GEOMETRY ASSESSMENT AND STRENGTH/STIFFNESS MONITORING OF LIME AND CEMENT MODIFIED SOILS VIA CHARACTERIZATION OF CURING-INDUCED PROPERTY CHANGES ESTIMATED FROM SEISMIC WAVE PROPAGATION TECHNIQUES AND ELECTRICAL RESISTIVITY

by

Richard G. Bearce
A thesis submitted to the Faculty and Board of Trustees of the Colorado School of Mines in partial fulfillment of the requirements for the degree of Doctor of Philosophy (Civil and Environmental Engineering)

Golden, CO

Date ________________

Signed ________________________
Richard G. Bearce

Signed ________________________
Dr. Michael Mooney
Thesis Advisor

Golden, CO

Date ________________

Signed ________________________
Dr. John McCray
Professor and Head
Department of Civil and Environmental Engineering
ABSTRACT

Soil modification via binding additives such as lime and cement grout is a commonly used practice for improving the engineering properties of soil in civil and underground construction applications. Such additives change the properties of the soil by forming cementing agents that bond soil grains together. This cementing process increases the strength/stiffness and reduces the hydraulic conductivity of the modified soil. To ensure adequate performance of chemically modified soil, assessment of both engineering properties and the spatial extent of modification is vitally important. A variety of performance verification techniques are used to assess engineering properties and geometry by industry, but these techniques have inherent limitations.

For roadway subgrade soil stabilized with lime and/or cement, an improved performance verification approach would utilize non-destructive monitoring of strength/stiffness growth of subgrade soils cured under field temperature conditions. This research employs wave propagation techniques to monitor the strength/stiffness growth of lime/cement stabilized subgrade and formulates a maturity function to predict modulus growth as a simultaneous function of both time and temperature. The maturity function is able to capture experimentally observed modulus growth from specimens cured at both constant (laboratory) and variable (field) temperature environments. A time/temperature dependent maturity function for lime/cement stabilized subgrade soils advances the current state of understanding and practice.

For soils modified with cement grout via in-situ applications such as jet grouting, an improved performance verification test would allow for immediate non-destructive assessment of production column geometry. DC resistivity testing is used to estimate soilcrete column diameter in a laboratory-scale study and validated via computational modeling. The computational modeling approach is extended to field geometries and a DC electrical resistivity push probe is developed. The push probe is tested on several deep soil mixed and jet grout columns on active construction sites. Computational modeling is used to interpret the experimental results and develop a procedure for estimating column geometry. The probe is able to estimate column geometry with an accuracy of ±5% of the as-constructed column diameter. Furthermore, the probe is a reusable/recoverable device that non-destructively evaluates fresh soilcrete column diameter within 1 hour of construction. This diameter verification approach is an improvement over any technique currently available.
# TABLE OF CONTENTS

ABSTRACT .............................................................................................................................. iii  

LIST OF FIGURES .................................................................................................................. vii 

LIST OF TABLES ...................................................................................................................... xiii 

ACKNOWLEDGEMENTS .......................................................................................................... xiv 

CHAPTER 1:  GENERAL INTRODUCTION .............................................................................. 1  
1.1 Introduction ...................................................................................................................... 1  
1.2 Thesis Organization ......................................................................................................... 4  
1.3 Literature Review ........................................................................................................... 7  
1.4 References Cited ............................................................................................................ 24 

CHAPTER 2:  A SEISMIC MODULUS MATURITY FUNCTION FOR LIME AND LIME-CEMENT STABILIZED CLAY ........................................................................... 31  
2.1 Abstract .......................................................................................................................... 31  
2.2 Introduction ..................................................................................................................... 31  
2.3 Experimental Procedure ............................................................................................... 33  
2.4 Results ............................................................................................................................. 35  
2.5 Development of an LSS and L-CSS Maturity Index ...................................................... 40  
2.6 Conclusions ..................................................................................................................... 49  
2.7 Acknowledgements ........................................................................................................ 50  
2.8 References Cited ............................................................................................................ 50 

CHAPTER 3:  CHARACTERIZATION OF SIMULATED SOILCRETE COLUMN CURING USING ACOUSTIC TOMOGRAPHY ................................................................. 54  
3.1 Abstract .......................................................................................................................... 54  
3.2 Introduction ..................................................................................................................... 54  
3.3 Experimental Setup ....................................................................................................... 56  
3.4 Results ............................................................................................................................. 57  
3.5 Conclusions ..................................................................................................................... 62  
3.6 References Cited ............................................................................................................ 65
CHAPTER 4: ELECTRICAL RESISTIVITY IMAGING OF LABORATORY SOILCREEPE COLUMN GEOMETRY .............................................................67
4.1 Abstract .................................................................................................67
4.2 Introduction ...............................................................................................67
4.3 Background ..............................................................................................70
4.4 Experimental Setup and Testing Protocol .................................................72
4.5 Finite Element Modeling ...........................................................................75
4.6 Results ......................................................................................................77
4.7 Conclusions ..............................................................................................85
4.8 Acknowledgements ...................................................................................86
4.9 References Cited .....................................................................................87

CHAPTER 5: DEVELOPMENT OF AN ELECTRICAL RESISTIVITY PUSH PROBE TO ESTIMATE THE DIAMETER OF JET GROUTED COLUMNS .........91
5.1 Abstract ...................................................................................................91
5.2 Introduction ...............................................................................................91
5.3 Background .............................................................................................94
5.4 Development of a Field-Scale Electrical Push Probe .........................96
5.5 Site Conditions and Testing Protocol .....................................................102
5.6 Results ....................................................................................................103
5.7 Conclusions ...........................................................................................111
5.8 Acknowledgements ...............................................................................114
5.9 References Cited ....................................................................................114

CHAPTER 6: DEVELOPMENT OF AN ELECTRICAL RESISTIVITY PUSH PROBE TO ESTIMATE THE DIAMETER OF JET GROUTED COLUMNS ..........118
6.1 Abstract ..................................................................................................118
6.2 Introduction .............................................................................................118
6.3 Background ............................................................................................120
6.4 Experimental Procedure .........................................................................123
6.5 Results and Discussion .........................................................................125
6.6 Conclusions ............................................................................................131
6.7 Acknowledgements ........................................................................................................ 132
6.8 References Cited ............................................................................................................. 132

CHAPTER 7: GENERAL CONCLUSIONS .................................................................................... 135
7.1 Specific Conclusions from Each Paper ............................................................................ 135
7.2 General Conclusions ........................................................................................................ 141
7.3 Recommendations for Future Work ................................................................................ 142

APPENDIX A: ADDITIONAL FINITE ELEMENT MODELING CONSIDERATIONS ........................................ 144

APPENDIX B: CO-AUTHOR AND PUBLISHER PERMISSIONS ............................................. 147
LIST OF FIGURES

Figure 1.1: a) Additional of quicklime slurry to field soil via tanker truck, and b) mechanical mixing of quicklime slurry via industrial mixing machinery ............2

Figure 1.2: Illustration of the cement grout soil improvement techniques a) deep soil mixing, and b) jet grouting (from Hayward-Baker 2015) ........................................3

Figure 1.3: a) Variable geometry in excavated jet grout columns (from Omran Ista 2015) and b) voids and soil inclusions in poorly mixed soilcrete (Stark et al. 2009) ...........4

Figure 1.4: Illustration of the bonds formed between sand grains and cementing compounds resulting from cement hydration (from Montgomery 1998) ..................9

Figure 1.5: Free-free resonance test exciting the longitudinal resonant frequency of a cylinder composed of lime stabilized clay (from Toohey 2009) ......................18

Figure 1.6: a) Tukey windowed time domain response from a free-free resonance test, and b) selection of the first resonant peak after performing a discrete Fourier transform on the response from a) (Toohey 2009) ........................................19

Figure 1.7: a) Illustration of CSL on a drilled pile (from Olson 2015) and b) selection of the first arrival time from a received p-wave (from Olson 2015) ..................20

Figure 1.8: Illustration of the electric field created by a traditional surface-based DC resistivity measurement (from Revil et al. 2012) ........................................24

Figure 2.1: (a) Five stacked time histories from FFR testing and (b) discrete Fourier transform of the time histories in (a) used to identify the resonant frequency ......33

Figure 2.2: (a) $E_0$ vs. $t$ and (b) field and lab temperatures vs. $t$ for site 1 L-CSS cylinders ....36

Figure 2.3: (a) $E_0$ vs. $t$ and (b) field and lab temperatures vs. $t$ for site 2 (zone 1) L-CSS cylinders ........................................................................................................36

Figure 2.4: (a) $E_0$ vs. $t$ and (b) field and lab temperatures vs. $t$ for site 2 (zone 2) L-CSS cylinders. Zone 2 soil is identical to zone 1 soil but additional temperature curing regimes were evaluated .........................................................37

Figure 2.5: (a) $E_0$ vs. $t$, and (b) lab and field temperatures vs. $t$ for site 3 LSS cylinders ........38

Figure 2.6: Correlation between $E_0$ and curing temperature for (a) day 3, (b) day 7, (c) day 14, and (d) day 28 .............................................................44
Figure 2.7: (a) Power model function from regression analysis for empirical parameter $\eta_t$ with individual data points from regression analysis ($\eta_t$) used to obtain this power model fit, and (b) $\beta$ values at each curing day obtained from regression analysis.

Figure 2.8: (a) A family of curves describing the progression of $E_0$ growth as a function of time and temperature (per Equation 12), and (b) average temperature inputs used to generate the growth curves in (a).

Figure 2.9: Comparison of constant curing temperature FFR $E_0$ data to $\tilde{E}_0$ predicted by the maturity function. Dashed line envelopes display $\tilde{E}_0$ for $T_\bar{t} \pm 1$ °C to illustrate the $\tilde{E}_0$ growth variation resulting from minimal temperature changes. Each $\tilde{E}_0$ curve is plotted with the parameter values obtained from regression analysis but at different constant temperature regimes.

Figure 2.10: (a) Variable (field) curing temperature $E_0$ compared to $\tilde{E}_0$ predicted by the maturity index for field-cured cylinders from sites 1 and 3 and (b) average temperature field temperature. Note that site 1 data corresponds to Figure 2.8a and b (path b, variable temperature).

Figure 3.1: (a) Photograph and (b) illustration of the experimental setup.

Figure 3.2: Example of raw data time history from CSL for (a) strong soilcrete, and (b) weak soilcrete.

Figure 3.3: Jet grout velocity vs. depth at increasing curing times using straight ray trace first arrival time approach.

Figure 3.4: Example ray trace pattern used for acoustic tomography (not taking refraction into account) and excavated specimen (to scale) to illustrate ray path coverage.

Figure 3.5: Verification of jet grout size/geometry prediction (Specimen 1) with excavated specimen, and tomogram from data acquired just before excavation.

Figure 3.6: Verification of jet grout size/geometry prediction (Specimen 2) with excavated specimen, and tomogram from data acquired just before excavation.

Figure 3.7: Verification of jet grout size/geometry prediction (Specimen 3) with excavated specimen and tomogram from data acquired just before excavation.

Figure 3.8: $V_p$ tomograms for Specimen 1 at increasing curing times.

Figure 3.9: $V_p$ tomograms for Specimen 2 at curing times 20 to 28 hours.

Figure 3.10: $V_p$ tomograms for Specimen 2 at curing times 48 to 120 hours.
Figure 3.11: $V_p$ tomograms for Specimen 3 at increasing curing times ....................... 65

Figure 4.1: a) Illustration of the jet grouting method, and b) the jet grouting method applied to foundation underpinning................................................................. 68

Figure 4.2: 2D axisymmetric cross section of current flow lines for the direct coupled ring electrode array at increasing electrode spacing. Estimation of $\rho_a$ using Equation 3 is shown for each $a$ based on the potential across electrodes M and N labeled in each plot................................................................. 72

Figure 4.3: a) Laboratory soil tank with 2 stages, b) staged construction of 30cm diameter soilcrete cylinder prior to addition of sand and soilcrete and c) soilcrete cylinder after extraction of form tube. A cross-sectional illustration of the sequential soilcrete/sand placement process is shown in d)-f) ......................... 73

Figure 4.4: Illustration of full length electrical array with expanded cross sectional diagram and photographs for a) direct coupled ring electrodes and b) ring electrodes in a water-filled slotted casing. Note that the photographs and FE renderings in b) do not show the geotextile to better appreciate the geometry of the slotted casing. The geotextile location is shown in the expanded illustration of and also visible in Figure 4.3b/c and Figure 4.5b ......................... 75

Figure 4.5: Diagram of soilcrete specimen/array/soil geometry with exhumed specimen for comparison for a) specimen 1 and b) specimen 2. Note that the height of the soil around the column corresponds to the height of the soil in the tank during the test and not the full height of the tank .............................................. 76

Figure 4.6: Comparison of experimental and FE apparent resistivity responses for specimen 1 after 1.5 hours of curing. Plots and column images are scaled such that horizontal dashed lines can be used to relate geometry changes in the column to changes in the data response. $z$ position of the data points represents the center of each four electrode array configuration....................... 78

Figure 4.7: Comparison of experimental and FE resistivity responses for specimen 2 after 1.5 hours of curing. Plots and column images are scaled such that horizontal dashed lines can be used to relate geometry changes in the column to changes in the data response................................................................. 79

Figure 4.8: 2D axisymmetric cross section of current flow lines of electrical array and homogeneous soil tank configuration for a) a direct coupled ring electrode array and b) a ring electrode array within a water-filled slotted casing at $a = 3$cm........................................................................................................ 81
Figure 4.9: Comparison of FE resistivity responses for columns with diameter profiles of $1.1D, D,$ and $0.9D$ to experimental resistivity response for a) $a = 3cm$, b) $a = 6cm$, c) $a = 9cm$, d) $a = 12cm$. e) Experimentally predicted column diameter $\bar{D}$ from each value of $a$........................................................................................................82

Figure 4.10: a) FE diameter study compared to experimental data for $a = 120cm$, and linear correlation between $D$ and $\rho_a$ for b) $z = 51cm$, and c) $z = 72cm$.............83

Figure 4.11: Experimental $\rho_{sc}$ with 1σ error bars from curing times of 1.5 hours to 240 hours. Specific curing times are highlighted and a best fit regression analysis function is shown.................................................85

Figure 5.1: Illustration of the electrical push probe geometry components. In a), the outer layer of probe is shown with inner/outer PVC pieces and ring electrodes. Figure b) shows a cross section of Figure a) to illustrate the probe’s internal supporting system via AWJ drill rod. In c), a close up of the connection is illustrated for the bottom nose cone section of the probe. This is the same connection mechanism used to attach each 1.5m section.............................................97

Figure 5.2: a) Illustration of the electrical push probe and measurement points obtained from the Wenner-α protocol at various electrode spacings, b) the electrical push probe attached to a placement rig, c) the push probe being submerged in a fresh DSM column, and d) the push probe after full placement..................98

Figure 5.3: Axisymmetric cross-section of the current flow and equipotential lines created by the electrical push probe in a 2m diameter soilcrete column for a) $a = 0.3m$, b) $a = 0.6m$, c) $a = 0.9m$, d) $a = 1.2m$ .................................................................99

Figure 5.4: Parametric study illustrating the effects of $D/a$ on measured $\rho_a$ for soil:soilcrete resistivity contrasts of a) 4:1, b) 10:1, c) 20:1, d) 40:1, e) 60:1, and f) 80:1 .................................................................101

Figure 5.5: Schematic of soil conditions, column geometry/resistivity, probe position, and axisymmetric measured region for a) site 1 column 1, b) site 2 column 2, and c) site 3 column 1. Soil conditions are assumed the same for each site across columns (Table 1), but column diameters and resistivities vary (Table 5.1).......104

Figure 5.6: $a = 0.3m \rho_a$ for a) Site 2 Column 1 with b) corresponding FE model. c) Site 1 Column 1 with d) corresponding FE model. The experimental data is averaged over particular regions to illustrate the effects of $\rho_s/\rho_{sc}$ and $D/a$ on the probe’s ability to capture $\rho_{sc}$ .................................................................106

Figure 5.7: a) Experimental resistance data from Site 2 Column 1 for $a = 0.6, 0.9,$ and 1.2m, b) FE-estimated geometric correction factor $k$ for each value of $a$ in a), and c) apparent resistivity response obtained by multiplying a) and b) for any given depth and value of $a$ (i.e., Equation 3.5) .................................................................107
Figure 5.8: Comparison of experimental vs. FE $\rho_a$ responses for Site 1 Column 1 ..........109

Figure 5.9: Comparison of experimental vs. FE $\rho_a$ responses for Site 2 Column 1 ..........110

Figure 5.10: Comparison of experimental vs. FE $\rho_a$ responses for Site 3 Column 1 ..........111

Figure 5.11: a) Experimental $\rho_a$ compared to FE $\rho_a$ for columns with diameters of $0.9D$, $D$, and $1.1D$ and $a = 0.9m$. Diameter is estimated via linear correlation for b) $z = 1.8m$ and c) $z = 4.3m$. Plot d) shows the predicted diameter $\bar{D}$ using each appropriate value of $a$. Plot e) shows the average $\bar{D}$ estimated by averaging the $\bar{D}$ estimates in d) using all values of $a$ ...............................................................112

Figure 5.12: Estimated soilcrete column diameter $\bar{D}$ with ±5%D bounding lines for a) site 1 column 1, b) site 1 column 2, c) site 2 column 1, d) site 2 column 2, e) site 2 column 3, and f) site 3 column 1. Reported depth intervals reflect the probe position and corresponding measurement profile (e.g., Figure 5.5) ..........................112

Figure 6.1: a) Illustration of the 20 electrode push probe with corresponding data points for a full protocol. An example array length (3a) is illustrated for each value of $a$ using the top electrode as injection electrode A. b) An illustration of column 1 and the current/equipotential lines resulting from an $a = 0.9m$ measurement ..................................................................................123

Figure 6.2: a) Jet grouting at the Horstwalde field site, and b) implementation of the electrical push probe immediately after grouting .................................................................125

Figure 6.3: Soil profile, grouting parameters, probe positions, and measured region for a) column 1, and b) column 2 ..........................................................127

Figure 6.4: Experimental $\rho_a$ profiles from the $a = 0.3m$ electrode spacing for a) column 1 and b) column 2 .................................................................128

Figure 6.5: Experimental $\rho_a$ responses with FE-predicted $\rho_a$ responses for column 1 with $a$ values of a) 0.6m, b) 0.9m, and c) 1.2m .........................................................129

Figure 6.6: Experimental $\rho_a$ responses with FE-predicted $\rho_a$ responses (from both constant and variable diameter columns) for column 2 with $a$ values of a) 0.6m, b) 0.9m, and c) 1.2m .........................................................130

Figure 6.7: $\bar{D}$ with depth for a) column 1 and b) column 2 .........................................................131

Figure A-1: Illustration of the volumetric current injection applied to push probe ring electrodes ..................................................................................144

Figure A-2: Geometric correction factors used for the push probe .................................................145
Figure A-3: Illustration of probe offset effects.
LIST OF TABLES

Table 2.1: Soil properties/classifications and lime/cement mix designs for all soils evaluated in this research. FFR testing regime with number of cylinders per temperature regime and average moisture content of field-mixed soil from each site..........................................................34

Table 2.2: Parameter values from least squares regression power model fits for curves in Figures 2.2-2.5 ........................................................................................................39

Table 4.1: Experimentally measured resistivity values of the sand ($\rho_S$), soilcrete ($\rho_{SC}$), and water ($\rho_W$) used in the laboratory experiments. FE resistivity inputs for each model are also reported.................................................................80

Table 5.1: Summary of test sites and columns tested with $\rho_{SC}$ values and tested probe interval ........................................................................................................................................103
ACKNOWLEDGEMENTS

I would like to express my appreciation to my advisor, Dr. Mike Mooney, for his support, guidance, and invaluable contributions to this research. I would also like to thank Dr. Pauline Kessouri, who assisted in the acquisition and interpretation of electrical data throughout this project. Furthermore, I would like to express my gratitude to my Ph.D. committee for the technical and editorial feedback they have provided throughout this research.

Special thanks to Dr. Ernst Niederleithinger for organizing my guest researcher stay at the BAM Federal Institute for Materials Testing and Research in Berlin, Germany during the summer of 2014. This stay resulted in an abundance of research collaboration on field testing of jet grout columns which is still actively underway. I would also like to thank BAM scientist Julio Galindo-Guerreros, whose assistance in field testing at the BAM site was invaluable.

ARS Inc. provided access to several lime/cement treated subgrade sites throughout the course of this research. I would like to express my gratitude to project manager Derek Garbin and the on-site personal for allowing me to test on their active construction sites. I would like to thank Olson Engineering for the use of their crosshole sonic logging system during this research. Industry collaborator Hayward-Baker provided multiple field testing sites with on-site equipment and personnel for soilcrete column evaluation. I would like to express my appreciation for their willingness to let us evaluate new technology on their construction sites.

Finally, I would like to thank my wife, parents, and friends for their ongoing encouragement and support.
CHAPTER 1: GENERAL INTRODUCTION

1.1 Introduction

Modification of soil with chemical additives such as lime and cement is frequently used in civil and underground construction to improve the soil’s engineering properties. Application of lime and cement can occur in many forms from lime/cement stabilized subgrade to in-situ cement grout applications such as jet grouting. Both lime and cement react with water (and sometimes soil minerals) to create cementing compounds that bond soil grains together. This process increases soil strength, reduces the potential for shrink/swell, and hydraulic conductivity. While soil modification techniques have been used in industry for decades, techniques used to verify the engineering behavior and geometry of the stabilized soil are often lacking. For stabilized subgrade soil layers, verification of strength/stiffness and layer thickness is desired. For jet grout columns, an assessment of soilcrete continuity and column diameter is desired. As sensing technology and computing capability advance, non-destructive wave propagation and electrical techniques provide attractive solutions for improved QA/QC of chemically modified soil.

Lime, sometimes in combination with cement, is typically used to stabilize near-surface subgrade soils (that lie directly beneath the placed aggregate base and surface layer) with high clay contents that are especially susceptible to shrink/swell. Generally, quicklime slurry is added to the soil via tanker truck (Figure 1.1a) and mechanically mixed into the in-situ soil with industrial mixing machinery (Figure 1.1b). Industry has adopted a wide array of QA/QC methods to evaluate stabilized soil depending on soil type and application; however, these techniques have inherent limitations (discussed in detail in section 1.3.2). Subgrade soils stabilized with lime/cement undergo a time and temperature dependent curing process that results in increased strength/stiffness and reduced plasticity. The input design parameter for stabilized subgrade is the 28-day resilient modulus ($M_r$). Because construction schedules cannot wait 28 days to verify performance, 28 day $M_r$ is estimated from specimens cured at accelerated temperatures (e.g., 7 days at 41°C) with the assumption that this accelerated curing will have the same results as 28 day 23°C curing. Given the range of soil types and lime/cement admixtures used to stabilize subgrade, a single accelerated curing regime is not logical and likely inaccurate. This QA/QC process for stabilized subgrade can be improved via laboratory and field assessment of stabilized soils with wave propagation techniques, which can non-destructively monitor curing-induced
strength/stiffness growth for a given mix design. Estimating modulus growth on the field-mixed/field-constructed subgrade that experiences the on-site temperature is an improvement over existing QA/QC processes.

Figure 1.1: a) Additional of quicklime slurry to field soil via tanker truck, and b) mechanical mixing of quicklime slurry via industrial mixing machinery.

While lime is typically used for roadway subgrade stabilization, cement has various applications within the civil and underground construction industries. Many applications are in-situ techniques that mix cement with soil such as jet grouting, injection grouting, compaction grouting, deep soil mixing, slurry/diaphragm walls, etc. Jet grouting is an in-situ soil improvement technique that sprays high pressure cement grout from a rotating drill string to erode in-situ soil (Figure 1.2b). The mixture of eroded soil and cement grout forms soilcrete columns in the subsurface. Soilcrete is a type of concrete composed of cement grout (cement-water mixtures) and in-situ soil. The turbulent mixing process used in jet grouting leads to uncertainty in the geometry of the resulting column (Figure 1.3a). Variation in machine parameters such as grout pressure, drill rotation speed, and cement content in the grout can lead to variable soilcrete column geometry and integrity. Mixing these grouts with various types of soil with varying in-situ stress states and groundwater conditions can further affect the ability of the jet grouting process to produce columns of precise geometry with well mixed (high integrity) soilcrete. Because columns are designed to perform with a set diameter, geometry/integrity verification is crucial. For example, jet grout columns can be overlapped to create hydraulic barriers. If the column diameter is too small or soil inclusions (Figure 1.3b) exist because of
inadequate mixing, leaks can occur in the hydraulic barrier. These types of performance issues are costly as they delay construction schedule and sometimes require the ground improvement technique be repeated to achieve desired results.

Figure 1.2: Illustration of the cement grout soil improvement techniques a) deep soil mixing, and b) jet grouting (from Hayward-Baker 2015).

Verification techniques for jet grout column geometry exists in many forms (discussed in detail in Section 1.3.3), but most techniques used by industry are destructive (e.g., coring, excavation). While some non-destructive geophysical techniques are used, these techniques often require permanent casings in or near the column. Because of the destructive nature of tests or the requirement of a permanent casing, many of these techniques are only used to evaluate test columns (i.e., a column produced on the same site with the same grouting conditions which is assumed to be the same geometry/integrity as production columns to be constructed at a later time). Given the turbulent nature of jet grouting and variation in field soils, this assumption is not always true. An improved verification test would evaluate soilcrete column geometry and integrity using a truly non-destructive approach (i.e., no damage to column or permanent casings in or near the column).
This research applies seismic/acoustic wave propagation techniques and direct current (DC) electrical resistivity to non-destructively assess both the engineering properties and geometry of soils improved with lime/cement and cement grout. Seismic/acoustic wave propagation techniques (free-free resonant column testing, surface wave testing, and crosshole ultrasonic logging) are used to assess subgrade stiffness and/or soilcrete integrity via the increased wave speed that results from curing. DC resistivity is used to assess the geometry of soilcrete columns in the laboratory and field by identifying the substantial resistivity contrast between the fresh soilcrete and in-situ soil.

1.2 Thesis Organization

This thesis is divided into seven chapters including an introduction, five papers that make up the body chapters, and a general conclusions chapter. A brief summary of each paper is described below.

Paper I: A Seismic Modulus Maturity Function for Lime and Lime-Cement Stabilized Clay

This paper develops a time and temperature dependent maturity function for lime and lime/cement stabilized clay based on observed effects on the curing behavior of clay subgrade soil. Free-free resonance testing is used to characterize the compressional wave velocity of specimens prepared from field-mixed stabilized subgrade obtained from several construction
sites near Denver, CO. This wave velocity assessment is used to determine the seismic (low strain) elastic modulus. The elastic modulus growth of approximately 60 specimens is monitored over a curing period of 28-60 days. Several maturity approaches from the cement/concrete literature are studied and were determined inadequate for capturing the simultaneous time/temperature dependent modulus growth that results from lime/cement stabilization. Elastic modulus growth data is analyzed using least squares regression analysis and a time and temperature dependent maturity function is developed to predict elastic modulus growth in lime and lime/cement stabilized clay subgrade. Co-author Dr. Michael Mooney served as the faculty advisor for this project and assisted with technical and editorial feedback to improve the quality of the manuscript.

**Paper II: Characterization of Simulated Soilcrete Column Curing using Acoustic Tomography**

This paper validates the use of crosshole sonic logging to monitor curing-induced compressional wave velocity growth of laboratory-constructed soilcrete to develop a soilcrete integrity testing program for field soilcrete columns produced via jet grouting. The soilcrete was constructed from masonry sand and cement grout with grout properties similar to those used in field jet grouting applications. Compressional wave velocity growth is monitored using traditional crosshole sonic logging (CSL) on two columns over the course of seven days and data inversion is performed to produce tomograms of column integrity. The laboratory tests suggest that CSL with tomography provides a high resolution 2D image of the soilcrete velocity profile. The profiles indicate that compressional wave velocity grows for 7+ days. In addition, the CSL captures the decreased energy attenuation that results from curing. Co-author Dr. Michael Mooney served as the faculty advisor for this project and assisted with technical and editorial feedback to improve the quality of the manuscript. Co-author Dr. Ernst Niederleithinger assisted with laboratory data acquisition and provided technical feedback for manuscript preparation. Co-author Dr. Andre Revil provided editorial feedback to improve the quality of the manuscript.

**Paper III: Electrical Resistivity Imaging of Laboratory Soilcrete Column Geometry**

This paper presents the results of a study to better understand the potential of DC resistivity for imaging soilcrete columns, including direct couple electrodes vs. electrodes in a slotted casing with indirect coupling. The laboratory-scale soilcrete experiments are similar to
those conducted in Paper II, but instead of wave propagation techniques to monitor column integrity, DC resistivity is used to estimate column geometry. Two different array/casing combinations are evaluated to assess the sensitivity of electrode coupling and electrical protocol type on the measured results. Finite element modeling is used to validate the laboratory data and study the effects of column diameter variation. The model is then used to devise a technique to estimate column diameter. Time lapse monitoring of curing behavior is monitored to determine the most appropriate time window for DC resistivity measurement. Co-author Dr. Michael Mooney served as the faculty advisor for this research project and provided technical and editorial feedback to improve the quality of the manuscript. Co-author Dr. Pauline Kessouri assisted with laboratory data acquisition and provided technical feedback on data interpretation during manuscript preparation.

**Paper IV: Direct Couple Electrical Resistivity Imaging of Freshly Constructed Deep Soil Mix Columns to Estimate Diameter**

The results of the laboratory experiments and computational modeling discussed in Paper III are used to design a field-scale electrical resistivity push probe for diameter assessment on field-constructed soilcrete columns. The modeling approaches used in Paper III are up-scaled to field geometries and several configurations are evaluated to determine the optimum size and electrode configuration for a field probe. The modeling also helps to understand scaling, probe-grout interaction, and end effects at the top/bottom of columns. A prototype probe is constructed and tested on several deep soil mixed (DSM) columns. The advantage of DSM columns is they are mechanically mixed by a mixing blade of precisely known size, thus resulting in a column with precisely known diameter. This approach is advantageous because ground truthing of soilcrete columns is inherently difficult (e.g., excavation is costly and inefficient). This paper validates the experimental results obtained from field probe testing using the finite element model. A diameter estimation technique similar to that of Paper III is used to estimate the accuracy with which the probe can estimate diameter (i.e., the columns are of precise diameter, so deviation from expected diameter is related to measurement inaccuracy and not actual diameter changes). Co-author Dr. Michael Mooney served as the faculty advisor for this research project, assisted with field data acquisition, and provided technical and editorial feedback to improve the quality of the manuscript. Co-author Dr. Pauline Kessouri assisted with field data.
acquisition and provided technical feedback on data interpretation during manuscript preparation.

**Paper V: Estimation of Jet Grout Column Geometry with a DC Electrical Resistivity Push Probe**

This paper presents the results of field tests on jet grouted columns using the electrical push probe developed in Paper IV. Because the jet grouting process can result in variable diameter columns, this research extends the testing of Paper IV by introducing the possibly of column diameter variation. The electrical push probe developed in Paper IV is used on two jet grout columns constructed at a test site near Berlin, Germany. The results are analyzed and diameters are predicted using the same technique as Paper IV. In addition to push probe diameter assessment, the column diameter is assessed via machine parameter monitoring from on-site contractors and crosshole seismic testing conducted by personnel from the BAM Federal Institute for Materials Research. The results of all employed diameter estimation techniques are compared to assess accuracy. Co-author Dr. Michael Mooney served as the faculty advisor for this research project and provided technical and editorial feedback to improve the quality of the manuscript. Co-author Dr. Pauline Kessouri provided technical feedback on data interpretation during manuscript preparation.

1.3 Literature Review

The addition of lime and/or cement to soil causes several chemical reactions that change the physiochemical and engineering properties of the soil. Lime stabilized subgrade is often treated with quicklime (CaO), a mixture of lime and water that produces hydrated lime. Subgrade soil improvement occurs via soil modification and soil stabilization when hydrated lime is mixed with fine-grained soils. The soil modification process is governed by cation exchange and flocculation-agglomeration which occur rapidly and result in unconfined compressive strength ($q_u$) increase and soil plasticity reduction (Little 1987, Mallela *et al.* 2004). Excess Ca$^{++}$ cations from the quicklime replace weaker metallic cations in the soil causing a size reduction in the diffuse water layer around the clay minerals. This reduction in diffuse layer size causes the clay particles to flocculate (Little 1987, Mallela *et al.* 2004). The second mechanism for soil improvement with lime is soil stabilization, which occurs via time and temperature dependent pozzolanic reactions. Depending on reactant supply, pozzolanic reactions can occur over a time
span of days to years. In addition to reduced diffuse water layer size, the addition of hydrated lime to fine-grained soil also causes a significant increase in soil pH. The higher pH increases the solubility of silica/alumina compounds present in the clay minerals and results in the formation of cementing agents (Little, 1987).

The chemical reactions governing the formation of cementing agents are outlined in the equations 1.1-1.4. Equation 1.1 shows the chemical reaction that produces calcium hydroxide (Ca(OH)$_2$) when quicklime (CaO) and water (H$_2$O) are mixed (Little et al. 1995).

\[ CaO + H_2O \rightarrow Ca(OH)_2 \]  \hspace{1cm} (1.1)

At sufficiently high pH, calcium hydroxide will react with pozzolans in the soil to form cementing agents. Pozzolans are compounds usually consisting of silicates, alumina, and/or alumino-silicates. The reactions between calcium hydroxide and various pozzolans will produce cementing agents calcium-silicate-hydrate (CSH) and calcium-aluminate-hydrate (CAH). An example reaction between calcium hydroxide and an alumino-silicate is shown in Eq. 1.2 (West and Carder 1997).

\[ 2(Al_2O_3 \cdot 2SiO_2) + 7Ca(OH)_2 \rightarrow 3CaO \cdot 2SiO_2(aq) + 2(2CaO \cdot Al_2O_3 \cdot SiO_2(aq)) \]  \hspace{1cm} (1.2)

The compound 3CaO-2SiO$_2$(aq) is a CSH. Similar reactions between calcium hydroxide and other pozzolans yield various CSH and CAH compounds. CSH and CAH are some of the same types of cementitious hydrates formed during the hydration of Portland cement (Terrel et al. 1979). The majority of Portland cement hydration occurs via the combination of water and either tricalcium silicate (Equation 1.3) or dicalcium silicate (Equation 1.4) to form CSHs (Taylor 1997).

\[ 2(CaO)_3(SiO_2) + 7H_2O \rightarrow (CaO)_3 \cdot (SiO_2)_2 \cdot 4H_2O + 3Ca(OH)_2 \]  \hspace{1cm} (1.3)

\[ 2(CaO)_2(SiO_2) + 5H_2O \rightarrow (CaO)_3 \cdot (SiO_2)_2 \cdot 4H_2O + Ca(OH)_2 \]  \hspace{1cm} (1.4)
Similar reactions occur between alumina compounds (e.g., \((\text{CaO})_3 \cdot \text{Al}_2\text{O}_3\)) and water to form CAHs. These reactions continue until reactant supply is consumed.

In lime stabilization, pozzolans are supplied by the soil itself (while calcium hydroxide is the primary chemical supplied by lime application). Portland cement is produced with a variety of compounds that, when hydrated, form cementing agents regardless of the type of soil being stabilized (i.e., Equations 1.3 and 1.4). Portland cement is typically composed of tricalcium silicate, dicalcium silicate, tricalcium aluminate \((\text{Ca}_3\text{Al}_2\text{O}_6)\), and tetracalcium aluminoferrite \((\text{Ca}_4\text{Al}_2\text{Fe}_2\text{O}_{10})\) compounds, depending on the source of the cement constituents. Hydration processes for tricalcium aluminate and tetracalcium aluminoferrite are similar to those of tricalcium and dicalcium silicate shown in Equations 1.3 and 1.4. The binding effects of hydrated cement is illustrated in Figure 1.4, where cement particles surrounded by water are hydrated and form crystalline structures that produce strong cementing bonds between soil grains.

![Figure 1.4: Illustration of the bonds formed between sand grains and cementing compounds resulting from cement hydration (modified from Montgomery 1998).](image)

The chemical reactions that occur when soil grains are bonded via lime and/or cement also have an effect on the geophysical properties of the soils. As lime stabilized subgrade cures (i.e., pozzolanic reactions occur), soil grains are cemented together, increasing the strength/stiffness of the stabilized soil. This increase in strength/stiffness can be characterized using wave propagation techniques. As the stabilized soil cures, the wave velocities (e.g., compressional, shear) will increase. This increase can be monitored with wave propagation techniques such as free-free resonance (discussed in Section 1.3.4). Soils treated with cement grout undergo the same strength/stiffness growth, though the time frame over which this growth
occurs can vary depending on additive type and quantity. The electrical properties of cement-grouted soil are notably different from the unmodified soil, especially in the time frame immediately after cement grout addition. In the case of soilcrete produced with cement grout, the wet/fresh soilcrete is in slurry form with highly connected pore space and a highly conductive pore fluid caused by the free ions from the hydrated cement. This porous connectivity and highly ionic pore fluid make the fresh soilcrete much more conductive than the in-situ soil. The contrast between fresh soilcrete and in-situ soil is an excellent target for DC resistivity, which can be used to assess the geometry of a newly constructed soilcrete body.

The lime/cement mix designs for stabilized soil are based on laboratory tests to determine the optimum additive content. Soils are tested with the Eades and Grim pH test (ASTM D6276) to determine lime demand, i.e., the lime content required to satisfy immediate lime-soil reactions and still provide sufficient residual calcium to maintain high system pH for the long-term pozzolanic reactions. Soils are also evaluated with the shrink/swell test (ASTM D3877) to determine their expansive potential at different additive contents in the interest of reducing the shrink/swell potential of treated roadway subgrade. Field-constructed subgrade is treated with the appropriate amounts of lime/cement/water to mimic the laboratory design, but the inherent variability of field conditions and application technique (Figure 1.1) can lead to variability in the mixed proportions of the resulting subgrade. Performance verification of lime and lime-cement stabilized subgrade requires the estimation of both shear strength (e.g., unconfined compressive strength, \( q_u \)) and resilient elastic modulus (\( M_r \)). Construction operations seek to maintain efficiency, and thus rapid achievement of \( q_u \) is desired to verify that the subgrade is strong enough for the passage of construction traffic that will construct the overlying layers of the pavement system. Suitable strength for construction traffic is typically reached within one week. \( q_u \) is verified by grab sampling field-mixed subgrade prior to compaction and compacting it into standard Proctor molds. After a specified curing duration (discussed below), unconfined compressive strength tests are performed in the laboratory to verify that the subgrade will perform according to design. Field-mixed soil is gathered from a design-specified number of points per roadway length (nominally 1 per 50-100m of roadway). After this laboratory verification, construction crews are permitted to begin construction of overlying pavement layers.
Pavement design guides such as AASHTO’s Mechanistic Empirical Pavement Design Guide (MEPDG) require 28-day resilient modulus ($M_r$) as the input parameter (i.e., $M_r$ determined from specimens cured for 28 days at 23°C). Because $M_r$ testing is more costly and involved than $q_u$ testing, $M_r$ is often obtained from a correlation to $q_u$ (Thompson 1966, Little 1994, Toohey et al. 2013). The requirement of a 28 day parameter for subgrade/pavement system design presents a problem for most construction schedules, which would seek to begin construction of the overlying pavement layer within a week of subgrade stabilization. Literature has adopted the notion of an equivalent curing regime, where specimens cured for 7 days at 41°C (Little 1999, Mallela et al. 2004) are assumed equivalent to 28 days at 23°C. While this approach provides a more rapid verification of performance (7 days vs. 28 days), the assumption of an equivalent accelerated curing regime for all soils and lime/lime-cement mix designs is an oversimplification. There is a significant body of literature on accelerated curing of lime/cement stabilized soil (e.g., Biswas 1972, Drake and Haliburton 1972, Townsend and Donaghe 1976, Alexander and Doty 1978, Little 1987, Little et al. 1994, Little 1999, Little 2000, Yusuf et al. 2001, Mallela et al. 2004, Little et al. 2004, Toohey et al. 2013); however, these studies all support the conclusion that there is no equivalent accelerated curing regime for all soil types and lime/lime-cement mix designs. Furthermore, none of these studies have proposed a quantitative relationship between the measured parameter ($q_u / M_r$) and both time and temperature simultaneously, i.e., a maturity function for lime/cement stabilized soil.

Verification of performance for lime/cement stabilized subgrade is further complicated by field curing conditions, where the soil experiences the variable temperature curing regime of the field site, e.g., day to night temperature cycles, changes in daily temperature, etc. Non-destructive techniques to assess the modulus growth of both laboratory specimens and field-constructed subgrade have been evaluated in literature. Low strain seismic modulus testing via free-free resonance (FFR) is a well-accepted non-destructive approach (Nazarian 2002, Ryden 2004, Ahnberg and Holmen 2008, Toohey and Mooney 2012) that can monitor modulus growth of specimens throughout curing. The low strain or seismic modulus ($E_0$) is also well correlated to $q_u$ (Toohey and Mooney 2012) and $M_r$ (Williams and Nazarian 2007) making it an attractive method for the characterization of stabilized soil as it provides simultaneous assessment of both strength and stiffness. FFR testing is performed on compacted specimens in a laboratory environment, but the specimens can be composed of field-mixed soil and cured in the field to
experience the same temperature regime as the field-constructed soil. Because the FFR method is extensively used for the research herein, a full description of this technique is presented later in this literature review.

Non-destructive evaluation of field-constructed subgrade is often performed with surface wave testing, which can estimate $E_0$ and stabilized layer thickness. Since its development (Heisey 1982, Nazarian 1984), spectral analysis of surface waves (SASW) has been used to non-destructively estimate modulus in pavement and soil systems. Accelerometers or geophones are placed on the soil surface, an impulse load is applied at a designated surface source location, and the resulting surface waves are measured (Kim et al. 2001). Advances to the method have resulted in SASW becoming a common modern method for soil/pavement evaluation (Ryden et al. 2006). Multichannel Analysis of Surface Waves (MASW) is similar to SASW, but utilizes multiple simultaneous measurements (i.e., multiple accelerometers or geophones) to estimate soil properties (Park et al. 1997). MASW data also contain information about higher order wave propagation modes because of the method’s richer spatial data sampling (Ryden et al. 2004). To this end, the MASW method is often used to evaluate multi-layer soil/pavement systems (Ryden and Park 2006, Ryden et al. 2004, Ryden 2004).

Another primary focus of the research herein is to improve QA/QC for jet grouted columns. The QA/QC techniques for similar applications (e.g., injection/compaction grouting, cutoff or diaphragm walls) are also discussed as there are inherent similarities in the methods that are applicable to jet grouting as well. For jet grouted columns, the density and geometry of a column can be estimated from design parameters and the actual volume of grout injected for each borehole with some expectation of grout/soil ratio (Ho 2011). During injection, properties such as grout pressure and drill string rotation rate are controlled via the machine operator. This type of QA/QC is commonly used in practice, and is the current definition of “monitoring” (Larsson 2005). However, Stark et al. (2009) concludes that the grout:soil ratios for field mixed columns is often highly variable and not consistent with mix design parameters.

Other common approaches used for jet grout geometry assessment in industry include radial coring/probing or column excavation (e.g., Duzceer and Gokalp 2004, Olgun and Martin 2008, Rollins et al. 2010, Yoshida 2010, Burke 2012, Bruce 2012, Wang et al. 2012, etc.), but these approaches require 2+ days for adequate curing and are difficult/unfeasible to perform below the water table. Furthermore, these tests are destructive and can only be performed on test
columns (not production columns). Nikbakhtan and Osanloo (2009) performed laboratory tests (uniaxial compression, triaxial compressive strength, and direct shear) on continuously cored samples from excavated in-situ jet grout columns and found significant variability from the properties observed in the laboratory mix design tests (up to 50% difference). A number of nondestructive approaches have been proposed in the past to measure jet grout column geometry, including mechanical downhole devices (Passlick and Doerendahl 2006); however, these devices are cumbersome and potentially not recovered from the column. Temperature monitoring (Meinhard 2002, Mullins 2010, Sellountou and Rausche 2013) has also been proposed, but the inherent heterogeneity of field soils and soilcrete can cause considerable error in the ability of these techniques to estimate diameter. In a related field, thermal imaging has been successfully used to assess diaphragm walls and diaphragm wall joints (Doornenbal et al. 2011, Spruit et al. 2011).

Several wave propagation approaches have been applied to jet grout columns and similar cement grout applications. The earliest attempt to characterize grout injections in the field using seismic methods was conducted by E. L. Majer (1989). Several downhole seismic arrays were placed around the injection area, which was composed of fractured crystalline rock. The goal of this research was to measure acoustic emission events caused by the pressurized injection of grout to track the flow of the grout in the subsurface. All active sensors detected acoustic emissions during the grout injection, and incoming events had frequency content ranging from 2-10kHz. However, the largest acoustic emission events occurred after the grout injection pressure had been released. The author believes these significant acoustic emissions are the result of the fractured rock and grout mixture setting to a final configuration in the absence of grout pressure.

Madhyannapu et al. 2010 focuses specifically on improved QA/QC for soilcrete columns by comparing field-constructed DSM columns to laboratory results. Both laboratory-mixed and field-sampled grout/soil slurry were prepared into cylindrical specimens. After curing, bender element, unconfined compression strength, free swell, and linear shrinkage tests were performed on the specimens. The authors found that the estimated shear modulus of the field-mixed soil only reached 43-67% that of the laboratory mixed soil. In addition, $q_u$ values for field mixed specimens only achieved 67-83% of the strength estimated for the laboratory mixed samples. In the field, downhole P-wave tests and SASW tests were used to assess performance. The average P-wave velocities recorded from downhole tests in and around jet grout columns were 1.4 to 2.3
times larger than those recorded in untreated areas. SASW results estimated shear wave velocities in jet grouted areas to be between 1.3 and 1.5 times higher than the shear wave velocities of the untreated soil.

Sénéchal et al. (2010) used acoustic imaging to estimate the properties of fresh mortar injected in the laboratory. The goal of this study was to characterize the spatial geometry and properties at the soil/mortar interface in the interest of monitoring the developing acoustic properties with mortar curing time. Mortar properties were measured from 15 minutes to 8 hours after placement at varying time intervals. Results indicate that the p-wave velocity of the fresh mortar is very low (ranging from 86-287 m/s). Furthermore, the mortar causes significant attenuation, decreasing the source signal's frequency content from 20kHz to between 4 and 6 kHz. This attenuation is most significant at early curing times and decreases with time.

The most common electrical method used to evaluate chemically stabilized soil is DC electrical resistivity. This technique is based on Ohms' Law and Earth material resistance to current flow. There is limited literature applying DC resistivity to jet grout columns, so additional DC resistivity studies related to similar cement grout applications are also discussed. The electric cylinder method (ECM) is a commercially-available DC resistivity technique used to estimate the geometry of a jet grouted column (Frappin and Morey 2001, Frappin 2011). The ECM employs a central borehole with a slotted casing in the center of the column (either pushed into the fresh column or drilled in after 1-2 days of curing). After casing placement, a chain of electrodes is lowered into the water-filled casing to allow electrical coupling between the jet grout and the electrodes (i.e., the electrodes are coupled to the water, which is coupled to the jet grout through the slots in the casing). This approach uses a type of pole-pole electrode array configuration that requires reference electrodes on the ground surface. Frappin and Morey (2001) conclude that the ECM can estimate to within 10% of the column diameter. However, in regions where geometry changes are the result of changing soil conditions, there is an additional 0.5m error. This can result in considerable uncertainty.

Daily and Ramirez (2000) performed several electrical resistivity tomography (ERT) surveys to assess engineered hydraulic subsurface barriers. The study compares thin diaphragm walls (constructed via high pressure high mobility grout) and thick mortar walls (i.e., low mobility compaction grouting). To assess these in-situ barriers as aquitards, the central region (inside an enclosed "wall" of grout formations) was flooded and additional ERT surveys were
acquired. Based on the results of their data, the authors conclude that ERT can produce 2- or 3-D images of subsurface mortar/grout barriers, and that the location of the water table is not important (i.e., method works equally well above or below).

Abu-Zeid et al. (2006) performed ERT on a portion of a Venice canal before and after mortar injection treatment. The canal walls, being several centuries old, are subject to internal deterioration. Internal voids can be injected with mortar to re-solidify these walls. To characterize the improvement from mortar injection, the authors collected DC resistivity data using Wenner and dipole-dipole arrays before injection, and again 4 weeks after injection. The authors concluded that 3-D ERT is an effective tool for mapping injected mortar volume. However, this study only considered mortar that had cured for 4 weeks. Ideally, the mortar curing process could be monitored from the time of injection.

Keersmaekers et al. (2006) performed a study to assess the application ERT on grout/mortar injection on the failing masonry foundations of Our Lady's Basilica at Tongeren, Belgium. Field resistivity surveys were conducted using a 48 electrode array with 10cm spacing. Although limited data were presented, the authors conclude that ERT is a useful technique for characterizing masonry/foundations injected with grout.

Abu-Zeid et al. (2009) used ERT to estimate the mortar injection volume into the walls of the church of Montepetriolo, Perugia, Central Italy. Resistivity surveys were conducted immediately before the injection of mortar, and again 10 days after. The mortar used in this injection (once cured) is highly resistive. The authors note the ability of the ERT to locate the areas affected by mortar injection via a large increase in resistivity from the before image. However, the authors conclude that while this method is a useful qualitative tool for locating injected mortar, it still lacks the quantitative capability to accurately estimate the volume of injected mortar present in a given location. Furthermore, this study again compares 10-day cured mortar, and does not assess the mortar while curing.

Santorato et al. (2011) performed 3D-ERT on soil treated with resin injection. This is fundamentally similar to mortar injection, but uses high resistivity (i.e., 1000Ωm) expanding polyurethane resins instead of mortar/grout. The study was conducted around a building with settlement cracks where the native soil has an assumed resistivity of 10Ωm, making the contrast between native soil and injected resin a good resistivity target. As this study is considering the immediate effects of additional resin injections, it is the closest approach to resistivity
monitoring of grout/resin injection currently in the literature. It should be noted however, that this study does not perform true time lapse measurements. In estimating the resistivity profiles before and after injections, sequential resistivity "snapshots" are compared. Because background noise level is often inconsistent between acquisitions, subtracting snapshots in time may result in non-existent anomalies. In true time lapse inversion, time would be inverted as a variable to minimize noise level between acquisitions.

A number of wave propagation techniques are discussed in the literature review in section 1.3.3; however, not all of these techniques were employed by the research herein. Paper 1 relies heavily on seismic Young’s modulus estimated from compressional wave velocity ($V_p$) obtained from the free-free resonant column test. The free-free resonant column test or free-free resonance (FFR) method is rooted in the work of Richart et al. (1970) on wave propagation in an elastic, homogeneous, isotropic rod of finite length. The one dimensional wave equation is defined as

$$\frac{\partial^2 u}{\partial t^2} = V^2 \frac{\partial^2 u}{\partial x^2} \quad (1.5)$$

where $x$ is the horizontal distance (m), $u$ is the displacement in the $x$ direction (m), $t$ is the time (s), and $V$ is the wave propagation velocity (m/s). For longitudinal wave propagation, the wave equation has the form

$$E \frac{\partial^2 u}{\partial x^2} = \rho \frac{\partial^2 u}{\partial t^2} \quad (1.6)$$

where $E$ is Young’s modulus (Pa) and $\rho$ is the mass density (kg/m$^3$). Equation 1.6 can also be written in the form

$$\frac{\partial^2 u}{\partial t^2} = V_p^2 \frac{\partial^2 u}{\partial x^2} \quad (1.7)$$

where
\[ V_p^2 = \frac{E}{\rho} \]  

(1.8)

and \( V_p \) is the compressional/longitudinal wave speed (m/s). The free-free resonant column test is built from this theory and seeks to characterize \( V_p \) by exciting a cylindrical specimen of length \( l \) and diameter \(< l/2\) with a low strain impulse load to excite the natural modes of vibration (ASTM C215). This theory was experimentally validated by Baker et al. (1995), where the arrival times of various types of waves were characterized. Ryden et al. (2006) determined that the compressional wave velocity for a rod of length \( l \) (m) with free-free boundary conditions is defined as

\[ V_p = f_p \lambda = f_p (2l) \]  

(1.9)

where, \( f_p \) is the frequency of the first natural mode of longitudinal resonance (s\(^{-1}\)), and \( \lambda \) is the wavelength (m). For a free-free rod, \( \lambda = 2l \). Rearranging Equation 1.8 to solve for \( E \) and substituting the \( V_p \) expression in Equation 1.9 into Equation 1.8, the FFR equation is formed,

\[ E_0 = \rho (2f_p l)^2 = \rho (V_p)^2 \]  

(1.10)

where \( E_0 \) is the low-strain seismic Young’s modulus (Pa). In practice, the cylinder’s vibration response is excited via an impulse load and measured from an accelerometer affixed to one end of the cylinder (Figure 1.5). This time-domain response (Figure 1.6a) is filtered with a Tukey window to remove forced vibration effects and is then transformed into the frequency domain via a discrete Fourier transform to locate the first longitudinal resonance frequency (Figure 1.6b). This technique is well established in the literature, and examples can be found in Nazarian et al. (1999), Nazarian et al. (2002), Ryden et al. (2006), Ahnberg and Holmen (2008), Toohey (2009), and Toohey and Mooney (2012).

Another wave propagation technique used in the research herein is crosshole ultrasonic logging (sometimes called crosshole sonic logging, CSL). This technique uses hydrophone transceivers to send and receive pressure waves between fluid-filled casing tubes embedded in cement grouted structures. This approach is traditionally used to assess the quality of drilled
shafts where casing tubes are attached to the reinforcing cage and waves are propagated through concrete (Figure 1.7a). Crosshole logging techniques are generally assumed to follow the 2D wave equation. Zhou et al. (1995) presents the following wave equation for crosshole logging applications,

$$\frac{1}{K(x_r)} \frac{\partial^2 p(x_r, t| x_s)}{\partial t^2} - \nabla \cdot \left[ \frac{1}{\rho(x_r)} \nabla p(x_r, t| x_s) \right] = s(x_r, t| x_s) \quad (1.11)$$

where $K$ is the bulk modulus, $\rho(x_r)$ is the mass density, $p(x_r)$ is the pressure at time $t$ at receiver location $x_r$, and $s(x_r, t| x_s)$ is the source function at $x_s$. The forward model problem is defined as determining the pressure field that satisfies Equation 1.11 with a given set of boundary conditions and initial conditions. The authors propose a fourth-order finite difference solution to the equation of motion. This research does not require determination or inversion of the full waveform, and thus a full derivation is not presented. Reynolds (2011) describes the propagation velocity of a wave travelling through an elastic medium as,

Figure 1.5: Free-free resonance test exciting the longitudinal resonant frequency of a cylinder composed of lime stabilized clay (from Toohey 2009).
where $K$ is the bulk modulus (Pa), $\mu$ is the shear modulus (Pa), and $\rho$ is the mass density (kg/m$^3$).

Because the casings used for hydrophone placement are fluid-filled, no shear waves are transmitted beyond the casing, and thus the arriving wave detected by the receiver is the result of compressional waves only. For the traditional CSL approach used herein, the first arrival time of wave (Figure 1.7b) is used in conjunction with the source/receiver separation distance to estimate the material’s wavespeed. $V_p$ is experimentally estimated by

$$V_p = \frac{d}{t}$$ (1.13)

where $d$ is the distance separating the two hydrophone transceivers (m), and $t$ is the time of the first measured arrival (s). Chan and Tsang (2006) discuss the apparent transmission velocity for a waves travelling from the source hydrophone to the receiving hydrophone. A typical wave path

---

**Figure 1.6:** a) Tukey windowed time domain response from a free-free resonance test, and b) selection of the first resonant peak after performing a discrete Fourier transform on the response from a) (from Toohey 2009).
goes from transmitting hydrophone → water → casing tube wall → concrete → casing tube wall → water → receiving hydrophone. Because of the multiple material interfaces encountered in this travel path, material impedances and transmission/reflection coefficients must be considered. The coefficients of transmission and reflection are shown in Equations 1.14 and 1.15, respectively.

\[ T = \frac{2Z_B}{Z_B - Z_A} \]  
\[ R = \frac{Z_B + Z_A}{Z_B - Z_A} \]

The crosshole logging approach estimates \( V_p \) via the first arrival time of the wave transmitted from the source to the receiver, i.e., reflections are not measured in this approach.

![Figure 1.7](image)

Figure 1.7: a) Illustration of CSL on a drilled pile (from Olson 2015) and b) selection of the first arrival time from a received p-wave (from Olson 2015).

DC resistivity testing is an electrical geophysical technique based on Ohm’s law that has been widely used in geophysical exploration for decades and is becoming more prominent in civil engineering quality assurance and quality control (QA/QC) applications. In practice, DC
resistivity measurements are usually performed using commutated direct current (i.e., a square-wave alternating current) or low frequency alternating current (AC) to assess the real component of the material’s resistivity and avoid material polarization resulting from sustained DC injection. The DC resistivity technique characterizes a material’s electrical resistivity, or ability to resist current flow. The equations governing DC resistivity are summarized by Revil et al. 2012. The principle behind the DC resistivity technique is macroscopically governed by Ohm’s law (Equation 1.16),

\[ j = \frac{E}{\rho} \]  

(1.16)

where \( j \) is the conduction current density (A/m\(^2\)), \( E \) is the electrical field in V/m, and \( \rho \) is the material’s electrical resistivity (\( \Omega \)m). The electric field is defined as the gradient of the electrical potential (Equation 1.17),

\[ E = -\nabla \psi \]  

(1.17)

where \( \psi \) is the electrical potential (V). Equation 1.17 satisfies \( \nabla \times E = 0 \) for the low-frequency limit of Maxwell’s equations. The continuity equation is written as

\[ \nabla \cdot j = \mathcal{S} \]  

(1.18)

where \( \mathcal{S} \) represents a volumetric charge (A/m\(^3\)). For a source term \( \mathcal{S} > 0 \), and for a sink term \( \mathcal{S} < 0 \). For a single point electrode on the surface of a homogeneous semi-infinite half sphere, Ohm’s law has the form

\[ j = \frac{\rho l}{\frac{1}{2}(4\pi r^2)} \]  

(1.19)

where \( r \) is the distance from the current injection electrode (m). In practice, DC resistivity measurements are often obtained from the ground surface (i.e., halfspace) and this case will be
derived herein. In the case of the full space (where the injection electrode is contained within a semi-infinite sphere), the $1/2$ factor in Equation 1.19 is dropped and

\[ j = \frac{\rho i}{(4\pi r^2)} \]  

(1.20)

For constant $\rho$, the first-order differential equation in Equation 1.19 is solved by

\[ \psi(r) = \frac{i\rho}{2\pi r} \]  

(1.21)

In practice, current is injected across two electrodes A and B and the potential difference is measured across electrodes M and N (Figure 7). Revil et al. (2012) describes the potential at any point $P$ as a function of distance from electrodes A and B as,

\[ \psi(P) = \frac{i_{AB}\rho}{2\pi} \left( \frac{1}{r_A} - \frac{1}{r_B} \right) \]  

(1.22)

where, $r_A$ and $r_B$ are the distances between point $P$ and electrodes A and B, respectively. For potentials $\psi(M)$ and $\psi(N)$, which represent the potential at measurement electrodes M and N, respectively,

\[ \psi(M) = \frac{i_{AB}\rho}{2\pi} \left( \frac{1}{AM} - \frac{1}{BM} \right) \]  

(1.23)

\[ \psi(N) = \frac{i_{AB}\rho}{2\pi} \left( \frac{1}{AN} - \frac{1}{BN} \right) \]  

(1.24)

where AM is the distance between electrodes A and M, BM is the distance between electrodes B and M, AN is the distance between electrodes A and N, and BN is the distance between electrodes B and N. Using the principle of superposition, the potential difference across electrodes M and N can be represented as,

\[ \psi_{MN} = \psi(M) - \psi(N) = \frac{i_{AB}\rho}{2\pi} \left( \frac{1}{AM} - \frac{1}{BM} - \frac{1}{AN} + \frac{1}{BN} \right) = \frac{i_{AB}\rho}{k} \]  

(1.25)
where

\[ k = 2\pi \left( \frac{1}{AM} - \frac{1}{BM} - \frac{1}{AN} + \frac{1}{BN} \right)^{-1} \]  

The DC resistivity applications in this research use the Wenner-\( \alpha \) array, which has equal spacing between all adjacent electrodes (i.e., \( AM = MN = NB \)). For the Wenner-\( \alpha \) array with point electrodes on the surface of an semi-infinite homogeneous halfspace,

\[ k = 2 \cdot \pi \cdot a \]  

where \( a \) is the distance between any two adjacent array electrodes (Figure 1.8). For a Wenner-\( \alpha \) full space condition (where in practice electrodes are deep enough in the ground that no surface boundary effects exist),

\[ k = 4 \cdot \pi \cdot a \]  

The difference between \( k \) for half and full space (Equations 1.27 and 1.28) stems from the difference in the representation of Ohm’s law shown in Equations 1.19 and 1.20. Borehole resistivity measurements have a variable \( k \) factor in the near surface region that transitions from a half space to full space condition as surface boundary effects become less prominent (Revil et. al 2012, Guo et al. 2014).

In practice, current is injected across a pair of electrodes (A and B, Figure 1.8) and the potential difference is measured across two or more measurement electrodes of known separation distance (M and N, Figure 1.8). Using the injected current and the measured potential, the material’s resistance \( R \) (\( \Omega \)) is calculated. To obtain the apparent resistivity \( \rho_a \) (\( \Omega m \)) from resistance, a geometric correction factor must be applied,

\[ \rho_a = \left( \frac{\psi_{MN}}{i_{AB}} \right) \cdot k = R \cdot k \]
where $\psi_{MN}$ is the potential difference measured across electrodes M and N (V), $i_{AB}$ is the current injected across electrodes A and B (A), and $k$ is the geometric correction factor (m). Note that $\rho_a$ (obtained from the DC resistivity test) is not the same as a material’s true resistivity $\rho$. $\rho_a$ is a weighted average of all $\rho$ in the volume of material influenced by the injected electrical field. For a homogeneous medium, $\rho = \rho_a$, but in heterogeneous media, $\rho_a$ is influenced by the different values of $\rho$. In practice, $\rho$ is often obtained using inversion of $\rho_a$ data (Revil et al. 2012).

Figure 1.8: Illustration of the electric field created by a traditional surface-based DC resistivity measurement (from Revil et al. 2012).

1.4 References Cited


Ho, C. (2011) "Evaluation of jet grout formation in soft clay for tunnel excavation." Geo-Frontiers 2011, ASCE.


CHAPTER 2:  
A SEISMIC MODULUS MATURITY FUNCTION FOR LIME AND LIME-CEMENT STABILIZED CLAY  

Modified from a paper accepted for publication in the *ASCE Journal of Materials in Civil Engineering* (with permission from ASCE)  

R.G. Bearce and M.A. Mooney  

2.1 Abstract  
Stabilization via lime and/or cement is commonly used to improve poor subgrade soil. The key design parameters for lime and lime-cement stabilized soils (LSS/L-CSS) are strength and stiffness, the growth of which are time and temperature dependent. It is generally understood that increased curing temperature will result in increased LSS/L-CSS strength/stiffness; however, there is no quantitative framework for predicting this behavior. This paper proposes a modulus maturity function for LSS/L-CSS that estimates low strain modulus as a function of the curing duration and the average curing temperature over that duration. To develop the maturity function, non-destructive seismic modulus tests were performed on cylinders cured at varying temperature regimes from three LSS/L-CSS construction sites. Variations in curing behavior were compared within and across sites and least squares regression analysis was performed to assess the functional behavior of the data. Results indicate that seismic modulus growth in LSS/L-CSS is non-linear in both time and temperature and should be characterized via a maturity function.  

2.2 Introduction  
In subgrade soils that are prone to deleterious shrink/swell, lime and/or cement are mechanically mixed into the moisture treated soil prior to compaction. This process increases soil strength and reduces plasticity. From a constructability standpoint, rapid achievement of shear strength (e.g., unconfined compressive strength, $q_u$) is desired to minimize the time until construction of overlying layers can begin. Suitable strength for construction traffic is typically reached within one week. From a pavement design perspective (e.g., AASHTO’s Mechanistic Empirical
Pavement Design Guide, MEPDG), the desired input parameter for a stabilized soil layer is 28-day resilient modulus ($M_r$) determined from specimens cured for 28 days at 23°C. In practice, $M_r$ is often obtained from a correlation to $q_u$ (Thompson 1966, Little 1994, Toohey et al., 2013). Because most construction schedules cannot wait 28 days for performance verification, acceptance of LSS/L-CSS is often based on $M_r/q_u$ testing of specimens cured for 7 days at 41°C (Little 1999, Mallela et al. 2004) with the assumption that this elevated temperature regime and duration is equivalent to 28 days at 23°C. However, an evaluation of the literature on elevated temperature LSS curing (e.g., Biswas 1972, Drake and Haliburton 1972, Townsend and Donaghe 1976, Alexander and Doty 1978, Little 1987, Little et al. 1994, Little 1999, Little 2000, Yusuf et al. 2001, Mallela et al. 2004, Little et al. 2004, Toohey et al. 2013) reveals that the use of a single accelerated curing regime is an over-simplification. While some of these studies attempt to relate LSS/L-CSS performance to curing temperature (e.g., an equivalent accelerated curing regime), none of these studies have proposed a quantitative relationship between the measured parameter ($q_u/M_r$) and both time and temperature simultaneously (i.e., a maturity function for LSS/L-CSS).

This paper presents the development of a time and temperature maturity index for predicting elastic modulus growth of LSS and L-CSS soils during curing. Low strain seismic modulus testing, a well-accepted non-destructive approach (Nazarian 2002, Ryden 2004, Ahnberg and Holmen 2008, Toohey and Mooney 2012), is used to enable testing on specimens throughout curing. The low strain or seismic modulus ($E_0$) is also well correlated to $q_u$ (Toohey and Mooney 2012) and $M_r$ (Williams and Nazarian 2007) making it an attractive method for the characterization of stabilized soil as it provides simultaneous assessment of both strength and stiffness. The maturity index was developed from a study across multiple LSS/L-CSS projects in which seismic modulus growth was monitored throughout the curing process. Specimens were cured at several constant and variable (field) temperature regimes. After considering maturity function approaches from related fields (e.g., the Portland cement community), least squares regression analysis was performed on the experimental data to develop a maturity index for LSS/L-CSS to characterize $E_0$ growth as a non-linear function of both time and temperature.
2.3 Experimental Procedure

The estimation of $E_0$ via FFR testing is a well-developed technique wherein a prismatic cylindrical specimen of length $L$ and diameter $< L/2$ is subjected to impulse loading under free-free boundary conditions to induce resonant vibration (Figure 2.1). $E_0$ is estimated from the observed resonant response via Eq. 2.1 derived assuming one-dimensional wave propagation theory.

$$E_0 = \rho(2f_p l)^2 = \rho(V_p)^2$$  (2.1)

Here, $\rho$ is the cylinder's mass density (kg/m$^3$), $f_p$ is the longitudinal resonant frequency (s$^{-1}$), $l$ is the cylinder length (m), and $V_p$ is the material's p-wave velocity (m/s). FFR testing was performed on each cylinder daily for the first 14 days, and every other day between days 15 and 28+.

Figure 2.1: (a) Five stacked time histories from FFR testing and (b) discrete Fourier transform of the time histories in (a) used to identify the resonant frequency.

FFR testing was performed on field-mixed LSS/L-CSS from three construction sites near Denver, Colorado, USA. All sites were treated with lime (in the form of quicklime slurry),
allowed to mellow for three days, and remixed with moisture conditioning prior to final compaction. Sites 1 and 2 were treated with cement powder prior to remix. Unmodified soil properties and lime/cement mix designs are summarized in Table 2.1. No additional mineralogical testing was performed on the soils in this study. Lime/cement mix designs for each site were selected by local geotechnical testing laboratories. Optimum mix design was determined based on the Eades and Grim test (ASTM D6276) and shrink/swell test (ASTM D3877). Given the range of soil types suitable for stabilization, the soil types and mix designs from this study are relatively similar. FFR testing was performed on 10cm diameter by 20cm long cylinders prepared from field-mixed soil and cured at varying temperatures to assess the effects of temperature on curing. The number of cylinders and corresponding temperature curing regimes for each site are summarized in Table 2.1. The recommended moisture content for each mix was to range from optimum moisture content (OMC) to 3% greater than OMC. Sites 2 and 3 meet this condition, while site 1 field soil moisture content was less than OMC.

Table 2.1: Soil properties/classifications and lime/cement mix designs for all soils evaluated in this research. FFR testing regime with number of cylinders per temperature regime and average moisture content of field-mixed soil from each site.

<table>
<thead>
<tr>
<th>Site</th>
<th>AASHTO/USCS Class.</th>
<th>LL/PI</th>
<th>Max Dry Unit Weight (kN/m$^3$)$^1$</th>
<th>Opt. Moisture Content (%)</th>
<th>$E_u^2$ (MPa)</th>
<th>% Lime/Cement</th>
<th>FFR Cylinders (# tested)</th>
<th>Avg. Moisture Content (%)$^4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A-7-6 / CH</td>
<td>55/36</td>
<td>1762</td>
<td>21.0</td>
<td>200</td>
<td>5.5/3.0</td>
<td>0 5 5 0</td>
<td>19.1</td>
</tr>
<tr>
<td>2</td>
<td>A-7-6 / CH</td>
<td>53/30</td>
<td>1729</td>
<td>18.8</td>
<td>270</td>
<td>5.0/2.0</td>
<td>5 10 10 5</td>
<td>21.2</td>
</tr>
<tr>
<td>3</td>
<td>A-7-6 / CH</td>
<td>51/32</td>
<td>1565</td>
<td>20.5</td>
<td>220</td>
<td>5.0/0</td>
<td>5 5 5 5</td>
<td>22.5</td>
</tr>
</tbody>
</table>

$^1$Obtained via standard Proctor testing  
$^2$Average $E_u$ of 3 FFR specimens constituted from unmodified field soil at OMC  
$^3$Field temperature is inherently variable and cannot be characterized with a single temperature value  
$^4$Average moisture content of the field-collected soil used to prepare FFR cylinders

To prepare FFR test cylinders, LSS/L-CSS was gathered prior to final field compaction and compacted in cylindrical molds in four 5cm lifts. This specimen preparation process mimics the technique recommended by AASHTO T294 and successfully used in other studies (e.g., Toohey and Mooney 2012). Compacted LSS/L-CSS cylinders were sealed in plastic bags to preserve moisture content. Cylinders to be cured at constant temperature were returned to the laboratory, while field-cured cylinders were stored in an on-site soil trench to mimic the
temperature experienced by the field-constructed soil. Temperature probes were placed in both laboratory storage containers and the on-site soil trench to monitor curing temperature.

2.4 Results

To illustrate modulus growth experienced by LSS/L-CSS, $E_0$ is plotted vs. curing time for each temperature curing regime across all three sites in Figures 2.2-2.5. Figure 2.2 shows that $E_0$ for $T=23^\circ\text{C}$ (site 1) increases significantly early on (doubles in 2 days) and continues to increase through the 28 day curing regime. While all site 1 specimens exhibit similar day 0 $E_0$ values, the elevated field temperature over the first 24 hours ($31^\circ\text{C}$ vs. $23^\circ\text{C}$) leads to a significant difference in $E_0$ after day 1. The slightly higher field temperature from day 2 through day 15 (average $26^\circ\text{C}$ vs. $23^\circ\text{C}$) leads to continued separation between field and lab $E_0$. Inspection of these data shows that the rate of $E_0$ increase, i.e., slope of the growth curve, is greater for the higher field temperature for days 1-14. From day 15 onward, the field and lab temperatures are more similar, and in general, the modulus curves remain parallel. Lab and field $E_0$ values continue to grow beyond 28 days as consistent with the literature (Boardman et al. 2001, Kavak and Baykal 2012, Zhang et al. 2014).

The nature of the temperature influence at site 2 is similar to site 1, as shown in Figures 2.3 and 2.4. Similar behavior would be expected for these two sites given the similarities between the soils and mix designs (Table 2.1). Site 2 contained two zones (Z1 and Z2) that correspond to two different areas on site prepared with the same soil type and lime/cement mix design. Site 2 $E_0$ growth rates for the higher temperature field specimens are greater from inception through day 15. Thereafter, when field and lab temperature are similar, the field cylinder growth rate appears less than lab cylinder growth rate for Z1 and Z2. Inspection of data from cylinders cured at more extreme temperatures (i.e., $T=8^\circ\text{C}, 41^\circ\text{C}$) reveals more significant variation in modulus growth behavior (Figure 2.4). $E_0$ for $T=41^\circ\text{C}$ grows significantly from day 0 to day 1 (i.e., day 1 $E_0$ is approximately five times greater than day 0 $E_0$). Growth from day 1 to day 2 is one-half of that observed between day 0 and day 1, but is still significantly greater than the $E_0$ growth rate of cylinders at any other curing regime for day 2. Day 3 $E_0$ growth is approximately one-half of day 2, and from day 3 onwards, no significant $E_0$ growth is observed. The exact reason for this halt in $E_0$ growth is unknown, but it is likely related to the complete consumption of one or more of the necessary reactants (i.e., lime/cement/water).
Figure 2.2: (a) $E_0$ vs. $t$ and (b) field and lab temperatures vs. $t$ for site 1 L-CSS cylinders. All cylinders were sampled and compacted from field-mixed L-CSS. Field results reflect cylinders buried in a field trench and subjected to field conditions; lab results reflect cylinders cured at constant temperature in sealed plastic bags.

Figure 2.3: (a) $E_0$ vs. $t$ and (b) field and lab temperatures vs. $t$ for site 2 (zone 1) L-CSS cylinders. Note that day 0 data were not recorded but all cylinders were prepared from the same soil at the same time in the same way and therefore would be expected to be similar.
Further evidence of complete reactant consumption is observed in the field-cured cylinders. Upon reaching the same level of $E_0$ growth as the $T=41^\circ C$ lab cylinders (after 14 field days compared to 4 lab days), the field-cured cylinders undergo no further $E_0$ growth (Figure 2.4). Low temperature also has a significant slowing effect on $E_0$ growth. $E_0$ for $T=8^\circ C$ cylinders (Figure 2.4) undergo limited growth. For example, day 28 $E_0$ is only three times greater than day 0 $E_0$. This result is consistent with literature as Mallela et al. 2004 reports that curing temperatures below 8 °C can significantly slow or halt the pozzolanic reactions responsible for long term strength/stiffness development.

Data from site 3 (Figure 2.5) reveal the same general trend as data from site 1 and 2 L-CSS cylinder tests, namely that increased temperature results in increased $E_0$ growth. In $T=41^\circ C$ site 3 cylinders, $E_0$ growth is discernibly less than $T=41^\circ C$ site 2 $E_0$ growth rates (Figure 2.4). Furthermore, appreciable $E_0$ growth in site 3 $T=41^\circ C$ cylinders continues for 12 days (compared to 3 days with site 2) and maximum achieved $E_0$ for site 3 (4380 MPa) is noticeably greater than that of site 2 (3100 MPa). A plausible explanation for this difference in behavior is the lack of cement in the site 3 material.
Figure 2.5: (a) $E_0$ vs. $t$, and (b) lab and field temperatures vs. $t$ for site 3 LSS cylinders.

During the stabilization process both lime and cement react with the same finite supply of both water and soil minerals. Thus, lime/cement reacting simultaneously will consume the available reactant supply faster than lime alone. A comparison of LSS and L-CSS $E_0$ growth curves suggests that the use of lime-cement mixes results in faster initial $E_0$ growth at the cost of faster reactant consumption (and thus an earlier halt in $E_0$ growth). This effect is best observed in the $T=41^\circ$C data, but is also observed in sites 1 and 2 field data (Figures 2.2-2.4), in which the rate of field-cured L-CSS $E_0$ growth is noticeably reduced after 15 days. A comparison of field curing between LSS and L-CSS is not readily available as the L-CSS sites were cured at significantly higher temperatures than the LSS site. $T=23^\circ$C cylinders of LSS and L-CSS experience appreciable $E_0$ growth through 28 days of curing and have not exhausted any reactant to cause a halt in $E_0$ growth.

Toohey and Mooney (2012) concluded that $E_0$ growth with curing time in LSS cylinders cured at 23°C follows a power model (Eq. 2.2),

$$E_0(t) = E_{1-day} \left(\frac{t}{t_0}\right)^\alpha \hspace{1cm} (2.2)$$
where \( t \) is the curing duration (days), \( t_0 \) is a normalizing time equal to 1 day, \( E_{1\text{-day}} \) is the \( E_0 \) value after 1 day of curing (MPa), and \( \alpha \) is an empirical parameter ranging from 0.3-0.7. To assess the ability of the power model to predict \( E_0 \) growth at varying temperature, least squares regression fitting was performed on \( E_0 \) data across each set of five specimens from all sites and temperature curing regimes. Best fit power models are plotted over corresponding experimental \( E_0 \) data in Figures 2.2-2.5 with corresponding \( E_{1\text{-day}} \), \( \alpha \), and \( R^2 \) values for each fit reported in Table 2.2. While the \( R^2 \) values for most power model fits are very high, visual inspection suggests that the power model alone cannot adequately characterize \( E_0 \) growth, especially at higher temperatures. Power model fits for the 41°C \( E_0 \) data appear nearly linear, underestimating \( E_0 \) early on and overestimating \( E_0 \) at later curing times (Figures 2.4 and 2.5). Furthermore, the best fit \( E_{1\text{-day}} \) (i.e., Eq. 2.2) obtained from regression analysis is different than the actual \( E_{1\text{-day}} \) (i.e., the experimental \( E_0 \) after one day of curing) (Table 2.2). This result suggests that the power model alone cannot capture the higher rate of \( E_0 \) growth that results from elevated curing temperature and additional functional components would be required to fully capture the behavior observed over a wide range of curing temperatures.

Table 2.2: Parameter values from least squares regression power model fits for curves in Figures 2.2-2.5.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Site 1</th>
<th>Site 2-Z1</th>
<th>Site 2-Z2</th>
<th>Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>23°C</td>
<td>Field</td>
<td>23°C</td>
<td>Field</td>
</tr>
<tr>
<td>( E_{1\text{-day}} ) (fit) (MPa)</td>
<td>690</td>
<td>683</td>
<td>710</td>
<td>1202</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>0.34</td>
<td>0.40</td>
<td>0.42</td>
<td>0.36</td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.95</td>
<td>0.92</td>
<td>0.99</td>
<td>0.99</td>
</tr>
<tr>
<td>( E_{1\text{-day}} ) (actual) (MPa)</td>
<td>750</td>
<td>1200</td>
<td>655</td>
<td>1108</td>
</tr>
</tbody>
</table>

\(^1\)Power model regression analysis of site 1 data was performed for days 1-62 (compared to days 1-28 on all other sites).

While there is limited literature on the time/temperature dependent \( E_0 \) growth of LSS/L-CSS, Silva et al. (2013) studied the \( E_0 \) growth of cement-stabilized river sand (SW). The authors found that 28-day \( E_0 \) values ranged between 4000-5000 MPa when treated with 7% cement and 9% water. Given the higher cement content and different soil type, the increased performance of this CSS is a plausible result. Specimens were cured at 19°C and \( E_0 \) values were determined using \( V_p \) and Eq. 1. Regression analysis of the experimental data predicted \( E_0 \) growth in CSS (SW) followed the form of Eq. 2.
\[ E(t) = a e^{-\frac{1}{2}(\frac{t}{\tau})^B} \]  

(2.3)

where \( a, \tau, \) and \( B \) are empirical fitting parameters related to cement content. This function results in higher initial modulus growth and a modulus growth rate that decreases with increased curing time, similar to the power model approach of Toohey and Mooney (2012).

### 2.5 Development of a LSS and L-CSS Maturity Index

Data from this study (Figures 2.2-2.5) indicate that \( E_0 \) growth in LSS/L-CSS is highly affected by the curing temperature regime. A summary of the similarity in LSS and CSS chemical reactions coupled with a review of the research on quantifying temperature effects on Portland cement reaction rates provides a basis for the development of a maturity index for LSS and L-CSS. Test sites in this study were treated with quicklime slurry (i.e., a mix of quicklime and water that produces hydrated lime). When fine-grained soils are mixed with hydrated lime, the improvement in soil properties occurs via modification and stabilization. The mechanisms governing soil modification are cation exchange and flocculation-agglomeration, which occur rapidly and result in soil plasticity reduction and \( q_u \) increase (Little 1987, Mallela et al. 2004). Excess \( \text{Ca}^{++} \) cations from the quicklime replace weaker metallic cations in the soil causing a size reduction in the diffuse water layer around the clay minerals. This reduction in diffuse layer size allows the clay particles to flocculate (Mallela et al. 2004). The effects of soil modification are apparent when comparing unmodified soil modulus \( E_u \) and 0-day \( E_0 \) from this study (i.e., the day 0 \( E_0 \) is 1.5-2.0 times greater than \( E_u \) for each site/soil). The effects of soil modification are further evidenced in the data as 0-day \( E_0 \) values are very similar for each site; however, \( E_0 \) values at later days diverge according to curing temperature.

Soil stabilization occurs via time/temperature dependent pozzolanic reactions that occur over a time span of days to years, depending on reactant supply. When a sufficient quantity of hydrated lime is introduced into fine-grained soil, there is a significant increase in soil pH. The higher pH increases the solubility of silica/alumina compounds present in the clay minerals and results in the formation of cementing agents (Little, 1987). The long term \( E_0 \) gain observed in the data (Figures 2.2-2.5) is the result of pozzolanic reactions. Eq. 2.4 shows the chemical reaction that produces calcium hydroxide \( (\text{Ca(OH)}_2) \) when quicklime (\( \text{CaO} \)) and water (\( \text{H}_2\text{O} \)) are mixed (Little et al. 1995).
CaO + H₂O → Ca(OH)₂ \hspace{1cm} (2.4)

In the presence of water, pozzolans (compounds usually consisting of silicates, alumina, and/or alumino-silicates) react with calcium hydroxide to form the cementitious compounds calcium-silicate-hydrate (CSH) and calcium-aluminate-hydrate (CAH). An example reaction between calcium hydroxide and an alumino-silicate is shown in Eq. 2.5 (West and Carder 1997).

2(Al₂O₃ · 2SiO₂) + 7Ca(OH)₂ → 3CaO · 2SiO₂(q) + 2(2CaO · Al₂O₃ · SiO₂(q)) \hspace{1cm} (2.5)

The compound 3CaO·2SiO₂(q) is a CSH. Similar reactions between calcium hydroxide and other pozzolans yield various CSH and CAH compounds. CSH and CAH are some of the same types of cementitious hydrates formed during the hydration of Portland cement (Terrel et al. 1979). The majority of Portland cement hydration occurs via the combination of water and either tricalcium silicate (Eq. 2.6) or dicalcium silicate (Eq. 2.7) to form CSHs (Taylor 1997).

2(CaO)₃(SiO₂) + 7H₂O → (CaO)₃ · (SiO₂)₂ · 4H₂O + 3Ca(OH)₂ \hspace{1cm} (2.6)

2(CaO)₂(SiO₂) + 5H₂O → (CaO)₃ · (SiO₂)₂ · 4H₂O + Ca(OH)₂ \hspace{1cm} (2.7)

Similar reactions occur between alumina compounds (e.g., (CaO)₃·Al₂O₃) and water to form CAHs. These reactions continue until reactant supply is consumed.

The maturity approach is well-accepted within the Portland cement concrete community, (e.g., ASTM C 1074), and thus when considering a maturity function for LSS/L-CSS, the maturity equations used by the concrete community are a natural starting point. Studies on accelerated concrete curing methods by McIntosh (1949), Nurse (1949), and Saul (1951) led to the development of the Nurse-Saul maturity function (Eq. 2.8).

\[ M(t, T) = \Sigma_0(T - T_0)\Delta t \hspace{1cm} (2.8) \]

Here, \( M \) is maturity (°C·days), \( T \) (°C) is the average concrete temperature during the curing interval \( \Delta t \), \( T_0 \) is the datum temperature usually taken to be -10 °C, \( t \) is the elapsed time (hours or
days), and $\Delta t$ is the time interval (hours or days). This function describes the maturity of curing concrete as a linear function of both time and temperature. Freiesleben et al. (1977) developed an equivalent age maturity function (Eq. 2.9),

$$t_e(t, T) = \sum_0^t e^{-\frac{E}{R} \left( \frac{1}{T} - \frac{1}{T_r} \right)} \Delta t$$

where $t_e$ is the equivalent age of the concrete at the reference temperature, $E$ is the apparent activation energy (J/mol), $R$ is the universal gas constant (8.314 J/mol-K), $T$ is the average absolute temperature of the concrete during the time interval $\Delta t$ (°K), and $T_r$ is the absolute reference temperature (°K). The equivalent age approach extends beyond the Nurse-Saul equation in that it describes strength growth as a linear function of time and an exponential Arrhenius function of temperature and activation energy (related to the cement content/type).

Kinetic energy within molecules increases when a system is heated, and more energetic molecules increase the rate of chemical reactions in the system. As kinetic energy is added to the system (i.e., $T$ increases) the reaction rate is increased via the exponential function. This temperature-induced increase in the reaction rate results in faster strength gain (and thus, a higher equivalent age). Numerous other studies on Portland cement concrete and cement stabilized soil have characterized the relationship between reaction rate and curing temperature using an Arrhenius equation (e.g., Kim et al. 1998, Carino and Lew 2001, Kim et al. 2001, Chitambira 2004, Yi et al. 2005, Chitambira 2007, Daniels et al. 2010, Zhang et al. 2014, etc.). Because the pozzolanic reactions that occur between lime/soil/water produce similar cementitious compounds to those of Portland cement hydration, it is plausible that LSS reaction rate as a function of curing temperature is also governed by an exponential equation.

The literature suggests that LSS/L-CSS $E_0$ growth at constant temperature is governed by a function that decays as time increases. Studies on Portland cement concrete and cement stabilized soil have proposed multiple exponential equation approaches to describe the increase in reaction rate (and therefore strength gain) that results from curing at elevated temperatures. To evaluate the simultaneous time/temperature dependence on LSS/L-CSS $E_0$ growth, least squares regression analysis was performed on $E_0$ data from all sites and curing regimes to correlate $E_0$ to curing temperature for each curing day. The relationship between $E_0$ and the cumulative average curing temperature ($\bar{T}_{\bar{t}}$) was assessed using several functional forms (e.g., linear, power model,
exponential, logarithmic, etc.). An exponential function provided the best fit across all data ($R^2 = 0.88-0.92$) and mimics the exponential temperature behavior observed in literature.

This relationship is shown for curing days 3, 7, 14, and 28 in Figure 2.6, where each $E_0$ data point is the average $E_0$ across five specimens from a specific site and temperature curing regime for a specific day (Table 2.1). $T=41^\circ C$ data is not included after $E_0$ growth plateau, and thus is only shown for 3 day (Figure 2.6a) and 7 day (Figure 2.6b). The functional form observed in Figure 2.6 is shown in Eq. 2.10,

$$E_0(T, \bar{\eta}_t, \beta) = \bar{\eta}_t e^{\beta(T/T_0)}$$

(2.10)

where $E_0$ is the seismic modulus predicted by the maturity index, $\bar{T}_t$ is the cumulative average curing temperature ($^\circ C$), $T_0$ is a normalizing parameter equal to $1^\circ C$, $\beta$ is an empirical parameter related to the lime/cement content.

For the regression analysis performed herein, $\beta$ remains relatively constant (Figure 2.7b) and is prescribed a constant value of 0.05. $\bar{\eta}_t$ is an empirical parameter that increases with curing day (Figure 2.7a) and is analogous to the $\sum \Delta t$ portion of Eq. 2.9; however, the increase in $\bar{\eta}_t(t)$ is not linear. As evidenced in Figure 2.6 (terms preceding the exponent) and shown in Figure 2.7a (for curing days 1-28), $\bar{\eta}_t(t)$ follows a power model ($R^2=0.99$) as expressed in Eq. 2.11.

$$\bar{\eta}_t(t, \eta_0, \alpha') = \eta_0 \left(\frac{t}{t_0}\right)^{\alpha'}$$

(2.11)

Here, $t$ is the curing day, $t_0$ is a normalizing parameter equal to 1 day, $\eta_0$ is an empirical parameter equal to 241, and $\alpha'$ is an empirical parameter equal to 0.35. Note the labeled data points in Figure 2.7a that correspond to the $\eta_t$ terms for days 3, 7, 14, and 28 obtained from regression analysis in Figure 2.6. Portland cement maturity functions (Eq. 2.8 and Eq. 2.9) have proposed that the reaction rate remains linear with curing time (i.e., $\sum \Delta t$); however, available reactants are being consumed throughout the course of curing, and thus the potential for additional stabilizing reactions would diminish over time (i.e., a power model). Eq. 2.11
Figure 2.6: Correlation between $E_0$ and curing temperature for (a) day 3, (b) day 7, (c) day 14, and (d) day 28.

describes the potential for $E_0$ growth as a function of curing time ($\bar{n}_{t}(t)$) which is modified by an exponential function (Eq. 2.10) to account for variation in curing temperature. By using the average curing temperature over the curing duration, the $\bar{E}_0$ function is stable at greater curing days (i.e., an increase in temperature at a later day would result in a smaller $\bar{E}_0$ gain than the same temperature change on an earlier day). This result is logical in that as time progresses,
fewer reactants would remain and thus an increase in temperature would have a smaller effect on the $\tilde{E}_0$ growth compared to earlier curing times when more reactants are unconsumed.

Substituting the expression for $\tilde{\eta}_t(t)$ (Eq. 2.11) into Eq. 2.10, a new expression for $E_0$ growth is developed that serves as a seismic modulus maturity index for LSS/L-CSS (Eq. 2.12).

$$\tilde{E}_0(\bar{T}_t, t) = \left[ \eta_0 \left( \frac{t}{t_0} \right)^{\alpha'} \right] e^{\beta(\bar{T}_t/T_0)}$$  \hspace{1cm} (2.12)

The power model function in Eq. 2.12 characterizes the time dependence on $\tilde{E}_0$ growth and is similar to the form Eq. 2.2 (Toohey and Mooney 2012). The exponential function in Eq. 2.12 characterizes the temperature-dependence in $\tilde{E}_0$ growth and is also similar to approaches used in literature (e.g., Chitambira 2007). Relatively similar soil types and mix designs, along with the inherent uncertainty of field-mixed soil, make it difficult to quantify $\eta_0$, $\alpha'$, and $\beta$ as a function of soil type or additive content. Given constant temperature and curing duration (and assuming

Figure 2.7: (a) Power model function from regression analysis for empirical parameter $\tilde{\eta}_t$ with individual data points from regression analysis($\eta_t$) used to obtain this power model fit, and (b) $\beta$ values at each curing day obtained from regression analysis.
adequate moisture conditioning), an increase in $E_0$ would likely be the result of increased additive content. Assuming constant $t$ and $T_e$, the maturity index captures variable $E_0$ behavior by adjusting the empirical parameter values. Increasing $\eta_0$, $\alpha'$, or $\beta$ results in a larger $E_0$, which would be physically indicative of a larger quantity of additive (and thus larger $E_0$); however, additional testing with greater variation in additive content would be necessary to fully validate this conclusion.

Figure 2.8: (a) A family of curves describing the progression of $E_0$ growth as a function of time and temperature (per Equation 2.12), and (b) average temperature inputs used to generate the growth curves in (a).

Using the empirical parameter values obtained from regression fitting ($\eta_0 = 241$, $\alpha' = 0.35$, and $\beta = 0.05$), Eq. 2.12 generates a family of curves to predict $E_0$ growth for both constant and variable temperature curing (Figure 2.8). Each curve represents an individual curing day over a range of temperatures. Following $T_e$ through each curing day curve, the $E_0$ growth profile emerges. For example, if a specimen was cured for 28 days at
a constant $T = 23^\circ C$ as shown in Figure 2.8b, the progression of $E_0$ per Eq. 2.12 would follow the vertical path a illustrated in Figure 2.8a (with $\bar{T}_t = 23^\circ C$ for $t = 1, 2, 3, \ldots 28$). Conversely, if the daily temperature varied (Figure 2.8b), the progression of $E_0$ per Eq. 2.12 would follow path b (Figure 2.8a). Using the average $T$ for day 1 through the current day helps to account for the temperature history of the curing and is consistent with approaches employed by previous studies (i.e., Eq. 2.8 and Eq. 2.9).

The application of the LSS/L-CSS maturity index is demonstrated in Figure 2.9 where each experimental data set (i.e., the average $E_0$ across all sites cured at a given temperature regime, Table 2.1) is compared to the $E_0$ predicted by Eq. 2.12 ($\tilde{E}_0$). Note that each $\tilde{E}_0$ curve in Figure 2.9 is surrounded by a dashed line envelope, which corresponds to $\bar{T}_t \pm 1^\circ C$. While the precision of the temperature monitoring was 0.01 $^\circ C$, the accuracy with which this temperature could be maintained in the laboratory was $\pm 1^\circ C$. These envelopes help to account for the range of behavior that could be expected as a result of small temperature variation. Experimental $E_0$ and predicted $\tilde{E}_0$ for $\bar{T}_t = 41^\circ C$ show reasonable agreement over all days except day 1 (it should be noted that data sets were no longer plotted after the $E_0$ growth plateau). Eq. 2.12 overpredicts day 1 $E_0$ for $\bar{T}_t = 41^\circ C$ but is very close to the experimental $E_0$ values for all other days prior to the $E_0$ plateau. $E_0$ and $\tilde{E}_0$ for $\bar{T}_t = 23^\circ C$ show reasonable agreement as well. The experimental $E_0$ points are greater than the $\bar{T}_t = 23^\circ C$ line after 10 days of curing, but are still within the dashed line envelope (i.e., lower than the values predicted by $\bar{T}_t = 24^\circ C$). Experimental $E_0$ and predicted $\tilde{E}_0$ for $\bar{T}_t = 8^\circ C$ show good agreement up to day 13, after which $E_0$ exceeds $\tilde{E}_0$. Some $E_0$ values also fall outside the dashed line envelope after day 13, but this envelope is notably narrower at $8^\circ C$ than at higher curing temperatures because of the exponential influence of temperature.

The LSS/L-CSS maturity function can also be used to predict $E_0$ growth in field-cured soil with varying temperature. The $\tilde{E}_0$ curves shown in Figure 2.10a are generated with the same process described above, but $E_0$ values for Figure 2.10a are from sites 1 and 3 field-cured cylinders. Figure 2.10b displays the $\bar{T}_t$ experienced by the field-cured cylinders from each site. These $\bar{T}_t$ values are used as the inputs to Eq. 2.12 to generate the $\tilde{E}_0$ curves for each site in Figure 2.10a. Results suggest that the LSS/L-CSS maturity index can also capture the behavior of soils cured at varying temperatures (as would always be experienced in the field). Furthermore, the maturity function captures variable temperature curing behaviors across the range of temperature regimes experienced in the field.
As the soils for all three sites were mixed with large-scale machinery and with relatively imprecise application of water/lime/cement (i.e., dropped/sprayed out of a truck as opposed to precise laboratory weighing/mixing), the data used to construct the LSS/L-CSS maturity function contain inherent scatter. This data scatter is reflected in the best fit empirical parameters ($\eta_0$, $\alpha'$, and $\beta$), which serve as average values across all three sites. Regression fitting across individual sites would result in variation of these reported parameter values. Furthermore, the effects of cement powder in addition to lime would also have an effect on empirical parameter values (i.e., sites 1/2 vs. site 3) but this study does not contain enough data with lime only to reasonably distinguish this difference. The sites evaluated in this study contain soil that is relatively similar (Table 2.1), and thus, variation in soil type would likely result different values of $\eta_0$, $\alpha'$, and $\beta$ as well. However, the authors believe that the functional form proposed in Eq. 2.12 would remain the same for most combinations of soil and lime/cement stabilizer.

Figure 2.9: Comparison of constant curing temperature FFR $E_0$ data to $\tilde{E}_0$ predicted by the maturity function. Dashed line envelopes display $\tilde{E}_0$ for $\bar{T}_t \pm 1$ °C to illustrate the $\tilde{E}_0$ growth variation resulting from minimal temperature changes. Each $\tilde{E}_0$ curve is plotted with the parameter values obtained from regression analysis but at different constant temperature regimes.
Figure 2.10: (a) Variable (field) curing temperature $E_0$ compared to $\bar{E}_0$ predicted by the maturity index for field-cured cylinders from sites 1 and 3 and (b) average temperature field temperature. Note that site 1 data corresponds to Figure 2.8a and b (path b, variable temperature).

2.6 Conclusions

FFR data were collected from LSS/L-CSS cylinders cured at several temperatures to assess $E_0$ growth behavior. Inspection of data between LSS and L-CSS sites suggests that the addition of cement powder with quicklime induces somewhat different behavior. Primarily, it appears that soil stabilized with lime undergoes more gradual $E_0$ growth than L-CSS. The LSS also achieves a higher peak $E_0$ than the L-CSS studied, but this difference is potentially related to variation in soil type and not a direct result of cement powder addition. The data scatter and similar range of modulus values among these sites make it difficult to accurately decouple the individual effects of lime and cement. The inherent variability associated with field soils and LSS/L-CSS application/production further increases data scatter, but any field site would be subject to this variability.

Regression analysis of FFR results revealed that $E_0$ growth in LSS/L-CSS should be characterized as a non-linear maturity function of both time and temperature. A maturity function was developed to describe $E_0$ growth in LSS/L-CSS as a function of curing duration and average
temperature over that curing duration. The maturity function exhibits power model behavior with curing time, exponential behavior with curing temperature, and also depends on three constant empirical parameters $\eta_0$, $\alpha'$, and $\beta$. Best fit parameter values used in this research are based on data averages across all sites. The maturity function adequately captures $E_0$ growth as a function of time and temperature for both constant and variable field curing temperatures (Figures 2.9 and 2.10).

Given the similarities in soil properties and mix designs (i.e., all three test site soils were A-7-6/CH with 5-5.5% lime and 0-3% cement), it is not unreasonable to expect that variation in soil type or mix design could result in different best fit values for $\eta_0$, $\alpha'$, and $\beta$. While all $E_0$ growth may not be described adequately with the parameter values used proposed in this paper, the form of the maturity function (i.e., Eq. 2.12) should adequately describe the behavior given appropriate empirical parameter values.

Mix design studies, which are frequently conducted prior to field-scale LSS/L-CSS application, could include FFR $E_0$ growth characterization and application of the LSS/L-CSS maturity function with best fit $\eta_0$, $\alpha'$, and $\beta$ for a given soil and mix design. $E_0$ growth for applications of the same soil and mix design could be more reliably predicted given $\bar{T}_t$, and $t$ of the field conditions, but additional study and application to field data would be necessary to fully verify this conclusion.

2.7 Acknowledgements:
The authors wish to thank the Colorado Department of Transportation for providing the funding to conduct this research. In addition, the authors wish to thank ARS, Inc. for providing access to field-constructed lime and lime-cement stabilized subgrade sites.

2.8 References:


CHAPTER 3:
CHARACTERIZATION OF SIMULATED SOILCREETE COLUMN CURING USING
ACOUSTIC TOMOGRAPHY

Modified from a paper published in the proceedings for the ACSE GeoCongress 2014
Conference in Atlanta, GA (with permission from ASCE)

R.G. Bearce, M.A. Mooney, E. Niederleithinger, and A. Revil

3.1 Abstract

Implementation of soilcrete columns via jet-grouting or deep soil mixing to stabilize
problematic subsurface soils is common in underground construction. However, industry is faced
with limited options to characterize column geometry and quality of the resulting soilcrete
without excavation or destructive testing. Laboratory-scale experiments were conducted on
simulated soilcrete columns using crosshole ultrasonic testing to evaluate the feasibility of
acoustic tomography to characterize soilcrete geometry and quality. Data were acquired on
multiple columns immediately after placement up to a curing time of 120 hours. Jet grout
compressional wave velocity (\(V_p\)) was estimated using a first arrival time approach and inverted
to construct acoustic tomograms. Acoustic tomograms indicate that crosshole ultrasonic testing is
able to characterize the changes in acoustic properties that result from jet grout curing, locate
contrasts between weaker/stronger regions in the jet grout, and estimate geometry of the column.

3.2 Introduction

Jet grouting is a widely adopted in-situ ground improvement technique used to create
cylinders of soil-grout or soilcrete (soil-grout mixes). The nature of the technique combined with
uncertainty in ground conditions leads to variability in column geometry and soilcrete properties.
To this end, quality control/quality assurance (QC/QA) inspection techniques are critical to the
successful implementation of jet grouting. Current inspection techniques for jet grouted columns
can be either destructive or non-destructive, and both approaches have limitations. Destructive
methods include the after-construction excavation of columns, penetrometer testing at the
assumed column perimeter, or coring (radial or vertical). These techniques can evaluate both
column geometry and soilcrete quality, but have low coverage (i.e., often only 1-2 locations are evaluated). Furthermore, destructive inspection is costly and inefficient, and often requires a several day curing period for sufficient soilcrete curing.

Non-destructive inspection techniques have included direct current (DC) electrical resistivity, ground penetrating radar, acoustics, and temperature monitoring. The CLYJET system (Frappin, 2011), also known as the electric cylinder method (Frappin and Morey 2001), employs a downhole DC electrical resistivity array on a movable cable that estimates the radius of the jet grout column due to the resistivity contrast between the fresh soilcrete and the in-situ soil. Ground penetrating radar (GPR) has also been used to assess the geometry of jet grout columns by measuring the reflection of incident radar waves at the interface between jet grout and the in-situ soil (T&A Survey 2013). Temperature monitoring of wet soilcrete has been used to estimate the column radius from the results of a sensitivity study as a function of measured temperature and predicted cement content (Meinhard et al. 2010). However, DC resistivity, GPR, and temperature monitoring can only estimate column geometry with no assessment of soilcrete quality. Seismic surface wave and combination surface/downhole acoustic methods have been used to assess soilcrete quality (Madhyannapu et al. 2010), but surface methods are not an ideal approach for jet grout QA because jet grout columns are often very deep in the soil. To evaluate deeper depths, surface wave techniques use lower frequencies, which also increase the wavelength of evaluation (i.e., reduce the resolution). Higher resolution inspection (e.g., two-dimensional, increased spatial coverage, etc.) of soilcrete quality is desirable as soilcrete quality is directly related to the effectiveness of grout infiltration (e.g., for jet grout columns used as permeability barriers, seepage can occur in weak zones and soil inclusions).

An ideal QC/QA approach for jet grouted columns would incorporate non-destructive testing that could estimate both column geometry and soilcrete quality in a shorter time frame (i.e., within 24-48 hours of construction). Parameters measured non-destructively (e.g., $V_p$) could then be correlated to design related properties such as strength and stiffness, although it may be necessary to develop these empirical relationships for each soilcrete mix. This paper investigates the use of crosshole sonic logging (CSL) as a QA inspection technique for curing soilcrete cylinders embedded in a laboratory sand bed. CSL logs are acquired at multiple angles to construct two-dimensional (2D) acoustic tomograms of curing soilcrete and the resulting tomograms are compared to excavated soilcrete specimens for technique evaluation.
3.3 Experimental Setup

A laboratory scale setup was constructed to evaluate the capability of a joint acoustic/electrical approach for determining soilcrete quality and geometry. Due to length limitations, this paper focuses specifically on the implementation and results of acoustic measurement using CSL. The laboratory soil box (Figure 3.1) was 1m on each side and sealed with a plastic sheet for groundwater table simulation. The box was filled with sand and water in five lifts, and a concrete vibratory probe was used to densify the sand every 20 cm. Acoustic and electric instrument arrays were installed during soil placement and densification. A cylindrical soilcrete casing was installed in the center of the box during soil placement. This casing (either 20 or 25 cm diameter) remained hollow until surrounding soil had been placed. Then, the casing was filled with wet soilcrete and extracted such that the soil confined the resulting soilcrete column and instrument arrays (i.e., ensuring contact between the soilcrete and CSL casings).

Traditional CSL was used to acquire acoustic data from 5cm diameter water-filled PVC pipe (CSL system courtesy of Olson Instruments, Wheat Ridge, CO). This system transmits and receives p-waves through two fluid filled casings on either side of the soilcrete column via two hydrophone transceivers with a 45kHz center frequency. The transceivers are placed in the bottom of the casings and moved upwards via a cable reel. The cables are fed through an encoder wheel which triggers the system to send/receive a signal at vertical intervals of 1.8cm. The CSL setup was used to collect amplitude vs. time histories across the curing soilcrete cylinders to estimate the p-wave velocity \( V_P \) of the soilcrete from 20 hours to approximately 120 hours. Starting at hour 20, data was collected every 1-2 hours until signal transmission was achieved (i.e., the wet soilcrete attenuates the CSL signal beyond measurement until a curing time of approximately 20-24 hours). After 24 hours, CSL logs were obtained in 24 hour intervals until no additional changes in soilcrete \( V_P \) were observed (generally after 120 hours). After the final round of testing, the specimen was excavated to verify actual geometry.

3.4 Results

\( V_P \) was estimated using the known spacing between CSL casings and the first arrival time of the P wave. Figure 3.2 illustrates typical arrival times and energy transmission levels for (a) strong soilcrete and (b) weak soilcrete. The strong soilcrete has a faster arrival time (i.e., higher
$V_p$) with significantly less signal attenuation than the weak soilcrete. After determining $V_p$ at each vertical interval, a plot of $V_p$ vs. depth is constructed (Figure 3.3). This profile illustrates both the increasing p-wave velocity as a function of curing time and the emergence of patterns in the velocity profile (i.e., velocity increases at all measurement locations, but the overall shape of the depth profile remains relatively similar). In locations where there is only in-situ soil between the CSL casings, little to no signal is transmitted. For this reason, at the top and bottom of the column, $V_p$ shows significant decreases. These decreases are the result of partial transmission (or guided waves) through some combination of soil and soilcrete, which reduces the signal amplitude and increases the arrival time. As the curing process progresses, the profiles also gain height (on both the top and bottom). This phenomenon occurs because the soilcrete column is hardening, providing a larger zone of possible signal transmission. Strong and weak sections can be identified in these profiles, but as Figure 3.3 only illustrates straight ray paths, the results are essentially one dimensional (as a function of depth). To further improve these results, CSL logs were acquired at angled ray paths and used to construct acoustic ($V_p$) tomograms. The use of angled ray paths and acoustic tomography is a more desirable approach as it allows for the identification of strong/weak zones in two dimensions, and better definition of soilcrete column boundaries.

![Figure 3.1: (a) Photograph and (b) illustration of the experimental setup.](image)
Figure 3.2: Example of raw data time history from CSL for (a) strong soilcrete, and (b) weak soilcrete.

To produce acoustic tomograms, CSL logs were acquired at angles of 0, 10, 20, 30, and 40 degrees, from both directions. An example of full ray path coverage and corresponding excavated specimen are shown in Figure 3.4. Inversion of this data was performed using the commercial software package ReflexW (Sandmeier Scientific Software). The first arrival times are fitted by a SIRT type algorithm (Simultaneous Iterative Reconstruction Technique). Synthetic travel times are calculated using ray tracing for a velocity model with rectangular grid cells. Curved (refracted) rays are considered in the inhomogeneous model. SIRT adapts the model until a sufficient fit between synthetic and measured data is reached. MATLAB algorithms developed in-
house were used for data preprocessing (e.g., discarding erroneous travel times) and display of results.

Figure 3.3: Jet grout velocity vs. depth at increasing curing times using straight ray trace first arrival time approach.

Figure 3.4: Example ray trace pattern used for acoustic tomography (not taking refraction into account) and excavated specimen (to scale) to illustrate ray path coverage.
Acoustic tomograms were calculated for Specimens 1, 2, and 3 (Figures 3.5, 3.6 and 3.7) and indicate that the CSL system can predict the geometry of the excavated specimens as long as the soilcrete is in contact with the casing. These comparisons are taken from 96-120 hour tomograms, and are representative of the final $V_P$ profile of the soilcrete specimen. These tomograms allow for the identification of stronger and weaker zones in the soilcrete, which is an important result as weaker zones represent areas in the soilcrete with lower grout infiltration and possibly soil inclusions. Specimen 3 (Figure 3.7) included intentionally placed soil inclusions that the acoustic tomography was able to locate (i.e., simple ray trace approach only shows weaker depths, not specific location of inclusions). For jet grout applications involving impermeable barriers, soil inclusions often result in seepage zones, and thus, identification of these zones could help stakeholders remediate this issue.

![V_p (m/s)](image)

Figure 3.5: Verification of jet grout size/geometry prediction (Specimen 1) with excavated specimen, and tomogram from data acquired just before excavation.

Time lapse tomograms reveal the curing behavior of the soilcrete and provide an assessment of quality via $V_P$ contrast for Specimens 1 (Figure 3.8), 2 (Figures 3.9 and 3.10), and 3 (Figure 3.11). For high quality soilcrete, $V_P$ in specimens ranges from 1200-
1400 m/s in early stages of curing, and can reach 3000-3200 m/s after 96 - 120 hours of curing. In low quality soilcrete (e.g., Specimen 3), $V_p$ is significantly lower (900-1500 m/s) and does not result in appreciable $V_p$ gain. The results also suggest that a field application of this technology for column geometry determination would be valid after a minimum curing time of 48 hours (i.e., even if it has not reached full curing strength, the entire specimen can be resolved in the acoustic tomogram after 48 hours of curing). Specimen 2 undergoes variable curing behavior (i.e., the top half of the column cures at a slower, less homogenous rate than the bottom half of the column). The specific reason for this anomaly is not known, but it should be noted that this specimen was poured into the casing in two batches. To this end, differences between the top and bottom halves of the column could occur as a result of any unintentional differences in the soilcrete mixes. Specimen 3 was intentionally weakened via a higher soil:grout ratio, and the addition of soil inclusions (Figure 3.7). This specimen is notably weaker than the others and does not undergo significant $V_p$ increase over the course of curing (Figure 3.11). This specimen is representative of poor soilcrete (e.g., low grout infiltration), demonstrating that the CSL/tomography approach can locate potentially problematic low-quality columns.

Figure 3.6: Verification of jet grout size/geometry prediction (Specimen 2) with excavated specimen, and tomogram from data acquired just before excavation.
Some tomograms contain high velocity zones close to the bottom of the model that cannot be explained by the soilcrete columns (e.g. Figure 3.8, 48 and 72 hours). There are several issues in the data that might be responsible for these artifacts. First, the amplitude of the signal in soil is very low and sometimes not recognizable. Thus many data were discarded from below the soilcrete columns, thinning the ray coverage in this area significantly. Second, the ray paths pictured in Figure 3.4 do not consider refraction of rays on the boundary between soil and soilcrete. The refraction leads to further thinning of ray coverage in the soil as travel paths tend to deviate through the faster soilcrete. Some of the remaining data might contain small errors which would cause no harm in zones with high ray coverage but have a significant influence on the tomography results in our case. A detailed investigation of ray coverage and model resolution will be conducted in the future to better address these issues.

Figure 3.7: Verification of jet grout size/geometry prediction (Specimen 3) with excavated specimen and tomogram from data acquired just before excavation.
Figure 3.8: $V_p$ tomograms for Specimen 1 at increasing curing times.

Figure 3.9: $V_p$ tomograms for Specimen 2 at curing times 20 to 28 hours.
Figure 3.10: $V_p$ tomograms for Specimen 2 at curing times 48 to 120 hours.

Figure 3.11: $V_p$ tomograms for Specimen 3 at increasing curing times.
3.5 Conclusions

A laboratory study was conducted to explore the spatial evolution of $V_p$ in simulated soilcrete columns. The results of this study demonstrate the use of CSL and 2D acoustic tomography for characterizing soilcrete geometry and quality for the 2D cross section of the column being evaluated. This method allows for the identification of high and low quality soilcrete, and characterizes the increase in soilcrete $V_p$ that results from curing. For high quality soilcrete, $V_p$ ranges from 1200-1400 m/s at early curing times (20-24 hours) and 3000-3200 m/s at late curing times (96-120 hours). For weak soilcrete, $V_p$ ranges from 900-1500 m/s after approximately 24 hours of curing, but may not undergo any significant $V_p$ gain at later curing times. Furthermore, 2D acoustic tomography can identify soil inclusions in the soilcrete column and characterize the geometry (height and diameter) of the soilcrete column, so long as the soilcrete is in contact with the CSL casings.

While the CSL system may not be applicable to a field scale setup (i.e., the frequency is too high and the source energy is too low to characterize a full size column), it provides a high resolution laboratory proof of concept supporting the use of acoustic tomography (and acoustic methods in general) for soilcrete column QC/QA. A more desirable field setup would allow measurement of jet grout quality even if no contact exists between the column and measurement array casing (i.e., an approach that could measure through in-situ soil and soilcrete). These results will help to inform the development of a coupled electric/acoustic field-scale test system capable of estimating both jet grout column geometry and soilcrete quality regardless of soilcrete/casing contact. Further extensions of this research include finite element modeling and joint inversion of both data sets to understand the simultaneous acoustic/electric behavior of soilcrete in the interest of improved predictive capability. Such a QC/QA system would offer higher quality results in an expedited time frame compared to current industry practice.

3.6 References


CHAPTER 4:
ELECTRICAL RESISTIVITY IMAGING OF LABORATORY SOILCRETE COLUMN GEOMETRY

Modified from a paper accepted for publication in the *ASCE Journal of Geotechnical and Geoenvironmental Engineering* (with permission from ASCE)

R. G. Bearce, M. A. Mooney, and P. Kessouri

4.1 Abstract

Ground improvement via jet grouting is commonly used to strengthen weak ground and/or create hydraulic barriers. Delivering soilcrete columns with tightly controlled and known diameters is critical to performance; however, techniques to assess jet grout geometry during construction are lacking. This paper reports the results of a study on electrical resistivity imaging of soilcrete by investigating the effects of electrode configuration and electrical protocol type on laboratory scale soilcrete columns constructed in a tank filled with sand. Experimental results are verified via numerical modeling and the model is used to analyze the changes in soilcrete resistivity that result from geometric variation. The results of this study indicate that resistivity imaging with direct contact electrodes can estimate the diameter of laboratory scale jet grout columns to within ±5% of the as built column diameter. A relationship between electrode spacing and column diameter is identified/quantified to more readily extend the diameter estimation approach developed herein to field scale geometries. Additionally, time lapse monitoring of soilcrete resistivity was performed over the course of curing. Results indicate that resistivity imaging should be performed as early as possible to obtain the greatest resistivity contrast between the soilcrete and in-situ soil.

4.2 Introduction

Jet grouting is an in-situ ground improvement technique used to strengthen weak/unstable ground and/or create hydraulic barriers via columns of soilcrete (i.e., a mixture of grout and in-situ soil). This process is illustrated in Figure 4.1a. Successful performance of jet grout columns and column assemblies requires constructing precise column geometries. However, the realized
diameter of jet grout columns is influenced by in-situ soil properties and stress state (Essler and Yoshida 2004). To this end, on-site inspection of geometry, preferably in real time, is critically important to jet grout construction.

![Image: Illustration of the jet grouting method, and the jet grouting method applied to foundation underpinning.]

Figure 4.1: a) Illustration of the jet grouting method, and b) the jet grouting method applied to foundation underpinning.

Jet grout column geometry is often assessed in practice by radial coring/probing or column excavation (e.g., Duzceer and Gokalp 2004, Yoshida 2010, Burke 2012, Bruce 2012, Wang et al. 2012, etc.). However, these approaches require waiting several days for sufficient soilcrete curing and are often unfeasible to perform below the water table. Further, the destructive nature of these approaches limits them to use on test columns; these techniques cannot be used on production columns.

A number of nondestructive approaches have been proposed in the past to measure jet grout column geometry, including mechanical downhole devices (Passlick and Doerendahl 2006) and temperature monitoring (Meinhard 2002, Mullins 2010, Sellountou and Rausche 2013). Thermal imaging has been successfully used to assess diaphragm walls and diaphragm wall joints (Doornenbal et al. 2011, Spruit et al. 2011). Geophysical approaches have also been proposed. Mechanical wave propagation techniques including downhole/surface seismic
(Madhyannapu et al. 2010) and crosshole sonic logging (CSL) (Niederleithinger et al. 2010, Bearce et al. 2014, Spruit et al. 2014) can characterize the changes in concrete/soilcrete strength (via increased wave speed), but cannot estimate geometry because the monitoring tubes are within the grouted structure. Furthermore, these methods require permanent casings and sufficient soilcrete curing time for ultrasonic/seismic wave propagation (2+ days). Borehole ground penetrating radar has also been proposed but requires a cased borehole directly outside the column (T&A Survey 2013).

Direct current (DC) electrical resistivity has been used to characterize soil improvement techniques such as injection grouting, compaction grouting, and hydraulic barrier walls (e.g., Daily and Ramirez 2000, Abu-Zeid et al. 2006, Abu-Zeid et al. 2009, Santarato et al. 2011, etc.). While these improvement techniques are not identical to jet grouting, the application of the geophysical technique is quite similar (i.e., DC resistivity exploits the resistivity contrast between improved/unimproved soil). The electric cylinder method (ECM) is a commercially-available DC resistivity technique used to estimate the geometry of a jet grouted column (Frappin and Morey 2001, Frappin 2011). The ECM employs a central borehole with a slotted casing in the center of the column (either pushed into the fresh column or drilled in after 1-2 days of curing). After casing placement, a chain of electrodes is lowered into the water-filled casing to allow electrical coupling between the jet grout and the electrodes (i.e., the electrodes are coupled to the water, which is coupled to the jet grout through the slots in the casing). This approach uses a type of pole-pole electrode array configuration that requires reference electrodes on the ground surface. Frappin and Morey (2001) conclude that the ECM can estimate to within 10% of the column diameter. However, in regions where geometry changes are the result of changing soil conditions, there is an additional 0.5m error. This can result in considerable uncertainty.

This paper presents the results of a study to advance DC resistivity imaging of jet grout column geometry. The study focused on examining the influence of direct coupling of electrodes to the jet grout column (vs. slotted casing where coupling is indirect through water) as well as utilizing the traditionally surface-based Wenner-α measurement protocol into a central borehole-based approach. Laboratory-scaled soilcrete column experiments were conducted to carry out the investigation. Finite element modeling was performed to support the experimental results and to assess accuracy.
4.3 Background

DC resistivity testing is an electrical geophysical technique based on Ohm’s law that has been widely used in geophysical exploration for decades and is becoming more prominent in civil engineering quality assurance and quality control (QA/QC) applications. In practice, DC resistivity measurements are usually performed using commutated direct current (i.e., a square-wave alternating current) or low frequency alternating current (AC) to assess the real component of the material’s resistivity and avoid material polarization resulting from sustained DC injection. The commutated direct current approach is utilized by the ABEM Terrameter LS used in this research. The DC resistivity technique characterizes a material’s electrical resistivity, or ability to resist current flow. The principle behind the DC resistivity technique is macroscopically governed by Ohm’s law (Eq. 4.1),

\[ j = \frac{E}{\rho} \] (4.1)

where \( j \) is the conduction current density (A/m\(^2\)), \( E \) is the electrical field in V/m, and \( \rho \) is the material’s electrical resistivity (Ωm). The electric field is defined as the gradient of the electrical potential (Eq. 4.2),

\[ E = -\nabla \psi \] (4.2)

where \( \psi \) is the electrical potential (V). In practice, current is injected across a pair of electrodes (A and B, Figure 4.2) to create an electric field in the subsurface. Figure 4.2 illustrates a borehole configuration; however, the more classical/common approach is on the ground surface (i.e., rotate image 90° clockwise). The electric field is characterized by measuring the potential difference across two or more measurement electrodes of known separation distance (M and N, Figure 4.2). Using the injected current and the measured potential, the material’s resistance \( R \) (Ω) is calculated. To obtain the apparent resistivity \( \rho_a \) (Ωm) from resistance, a geometric correction factor must be applied,

\[ \rho_a = \left( \frac{\psi_{MN}}{i_{AB}} \right) \cdot k = R \cdot k \] (4.3)
where $\psi_{MN}$ is the potential difference measured across electrodes M and N (V), $i_{AB}$ is the current injected across electrodes A and B (A), and $k$ is the geometric correction factor (m). Note that $\rho_a$ (obtained from the DC resistivity test) is not the same as a material’s true resistivity $\rho$. $\rho_a$ is a weighted average of all $\rho$ in the volume of material influenced by the injected electrical field. For a homogeneous medium, $\rho = \rho_a$, but in heterogeneous media, $\rho_a$ is influenced by the different values of $\rho$. In practice, $\rho$ is often obtained using inversion of $\rho_a$ data (Revil et al. 2012). The geometric factor $k$ is related to the electrode spacing $a$ for a Wenner-$\alpha$ array with point electrodes on the surface of an infinite homogeneous halfspace,

$$k = 2 \cdot \pi \cdot a \quad (4.4)$$

where $a$ is the distance between any two adjacent array electrodes ($a = AM = MN = NB$, Figure 4.2). For a full space condition (where in practice electrodes are deep enough in the ground that no surface boundary effects exist),

$$k = 4 \cdot \pi \cdot a \quad (4.5)$$

Borehole resistivity measurements have a variable $k$ factor in the near surface region that transitions from a half space to full space condition (Revil et al 2012, Guo et al. 2014). The laboratory setup used in this study mimics a field soilcrete column and will be subject to a variable $k$ factor near the surface. Other geometric complexities of the laboratory setup are not as easily addressed with analytical solutions (e.g., ring electrodes, soil tank boundaries, etc.). Therefore, finite element (FE) modeling (via COMSOL Multiphysics®) is used to obtain a more accurate estimate of $k$. The application of Equation 4.3 for a ring electrode array in a homogeneous medium constrained to the laboratory soil tank geometry is illustrated in Figure 4.2. The figure depicts a 2D radial cross section of the Wenner-$\alpha$ measurement protocol at increasing values of $a$. Current flow lines near the tank boundary and ground surface are influenced by the finite volume of the laboratory soil tank. The Wenner-$\alpha$ measurement protocol injects current ($i_{AB}$) across electrodes A and B and measures the potential ($\psi_{MN}$) across electrodes M and N. In the case of the borehole Wenner-$\alpha$ array, each measurement corresponds
to $\rho_a$ at a depth $z_{MN}$ (i.e., the midpoint of electrodes M and N). In practice, $a$ is increased by using every other electrode, every third electrode, etc. For the Wenner-$\alpha$ protocol, increasing $a$ results in increased volume of measurement influence with decreased measurement sensitivity (i.e., the measurement is a weighted average of the resistivity in a larger region, the boundary of which is further from the array).

Figure 4.2: 2D axisymmetric cross section of current flow lines for the direct coupled ring electrode array at increasing electrode spacing. Estimation of $\rho_a$ using Equation 4.3 is shown for each $a$ based on the potential across electrodes M and N labeled in each plot. Note that these simulations show the current/equipotential lines in the laboratory tank, and thus boundary effects are noticeable for lines near the tank boundary.

### 4.4 Experimental Setup and Testing Protocol

To perform the investigation, scaled soilcrete cylinders were constructed within a cylindrical soil tank of 1m diameter and 2m height (Figure 4.3a). A poorly-graded masonry sand was used as the medium surrounding the soilcrete columns. The SP material has a $D_{50} = 0.81\text{mm}$ and $C_u = 1.5$. The resistivity of the sand was verified by conducting a background resistivity profile (i.e., a Wenner- $\alpha$ profile from a soil tank full of sand prepared with the same technique as the soilcrete/sand system). Increased sand density can result in higher resistivity, but this effect
was not observed in the laboratory data (where the soil column height was only 1.3m, providing insufficient overburden pressure to meaningfully alter the density and thus resistivity profile with depth). The resistivity of the sand $\rho_s$ (Ωm) under saturated conditions had an average value of 20 ± 3.5Ωm. Grout was prepared by mixing tap water and Portland Type I-II cement with a 2:1 water:cement ratio. This grout mixture was combined with dry sand (SP) to form soilcrete with an approximate cement content of 8-9% by volume. In practice, field jet grout columns can have widely variable cement contents, but this value is reasonable for field jet grout columns in sand.

Figure 4.3: a) Laboratory soil tank with 2 stages (2m height, 1m inside diameter), b) staged construction of 30cm diameter soilcrete cylinder prior to addition of sand and soilcrete and c) soilcrete cylinder after extraction of form tube. A cross-sectional illustration of the sequential soilcrete/sand placement process is shown in d)-f).
To prepare soilcrete specimens, a PVC casing (outside diameter = 7.6cm) was placed in the center of the soil tank. Dry sand was deposited uniformly to a height of 20-30 cm by air pluviation from a consistent drop height (25-30 cm). The sand was not mechanically compacted, but this was not important for the study. The sand was saturated by continuously raising and maintaining the water table 1-2 cm below the surface of the placed sand throughout sample preparation. A form tube was slipped over the PVC casing and 10cm of sand was filled around the outside of the form tube to fix its position and prevent soilcrete from leaking out the bottom of the tube (Figure 4.3b and d). Pre-mixed soilcrete was poured into the form tube to achieve the desired column section height (Figure 4.3d). Sand was deposited around the soilcrete-filled form tube until the height of soilcrete and confining sand were equal (Figure 4.3e). Then, the form tube was extracted, allowing the sand to confine the fresh/wet soilcrete and form a soilcrete cylinder (Figure 4.3c and f). This staged construction process was used to produce the post-test exhumed soilcrete columns shown in Figure 4.5.

This study employed two types of central casing/electrode array combinations. The first array type consists of PVC casing with externally mounted ring electrodes in direct contact with the measured media (Figure 4.4a). The second array mimics the slotted casing approach used by the ECM (Frappin and Morey 2001) and is illustrated in Figure 4.4b. The slotted casing used in the laboratory was wrapped in a fluid-permeable geotextile to prevent soilcrete/soil infiltration. Both images show the full length array with an expanded detail window of array photographs and 2D axisymmetric cross sections of the cylindrical setup. The inside of both array tubes are hollow/water tight and contain instrumentation wiring.

Two soilcrete specimens were prepared and tested. Specimen 1 (Figure 4.5a) was tested with a direct coupled ring electrode array and specimen 2 (Figure 4.5b) was tested with a slotted casing and internal ring array. The slots are not visible on the exhumed specimen 2 casing because of the geotextile. Because the column construction process resulted in minor geometric variations, the dimensions shown in Figure 4.5 are average values obtained from multiple measurement profiles of the exhumed columns. In regions where the as-built diameter \( D = 30 \)cm, the average diameter of the exhumed specimen was equal to 29.8±1.7cm (i.e., ± 1\( \sigma \)). For the reduced diameter regions (\( D = 18 \)cm), average exhumed specimen diameter was equal to 17.8±1.2cm.
Figure 4.4: Illustration of full length electrical array with expanded cross sectional diagram and photographs for a) direct coupled ring electrodes and b) ring electrodes in a water-filled slotted casing. Note that the photographs and FE renderings in b) do not show the geotextile to better appreciate the geometry of the slotted casing. The geotextile location is shown in the expanded illustration of and also visible in Figure 4.3b/c and Figure 5b.

4.5 Finite Element Modeling

A 3D scale model of the laboratory soil tank and test arrays were constructed in COMSOL Multiphysics®. This software package is extensively used as a forward modeling tool for the DC resistivity test (Kumar et al. 2008, Chou et al. 2010, Clement et al. 2011) and DC resistivity applied to geotechnical/geological problems (Kim et al. 2009, Huang and Lin 2010, Wang et al. 2011, Araji et al. 2012, etc.). The laboratory tank with ring electrodes contains complex geometries and finite boundaries. The FE model was constructed using free tetrahedral elements with uniform directional scaling. High resolution regions (e.g., ring electrodes) had a minimum element dimension of 1mm, and regions of near zero current (e.g., soil near the tank boundary) had a maximum element size of 7cm. The FE model is governed by the electric currents physics interface, which solves a current conservation equation (based on Ohm’s law) using the scaler electric potential as the dependent variable (COMSOL 2014). The tank boundaries were defined using the electric insulation boundary condition, which allows no current to flow into the boundary. A series of stationary electrical models is used to simulate the
DC resistivity test. The modeling mimics the experimental protocol by sequentially injecting/measuring each electrode combination in the laboratory Wenner-α protocol. To simulate an individual DC resistivity measurement, a volumetric current source (A/m³) is applied to ring electrode A (of known volume) and sinked to an identical ring electrode B such that the desired $i_{AB}$ is injected (e.g., Figure 4.2). Volumetric potentials (V/m³) are obtained from ring electrodes M and N and converted to $\psi_{MN}$ (V) via the known ring volume. Equation 3 is used to estimate $\rho_a$ with FE $i_{AB}$, $\psi_{MN}$, and $k$.

Figure 4.5: Diagram of soilcrete specimen/array/soil geometry with exhumed specimen for comparison for a) specimen 1 and b) specimen 2. Note that the height of the soil around the column corresponds to the height of the soil in the tank during the test and not the full height of the tank.

Geometric factors for the arrays and protocols used in this research were estimated by simulating the DC resistivity test in a homogeneous 10Ωm material (confined to the geometry of the lab tank) for each value of $a$ (e.g., Figure 4.2). The geometric correction for this data is affected by more aspects of the system geometry than $a$ alone. The laboratory scale setup is limited in size, and will have boundary effects compared to the half/full space conditions in the field. In addition, the ring electrode diameter (8cm) is relatively large compared to the minimum $a$ for the array (3cm). The combination of these geometric complexities requires the use of a
custom $k$ factor determined from FE modeling of the DC resistivity test. This technique is widely used/accepted in literature (e.g., Rücker et al. 2006, Kumar et al. 2008, Yi et al. 2009, Guo et al. 2014, etc.). To mimic the laboratory array setup, $k$ was estimated at 3cm intervals over the entire height of the array in a homogeneous 10Ωm material for each value of $a$.

To assess the validity of the FE model, a simplified version of the laboratory setup was modeled with point electrodes (to eliminate ring electrode geometric complexity) in a 10m diameter by 10m height homogeneous cylinder (to eliminate boundary effects). The FE model results show excellent agreement with Equation 4.3. Surface electrode simulations have $k$ values consistent with Equation 4.4 and measurements at midpoint depth (i.e., full space) have $k$ values consistent with Equation 4.5. Near surface measurements have $k$ values that transition from half to full space conditions. These validation tests were performed for $a = 3, 6, 9, \text{and } 12\text{cm}$.

4.6 Results

Apparent resistivity results for the two soilcrete cylinders are shown in Figure 4.6 (direct coupled electrodes) and Figure 4.7 (slotted casing) for data acquired after 1.5 hours of curing using the Wenner-$\alpha$ protocol ($a = 3, 6, 9 \text{ and } 12\text{cm}$). Scaled images of the exhumed specimens and corresponding FE models are shown for reference. All data are presented in terms of $\rho_a$ (Equation 4.3) obtained from experimental $\psi_{MN}$ and $i_{AB}$ with FE $k$ factor.

The $a = 3\text{cm}$ data for the direct coupled electrodes (Figure 4.6) reveal $\rho_a$ values of approximately 1.6Ωm in the regions of the soilcrete column where the as-built/modeled column diameter $D$ is equal to 30cm ($z = 40-62, 82-110\text{ cm}$). These $\rho_a$ values are very similar to the soilcrete’s true resistivity ($\rho_{sc}$) of 1.5-2.0Ωm as determined from benchtop tests on soilcrete of the same mix/age. This result suggests that $a = 3\text{cm}$ $\rho_a$ measurements are sensitive only to $\rho_{sc}$ and are not influenced by the surrounding untreated sand resistivity ($\rho_s$). A very slight increase in $a = 3\text{cm}$ $\rho_a$ is observed in the $D = 18\text{cm}$ region ($z = 62-82\text{cm}$), suggesting that $\rho_a$ is influenced by both $\rho_{sc}$ and $\rho_s$ in this region (recall that $\rho_s$ is 20Ωm). The increase in $\rho_a$ in the reduced diameter region ($z = 62-82\text{cm}$) is manifested more prominently in the data as $a$ increases. This response is expected as an increase in $a$ will increase the depth of measurement. These trends support a relationship between $a$ and $D$. As will be confirmed later in the paper, for cases in which $D \geq 10a$, $\rho_a$ measurements can be used as an indicator of $\rho_{sc}$ with depth.
Figure 4.6: Comparison of experimental and FE apparent resistivity responses for specimen 1 after 1.5 hours of curing. Plots and column images are scaled such that horizontal dashed lines can be used to relate geometry changes in the column to changes in the data response. z position of the data points represents the center of each four electrode array configuration.

The $a = 3$cm data from the slotted casing electrodes (Figure 4.7) reveal $\rho_a$ values of 17$\Omega$m, which unlike the direct coupled approach, are not indicative of $\rho_{sc}$ (1.5-2$\Omega$m). The water in the casing ($\rho_w = 33\Omega$m) results in $\rho_a$ measurements that are influenced by $\rho_w$, $\rho_{sc}$, and $\rho_s$. Further, slotted casing data exhibits a more gradual change in response when transitioning from sand to soilcrete ($z = 40$ cm and $z = 95$ cm). The water column forms a homogeneous buffer zone between the electrodes and the soilcrete or sand. Compared to the sharp changes in response of the direct coupled electrodes (Figure 4.6), material transitions and changes in $D$ are difficult to detect in the slotted casing data.

A comparison of the two data sets suggests that the direct coupled ring electrodes provide a considerable measurement advantage over the slotted casing approach for a Wenner-$\alpha$ array. The direct coupled electrodes provide a high resolution estimate of $\rho_{sc}$ from the $a = 3$cm measurement and reveal sharper contrasts when transitioning between materials and variations in $D$. The slotted casing approach cannot directly estimate $\rho_{sc}$ due to the influence of borehole water, and exhibits less sensitivity to changes in $D$ than the direct coupled approach.
Figure 4.7: Comparison of experimental and FE resistivity responses for specimen 2 after 1.5 hours of curing. Plots and column images are scaled such that horizontal dashed lines can be used to relate geometry changes in the column to changes in the data response. Data sets are clipped at the bottom due to electrode failure in the bottom two electrodes.

FE models of both specimens and array configurations were constructed to further inform the experimental results. Each soilcrete column was modeled with the geometric specifications shown in Figure 4.5. FE model inputs for $\rho_{sc}$, $\rho_s$, and $\rho_w$ were obtained from benchtop resistivity tests and/or $a = 3\text{cm}$ direct coupled Wenner-$\alpha$ data (Table 4.1). FE modeling of the direct coupled array (Figure 4.6) shows excellent agreement with the experimental data. There is minor disagreement in the regions above and below the soilcrete column, but results are very close over the soilcrete column depth interval for all values of $a$. FE modeling of the slotted casing array does not exhibit the same quality of fit as the direct coupled electrodes. The $a = 3\text{cm}$ result shows reasonable agreement, but there is more scatter in the experimental data. FE results for $a = 6$ and $12\text{cm}$ are reasonably well fit over the soilcrete column interval, but $a = 9\text{cm}$ FE response deviates from experimental results at $z = 60$-100cm. The reason for this lack of fit is not entirely clear given that the other data sets show reasonable fit in this region; however, it is likely due to the increased modeling complexity of the slotted casing setup. No additional variation of
input parameters can be justified to improve these fits (e.g., no reason to assume variable $\rho_w$ or $\rho_s$ with depth).

Table 4.1: Experimentally measured resistivity values of the sand ($\rho_s$), soilcrete ($\rho_{sc}$), and water ($\rho_w$) used in the laboratory experiments. FE resistivity inputs for each model are also reported.

<table>
<thead>
<tr>
<th></th>
<th>Experimental</th>
<th>FE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Benchtop</td>
<td>Array</td>
</tr>
<tr>
<td>$\rho_s$ ($\Omega m$)</td>
<td>18-26</td>
<td>20±3.5</td>
</tr>
<tr>
<td>$\rho_{sc}$ ($\Omega m$)</td>
<td>1.5-2.0</td>
<td>1.6±0.1</td>
</tr>
<tr>
<td>$\rho_w$ ($\Omega m$)</td>
<td>33</td>
<td>n/a</td>
</tr>
</tbody>
</table>

To further understand the measurement capability of the two electrical arrays, FE modeling was used to estimate the percent of current $i_{AB}$ propagating into a homogeneous medium ($10\Omega m$). Axisymmetric cross sections for $a = 3cm$ are shown for the direct coupled array (Figure 4.8a) and the slotted casing array (Figure 4.8b). Current flow lines for each model are separated into eight magnitude-based regions. The area between any adjacent pair of contour lines contains 12.5% of the total current in the system (i.e., areas with more closely spaced contour lines have higher current density). The images are zoomed to better illustrate the detail of the more closely spaced current lines near the array, and thus not all eight regions are visible. For the DC resistivity test, a higher current density is indicative of higher measurement sensitivity.

The coupling between the ring electrodes and soil allows the injected current to propagate directly into the soil (in this simulation, $\rho_s = 10\Omega m$). On the contrary, the slotted casing approach (Figure 4.8b) loses a significant amount of measurement sensitivity because approximately 40% of the current does not propagate beyond the borehole water column. Flow lines in the water column are more closely spaced than in the soil, suggesting that the Wenner-$\alpha$ protocol with a slotted casing array has the highest sensitivity to $\rho_w$. The direct coupled approach also has the largest current density near the array (Figure 4.8a), which results in the highest sensitivity to the soil/soilcrete. This observation explains why the experimental $a = 3cm \rho_a$ (Figure 4.6) is sensitive to the $\rho_{sc}$ only.
Figure 4.8: 2D axisymmetric cross section of current flow lines of electrical array and homogeneous soil tank configuration for a) a direct coupled ring electrode array and b) a ring electrode array within a water-filled slotted casing at a = 3cm.

The protocol used by the ECM would not suffer from this issue as significantly (i.e., electrode B is on the ground surface so current naturally flows out of the borehole). The geometric constraints of the laboratory prevented the evaluation of pole-pole arrays in this research; however, it is reasonable that the pole-pole protocol could also see improved results using direct coupled electrodes instead of a slotted casing.

To better characterize the relationship between \( a \) and \( D \), additional FE models of specimen 1 were constructed with diameter profiles of 1.1\( D \) and 0.9\( D \) (i.e., \( \pm 10\% \) of the as-built diameter \( D \)). FE \( \rho_a \) responses for columns with diameter profiles of 0.9\( D \), \( D \), and 1.1\( D \) are defined as \( \rho_a(0.9D) \), \( \rho_a(D) \), and \( \rho_a(1.1D) \), respectively (Figure 4.9). For each response, \( \rho_a(1.1D) < \rho_a(D) < \rho_a(0.9D) \) because increasing \( D \) results in the measurement imaging more of the lower resistivity soilcrete.

As shown in Figure 4.9a, a 10\% variation in the \( D = 30\text{cm} \) diameter regions (\( z = 42-62\text{cm}, z = 82-105\text{cm} \)) has relatively little or no influence on the response (i.e., \( \rho_a(0.9D) \approx \rho_a(D) \))
$\approx \rho_a(1.1D)$. $D/a$ varies from 9-11 in this region, and $\rho_a$ is influenced by $\rho_{sc}$ only. For the $a = 6$cm measurements, however, $\rho_a(1.1D) < \rho_a(D) < \rho_a(0.9D)$, indicating that the 10% variation in the $D = 30$cm regions does influence $\rho_a$ (Figure 4.9b). Here, $D/a$ varies from 4.5-5.5 and $\rho_a$ is influenced by both $\rho_{sc}$ and $\rho_s$. As $a$ increases, the deviation between $\rho_a(0.9D)$, $\rho_a(D)$, and $\rho_a(1.1D)$ increases (Figure 4.9c, d). The ± 10% variation in the $D = 18$cm diameter region has a more significant influence on $\rho_a$ modeled at all values of $a$.

![Figure 4.9: Comparison of FE resistivity responses for columns with diameter profiles of 1.1D, D, and 0.9D to experimental resistivity response for a) $a = 3$cm, b) $a = 6$cm, c) $a = 9$cm, d) $a = 12$cm. e) Experimentally predicted column diameter $\overline{D}$ from each value of $a$.](image)

The sensitivity of $\rho_a$ to $D$ (for $D/a < 5$) is exploited to estimate column diameter. For example, in Figure 4.10a ($a = 12$cm), the experimental $\rho_a$ value at $z = 51$cm deviates from the expected response $\rho_a(D)$. Because the value falls between the $\rho_a(D)$ and $\rho_a(0.9D)$ responses, the experimentally measured $\rho_a$ suggests that the column diameter at this depth lies between $D$ and $0.9D$. To estimate the diameter that best fits the experimental data, defined here as $\overline{D}$, a linear correlation is assumed using points ($\rho_a(1.1D)$, 1.1D), ($\rho_a(D)$, D), and ($\rho_a(0.9D)$, 0.9D) (Figure 4.10b, c). Over the range of $D$ evaluated (± 10% $D$), the correlation between $D$ and $\rho_a$ is sufficiently linear ($R^2 \geq 0.98$ for all combinations of $z$ and $a$). This correlation and corresponding
$D$ are shown at $z = 51\text{cm}$ (where $D = 30\text{cm}$) and $z = 71\text{cm}$ (where $D = 18\text{cm}$) in Figures 4.10b and c, respectively. No single correlation equation is reported because this relationship differs for every value of $a$ and $z$, and also depends on $D/a$. This correlation approach is applied to experimental $\rho_a$ to provide an estimate of column diameter $\bar{D}$ (Figure 4.9e).

![Diagram showing correlation between $D$ and $\rho_a$](image)

Figure 4.10: a) FE diameter study compared to experimental data for $a = 120\text{cm}$, and linear correlation between $D$ and $\rho_a$ for b) $z = 51\text{cm}$, and c) $z = 72\text{cm}$.

The estimated $\bar{D}$ are in good agreement with the as-built $D$. For the $D = 30\text{cm}$ regions, the average $\bar{D}$ was equal to $30.2\pm1.5\text{cm (5\% D)}$. In the $D = 18\text{cm}$ region, average $\bar{D} = 17.9\pm0.6\text{cm (3\% D)}$. Because the measurement is taking an axisymmetric average of the volume around the ring electrode array, it is important to consider the diameter variation in the exhumed specimens to address the accuracy with which $\bar{D}$ can be estimated. The average as-built $D$ of exhumed specimen 1 was equal to $29.8\pm1.7\text{cm}$ for the larger regions, which is very similar to the average $\bar{D} = 30.2\pm1.5\text{cm}$ estimated via resistivity imaging. In the $D = 18\text{cm}$ region, average $D$ of the exhumed specimen was equal to $17.8\pm1.2\text{cm}$ and $\bar{D} = 17.9\pm0.6\text{cm}$.

$\bar{D}$ is also well fit in transitional regions corresponding to column diameter reduction ($z = 62$ and $82\text{cm}$). Uniform diameter regions and diameter reduction transition regions show similar
misfit between experimental $\rho_a$ and the $\rho_a(D)$ response, suggesting that column diameter reduction does not reduce the accuracy of the measurement. For soil to soilcrete transitions (i.e., the top and bottom of the column), experimental $\rho_a$ values occasionally fall outside the $\pm10\% D$ envelopes (e.g., $z = 100$-105cm, Figure 4.9a-d). The sharp transition between soil and soilcrete at the array interface results in a $\rho_a$ response that is significantly less sensitive to diameter variation. This observation helps to explain why $\bar{D}$ near the top and bottom of the column show the greatest misfit from $D$. In general, measurements where $D/a < 5$ provide sufficient change in resistivity response for use as estimators of diameter. The lower the value of $D/a$, the more sensitive the measurement will be to changes in $D$. This relationship is also affected by the resistivity contrast between the soilcrete and the soil.

Electrical resistivity profiling exploits the contrast in resistivity between the surrounding soil and the soilcrete. In this study, $\rho_s/\rho_{sc} \approx 12$ and Frappin and Morey 2001 recommend $\rho_s/\rho_{sc} > 10$. The highest $\rho_s/\rho_{sc}$ ratio occurs within the first two hours of mixing (depending on cement type and water to cement ratio of the grout). The relatively low resistivity of fresh soilcrete stems from the conductive ionic electrolyte solution making up the pore fluid in the concrete/soilcrete (Rajabipour et al. 2007). The resistivity of the pore fluid depends on the cement composition and ionic concentration in the pore fluid (Backe et al. 2001, Chung 2004). Ions in the pore fluid (e.g., Na$^+$, K$^+$, Ca$^{2+}$, OH$^-$, SO$^{4-}$) are associated with the formation of cementitious compounds that bond soil grains together (e.g., calcium-silicate-hydrates) (Taylor 1997). This cementing process reduces ionic concentration and causes the concrete/soilcrete to become more resistive with curing time. Additionally, bonds formed between soil grains and cementing compounds reduce porosity, which further increases resistivity (Rajabipour et al. 2007).

To assess the change in $\rho_{sc}$ that results from curing, average values of $\rho_{sc}$ were determined using direct coupled $a = 3$cm specimen 1 data from column sections where $D/a \geq 10$. Average $\rho_{sc}$ values with $1\sigma$ error bars are displayed for curing times ranging from 1.5-240 hours in Figure 4.11. As expected, $\rho_{sc}$ increases with curing time. The growth is significant in the first 10 hours of curing, and continues to grow at a slower rate thereafter. Growth continues for the duration of monitored curing and would likely continue to grow past 10 days. Least squares regression fitting indicated that resistivity growth follows a logarithmic behavior with curing time.
Figure 4.11: Experimental $\rho_{sc}$ with 1σ error bars from curing times of 1.5 hours to 240 hours. Specific curing times are highlighted and a best fit regression analysis function is shown.

4.7 Conclusions

Borehole DC resistivity tests were performed on laboratory scale soilcrete columns to assess the usability of the Wenner-$\alpha$ protocol with direct coupling of electrodes to improve column geometry estimation. The protocol was tested on two array configurations: electrodes in direct contact with the soilcrete and electrodes in a slotted, water-filled casing encased in soilcrete. FE models of the soil tank and specimens/arrays were constructed and compared with experimental data to assess the capability of the model as a tool for column geometry prediction.

The study revealed that the Wenner-$\alpha$ array with direct coupled electrodes provides several advantages over the slotted casing configuration. With direct coupled electrodes, the current is injected directly into the soilcrete, resulting in a higher current density in the soilcrete compared to the slotted casing approach. In the slotted casing configuration, considerable current remains in the column fluid, e.g., 40% per FE analysis. To this end, the direct coupled electrodes provide significantly better geometric resolution than the slotted casing array using same Wenner-$\alpha$ protocol. This conclusion is evidenced in both experimental and FE results.

$\rho_a$ measurements from the direct coupled data are compared to FE $\rho_a$ predictions to estimate column diameter via a linear correlation between $D$ and $\rho_a$. For $D = 30$cm regions,
exhumed specimen 1 $D = 29.8 \pm 1.7\text{cm}$ and average $\overline{D} = 30.2 \pm 1.5\text{cm}$. In the $D = 18\text{cm}$ region, exhumed specimen 1 $D = 17.8\pm1.2\text{cm}$ and $\overline{D} = 17.9\pm0.6\text{cm}$. The uncertainties in $\overline{D}$ correspond to $\pm 5\% D$ in the $D = 30\text{cm}$ regions and $\pm 3\% D$ in the $D = 18\text{cm}$ region. There is no appreciable difference in $\overline{D}$ accuracy when transitioning between different column diameters; however, when making the more drastic transition from soil to soilcrete at the array interface (i.e., the top and bottom of the column), the method loses its sensitivity to diameter change.

An analysis of experimental results and FE modeling reveals an important relationship between electrode spacing $a$ and column diameter $D$ using the direct coupling configuration. If $D/a \geq 10$, the measured $\rho_a$ will be influenced by the soilcrete only. $\rho_a$ measurements where $D/a \leq 5$ are influenced by $\rho_{sc}$ and $\rho_s$. $D/a \leq 5$ $\rho_a$ is sensitive to the soilcrete and the soil, indicating that $D/a \leq 5$ measurements are best-suited for characterizing column geometry. In general, the lower the value of $D/a$, the more sensitive the measurement will be to changes in column geometry. This conclusion can be applied to field jet grout construction. Direct couple electrode configurations can be implemented with push-probe technology (effort underway). The normalized observations with $D/a$ observed in this study can be readily extended to field scale $D/a$ values (e.g., a 3m diameter column and an array with $a_{min} = 30\text{cm}$).

Time lapse Wenner-$\alpha$ data (using direct coupled electrodes) suggest that $\rho_{sc}$ values range from $1.6\Omega\text{m}$ (1.5 hours) to $8.5\Omega\text{m}$ (10 days), resulting in a significant reduction in the resistivity contrast between the soil and soilcrete as curing time increases. Furthermore, measurement uncertainty increases significantly with curing time ($\sigma = 0.1\Omega\text{m}$ after $t = 1.5\text{ hours}$, $\sigma = 1.6\Omega\text{m}$ after $t = 10\text{ days}$). While the exact temporal variation in soilcrete resistivity would be mix-dependent, this result indicates that soilcrete resistivity testing should be performed as early as possible to maximize the resistivity contrast between the soilcrete and in-situ soil and minimize the uncertainty in $\rho_{sc}$. This is well-suited for field conditions where testing immediately after jet grouting is ideal.

### 4.8 Acknowledgements:

Funding for this study was provided by the National Science Foundation under the Partnership for International Research and Education (PIRE) Program (OISE-1243539). The authors also wish to thank Dr. Ernst Niederleithinger of the BAM Federal Institute for Materials Testing and CSM student Justin Downs for their support and assistance in this research.
4.9 References


5.1 Abstract

Accurate assessment of soilcrete columns, e.g., columns produced via jet grouting, is necessary to ensure proper performance, but current assessment techniques are usually limited to evaluation of test columns. Issues such as variable soil conditions and imprecise repeatability of construction parameters can lead to differences in the geometry of test and production columns. An ideal soilcrete column geometry assessment approach would be capable of rapidly evaluating production columns with no lasting column defects. This study outlines the development and implementation of a direct current electrical resistivity push probe to estimate the diameter of soilcrete columns. Computational modeling is used to inform the design of the probe, interpret experimental data obtained from probe measurements on field scale soilcrete columns, and develop a data analysis routine to predict constructed column diameter. The results of this study indicate that the electrical probe can provide an in-situ estimate of soilcrete resistivity (used to inform computational modeling) and estimate soilcrete column diameter to within ±5% of the as-constructed diameter. Furthermore, the push probe is a recoverable/reusable tool that is truly non-destructive as it leaves no lasting defects in tested columns.

5.2 Introduction

Jet grouting is an in-situ ground improvement technique that creates soilcrete columns in the subsurface to strengthen unstable ground, underpin foundations, stabilize slopes, and/or create hydraulic barriers (via overlapping columns). Given the volatile nature of the jet grouting process, wherein high pressure fluid is used to erode in-situ soil and mix it with cement grout to create soilcrete, the realized diameter of jet grout columns is often variable. Specific factors
influencing column geometry include in-situ soil type, stress state, and groundwater conditions. Machine parameters such as grout pressure and drill string rotation speed also affect column geometry (Essler and Yoshida 2004, Yoshida 2010, Burke 2012). To this end, real time verification of production jet grout column geometry is necessary to ensure proper performance.

Column geometry assessment in industry is often destructive, and thus performed only on test columns. Common approaches include radial coring/probing or column excavation (Duzceer and Gokalp 2004, Yoshida 2010, Burke 2012, Bruce 2012, Wang et al. 2012, etc.). These approaches cannot be used to assess production columns as they are destructive in nature. Further, they often require 2+ days for adequate curing and are difficult/unfeasible to perform below the water table. Pseudo-nondestructive approaches have been used to estimate soilcrete column geometry; however, the approaches are not truly non-destructive as they often require permanent casings in or near the column. Such approaches include temperature monitoring (Meinhaerd 2002, Mullins 2010, Sellountou and Rausche 2013), downhole/surface seismic (Madhyannapu et al. 2010), crosshole ultrasonic/seismic (Niederleithinger et al. 2010, Bearce et al. 2014, Spruit et al. 2014, Mackens et al. 2015, Galindo-Guerreros et al. 2015a,b), borehole ground penetrating radar (T&A Survey 2013), and direct current (DC) electrical resistivity (Frappin and Morey 2001, Frappin 2011).

DC resistivity is the well-suited for soilcrete column diameter estimation (Frappin and Morey 2001, Frappin 2011, Bearce et al. 2015). As applied to soilcrete testing, electrical resistivity profiling exploits the contrast in electrical resistivity between the soilcrete and surrounding in-situ soil. Soil resistivity can vary drastically (10s-100s of Ωm) depending on soil type and groundwater presence/type. Fresh soilcrete has a relatively low resistivity, e.g., 1.5-3Ωm observed in this study, that is caused by the conductive ionic electrolyte solution making up the the pore fluid in the soilcrete (Rajabipour et al. 2007). Pore fluid resistivity depends on cement composition and ionic concentration (Backe et al. 2001, Chung 2004). Ions in the pore fluid (e.g., Na⁺, K⁺, Ca²⁺, OH⁻, SO₄²⁻) undergo reactions that form cementitious compounds and bond soil grains together, e.g., calcium-silicate-hydrates (Taylor 1997). This cementing process reduces ionic concentration (via consumption of free ions) and porosity (by isolating pore connectivity), which results in an increase in resistivity (Rajabipour et al. 2007). Literature indicates that resistivity imaging of soil/soilcrete boundaries should be performed as immediately
as possible, when the contrast between the fresh soilcrete and in-situ soil is the largest (Frappin and Morrey 2001, Bearce et al. 2015).

The electric cylinder method (a.k.a Cyljet) is a commercially available technique that uses DC resistivity to estimate soilcrete column diameter. Measurements are obtained from a central slotted casing either pushed into the fresh soilcrete column or drilled in after 1-2 days of curing. After casing placement, a chain of electrodes is lowered into the water-filled casing to allow electrical coupling between the soilcrete and the electrodes (i.e., the electrodes are coupled to the water, which is coupled to the soilcrete through the slots in the casing). This approach uses a type of pole-pole electrode array configuration that requires reference electrodes on the ground surface. Frappin and Morey (2001) conclude that the ECM can estimate column diameter to within 10% of the constructed diameter. However, in regions where geometry changes are the result of changing soil conditions, there is an additional 0.5m error. Furthermore, subsurface anomalies within 1m of the column can affect measurement accuracy. These limitations can result in considerable uncertainty in column diameter estimation.

In a recent laboratory-based study, Bearce et al. 2015 showed that electrodes directly coupled to the soilcrete combined with a Wenner-α electrode configuration can provide compelling advantages compared to a pole-pole electrode configuration in a slotted casing (such as the electric cylinder method). Namely, the use of electrodes directly coupled to the soilcrete (as opposed to indirect coupling via a fluid-filled slotted casing) provides an in-situ estimate of the soilcrete’s resistivity, which can vary with depth. Accurate prediction of column diameter requires accurately known soil and soilcrete resistivity. To this end, obtaining an in-situ estimate of soilcrete resistivity can increase the accuracy of diameter predictions. Furthermore, the borehole based Wenner-α approach does not require reference electrodes on the surface (an approach that is sometimes unfeasible in real construction environments).

This paper presents the results of a study to extend direct coupled DC resistivity imaging to field scale deep soil mix (DSM) columns. DSM columns are evaluated because the diameter is precisely known, and having ground truth data is key to investigating the strengths and limitations of this approach. A 6m long DC resistivity push probe was fabricated and used to image 1.8-2.4m diameter DSM columns on two active construction sites. Multiphysics computational modeling was used to both inform the probe design and to aid in the interpretation of the field measurements. The paper describes the principles behind the direct coupled approach
and electrical probe-soilcrete-soil interaction, describes the probe and modeling efforts that informed its development, presents the results from field testing, and characterizes the accuracy with which the direct couple probe approach can measure DSM column diameters.

5.3 Background:

Direct current (DC) resistivity is an electrical geophysical technique that characterizes a material’s electrical resistivity $\rho$, or ability to resist current flow. The principle behind the DC resistivity technique is macroscopically governed by Ohm’s law (Eq. 5.1),

$$ j = \frac{E}{\rho} \quad (5.1) $$

where $j$ is the conduction current density (A/m$^2$), $E$ is the electrical field in V/m, and $\rho$ is the material’s electrical resistivity (Ωm). The electric field is defined (Eq. 5.2) as the gradient of the electrical potential $\psi$ (V).

$$ E = -\nabla \psi \quad (5.2) $$

To obtain an individual measurement, current is injected across a pair of electrodes (A and B) to create an electric field in the subsurface that is sampled by measuring the potential difference across two or more measurement electrodes of known separation distance (M and N). Each measurement yields a value of resistance $R$ (Ω) that is converted to an apparent resistivity $\rho_a$ (Ωm) using a geometric correction factor $k$,

$$ \rho_a = \left( \frac{\psi_{MN}}{i_{AB}} \right) \cdot k = R \cdot k \quad (5.3) $$

where $\psi_{MN}$ is the potential difference (V) measured across electrodes M and N, $i_{AB}$ is the current (A) injected across electrodes A and B, and $k$ is the geometric correction factor (m). The $\rho_a$ obtained from DC resistivity measurements (i.e., Eq. 5.3) is not the same as a material’s constitutive resistivity $\rho$. $\rho_a$ is a weighted average of all $\rho$ in the volume of material influenced by the injected electrical field. For a homogeneous medium, $\rho = \rho_a$. In a heterogeneous media,
e.g., layered media, $\rho_a$ is affected by the $\rho$ values of all materials influenced by the injected electrical field. $\rho$ profiles in heterogeneous media are often obtained by inverting $\rho_a$ data from many DC resistivity measurements at various electrode spacings (Revil et al. 2012). For the Wenner-\(\alpha\) array with point electrodes on the surface of an infinite homogeneous halfspace,

$$k = 2 \cdot \pi \cdot a$$  \hfill (5.4)$$

where $a$ is the distance between any two adjacent array electrodes (m). When electrodes are sufficiently deep in the ground and no surface boundary effects are present (i.e., full space conditions),

$$k = 4 \cdot \pi \cdot a$$  \hfill (5.5)$$

For the borehole Wenner-\(\alpha\) array, each measurement corresponds to $\rho_a$ at a depth $z_{MN}$ (i.e., the midpoint of electrodes M and N). Borehole resistivity measurements in the near surface have a variable $k$ factor that is similar to half space condition near the surface when boundary effects are present. As measurements are obtained at greater depths, the surface boundary effects are reduced and the $k$ factor transitions to a full space condition (Revil et al. 2012, Guo et al. 2014). The depth over which this transitions occurs depends on the electrode spacing utilized.

While the name implies that direct current is used to perform the DC resistivity test, sustained DC current can cause material polarizations that will affect the accuracy of the test results. When a sustained direct current is applied to a polarizable material, the electrical field influences the distribution of charges in the medium by aligning molecules/ions according to the direction of the electrical field. This changes the electrical properties of the material and will affect the accuracy of the measurement. For this reason, DC resistivity measurements are often performed using commutated direct current (e.g., a square-wave alternating current) or low frequency alternating current (AC) to assess the real component of the material’s resistivity. The commutated direct current approach is used in this research.
5.4 Development of a Field-Scale Electrical Resistivity Push Probe

Building on the findings in Bearce et al. 2015, a direct coupled electrical resistivity push probe was developed for field scale testing. The probe, shown in Figures 5.1 and 5.2, is composed of four 1.5 m long sections constructed from PVC pipe with ring electrodes spaced every 0.3m (Figure 5.1a). The PVC pipe assembly is nested, with a full-length 1.6m inner pipe and 10cm outer diameter sleeves. AWJ drill rod is used to provide structural integrity to the sections (Figure 5.1b and c). The probe is connected to the AWJ rod via metal support washers at the end of each section. 10cm diameter by 0.8cm height ring electrodes (5 per 1.5m section) are placed at 0.3m spacing with 10cm diameter PVC sections between each electrode (Figure 5.1a). The alternating ring electrode and PVC spacer configuration overlays the full-length inner section of PVC. The space between the AWJ rod and the inner PVC pipe is hollow and contains instrumentation wiring. Prior to testing, the sections are assembled on-site to form the full 6m probe shown in Figure 5.2b. The probe was designed for a Wenner-α electrode protocol with equally spaced electrodes throughout; however, other protocols (e.g., Wenner-β/γ/Schlumberger) can be used. With surface references electrodes, pole-pole configurations could also be employed.

The push probe design and geometry was informed by the results of Bearce et al. (2015). The authors used a direct coupled ring electrode approach with a Wenner-α protocol to estimate soilcrete column geometry in a laboratory environment and validated these results with computational modeling. This research uses a similar computational modeling approach to inform the design of the probe, validate the experimental data obtained from the probe, and estimate the diameter of tested DSM columns. Modeling was conducted using the finite element (FE) software package COMSOL Multiphysics®. This software is well suited for modeling the DC resistivity test (Kumar et al. 2008, Chou et al. 2010, Clement et al. 2011) and DC resistivity data related to geotechnical/geological problems (Kim et al. 2009, Huang and Lin 2010, Wang et al. 2011, Araji et al. 2012, etc.).

The push probe, ring electrodes, and surrounding ground were modeled using free tetrahedral elements with uniform directional scaling. High resolution regions requiring precise geometry (e.g., ring electrodes) had a minimum element dimension of 1mm, and regions of near zero current (e.g., soil elements far from the column) had a maximum element size of 0.5m. The FE model is governed by the electric currents physics interface, which solves a current...
conservation equation (based on Ohm’s law) using the scaler electric potential as the dependent variable (COMSOL 2014). Columns were modeled in a 30m diameter by 20m height cylinder to eliminate boundary effects. Boundary conditions at the border of the soil mass were defined with the electric insulation boundary condition, which prevents current from flowing through the boundary. For the depths evaluated by the probe, geometric factor $k$ varies in the near surface

Figure 5.1: Illustration of the electrical push probe geometry components. In a), the outer layer of probe is shown with inner/outer PVC pieces and ring electrodes. Figure b) shows a cross section of Figure a) to illustrate the probe’s internal supporting system via AWJ drill rod. In c), a close up of the connection is illustrated for the bottom nose cone section of the probe. This is the same connection mechanism used to attach each 1.5m section.

between half and full space conditions. Other geometric complexities such as 10cm diameter ring electrodes (as opposed the point electrodes used in the formulation of Eqs. 5.4 and 5.5) will also affect $k$. The FE model is used to estimate $k$ for each value of $a$ over the range of depths studied.
The differences between DC resistivity results obtained from idealized point electrodes compared to electrodes of finite volume is well studied in literature, and obtaining a $k$ factor using computational modeling of finite volume electrodes is a well-accepted approach (e.g., Rücker et al. 2006, Rücker and Gunther 2011, Kumar et al. 2008, Yi et al. 2009).

Figure 5.2: a) Illustration of the electrical push probe and measurement points obtained from the Wenner-$\alpha$ protocol at various electrode spacings, b) the electrical push probe attached to a placement rig, c) the push probe being submerged in a fresh DSM column, and d) the push probe after full placement.

A series of stationary electrical models is used to simulate the DC resistivity test. The modeling mimics the field test by sequentially injecting/measuring each electrode combination in the Wenner-$\alpha$ protocol used by the field probe. To simulate an individual DC resistivity measurement, a volumetric current source ($A/m^3$) is applied to ring electrode A (of known volume) and sunk to an identical ring electrode B such that the desired $i_{AB}$ is injected. Volumetric potentials ($V/m^3$) are obtained from ring electrodes M and N and converted to $\psi_{MN}$ (V) via the known ring volume. Eq. 5.3 is used to estimate $\rho_a$ with $FE_{i_{AB}}$, $\psi_{MN}$, and $k$. The application of Eq. 5.3 for the push probe in a homogeneous 2m diameter soilcrete column in
homogenous soil is illustrated in Figure 5.3. Here, the soilcrete resistivity $\rho_{sc} = 2\Omega m$ and the soil resistivity $\rho_s = 20\Omega m$ (i.e., $\rho_s/\rho_{sc} = 10$). A 2D radial cross section of the Wenner-$\alpha$ measurement protocol is shown at increasing values of $a$, illustrating the increased volume of electrical field influence that results from increasing the electrode spacing. The values of $a$ shown in Figure 5.3 correspond to the $a$ values used by the probe in field implementation (Figure 5.2a). The potential surfaces $V_M$ and $V_N$ correspond to the potential at measurement electrodes M and N, i.e., $\psi_{MN}$ from Eq. 5.3. The DC resistivity technique is proficient at locating boundaries between materials with different resistivities. The greater the difference in resistivities between two adjacent materials, the more sensitive the technique becomes to the material boundary. This concept is illustrated by the current and equipotential lines in Figure 5.3, where sharp changes in the field lines exist at the soilcrete/soil boundary.

Figure 5.3: Axisymmetric cross-section of the current flow and equipotential lines created by the electrical push probe in a 2m diameter soilcrete column for a) $a = 0.3m$, b) $a = 0.6m$, c) $a = 0.9m$, d) $a = 1.2m$.

A parametric study was performed to assess the necessary electrode spacing(s) needed to estimate column diameter over the range of constructible soilcrete column diameters in a variety of soil conditions. Bearce et al. 2015 identified a relationship between column diameter $D$ and
electrode spacing \( a \). The authors conclude that when \( D/a \geq 10 \), a direct coupled ring electrode array using a Wenner-\( \alpha \) protocol will be sensitive only to the soilcrete in the column, i.e., an in-situ measurement of \( \rho_{sc} \). Furthermore, when \( D/a \leq 5 \), Wenner-\( \alpha \) \( \rho_a \) measurements can be used to estimate column diameter \( D \). This concept can be extended to field scale columns/geometries, but additional modeling must be performed to accurately extrapolate these \( D/a \) relationships for a variety of field geometries and soil conditions.

The reported sensitivities to \( D/a \) depend on the resistivity contrast between the soil and soilcrete, i.e., \( \rho_s/\rho_{sc} \). To evaluate the feasibility of this approach on field-scale soilcrete columns, FE modeling was performed over a range of soilcrete column diameters, soil types, and electrode spacings. \( \rho_{sc} \) was assumed to be a constant 2\( \Omega \)m, which represents an average value of \( \rho_{sc} \) over the range of values reported in Bearce et al. 2015 and observed in this study. \( \rho_s \) was varied from 8-160\( \Omega \)m to simulate a variety of homogenous field conditions. These parametric study results are summarized in Figure 5.4a-f for increasing values of \( \rho_s/\rho_{sc} \), i.e., increasing soil resistivity. The x-axis of the plot shows the FE-estimated \( \rho_a \) normalized by the input soilcrete resistivity \( \rho_{sc} \). When the \( \rho_a/\rho_{sc} = 1 \), the measured \( \rho_a \) is not influenced by the surrounding soil and is only sensitive to the soilcrete. \( \rho_a/\rho_{sc} \) is plotted against a range of \( D/a \) values on the y-axis. The study was performed using \( D = 1-5 \)m (with \( D \) intervals of 0.2m) for electrode spacings of 0.3, 0.6, 0.9, and 1.2m. \( \rho_a \) was estimated using Eq. 5.3 and FE \( k \) factor at \( z_{MN} = 4 \)m. Figure 5.3 illustrates the parametric study results for Figure 5.4b, where \( \rho_s/\rho_{sc} = 10 \). For example, if \( a = 0.3 \)m, \( D/a = 6.7 \) and \( \rho_a/\rho_{sc} \approx 1 \) (i.e., the measurement is sensitive to the \( \rho_{sc} \) only). This is visible in Figure 5.3a where equipotential surfaces \( V_M \) and \( V_N \) are completely within the soilcrete column. As \( a \) increases (e.g., Figure 5.3d), the electric field is imaging both the soilcrete and the soil. \( D/a = 1.7 \) and this measurement is suitable for column geometry estimation.

The following observations can be made from the parametric study results shown in Figure 5.4. When \( \rho_a/\rho_{sc} = 1 \), the measurement is only sensitive to the soilcrete. As \( D/a \) decreases, \( \rho_a/\rho_{sc} \) increases, and the measurement becomes more sensitive to the soil (and therefore the column diameter). As \( \rho_s \) increases, \( \rho_a \) is affected by the soil at lower values of \( D/a \); however, \( \rho_a \) obtained from measurements where \( D/a \geq 6 \) is indicative of \( \rho_{sc} \) only, regardless of \( \rho_s/\rho_{sc} \). This observation holds true for any soil with \( \rho_s \leq 160\Omega \)m. The probe test protocol uses a minimum electrode spacing of 0.3m. With this configuration, the parametric study results suggest that the probe should be capable of estimating \( \rho_{sc} \) in any column where \( D \geq 1.8 \)m (i.e.,
$D/a \geq 6$). As $\rho_s$ decreases, the probe is suitable for evaluating $\rho_{sc}$ in smaller diameter columns due to the lower contrast and lessened influence of the soil on the measurement, i.e., $\rho_a/\rho_{sc}$ approaches 1 at progressively smaller values of $D/a$. The probe test protocol uses a maximum electrode spacing of 1.2m. Measurements where $\rho_a/\rho_{sc} \geq 2$ are sufficiently sensitive to diameter change and can be used to estimate column diameter. For these conditions, the probe can resolve $D$ for any column with $D \leq 2.4$m (i.e., $D/a \leq 2$). With $\rho_s/\rho_{sc} \geq 10$, the probe can resolve columns with $D \leq 3$m, and with $\rho_s/\rho_{sc} \geq 60$, the probe can resolve columns with $D \leq 3.6$m. Electrode spacings greater than 1.2m could be utilized to estimate diameter in columns of greater diameter, but this extension is not considered as the largest columns evaluated in this study are 2.43m diameter.

![Figure 5.4: Parametric study illustrating the effects of $D/a$ on measured $\rho_a$ for soil:soilcrete resistivity contrasts of a) 4:1, b) 10:1, c) 20:1, d) 40:1, e) 60:1, and f) 80:1.](image)

The parametric study indicates that a borehole Wenner-$\alpha$ protocol with direct coupled ring electrodes can provide as in-situ estimate of $\rho_{sc}$ and characterize column diameter for a range of soil conditions given an appropriate range of $D/a$ values. To assess the parametric study findings in a field environment, the DC resistivity electrical push probe in Figures 5.1 and 5.2
was developed by a research team at the Colorado School of Mines and implemented on several production DSM columns. A primary goal of this research was to design a recoverable tool that can evaluate columns immediately after construction and leave no lasting defects. Immediate and rapid post-construction testing is ideal to maximize resistivity contrast between the soil and the soilcrete. Furthermore, immersing the probe in soilcrete for and extended duration will result in soilcrete curing that will bond the probe in place, leading to lasting column defects and a non-recoverable probe. Due to this time constraint, it is important to obtain accurately estimate diameter with the minimum number of required measurements. The minimum, maximum, and intermediate electrode spacings with corresponding data point locations for the full test protocol are illustrated in Figure 5.2a.

### 5.5 Site Conditions and Testing Protocol

The electrical push probe was evaluated on two active DSM column construction sites with a variety of soil/groundwater conditions and column diameters (Figure 5.5, Table 5.1). Push probe implementation is shown in Figure 5.2b-d for a 2.43m diameter DSM column in salt water saturated sand (Figure 5.5a). An example of column/soil conditions, probe location, and resulting measurement zone is shown in Figure 5.5a for site 1. Figures 5.5b-c show the ground conditions for site 2 with a 1.83m diameter column and a 2.43m diameter column, respectively. Because the instrumented section of the probe is 6m in length, an uninstrumented extension was developed to evaluate columns at depths greater than 6m (Figure 5.5b and c).

To perform a test, the probe was hoisted from a crane or specialized drill rig (Figure 5.2b) and lowered into fresh soilcrete columns within 20-30 minutes of construction (Figure 5.2d). The probe was easily immersed in the wet soilcrete. Centering of the probe in the column was challenging and required personnel within the immediate vicinity of the column to ensure straightness. Acquisition for the push probe Wenner-α protocol took 25-30 minutes, after which the probe was removed. This approach allowed for the rapid testing of fresh columns with no post-test defects. A background electrical profile was performed using a vertical electrical sounding. These measurements were used to inform the soil resistivity profile for data interpretation and FE modeling (Figure 5.5). Soilcrete resistivity was estimated using the $a = 0.3m$ data and verified with resistivity measurements of grab sampled soilcrete obtained during column construction.
Table 5.1: Summary of test sites and columns tested with $\rho_{sc}$ values and tested probe interval.

<table>
<thead>
<tr>
<th>Site</th>
<th>Column</th>
<th>Diameter (m)</th>
<th>Column z Interval (m)</th>
<th>$\rho_{sc}$ ((\Omega m))</th>
<th>Probe Interval (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2.43</td>
<td>0-1</td>
<td>1.6</td>
<td>0-6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1-3</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3-9</td>
<td>2.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2.43</td>
<td>0-1</td>
<td>1.5</td>
<td>0-6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1-3</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3-9</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>1.83</td>
<td>0-9</td>
<td>2.1</td>
<td>0.5-6.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.83</td>
<td>0-9</td>
<td>2.9</td>
<td>0.5-6.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2.43</td>
<td>0-9</td>
<td>2.1</td>
<td>1.5-7.5</td>
</tr>
</tbody>
</table>

The probe was evaluated on increasingly complex column/soil conditions. Site 1 studied production DSM columns with a 2.43m diameter mixed to a depth of 9m in saltwater saturated sand (g.w.t. at surface). While the resistivity of the sand is fairly low ($\rho_{sc} = 10\Omega m$), it has a constant resistivity with depth (as estimated by on-site vertical electrical sounding). These field conditions provide an initial test of the push probe with fairly homogenous ground. Furthermore, the DSM process creates soilcrete columns using a physical mixing tool (unlike the erosive jet grout process) and results in a column with precisely-constructed and known diameter. Site 2 studied production DSM columns with diameters of 2.43m or 1.83m mixed to a depth of 9m in stratified SP/SM/ML. This field environment has variable soil resistivity with depth and a uniform diameter column, providing a more complex environment for probe evaluation.

5.6 Results and Discussion

The diameter estimation approach utilized in this paper is based on experimental data interpretation via FE modeling, which requires an accurately known soilcrete resistivity profile. The parametric study indicates that the push probe can provide an in-situ estimate of $\rho_{sc}$ to inform FE modeling given of $D/a \geq 6$. This observation is validated using experimental probe data, resistivity measurements on grab sampled soilcrete, and FE modeling. To provide an independent assessment of $\rho_{sc}$, on-site resistivity testing was performed on grab sampled soilcrete from the probe-tested columns. Grab sampling for site 2 column 3 (Figure 5.5c, 5.6a)
was performed at $z = 3\text{m}$ and $\rho_{sc}$ was estimated to be $1.9\Omega\text{m}$. The $a = 0.3\text{m}$ data from this column (Figure 5.6a) has a fairly uniform $\rho_a$ profile over the evaluated depth interval with average $\rho_a = 2.1 \pm 0.1\Omega\text{m}$ (i.e., $\pm 1$ standard deviation). $\rho_a = 2.1 \pm 0.1\Omega\text{m}$ is very close to the grab sampled $\rho_{sc}$ of $1.9\Omega\text{m}$. Furthermore, the uniformity in the $\rho_a$ profile suggests that the $a = 0.3\text{m}$ probe measurements are not influenced by the soil. $\rho_s$ varies from $171\Omega\text{m}$ ($z = 1\text{-}3\text{m}$) to $36\Omega\text{m}$ ($z = 3\text{+}m$) as shown in Figure 5.5c. If $a = 0.3\text{m}$ measurements were influenced by the soil, this drastic change in the $\rho_s$ profile would be visible in the $\rho_a$ response (Figure 5.6a).

**Figure 5.5:** Schematic of soil conditions, column geometry/resistivity, probe position, and axisymmetric measured region for a) site 1 column 1, b) site 2 column 2, and c) site 3 column 1. Soil conditions are assumed the same for each site across columns (Table 1), but column diameters and resistivities vary (Table 5.1).

Site 1 column 1 soilcrete was grab sampled from a depth of $z = 2.5\text{m}$ and $\rho_{sc}$ was estimated to be $2.3\Omega\text{m}$. $a = 0.3\text{m}$ $\rho_a$ is approximately $2.5\Omega\text{m}$ at $z = 2.5\text{m}$ (Figure 5.6c), which is in good agreement with the grab sampled measurement. The $\rho_a$ profile for site 1 column 1 shows variation with depth. The largest variation occurs between $z = 0\text{-}1\text{m}$ and is the result of a conductive slurry composed of cement grout and salt water with no soil grains that formed at the top of column. The change in $\rho_a$ with depth between 1 and 5m is less significant. Given the
homogeneous soil conditions, the changes observed in the $\rho_a$ profile are the result of variation in $\rho_{sc}$. The reason for this variation is not known; however, site 1 columns were mixed with a curing retarder to facilitate a longer available test window. This retarder was not used on site 2 columns.

To further validate the probe’s ability to provide an in-situ estimate of $\rho_{sc}$, FE modeling was performed to assess the sensitivity to diameter changes in the $a = 0.3\text{m}$ measurements. FE models of the push probe, soilcrete column, and soil profiles were evaluated and the FE-predicted $\rho_a$ responses are plotted in Figures 5.6b and 5.6d for site 2 column 3 and site 1 column 1, respectively. The $\rho_a$ response for a column with expected 2.43m diameter is defined as the solid line response $\rho_D$. To evaluate the measurement’s sensitivity to diameter changes, columns are modeled with diameters $0.9D (D = 2.19\text{m})$ and $1.1D (D = 2.67\text{m})$. The $\rho_a$ responses for these $\pm10\%D$ variations are plotted as the dashed lines $\rho_a (0.9D)$ and $\rho_a (1.1D)$ on either side of the solid line $\rho_a (D)$ response (Figures 5.6b and 5.6d). The change in FE $\rho_a$ response resulting from $\pm10\%D$ variation is minimal, and the response curves are difficult to distinguish. For site 2 column 3 (Figure 5.6b), scatter in experimental $\rho_a$ is greater than the variation in response from the $\pm10\%D$ variation. Increasing or decreasing column diameter by 10% results in a $\rho_a$ variation of approximately $\pm2\%$, suggesting that these measurements are not sensitive to diameter changes. Because the $\rho_a$ response is not sensitive to diameter, i.e., the soil/soilcrete interface, the measurement is not influenced by the soil and is thus only indicative of $\rho_{sc}$. For site 1 column 1 (Figure 5.6d), scatter in experimental $\rho_a$ is again greater than the change in response from the $\pm10\%D$ variation. In addition, the FE model captures the curvature observed in the experimental $\rho_a$ response using the variable $\rho_{sc}$ and homogeneous soil profile shown in Figure 5.5a. Experimental results are in generally good agreement with the parametric study regarding $\rho_{sc}$ prediction using the $a = 0.3\text{m}$ measurement. This conclusion is further supported by the agreement between grab sampled $\rho_{sc}$ and probe-measured $\rho_a$. Furthermore, FE modeling of the probe/column/soil indicates minimal sensitivity to diameter variation and is able to capture the behavior in the experimental data for columns with uniform and variable $\rho_{sc}$ and $\rho_s$ profiles. This is an important conclusion as probe-estimated $\rho_{sc}$ values are used as inputs for the FE modeling (and thus column diameter estimation).
Experimental $\rho_a$ is calculated using Eq. 5.3. Resistance is estimated with experimental $\psi_{MN}/i_{AB}$ and converted to $\rho_a$ using the FE-estimated $k$ factor. A visual example of the conversion from $R$ to $\rho_a$ is shown in Figure 5.7 for $a = 0.6$, 0.9, and 1.2m, i.e., the electrode spacings used to estimate column diameter. Each response in Figure 5.7a is multiplied by the corresponding value in 5.7b (same symbol at same depth) to obtain the response in 5.7c. Note the variation of the $k$ factor with depth. The greater the value of $a$, the more volume is influenced by the injected electrical field. This larger volume of influence causes surface boundary effects to be more prominent at greater depths. Thus, the larger the value of $a$, the greater the depth required to reach full space conditions. The probe $k$ factors shown in Figure 5.7b are used to obtain the $\rho_a$ values in Figures 5.8-5.10.

Experimental $\rho_a$ is shown in Figures 5.8-5.10 for site 1 column 1, site 2 column 1, and site 2 column 3, respectively. These columns represent the range of diameters evaluated for each site and soil profile (Figure 5.5, Table 5.1). In addition to the experimental data, FE responses for columns of diameter $0.9D$, $D$, and $1.1D$ are plotted and again defined as $\rho_a(0.9D)$, $\rho_a(D)$, and $\rho_a(1.1D)$, respectively. This is the same approach utilized for the $a = 0.3m$ data; however,
measurements from the larger $a$ values have an observable sensitivity to changes in diameter. The $D \pm 10\% D$ approach helps to illustrate both uncertainty in the experimental data and FE sensitivity to column diameter change as a function of $a$.

Figure 5.7: a) Experimental resistance data from Site 2 Column 1 for $a = 0.6, 0.9,$ and $1.2$m, b) FE-estimated geometric correction factor $k$ for each value of $a$ in a), and c) apparent resistivity response obtained by multiplying a) and b) for any given depth and value of $a$ (i.e., Eq. 5.3).

Site 1 column 1 (Figure 5.8) shows an experimental data set from a 2.43m diameter column constructed in salt water saturated sand (Figure 5.5a). While the sand has a homogeneous resistivity over the range of depths evaluated (determined via vertical electrical sounding), the soilcrete has a variable resistivity profile with depth (Figure 5.6c). As expected, the measured $\rho_a$ increases with increased $a$ because the array is imaging a proportionally larger volume of the higher resistivity soil; however, changes in $\rho_a$ response with depth are the result of the variation in $\rho_{sc}$ because $\rho_s$ does not change.

Site 1 column 1 $\rho_a$ data (Figure 5.8) are well fit to the $\rho_D$ responses for $a = 0.9$m and 1.2$m$, but $a = 0.6$m $\rho_a$ data are scattered around the expected $\rho_D$ response. The salt water saturated sand has a relatively low resistivity ($\rho_s = 10\Omega$m and $\rho_s/\rho_{sc} = 4$) which leads to a lower sensitivity to changes in $D$ for all values of $a$. Applying the results of the parametric study to the $a = 0.6$m measurement in these soil conditions ($D/a = 4$, Figure 5.3a), $\rho_a/\rho_{sc} \approx 1.2$ and the
scatter in experimental $\rho_a$ (Figure 5.8a) is greater than the response change from ±10$D\%$
variation. Since the column diameter is precisely constructed, this implies that the error in the
measurement can be larger than ±10$D\%$ (e.g., $z = 3\,m$) for this combination of $D/a$ and soil
conditions. For $a = 0.9\,m$ ($D/a = 2.7$) and $a = 1.2\,m$ ($D/a = 2$), the misfit between experimental $\rho_a$
and the expected $\rho_D$ is considerably smaller than the $a = 0.6\,m$ data. The increased sensitivity to
changes in $D$ that results from increasing $a$ is evidenced in the width of the ±10$D\%$ dashed line
envelopes ($\rho_a(0.9D)$ and $\rho_a(1.1D)$). For $a = 0.6\,m$ (Figure 5.8a), the difference between $\rho_a(0.9D)$
and $\rho_a(1.1D)$ is approximately 0.25Ωm. When $a = 0.9\,m$ (Figure 5.8b), the $\rho_a(0.9D)$ to $\rho_a(1.1D)$
envelope has a width of 0.5Ωm. As $a$ is increased to 1.2m, the $\rho_a(0.9D)$ to $\rho_a(1.1D)$ envelopes has
a width of approximately 1Ωm.

Site 2 column 1 experimental data from a 1.83m diameter column constructed in
stratified SP/SM/ML (Figure 5.5b) are shown in Figure 5.9. The curved shape of the $\rho_a$ response
with depth is the result of homogenous soilcrete with a variable soil resistivity profile. This is
different than site 1 (Figure 5.8) where changes in $\rho_a$ response are the result of variable soilcrete
with homogeneous soil. As the measured $\rho_a$ is an average of all media in the volume influenced
by the injected electrical field, there are discernible trends in the data related to the soil
stratification. For example, the more resistive $\rho_a$ responses around $z = 1.5\,m-4\,m$ are the result of
the 171Ωm SM layer (Figure 5.5b). As the measurements are acquired at greater depths,
proportionally more of the 26Ωm ML layer is being imaged, and thus the responses become less
resistive.

Site 2 column 1 data (Figure 5.9) are well fit to the $\rho_a(D)$ responses for all values of $a$.
The stratified SP/SM/ML soil around these columns provides $\rho_s/\rho_{sc} \approx 13-80$. Under these
conditions, $D = 1.83\,m$ and $D/a$ ranges from 1.5-3. The experimental $\rho_a$ is very close to the
predicted $\rho_a(D)$ (Figure 5.9) despite the significant $\rho_s$ variation with depth. The $a = 0.9\,m$
response (Figure 5.9b) deviates from the expected response from $z = 4\,m-5\,m$. The reason for this
misfit is unknown given the generally good agreement in the $a = 0.6$ and 1.2m responses. Given
the larger contrast between soil and soilcrete and smaller column diameter, the width of the
$\rho_a(0.9D)$ to $\rho_a(1.1D)$ envelope is greater for site 2 column 1 (Figure 5.9) than in site 1 column 1
(Figure 5.8) for all values of $a$. For $a = 0.6$, 0.9, and 1.2m, the $\rho_a(0.9D)$ to $\rho_a(1.1D)$ envelope has
an average width of 0.8Ωm, 1.5Ωm, and 2Ωm, respectively. The stratified soil (especially the
171Ωm ML layer) causes minor variation in the width of this envelope, further illustrating the increased sensitivity to diameter with increased $\rho_s/\rho_{sc}$.

![Figure 5.8: Comparison of experimental vs. FE $\rho_a$ responses for Site 1 Column 1.](image)

Site 2 column 3 experimental $\rho_a$ data from a 2.43m diameter column in stratified SP/SM/ML (Figure 5.5a) are shown in Figure 5.10. While this column is constructed in the same soil conditions as site 2 column 1 (Figure 5.5b, 5.9), the column has a larger diameter and was evaluated at a greater depth interval. The upper portion of these responses is most resistive because the probe is already within the 171Ωm ML layer. As measurements are acquired at greater depths, the responses become gradually less resistive. Site 2 column 3 $\rho_a$ responses are well fit to the $\rho_a(D)$ responses for all values of $a$. The stratified SM/ML soil around these columns provides $\rho_s/\rho_{sc} \approx 13-80$. Under these conditions, $D = 2.43m$ and $D/a$ ranges from 2-4. The experimental $\rho_a$ is very close to the predicted $\rho_a(D)$ (Figure 5.10) for all values of $a$. The $\rho_a(0.9D)$ to $\rho_a(1.1D)$ envelopes have similar values to those observed in site 2 column 1 (Figure 5.9).

To assess the probe’s capability for diameter prediction, a linear correlation is formed using the points $(\rho_a(0.9D), 0.9D), (\rho_a(D), D)$ and $(\rho_a(1.1D), 1.1D)$. This process is illustrated in Figure 5.11 using site 2 column 1 (Figure 5.9) as an example. Each combination of $z$ and $a$ results in a different correlation ($a = 0.9m$ example shown in Figure 5.11a). Figure 5.11b shows
the correlation obtained from Figure 5.11a at \( z = 1.8m \). Here, \( \rho_a(1.1D) = 4.0\Omega m \) and this FE response is indicative of a column with a diameter of \( 1.1D \) (2.01m). This relationship is used to form the \( (x, y) \) correlation point (4.0\( \Omega m \), 2.01m) as shown in Figure 5.11b. For the FE response obtained from a column with diameter \( D \) (1.83m), \( \rho_a(D) = 4.7\Omega m \) and the correlation point (4.7\( \Omega m \), 1.83m) is formed. Similarly, the correlation point (5.5\( \Omega m \), 1.65m) is formed from the \( \rho_a(0.9D) \) response. These three correlation points are plotted in Figure 5.11b and the corresponding experimental \( \rho_a \) value is plotted on the correlation line to formulate a diameter prediction. For \( z = 1.8m \) the predicted diameter, termed \( \bar{D} \), equals 1.87m. A similar example is shown in Figure 5.11c for \( z = 4.3m \) (Figure 5.11a). Note that the diameters for each correlation point in Figure 5.11c are the same as Figure 5.11b as they are based on the response obtained from the modeling of a particular constant diameter FE column; however, the \( \rho_a \) responses are different for each point because \( \rho_a \) varies with depth depending on soilcrete/soil geometry and resistivity profile. The correlations illustrated in Figures 5.11b and c are formulated for each value of \( z \) and \( a \). \( \bar{D} \) is estimated over the measured depth interval for each spacing (Figure 5.11d) and \( \bar{D} \) values are averaged across all values of \( a \) to obtain the \( \bar{D} \) estimate shown in Figure 5.11e.

A total of five columns were tested (Table 5.1) and the same modeling approach illustrated in Figures 5.8-5.10 was applied to the data. These figures illustrate an example column for each combination of soil conditions and column diameter. The diameter estimation
approach (Figure 5.11) is applied to all tested columns (Figure 5.12) with solid line responses on either side of the data indicating ±5% $D$. For each column, the predicted diameter matches the constructed diameter to within ±5% $D$.

![Figure 5.10](image1.png)

Figure 5.10: Comparison of experimental vs. FE $\rho_a$ responses for Site 3 Column 1.

![Figure 5.11](image2.png)

Figure 5.11: a) Experimental $\rho_a$ compared to FE $\rho_a$ for columns with diameters of 0.9$D$, $D$, and 1.1$D$ and $a = 0.9$m. Diameter is estimated via linear correlation for b) $z = 1.8$m and c) $z = 4.3$m. Plot d) shows the predicted diameter $\bar{D}$ using each appropriate value of $a$. Plot e) shows the average $\bar{D}$ estimated by averaged the $\bar{D}$ estimates in d) using all values of $a$.  

111
Figure 5.12: Estimated soilcrete column diameter $D$ with ±5% D bounding lines for a) site 1 column 1, b) site 1 column 2, c) site 2 column 1, d) site 2 column 2, e) site 2 column 3, and f) site 3 column 1. Reported depth intervals reflect the probe position and corresponding measurement profile (e.g., Figure 5.5).

5.7 Conclusions

A parametric study was conducted using FE modeling of the DC electrical resistivity test in field-scale soilcrete columns to inform the design of a push probe for field implementation. The resulting 6m probe had twenty ring electrodes (10cm diameter) with a minimum spacing of 0.3m. The probe utilized electrode spacings of 0.3, 0.6, 0.9, and 1.2m. These spacings were carefully selected from modeling results for their ability to provide both an in-situ estimate of soilcrete resistivity and an estimate of column geometry with a minimum number of necessary measurements.

The push probe was tested on five columns at two DSM column construction sites over a variety of column diameters and soil/groundwater conditions. Trends in experimental results were analyzed, compared to the observations from the parametric study, and found to be in good agreement. This conclusion helps validate the parametric study, making it a useful guide for selecting appropriate electrode spacings for DC resistivity push probe testing if the expected column diameter and soil conditions are known prior to construction.

The probe provides an additional measurement advantage in its ability to characterize $\rho_{sc}$ throughout the column. Because $\rho_{sc}$ can vary within an individual column (e.g., Figure 5.6b) a grab sample measurement from a single depth may not be indicative of $\rho_{sc}$ for the entire column.
Because an accurate estimate of $\rho_{sc}$ is critically important in obtaining an accurate FE model (and thus diameter estimate), the in-situ $\rho_{sc}$ measurement provided by the probe is an improvement over the existing grab sample practice.

Additional FE modeling was performed with columns of diameters $\pm 10\% D$ and used to estimate the diameter of tested columns in conjunction with experimental data. Using $\rho_{sc}$ inputs obtained from $a = 0.3m$ data and $\rho_s$ values obtained from background electrical profiling, the push probe modeling approach is able to estimate the column diameter to within $\pm 5\% D$ (Figure 5.12). The electric cylinder method (Frappin and Morey 2001) recommends measuring the geometry of columns in salt water saturated media after full curing. Cured soilcrete becomes more resistive than the surrounding ground, providing a better contrast to the less resistive salt water saturated soil than fresh soilcrete. The push probe is able to estimate column diameter on 2.43m diameter soilcrete columns in salt water saturated sand immediately after curing to within $\pm 5\% D$ (Figure 5.8, 5.12a and b). This is an improvement over the existing technique as column performance verification testing can be performed immediately.

This study highlights a promising new technology for diameter estimation of soilcrete columns. The test is relatively quick (completed within an hour of column construction) and truly non-destructive (i.e., the probe is removed while the soilcrete is still wet/fresh and causes no lasting defects). To this end, the probe is ideal for immediate geometric assessment of production columns (which is an improvement over any existing technique). Extensions of this research will seek to evaluate additional columns to further assess the probe’s diameter prediction capability. This study validates the probe’s diameter estimation capability on uniform diameter DSM columns with precisely constructed geometry and indicates that the probe can estimate diameter to within $\pm 5\% D$. Future testing will be extended to jet grouted columns where diameter variation is possible. In addition, a data inversion routine will be developed (effort underway) to better quantify diameter variations in variable diameter columns.

5.8 Acknowledgements
The authors would like to thank industry collaborator Hayward-Baker & Associates for providing DSM test sites and on-site support for implementation of testing equipment.
5.9 References


CHAPTER 6:
ESTIMATION OF JET GROUT COLUMN GEOMETRY USING A DC ELECTRICAL RESISTIVITY PUSH PROBE

Modified from a paper to be presented at the International Symposium on Non-Destructive Testing in Civil Engineering and submitted to a special conference issue of Near Surface Geophysics

R. G. Bearce, M. A. Mooney, and P. Kessouri

6.1 Abstract:
Jet grouting is common ground improvement technique that mixes grout (water and cement) with in-situ soil to form soilcrete columns in the subsurface. These columns have many uses including underpinning foundations, creating hydraulic barriers, slope stabilization, etc. By design, soilcrete columns are cylindrical, but the turbulent grout/soil mixing process used in jet grouting can result in geometric uncertainty. The inherent heterogeneity of field soils further complicates this issue. Adequate jet grout column performance requires diligent quality assurance and quality control, but current assessment techniques have limitations. Due to the required curing time and destructive nature, tests for geometry estimation are usually only performed on test columns, leading to uncertainty in performance of production columns. The DC electrical resistivity method exploits the resistivity contrast between the low resistivity soilcrete and the relatively higher resistivity in-situ soil to estimate the boundary between the two materials. While DC resistivity has been applied to jet grout columns in the past, a new approach utilizing a recoverable probe with directly coupled electrodes provides advantages over the existing approach. An electrical push probe was developed at the Colorado School of Mines and used to test jet grout columns near Berlin, Germany. Results of preliminary field testing suggest that the electrical push probe approach is readily implementable on production columns and can provide a rapid, non-destructive estimate of jet grout column diameter using a recoverable/reusable device.

6.2 Introduction:
Ground improvement via jet grouting is commonly used in civil and underground construction to stabilize weak/fractured ground, underpin foundations, and create hydraulic
barriers in the subsurface. The jet grouting process uses high pressure fluid and/or grout (water/cement mixtures) sprayed radially from a spinning drill string to erode the in-situ soil and create columns of soilcrete (i.e., a type of concrete composed of grout mixed with in-situ soil). Variation in grouting parameters (grout pressure, cement content, etc.) and ground conditions (variable soil type/density, water table, etc.) leads to uncertainty in the realized column diameter. Jet grout column diameters must be precisely constructed to perform according to design. The inherent diameter variations resulting from the volatile nature of jet grouting process can lead to inadequate performance (e.g., leaking hydraulic barriers, insufficient foundation support, etc.). To ensure proper performance, verification of jet grout column diameter is critically important.

Jet grouting contractors have adopted several techniques to estimate column diameter, but these approaches have inherent limitations. Destructive tests such as radial coring/probing and column excavation can be used but require 2+ days for sufficient curing, are difficult to perform below the water table, and can only be performed on test columns due to the destructive nature of the tests (e.g., Duzceer and Gokalp 2004, Yoshida 2010, Burke 2012, Bruce 2012, Wang et al. 2012, etc.). For verification purposes, grouting contractors often verify performance of one or more test columns and assume that the production columns constructed in the same environment will have the same geometry because the ground/grouting conditions are the same. Due to inherent geological heterogeneity and lack of precise repeatability in grouting parameters, this assumption is not always true.

Non-destructive geophysical inspection approaches have also been proposed. Surface/downhole seismic and crosshole acoustic/seismic techniques for geometry estimation have been successfully implemented, but these approaches require permanent casings in or near the jet grout column (Madhyannapu et al. 2010, Galindo-Guerreros et al. 2015a,b, Mackens et al. 2015). Ground penetrating radar (T&A Survey 2013) and DC resistivity (Frappin and Morey 2001, Frappin 2011) have been applied to jet grout column geometry assessment, but these techniques also require a permanent casing placed in or near the column. Because these approaches all require permanent casings in or near the column, they are not feasible for rapid assessment of multiple production columns. Furthermore, the casings are not recoverable, adding an additional cost per tested column that is not efficient for evaluating production columns (as columns can number in the 10’s to 100’s, depending on site and application).
This paper presents the results of a field study on jet grout columns constructed in sand at a field site near Berlin, Germany. The study uses a recoverable/reusable DC electrical resistivity push probe developed by researchers at the Colorado School of Mines (Bearce et al. 2015b). The probe uses direct coupled electrodes and a borehole Wenner-α protocol to non-destructively estimate the diameter of jet grout columns immediately after construction. Computational modeling is used to validate the experimental results. Additional modeling is used to estimate column diameter. This research presents an improved method for jet grout column diameter inspection that can be performed immediately after construction and leave no lasting defects in the column, thus making it suitable for rapid evaluation of production columns.

6.3 Background:

DC resistivity is a classical electrical geophysical technique that characterizes a material’s ability to resist current flow. The technique works by injecting current across two electrodes (A and B, Figure 6.1b) to create an electric field in the ground. This electric field is sampled by measuring the potential difference across two (or more) measurement electrodes (M and N, Figure 6.1b). Using a series of measurements at various electrode spacings (e.g., Figure 6.1a), DC resistivity can identify boundaries between materials with different resistivities. The DC resistivity test provides an estimate of the ground’s apparent resistivity $\rho_a$ ($\Omega \text{m}$)

$$\rho_a = \left(\frac{\psi_{MN}}{i_{AB}}\right) \cdot k = R \cdot k \quad (6.1)$$

where $\psi_{MN}$ is the potential difference measured across electrodes M and N (V), $i_{AB}$ is the current injected across electrodes A and B (A), and $k$ is the geometric correction factor (m). The $\rho_a$ obtained from DC resistivity measurements is not the same as a material’s true resistivity $\rho$. $\rho_a$ is a weighted average of all $\rho$ in the volume of material influenced by the injected electrical field. In homogeneous media, $\rho = \rho_a$, but in heterogeneous media, $\rho_a$ is affected by all values of $\rho$ through which the injected electrical field is propagated. In practice, $\rho$ is often obtained by inverting $\rho_a$ data from many DC resistivity measurements at various electrode spacings (Revil et al. 2012).
The push probe can acquire data from just below the ground surface to a maximum depth of 7m, which results in a variable $k$ factor in the near surface. For the Wenner-$\alpha$ on the surface of an infinite homogenous halfspace using point electrodes,

$$k = 2 \cdot \pi \cdot a$$  (6.2)

where $a$ is the distance between any two adjacent array electrodes for a given measurement protocol. For the Wenner-$\alpha$ protocol used by the probe, $a = AM = MN = NB$. For an infinite homogeneous full space with point electrodes,

$$k = 4 \cdot \pi \cdot a$$  (6.3)

In practice, full space conditions apply when measurements are sufficiently deep in the ground such that no surface boundary effects are present. The near surface geometric factor transitions from half to full space conditions. The depth required to reach a full space condition is dependent on the electrode spacing, but occurs between 4-8m depth for the electrode spacings used by the push probe. $k$ factors for the probe are similar to Equation 6.2 near the surface and Equation 6.3 at sufficient depth (which depends on electrode spacing); however, the 8cm diameter ring electrodes (as opposed to the point electrodes assumed for Equations 6.2 and 6.3) also affect $k$. The push probe $k$ factors were determined for each applicable depth and $a$ value (at 0.3m intervals) using finite element (FE) modeling in Cosmol Multiphysics® performed by Bearce et al. 2015b.

As applied to jet grout column testing, DC resistivity identifies the boundary between the low resistivity in freshly mixed soilcrete and the relatively more resistive in-situ soil. This concept is illustrated in Figure 6.1b for a measurement with $a = 0.9$m on a 1.2m diameter column in stratified soil. The sharp changes in the current/equipotential lines at the soil/soilcrete boundary illustrate the measurement’s sensitivity to the material interface when there is a large resistivity contrast. While soil resistivity can vary greatly depending on soil type and groundwater conditions (10s-100s of $\Omega$m), freshly mixed soilcrete has a much lower resistivity (approximately 1.5-3$\Omega$m) that depends on the cement type and the grout to soil ratio in the soilcrete (Bearce et al. 2015a,b).
The low resistivity of fresh soilcrete is due to the highly ionic pore fluid in the cement grout being mixed with in-situ soil. Immediately after mixing, soilcrete is in a wet slurry form with highly connected pore space. This initial slurry state maximizes the porosity and the ionic concentration in the pore fluid. As curing occurs, the pore fluid ions undergo chemical reactions that form cementing compounds and bond soil grains together (Taylor 1997). As pore fluid ions are transformed in chemical reactions, the soilcrete’s resistivity increases. The cementing of soil grains results in reduced/disconnected porosity and also increases the soilcrete resistivity (Rajabipour et al. 2007). Because of the large resistivity contrast between the in-situ soil and fresh soilcrete, DC resistivity is able to locate the boundary between these two materials (and thus the column diameter) if measurements are acquired shortly after column construction.

The finite element modeling used to obtain $k$ factors and validate experimental results was performed using COMSOL Multiphysics®. This software package is commonly used for modeling the DC resistivity test (Kumar et al. 2008, Chou et al. 2010, Clement et al. 2011) and geotechnical/geological DC resistivity applications (Kim et al. 2009, Huang and Lin 2010, Wang et al. 2011, Araji et al. 2012, etc.). Bearce et al. 2015b used this modeling approach to inform the development of the push probe used in this study and to interpret the experimental results obtained from soilcrete columns constructed with deep soil mixing (DSM). The FE model utilized 3D geometry with free tetrahedral elements of uniform directional scaling. High resolution regions (e.g., ring electrodes) had a minimum element dimension of 1mm, and regions of near zero current (far from the column) had a maximum element size of 0.5m. The FE model is governed by the electric currents physics interface that solves a current conservation equation (based on Ohm’s law) using the scaler electric potential as the dependent variable (COMSOL 2014). The soilcrete columns were modeled in a 30m diameter by 20m height cylinder with electric insulation on the external boundaries. The FE model simulated the DC resistivity test with a series of stationary electrical models (one for each measurement shown in the protocol in Figure 6.1a). To simulate an individual DC resistivity measurement, a volumetric current source ($A/m^3$) is applied to ring electrode A (of known volume) and sunk to an identical ring electrode B such that the desired $i_{AB}$ is injected. Volumetric potentials (V/m$^3$) are obtained from ring electrodes M and N and converted to $\psi_{MN}$ (V) via the known ring volume. Equation 1 is used to estimate $\rho_a$ with FE $i_{AB}$, $\psi_{MN}$, and $k$. 

122
Figure 6.1: a) Illustration of the 20 electrode push probe with corresponding data points for a full protocol. An example array length (3\(a\)) is illustrated for each value of \(a\) using the top electrode as injection electrode A. b) An illustration of column 1 and the current/equipotential lines resulting from an \(a = 0.9\)m measurement.

6.4 Experimental Procedure:

The jet grout columns evaluated in this study were constructed at a depth (\(z\)) of 3-10m at a test site south of Berlin, Germany operated by the BAM Federal Institute for Material’s Research and Testing. Construction of the jet grouted columns is illustrated in Figure 6.2a. A profile of the constructed columns and background soil profile is illustrated in Figure 6.3. Column 1 was grouted at a constant pressure of 40MPa (Figure 6.3a), and on-site contractors estimated column 1 diameter to be between 1.2 and 1.3m. Crosshole seismic testing performed by Galindo-Guerreros et al. (2015b) indicates that column 1 diameter is approximately 1.25m over the \(z = 3-7\)m depth interval (i.e., the depth interval tested by the push probe). Column 2 was
grouted at a pressure of 30MPa from \( z = 3\text{-}6.5 \text{m} \) and 40MPa from \( z = 6.5\text{-}10 \text{m} \) (Figure 6.3b). Crosshole seismic results indicate a diameter of 1.25-1.3m from \( z = 6.5\text{-}7 \text{m} \) and a reduced column diameter of 0.9-1.1m from \( z = 3\text{-}6.5 \text{m} \); however the authors note that there is some uncertainty in results near the top of the column due to difficulty in accurate selection of arrival times in/near the unsaturated sand layer Galindo-Guerreros et al. (2015b). After grouting, the electrical push probe was hoisted from a crane and lowered into the freshly mixed jet grout column (Figure 6.2b). Because the probe length is 6m and the jet grout columns start at a depth of 3m, it was necessary to outfit the probe with an uninstrumented extension to allow the instrumented probe section to reach adequate depth (Figures 6.2b, 6.3).

The site contains post glacial sediments consisting of sandy layers of varying grain size with some silts and organic materials. The groundwater table varies seasonally, but has a depth of approximately \( 3 \pm 1 \text{m} \) (Niederleitner et al. 2012). At the time of data acquisition, the groundwater table was measured at 3m. Ensuring accurate modeling results (and thus diameter prediction) requires precisely known resistivity values for both the in-situ soil and the soilcrete. To obtain a resistivity background profile for the in-situ soil (prior to jet grouting), crosshole dipole-dipole measurements were conducted using permanently embedded ring electrode casings with a 14m length, 3m horizontal separation, and 0.5m minimum electrode spacing. While the depth interval evaluated is composed predominantly of sand, crosshole DC resistivity tests indicate that the sand has stratified resistivity with depth (Figure 6.3). The most resistive layer is the dry layer above the water table. Below the water table, the ground becomes increasingly resistive with depth, which is likely due to the increased density (and thus reduced porosity) of the saturated sand.

The electrical push probe used in this research had a 6m length, 10cm diameter, and twenty ring electrodes spaced at 0.3m (Figure 6.1a). Measurements were acquired using a downhole variation of the traditionally surface-based Wenner-\( \alpha \) protocol with electrode spacing \( a = 0.3, 0.6, 0.9, \text{and } 1.2 \text{m} \). The illustration in Figure 6.1a conveys the array width for the topmost measurement point (at a depth \( Z_{MN} \)) for each value \( a \). Subsequent measurement points are acquired by moving these configurations down the probe at 0.3m intervals to obtain the full measurement profile illustrated by the field of data points. While additional measurements could be acquired using this twenty electrode configuration (e.g., \( a = 1.5 \text{m}, 1.8 \text{m}, \text{etc.} \)), the test uses the minimum number of required measurements to facilitate rapid testing and ensure adequate time
for instrument recovery in the curing/hardening soilcrete. The measurements obtained from the push probe are an axisymmetric average of the volume around the probe, and for the purpose of diameter estimation, the probe estimates an average diameter with depth.

6.5 Results and Discussion:

$\rho_a$ responses are estimated using field-measured $\psi_{MN}$ and $i_{AB}$, FE-determined $k$ factors, and Equation 6.1. The probe can provide an in-situ estimate of soilcrete resistivity ($\rho_{sc}$) for inputs the FE model. For sufficiently large column diameter ($D$) and sufficiently small electrode spacing ($a$), the probe is sensitive only to the soilcrete resistivity because the depth of measurement influence is not deep enough to image the soil. Bearce et al. 2015a,b define this relationship as a unitless parameter $D/a$. The larger the value of $D/a$, the more sensitive the measurement is to $\rho_{sc}$ only. For the smallest electrode spacing on the probe ($a = 0.3m$ and $D/a = 4$ for column 1), experimental $\rho_a$ data are used as an in-situ estimate of the $\rho_{sc}$ (Figure 6.4a). The $a = 0.3m$ measurements from column 1 are representative of the soilcrete only. This is

Figure 6.2: a) Jet grouting at the Horstwalde field site, and b) implementation of the electrical push probe immediately after grouting.
evidenced by the relatively uniform response over the entire depth profile (Figure 6.4a). If the measurement was also imaging soil, the $\rho_a$ response would increase with depth according to the soil resistivity profile (Figure 6.3a). Furthermore, laboratory and field-sampled soilcrete from previous studies with similar grout composition and soil type has a $\rho_{sc}$ of approximately 2\,Ωm immediately after mixing (Bearce et al. 2015a,b), which is consistent with the $\rho_{sc}$ predicted by column 1 measurements where $D/a = 4$. In column 2, $D/a = 4$ below $z = 6.5$\,m and the measurement is representative of $\rho_{sc}$. In the region above $z = 6.5$\,m, the column diameter is reduced ($D/a = 3$), and the $a = 0.3$\,m measurement is also imaging the soil profile outside the column. The data in this region has significantly more scatter and is not indicative of $\rho_{sc}$. Given the homogeneity of the soilcrete in column 1 and the general agreement in $\rho_{sc}$ values between both columns below $z = 6.5$, $\rho_{sc}$ is assumed to be a homogeneous 1.9\,Ωm throughout the entire column 2. An electrode malfunction during testing resulted in a gap in the data profiles as illustrated in Figure 6.4.

The responses for $a = 0.6$, 0.9, and 1.2\,m are shown for columns 1 and 2 in Figures 6.5 and 6.6, respectively. $a = 0.3$\,m measurements indicate that $\rho_{sc}$ is constant with column depth (Figure 6.4a), and thus, the curvature of the $\rho_a$ responses for $a = 0.6$, 0.9, and 1.2\,m are the result of the changing soil conditions. The gradual decrease then increase between $z = 3$-7\,m is stems from the soil resistivity profile shown in Figure 6.3a. The topmost layer of sand is dry, causing a larger $\rho_a$ response. The second layer is below the water table and significantly less resistive, causing the $\rho_a$ responses to also become gradually less resistive. As measurements are acquired deeper in the column, the increasing soil resistivity with depth causes the $\rho_a$ responses to again become more resistive. The sharp change in $\rho_a$ responses at $z = 3$\,m results from the transition from in-situ soil to soilcrete column. While there is a small amount of soilcrete surrounding the probe from 0-3\,m (a monitor hole of approximately 15\,cm), this diameter is too small to influence the measurements and $\rho_a$ is most sensitive to the in-situ soil profile above $z = 3$\,m.

FE modeling of column 1 was conducted assuming the as-built column diameter $D = 1.25$\,m and the soil/soilcrete properties shown in Figure 6.3a. The $\rho_a$ response obtained from modeling the geometry and soil/soilcrete properties shown in Figure 6.3a results in the solid line response $\rho_a(D)$ (Figure 6.5). Here, $\rho_a(D)$ is defined as the apparent resistivity response obtained from modeling a column with diameter $D$ as shown in Figure 6.3a. For each value of $a$, $\rho_a(D)$ is generally well fit to the experimental $\rho_a$. The model captures the sharp transition between in-situ
soil and soilcrete column at $z = 3\text{m}$ and generally follows the same trends with depth as the experimental $\rho_a$ data.

Figure 6.3: Soil profile, grouting parameters, probe positions, and measured region for a) column 1, and b) column 2.

To assess the sensitivity of apparent resistivity to diameter changes, additional FE modeling is performed assuming $D$ varies by ±10% (i.e., $0.9D$ and $1.1D$, where $D = 1.25\text{m}$). The responses obtained from the modeled diameter variations are shown as dotted lines on either side of the $\rho_a(D)$ response (defined as $\rho_a(1.1D)$ and $\rho_a(0.9D)$). As expected, there is notable symmetry among the three FE responses. As the column diameter is reduced to $0.9D$, the probe is imaging more of the in-situ soil and the response is accordingly more resistive. Similarly, as the diameter is increased to $1.1D$, the response becomes less resistive as there is proportionally more soilcrete.
in the volume of measurement. In Figure 6.5a, where \( a = 0.6 \) m, the difference between \( \rho_a (1.1D) \) and \( \rho_a (0.9D) \) is approximately 1.5\( \Omega \)m over the column interval \( (z = 3-7) \) m. As \( a \) increases to 0.9m, the width of the envelope curves is approximately 2\( \Omega \)m (Figure 6.5b). Note that sensitivity to both the column and soil is also visible in Figure 6.1b, which corresponds to column 1 geometry/resistivity profile obtained from an \( a = 0.9 \) m measurement. At \( a = 1.2 \) m, the envelope width equals 8\( \Omega \)m (Figure 6.5c), suggesting even greater sensitivity to the soil/soilcrete boundary.

Figure 6.4: Experimental \( \rho_a \) profiles from the \( a = 0.3 \) m electrode spacing for a) column 1 and b) column 2.

Bearce et al. 2015a,b also uses the \( D/a \) parameter to estimate sensitivity to diameter changes. In general, the smaller the value of \( D/a \), the more sensitive the \( \rho_a \) measurement is to changes in column diameter. This trend is evidenced in Figure 6.5 where decreasing values of \( D/a \) correspond to increased width in the \( \pm 0.1D \) dashed line envelopes. Bearce et al. 2015b exploit these changes in response to provide an experimental estimate of actual column diameter \( (\bar{D}) \) with a linear correlation from points \( (\rho_a (0.9D), 0.9D), (\rho_a (D), D), \) and \( (\rho_a (1.1D), 1.1D) \). The authors validated this approach on DSM columns because of their precisely constructed
diameter. While the soilcrete created by jet grouting and DSM is similar, DSM columns are mixed with a mechanical mixing tool of precise size (as compared to the often variable erosive mixing process used by jet grouting). The precisely constructed column diameters created by DSM provide a ground truth for analyzing probe data and informing/validating FE modeling of the experimental results. Bearce et al. 2015b conclude that the push probe measurements in conjunction with FE modeling can estimate DSM column diameter, termed $\bar{D}$, to within $\pm 5\%$ of the as-constructed diameter $D$ (which for DSM columns is accurately known). This modeling and linear correlation approach is used on Figure 6.5 data to predict the actual diameter of the potentially variable diameter jet grout columns (Figure 6.7a).

$\rho_a$ responses for column 2 are plotted in Figure 6.6. The variation in $\rho_a$ response with depth is significant compared to column 1. For the $a = 0.6m$ response, column 1 data varies from 6-6.5Ωm over the column interval. In the column 2 $a = 0.6m$ response, $\rho_a$ varies from 5.5Ωm ($z = 7m$) to around 10Ωm ($z = 5m$). This larger variation and general trend with depth are observed for all three values of $a$. These responses suggest that column 2 does not have a uniform diameter with depth. The reduced diameter from $z = 3$-6.5m is supported by the 25% reduction in grouting pressure over this interval (Figure 6.3b). Galindo-Guerreros et al. (2015b) also predict a reduced column diameter over the $z = 3$-6.5m depth interval from crosshole seismic testing.

![Figure 6.5: Experimental $\rho_a$ responses with FE-predicted $\rho_a$ responses for column 1 with $a$ values of a) 0.6m, b) 0.9m, and c) 1.2m.](image)
To further assess the observed changes in $\rho_a$, additional FE modeling is performed with several constant and variable diameter columns. Uniform diameter columns were modeled for each electrode spacing over a diameter range of $D = 0.8$-1.3m with 0.1m intervals (grey lines, Figure 6.6). In all three cases (Figure 6.6a-c) no uniform diameter column model is able to capture the behavior observed in the experimental data. For this reason, the diameter from $z = 3$-6.5m is varied. $D$ is assumed to be 1.25m from $z = 6.5$-10m given the same soil conditions and grouting parameters to column 1 over this depth interval. Using observed similarities between constant diameter responses at various depths, column 2 is modeled with $D_1 = 0.9$m ($z = 3$-6.5m) and $D_2 = 1.25$m ($z = 6.5$-10m) and is plotted as the solid line $\rho_a(D)$ in Figure 6.6. The model geometry/resistivity profile used to obtain the $\rho_a(D)$ response in Figure 6.6 is shown in Figure 6.3b. $D = \pm 10\% D$ dashed line envelopes are plotted with $\pm 10\% D_1$ from $z = 3$-6.5m and $\pm 10\% D_2$ from $z = 6.5$-10m. The experimental $\rho_a$ data points are mostly contained within the $\pm 10\% D$ envelopes (with the exception of a few outliers, e.g., $z = 4.2$-4.6m in Figure 6.6c). The sensitivity to diameter changes is notably reduced around the region of diameter change ($z = 6.5$m), but the predicted diameter change is significant (28% diameter reduction). Given the generally good fit between the experimental data and the $\pm 10\% D$ envelopes, the same linear correlation approach used for column 1 is applied to column 2 (Figure 6.7b).

![Figure 6.6: Experimental $\rho_a$ responses with FE-predicted $\rho_a$ responses (from both constant and variable diameter columns) for column 2 with $a$ values of a) 0.6m, b) 0.9m, and c) 1.2m.](image)
6.6 Conclusions:

Freshly constructed jet grout columns were tested using a DC electrical resistivity push probe, and the results of these tests were analyzed using FE modeling. A linear correlation between modeled diameter and FE $\rho_a$ response is used to estimate column diameter from experimental $\rho_a$ at 0.3m intervals over the evaluated region. The modeling approach described herein has been previously validated on constant diameter DSM columns, but this is the first application to jet grouted columns with a potential for diameter variation. The columns were constructed in ground with heterogeneous/stratified resistivity, which adds further complexities to the interpretation of the measured response via FE modeling. The probe measurements for both columns are generally well-predicted by the modeling, indicating that the FE model is a robust forward modeling tool for push probe DC resistivity testing applied to jet grout columns.

For column 1, modeling suggests a relatively constant diameter of $\bar{D} \approx 1.25$m. This is in good agreement with the contractor’s assessment of 1.2-1.3m. This diameter estimate is further supported by the results of crosshole seismic tests performed by Galindo-Guerreros et al. (2015), where the predicted diameter was 1.25m. Column 2 data is inherently more complex and no
uniform diameter column model can predict the measured $\rho_a$ responses. To estimate column 2 geometry, the experimental results were compared to several uniform and variable diameter column models. The modeling suggests that from $z = 3$-6.5m, where the grouting pressure was reduced to 30MPa, $\bar{D} \approx 0.8$-1m. Below 6.5m, $\bar{D} \approx 1.25$m. Galindo-Guerreros et al. (2015) predicted that the diameter from $z = 3$-6.5m was 0.9-1.1m. From $z = 6.5$-7m crosshole seismic results suggest a diameter of 1.25-1.3m. The push-probe predicted diameters for both columns are in generally good agreement with contractor estimates and crosshole seismic results over the depth intervals evaluated.

The results of this study suggest that the electrical push probe is readily implementable on field-constructed jet grout columns. The probe placement, testing, and removal can be performed within a sufficiently small time frame to ensure probe recovery with no lasting column defects. Furthermore, the DC resistivity protocol can capture changes in resistivity behavior that result from jet grout column diameter variation (and can do so in variable resistivity ground). The FE model used in this study can capture the experimental behavior, indicating that it is an adequate forward modeling tool to predict column diameter. Given the general success of this study, future research will seek to develop an inversion routine to work in conjunction with the FE forward model.

6.7 Acknowledgements: The authors would like to thank the BAM Federal Institute for Materials Research for providing the jet grout field site tested in this research. The authors would particularly like to thank BAM collaborators Ernst Niederleithinger and Julio Galindo-Guerreros for their assistance in probe implementation and data acquisition during the field test phase.

6.8 References:


CHAPTER 7:
GENERAL CONCLUSIONS

The research in this thesis seeks to advance the current state of understanding for monitoring the geometrical and engineering properties of soils treated with lime and cement grout using non-destructive geophysical methods. This chapter discusses specific conclusions from each portion of the research, outlines general conclusions related to the overarching implications of the research, and discusses future applications and extensions to the research.

7.1 Specific Conclusions from Each Paper

Paper 1:

Paper 1 develops a new approach for characterizing the time-temperature dependent seismic modulus maturity of lime and lime-cement stabilized subgrade soil. To develop this maturity function, FFR data were collected from LSS/L-CSS cylinders cured at several temperatures to assess $E_0$ growth behavior. Regression analysis of FFR results revealed that $E_0$ growth in LSS/L-CSS should be characterized as a non-linear maturity function of both time and temperature. The maturity function developed in this research (Eq. 2.12) exhibits power model behavior with curing time, exponential behavior with curing temperature, and also depends on three constant empirical parameters $\eta_0$, $\alpha'$, and $\beta$. The maturity function adequately captures $E_0$ growth as a function of time and temperature for both constant and variable field curing temperatures (Figures 2.9 and 2.10). Given the similarities in soil properties and mix designs (i.e., all three test site soils were A-7-6/CH with 5.0-5.5% lime and 0-3% cement), it is not unreasonable to expect that variation in soil type or mix design could result in different best fit values for $\eta_0$, $\alpha'$, and $\beta$; however, the maturity function should adequately describe the $E_0$ growth behavior given appropriate empirical parameter values.

Inspection of data between LSS and L-CSS sites suggests that the addition of cement powder with quicklime induces somewhat different behavior. Primarily, it appears that soil stabilized with lime undergoes more gradual $E_0$ growth than L-CSS. The LSS also achieves a higher peak $E_0$ than the L-CSS studied, but this difference is potentially related to variation in soil type and not a direct result of cement powder addition. The data scatter and similar range of modulus values among these sites make it difficult to accurately decouple the individual effects of lime and cement. The inherent variability associated with field soils and LSS/L-CSS

135
application/production further increases data scatter, but any field site would be subject to this variability.

This paper also validates the use of field-cured specimens to mimic the curing temperature regimes (and thus $E_0$ growth) experienced by the field-constructed soil. Mix design studies, which are frequently conducted prior to field-scale lime/cement treatment, could include FFR $E_0$ growth characterization and application of the maturity function with best fit $\eta_0$, $\alpha'$, and $\beta$ for a given soil and mix design. $E_0$ growth for applications of the same soil and mix design could be more reliably predicted given $T$, and $t$ of the field conditions, but additional study and application to field data would be necessary to fully verify this conclusion.

**Paper 2:**

Paper 2 outlines the results of a laboratory study to explore the spatial evolution of $V_P$ in simulated soilcrete columns. The results of this study demonstrate the use of CSL and 2D acoustic tomography for characterizing soilcrete geometry and quality for the 2D cross section of the column being evaluated. This method allows for the identification of high and low quality soilcrete, and characterizes the increase in soilcrete $V_P$ that results from curing. For high quality soilcrete, $V_P$ ranges from 1200-1400 m/s at early curing times (20-24 hours) and 3000-3200 m/s at late curing times (96-120 hours). For weak soilcrete, $V_P$ ranges from 900-1500 m/s after approximately 24 hours of curing, but may not undergo any significant $V_P$ gain at later curing times. Furthermore, 2D acoustic tomography can identify soil inclusions in the soilcrete column.

While the CSL system may not be applicable to a field scale setup (i.e., the frequency is too high and the source energy is too low to characterize a full size column), it provides a high resolution laboratory proof of concept supporting the use of acoustic tomography (and acoustic methods in general) for soilcrete column QC/QA. A more desirable field setup would allow measurement of jet grout quality even if no contact exists between the column and measurement array casing (i.e., an approach that could measure through in-situ soil and soilcrete). These results have led to research extensions performed in collaboration with the BAM Federal Institute for Materials Research and Testing in Berlin, Germany. Based on this laboratory proof of concept, BAM co-author Dr. Ernst Niederleithinger, along with industry partner Geotomographie, developed a new source/receiver setup to perform crosshole seismic testing on field scale jet grout columns. This system was evaluated on the same jet grouted columns studied.
in Paper 5 with the DC resistivity push probe developed in this research; however, the crosshole seismic tests were conducted after full curing (7-30 days) and thus do not provide an immediate assessment of diameter.

**Paper 3:**

Paper 3 presents the results of a laboratory study used as a proof of concept for the development and implementation of the DC resistivity push probe used in Papers 4 and 5. Borehole DC resistivity tests were performed on laboratory scale soilcrete columns to assess the usability of the Wenner-α protocol with direct coupling of electrodes to improve column geometry estimation. The protocol was tested on two array configurations: electrodes in direct contact with the soilcrete and electrodes in a slotted, water-filled casing encased in soilcrete. FE models of the soil tank and specimens/arrays were constructed and compared with experimental data to assess the capability of the model as a tool for column geometry prediction.

The study revealed that the Wenner-α array with direct coupled electrodes provides several advantages over the slotted casing configuration. With direct coupled electrodes, the current is injected directly into the soilcrete, resulting in a higher current density in the soilcrete compared to the slotted casing approach. In the slotted casing configuration, considerable current remains in the column fluid, e.g., 40% per FE analysis. To this end, the direct coupled electrodes provide significantly better geometric resolution than the slotted casing array using same Wenner-α protocol. This conclusion is evidenced in both experimental and FE results.

\( \rho_a \) measurements from the direct coupled data are compared to FE \( \rho_a \) predictions to estimate column diameter via a linear correlation between \( D \) and \( \rho_a \). For \( D = 30 \text{cm} \) regions, exhumed specimen 1 \( D = 29.8 \pm 1.7 \text{cm} \) and average \( \bar{D} = 30.2 \pm 1.5 \text{cm} \). In the \( D = 18 \text{cm} \) region, exhumed specimen 1 \( D = 17.8 \pm 1.2 \text{cm} \) and \( \bar{D} = 17.9 \pm 0.6 \text{cm} \). The uncertainties in \( \bar{D} \) correspond to \( \pm 5\%D \) in the \( D = 30 \text{cm} \) regions and \( \pm 3\%D \) in the \( D = 18 \text{cm} \) region. There is no appreciable difference in \( \bar{D} \) accuracy when transitioning between different column diameters; however, when making the more drastic transition from soil to soilcrete at the array interface (i.e., the top and bottom of the column), the method loses its sensitivity to diameter change.

An analysis of experimental results and FE modeling reveals an important relationship between electrode spacing \( a \) and column diameter \( D \) using the direct coupling configuration. If \( D/a \geq 10 \), the measured \( \rho_a \) will be influenced by the soilcrete only. \( \rho_a \) measurements where \( D/a \)
≤ 5 are influenced by \( \rho_{sc} \) and \( \rho_s \). \( D/\alpha < 5 \) \( \rho_a \) is sensitive to the soilcrete and the soil, indicating that \( D/\alpha < 5 \) measurements are best-suited for characterizing column geometry. In general, the lower the value of \( D/\alpha \), the more sensitive the measurement will be to changes in column geometry. This conclusion can be applied to field jet grout construction. Direct couple electrode configurations can be implemented with push-probe technology. The normalized observations with \( D/\alpha \) observed in this study can be readily extended to field scale \( D/\alpha \) values (e.g., a 3m diameter column and an array with \( a_{min} = 30 \text{cm} \)).

Time lapse Wenner-\( \alpha \) data (using direct coupled electrodes) suggest that \( \rho_{sc} \) values range from 1.6Ωm (1.5 hours) to 8.5Ωm (10 days), resulting in a significant reduction in the resistivity contrast between the soil and soilcrete as curing time increases. Furthermore, measurement uncertainty increases significantly with curing time (\( \sigma = 0.1\Omega \text{m after } t = 1.5 \text{ hours}, \sigma = 1.6\Omega \text{m after } t = 10 \text{ days} \)). While the exact temporal variation in soilcrete resistivity would be mix-dependent, this result indicates that soilcrete resistivity testing should be performed as early as possibly to maximize the resistivity contrast between the soilcrete and in-situ soil and minimize the uncertainty in \( \rho_{sc} \). This is well-suited for field conditions where testing immediately after jet grouting is ideal.

**Paper 4:**

In Paper 4, the laboratory proof of concept study in Paper 3 is extended to field-scale testing. A parametric study was conducted using FE modeling of the DC electrical resistivity test in field-scale soilcrete columns to inform the design of a push probe for field implementation. The resulting 6m probe had twenty ring electrodes (10cm diameter) with a minimum spacing of 0.3m. The probe utilized electrode spacings of 0.3, 0.6, 0.9, and 1.2m. These spacings were carefully selected from modeling results for their ability to provide both an in-situ estimate of soilcrete resistivity and an estimate of column geometry with a minimum number of necessary measurements over a range of column diameters. The push probe was tested on five DSM columns at two active construction sites over a variety of column diameters and soil/groundwater conditions. Trends in experimental results were analyzed, compared to the observations from the parametric study, and found to be in good agreement. The development of this parametric study and subsequent validation with field data advances the understanding of DC resistivity testing on soilcrete columns. The parametric study is a useful guide for selecting appropriate electrode
spacings for DC resistivity push probe testing. Background measurements to estimate a soil resistivity profile for a particular site can be obtained prior to column construction. In conjunction with designed column diameter, the parametric study can be used to design the ideal measurement protocol on a site-specific basis.

FE modeling was performed with columns of diameters ±10% D and used to estimate the diameter of tested columns in conjunction with experimental data. Using \( \rho_{sc} \) inputs obtained from \( a = 0.3 \text{m} \) data and \( \rho_s \) values obtained from background electrical profiling, the push probe modeling approach is able to estimate the column diameter to within ±5% D (Figure 5.12). The probe provides an additional measurement advantage in its ability to characterize \( \rho_{sc} \) throughout the column. Because \( \rho_{sc} \) can vary within an individual column (e.g., Figure 5.6b) a grab sample measurement from a single depth may not be indicative of \( \rho_{sc} \) for the entire column. Because an accurate estimate of \( \rho_{sc} \) is critically important in obtaining an accurate FE model (and thus diameter estimate), the in-situ \( \rho_{sc} \) measurement provided by the probe is an improvement over the existing grab sample practice.

This study highlights a promising new technology for diameter estimation of soilcrete columns. The test is relatively quick (completed within an hour of column construction) and truly non-destructive as the probe is removed while the soilcrete is still wet/fresh and causes no lasting defects. To this end, the probe is ideal for immediate geometric assessment of production columns (which is an improvement over any existing technique). Extensions of this research will seek to evaluate additional columns to further assess the probe’s diameter prediction capability. This study validates the probe’s diameter estimation capability on uniform diameter DSM columns with precisely constructed geometry and indicates that the probe can estimate diameter to within ±5% D. Future testing will be extended to jet grouted columns where diameter variation is possible. In addition, a data inversion routine will be developed (effort underway) to better quantify diameter variations in variable diameter columns, where measured data responses are inherently more complex.

**Paper 5:**

Paper 5 uses the electrical push probe developed in Paper 4 to assess the diameter of two freshly constructed jet grout columns. The results of these tests were analyzed using FE modeling and diameter was estimated using the same linear correlation approach as Papers 3 and
4 with $\rho_{sc}$ profiles obtained from the $a = 0.3$m data. The modeling approach described herein has been previously validated on constant diameter DSM columns, but this is the first application to jet grouted columns with a potential for diameter variation. The columns were constructed in ground with heterogeneous/stratified resistivity, which adds further complexities to the interpretation of the measured response via FE modeling. The probe measurements for both columns are generally well-predicted by the modeling, indicating that the FE model is a robust forward modeling tool for push probe DC resistivity testing applied to jet grout columns.

For column 1, modeling suggests a relatively constant diameter of $\bar{D} \approx 1.25$m. This is in good agreement with the contractor’s assessment of 1.2-1.3m. This diameter estimate is further supported by the results of crosshole seismic tests performed by Galindo-Guerreros et al. (2015), where the predicted diameter was 1.25m. Column 2 data is inherently more complex and no uniform diameter column model can predict the measured $\rho_a$ responses. To estimate column 2 geometry, the experimental results were compared to several uniform and variable diameter column models. The modeling suggests that from $z = 3$-6.5m, where the grouting pressure was reduced to 30MPa, $\bar{D} \approx 0.8$-1m. Below 6.5m, $\bar{D} \approx 1.25$m. Galindo-Guerreros et al. (2015) predicted that the diameter from $z = 3$-6.5m was 0.9-1.1m. From $z = 6.5$-7m crosshole seismic results suggest a diameter of 1.25-1.3m. The push-probe predicted diameters for both columns are in generally good agreement with contractor estimates and crosshole seismic results over the depth intervals evaluated.

The results of this study suggest that the electrical push probe is readily implementable on field-constructed jet grout columns. The probe placement, testing, and removal can be performed within a sufficiently small time frame to ensure probe recovery with no lasting column defects. Furthermore, the DC resistivity protocol can capture changes in resistivity behavior that result from jet grout column diameter variation (and can do so in variable resistivity ground). The crosshole seismic approach informed by the results of Paper 2 were used in conjunction with the electrical push probe. Push probe diameter predictions are in good agreement with crosshole seismic diameter predictions. Both of these approaches advance the current state of understanding for geometry assessment on jet grout columns. Furthermore, this research validates the ability of the FE model to capture the experimental behavior in variable diameter columns constructed in stratified resistivity ground, indicating that it is an adequate forward modeling tool to predict column diameter. Given the general success of this study, future
research will seek to develop an inversion routine to work in conjunction with the FE forward model.

7.2 General Conclusions

The specific conclusions from each portion of this study can be generalized to more broadly address the primary goals of the research. This thesis seeks to identify the best-suited geophysical techniques to assess both the engineering properties and geometry of soil volumes modified with lime and cement grout. Further, this thesis seeks to advance the state of understanding regarding both the implementation of geophysical techniques and the understanding/interpretation of data from said techniques as applied to modified soil testing. Several geophysical techniques were used in this research, and these techniques have inherent strengths and weaknesses for evaluating various properties in various types of modified soil.

The FFR technique is well-suited for characterizing the growth in the engineering properties (strength/stiffness) of soils stabilized with lime and cement grout. In the case of stabilized subgrade, soil is compacted into a cylindrical specimen and can be tested immediately after compaction. Modulus growth is measurable from inception to the time at which the growth ceases due to reactant consumption (up to 60+ days as evidenced by the research herein). For cement-grouted soils, soilcrete is cast into cylindrical molds in slurry form and cured for approximately 24 hours before having sufficient solidity for testing. The specimen must be able to hold its form without the support of a mold to satisfy the boundary conditions of the FFR test. Furthermore, wet soilcrete is highly attenuative and both FFR and CSL have difficulty propagating waves through wet soilcrete prior to 24 hours of curing. Once sufficient solidity is reached (either via immediate compaction or cast in place curing), the FFR method is well-suited for monitoring strength/stiffness growth. Because the FFR test requires the specimen geometry as an input to estimate strength/stiffness, the test is unsuitable for geometric assessment.

CSL can estimate the compressional wave velocity of curing soilcrete, but only when the casings are in contact with the grouted structure. In a field application, this constraint would require known geometry of the column to ensure proper casing placement, thus making it unsuitable for geometric assessment. The frequencies used in traditional CSL (transceiver center frequency of 45kHz) are well suited for \( V_p \) estimation in field scale concrete structures such as...
drilled piles, but the addition of a soil layer on either side of the soilcrete volume results in significant attenuation. This issue has been overcome by propagating waves of lower frequencies to reduce attenuation, and this topic is an active area of research (Mackens 2015, Galindo-Guerreros et al. 2015a,b). Crosshole seismic testing was utilized on the same jet grout columns in Paper 5 and the column diameter predictions between the crosshole seismic and DC resistivity were found to be in generally good agreement. Although the crosshole seismic testing results are in good agreement with push probe results, these tests were conducted after 7-30 days of curing. The crosshole seismic approach provides a good estimate of geometry (with the inability to excavate the columns for verification), but does not provide the results with the same immediacy as the push probe (which can provide results within 1 hour of column construction).

DC electrical resistivity excels at identifying the geometry of cement grouted soil volumes by identifying the large resistivity contrast between the grouted and un-grouted soil. The largest resistivity contrast occurs earliest in the curing process when cement grouted soils are still in slurry form. Direct coupling of electrodes to soilcrete for DC resistivity testing was extensively evaluated in this research, and results indicate it can provide significant measurement advantages over indirect electrode coupling via a fluid-filled slotted casing. This direct coupling approach was used in the development of an electrical push probe that can characterize soilcrete column geometry. A recoverable probe with direct coupled electrodes facilitates rapid non-destructive testing of production columns, and is a significant advancement over any approach currently utilized. DC resistivity was not evaluated on lime/cement stabilized subgrade, but given the similarity in chemical reactions occurring in both lime/cement treated subgrade and cement grouted soil, it is plausible that DC resistivity could be used to estimate the thickness of the stabilized soil layer immediately after construction by identifying the contrast between treated and untreated soil. DC resistivity is not well-suited for assessment of strength/stiffness of soilcrete.

7.3 Recommendations for Future Work

The seismic modulus maturity function developed in Paper 1 is an improved methodology for characterizing time/temperature depending strength/stiffness growth in lime/cement stabilized soil. Application of this maturity function requires the determination of
three empirical parameters via regression fitting. For the sites evaluated, the soil/additive contents were relatively similar. Seismic modulus growth monitoring on specimens constructed with a greater variety of soil types and mix designs could further validate the maturity function.

Paper 2 provides the laboratory framework for extending crosshole wave propagation testing to field-scale jet grout columns. Field scale testing using this approach has already been implemented in conjunction with electrical push probe testing. The two methods are in generally good agreement regarding diameter prediction, but both of these techniques would benefit from additional jet grout column testing with a greater variety of column diameters and soil conditions.

The research in Papers 3-5 support the use of direct coupled electrode DC resistivity testing to determine geometry of cement grouted volumes. The probe developed and implemented in this research is a first prototype and had inherent limitations. Efforts are currently underway to develop a second generation push probe that will utilize more electrode spacings and variations in electrical protocol. Furthermore, the probe will be of smaller diameter with fewer air voids to overcome the buoyancy issue encountered when placing the probe at depths greater than 8m. A robust forward model is developed via COMSOL Multiphysics® that is able to capture the behavior of experimental data responses in both laboratory and field environments. The model exhibits sensitivity to diameter changes related to column diameter and electrode spacing, as extensively studied in this research. To improve the diameter estimation of this approach, especially in jet grout columns where diameter is not known, an inversion routine can be used to better interpret the experimental data in conjunction with the forward model. Development of this full inversion code is currently underway.
APPENDIX A:
ADDITIONAL CONSIDERATIONS FOR DC RESISTIVITY MODELING

The COMSOL finite element model used to simulate the DC resistivity tests (for both laboratory and field scale simulations) injects current used a volumetric current source (A/m$^3$). The model distributes this current source over a known ring volume to obtain the desired injection current $i_{AB}$ (A). Figure A-1 shows a model rendition of a ring electrode volume over which the source current is distributed. The volume of the ring is calculated using the ring’s geometry as shown in Equation A-1.

$$V = \pi h (r_2^2 - r_1^2) = 0.01\pi(0.043 - 0.038) = 1.27 \cdot 10^{-5} \, m^3$$ \hspace{1cm} (A-1)

To obtain the desired injection current (example shown with 5mA), the current (in A) is divided by the volume of the ring to obtain the required volumetric current in A/m$^3$ (Equation A-2).

$$i_{AB} = 0.005A \rightarrow \frac{0.005A}{1.27 \cdot 10^{-5} \, m^3} = 392.8A/m^3$$ \hspace{1cm} (A-2)

Figure A-1: Illustration of the volumetric current injection applied to push probe ring electrodes.
The geometric correction factors $k$ used for the push probe in Papers 4 and 5 depend on the ring geometry, electrode spacing, and depth from the surface. These $k$ factors for all evaluated push probe depths and electrode spacings are shown in Figure A-2.

![Figure A-2: Geometric correction factors used for the push probe.](image)

Another important consideration for push probe testing is to characterize the effect of probe offset (i.e., if the push probe is not perfectly centered in the soilcrete column). Because the probe provides an axisymmetric average of the volume around the electrodes, the effects of a minor offset (10% column diameter) are minimal. This change in response due to offset is illustrated in Figure A-3 using Site 1 Column 1 as an example. The change in response resulting from 10% offset is shown as the dotted line response $\rho_{a\text{(offset)}}$ for $a = 0.6, 0.9, \text{ and } 1.2\text{m}$. Because the probe provides an axisymmetric measurement, this response is representative of a probe shift in any radial direction from the center. This consideration is important for field testing, where placing the probe precisely in the center of soilcrete columns can be difficult; however, the modeling results suggest that the measurement obtained with a slight probe offset will not be significantly different from those obtained from a perfectly centered probe.
Figure A-3: Illustration of probe offset effects.
APPENDIX B: CO-AUTHOR AND PUBLISHER PERMISSIONS

Chapter 2 is modified from the *ASCE Journal of Materials in Civil Engineering* paper
“A SEISMIC MODULUS MATURITY FUNCTION FOR LIME AND LIME-CEMENT STABILIZED CLAY”
Co-author Dr. Michael Mooney was my faculty advisor for this paper

Chapter 3 is modified from the ASCE GeoCongress 2014 conference paper
“CHARACTERIZATION OF SIMULATED SOILCRETE COLUMN CURING USING ACOUSTIC TOMOGRAPHY”
Co-author Dr. Michael Mooney was my faculty advisor for this paper, and co-author Dr. Andre Revil was a member of my Ph.D. committee. Co-author Dr. Ernst Niederleithinger approves the publication of this paper in my dissertation as indicated by his signature below.

Signed: [Signature]
Dr. Ernst Niederleithinger

Date: 2015-08-30

Chapter 4 is modified from the *ASCE Journal of Geotechnical and Geoenvironmental Engineering* paper
“ELECTRICAL RESISTIVITY IMAGING OF LABORATORY SOILCRETE COLUMN GEOMETRY”
Co-author Dr. Michael Mooney was my faculty advisor for this paper.

Chapter 5 is modified from a paper to be submitted to the *ASCE Journal of Geotechnical and Geoenvironmental Engineering*
“DEVELOPMENT OF AN ELECTRICAL RESISTIVITY PUSH PROBE TO ESTIMATE THE DIAMETER OF JET GROUTED COLUMNS”
Co-author Dr. Michael Mooney was my faculty advisor for this paper.

Chapter 6 is modified from a paper to be submitted to the Near Surface Geophysics
“ESTIMATION OF JET GROUT COLUMN GEOMETRY USING A DC ELECTRICAL RESISTIVITY PUSH PROBE”
Co-author Dr. Michael Mooney was my faculty advisor for this paper.

Co-author Dr. Pauline Kessouri approves the publication of the papers in Chapters 4-6 in my dissertation as indicated by her signature below.

Signed: [Signature]
Dr. Pauline Kessouri

Date: 08/25/15
Papers 1, 2, and 3 are already published or have been accepted for publication in ASCE conferences or journals. Publisher permissions for each paper are shown in the text below.

**Papers 1 and 3 have been accepted for publication but are not yet in specific journal volumes/issues. Permission to publish these papers in this dissertation was obtained by directly contacting the permissions department of ASCE. ASCE’s response (from Product and Subscription Services Manager Joann Fogleson) is shown below.**

Dear Richard Bearce:

Permission is granted for you to reuse your ASCE papers as stated in the email below. A full credit line must be added to the material being reprinted. For reuse in non-ASCE publications, add the words "With permission from ASCE" to your source citation. For Intranet posting, add the following additional notice: "This material may be downloaded for personal use only. Any other use requires prior permission of the American Society of Civil Engineers."

Manuscript #: GTENG-4720R1  
Title: Electrical Resistivity Imaging of Laboratory Soilcrete Column Geometry  
Authors: Richard Bearce, M.S.; Michael Mooney, Ph.D., P.E.; Pauline Kessouri, Ph.D.  
Publication: Journal of Geotechnical and Geoenvironmental Engineering

Manuscript #: MTENG-3366R2  
Title: A Seismic Modulus Maturity Function for Lime and Lime-Cement Stabilized Clay  
Authors: Richard Bearce, M.S.; Michael Mooney, Ph.D., P.E.  
Publication: Journal of Materials in Civil Engineering

Regards,

Joann

Joann Fogleson  
Manager, Product and Subscription Services  
American Society of Civil Engineers  
1801 Alexander Bell Drive  
Reston, VA 20191

PERMISSIONS@asce.org

703-295-6112

E-mail: jfogleson@asce.org  

A full credit line must be added to the material being reprinted. For reuse in non-ASCE publications, add the words "With permission from ASCE" to your source citation. For Intranet posting, add the following additional notice: "This material may be downloaded for personal use only. Any other use requires prior permission of the American Society of Civil Engineers."
Paper 2 is already published and permissions for publication in this dissertation were obtained using ASCE’s automated copyright clearance tool RightsLink.

**Permissions Request**

As an ASCE author, you are permitted to reuse you own content for another ASCE or non-ASCE publication.

Please add the full credit line "With permission from ASCE" to your source citation. Please print this page for your records.

**Type of use:** Dissertation/Thesis

**Portion:** full article

**Format:** print and electronic

**Use of this content will make up more than 25% of the new work:** no

**Author of this ASCE work or ASCE will publish the new work:** yes