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16. ABSTRACT

The purpose of this document is to summarize the in situ physical properties of soil and rock that can be measured by different geophysical methods. These methods can provide crucial subsurface information for geotechnical design of transportation infrastructure on a more comprehensive scale than typical subsurface investigation techniques. This broader scale often allows greater insight regarding highly variable subsurface soil conditions and can result in reduced risk and uncertainty. This document attempts to fill in a gap in the literature by providing a comprehensive reference for geotechnical engineers with an introductory knowledge of geophysics. The document emphasizes the measurements obtained using common geophysical techniques (e.g., seismic methods, ground penetrating radar, electrical resistivity, etc.) and the relationships between these measurements and soil and rock properties. Guidance is provided regarding selection of geophysical methods for particular applications, physical scales involved in each method, limitations in measuring a particular physical property, and uncertainty in the geophysical measurements. The reader is assumed to have familiarity with the geophysical principles, equipment, and data acquisition procedures appropriate for each method. For those readers who desire background information on each of the techniques, an appendix is provided as well as citations to several references that discuss this information in much greater detail. The ultimate goal of this document is to help geotechnical engineers with an entry-level understanding of geophysics make better-informed decisions regarding the use of geophysical methods for geotechnical engineering and transportation infrastructure projects.

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GEOPHYSICAL METHODS FOR DETERMINING THE GEOTECHNICAL ENGINEERING PROPERTIES OF EARTH MATERIALS









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Prepared By:

Principal Investigator: Dr. Joseph T. Coe, Jr. Assistant Professor, Temple University

Co-Principal Investigator: Dr. Scott J. Brandenberg Associate Professor, University of California Los Angeles

Sean K. Ahdi Ph.D. Candidate, University of California, Los Angeles

> Alireza Kordjazi Ph.D. Candidate, Temple University

EXECUTIVE SUMMARY

The purpose of this document is to summarize the in situ physical properties of soil and rock that can be measured by different geophysical methods. These methods can provide crucial subsurface information for geotechnical design of transportation infrastructure on a more comprehensive scale than typical subsurface investigation techniques. This broader scale often allows greater insight regarding highly variable subsurface soil conditions and can result in reduced risk and uncertainty. Appreciation of this fact has led to growing interest in the value of geophysical measurements within the geotechnical profession.

Despite growing interest in geophysical methods, these methods are underutilized for transportation infrastructure. A major contributing factor is that literature on geophysical methods tends to be concentrated at either end of the novice-expert spectrum. References are either introductory in nature and intended for novice geophysical users or rather advanced and intended for geophysical specialists in various fields. This document attempts to fill in this gap in the literature by providing a comprehensive reference for geotechnical engineers with an introductory knowledge of geophysics. The document emphasizes the measurements obtained using common geophysical techniques (e.g., seismic methods, ground penetrating radar, electrical resistivity, etc.) and the relationships between these measurements and soil and rock properties. Guidance is provided regarding selection of geophysical methods for particular applications, physical scales involved in each method, limitations in measuring a particular physical property, and uncertainty in the geophysical measurements. The reader is assumed to have familiarity with the geophysical principles, equipment, and data acquisition procedures appropriate for each method. For those readers who desire background information on each of the techniques, an appendix is provided as well as citations to several references that discuss this information in much greater detail. The ultimate goal of this document is to help geotechnical engineers with an entry-level understanding of geophysics make better-informed decisions regarding the use of geophysical methods for geotechnical engineering and transportation infrastructure projects.

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1. INTRODUCTION

This chapter describes the background regarding subsurface exploration in geotechnical engineering and the role of geophysics in such explorations. This discussion provides the rationale for the research performed in this study. The scope of this document is discussed based on the goals of the research. Finally, a summary of the report is provided.

1.1 BACKGROUND

Efficient design of transportation projects requires a thorough understanding of the subsurface. Unfortunately, the characterization of subsurface properties and geometry remains one of the biggest issues in geotechnical engineering. The majority of subsurface investigations rely on sampling subsurface soils for laboratory testing and on correlations from in situ testing techniques such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT), among others. Such methods have proven capable of providing high-quality information regarding the subsurface, though the extent of site characterization is often constrained by the associated costs and the limited volume of material actually tested within the site of interest. Therefore, the current state of practice for subsurface investigations can provide an inadequate amount of information regarding the subsurface to develop reliable, efficient geotechnical designs.

Geologists have faced a similar issue of proper characterization of the subsurface, although typically at a larger scale than in geotechnical engineering. Applications related to exploration for natural resources (i.e., petroleum, gas, etc.) have spurred tremendous advances in geophysical methods during the turn of the last century, especially as the use of computers has proliferated (Telford et al. 1990). Geophysics involves the measurement of properties of earth materials based on principles of physics (e.g., seismic wave propagation, electromagnetism, etc.). Geophysical methods detect discontinuities in material properties and allow determination of the nature and distribution of materials beneath the surface (Wightman et al. 2003). Geophysical methods are now routinely the initial mode of testing for the exploration of petroleum. Successful implementation of geophysical methods provides the necessary information to guide drilling operations for such exploration in the petroleum industry (Sirles 2006). Additionally, geophysics is routinely utilized in a wide range of geologic studies, including the delineation of faults (Ivanov et al. 2006; Blakely et al. 2002; Bleibinhaus et al. 2007),

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characterization of sub-bottom stratigraphy in streams and the ocean (Nielsen et al. 2005; Rebescoa et al. 2011; Pinson et al. 2008), location of karst features (e.g., sink-holes) (Hackert and Parra 2003; Nyquist et al. 2007; Legchenko et al. 2008), and evaluation of aquifers (Harry et al. 2005; Francese et al. 2002; Bradford and Sawyer 2002), among others applications. Geophysical methods have proven to be an effective tool for geologists to better understand the inner workings of our planet.

1.2 MOTIVATION FOR RESEARCH

In addition to geologic applications, geophysical methods can measure in situ properties of soil and rock that are often valuable for geotechnical design of transportation infrastructure. This is especially true for seismic design purposes where shear wave velocity/modulus and material damping are input parameters for site class, estimation of ground response, and seismic hazard analysis. Geophysical measurements are also distributed over a larger area than typical geotechnical site investigations and can therefore provide a higher level of detail regarding site conditions for a project. As such, the application of geophysical methods has demonstrated cost savings through reduced design uncertainty and lower investigation costs. Routine use of geophysical methods, however, remains limited due to the specialized nature of the work and limited industry experience with its application to real-world projects. Literature on the topic tends to either be qualitative and introductory, intended for readers with little knowledge of geophysical methods, or rather advanced and complex, intended for geophysicists with a thorough understanding of geophysical techniques and the measurements they provide.

1.2.1 ENGINEERING APPLICATION OF GEOPHYSICAL METHODS

As previously noted, geophysical methods have been routinely utilized to explore the subsurface as part of geologic investigations. In many cases, there is significant overlap between applications of these methods for geological purposes and for engineering purposes. This document aims to explore this overlap in more detail and provide guidance regarding measurements of earth material properties using geophysical techniques, particularly as relevant for transportation infrastructure.

Geophysical methods are essentially measuring the same parameters when applied to engineering investigations and geologic studies. Often, the main difference between these applications is a question of scale. Engineers are often preoccupied with the near surface (i.e., upper tens of meters), which is the outermost part of the earth's crust that interacts with our built environment the most (Butler 2005). Moreover, the spatial scale with which engineers are interested is often smaller given the modest size of sites associated with even the largest engineering projects (at least in relation to regional or planetary spatial scales). Traditional geological studies, particularly as related to oil exploration and understanding the inner workings and history of our planet are often interested in deeper strata (e.g., hundreds of meters and more) over a broader spatial coverage (e.g., across a geologic region such as an entire city, state, country, etc.). Given these differences, there have been a number of unique challenges associated with adoption of various geophysical methods for engineering purposes. These challenges have spurred extensive research and the marriage of near surface geophysics with engineering has allowed tremendous advances in both fields, including better understanding of complicated site conditions and the development of specialized techniques that focus on the near surface [e.g., Multichannel Analysis of Surface Waves (MASW)].

There are a significant number of engineering and environmental applications where geophysical methods have proven extremely beneficial both domestically and abroad. The literature is filled with case studies where geophysical methods have been successfully applied to map groundwater contamination/salinity (Fitterman and Deszcz-Pan 1998; Ackman 2003; Zelt et al. 2006; Siemon et al. 2009; Metwaly et al. 2014), evaluate conditions on natural and engineered structures such as dams, slopes, levees, and landfills (Nakazato and Konishi 2005; Hodges et al. 2007; Amine et al. 2009; Pfaffhuber et al. 2010; Inazaki et al. 2011; Doll et al. 2012b, Suto 2013; Hayashi et al. 2014; Konstantaki et al. 2015), locate buried objects (Takata et al. 2001; Hanafy and Gamal 2005; Porsani and Sauck 2007; Omolaiye and Ayolabi 2010; Doll et al. 2012a), evaluate seismicity and seismic hazards (Hardesty et al. 2010; Cox et al. 2011; Hayashi et al. 2013; Khan et al. 2013; Stephenson et al. 2013), and monitor karst bedrock conditions such as sinkholes (Hackert and Parra 2003; Nyquist et al. 2007; Legchenko et al. 2008), among a wide range of other engineering and environmental applications. In each of these cases, geophysics improved assessment of the desired features and allowed increased confidence in design, construction, and/or remediation strategies. Given these potential improvements, the prevalence of geophysics has increased in engineering and environmental investigations. Table 1.1 as adapted from ASTM 6429 provides a general overview of various

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engineering and environmental applications for a number of common geophysical methods. Several of these applications are related to situations encountered during the design, construction, and management of transportation infrastructure. For example, stratigraphic identification of unconsolidated sediments, determination of depth to bedrock and water table, and location of voids, sinkholes, and utilities are all likely necessary steps in typical transportation projects. Other applications as listed in Table 1.1 are less applicable in those regards (e.g., location of inorganic contaminants in landfills) but still very useful for other engineering and environmental purposes.

						ELECTROMAGNETIC			OTHER TECHNIQUES				
	SEIS	SMIC		ELECTRICAL		Frequency	Time		Pipe/Cable	Metal			
Application	Refraction	Reflection	D.C.	I.P./Complex	S.P.	Domain	Domain	VLF	Locator	Detector	Radar	Magnetics	Gravity
Natural Conditions													
Soil/Unconsolidated Strata	A	В	A	-	-	В	A	в	-	-	А	-	-
Rock Strata	В	A	-	-	-	-	-	-	-	-	В	-	-
Depth to Bedrock	A	A	В	-	-	В	В	В	-	-	A	-	В
Depth to Water Table	A	A	В	-	-	В	В	в	-	-	Α	-	-
Fractures and Fault Zones	В	В	В	-	-	A	В	Α	-	-	Α	-	В
Voids/Sinkholes	В	-	В	-	-	A	-	-	-	-	Α	-	Α
Soil and Rock Properties	A	-	A	-	-	A	-	-	-	-	-	-	-
Dam/Lagoon Leakage	-	-	В	-	А	В	-	-	-	-	В	-	-
Inorganic Contaminants													
Landfill	-	-	A	-	-	A	А	в	-	-	В	-	
Saltwater Intrusion	-	-	A	-	-	A	A	в	-	-	в	-	-
Soil Salinity	-	-	Α	-	-	A	-	-	-	-	-	-	-
Organic Contaminants													
Light (Non-Aqueous) Phase Liquids	-	-	в	-	-	в	В	-	-	-	В	-	
Dissolved	-	-	-	-	-	-	-	-	-	-	-	-	-
Dense (Non-Aqueous) Phase Liquids	-	-	-	-	-	-	-	-	-	-	-	-	-
Man Made Buried Objects													
Utilities	-	-	-	-	-	в	-	-	A	В	А	-	-
Drums/USTs	-	-	- I		-	Ā	-	-	A	Ā	A	А	
UXO	-	-		-			-	-	A	В	A	A	
Abandoned Wells	-	-		-		в	-	-	В	В	-	A	
Landfill/Trench Boundaries	в	-	в	-	-	Â	В	-			А	-	
Forensics		-	В	-	-	A	-	-	в	В	A	В	
Archaeological	В	В	В	-	-	A	-	-	-	-	А	А	В



Due to the benefits offered by geophysical methods and the significant overlap between near surface geophysics and engineering, representation of geophysical interests has increased in professional organizations traditionally populated by engineers. For example, the American Society of Civil Engineers (ASCE) Geo-Institute has a Geophysical Engineering technical committee and ASTM's Committee D18 on Soil and Rock has a subcommittee devoted to Geophysics (D.18.01.02). There is also representation of geophysics in committee AFP20 Exploration and Classification of Earth Materials of the Transportation Research Board (TRB). Additionally, there is increasing collaboration between engineers and the near surface

geophysics community through various professional organizations [e.g., Environmental and Engineering Geophysical Society (EEGS), the Near Surface Geophysics Section (NSGS) of the Society of Exploration Geophysicists (SEG), the Near Surface Geophysics Focus Group (NSFG) of the American Geophysical Union (AGU), and the Near Surface Geoscience Division (NSGD) of the European Association of Geoscientists and Engineers (EAGE)]. Together, these committees (and affiliated professional and government organizations) are responsible for a significant amount of literature regarding geophysical subsurface exploration for engineering purposes (e.g., ASTM D4428, D5195, D5753, D5777, D6167, D6274, D6429, D6430, D6431, D6432, D6639, D6726, D6727, D6820, D7046, D7128, G57; Ward 1990; USACE 1995; McCann et al. 1997; Wightman et al. 2003; Butler 2005; Sirles 2006; Anderson et al. 2008; SEGJ 2014). This literature includes standards as well as review documents that provide guidance to engineers about appropriate deployment of various geophysical methods.

The purpose of this document is to expand on the overlap between near surface geophysics and engineering, particularly as related to transportation infrastructure projects. Much like the aforementioned case studies where geophysics have led to improvements in various engineering and environmental applications, near surface geophysical techniques can improve the current state of practice in transportation infrastructure design, construction, and maintenance. As more engineers are exposed to these benefits of geophysics, the goal of this document is to encourage responsible use of geophysics to supplement traditional engineering subsurface investigations. This document will support such improvements by providing a reference suitable for engineers with some experience in geophysics that summarizes the pertinent earth material properties that can be measured using those techniques for transportation projects.

1.2.2 TRANSPORTATION AGENCIES EXPERIENCES WITH GEOPHYSICAL TECHNIQUES

Despite the limited availability of intermediate-level literature focusing on quantitative aspects of geophysics, the role of geophysics in characterizing the subsurface for transportation projects has been increasingly recognized in recent years as more engineers are exposed to near surface geophysical methods. In 2005 the National Cooperative Highway Research Program (NCHRP) sponsored a project in conjunction with the TRB to explore the experiences of various transportation agencies with geophysical methods (Sirles 2006). As part of this project, a questionnaire was sent to representatives from state DOT's, various federal highway agencies, and Canadian transportation agencies. The questionnaire was meant to clarify the role of geophysics in geotechnical investigations for participants in the survey, including: familiarity and level of comfort with geophysical methods; amount and type of investigations performed; typical engineering applications of geophysical methods; annual budgeting and in-house capabilities; common practices for solicitation and contracting of geophysical services; and future research, educational, and training ideas. The response rate was 90%, which corresponded to 67 completed questionnaires, including all 50 state DOT's, the District of Columbia, the Port Authority of New York and New Jersey, 8 Canadian agencies, and 3 federal agencies.

The vast majority of survey respondents (close to 90%) disclosed that they utilized geophysical methods as part of their subsurface investigations (Fig. 1.1). However, as noted in Fig. 1.2, a large percentage (45%) of these agencies has only started implementing geophysical methods within the last 10 years. This statistic, along with corresponding increase in use of geophysical methods – 21% of agencies noting an increase by more than 50% – points to a real need for formal trainings and standards to be developed so that transportation agencies are increasingly comfortable with the appropriate use and limitations associated with geophysical methods. In fact, survey responses noted that the greatest deterrent to use of geophysical methods is related to lack of understanding (Fig. 1.3). That being said, many transportation agencies did recognize the benefits of geophysical methods, with the top responses related to speed of data acquisition, cost-benefits to projects, and better subsurface characterizations. The most common engineering application for geophysical methods was related to mapping subsurface lithology (bedrock and soils), particularly as related to depth, topography, rippability, and other engineering properties (Fig. 1.4). These common applications are to be expected given that the two most common geophysical methods were seismic methods (i.e., refraction and crosshole/downhole techniques) and GPR (Fig. 1.5). Electrical resistivity and borehole logging methods were also fairly prevalent in survey responses. However, it appears that electromagnetic methods were less favored by respondents. It should be noted that a significant percentage of respondents (24%) reported that Nondestructive Testing (NDT) was their most common geophysical application (Fig. 1.4). This highlights the ambiguity between geophysical methods and NDT. Many of the geophysical exploration methods used in practice have been

adapted for use in NDT studies as the underlying theory is often identical. The typical distinction is that geophysical methods apply principles of physics to explore underlying subsurface conditions of the earth and NDT focuses on evaluation of engineered structures (e.g., concrete, asphalt, steel, etc.) (Wightman et al. 2003). The focus of this report will be geophysical applications and the distinction provided by Wightman et al. (2003) will serve as a criterion for distinguishing between geophysics and NDT. However, this report does provide a brief discussion of common NDT applications for transportation infrastructure, the geophysical methods employed, and the properties evaluated.



Figure 1.1: Agency response to survey question regarding use of geophysical methods for subsurface investigations (Sirles 2006).



Figure 1.2: Agency response to survey question regarding initial implementation of geophysical methods (Sirles 2006).



Figure 1.3: Agency response to survey question regarding greatest deterrent to use of geophysical methods (Sirles 2006).



Figure 1.4: Agency response to survey question regarding most common geophysical application (Sirles 2006).





The results from Sirles (2006) illustrate the increasing role that geophysics plays in transportation projects. More transportation agencies are aware of geophysical methods as

tools to augment current subsurface investigations efforts. However, transportation agencies still rely heavily on subsurface drilling and in situ testing, even when geophysical methods can potentially save money, time, and reduce the risk associated with unknown subsurface conditions. For example, 68% of respondents in Sirles (2006) noted only "occasional" use of geophysical methods. As such, engineers in these transportation agencies still have limited familiarity with these methods. This leads to a lack of understanding and confidence in data processing/interpretation, which further deters use of geophysical methods when appropriate. The results from this study are meant to address this issue by synthesizing the current state of practice of geophysical methods for quantitative measurements of geotechnical properties. This report will address the existing gap in the literature for geotechnical engineers with an introductory knowledge of geophysics and it will provide guidance regarding acquisition of geotechnical design parameters. Examples are also provided with real subsurface data that demonstrate the value of geophysical measurements for transportation infrastructure projects.

1.2.3 CALTRANS EXPERIENCES WITH GEOPHYSICAL METHODS

Caltrans is unique in comparison to other state DOT's since it has a centralized branch in its organizational structure devoted to geophysics. The Branch of Geophysics and Geology is part of the Division of Engineering Services (Geotechnical Services Subdivision) and is responsible for providing support on geo-related capital development projects throughout the state of California. As of the writing of this report, the Branch of Geophysics and Geology is composed of eight personnel led by Branch Chief William Owen based out of the main Caltrans offices in Sacramento, California. The Branch of Geophysics and Geology essentially operates as an internal consulting group to serve the geophysical needs of the rest of the Department. Their work is typically performed under the auspices of the Chief Engineer, though sometimes the branch does work directly with local districts to identify appropriate geophysical solutions, deploy equipment, and analyze the resulting geophysical data. In limited cases, the internal capabilities of the branch may be exceeded (e.g., workflow cannot keep up with demand, specialized equipment is necessary for a particular application). In those cases, outside geophysical consultants can be brought in on Caltrans projects. Otherwise, geophysical testing on Caltrans projects is primarily performed by the Branch of Geophysics and Geology.

The Branch of Geophysics and Geology currently provides services with the following geophysical methods: Seismic Refraction, Refraction Tomography, Magnetometry, Electrical Resistivity, Electromagnetic Conductivity, Ground Penetrating Radar, Borehole Acoustic Televiewer, Borehole Resistivity, Borehole PS Suspension Log, Borehole Conductivity, Borehole Caliper, Borehole Natural Gamma, and Borehole Full Waveform Sonic Testing. The branch is typically involved fairly early in the design process of a project (i.e., less than approximately 60% complete). This corresponds to the 0- and 1- phase based on typical Caltrans terminology. In some cases, branch efforts may take place in the 2-phase or later when issues arise as part of construction. The primary application encountered by the branch is mapping stratigraphy and bedrock for foundation design or for excavations (e.g., cut-slope design, landslide mitigation, etc.). To that effect, seismic refraction/tomography is one of the most commonly employed geophysical methods by the Branch of Geophysics and Geology. The goal of this work is often to extrapolate information into the areas away from boreholes and to interpolate conditions between boreholes. Also, geophysics is sometimes used to make decisions about where to locate certain boreholes. For example, a key aspect of site subsurface investigations is locating boreholes away from any existing infrastructure or utilities beneath the surface. GPR can prove quite beneficial in such applications, which contributes to why GPR is another one of the most commonly employed methods by the Branch of Geophysics and Geology. Velocity logging is also performed by the branch to obtain the shear wave velocity profile and other relevant soil material properties for seismic design of the foundation/structure. P-S suspension logging was once commonly used in this context. However, the number of projects related to foundation design (particularly deep foundations) has diminished, which has reduced demand for logging velocity and the P-S logging tool. Other methods that are quite beneficial but have seen limited usage include surface wave testing techniques such as MASW and ReMi to estimate shear wave velocity. Finally, borehole imaging techniques such as acoustic televiewer and optical televiewer have been underutilized despite their abilities to provide direct inspection and measurement of in situ orientation of bedding planes and fractures. Whether the branch's geophysical efforts are utilized to guide the drilling program or to augment it once it is already taking place has often been dependent on the client for the particular project. However, Caltrans continues to recognize the importance of judicious use of geophysics to help guide the drilling program. To that effect, recent revisions to Caltrans project delivery/development documents have encouraged increased use of geophysics in the 0-phase. Geophysical work is rarely performed in

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the project initiation phase (i.e., K-phase) since there is no budget approved for such work at that stage of the process.

Generally, Caltrans seems to be shifting away from a construction focus to an operations and maintenance focus. There is not a large amount of construction taking place of "new" bridges and structures. Often, much of the present work centers on maintaining or replacing existing facilities (e.g., bridges that have been deemed structural deficient). Given these trends, it is unsurprising that some of the aforementioned geophysical techniques are falling out of favor and that NDT applications are increasing. For example, through the Branch of Geophysics and Geology, Caltrans has been involved in a number of National Cooperative Highway Research Program (NCHRP) Strategic Highway Research Program (SHRP2) initiatives that have been exploring the role of NDT in maintaining highway related components. This involvement has included a number of proof-of-concept applications of NDT for bridge deck investigations, pavement delamination, subsurface utilities, and tunnel linings.

Moving forward, the Branch of Geophysics and Geology will continue to lead the way in promoting and applying geophysical methods for highway related applications within the state of California. It is anticipated that part of its role will be to encourage more consistent application of geophysics for geotechnical projects. In that manner, early identification can occur of those projects where geophysics is applicable so that equipment can be swiftly mobilized on site early enough (i.e., 0-phase) to aid in the development of drilling plans. The goal would be to exploit the reconnaissance capabilities of geophysics and reduce the number of boreholes to the absolute minimum necessary in order to better manage subsurface investigation budgets. Regarding Caltrans experience with NDT, utility locating will continue to be vital for future projects. Many internal studies within Caltrans and others across the country have demonstrated that the return on investment is high when using NDT early in a project to locate utilities as part of construction efforts. A limited investment in NDT efforts up front leads to fewer cost overruns related to change orders and construction claims for utility relocation, protection in place or project redesign. It is anticipated that the Branch of Geophysics and Geology will place a larger focus on NDT moving forward and will continue to engage in activities such as the NCHRP SHRP2 initiatives that study the role of NDT in highway related construction and asset management. Finally, a large contribution to limited use of geophysics is likely related

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to lack of familiarity with these methods and their capabilities. To address this issue, the Branch of Geophysics and Geology will continue to engage stakeholders and provide formal and informal training opportunities regarding the work it performs and the importance of geophysics in modern DOT practice.

1.3 SCOPE

Given the previous discussions regarding the increasing importance of geophysics in transportation projects, the primary purpose of this report is to provide a review of the quantitative measurements possible using geophysical techniques. It is intended to address the existing gap in the literature regarding geophysical measurements for geotechnical purposes. This review will focus on soil and rock parameters that are particularly useful for geotechnical applications in transportation infrastructure projects. The majority of the information will be obtained from a compilation of literature where geophysical measurements have been utilized to obtain various soil and rock properties. Such literature will include case histories and comprehensive studies relating physical parameters to geophysical measurements. However, certain sections of the report will highlight potential knowledge gaps in the literature and will also address issues related to uncertainty in dynamic soil properties. Those discussions will highlight the importance of geophysical measurements and the potential impacts on geotechnical design. Recommendations will also be provided regarding use of geophysical measurements for subsurface investigations for typical Caltrans project applications.

1.4 ORGANIZATION

Chapter 2 introduces the typical geophysical techniques utilized in the field of geotechnical engineering. The initial focus of this chapter is a qualitative description of the methods, though it is not intended as a replacement for full texts on that subject (e.g., Telford et al. 1990, Wightman et al. 2003, Anderson et al. 2008) and other recent guidance resources (e.g., ASTM D6429, CFLHD website). This chapter also discusses typical subsurface investigation techniques used to estimate parameters often obtained from geophysical measurements.

Chapter 3 discusses the applications of geophysical measurements in geotechnical design of transportation projects. This chapter summarizes earth material properties and design

parameters that can be obtained from geophysical measurements. Included in this discussion are properties that are broad in scope and usage (e.g., porosity of bedrock, clay content, etc.), properties that are directly pertinent to seismic design (i.e., shear wave velocity), and nondestructive testing (NDT) applications. Additionally, uncertainty in shear wave velocity measurements is discussed and comparisons are made between results from geophysical measurements and those from traditional geotechnical subsurface investigations.

Chapter 4 summarizes the overall findings of the study. Significant overlap exists between geotechnical applications of geophysics and applications related to geological exploration and NDT. For example, many geophysical methods are utilized to detect voids and delineate subsurface features such as stratigraphic contacts, bedrock topography, fault traces, landslide slip surfaces, and similar features. Likewise, many NDT methods are related to geophysical methods and are similarly used to detect features (e.g., cracks, delamination, corrosion, voids, etc.) in engineered materials such as concrete and pavements. Though these qualitative evaluations for geological and NDT purposes are very useful and important applications, they are outside the scope of this report. Instead, Chapter 4 focuses on providing recommendation regarding appropriate use of geophysical measurements for geotechnical applications as related to quantifying earth material properties. The scope of these recommendations primarily focuses on shear wave velocity and related seismic design applications, though discussion is included for a number of non-earthquake applications such as strength of rock, voids/porosity, presence of water, and soil composition. Finally, Chapter 4 highlights any remaining knowledge gaps in the current state of geophysical practice and research as related to geotechnical engineering.

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of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Continued on Next Page

2. GEOPHYSICAL METHODS FOR TRANSPORTATION PROJECTS

This chapter provides a brief summary of the various geophysical methods that are common in geotechnical subsurface investigations. The focus is not on qualitatively describing each method, but rather to provide a current state of geophysical practice within the context of transportation infrastructure applications. More details on each method are available in a wide range of references available in the literature (e.g., Steeples and Miller 1988; Sheriff and Geldart 1995; USACE 1995; Ellis and Singer 2007; Jol 2008; Ashcroft 2011, etc.). An online resource based on Wightman et al. (2003) is also maintained at the Central Federal Lands Highway Division website (CFLHD 2013). A discussion is also provided in this chapter regarding the current state of practice for estimating pertinent soil properties based on in situ and laboratory investigations. Quantitative examples are developed and case histories are discussed where geophysical measurements prove more suitable to acquire such properties.

2.1 SURFACE METHODS

The following sections provide a brief summary of common surface geophysical methods that have been employed for geotechnical purposes. These methods rely on measurements obtained using equipment and instrumentation at the ground surface. The focus is to provide context for these methods as used in geotechnical engineering. The descriptions are qualitative in nature and assume basic understanding of fundamental geophysical concepts and familiarity with the methods described.

2.1.1 SEISMIC METHODS

Several methods rely on interpreting the subsurface based on the propagation of seismic waves. Seismic waves produce mechanical strains in the materials through which they propagate. The velocities at which the waves propagate depend on material elastic moduli and density. The resulting particle motions depend on seismic wave type, of which P-, S-, and surface (i.e., Rayleigh and Love) waves are the most commonly utilized. These motions can be measured using sensors and can be used to determine information regarding the material through which the wave propagates. These methods have been routinely used in the field of geophysics, particularly for mapping lithology as related to exploration for hydrocarbons (e.g., see history of seismic techniques in Sheriff and Geldart 1995).

Application	Seismic Refraction	Seismic Reflect.	Seismic Tomo.	GPR	EM	Resist.	IP	SP	Mag.	Grav.
Mapping lithology (<10m depth)	М	Х		M	х	х				
Mapping lithology (>10m depth)	x	М	Х		х	х				
Estimating clay/mineral content					M	х	х			
Locating shallow sand and gravel deposits				м	М					
Locating sand and gravel deposits (that contain heavy minerals)									м	
Determining volume of organic material in filled-in lakes or karsted features	М	М			м					М
Mapping top of ground water surface	M (P-wave)	M (P-wave)		м	м	М				
Determining water depths (including bridge scour)				м						
Mapping groundwater cones of depression	×	×		м	×	x				
Subsurface fluid flow								М		
Mapping contaminant plumes				M	M	х		х		
Mapping crop land salination and desalination over time					м	М				
Locating underwater ferromagnetic objects				м					М	
Mapping bedrock topography (<10m depth)	м			м	×	×				×
Mapping bedrock topography (>10m depth)	×	м			×	x				x
Mapping sub-bedrock structure	х	М		x	х	х				
Delineating steeply dipping geologic contacts (<10m depth)	м			м	м	м				
Delineating steeply dipping geologic contacts (>10m depth)	x	м	x		х	x			x	
Mapping fracture orientation (near- surface bedrock)	м			м						
Mapping fracture orientation	М		М							
Identifying regions of potential weakness (e.g., shear zones & faults; <10m depth)	М		x	м	x	x			x	
Identifying regions of potential weakness (e.g., shear zones & faults; >10m depth)	x	x	м		x	x			x	
Identifying near-surface karstic sinkholes and the lateral extent of their chaotic, brecciated, and otherwise disrupted ground	М	М		м	x	x				x
Mapping air-filled cavities, tunnels, (<10m depth)	×	×	x	м	x	м				x
Mapping air-filled cavities, tunnels, (>10m depth)	x	М	М		x	x				x
Mapping water-filled cavities, tunnels	X (P-wave)	M (P-wave)	М	×						
Mapping clay-filled cavities, tunnels	x	М	M		x	x				
Estimating rippability	M		х							
Foundation integrity studies	M		x	M						
Dam-site integrity studies	M	M	M	M	х	x		М		
Landslide site evaluation	M		M	x	M	M				
Locating buried well casings (metal)				м	м				м	

Table 2.1: Potential engineering applications of various geophysical methods (Anderson 2006).Note: M = Major Application, X = Minor Application

Application	Seismic Refraction	Seismic Reflect.	Seismic Tomo.	GPR	EM	Resist.	IP	SP	Mag.	Grav.
Locating buried drums, pipelines and other ferromagnetic objects			м	М				м		
Locating buried non-magnetic utilities			м							
Locating buried non-magnetic utilities				м						
Mapping archeological sites (buried				M	м				м	
ferro-magnetic objects, fire beds.										
burials, etc)										
Mapping archeological sites (non				м						
magnetic - excavations, burials, etc)										
Concrete integrity studies and				M						
inspection										
Detection of delamination and				M						
incipient concrete spallage on bridge										
decks										
Locating rebar in concrete				M	M				M	
Detection of corrosion of rebar				M						
embedded in concrete										
Evaluation of presence, pattern and				M	x				x	
density of rebar embedded in										
concrete destined for demolition										
Pavement integrity studies				M						
Detection of voids beneath pavement				M						
Detection and delimitation of zones of				M						
relatively thin sub-grade or base										
course material										
Detection and monitoring of areas of				M						
insufficiently dense sub-base										
Large-area differentiation and				M						
monitoring of insufficient thickness of										
pavement as a quality assurance										
measure during construction										
Large-area differentiation and				M						
monitoring of insufficient pavement										
thickness as post-construction										
monitoring technique										
Detection of bodies of sub-grade in				м						
which moisture content is										
anomalously high, as a precursor to										
development or pitting and potholes										
Mapping/locating landfills	X			×	IVI	×			M	
(bulk, shear and Young's moduli)	IVI		171							
Estimating in situ rock properties					M	M				
(saturation, porosity, permeability)										
Determining in situ rock densities										м
Determining in situ rock properties				x						
(dielectric constant)										
Mapping abandoned, in-filled open-pit	М	м		x	x	x			x	x
mines and guarries									~	
Mapping abandoned underground		М	x			x				
mines										
Detecting abandoned		х	x	M	M	x			x	
Mine shafts										

Table 2.1 (cont.): Potential engineering applications of various geophysical methods (Anderson 2006). Note: M = Major Application, X = Minor Application

The most common seismic surface methods as employed for geotechnical purposes include Seismic Reflection, Seismic Refraction, Spectral Analysis of Surface Waves (SASW), Multichannel Analysis of Surface Waves (MASW), and passive methods such as passive MASW, Microtremor Survey Method (MSM), Refraction Microtremor (ReMi), and the Horizontal-to-Vertical Spectral Ratio (HVSR). Active methods such as Seismic Reflection/Refraction, SASW, and MASW employ waves that are actively generated by seismic sources at the site, including sledge hammers, air guns, explosives, mass shakers, accelerated weight drops (AWD), and vibroseis vehicles, among others. Passive methods such as passive MASW, MSM, ReMi, and HVSR measure ambient seismic energy from various sources (e.g., traffic, ocean tidal activity, industrial and construction
noise, etc.). These methods are typically utilized to map and delineate geologic features (e.g., soil layer contacts, top of bedrock, faults/fractures, voids/tunnels, sub-bottom profiles, depth to water table, etc.) and to measure elastic wave velocities, particularly the time-averaged shear wave velocity in the upper 30 m of the site (i.e., V_{s30}) to determine the National Earthquake Hazards Reduction Program (NEHRP) site class (Tables 2.1 and 2.2). From velocity measurements it is possible to obtain the corresponding elastic modulus and the density of the material, which can be correlated to other earth material properties.

						Ν	lethods						
			Seismi	с		Elec	ctro-Ma	gnetic	Elect	rical		Other	r
						Techniques							
	Seismic Refraction	Seismic Reflection	Seismic Tomography	Shear Wave	Surface Wave (SASW, MASW, and passive)	Frequency–Domain EM (terrain conductivity)	Time-Domain EM (metal detector)	Time Domain EM Soundings	Electrical Resistivity	Induced Polarization	Gravity	Magnetics	Ground Penetrating Radar
Investigation Objectives													
Bedrock depth	Р	Р	Р		Р			S	S	S	S		
Rippability	Р		Р	Р									
Lateral and vertical variation in rock or soil strength	Р		Р	Р	Р								
Location of faults and fracture zones	S	Р	Р	Р	S	S		S	S	S	S	S	S
Karst features		S	Р		Р	S				Р			Р
Near-surface anomalous conditions					S	Р		Р	S				Р
Soil characterization and lithology			S	S	S	Р		S	Р	Р			
Locating landfill boundaries, waste pits, waste trenches, buried drums					S	Р	Р		Р	Р		Р	Р
Water table	S		S			Р		Р	Р	Р			
Water quality, fresh-saline water interfaces						Р		Р	Р	S			

Notes: This matrix is intended to aid in the selection of an appropriate geophysical method and respective technique for typical geotechnical investigation objectives. The table *does not* account for geologic conditions, site cultural features, target size, and depth. Refer to Appendix A for additional information regarding methods and techniques. SASW = Spectral Analysis of Surface Waves; MASW = Multi-Channel Analysis of Surface Waves; P = primary technique; S = secondary technique; blank space = technique should not be used.

Table 2.2: Potential engineering applications of various geophysical methods (Sirles 2006).

Seismic methods are generally robust methods that can delineate density contrasts deep below the surface. These methods are particularly well suited for obtaining properties of subsurface materials that are very difficult to sample using traditional geotechnical investigations or subsurface geophysical methods (e.g., glacial till, pavements, etc.). However, they do suffer from a number of limitations, primarily related to data post-processing and interpretation. The techniques rely on arrivals from waves that interact in a complex manner with the underlying subsurface soils. At particularly challenging sites, it may be difficult or impossible to separate the effects of different wave types (e.g., reflected/refracted body waves, first arrival surface waves, etc.) from the recorded ground signals. There are also limitations related to vertical and lateral resolution as well as signal-to-noise ratio. As a result, care must be exercised in designing field testing parameters to prevent spatial aliasing, near-field effects, and excessive signal attenuation. Optimal field testing parameters may be restricted by the logistics of the particular field site. For example, it may be impossible to string a survey line to the length necessary or the site may be subject to excessive background seismic noise. Non-unique solutions are possible for the results from surface-wave testing because their analysis is based on performing an inversion of the measured dispersion curve. Moreover, for passive methods, assumptions must be made regarding the directionality of the background seismic energy. As a result of these limitations, it is advisable to perform multiple seismic tests at a site to constrain results and provide supplementary information. Fortunately, the similarity in equipment used for each of the seismic methods encourages complementary testing.

2.1.2 ELECTROMAGNETIC METHODS

Electromagnetic methods [i.e., Ground Penetrating Radar (GPR), Borehole Radar (BHR), and Time-Domain Reflectometry (TDR)] are often utilized to explore the subsurface and differentiate between materials with different electric properties. These methods are distinguished from electrical/magnetic methods by the fact that they typically introduce combined electromagnetic waves into the domain of interest, rather than a direct current (DC) or alternating current (AC) electrical potential.

Regarding radar methods, much of the processing and interpretation of GPR reflection data is similar to that used for seismic wave reflection testing. However, the propagating waves are high-frequency (usually polarized) radar waves that are sensitive to the electromagnetic properties of the soil instead of mechanical properties. The results from radar methods are useful in determining stratigraphy (e.g., Jol et al. 2003), location of faults and fractures (e.g., Theune et al. 2006), presence of voids and tunnels (e.g., Di Prinzioa et al. 2010), location of utilities (e.g., Al-Nuaimy et al. 2000), and non-destructive testing purposes for concrete and pavements (e.g., Bungey 2004; Barrile and Paccinotti 2005; Chang et al. 2009; Chen and Wimsatt 2010; Willett et al. 2006). The primary measurement in radar methods is the propagation velocity of electromagnetic waves in the medium (i.e., the electromagnetic analog to elastic wave velocity as obtained in seismic reflection/refraction testing). The propagation velocity can be estimated from reflected waves, critically refracted waves, and ground waves generated by the transmitter and measured at the receiver. From this velocity it is possible to obtain the corresponding dielectric permittivity of the materials.

In its simplest form, TDR relies on measurements of electromagnetic waveforms along a waveguide (e.g., transmission line, cable) of known length and constant dielectric constant. A pulse generator attached to the cable inputs the appropriate input voltage signal and any reflections are recorded using an oscilloscope. In the case of applications related to soils, the "cable" is actually a probe and the soil functions as the dielectric material between the "cable" elements. During operation, reflections of the input pulse occur at the initial and final contact locations between the probe and soil. Since the length of the "cable" is known, a travel time analysis of the reflected signals can be performed to determine the velocity of the electromagnetic wave and the corresponding dielectric constant. TDR has proven useful in evaluating soil moisture and density, particularly in the context of compaction quality control (Lin et al. 2000; Yu and Drnevich 2004; Fratta et al. 2005; Lin et al. 2012) and agricultural/environmental applications (Dalton and Van Genuchten 1986; Inoue et al. 2001; Wraith et al. 2005; Oberdörster et al. 2010; Kallioras et al. 2016).

2.1.3 ELECTRICAL AND MAGNETIC METHODS

A number of surface methods employ measurements related to electrical and/or magnetic potentials. Some methods utilize passive instrumentation to measure the intrinsic electrical/magnetic properties of the subsurface and others utilize active sources of electrical current. Examples of passive methods include magnetic surveys that measure local perturbations in the Earth's magnetic field using a magnetometer and self potential methods

that measure the voltage difference between two points on the ground caused by the small, naturally produced currents that occur beneath the Earth's surface. Active methods include Electrical Resistivity and Induced Polarization (IP), which determine the resistivity of soils by measuring their response to applied current.

The most commonly employed electrical/magnetic geophysical methods include ER/IP and magnetic surveys. These methods are typically used to measure the electrical properties of the soil and to map the subsurface (e.g., delineate layer contacts between soils or to determine the depth to the ground water table). The electrical characteristics of soil are inherently related to other properties, including void ratio and porosity, water content, hydraulic conductivity, and density. As such, these methods can be used to estimate these properties based on correlations to electrical resistivity/conductivity and magnetic susceptibility.

Electrical and magnetic methods are well suited to sites where significant contrasts exist in the electrical/magnetic response of the underlying subsurface materials. It is for that reason that these methods are often used to locate and evaluate the condition of embedded man-made materials (e.g., steel, pipelines, utilities, etc.). As such, a number of these methods have been routinely utilized for nondestructive testing (NDT) purposes (e.g., Table 2.1). However, due to the nature of the measured parameters in these studies, these methods suffer from poor performance at sites where significant background electrical noise is prevalent (e.g., urban sites, power lines, grounded metallic objects, etc.). Moreover, saturated clayey soils can present challenges because their electrical properties cause severe attenuation in the input energy. Similar to seismic methods, equipment layout can be negatively impacted by field logistics. For example, very long lines are necessary to string out the large number of sensors necessary for sufficient resolution in ER/IP surveys. Finally, data analysis and interpretation is not trivial for a number of these methods (e.g., ER/IP) as appropriate inversion techniques and modeling of the subsurface is necessary.

2.1.4 GRAVITY METHODS

Methods that measure gravitational forces associated with an object have an extensive history within the geophysical community and have been used over a wide range of scales and purposes (for a good summary, see Nabighian et al. 2005). For example, at a global and interstellar scale,

measurements of gravity are vital to understand the complex workings of planetary bodies. For geophysical exploration purposes, the gravity method has been widely used for mining and oil exploration. In engineering and environmental applications the gravity method can be performed at a much more localized scale (i.e., microgravity surveys) to locate subsurface features (e.g., voids, changes in depth to bedrock, buried stream valleys, water table levels, etc.) and to estimate fluctuations in mass density across a site. Generally, the gravity method relies on gravimeters to measure small changes in the gravitational field at the Earth's surface due to a gravity anomaly. The magnitude of these changes can be attributed to lateral density changes in the subsurface (e.g., a mass concentration or void) as well as terrain, tidal, equipment drift, elevation, and motion-induced variations in the total Earth gravity field. Though they are not as readily available as surface modules, borehole gravimeters have been produced as early as the 1950's to perform similar gravity measurements within a borehole.

The measurements required in the gravity method are relatively simple to perform. However, the challenge in the method results from minimizing the issues associated with sources of "noise" (e.g., equipment drift, tidal variations in gravity measurements, etc.) and in accurately determining station locations and elevations from a high precision site survey. For locating features at the engineering scale of interest, high station density is necessary and the most time-consuming aspect of a microgravity study is often surveying the area of interest. Moreover, since the gravity measurements can vary with time due to tidal changes and equipment drift, measurements must be repeated several times at each station. The gravity method is less affected by issues found in electrical and magnetic methods, such as limitations in investigation depth due to highly conductive clay-rich soils near the surface. However, one of the main drawbacks is related to data interpretation. For example, a gravity anomaly from a distribution of small masses at a shallow depth can produce the same effect as a large mass at depth. Resolving this ambiguity can necessitate external information from other geophysical methods or geotechnical subsurface investigations.

Gravity methods have been increasingly used for engineering purposes (e.g., evaluation of sinkholes, soft surficial anomalies, etc.). This increase has been driven by improvements in gravimetry equipment that have allowed more consistent and higher resolution measurements of the gravitational field. Typical gravity anomalies for near-surface engineering applications

have magnitudes in the range of $10 - 1000 \,\mu$ Gal (hence the use of the microgravity term), where Gal is a unit of acceleration measurement equal to 1 cm/s² (Butler 2007). This implies that gravimeter sensitivity, accuracy, and precision must be on the order of 1 μ Gal (i.e., 1 part in 10⁹ of the earth's gravitational field) for engineering applications, an achievement not realized until the 1960's and 1970's (Butler 1980; Nabighian et al. 2005). Since this equipment has been available, there has been growing interest in performing high resolution microgravity surveys for engineering issues ranging from the delineation of fracture zones to estimating aquifer porosity and depth to bedrock, delineating substrata depth variations and fill thickness, and verifying bedrock conditions (Hall and Hajnal 1962; Eaton et al. 1965; Domenico 1967; Wolters 1973; Arzi 1975; Carmichael and Henry 1977; Wang et al. 1986; Benson and Baer 1989; Roberts et al. 1990; Tønnesen 1995; Benson and Floyd 2000; Hayashi et al. 2005; Davis et al. 2008; Mankhemthong et al. 2012). The most common application of microgravity surveys is to evaluate the presence of subsurface cavities such as sinkholes and other karst topography (Butler 1984; Wenjin and Jiajian 1990; Camacho et al. 1994; Yule et al. 1998; Beres et al. 2001; Benson et al. 2003; Styles et al. 2005; Mochales et al. 2008; Tuckwell et al. 2008; Whitelaw et al. 2008; Orfanos and Apostolopoulos 2011; Paine et al. 2012).

The unifying earth material property in each of the aforementioned microgravity applications is the density of the subsurface materials. Fluctuations in gravitational fields are directly dependent on five factors: latitude, elevation, topography, tidal changes, and density variations (Telford et al. 1990). In application of the gravity method for engineering and geological purposes, the density variation is typically the relationship of interest. The variations in gravity that result from differences in density are small in relation to fluctuations that result from the other factors. Fortunately, post processing techniques exist to remove the effects of latitude, elevation and topographical changes, and temporal (i.e., tidal) fluctuations, though utmost care must be exercised during field operations to minimize the influence of these factors. Various references provide relevant background information regarding the field procedures and associated data post-processing steps involved in isolating the effects of density on gravimetric measurements (e.g., Neumann 1977; Butler 1980; Hinze 1990; Telford et al. 1990; Mickus 2003). The majority of microgravity surveys for engineering purposes are performed with relative gravimeters, which determines the difference in gravity between measurement locations. Absolute gravity instruments are more expensive, physically larger, require longer acquisition

times, and are generally less user-friendly compared to relative gravity instruments (Nabighian et al. 2005). The raw data collected by a relative gravimeter is post-processed and corrected for the aforementioned factors. Post-processing also typically includes the separation of the anomaly of interest (residual) from the remaining background anomaly (regional) using manual or polynomial surface fitting techniques (Hinze 1990), among other approaches. The end result is a spatial distribution map of residual gravity measurements. This map can typically be used in conjunction with other subsurface investigation techniques to provide a qualitative assessment of subsurface conditions based on changes in gravity. For example, minima in the gravity measurements (i.e., negative gravity anomalies) typically correspond to potential cavities (Styles et al. 2005). Some of the earliest applications of microgravity surveys have relied on such spatial distribution maps to evaluate subsurface conditions (e.g., Arzi 1975; Fountain et al. 1975; Neumann 1977). More detailed analysis such as numerical modeling can be performed on spatial measurements of gravity to quantify the nature (e.g., depth and geometry) of subsurface features causing the gravity variations. Additionally, examining the vertical and horizontal gradients (i.e., first derivatives) of the gravity measurements can be of considerable importance and provide additional information regarding subsurface conditions, particularly for anomalies caused by shallow subsurface structures (Evjen 1936; Heiland 1943; Butler 1980; Butler 1984). A number of case histories have demonstrated the use of gravity gradient measurements to evaluate the subsurface for engineering applications (e.g., Fajklewicz 1976; Butler 1984; Pan 1989; Pajot et al. 2008; Erkan et al. 2012).

2.1.5 REMOTE SENSING

Remote sensing refers to a broad range of techniques where information is obtained based on measurements made at a distance without making physical contact with the object. This definition is inherently broad [e.g., see summary in Campbell and Wynne (2011)] and includes digital imagery methods, thermal radiometry, remote acoustics, radar-based technology [synthetic aperture radar (SAR), in combination with interferometry (InSAR)], and LiDAR among others. Remote sensing technologies typically rely on measurements of propagated signals (e.g., electromagnetic radiation) that are either actively emitted from a source or passively collected from the object being measured. As such, there is overlap in the physics and fundamental operating theory between remote sensing and many geophysical methods. This overlap is even more readily apparent in airborne applications of geophysical surface techniques such as

electrical/magnetic and gravity methods. Additionally, there are growing opportunities in remote sensing techniques (i.e., InSAR and LiDAR) for the evaluation of transportation infrastructure and other civil engineering applications, either as standalone methods or to provide complementary information to geotechnical investigations and/or geophysical studies.

The leading transportation application of remote sensing is for imaging purposes (e.g., deformation imaging, landslide and rockslide characterization, etc.). As such, in depth discussion of these technologies and their transportation applications are outside the scope of this report, which focuses on the measurement of earth material properties. However, a number of case studies will be briefly mentioned below and readers are encouraged to review these studies and Power et al. (2006), Kemeny and Turner (2008), and Morgan et al. (2011) for more thorough discussions of these technologies, particularly as related to InSAR and LiDAR. Additionally, Brown and Hodges (2005) provides a broad overview of application of airborne geophysical methods for engineering purposes and Vaghefi et al. (2012) provides an overview of commercially available remote sensing technology as applicable to evaluation of bridges.

Among the commonly applied remote sensing technologies for engineering purposes are airborne surveys of surface geophysical methods. A number of studies have utilized aircraftbased electrical/magnetic and gravity methods to develop subsurface maps for various engineering purposes, including ground water quality studies (Fitterman and Deszcz-Pan 1998; Ackman 2003; Siemon et al. 2009), tunnel and pipeline construction (Hodges et al. 2000; Okazaki et al. 2011), location of buried metallic structures such as underground storage tanks (UST) and unexploded ordnances (UXO) (Takata et al. 2001; Doll et al. 2012a), evaluation of levee condition (Hodges et al. 2007; Amine et al. 2009; Doll et al. 2012b), and rockslide/landslide investigations (Nakazato and Konishi 2005; Pfaffhuber et al. 2010). Among established remote sensing techniques, InSAR (satellite and aerial) and LiDAR (aerial and terrestrial) are increasingly observed in engineering studies. For example, recent CFLHD projects have explored the use of InSAR to measure landslide movements (Anderson et al. 2004; Power et al. 2006; Sato et al. 2009; Morgan et al. 2011). These studies have demonstrated the ability to resolve movements on the order of centimeters. Similar efforts have also been performed to monitor dams and bridges (Tarchi et al. 1999; Pieraccini et al. 2006; Soergel et al. 2008; Zhang et al. 2010; Talich et al. 2014), pavements (Suanpaga and Yoshikazu 2010), levees (Dabbiru et al. 2010; Bennett et al.

2014; Han et al. 2015), rock slopes (Bruckno et al. 2013), road subsidence (Yu et al. 2013; Lazecký et al. 2014), and sinkholes (Vaccari et al. 2013). In the case of LiDAR, applications have ranged from evaluating levee integrity (aerial: Bishop et al. 2003; Casas et al. 2012; terrestrial: Kemeny and Turner 2008; Collins et al. 2009) to estimating ground deformations due to underground construction (Hashash et al. 2005), characterizing landslides (Conte and Coffman 2013), estimating deformations due to expansive clays (Garner and Coffman 2014), and mapping ground deformations and structural failures due to seismic events (Kayen et al. 2006).

Remote sensing and airborne geophysical studies provide some advantages over traditional surface and subsurface geophysical methods. They often have higher production rates capable of providing measurements over a larger area in a smaller amount of time. Moreover, these methods can provide measurements in rugged terrains that are often difficult to traverse by foot or ground vehicles (e.g., across rivers, marshes, mountains, etc.). However, remote sensing techniques suffer from limitations associated with equipment costs, complex data interpretation, limited temporal resolution for satellite-based measurements, and coarse spatial resolution in comparison to surface/subsurface geophysical methods. Despite these limitations, there is tremendous potential offered by remote sensing techniques in management and evaluation of transportation infrastructure, as evidenced by a growing amount of interest at the state and federal levels [e.g., the USDOT's Commercial Remote Sensing & Spatial Information (CRS&SI) program]. Application of remote sensing and aerial geophysical methods should only continue to grow, particularly as the use of unmanned aerial systems (UAS) proliferates and such systems are retrofitted to enable remote sensing measurements. One such example is the Jet Propulsion Laboratory's (JPL) Uninhabited Aerial Vehicle Synthetic Aperture Radar (UAVSAR) project that was funded in 2004 by the National Aeronautics and Space Administration's (NASA) Earth Science Technology Office (Fore et al. 2015). Since the system became operational in 2009, it has been utilized for a number of engineering applications, including visualization of fault slip (Rymer et al. 2011; Donnellan et al. 2014), monitoring of sinkholes (Jones and Blom 2013), evaluation of levees (Aanstoos et al. 2011), and estimation of fault-induced landslide movements (Scheingross et al. 2013). Similar efforts are underway to develop more transportation-focused UAS technology with the support of FHWA and the USDOT CRS&SI program.

2.2 SUBSURFACE METHODS

The following sections provide a brief summary of common subsurface geophysical methods that have been employed for geotechnical purposes. These methods typically deploy sensors within the subsurface either from boreholes or with the use of a cone penetration testing (CPT) rig. The focus herein is to provide context for these methods as used in geotechnical engineering. The descriptions are qualitative in nature and assume basic understanding of fundamental geophysical concepts and familiarity with the methods described.

2.2.1 ACOUSTIC METHODS

Acoustic subsurface methods utilize the travel time of mechanical stress waves (i.e., compressional) that are generated by transmitters within a borehole. This wave energy travels through the fluid of the borehole (i.e., pressure or tube wave) and along the borehole walls, often refracting and converting into other modes of wave propagation (i.e., shear waves). The probes utilized for acoustic logging often contain a number of receivers to record the wave arrivals (both compressional and shear). The travel time of the waves is related to the lithology and porosity of the borehole wall materials.

A number of methods are included in this category of geophysical testing, including acoustic velocity logging, full waveform sonic logging, and suspension logging. These methods primarily differ in the analytical methods used, the frequency of the input signals, and the purpose of the corresponding data. Typically, a plot of wave velocity (compressional and shear) is obtained with depth that illustrates the stratigraphy at the borehole location. However, it is also possible to obtain information regarding the location of factures and to correlate the measurements to porosity, permeability, bulk density, and other elastic properties. Finally, the results from acoustic borehole methods can be used to evaluate the condition of the borehole for quality assurance purposes.

Borehole acoustic methods typically provide a higher level of resolution relative to surface methods, and are therefore well suited to determine localized fluctuations in velocity. For example, sonic logging tools can have a fixed receiver interval as low as 0.3 m (1.0 ft) and can therefore resolve soil layers with thicknesses on the order of this value. Moreover, borehole acoustic methods can reliably acquire data at this resolution at up to kilometers of depth

because the interval between source and receivers is fixed throughout testing (unlike surface methods where deeper layers are significantly farther away from the source-receiver pair or down-hole methods where the source and receiver are increasingly separated for deeper measurements). However, there are some limitations to these methods. The velocity measurements are much more localized than typical surface methods and only a limited amount of material is sampled in the immediate vicinity of the borehole [i.e., within three times the wavelength (Pirson 1963)]. As with other borehole geophysical logging techniques, disturbance due to drilling (e.g., stress release, drilling mud infiltration, fracturing, etc.) can affect the measurements (Hodges and Teasdale 1991). Care must be exercised during testing operations that the probe is vertical and equidistant from the borehole wall and that borehole verticality is consistent after construction. Finally, data interpretation can be problematic in certain profiles because the nature of the transmitted/refracted waves can be complex. For example, based on Snell's Law a refracted shear wave may not be generated from the input compressional wave if the soil shear wave velocity is slow enough. In this situation acoustic logging would be unable to resolve the shear wave velocity of such a formation.

2.2.2 TELEVIEWERS

Televiewer methods involve the use of equipment to image the borehole wall. Measurements are performed using specialized submersible charge coupled device (CCD) cameras [i.e., Optical Televiewer (OTV)] that produce direct images of the borehole wall or with ultrasonic transducer systems [i.e., Acoustic Televiewer (ATV)] that operate in a pulse-echo arrangement and generate synthetic images of the borehole wall based on wave travel time and amplitude. In both cases, the measurements are performed as the sensor is rotated within the borehole and the resulting image captures a 360° scan of the borehole wall.

The primary use of OTV and ATV measurements is to identify stratigraphic layers, determine the location and extent of fracturing/voids, and to evaluate borehole construction. Most commercial systems operate using software that is capable of analyzing the corresponding images and providing information regarding planar features, such as strike and dip, frequency, and aperture size. Moreover, the images can be further examined for indications of water flow and/or contamination and changes in borehole diameter and wall roughness (either due to drilling or lithology). Therefore these techniques focus less on direct measurement of soil

properties and more on locating specific features such as fractures. However, such information can often allow evaluation of other pertinent information such as the orientation of stress fields (Wolff et al. 1974; Keys et al. 1979).

ATV is capable of resolving very small features on the order of 1 mm under ideal conditions (Wightman et al. 2003) and OTV resolution is restricted essentially by the quality of the camera. However, there are a number of limitations to these methods. Both methods examine only a limited area in the vicinity of the borehole wall, which may not be representative of the entire formation. Aberrations in the magnetic field (e.g., significant presence of metallic objects such as a steel casing) render inaccurate the magnetometer readings that orient the televiewers during data acquisition. OTV is affected by the clarity of the drilling fluid in the hole and ATV can only be performed in a fluid-filled borehole.

2.2.3 SEISMIC METHODS

A number of seismic methods have been developed that utilize the travel time of mechanical stress waves as measured by geophones and/or hydrophones within a borehole or set of boreholes. These stress waves (typically shear waves) are often generated either at the surface [e.g., Down-hole survey and Seismic Cone Penetration Test (SCPT)] or within a borehole (e.g., Cross-hole survey). A wide array of seismic sources is possible, including sledgehammers, sparker sources, and air guns. These methods are similar in approach to the acoustic methods discussed in 2.2.1. However, the fundamental difference is that the seismic source and the corresponding receiver are not collocated in the same borehole. The source is either at the surface or in another borehole away from the receiver(s).

The most commonly used seismic subsurface methods in geotechnical engineering include Down-hole surveys, Cross-hole surveys, and SCPT. Typically, the underlying goal of seismic subsurface methods is the development of an accurate profile of shear wave velocity. Compressional wave velocity – and, by extension, Poisson's ratio – and attenuation can sometimes be estimated, particularly in cross-hole tomographic studies. SCPT has an added time efficiency benefit that testing can take place concurrently with the acquisition of detailed penetration resistance information. However, SCPT is limited to testing in materials and over depths applicable to typical cone penetrometer rigs. For example, SCPT may be ineffective in

glacial tills as cone refusals may prevent the acquisition of data over an appropriate range of depths. Down-hole surveys and Cross-hole surveys are typically performed with one measurement at each depth. For example, in down-hole surveys the source at the surface will be activated each time the receiver in the borehole is lowered to a new location and the travel time with depth will be utilized to develop the velocity profile. Similarly, the source and receiver will each be moved to occupy the same depth in different boreholes as part of cross-hole data acquisition. However, as previously noted, data acquisition in cross-hole surveys can also be performed in such a way as to allow seismic tomography to be performed. Seismic tomography refers to the development of two- or three-dimensional (2D and 3D) velocity images between boreholes by performing an inversion algorithm on the acquired waveforms. During data acquisition, the source and receivers are relocated to occupy a number of stations in their respective boreholes. For example, a string of receivers may be placed in one borehole and the source moved within its borehole from bottom to top at a specific interval. Once the data has been acquired, algorithms are utilized to solve the system of thousands of nonlinear equations to reconstruct the velocity field between the boreholes. These algorithms are often based on ray tracing or some form of the wave equation that models the manner in which waves propagate between source and receivers. As with other methods where seismic wave velocities are measured, the results from borehole seismic methods can be correlated to other elastic material properties.

The resolution of seismic subsurface methods lies somewhere between borehole acoustic methods as described in 2.2.1 and surface seismic methods such as SASW and MASW as described in 2.1.1. Additionally, data interpretation for Down-hole, SCPT, and non-tomographic Cross-hole surveys can be simplified relative to SASW/MASW because the receiver is located within the soil column at a specific depth for a given seismic input wave. In these cases, velocities can be estimated by distinguishing first arrivals in the wave record. The layout of these methods also allows a larger volume of material to be sampled in relation to acoustic methods. However, the amplitude of the source input wave attenuates in the Down-hole and SCPT methods when the receiver is lowered, which makes interpretation of first arrivals increasingly difficult as the test progresses. Cross-hole surveys address this issue and also allow information regarding anisotropy of the soils, however at additional costs and complexities with field setup. Finally, the quality of borehole construction can significantly affect the results. It is particularly

important to ensure adequate coupling for any borehole casings that may be installed and also to maintain borehole verticality during construction (or otherwise measure the borehole alignment during testing). This is also true for SCPT, where care must be exercised to ensure the cone tip does not excessively wander as it is pushed into the ground.

2.2.4 BOREHOLE RADAR

Borehole radar (BHR) as a subsurface method essentially mimics the functionality of GPR within a borehole. A transmitter and receiver antenna is lowered into the borehole, where electromagnetic pulses (often in the MHz range) are radiated and reflected energy is recorded with depth. The reflections occur at boundaries between materials with different electrical characteristics (i.e., dielectric constant). Antennas have been developed at different central operating frequencies to allow tailoring the resolution and penetration depth of the system to particular site needs and conditions. Generally, BHR is run within a single borehole as a reflection survey. However, it is possible with certain systems to perform surface to borehole measurements and/or cross-hole measurements as well.

As with GPR, BHR finds preliminary use as a tool to delineate geological features, particularly in cases where the depth of coverage for GPR is inadequate. For example, BHR can be used to map fractures, voids and cavities, and contacts between layers, up to kilometers in depth below the surface. Additionally, the electrical characteristics of the soil that are measured by BHR can be correlated to other soil properties.

As the fundamental mechanisms are nearly identical, BHR shares a number of limitations with GPR. For example, saturated clayey soils still drastically attenuate the radar signals in BHR and affect its ability to transmit signals a significant distance away from the borehole. Background interference from electrical transmission sources (e.g., cellular towers, radio transmitters, etc.) can negatively impact BHR signals. However, since the antenna is lowered into a borehole, BHR is able to resolve deeper profiles than possible using GPR.

2.2.5 ELECTRICAL METHODS

As is the case for surface methods, a number of methods have been developed for use in borehole geophysical investigations that gather information regarding the response of subsurface materials to electrical currents and potentials. Data acquisition is typically accomplished using electrodes or coil probes that transmit and/or measure either direct current (DC) or alternating current (AC) signals. Some of the methods are passive and measure the telluric currents present in the soils and formations [e.g., Spontaneous Potential (SP) logging], while others actively induce currents into the surrounding materials and measure the corresponding response (e.g., Resistivity and/or Induction logging).

Most of the electrical methods trace their origins to geophysical borehole logging in the petroleum industry. As such, the most common applications of these methods include mapping lithology of underlying soils and rocks, determining layer thicknesses, and determining salinity of groundwater (Wightman et al. 2003). The most common of the subsurface electrical methods as utilized for geotechnical engineering purposes include SP logging, resistivity techniques, and induction logging. As with other electrical geophysical methods, the electrical characteristics of the soil can be correlated to other soil properties, including clay content and porosity.

Borehole electrical methods typically allow a greater depth of coverage than surface based electrical methods as transmitter and receiver are often collocated at approximately the same location within the borehole. However, they suffer from similar limitations given that the fundamental theory behind their operation is practically identical. Added complications include the effects of borehole construction on the measurements as well as the additional borehole fluid interface that can alter the electrical characteristics of the surrounding formations.

2.2.6 NUCLEAR METHODS

Nuclear methods include a number of techniques that rely on detecting the presence of unstable isotopes in the vicinity of the borehole. Measurements can be made in a passive manner that sample the background levels of radiation (e.g., gamma logging) or in an active manner that introduces small levels of radiation and measures backscatter (e.g., gamma-gamma logging). Different isotopes are utilized depending on the test performed (e.g., gamma logging versus neutron logging).

The most commonly used nuclear methods for geotechnical purposes include gamma logging, gamma-gamma logging, and neutron logging. These methods are primarily used to map

subsurface stratigraphy as the amount of radioactivity is a function of bulk density, porosity, and moisture content. As a result, nuclear methods also find applications related to quality control of compaction (i.e., nuclear gauge test) and for non-destructive testing (NDT) of drilled shafts to ensure integrity of concrete. Moreover, the sensitivity of the results to moisture changes allows these methods to be used to monitor groundwater movement (e.g., between waste containment facilities and underlying aquifers). It should be noted that though nuclear methods are presented in the subsurface section of this chapter, surface methods that rely on the same concepts do exist in practice and do see routine use (e.g., nuclear gauge test, neutron moisture probe, etc.). However, these surface nuclear methods will be limited to very shallow investigations because measurements only occur in their immediate vicinity (i.e., 4 - 6 in) (Timm et al. 2005).

If calibrated appropriately and interpreted relative to other background information at a site, nuclear methods can provide accurate information regarding density, moisture content, and identification of geological units and rock types. However, there are a number of unique aspects of nuclear methods that must be appreciated in order to ensure successful testing. To start, any contamination of the surrounding materials by artificial radioisotopes will alter the readings and can be difficult to isolate. Measurement accuracy of the probe is increased as the counting rate and length of data acquisition at a given point is increased due to the decaying nature of radioactive isotopes. This must be balanced against the logging speed and vertical resolution requirements for a given project site. Additionally, the use and transportation radioactive materials are regulated by both Federal and State agencies. Care must be exercised when handling equipment to ensure the radioactive sources are not subject to excessive wear and tear. Therefore nuclear methods are subject to extra logistical concerns relative to other geophysical methods.

2.3 SUBSURFACE INVESTIGATION CURRENT STATE OF PRACTICE

As previously discussed, all transportation projects are built on or with earthen materials and it is important to understand their behavior and properties to ensure adequate long-term function. The role of the geotechnical engineer can vary by project, but commonly entails the development of suitable subsurface investigation operations to characterize the site conditions. The data derived from subsurface investigations are evaluated to define stratification and groundwater conditions and to develop appropriate soil/rock parameters for use in design. As previously noted, several methods are available to successfully perform suitable subsurface investigations, including geophysical methods. The following sections discuss the current state of practice for subsurface investigations as related to transportation projects. Included in the discussion are typical geotechnical investigation techniques, in situ methods to determine geophysical parameters when geophysical techniques are unavailable or cannot be reasonably obtained, and laboratory methods to determine geophysical parameters (e.g., shear wave velocity) from soil samples. These sections do not provide detailed information on all available subsurface methods; rather the focus is on providing an overview of typical subsurface investigations and how geophysics can fit into this process. The reader is encouraged to review other references that specifically focus on site investigations, including Mayne et al. (2002) and Sabatini et al. (2002), for detailed information on the various subsurface investigation methods.

2.3.1 TYPICAL GEOTECHNICAL SUBSURFACE INVESTIGATIONS

The design and execution of a geotechnical subsurface investigation is a multi-step process that involves appropriate communication often among several parties, including the geoprofessional, the client, other engineers (e.g., structural engineer, project engineer, etc.), field staff (e.g., maintenance, environmental, traffic coordinators, etc.), subsurface drillers, permitting agencies, and other outside consultants (e.g., specialty drilling operators, geophysical services, etc.). The type of investigation performed will vary depending on the nature of the project (e.g., size, scope, new construction versus rehabilitation, etc.) and the site conditions (e.g., topography, environmental constraints, etc.). Generally, geo-professionals are approached to provide recommendations regarding subsurface conditions for new construction projects (initial planning purposes or geotechnical design), for rehabilitation projects, and/or for geoenvironmental concerns (e.g., contaminated sites) (Mayne et al. 2002). The most common type of subsurface investigation project is performed for new construction (e.g., new foundation). In these projects, the main purpose of the subsurface investigation is to obtain the stratigraphy and engineering properties of the soil or rock at the site that could affect the design of the project, while minimizing exploration costs (Caltrans 2015, ASTM D420). A typical subsurface investigation for new construction involves multiple stages, progressing from preliminary office/field reconnaissance to designing/planning an appropriate subsurface investigation plan, and finally to executing the investigation, interpreting the results, and

developing a corresponding geotechnical report summarizing the findings. A number of documents address the general development and execution of subsurface exploration plans (NAVFAC 1986; AASHTO 1988, currently being revised under NCHRP Project 21-10; USACE 2001; Mayne et al. 2002; Sabatini et al. 2002; ASTM D420). Moreover, the Caltrans Geotechnical Manual has a section devoted to geotechnical investigations that documents the Department's standards of practice for characterizing subsurface conditions (Caltrans 2015). The following sections are not meant to replace these references and the reader is encouraged to review them as appropriate. Instead, the following sections synthesize these references and briefly discuss major highlights within the investigation phases and the role of geophysical methods in the process.

2.3.1.1 Subsurface Exploration Plan

Prior to the initiation of field subsurface investigations, it is imperative that a well-defined exploration plan is established to ensure that the engineer is able to obtain all the necessary data to perform engineering analyses and design. The required subsurface data and corresponding exploration plan will depend on the nature of the proposed project (e.g., Table 2.3). Therefore, it is vital to review the proposed project request and plans with the client so that any questions regarding the scope of the work are clarified. For projects in the planning phase (i.e., K and 0 phase in Caltrans terminology), the purpose of the field investigation is to gather existing site information, evaluate if the proposed work is appropriate, and to support preliminary recommendations (Caltrans 2015). Design-phase (i.e., 1 phase in Caltrans terminology) subsurface exploration must adequately define stratigraphy and engineering properties of the soils and rocks that can impact the proposed project (Caltrans 2015). This subsurface exploration plan should consider all available investigation techniques, including hand augers and/or test pits, subsurface drilling (with disturbed and undisturbed sampling), in situ testing, geophysical investigations, and remote sensing. Generally, the subsurface exploration plan should stipulate that remote sensing and geophysical techniques (if necessary) be conducted prior to subsurface drilling as these methods are faster, less invasive, and can provide supplementary information to guide subsurface drilling (Mayne et al. 2002).

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing ⁽¹⁾	Laboratory Testing ⁽¹⁾
Shallow Foundations	 bearing capacity settlement (magnitude & rate) shrink/swell of foundation soils (natural soils or embankment fill) chemical compatibility of soil and concrete frost heave scour (for water crossings) extreme loading 	 subsurface profile (soil, groundwater, rock) shear strength parameters compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) frost depth stress history (present and past vertical effective stresses) chemical composition of soil depth of seasonal moisture change unit weights geologic mapping including orientation and characteristics of rock discontinuities 	 vane shear test SPT (granular soils) CPT dilatometer rock coring (RQD) nuclear density plate load testing geophysical testing 	I-D Oedometer tests direct shear tests triaxial tests grain size distribution Atterberg Limits pH, resistivity tests moisture content unit weight organic content collapse/swell potential tests rock uniaxial compression test and intact rock modulus point load strength test
Driven Pile Foundations	 pile end-bearing pile skin friction settlement down-drag on pile lateral earth pressures chemical compatibility of soil and pile driveability presence of boulders/ very hard layers seour (for water crossings) vibration/heave damage to nearby structures extreme loading 	 subsurface profile (soil, ground water, rock) shear strength parameters horizontal earth pressure coefficients interface friction parameters (soil and pile) compressibility parameters chemical composition of soil/rock unit weights presence of shrink/swell soils (limits skin friction) geologic mapping including orientation and characteristics of rock discontinuities 	 SPT (granular soils) pile load test CPT vane shear test dilatometer piezometers rock coring (RQD) geophysical testing 	triaxial tests interface friction tests grain size distribution 1-D Ocdometer tests pH, resistivity tests Atterberg Limits organic content unit weight collapse/swell potential tests slake durability rock uniaxial compression test and intact rock modulus point load strength test
Drilled Shaft Foundations	shaft end bearing shaft skin friction constructability down-drag on shaft quality of rock socket lateral earth pressures settlement (magnitude & rate) groundwater seepage/ dewatering presence of boulders' very hard layers scour (for water crossings) extreme loading	 subsurface profile (soil, ground water, rock) shear strength parameters interface shear strength friction parameters (soil and shaft) compressibility parameters horizontal earth pressure coefficients chemical composition of soil/rock unit weights permeability of water-bearing soils presence of shrink/swell soils (limits skin friction) geologic mapping including orientation and characteristics of rock discontinuities degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales) 	technique shaft shaft load test vane shear test CPT SPT (granular soils) dilatometer piezometers rock coring (RQD) geophysical testing	1-D Oedometer triaxial tests grain size distribution interface friction tests pH, resistivity tests permeability tests Atterberg Limits moisture content unit weight organic content collapse/swell potential tests rock uniaxial compression test and intact rock modulus point load strength test slake durability
Embankments and Embankment Foundations	settlement (magnitude & rate) bearing capacity slope stability lateral pressure internal stability borrow source evaluation (available quantity and quality of borrow soil) required reinforcement	subsurface profile (soil, ground water, rock) compressibility parameters shear strength parameters unit weights time-rate consolidation parameters horizontal earth pressure coefficients interface friction parameters pullout resistance geologic mapping including orientation and characteristics of rock discontinuities shrink/swell/degradation of soil and rock fill	nuclear density plate load test test fill CPT SPT (granular soils) dilatometer vane shear rock coring (RQD) geophysical testing	1-D Oedometer triaxial tests direct shear tests grain size distribution Atterborg Limits organic content moisture-density relationship hydraulic conductivity geosynthetic/soil testing shrink/swell slake durability unit weight
Excavations and Cut Slopes	slope stability bottom heave liquefaction dewatering lateral pressure soil softening/progressive failure pore pressures	 subsurface profile (soil, ground water, rock) shrink/swell properties unit weights hydraulic conductivity time-rate consolidation parameters shear strength of soil and rock (including discontinuities) geologic mapping including orientation and characteristics of rock discontinuities 	test cut to evaluate stand-up time piezometers CPT SPT (granular soils) vane shear dilatometer rock coring (RQD) in situ rock direct shear test geophysical testing	hydraulic conductivity grain size distribution Atterberg Limits triaxial tests direct shear tests moisture content slake durability rock uniaxial compression test and intact rock modulus point load strength test
Fill Walls/ Reinforced Soil Slopes	internal stability external stability external stability settlement horizontal deformation lateral earth pressures bearing capacity ehemical compatibility with soil and wall materials pore pressures behind wall borrow source evaluation (available quantity and quality of borrow soil)	subsurface profile (soil, ground water, rock) horizontal earth pressure coefficients interface shear strengths foundation soil/wall fill shear strengths compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) chemical composition of fill/foundation soils hydraulic conductivity of soils directly behind wall time-rate consolidation parameters geologic mapping including orientation and characteristics of rock discontinuities	 SPT (granular soils) CPT dilatometer vane shear piezometers test fill nuclear density pullout test (MSEW/RSS) rock coring (RQD) geophysical testing 	I-D Oedometer triaxial tests direct shear tests grain size distribution Atterberg Limits PH, resistivity tests moisture content organic content moisture-density relationships hydraulic conductivity
Cut Walls	internal stability excernal stability excavation stability dewatering chemical compatibility of wall/soil lateral earth pressure down-drag on wall pore pressures behind wall obstructions in retained soil	 subsurface profile (soil, ground water, rock) shear strength of soil horizontal earth pressure coefficients interface shear strength (soil and reinforcement) hydraulic conductivity of soil geologic mapping including orientation and characteristics of rock discontinuities 	test cut to evaluate stand-up time well pumping tests piezometers SPT (granular soils) CPT vane shear dilatometer pullout tests (anchors, nails) geophysical testing	triaxial tests direct shear grain size distribution Atterberg Limits PH, resistivity tests organic content hydraulic conductivity moisture content unit weight

Table 2.3: Summary of typical transportation project requirements and necessary subsurface information (adapted from Sabatini et al. 2002).

No matter the intended purpose of the subsurface investigations (i.e., planning versus design phase project), the development of a thorough subsurface exploration plan begins with office reconnaissance performed to identify any existing information regarding the project site (Mayne et al. 2002; ASTM D420; Caltrans 2015). The results from such an investigation can provide a wealth of geologic and historic information that will benefit subsequent planning of subsurface investigations and minimize surprises in the field (Mayne et al. 2002). For example, Sabatini et al. (2002) provides a useful flow chart to aid engineers in selecting appropriate properties for earth materials and includes a review of existing documents as the first step in that process (Fig. 2.1). Existing information regarding site conditions can be found within a number of potential data sources, many of which are publically accessible or available at a modest cost (Mayne et al. 2002):

- Prior subsurface investigations (historical data) from areas nearby the project site. Caltrans maintains an internal website for archiving geotechnical reports, laboratory tests, and boring logs [Digital Archive of Geotechnical Services (GeoDOG)].
- Construction records from prior projects at or nearby the site.
- Geologic and topographic maps, reports, and publications [available from the United States Geological Survey (USGS) and California Geological Survey, geological societies, university libraries and geology departments, Library of Congress, DOT libraries, public libraries, etc.]
- Flood zone maps [available from USGS, California Geological Survey, and/or the Federal Emergency Management Agency (FEMA)].
- Soil survey maps [e.g., Department of Agriculture Soil Conservation Service (SCS) Soil Maps]
- Aerial photographs (USGS, SCS, Earth Resource Observation System, Google Earth).
- Remote sensing images (LANDSAT, Skylab, and NASA).
- Environmental studies and ground water information [e.g., USGS, United States Environmental Protection Agency (EPA), California Department of Water Resources (DWR), etc.].
- Earthquake data, seismic hazard maps, and fault information (available from various agencies, including USGS, California Geological Survey, Earthquake Engineering Research Center (EERC), Earthquake Engineering Research Institute (EERI), National Earthquake Engineering Research Program (NEERP), Multidisciplinary Center of Earthquake Engineering Research (MCEER), Advanced Technology Council (ATC), Mid-America Earthquake Center (MAEC), and the Pacific Earthquake Engineering Center (PEER)]

In addition to the aforementioned sources, consultations with other geo-professionals who may have some experience with the site or nearby locations can prove very useful.



Figure 2.1: Flow chart for selecting appropriate engineering properties of soil and rock for use in design (Sabatini et al. 2002).

Once existing data has been reviewed, a reconnaissance site visit should be performed to better understand the geotechnical, topographic, and geological features of the site and to become knowledgeable of access and working conditions (e.g., traffic control requirements, proximity to nearby structures and utilities, presence of environmentally sensitive areas, etc.) (Mayne et al. 2002; Caltrans 2015). It may be necessary to perform multiple visits for more complicated site conditions. The goal is to develop a working preliminary model of the site that can guide the development of an appropriate subsurface investigation plan and the selection of potential design options (Sabatini et al. 2002). For example, a preliminary site model may note the presence of significant alluvial soils that may provide inaccurate "top of rock" estimates with traditional drilling procedures, prevent the use of certain in situ test methods, and potentially preclude the use of driven pile foundation designs (Sabatini et al. 2002). Or the preliminary site model could identify significantly heterogeneous strata across the site that must be better characterized by in situ testing or geophysical methods. Therefore, depending on the nature of the site and the project, it may be necessary to incorporate geophysical methods as part of the initial reconnaissance to better understand site subsurface conditions for subsequent explorations. Finally, the reconnaissance visit(s) can also be used to mark the site for utility clearance and to establish a benchmark for any future potential borings.

Following the review of existing data and the field reconnaissance of the site, a subsurface exploration plan can then be established that is best suited based on the project design requirements, previously available subsurface information, current site conditions, availability of equipment, and local practice. The types of subsurface investigation methods and spatial frequency with which they are performed will be tailored to the specific project needs. The subsurface investigation plan should also take into account anticipated needs for laboratory testing so that appropriate sampling of soils is performed. The subsurface conditions once the field investigations are initiated. Given the wide range in drilling, sampling, in situ testing, and geophysical testing techniques combined with the uniqueness of each site and project, a prescriptive approach is not advisable nor provided in this document (and other documents regarding subsurface investigations). Instead, the following sections provide a summary of items to consider when developing the subsurface exploration plan and the role of geophysical methods within that framework.

2.3.1.2 Subsurface Drilling and In Situ Testing

For a significant percentage of projects, subsurface exploration entails the drilling of boreholes to obtain information about the on-site earth materials. Borings can be used to obtain high quality "undisturbed" and lower quality "disturbed" samples for laboratory testing. To avoid issues related to sample disturbance, in situ testing may be performed within borings or as standalone tests to evaluate the earth materials properties. The focus of this section is to briefly summarize general topics related to subsurface drilling and in situ testing. Sampling techniques and the use of laboratory tests to determine engineering properties is presented in 2.3.1.3. It is assumed that the reader is familiar with typical subsurface drilling techniques such as auger borings and rotary wash techniques and in situ testing methods such as the standard penetration test (SPT), cone penetration test (CPT), flat plate dilatometer (DMT), and other similar techniques. For review, the reader is encouraged to review the various references available that discuss subsurface drilling techniques and in situ test methods in more detail (e.g., ASTM D4700; AASHTO 1988; Schmertmann 1988; Briaud 1989; USACE 2001; Briaud and Miran 2002; Mayne et al. 2002; Sabatini et al. 2002).

The use of subsurface drilling and in situ testing should not take a "one size fits all" approach, as many factors will influence decisions regarding drilling method, boring locations and depths, and number/types of in situ tests to perform. These factors include the proposed structure, geologic constraints, expected stratigraphy and heterogeneity, and access issues for drilling equipment, among others. Additionally, augmenting site explorations with geophysical methods can help fine tune the location, amount, and depths of drilling operations and in situ tests. In some cases, subsurface stratigraphy and material properties can be developed by supplementing limited laboratory testing (and associated drilling for samples) with rapid in situ test methods such as CPT and DMT. Therefore, drilling frequency and depth will be limited. General guidelines exist regarding minimum number of borings as well as depth to extend borings depending on project type (Tables 2.4 - 2.6). However, these guidelines are by no means definitive and they should be considered as initial recommendations because actual boring spacing and depth will be highly project- and site-dependent. In addition to general guidelines provided in Tables 2.4 - 2.6, ASTM standards exist regarding subsurface drilling and various in situ testing techniques. Relevant ASTM standards are summarized in Table 2.7.

Areas of Investigation	Recommended Boring Depth
Bridge Foundations* Highway Bridges	
1. Spread Footings	For isolated footings of breadth L_f and width $\leq 2B_f$, where $L_f \leq 2B_f$, borings shall extend a minimum of two footing widths below the bearing level.
	For isolated footings where $L_f \ge 5B_f$, borings shall extend a minimum of four footing widths below the bearing level.
	For $2B_f \le L_f \le 5B_f$, minimum boring length shall be determined by linear interpolation between depths of $2B_f$ and $5B_f$ below the bearing level.
2. Deep Foundations	In soil, borings shall extend below the anticipated pile or shaft tip elevation a minimum of 6 m, or a minimum of two times the maximum pile group dimension, whichever is deeper.
	For piles bearing on rock, a minimum of 3 m of rock core shall be obtained at each boring location to verify that the boring has not terminated on a boulder.
	For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.
Retaining Walls	Extend borings to depth below final ground line between 0.75 and 1.5 times the height of the wall. Where stratification indicates possible deep stability or settlement problem, borings should extend to hard stratum.
	For deep foundations use criteria presented above for bridge foundations.
Roadways	Extend borings a minimum of 2 m below the proposed subgrade level.
Cuts	Borings should extend a minimum of 5 m below the anticipated depth of the cut at the ditch line. Borings depths should be increased in locations where base stability is a concern due to the presence of soft soils, or in locations where the base of the cut is below groundwater level to determine the depth of the underlying pervious strata.
Embankments	Extend borings a minimum depth equal to twice the embankment height unless a hard stratum is encountered above this depth. Where soft strata are encountered which may present stability or settlement concerns the borings should extend to hard material.
Culverts	Use criteria presented above for embankments.
*Note: Taken from AA	SHTO Standard Specifications for Design of Highway Bridges

Table 2.4: Minimum recommendations	s for boring dep	ths (Mayne et al. 2002).
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Geotechnical Features	Boring Layout
Bridge Foundations	For piers or abutments over 30 m wide, provide a minimum of two borings.
	For piers or abutments less than 30 m wide, provide a minimum of one boring.
	Additional borings should be provided in areas of erratic subsurface conditions.
Retaining Walls	A minimum of one boring should be performed for each retaining wall. For retaining walls more than 30 m in length, the spacing between borings should be no greater than 60 m. Additional borings inboard and outboard of the wall line to define conditions at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage capacities should be considered.
Roadways	The spacing of borings along the roadway alignment generally should not exceed 60 m. The spacing and location of the borings should be selected considering the geologic complexity and soil/rock strata continuity within the project area, with the objective of defining the vertical and horizontal boundaries of distinct soil and rock units within the project limits.
Cuts	A minimum of one boring should be performed for each cut slope. For cuts more than 60 m in length, the spacing between borings along the length of the cut should generally be between 60 and 120 m.
	At critical locations and high cuts, provide a minimum of three borings in the transverse direction to define the existing geological conditions for stability analyses. For an active slide, place at least one boring upslope of the sliding area.
Embankments	Use criteria presented above for Cuts.
Culverts	A minimum of one boring at each major culvert. Additional borings should be provided for long culverts or in areas of erratic subsurface conditions.

Table 2.5: Minimum recommendations for boring layout (Mayne et al. 2002).

	Minimum Number of Investigation Points	
Application	and Location of Investigation Points	Minimum Depth of Investigation
Retaining walls	A minimum of one investigation point for ea retaining wall. For retaining walls more than 30 m length, investigation points spaced every 30 to 60 with locations alternating from in front of the wall behind the wall. For anchored walls, addition investigation points in the anchorage zone spaced at to 60 m. For soil-nailed walls, additional investigati points at a distance of 1.0 to 1.5 times the height of t wall behind the wall spaced at 30 to 60 m.	ch Investigate to a depth below bottom of wall between 1 and 2 in times the wall height or a minimum of 3 m into bedrock. Investigation depth should be great enough to fully penetrate to soft highly compressible soils (e.g. peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock).
Embankment Foundations	A minimum of one investigation point every 60 (erratic conditions) to 120 m (uniform conditions) embankment length along the centerline of t embankment. At critical locations, (e.g., maximu embankment heights, maximum depths of soft strata) minimum of three investigation points in the transver direction to define the existing subsurface conditio for stability analyses. For bridge approa embankments, at least one investigation point abutment locations.	m Investigation depth should be, at a minimum, equal to twice the embankment height unless a hard stratum is encountered above this depth. If soft strata is encountered extending to a depth greater than twice the embankment height, a investigation depth should be great enough to fully penetrate the soft strata into competent material (e.g., stiff to hard ns cohesive soil, compact to dense cohesionless soil, or bedrock).
Cut Slopes	A minimum of one investigation point every 60 (erratic conditions) to 120 m (uniform conditions) slope length. At critical locations (e.g., maximum c depths, maximum depths of soft strata) a minimum three investigation points in the transverse direction define the existing subsurface conditions for stabili analyses. For cut slopes in rock, perform geolog mapping along the length of the cut slope.	m Investigation depth should be, at a minimum, 5 m below the minimum elevation of the cut unless a hard stratum is encountered below the minimum elevation of the cut. Investigation depth should be great enough to fully penetrate through soft strata into competent material (e.g., stiff to hard cohesive soil, compact to dense cohesionless soil, or bedrock). In locations where the base of cut is below ground-water level, increase depth of investigation as needed to determine the depth of underlying pervious strata.
Application	Minimum Number of Investigation Points and Location of Investigation Points	Minimum Depth of Investigation
Shallow Foundations	For substructure (e.g., piers or abutments) widths less than or equal to 30 m, a minimum of one investigation point per substructure. For substructure widths greater than 30 m, a minimum of two investigation points per substructure. Additional investigation points should be provided if erratic subsurface conditions are encountered.	Depth of investigation should be: (1) great enough to fully penetrate unsuitable foundation soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g. stiff to hard cohesive soil, compact to dense cohesionless soil or bedrock) and; (2) at least to a depth where stress increase due to estimated footing load is less than 10% of the existing effective overburden stress and; (3) if bedrock is encountered before the depth required by item (2) above is achieved, investigation depth should be great enough to penetrate a minimum of 3 m into the bedrock, but rock investigation should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities.
Deep Foundations	For substructure (e.g., bridge piers or abutments) widths less than or equal to 30 m, a minimum of one investigation point per substructure. For substructure widths greater than 30 m, a minimum of two investigation points per substructure. Additional investigation points should be provided if erratic subsurface conditions are encountered.	In soil, depth of investigation should extend below the anticipated pile or shaft tip elevation a minimum of 6 m, or a minimum of two times the maximum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine- grained soils, and loose coarse-grained soils to reach hard or dense materials.
	Due to large expense associated with construction of rock-socketed shafts, conditions should be confirmed at each shaft location.	For piles bearing on rock, a minimum of 3 m of rock core shall be obtained at each investigation point location to verify that the boring has not terminated on a boulder.
		For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

Table 2.6: Minimum recommendations for number and depths of borings (Sabatini et al. 2002).

Standard	Title
D420	Standard Guide to Site Characterization for Engineering Design and Construction Purposes
D653	Standard Terminology Relating to Soil, Rock, and Contained Fluids
D1452	Standard Practice for Soil Exploration and Sampling by Auger Borings
D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
D2113	Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration
D2487	Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
D2488	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
D2573	Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils
D2944	Standard Practice of Sampling Processed Peat Materials
D3213	Standard Practices for Handling, Storing, and Preparing Soft Intact Marine Soil
D3282	Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes
D3441	Standard Test Method for Mechanical Cone Penetration Tests of Soil
D3550	Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils
D4083	Standard Practice for Description of Frozen Soils (Visual-Manual Procedure)
D4220	Standard Practices for Preserving and Transporting Soil Samples
D4394	Standard Test Method for Determining In Situ Modulus of Deformation of Rock Mass Using Rigid Plate Loading Method
D4395	Standard Test Method for Determining In Situ Modulus of Deformation of Rock Mass Using Flexible Plate Loading Method
D4429	Standard Test Method for CBR (California Bearing Ratio) of Soils in Place
D4544	Standard Practice for Estimating Peat Deposit Thickness
D4553	Standard Test Method for Determining In Situ Creep Characteristics of Rock
D4554	Standard Test Method for In Situ Determination of Direct Shear Strength of Rock Discontinuities
D4555	Standard Test Method for Determining Deformability and Strength of Weak Rock by an In Situ Uniaxial Compressive Test
D4623	Standard Test Method for Determination of In Situ Stress in Rock Mass by Overcoring Method—USBM Borehole Deformation Gauge
D4630	Standard Test Method for Determining Transmissivity and Storage Coefficient of Low- Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test
D4631	Standard Test Method for Determining Transmissivity and Storativity of Low Permeability Rocks by In Situ Measurements Using Pressure Pulse Technique
D4633	Standard Test Method for Energy Measurement for Dynamic Penetrometers
D4700	Standard Guide for Soil Sampling from the Vadose Zone
D4719	Standard Test Methods for Prebored Pressuremeter Testing in Soils
D4729	Standard Test Method for In Situ Stress and Modulus of Deformation Using Flatjack Method
D4750	Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)

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Table 2.7: Relevant ASTM standards regarding subsurface drilling, sampling, and in situ testing.

Standard	Title
D4879	Standard Guide for Geotechnical Mapping of Large Underground Openings in Rock
D4971	Standard Test Method for Determining In Situ Modulus of Deformation of Rock Using Diametrically Loaded 76-mm (3-in.) Borehole Jack
D5079	Standard Practices for Preserving and Transporting Rock Core Samples
D5092	Standard Practice for Design and Installation of Groundwater Monitoring Wells
D5195	Standard Test Method for Density of Soil and Rock In-Place at Depths Below Surface by Nuclear Methods
D5220	Standard Test Method for Water Mass per Unit Volume of Soil and Rock In-Place by the Neutron Depth Probe Method
D5434	Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock
D5730	Standard Guide for Site Characterization for Environmental Purposes With Emphasis on Soil, Rock, the Vadose Zone and Groundwater
D5778	Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils
D5878	Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes
D5911	Standard Practice for Minimum Set of Data Elements to Identify a Soil Sampling Site
D6032	Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core
D6066	Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential
D6067	Standard Practice for Using the Electronic Piezocone Penetrometer Tests for Environmental Site Characterization
D6151	Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling
D6168	Standard Guide for Selection of Minimum Set of Data Elements Required to Identify Locations Chosen for Field Collection of Information to Describe Soil, Rock, and Their Contained Fluids
D6169	Standard Guide for Selection of Soil and Rock Sampling Devices Used With Drill Rigs for Environmental Investigations
D6282	Standard Guide for Direct Push Soil Sampling for Environmental Site Characterizations
D6286	Standard Guide for Selection of Drilling Methods for Environmental Site Characterization
D6517	Standard Guide for Field Preservation of Groundwater Samples
D6519	Standard Practice for Sampling of Soil Using the Hydraulically Operated Stationary Piston Sampler
D6635	Standard Test Method for Performing the Flat Plate Dilatometer
D6911	Standard Guide for Packaging and Shipping Environmental Samples for Laboratory Analysis
D6914	Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices
D6938	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)
D7015	Standard Practices for Obtaining Intact Block (Cubical and Cylindrical) Samples of Soils
D7380	Standard Test Method for Soil Compaction Determination at Shallow Depths Using 5-lb (2.3 kg) Dynamic Cone Penetrometer

Table 2.7 (cont.): Relevant ASTM standards regarding subsurface drilling, sampling, and in situ testing.

In addition to sampling and in situ testing, borings performed as part of subsurface drilling operations can potentially be used for borehole geophysical methods (e.g., down-hole seismic, cross-hole seismic, televiewer, borehole radar, etc.). However, special care must be taken when constructing these boreholes as the requirements for high quality data may necessitate different techniques and extra precautions during drilling. For example, in cases of soft soils and running sands, the borehole will need to be stabilized prior to borehole geophysical tests, otherwise the user runs the risk of losing equipment due to caving of the borehole. Often, this entails the installation of a rigid casing such as PVC or steel piping to line the borehole walls. In such cases, the quality of the geophysical test results is highly dependent on the coupling between the borehole wall and casing. Any gaps between the borehole wall and casing must be filled with a suitable grout mixture to ensure adequate coupling. ASTM standards exist for a number of borehole geophysical methods that provide directions regarding borehole construction specifically for these geophysical operations (e.g., ASTM D4428). A list of relevant ASTM standards for borehole and surface geophysical methods typically used in geotechnical subsurface exploration operations is provided in Table 2.8.

Standard	Title
C1383	Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method
D4428	Standard Test Methods for Crosshole Seismic Testing
D4695	Standard Guide for General Pavement Deflection Measurements
D4748	Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short- Pulse Radar
D4788	Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography
D5518	Standard Guide for Acquisition of File Aerial Photography and Imagery for Establishing Historic Site-Use and Surficial Conditions
D5753	Standard Guide for Planning and Conducting Borehole Geophysical Logging
D5777	Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation
D5882	Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations
D6167	Standard Guide for Conducting Borehole Geophysical Logging: Mechanical Caliper
D6274	Standard Guide for Conducting Borehole Geophysical Logging – Gamma
D6429	Standard Guide for Selecting Surface Geophysical Methods
D6430	Standard Guide for Using the Gravity Method for Subsurface Investigation

Table 2.8: Relevant ASTM standards regarding geophysical and non-destructive testing.

Standard	Title
D6431	Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation
D6432	Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface
	Investigation
D6639	Standard Guide for Using the Frequency Domain Electromagnetic Method for Subsurface Investigations
D6726	Standard Guide for Conducting Borehole Geophysical Logging - Electromagnetic Induction
D6727	Standard Guide for Conducting Borehole Geophysical Logging – Neutron
D6758	Standard Test Method for Measuring Stiffness and Apparent Modulus of Soil and Soil- Aggregate In-Place by Electro-Mechanical Method
D6760	Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing
D6780	Standard Test Method for Water Content and Density of Soil In situ by Time Domain Reflectometry (TDR)
D6820	Standard Guide for Use of the Time Domain Electromagnetic Method for Subsurface Investigation
D7046	Standard Guide for Use of the Metal Detection Method for Subsurface Exploration
D7128	Standard Guide for Using the Seismic-Reflection Method for Shallow Subsurface Investigation
D7383	Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations
D7400	Standard Test Methods for Downhole Seismic Testing
D7698	Standard Test Method for In-Place Estimation of Density and Water Content of Soil and Aggregate by Correlation with Complex Impedance Method
D7759	Standard Guide for Nuclear Surface Moisture and Density Gauge Calibration
D7830	Standard Test Method for In-Place Density (Unit Weight) and Water Content of Soil Using an Electromagnetic Soil Density Gauge
E1543	Standard Practice for Noise Equivalent Temperature Difference of Thermal Imaging Systems
E2583	Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)
G 57	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four- Electrode Method

Table 2.8 (cont.): Relevant ASTM standards regarding geophysical and non-destructive testing.

Finally, on site drilling operations can potentially be used as a seismic source for geophysical testing (Fig. 2.2). Such seismic-while-drilling (SWD) techniques were originally developed in the 1980's by the petroleum engineering industry for application to oil and gas exploration (e.g., Angeleri et al. 1990). SWD typically consists of using surficial sensors to record the waveforms caused by a rotary-cone bit as each tooth impacts and chisels the rock during drilling operations. A reference sensor at the top of the drill string is also used to record the source signature. The reference sensor response is cross-correlated to the surface sensor response to compute a travel time. Data analysis essentially proceeds in a similar manner to vertical seismic profiling (Gal'perin 1974; Hardage 1985), a form of downhole seismic testing, since SWD reverses the source and receiver positions (Rector and Marion 1991). Literature on SWD is plentiful within

the exploratory geophysics and petroleum engineering community (e.g., Rector et al. 1989; Rector and Marion 1991; Asanuma and Niitsuma 1992; Haldorsen et al. 1995; Petronio et al. 1999; Malusa et al. 2002; Rocca et al. 2005; Anchliya 2006; Reppert 2013) and Poletto and Miranda (2004) contains a thorough discussion of the technique and history of SWD. However, its usage has declined as drilling operations for petroleum sources have increasingly utilized poly-diamond-composite (PDC) bits. PDC bits scrape through the rock, which proves to be a less effective seismic source for SWD compared to the impact and chiseling action of rotary-cone bits (Poletto and Miranda 2004). Additionally, the technique has seen little usage for geotechnical purposes, likely due to expense and the differences in drilling operations through softer earth materials encountered near the surface.



Figure 2.2: Schematic of the seismic-while-drilling (SWD) technique (Rocca et al. 2005).

2.3.1.3 Sampling and Laboratory Testing

As previously noted, borings are routinely used in geotechnical subsurface exploration to sample the on-site earth materials and evaluate their engineering properties via laboratory testing. The focus of this section is to briefly summarize general topics related to subsurface sampling and laboratory testing, particularly as relevant to geophysical methods. As before, it is assumed that the reader is familiar with typical subsurface drilling and sampling techniques and is encouraged to review the various references available that discuss them in more detail (e.g., NAVFAC 1986; AASHTO 1988; USACE 2001; Mayne et al. 2002; Sabatini et al. 2002).

When designing and performing a subsurface exploration plan involving subsurface sampling, special care must be exercised when prescribing the sampling method to ensure enough appropriate samples are available for laboratory testing. The quality of the sample is highly dependent on sampling technique, which also dictates the suitability of different laboratory tests for a given sample. For example, highly disturbed sampling techniques [e.g., augering (ASTM D1452), split-spoon sampling via SPT (ASTM D1586, D3550), etc.] can completely destroy the in situ soil structure/fabric. Such samples are only suitable for index type laboratory tests such as sieve analysis, plasticity testing, compaction, and similar tests. Undisturbed soil samples are required for performing laboratory strength and consolidation testing. Since it is impossible to collect truly undisturbed samples, the goal of high-quality undisturbed sampling in geotechnical practice is to minimize changes in soil structure, moisture content, void ratio, and chemical composition during sampling. This is typically accomplished with thin-walled (Shelby) tube sampling (ASTM D1587), though alternative methods exist such as piston (ASTM D6519) and pitcher samplers and block sampling techniques (ASTM D7015). Selection of sampling technique is a function of geologic conditions, depth and spacing of boreholes, and project needs. For example, undisturbed sampling may not be necessary for all boreholes in cases when they are closely spaced and when the subsurface stratigraphy is relatively uniform. For planning phase projects, sampling may not be necessary at all, particularly if geophysical methods can be used to provide enough level of detail regarding site conditions to perform preliminary design assessments. Finally, certain soils are particularly difficult to sample (e.g., alluvial soils with significant gravel and/or stone content, highly cemented granular soils, etc.) and may require alternative sampling techniques outside of typical split-barrel or Shelby tube sampling. Geophysical techniques may prove useful in such circumstances where alternative sampling techniques are required and cost-prohibitive. Sampling of rock is typically accomplished using a number of rock coring techniques/equipment (as summarized in ASTM D2113), with doubletube core barrels commonly employed and wireline techniques preferred for their efficiency.

In terms of frequency of sampling, many factors will affect selection of appropriate sampling depths and intervals, including site conditions, nature of the project, and required design

parameters. Common practice typically involves sampling continuously or with a very small (i.e., 0.75 m) interval in the upper 3 m of a site, sampling at 1.5 m intervals up to 30 m, and increasing sampling interval to every 3 m at depths greater than 30 m (Mayne et al. 2002; Sabatini et al. 2002). However, this selection of sampling intervals is by no means definitive given the wide ranges in site geologic conditions and project requirements. Generally speaking, site subsurface conditions that are more homogeneous will require fewer samples for testing. For sites with soils that may prove difficult to sample, increasing the sampling frequency should be considered to offset the number of samples that may be unusable in the laboratory. As a general guideline, a minimum of one undisturbed sample should be taken for each fine-grained stratum (Sabatini et al. 2002). Therefore, in profiles where the strata are relatively thin and change frequently with depth, sampling intervals may need to increase. However, this must be balanced against design requirements. For example, frequent sampling may not be necessary in granular soils for designs where settlement is of particular concern. Geophysical methods can be used to rapidly provide information regarding site stratigraphy as part of the development of an initial site subsurface model and can therefore help to fine tune the selection of sampling frequency. For example, seismic methods such as seismic refraction can identify areas of significant heterogeneity and electrical methods such as resistivity imaging can identify thin lenses of finegrained cohesive soils. These observations can be used to select an appropriate sampling frequency to ensure the necessary strata are sampled.

Once sampled, laboratory testing can be used to evaluate earth material properties as appropriate for design purposes. This includes a number of index tests (e.g., plasticity indices, unit weight, water content, etc.) as well as strength and consolidation tests. This document assumes the reader is familiar with these typical laboratory tests as used to evaluate soil and rock properties, and Table 2.9 provides a list of relevant ASTM standards. Any number of introductory soil and rock mechanics texts and/or relevant subsurface investigation literature (e.g., NAVFAC 1986; AASHTO 1988; USACE 2001; Mayne et al. 2002; Sabatini et al. 2002) can provide additional background information as necessary. A laboratory testing program must be developed in collaboration with the subsurface exploration plan to ensure drilling operations provide adequate samples of the appropriate type for laboratory testing. Table 2.3 highlights typical laboratory testing requirements for a range of transportation-related design projects. In situ testing and geophysical methods can be used to provide complementary information and

avoid sampling disturbance effects. As previously noted, sampling disturbance is a major concern regarding selection of appropriate samples. Once sampling has been performed, care must also be exercised to ensure the samples are handled appropriately to prevent any additional disturbances (e.g., ASTM D3213, D4220, and D6911). Finally, it should be noted that laboratory testing can be performed to evaluate geophysical parameters of earth materials in lieu of performing geophysical methods in the field. For example, ultrasonic testing (ASTM D2845) can be performed using bender element on intact soil and rock samples to evaluate shear wave and/or primary wave velocity (as a proxy for elastic moduli parameters). Specialized laboratory equipment such as the resonant column test (ASTM D4015) can also be used to establish modulus and damping parameters. Depending on the geophysical method considered for field testing, resolution may be limited for deep thin strata. Sampling and application of the aforementioned laboratory tests may therefore provide more thorough information for the purposes of design, though the issue of sampling disturbance must be considered. Section 3.5 of this document provides more detailed discussion of laboratory approaches to evaluate geophysical parameters.

Standard	Title
D421	Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
D422	Standard Test Method for Particle-Size Analysis of Soils
D698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft ³ (600 kN-m/m ³))
D854	Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer
D1140	Standard Test Methods for Determining the Amount of Material Finer than 75- μ m (No. 200) Sieve in Soils by Washing
D1557	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft ³ (2,700 kN-m/m ³))
D1997	Standard Test Method for Laboratory Determination of the Fiber Content of Peat Samples by Dry Mass
D2166	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
D2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D2435	Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
D2845	Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock

Table 2.9: Relevant ASTM standards regarding laboratory testing for earth materials.

Standard	Title
D2850	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
D2936	Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens
D3080	Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions
D3967	Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens
D3999	Standard Test Methods for the Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus
D4015	Standard Test Methods for Modulus and Damping of Soils by Fixed-Base Resonant Column Devices
D4186	Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading
D4221	Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer
D4253	Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table
D4254	Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
D4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D4373	Standard Test Method for Rapid Determination of Carbonate Content of Soils
D4427	Standard Classification of Peat Samples by Laboratory Testing
D4543	Standard Practices for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerances
D4546	Standard Test Methods for One-Dimensional Swell or Collapse of Soils
D4643	Standard Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating
D4648	Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil
D4767	Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils
D4829	Standard Test Method for Expansion Index of Soils
D4943	Standard Test Method for Shrinkage Factors of Soils by the Wax Method
D4959	Standard Test Method for Determination of Water (Moisture) Content of Soil By Direct Heating
D5084	Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
D5311	Standard Test Method for Load Controlled Cyclic Triaxial Strength of Soil
D5550	Standard Test Method for Specific Gravity of Soil Solids by Gas Pycnometer
D5607	Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force
D5856	Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter
D6467	Standard Test Method for Torsional Ring Shear Test to Determine Drained Residual Shear Strength of Cohesive Soils

Table 2.9 (cont.): Relevant ASTM standards regarding laboratory testing for earth materials.

Standard	Title
D6528	Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils
D6913	Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis
D7012	Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures
D7070	Standard Test Methods for Creep of Rock Core Under Constant Stress and Temperature
D7181	Method for Consolidated Drained Triaxial Compression Test for Soils
D7263	Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens
D7608	Standard Test Method for Torsional Ring Shear Test to Determine Drained Fully Softened
	Shear Strength and Nonlinear Strength Envelope of Cohesive Soils (Using Normally
	Consolidated Specimen) for Slopes with No Preexisting Shear Surfaces

Table 2.9 (cont.): Relevant ASTM standards regarding laboratory testing for earth materials.

2.3.1.4 Role of Geophysics

As has been previously noted, geophysical methods can play an integral role at various junctures in the subsurface investigation process because of their ability to quickly provide information over a much larger area than subsurface drilling, in situ testing, and laboratory testing of acquired samples. For planning phase projects (i.e., K and 0 phase), subsurface investigations involving drilling, in situ testing, and geophysics are not typically performed (Caltrans 2015). However, should some form of subsurface investigation prove necessary or highly beneficial, geophysics may potentially provide all the information necessary for planning phase projects. In the initial stages of design phase projects (i.e., 1 phase) during which the subsurface exploration plan is being developed, geophysical methods can aid in tailoring any drilling operations and in situ tests based on site subsurface conditions. For example, during initial review of existing sources of data, previous geophysical reports can serve to highlight various aspects of subsurface conditions in and around the project site. Additionally, depending on the nature of the site and project, the initial site reconnaissance can also serve as an opportunity to perform some rapid geophysical tests (e.g., seismic refraction) to establish baseline subsurface conditions. In this context, often the role of geophysics is more qualitative, whereby engineering properties of soil and rock are not necessarily estimated using geophysical measurements. Instead, geophysical methods provide a means of rapid and thorough visualization of site subsurface conditions, including the location of utilities and other embedded objects that could affect drilling operations, depth to water table, and general stratigraphy.
In the latter stages of subsurface exploration plans during which field operations have been initiated, geophysical methods serve a more quantitative role where they can be used to estimate engineering properties of subsurface materials. The main focus of this document is to provide feedback regarding the various relationships that exist where geophysical measurements can be correlated to such engineering properties for soil and rock. Depending on the nature of the project and availability of existing subsurface data, geophysical methods can be combined with the relationships summarized in this document to potentially replace typical drilling, sampling, and in situ testing procedures. In other cases, geophysical methods in isolation may not sufficiently provide conclusive information to adequately constrain designs. The uncertainty and scatter inherent in the relationships between geophysical measurements and engineering properties of soil and rock may prove too high for sensitive projects or the results from geophysical methods may be too ambiguous. In such cases, geophysical methods can be used to augment typical drilling, sampling, and in situ testing procedures. For example, borehole geophysical techniques can be used selectively at certain borings to provide complementary information regarding different stratigraphic units. Surface geophysical methods can also augment drilling operations by providing a means to bridge the gap between successive boring locations (sometimes in near real-time as in SWD). Geophysical methods can also be combined with the results from other subsurface investigation techniques to allow for the development of site-specific correlations from which to assign soil and rock properties. These site-specific correlations will suffer from less uncertainty and scatter in relationship to those presented in this document. Moreover, the broad area of coverage and rapid nature of data acquisition can provide a much larger amount of information about the site for a modest cost, particularly when geophysical methods are correlated to site borings and the costs associated with surface wave methods are compared to the implementation of additional borings. As with drilling, sampling, and laboratory operations, ASTM standards have been developed that provide guidance regarding appropriate implementation of various geophysical methods. A summary of relevant ASTM geophysical standards is provided in Table 2.8.

3. APPLICATIONS OF GEOPHYSICAL METHODS

This chapter discusses the various earth material properties that are obtained from several of the methods introduced in Chapter 2. The material presented includes a summary of various case histories from the literature where these geophysical methods were utilized as well as the results from comprehensive projects to relate various parameters to geophysical measurements. The initial focus of the chapter is on earth material properties that are broad in scope and applicable across a number of geotechnical applications (e.g., void ratio, grading factor, etc.). The latter sections focus specifically on the use of geophysical measurements to obtain shear wave velocity for seismic design purposes.

3.1 APPLICATIONS RELATED TO GEOTECHNICAL PROPERTY MEASUREMENTS

As discussed in Chapter 2, traditional subsurface exploration operations often rely on drilling to sample the subsurface and estimate geotechnical earth properties necessary for geotechnical design (e.g., unit weight, void ratio, water content, strength, compressibility, etc.). Geophysical methods, if utilized, are deployed to augment drilling operations and to provide qualitative assessments of subsurface conditions by delineating boundaries between strata (e.g., locate voids, top of rock, etc.). However, geophysical methods do allow for the determination of earth material properties in addition to evaluating subsurface geometry. These measurements can occur rapidly over a significant area of investigation and can reduce dependency on traditional drilling and sampling approaches for site subsurface exploration. The following sections describe the earth material properties that can be determined based on the corresponding geophysical measurements. Care should be exercised with these relationships as many are highly empirical. Citations are provided to allow the reader to locate the databases used when developing the empirical expressions provided in this study. Should the site conditions significantly differ from the conditions established in the databases, it is recommended that site-specific correlations be established using the provided relationships as motivation for an appropriate functional form.

3.1.1 ELASTIC PARAMETERS

More than a quarter of DOT use of geophysical methods implements methods based on seismic wave propagation (Sirles 2006) (Fig. 1.5). This is unsurprising since wave velocity (P-wave and/or S-wave) is the primary measurement from seismic methods, and it is possible to obtain

the corresponding elastic parameters of the earth material from these measurements. The velocities of these waves are directly linked to the stiffness and density of the material by the following relationships in solid mechanics:

$$V_p = \sqrt{\frac{M}{\rho}} = \sqrt{\frac{B + \frac{4}{3}G}{\rho}}$$
(3-1)

$$V_s = \sqrt{\frac{G}{\rho}}$$
(3-2)

where V_P is the P-wave velocity of the material, V_S is the S-wave velocity, ρ is the density of the material, M is the constrained modulus, B the bulk modulus, and G the shear modulus. Young's modulus (E) can be derived from knowledge of the bulk/constrained modulus and shear modulus (and/or Poisson' ratio, v):

$$E = \frac{G(3M - 4G)}{M - G} = \frac{9BG}{3B + G} = 2G(1 + \nu)$$
(3-3)

Poisson's ratio can be derived based on the ratio of V_P to V_S :

$$\nu = 0.5 \frac{\left(\frac{V_p}{V_s}\right)^2 - 2}{\left(\frac{V_p}{V_s}\right)^2 - 1}$$
(3-4)

Given Eqs. 3-1 to 3-4, elastic parameters of earth materials can therefore be estimated based on velocities measured using seismic methods (with assumptions regarding mass density of the soil/rock). There are a number of situations where these moduli and Poisson's ratio are important for geotechnical purposes (e.g., estimating immediate settlements due to foundation loads, estimating small-strain stiffness of soils for dynamic analysis, numerical modeling, etc.). Moreover, the moduli themselves can often be related to a number of important engineering

parameters including soil strength (e.g., undrained shear strength of a clay). Discussion of the specific relationships that have been developed to relate soil moduli and other soil parameters is outside the scope of this reference document, and readers are encouraged to review textbooks related to soil mechanics (e.g., Lambe and Whitman 1979; Holtz and Kovacs 1981) as well as various manuals that have compiled such relationships (e.g., EPRI EL-6800 manual, Kulhawy and Mayne 1990). Seismic methods provide an approach to directly estimate elastic moduli in situ rather than rely on laboratory testing on potentially disturbed samples (highly likely for sands unless freezing techniques are used) or on empirical relationships.

3.1.2 STRENGTH PARAMETERS

Strength parameters of earth materials are arguably one of the most important parameters affecting geotechnical design. Knowledge of soil and/or rock strength is vital to many geotechnical projects, including the design of foundations, retaining systems, and slopes. As noted in 3.1.1, geophysical methods that generate mechanical waves inherently measure small-strain (e.g., $\gamma \leq 10^{-3}$ %) parameters such as shear modulus. Typically, shear strength of earth materials is a large strain phenomenon (e.g., $\gamma \approx 1 - 30$ %). However, many of the factors that affect small strain stiffness share a physical link with large strain phenomena. For example, shear strength of a sand is largely governed by void ratio and confining stress (e.g., Rowe 1962; Lambe and Whitman 1979), which also affect V_{5} . This forms the basis of a number of relationships between elastic moduli and soil strength (as noted in 3.1.1). However, a handful of studies have developed direct relationships between small strain and large strain phenomena. The following sections describe the results and proposed relationships from such studies for both soil and rock.

3.1.2.1 Shear Strength of Soils

A significant amount of research has been performed to develop relationships between geophysical measurements of small strain stiffness (i.e., wave velocities) and strength parameters of sands and clays. Much of the work has exploited the fact that both small strain stiffness and strength are affected by void ratio, effective stress, stress history, soil fabric, age, and degree of cementation among other factors (Guadalupe et al. 2013). Much of the research

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has focused on establishing relationships between V_s and undrained shear strength (s_u) of clays, particularly in marine and/or offshore applications (Table 3.1).

Study/reference	Type of clays	V_s (m/s) or G_{max} (kPa)	s_u determined from
Larsson and Mulabdic (1991)	Swedish (10) and Norwegian (4) sites. Medium-high plasticity	$G_{\max} = \left(\frac{208}{I_p} + 250\right) s_u$	Unspecified
Larsson and Mulabdic (1991)	Swedish (10) and Norwegian (4) sites. Low-plastic clays to high-plastic clayey organic soils	$G_{\max} = 504 \cdot s_u / w_L$	Unspecified
Dickenson (1994)	San Francisco Bay clay	$V_s = 23 s_u^{0.475}$	Fall cone tests
Blake and Gilbert (1997)	Offshore NW United States (55 tests)	$s_u = 1.87 V_s^{1.12}$	Triaxial
Ashford et al. (1997)	Bangkok clays (13 sites)	$V_s = 23 s_u^{0.475}$	Unspecified
Likitlersuang and Kyaw (2010) and Likitlersuang et al. (2013)	Bangkok clays (3 sites) based on downhole and MASW, respectively	$V_s = 187 (\frac{s_u}{p_a})^{0.372}, V_s = 228 (\frac{s_u}{p_a})^{0.510}$	Unspecified
Andersen (2004)	Normally consolidated clays	$\frac{G_{\text{max}}}{s^{\text{DSS}}} = 325 + 55 / (\frac{I_p}{100})^2$	DSS
Andersen (2004)	Sensitive and quick clays (remolded strength; $s_{ur} < 0.5$ kPa)	$\frac{G_{\max}}{s_{x}^{DSS}} = 800$ to 900	DSS
Yun et al. (2006)	Gulf of Mexico (38 tests)	$V_s = 19.4 s_u^{0.36}$	Unspecified
Kulkarni et al. (2010)	Indian coastal soils (130 tests, $R^2 = 0.82$)	$s_u = 5 \times 10^{-4} V_s^{2.5}$	Unconsolidated undrained triaxial
Taboada et al. (2013)	Bay of Campeche clay	$V_s = 31 s_u^{0.414}$	Unconsolidated undrained triaxial and in situ vane tests
Baxter et al. (2015), Baffer (2013)	Gulf of Mexico clay, Presumpscot clay (Gulf of Maine), and organic silt	Follows same relationship with I_p as proposed by Andersen (2004)	DSS
Agaiby and Mayne (2015)	Worldwide soils (360 tests, $R^2 = 0.76$)	$s_u = 0.152 V_s^{1.142}$	Triaxial compression
Agaiby and Mayne (2015)	Worldwide soils (362 tests, $R^2 = 0.87$	$s_u = 0.038 V_s^{1.063} I_p^{0.14} \text{OCR}^{0.31} e_0^{0.07} \sigma_{v0}^{\prime 0.23}$	Triaxial compression
Andersen (2015)	Worldwide soils I_p in range 10–100%	$\frac{G_{\text{max}}}{s_w^{\text{DSS}}} = \left[30 + \frac{300}{(\frac{I_p}{100} + 0.03)}\right] \text{OCR}^{-0.25}$	DSS

Table 3.1: Examples of available correlations between the s_u of clays and V_s or G_{max} (L'Heureux and Long 2017).

In many cases, the relationship between s_u and V_s (or G_{max}) is expressed using an exponential functional form:

$$V_s = a s_u^b \tag{3-5}$$

where *a* and *b* are coefficients that result from the regression analysis. In establishing these relationships (Table 3.1), multiple approaches have been used to measure both the V_s (e.g., in situ geophysical methods such as downhole seismic, laboratory bender element testing, etc.) and the corresponding s_u (e.g., triaxial, simple shear, Fall cone test, etc.). An example of the relative amount of scatter in these relationships is provided in Figs. 3.1 – 3.3. Fig. 3.1 plots the Agaiby and Mayne (2015) V_{s} - s_u variation for normally consolidated and/or lightly overconsolidated clays derived from a database of 31 sites across the world (360 total measurements). Fig. 3.2 plots the Levesques et al. (2007) V_{s} - s_u variation for intact post-glacial clays from eastern Canada and the North Sea. It should be noted that most of the proposed relationships relate V_s to s_u directly in an empirical manner. However, a handful of empirical

relationships also attempt to include the effects of other clay parameters that affect s_u , including Plasticity Index (*PI*), *OCR*, and/or void ratio (e.g., Andersen 2015; Agaiby and Mayne 2015).



Figure 3.1: Relationships between triaxial compression s_u and downhole V_s for: (a) NC to LOC intact clays; and (b) intact NC, LOC, to OC and HOC fissured clays (Agaiby and Mayne 2015).



Figure 3.2: Relationship between s_u and V_s for intact post-glacial clays from eastern Canada and the North Sea (Levesques et al. 2007).



Figure 3.3: Relationships between s_u and V_s for Norwegian clays: (a) CAUC triaxial tests; and (b) CAUE triaxial tests (L'Heureux and Long 2017).

Less research is available in the literature that discusses the estimation of shear strength of sands. A handful of studies have done so for grouted or lightly cemented sands (Sharma et al. 2011; Lee et al. 2014) where the strength parameter of interest is often the uniaxial compressive strength for undrained loading. The functional form for estimating uniaxial compressive strength is typically an exponential or power function similar to those proposed for use with concrete and/or bedrock and similar to relationships proposed between s_u and V_s for clays (Eq. 3-5):

$$UCS = ae^{bV_p}$$
(3-6)

$$UCS = aV_p^b \tag{3-7}$$

where UCS is the uniaxial compressive strength of the cemented sand, a and b are regression coefficients, and V_P is the compression wave velocity of the cemented sand. However, a handful of studies have examined drained strength of sands. For example Cha and Cho (2007) developed a methodology that estimates the drained friction angle (φ) of a sandy soil using V_s as measured using field seismic methods (suspension PS logging in their study). The method utilized the inherent link between φ and void ratio at a given stress. This relationship was established for the four sandy soils tested in their study using oedometer and direct shear testing. A correlation was then developed for the variation of V_s with vertical effective stress at the maximum and minimum possible void ratios (e_{max} and e_{min}) for each of the sandy soils. This correlation was obtained using bender element testing (3.4.2) in the oedometer as the sandy soils were reconstituted to specific e values. A linear variation for V_s was assumed between e_{max} and e_{min} at a given stress so that V_s could be computed for any given e for that stress. Combining these relationships together, an estimate for φ in the field proceeded as follows: (1) estimate vertical effective stress profile with depth; (2) measure field V_s using PS logging; (3) estimate e using the field V_s value; (4) estimate φ based on the known vertical effective stress at a given depth and from the e computed using V_s. Cha and Cho (2007) found reasonable agreement between the estimated φ profile with depth and the profile estimating using in situ testing results (i.e., corrected blowcounts) (Fig. 3.4).



Figure 3.4: φ estimates using SPT and V_s at one site in Korea (Cha and Cho 2007).

Other researchers have proposed empirical relationships between G_{max} (as related to V_s from Eq. 3-2) and the maximum principal stress at failure (σ'_{1f}) based on transducer or bender element testing and triaxial strength testing in the laboratory. Such relationships have been proposed for clays (e.g., Baxter et al. 2015), lightly cemented sand (e.g., Sharma et al. 2011), calcareous sands (Guadalupe-Torres 2013), non-plastic silts (Guadalupe-Torres 2013), and quartz sand (Guadalupe-Torres 2013). In these studies, the ratio of G_{max}/σ'_{1f} has been shown to be have a relatively small range in values (i.e., typically between 100 and 200) (Table 3.2). Based on these results, the G_{max}/σ'_{1f} can serve as a parameter analogous to the S_u/σ'_v ratio in cohesive soils. In this manner, field measurements of in situ V_s using geophysical methods can be used to estimate the variation of σ'_{1f} with depth at a site. Such a profile can be combined with estimated or measured values of σ'_h to estimate shear strength parameters of soils for which undisturbed sampling is difficult.

Soil	No. Tests	Stress range [kPa]	Bulk density [†] , ρ _b [g/cc]	Void ratio ^{††} , e ₀	$rac{G_0}{\sigma'_{1f}}$
Non-plastic silt	14	50 - 200	1.57 - 1.74	0.57 - 0.74	219
Cemented sand	22	100 - 300	1.67 – 2.13	0.26 - 0.60	188
Quartz sand	15	50 - 200	1.58 - 1.74	0.52 - 0.68	180
Sensitive clay	4	30 - 70	1.73 - 1.80	1.27 - 1.34	146
High plasticity clay	5	25 - 400	1.63 – 1.66	1.52 - 1.65	134
Calcareous sand	8	50 - 200	1.09 - 1.23	1.32 - 1.62	128

[†]Sample preparation

^{††} End of consolidation

Table 3.2: Summary of ratios between G_{max} and maximum principal stress at failure for soils tested in Guadalupe-Torres (2013).

Finally, there are a limited number of relationships in the literature that directly relate the peak drained friction angle φ'_p to V_s . One such example was proposed by Uzielli et al. (2013) for quartz-silica sands having trace to little fines content (FC < 10%):

$$\varphi_p' = 3.9(V_{s1})^{0.44} \tag{3-8}$$

$$V_{s1} = \frac{V_s}{\left(\frac{\sigma_{vo}'}{\sigma_{atm}}\right)^{0.25}}$$
(3-9)

where V_s is input in m/s and φ'_p is given in degrees. One particular aspect of the Uzielli et al. (2013) relationship is that it was developed using a probabilistic framework so that a target probability (p_t) of non-exceedance can be assigned (Fig. 3.5). Eq. 3-8 is a best fit deterministic line through the data.



Normalized Shear Wave Velocity, V_{S1} (m/s)

Figure 3.5: Relationship between φ' and V_s developed using a probabilistic framework that assigns a probability of non-exceedance (Uzielli et al. 2013).

Care should be exercised when applying the relationships highlighted in this section. Determination of soil shear strength is a vital aspect of many geotechnical projects and misidentification of strength can lead to failures. The empirical relationships in this section contain anywhere from a reasonable amount to a large amount of scatter. Additionally, some of the correlations are not derived from robust databases representing a wide range of conditions. However, the general approach utilized to develop some of these relationships can be performed at a smaller scale (e.g., across a single site, within a general metropolitan region, etc.) to develop relationships that are better calibrated for site-specific analyses beyond the preliminary design phase. Given the efficiency and broad spatial coverage offered by

geophysical methods, such relationships can prove highly beneficial for design, particularly at sites where undisturbed soil sampling is difficult.

3.1.2.2 Unconfined Compressive Strength of Rock

The unconfined compressive strength (UCS) (also referred to as uniaxial compressive strength) of rock plays an important role in geotechnical design, particularly as related to estimating the capacity of deep foundations socketed into bedrock. Moreover, bedrock properties may change drastically throughout a site based on weathering and fracturing patterns. As a result, a number of studies have attempted to correlate the UCS of rock to seismic velocities since the velocities can be established on a broader scale throughout a site using seismic reflection/refraction. Tables 3.3 – 3.6 and Figs. 3.6 – 3.12 highlight a number of such correlations as available in the literature. Rucker (2008) argues that UCS values estimated from seismic velocities are likely conservative as the seismic waves propagate through the entire rock mass and are slower due to fracturing whereas UCS laboratory testing is often performed on intact specimens of the rock. However, such correlations between UCS and seismic velocities may be inappropriate for shales as noted in Barton (2007). Finally, as has been the case with many of the earth material properties correlated to seismic velocities, there is appreciable scatter in the UCS-velocity data and the selected UCS values should be purposefully selected to be conservative.

Researchers	Equations	<i>r</i> value	Rock type/ lithology	UCS (MPa) $\rho(g/cm^3)$	$V_{\rm p}({\rm km/s})$
Tugrul and Zarif (1999) Kahraman (2001) Yasar and Erdogan (2004)	$\begin{array}{l} UCS = 35.54 \cdot V_{\rm p} - 55 \\ UCS = 9.95 \cdot V_{\rm p}^{1.21} \\ UCS = (V_{\rm p} - 2.0195)/0.032 \\ \rho = (V_{\rm p} + 7.707)/4.3183 \end{array}$	0.80 0.83 0.81 0.80	Igneous rocks Limestone, marble Lime, marble, dolomite	100–200 10–160 38–120 2·43–2·97	4.5-6.5 1.2-6.4 2.9-5.6 2.9-5.6
Sharma and Singh (2007)	$\begin{aligned} &\text{UCS} = 0.0642 \cdot V_{\text{p}} - 117.99 \\ &\text{Id}_2 = 0.069 \cdot V_{\text{p}} + 78.577 \end{aligned}$	$0.90 \\ 0.88$	7 types of rocks	10–1970	2-3.2.0
Kahraman and Yeken (2008)	$\rho = 0.213 \cdot V_{\rm p} + 1.256$	0.82	Carbonate rocks	2.0-2.6	3.6-6.1
This study	$\begin{array}{l} UCS = 0.258 \cdot V_{\rm p}^{3.543} \\ UCS = 49.4 \cdot V_{\rm p} - 167 \\ \rho = 0.19 \cdot V_{\rm p} + 1.61 \\ Id_2 = 0.71 \cdot V_{\rm p} + 95.7 \end{array}$	0·92 0·89 0·58 0·69	9 types of rock	20–125 - 2·15–2·85 -	1·89–6·1 - 1·8–6·1 -

*UCS = uniaxial compressive strength (MPa), V_p = *P*-wave velocity (km/s), ρ = density (g/cm³)

Table 3.3: Summary of published relationships between V_P and UCS (Yagiz 2011).

Eq. no.	UCS (MPa)	Region where developed	General comments	Reference
(1)	$0.035V_p - 31.5$	Thuringia, Germany	_	Freyburg (1972)
(2)	$1200\exp(-0.036\Delta t)$	Bowen Basin, Australia	Fine grained, both consolidated and unconsolidated sandstones with all porosity range	McNally (1987)
(3)	$1.4138 \times 10^{7} \Delta t^{-3}$	Gulf Coast	Weak and unconsolidated sandstones	
(4)	$3.3 \times 10^{-20} \rho^2 V_p^4 [(1+\nu)/(1-\nu)]^2 (1-2\nu) [1+0.78 V_{clav}]$	Gulf Coast	Applicable to sandstones with UCS>30 MPa	Fjaer et al. (1992)
(5)	$1.745 \times 10^{-9} \rho V_{\rm p}^2 - 21$	Cook Inlet, Alaska	Coarse grained sandstones and conglomerates	Moos et al. (1999)
(6)	$42.1\exp(1.9 \times 10^{-11} \rho V_{\rm p}^2)$	Australia	Consolidated sandstones with $0.05 < \phi < 0.12$ and UCS>80 MPa	
(7)	$3.87 \exp(1.14 \times 10^{-10} \rho V_p^2)$	Gulf of Mexico	_	
(8)	46.2exp(0.027E)	_	_	
(9)	$2.28 \pm 4.1089E$	Worldwide	_	Bradford et al. (1998)
(10)	$254 (1-2.7\phi)^2$	Sedimentary basins worldwide	Very clean, well-consolidated sandstones with $\phi < 0.3$	Vernik et al. (1993)
(11)	$277\exp(-10\phi)$	-	Sandstones with $2 \le UCS \le 360$ MPa and $0.002 \le \phi \le 0.33$	

Table 3.4: Summary of published relationships between UCS and physical properties of sandstone rocks (Chang et al. 2006). Note: $\Delta t = 1/V_P$ represents the interval transit time.

Eq. no.	UCS (MPa)	Region where developed	General comments	Reference
(12)	$0.77 (304.8/\Delta t)^{2.93}$	North Sea	Mostly high porosity Tertiary shales	Horsrud (2001)
(13)	0.43 $(304.8/\Delta t)^{3.2}$	Gulf of Mexico	Pliocene and younger	
(14)	$1.35 (304.8/\Delta t)^{2.6}$	Globally	_	
(15)	$0.5 (304.8/\Delta t)^3$	Gulf of Mexico	_	
(16)	$10(304.8/\Delta t - 1)$	North Sea	Mostly high porosity Tertiary shales	Lal (1999)
(17)	$7.97E^{0.91}$	North Sea	Mostly high porosity Tertiary shales	Horsrud (2001)
(18)	$7.22E^{0.712}$	_	Strong and compacted shales	
(19)	$1.001\phi^{-1.143}$	_	Low porosity ($\phi < 0.1$) high strength	Lashkaripour and
			(~79 MPa) shales	Dusseault (1993)
(20)	$2.922\phi^{-0.96}$	North Sea	Mostly high porosity Tertiary shales	Horsrud (2001)
(21)	$0.286\phi^{-1.762}$	_	High porosity ($\phi > 0.27$) shales	

Table 3.5: Summary of published relationships between UCS and physical properties of shale rocks (Chang et al. 2006). Note: $\Delta t = 1/V_P$ represents the interval transit time.

Eq. no.	UCS (MPa)	Region where developed	General comments	Reference
(22)	$(7682/\Delta t)^{1.82}/145$	_	-	Militzer and Stoll (1973)
(23)	$10^{(2.44+109.14/\Delta t)}/145$	_	_	Golubev and Rabinovich (1976)
(24)	$13.8E^{0.51}$	_	Limestone with 10 <ucs<300 mpa<="" td=""><td></td></ucs<300>	
(25)	$25.1E^{0.34}$	_	Dolomite with 60 <ucs<100 mpa<="" td=""><td></td></ucs<100>	
(26)	276 $(1-3\phi)^2$	Korobcheyev deposit, Russia	-	Rzhevsky and Novick (1971)
(27)	$143.8 \exp(-6.95\phi)$	Middle East	Representing low to moderate porosity ($0.05 \le \phi \le 0.2$) and high UCS ($30 \le UCS \le 150$ MPa)	
(28)	$135.9 \exp(-4.8\phi)$	-	Representing low to moderate porosity $(0 \le \phi \le 0.2)$ and high UCS $(10 \le UCS \le 300 \text{ MPa})$	

Table 3.6: Summary of published relationships between UCS and physical properties of limestone/dolomite rocks (Chang et al. 2006). Note: $\Delta t = 1/V_P$ represents the interval transit time.



Figure 3.6: Relationship between UCS and seismic velocity developed through relationships of low strain to high strain modulus derived from seismic velocities and static UCS testing. (Rucker 2008).



Figure 3.7: Relationship between UCS, V_P, and degree of weathering (Hiltunen et al. 2011).



Figure 3.8: Relationship between UCS and V_P (adapted from Barton 2007).



Figure 3.9: Comparison of a number of relationships for UCS as a function of V_P (Yagiz 2011).



Figure 3.10: Empirical relationships between UCS and V_P for 260 sandstones (adapted from Chang et al. 2006). Note: Numbers within plot denote equations in Table 3.4.



Figure 3.11: Empirical relationships between UCS and V_P for 100 shales (adapted from Chang et al. 2006). Note: Numbers within plot denote equations in Table 3.5.



Figure 3.12: Empirical relationships between UCS and V_P for 140 limestones/dolomites (adapted from Chang et al. 2006). Note: Numbers within plot denote equations in Table 3.6.

3.1.3 CONSOLIDATION PARAMETERS

The volumetric response of soils is an important aspect of geotechnical design. Consolidation reflects changes in stress state and the corresponding transfer of loading from pore pressure to skeletal stresses. Secondary compression reflects the effects of long term rearrangement of soil fabric (i.e., creep). The resulting changes in effective stress and/or void ratio can impact small strain stiffness (and thereby seismic velocity). Based on this mechanical link, a number of researchers have explored the relationship between consolidation parameters and geophysical measurements of seismic velocity.



Figure 3.13: V_s for a marine clay as a function of vertical effective stress and loading conditions during consolidation in an oedometer (Lee et al. 2008).

Typically, these studies have attempted to examine the V_s at the end of primary consolidation to develop relationships between V_s , void ratio, and/or effective stress (e.g., Viggiani and Atkinson 1995; Rampello et al. 1997). Some, such as Lee et al. (2008), examined this relationship as a function of effective stress, load stage, and time of consolidation (Fig. 3.13). In both approaches, the link between V_s and consolidation behavior of a soil was quite evident given the similarity in the resulting trends when plotted against effective stress. Moreover, Yoon et al. (2011) demonstrated that predictions of preconsolidation pressure (σ_p') from geophysical measurements at Korean sites agreed favorably with many of the existing approaches that estimate from the void ratio-pressure (e- σ') plot (Fig. 3.14). This approach was simply based on examining the inflection point of the V_{s} - σ' trendlines as the soil transitioned from recompression to virgin compression (Fig. 3.15).



Figure 3.14: Comparison of σ'_p estimates for 11 Korean clays: C = Casagrande, J = Janbu, B = Becker, S = Sridharan, O = Onitsuka, and V_s = Shear wave method (Yoon et al. 2011).



Figure 3.15: Example of approach used in Yoon et al. (2011) to estimate σ'_p (p' in figure) from V_s measurements during consolidation in an oedometer.

In a similar manner, L'Heureux and Long (2017) presents a best fit trendline between V_s and $\sigma_{p'}$ [determined using the Janbu (1963) procedure] for data from 14 sites in Norway (Fig. 3.16):

$$\sigma_p' = 0.00769 V_s^{2.009} \tag{3-10}$$



Figure 3.16: Relationship between σ'_p (p'_c in figure) and V_s for Norwegian clays (adapted from L'Heureux and Long 2007).

Mayne et al. (1998) developed a similar relationship based on 262 pairings of V_{S} - σ_{p} ' data at various clay sites:

$$\sigma_p' = \left(\frac{V_s}{4.59}\right)^{1.47} \tag{3-11}$$

where V_s is express in m/s and σ_p' in kPa. Finally, Lok et al. (2015) developed an empirical relationship that allowed the secondary compression index to be estimated from V_s measurements, assuming information is known about the end of consolidation parameters (i.e., t_p and e_p):

$$\frac{V_s}{V_{sp}} = 0.4343C_{\alpha} (1+e_p) \ln\left(\frac{t}{t_p}\right) + 1$$
(3-12)

where V_s is the measured shear wave velocity (measured using bender elements in this study), $V_{s,p}$ is the shear wave velocity at the end of primary consolidation, C_{α} is the secondary compression index, e_p is the void ratio at the end of primary consolidation, and t_p is the time at the end of primary consolidation. This relationship was based on comparing the time rate of deformations to the changes in $V_{\rm S}$ with time and normalizing based on the conditions present at the end of consolidation for two undisturbed and two reconstituted Macau clay samples (Fig. 3.17). Given the limited database, this relationship is highly site/soil specific and is not recommended for general usage. However, this approach demonstrates tremendous potential as a field monitoring tool. A site specific correlation similar to Eq. 3-12 could be developed based on a relatively small amount of laboratory consolidation testing in an oedometer fitted with bender elements. It would then be possible to use field-based seismic methods to monitor long term secondary compression effects after construction to ensure compatibility with analytical results during design.



Figure 3.17: Relationship between secondary compression and V_s in Lok et al. (2015): (a) time histories for all four samples; and (b) normalized time histories for all four samples.

3.1.4 COEFFICIENT OF LATERAL EARTH PRESSURE

The coefficient of lateral earth pressure for at rest conditions (K_o) represents the ratio of the in situ horizontal effective stress (σ'_{ho}) to the in situ vertical effective stress (σ'_{vo}). Since this parameter relates to the in situ stress state in soil, it is routinely found in a number of geotechnical applications, including the interpretation of laboratory and in situ tests, the design of retaining and excavation support systems, and the evaluation of the shaft friction in deep foundations (Simpson 1992; Fioravante et al. 1998). Given the many applications, K_o is an important parameter in the design of geotechnical transportation projects. However, it can be quite difficult to reliably measure K_o due to the many factors that affect the stress state in soils. For example K_o in clays is highly influenced by the many mechanisms that affect its structure, including mechanical overconsolidation, ageing, cementation, and physico-chemical changes (Jamiolkowski et al. 1985; Fioravante et al. 1998; Puppala et al. 2006). The current state of practice typically estimates K_o from the Jaky (1944) empirical relationships for normally consolidated (NC) clays with modifications that include the effects of overconsolidation ratio typically estimate from laboratory testing on high quality undisturbed samples (e.g., Sivakumar et al. 2001):

$$K_o = (1 - \sin\varphi)OCR^m \tag{3-13}$$

where φ is the friction angle, OCR is the overconsolidation ratio, and m is often computed as $m = \sin\varphi$ and typically varies between 0.4 and 0.7 depending on clay mineralogy (Mayne and Kulhawy 1982; Lunne and Christophersen 1983). In situ testing methods have been developed that allow for rapid estimates of K_o profiles under field conditions. These methods include direct tests [e.g., self-boring pressuremeter test (SBPMT)], semi-direct tests (e.g., total stress cells, lowa stepped blade, and Marchetti's flat dilatometer test), and empirical correlations to large strain penetration tests (e.g., SPT, CPT, etc.) (Robertson 1986). For example, K_o can be estimated based on measurements of CPT trip resistance (Kulhawy and Mayne 1990):

$$K_o = 0.10 \left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right) \tag{3-14}$$

where q_t is the corrected cone tip resistance, σ_{vo} is the in situ total vertical stress, and σ'_{vo} is the in situ effective vertical stress. In situ approaches are quite popular in practice as they allow rapid determination of K_o . However, it should be noted that the testing process necessary for many in situ methods can still appreciably alter the stress state and introduce uncertainty in the measurements.

There has been growing interest in using seismic-based geophysical methods to evaluate K_o because of the aforementioned limitations in the current state of practice. It has long been recognized that the velocity of body waves in soil are fundamentally dependent on the existing effective stress state (e.g., Roesler 1979). For example, V_s is fundamentally related to small strain shear stiffness (G_{max}), which itself is primarily a function of stress state and soil fabric (Hardin 1978; Stokoe et al. 1985). Therefore, independent measurements of body wave propagation with different polarizations should relate to K_o . Two methods have utilized this concept to develop relationships between measurements of V_s and K_o . The first was proposed by Sully and Campanella (1995) using seismic CPT and crosshole testing at multiple test sites in Vancouver, Canada. In this methodology, V_s can be related to stress state based on the following expression:

$$V_s = C_s(\sigma')^n \tag{3-15}$$

where V_s is the measured shear wave velocity, C_s is a constant that is dependent on soil state/anisotropy, σ' is the effective confining stress, and n is a stress exponent. There are two possible ways to define the σ' term, either as the average effective stress (σ'_m) or as the mean octahedral normal stress (σ'_o) (Knox et al. 1982):

$$\sigma'_{m} = \frac{\sigma'_{1} + \sigma'_{3}}{2}$$
(3-16)

$$\sigma'_{o} = \frac{\sigma'_{1} + \sigma'_{2} + \sigma'_{3}}{3}$$
(3-17)

where σ'_1 , σ'_2 , and σ'_3 are the principal stresses in the vertical and horizontal directions, respectively. Input of each of these confining stresses into Eq. 3-15 yields two potential configurations for the relationship between V_s and confinement:

$$V_s = C_s (\sigma'_m)^{n_t} \tag{3-18}$$

$$V_s = C_s(\sigma_o')^{n_m} \tag{3-19}$$

where the modified *n* terms represent different stress exponents based on their respective confining stress definitions. Based on assuming that the stresses in the horizontal plane are isotropic (i.e., $\sigma'_2 = \sigma'_3 = \sigma'_h$) and by applying the individual stress components into Eqs. 3-18 and 3-19 on two polarizations of the shear wave, the following expression can be derived:

$$\frac{V_s(HV)}{V_s(HH)} = \frac{C_s(HV)}{C_s(HH)} (K_o)^{-n_1}$$
(3-20)

where $V_{s}(HV)$ represents the velocity of a shear wave propagating in the horizontal direction with vertical particle motion (Fig. 3.18), $V_{s}(HH)$ represents the velocity of a shear wave propagating in the horizontal direction with horizontal particle motion (Fig. 3.18), $C_{s}(HV)$ is the anisotropic shear wave velocity constant, $C_{s}(HH)$ is the isotropic shear wave velocity constant, K_{o} is the coefficient of lateral earth pressure at rest, and n_{1} is a stress exponent that is dependent on the vertical effective stress ($\sigma'_{1} = \sigma'_{v}$). A complete derivation of Eq. 3-20 can be found in Cai et al. (2011). The ratio of $C_{s}(HV)/C_{s}(HH)$ is directly related to the inherent structural anisotropy of the soil and is independent of the stress conditions (and any corresponding anisotropic stresses). Recent studies have demonstrated that electrical resistivity measurements can be used to determine this ratio (Tong et al. 2013). Though this ratio can be as large as 1 to 1.1 (e.g., Lee and Stokoe 1985; Yan and Byrne 1990), a value of 0.93 is recommended for granular soils (Fioravante et al. 1998) and 0.85 for clays (Jamiolkowski et al. 1995). As an alternative to Eq. 3-20, the derivation can proceed using the average effective stress (σ'_{m}) instead of the individual stress components:

$$\left(\frac{V_{s}(HV)C_{s}(HV)}{V_{s}(HH)C_{s}(HH)}\right)^{\frac{1}{n_{1}+n_{2}}} = \frac{2K_{o}}{1+K_{o}}$$
(3-21)

where n_1 and n_2 represent the exponential stress components related to the principal stresses acting in the direction of wave propagation and particle motion, respectively. Fioravante et al. (1998) noted that the uncertainty in the *n* exponent terms is less significant than the uncertainty in $C_s(HV)/C_s(HH)$. Additionally, as noted from a summary of n_1 and n_2 values in Table 3.7, n_1 can be assumed to be equal to n_2 . By assuming perfectly isotropic conditions, K_o can be estimated from measurements using only a single polarization of shear waves from any of the downhole/cross-hole seismic tests (Hatanaka and Uchida 1995):

$$K_{o} = \frac{1}{2} \left[\frac{3}{\sigma_{v}'} \left(\frac{G_{max}}{A} \right)^{\frac{1}{4n_{1}}} - 1 \right]$$
(3-22)

where A is a material constant that can be obtained from laboratory tests on undisturbed samples. Given the dependence on isotropic conditions for Eq. 3-22, it is expected that estimates of K_o would be less reliable than those determined using Eqs. 3-20 and 3-21.



Figure 3.18: Wave polarization in different forms of seismic testing (Ku and Mayne 2013).

References	n_1	<i>n</i> ₂	Notes
Roesler (1979)	0.15	0.11	Pulse test
Knox et al. (1982)	0.12	0.09	Pulse test
Allen (1982)	0.12	0.11	Resonant column tests
Yu and Richart (1984)	0.12-0.14	0.11 - 0.14	Resonant column tests
Lee and Stokoe (1985)	0.10	0.10	Resonant column tests
Thomann and Hryciw (1990)	0.13	0.13	Bender element tests
Yan and Byrne (1990)	0.12	0.12	Hydraulic gradient similitude
Fioravante et al. (1998)	0.12	0.12-0.14	Pulse test

Table 3.7: n_1 and n_2 values for use in estimating K_0 from V_s measurements (Cai et al. 2011).

Recent studies by Cai et al. (2011) and Tong et al. (2013) at two test sites in China demonstrated generally good agreement between K_o values from Eqs. 3-20 and 3-21 and those obtained from the Jaky (1944) relationship using laboratory-derived soil properties and those obtained from the Kulhawy and Mayne (1990) relationship to CPT tip resistance. The K_o predicted from the seismic measurements matched the Jaky (1944) and Kulhawy and Mayne (1990) values better for normally consolidated clays located at larger depths. There was more discrepancy for overconsolidated soils at the near surface (less than 15.0 m) (Fig. 3.19). Based on these results, Cai et al. (2011) and Tong et al. (2013) concluded that an OCR-based correction factor may improve the formulations, though more research is necessary to establish a robust functional form for this correction factor. Past studies have also demonstrated similarly effective performance of Eqs. 3-20 and 3-21 when estimating K_o values (e.g., Fioravante et al. 1998). However, another recent study by Ku and Mayne (2013) explored the relationship between K_o and V_s at 16 well-documented test sites with different soil types and found that modification factors may be necessary for the exponent in Eq. 3-20 (Fig. 3.20). Additionally, Ku and Mayne (2013) found that the predictions from Eq. 3-20 were improved by including terms that accounted for age and depth of the formation (Fig. 3.21).



Figure 3.19: Comparison of estimated K_o values using Eqs. 3-20 and 3-21 [Eqs. (3) and (4) in figure legend], Jaky (1944) relationship, and Kulhawy and Mayne (1990) relationship (Mayne in figure legend) (Tong et al. 2013).



Figure 3.20: Regression analysis between K_o and V_s and corresponding sensitivity analysis of MF_1 and MF_2 on exponent *n* in Eq. 3-20: (a) $K_o V_s$. V_{SHH}/V_{SVH} with MF_1 ; (b) $K_o V_s$. V_{SHH}/V_{SHV} with MF_1 ; (a) $K_o V_s$. V_{SHH}/V_{SVH} with MF_2 ; (b) $K_o V_s$. V_{SHH}/V_{SHV} with MF_2 (Ku and Mayne 2013).



Figure 3.21: Comparison of regression analysis on K_o as a function of: (a) V_{SHH}/V_{SVH} and soil age; and V_{SHH}/V_{SVH} , soil age, and depth (Ku and Mayne 2013).

3.1.5 RIPPABILITY

Rippability is defined as the ease with which soil or rock can be mechanically excavated (Wightman et al. 2003). Excavation of rocks, in particular, is inherently related to several influential factors, including the extent and location of weathering, unconfined compressive strength of the intact rock, and the equipment used for excavation. It is extremely beneficial to ensure estimates are made regarding rippability during the planning stages of excavations, so that appropriate equipment is selected and the use of explosives is considered to fragment the rock as necessary. Assessment of rocks has been a concern for as long as excavations have taken place for construction purposes, and several researchers have proposed rock mass classification schemes (e.g., Wickham et al. 1972; Bieniawski 1973, 1976, 1989; and Barton et al. 1974). However, these methods often rely on parameters derived from laboratory testing and other localized measurements of rock properties. Atkinson (1971) was one of several researchers to propose relationships for rippability of rock based on P-wave velocity as a proxy for strength and weathering characteristics. This allows a fast assessment of overall rippability since seismic reflection and/or refraction can be utilized to determine the P-wave velocity on a larger scale. Several of these relationships are summarized in Church (1981), and Caterpillar Inc. publishes a performance handbook for use with their equipment that includes estimates of rippability based on P-wave velocity (Fig. 3.22). MacGregor et al. (1994) developed regression relationships for the productivity of rock excavation operations based on P-wave velocity among other factors (Fig. 3.23). Productivity was defined as the volume of intact material excavated in a given time frame. The proposed relationships for productivity are provided in Table 3.8. Productivity was compared to qualitative assessments of the ease of rippability, based on operator feedback (Fig. 3.24), which ultimately allows a direct comparison between P-wave velocity and rippability. Caltrans has developed their own correlations for rippability from P-wave velocity based on their experiences, which has proven to be more conservative and reliable across a wider range of materials compared to commonly used correlations such as Caterpillar (2008) (see Table 3.9 and Leeds 2002). As noted previously, equipment plays a role in rippability and the applicability of the Caltrans correlations in Table 3.9 are limited to the Caterpillar D-9 series. The primary function of Table 3.9 is to serve as a contract specification whereby risks associated with blasting are delineated. The blasting specifications included in bid documents for a project maintain that Caltrans assumes cost risk for blasting when the material velocity is unrippable based on the classification in Table 3.9 (i.e., material velocity above 2000 m/s). If desired,

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contractors can place more competitive bids by proposing different equipment (e.g., Caterpillar D10, D11, etc.) but project cost risks are transferred to the contractor in those situations. This demonstrates how the relationships discussed in this document can form the basis for contractual exchanges, in addition to estimation of material properties.



Figure 3.22: Estimates for rippability based on V_P and Caterpillar D10R equipment (Caterpillar 2008).



Figure 3.23: Case history data used for regression analysis of productivity versus V_P in MacGregor et al. (1994).



Figure 3.24: Qualitative relationship between productivity and rippability (MacGregor et al. 1994).

Equation	Rock type	Correlation coefficient R^2
$\ln(\text{Productivity}) = 9.25 - 0.0015(S)$	All	0.32
$\ln(\text{Productivity}) = 8.95 - 0.0012(S)$	Sedimentary	0.23
$\ln (\text{Productivity}) = 8.65 - 0.0019(S)$	Igneous	0.63

Notes: S = seismic velocity in metres per second. These relationships do not apply for seismic velocities less than 300 m/sec.

Table 3.8: Proposed equations for productivity as a function of V_P in MacGregor et al. (1994).

Velocity (m/s)	Rippability
< 1050	Easily Ripped
1050 – 1500	Moderately Difficult
1500 – 2000	Difficult Ripping
> 2000	Unrippable

Table 3.9: Caltrans rippability chart (adapted from Leeds 2002). Note: Limited to Caterpillar D9 series.

3.1.6 EARTHWORK/GRADING FACTOR

Earthwork operations are typically required for transportation projects. In most cases, earthwork operations will consist of excavating in situ soils and bringing the site back to design grade by compacting the excavated soil as fill material. In doing so, volumetric changes can occur in the excavated materials during excavation and placement, which can alter the amount of material necessary to complete construction. Earthwork factors quantify the volumetric changes so that they can be accounted for during design and construction. The Shrink Factor is a ratio of the unit weight of in situ soils at the site to the unit weight of the soils after compaction. The Swell (or Load) Factor is a ratio of the unit weight of the soils at the site in their natural state. The Earthwork Factor simplifies this into a single step by comparing the volumetric amount of compacted fill that results from a given volume of excavated material. Shrinkage would be signified by an Earthwork Factor less than 1 and swelling would be signified by a value larger than 1.



Figure 3.25: Earthwork Factor as a function of V_P for various rock types based on Smith et al. (1972) and Stephens (1978).

Attempts have been made to develop relationships between earthwork factors and P-wave velocities so that volumetric calculations can be performed and quantifies of fill can be estimated based on geophysical measurements of site materials. Accurate estimates of earthwork materials prior to construction reduce construction costs associated with shortages and/or excesses in fill material. Smith et al. (1972) and Stephens (1978) summarize a number of studies performed by Caltrans to estimate earthwork factors based on the seismic P-wave velocities of different rock types (Fig. 3.25). These curves were generated empirically based on

data acquired at a number of project sites (10+) across the state of California. Rucker (2008) and Rucker (2000) suggest that earthwork factors can be estimated for site materials by computing in situ dry unit weights as correlated to P-wave velocity (e.g., Fig. 3.26) and comparing these values to anticipated fill unit weights, laboratory Proctor tests on sampled materials, or to dry unit weights estimated using seismic refraction for existing fill areas. Hiltunen et al. (2011) provides another such relationship between unit weight and P-wave velocity (Fig. 3.27).



Figure 3.26: Example of geo-material γ_d as a function of V_P (Rucker 2008).



Figure 3.27: Estimate of γ for rock as a function of V_P and weathering (Hiltunen et al. 2011).

3.1.7 ROCK MASS CLASSIFICATION

Rock masses differ quite significantly from other earth materials as they contain a number of structural discontinuities (e.g., joints, shear zones, bedding planes, faults, folds, etc.) that ultimately govern their engineering behavior (Bieniawski 1989). Often it is more important to evaluate the type and frequency of these discontinuities than it is to determine the types of rocks involved or the strength of the intact rock itself (Palmström et al. 2002). Analytical techniques highly depend on the relative scale between the problem domain and the size of the intact rock blocks formed by the discontinuities (Zhang 2016). It is for this reason that classification of rocks often involves a quantitative and/or qualitative assessment of rock discontinuities. Over the last 50+ years, various rock classification systems have been proposed that attempt to account for the effects of discontinuities [e.g., Rock Quality Designation (RQD) (Deere et al. 1967); Rock Mass Rating (RMR) (Bieniawski 1978); Q-System (Barton et al. 1974); Geological Strength Index (GSI) (Hoek and Brown 1997); etc.]. Often, these systems allow for an estimate of rock strength, stiffness, and/or compressibility, in addition to providing information regarding the nature of jointing. Small strain stiffness (and, by extension, seismic wave velocity) is also significantly influenced by the nature of jointing in a rock mass. Therefore, it is not surprising that a number of investigators have proposed empirical relationships between seismic velocity (typically V_P) and various parameters associated with different rock mass classification systems. The following sections describe these relationships.

Rock Quality Designation (RQD)	Classification of Rock Quality	
0 – 25%	Very Poor	
25% – 50%	Poor	
50% – 75%	Fair	
75% – 90%	Good	
90% - 100%	Excellent	

Table 3.10: Classification of rock quality based on RQD (ASTM D6032).

3.1.7.1 Rock Quality Designation (RQD)

RQD was first developed by Deere (1964) and Deere et al. (1967) as a way to quantify the quality of borehole core samples of rock (Table 3.10). There are a few established methods by which to define *RQD*. The direct method is based on quantifying the length of intact core samples. In this

manner, *RQD* is defined as the ratio (in percentage) of the total length of sound core pieces that are 0.1 m (4 inch) or longer to the total length of the core run (Fig. 3.28).



Figure 3.28: Procedure for determination of RQD from rock coring (after Deere 1989).

RQD provides an index value that represents rock quality since poor rock recovery indicates excessive weathering, jointing, fracturing, and similar issues. A core size of at least NX (size 54.7 mm) or NQ-size (47.5 mm [1.87 in.]) is recommended with drilling taking place using a double-tube core barrel with a diamond bit (ASTM D6032). *RQD* can also be determined based on the frequency of discontinuities observed during scanline surveying (i.e., tape measure along an outcropped rock surface). Various correlations between *RQD* and linear discontinuity frequency have been developed based on different assumptions regarding the spatial distribution of jointing (Priest and Hudson 1976; Sen and Kazi 1984; Sen 1993). For example, the Priest and Hudson (1976) relationship specifies an exponential decay function for the discontinuity spacing, resulting in the following expression for *RQD*:

$$RQD = 100e^{-\lambda t}(\lambda t + 1) \tag{3-23}$$

where λ is the frequency of discontinuities per unit length and *t* represents the length threshold used to define *RQD* (i.e., typically *t* = 0.1 m as previously discussed in original *RQD* definition). In terms of geophysical measurements, the seismic compressional velocity is highly affected by the presence of jointing in rock. Correspondingly, comparison of the seismic velocity of intact rock samples to an in situ rock mass with jointing can provide a reliable method by which to estimate *RQD*. The following functional form has been proposed for this approach:

$$RQD = \left(\frac{V_{p,f}}{V_{p,o}}\right)^2 \times 100\% \tag{3-24}$$

where $V_{P,f}$ is an in situ measurement of seismic compressional wave velocity and $V_{P,o}$ is a measurement of seismic compressional wave velocity for the intact rock (Deere et al. 1967). $V_{P,o}$ can be measured directly in the laboratory using an ultrasonic pulse approach on sound rock core samples or can be indirectly estimated based on lithology of the rock. Similar expressions have been established by other researchers (e.g., El-Naqa 1996; Bery and Saad 2012):

$$RQD = 0.77 \left(\frac{V_{p,f}}{V_{p,o}}\right)^{1.05} \times 100\%$$
(3-25)

$$RQD = 0.97 \left(\frac{V_{p,f}}{V_{p,o}}\right)^2 \times 100\%$$
 (3-26)

where the variables are as defined previously in Eq. 3-24. Other functional forms have also been proposed, including a hyperbolic relationship (Sjogren et al. 1979 and Palmström 1995):

$$RQD = \frac{V_{p,q} - V_{p,f}}{V_{p,q}V_{p,f}k_q} \times 100\%$$
(3-27)

where $V_{P,q}$ is the seismic compressional wave velocity for a rock mass with RQD = 0, $V_{P,f}$ is an in situ measurement of V_P as described in previous equations, and k_q is a fitting parameter that accounts for in situ rock conditions. This functional form can be used by performing a regression to determine $V_{P,q}$ and k_q from data acquired from a single rock core at the site. Eq. 3-27 can then be applied to other parts of a site to estimate *RQD* from seismic surveys assuming the rock shares similar lithological characteristics. For example, Budetta et al. (2001) performed such an analysis and determined that $V_{P,q} = 1.22$ km/s and $k_q = -0.69$ in Eq. 3-27 for a heavily fractured calcareous rock in southern Italy (Fig. 3.29).



Figure 3.29: Relationship between V_P and RQD for heavily fractured calcareous rock masses in southern Italy (Budetta et al. 2001).

In some cases, other researchers have developed direct relationships between *RQD* and measured V_P from a seismic survey (e.g., Sjogren et al. 1979; El-Naqa 1996; Leucci and De Giorgi 2006). Eq. 3-28 below and Figs. 3.30 – 3.32 provide examples of such empirical relationships:
$$RQD = 36.7V_p^{0.52} \tag{3-28}$$

where V_P is input in km/s (El-Naqa 1996). It should be emphasized that such relationships are highly site-specific and should not be extrapolated outside their intended scope.



Figure 3.30: Relationship between V_P and RQD for limestones, mudstones, marls and shales beneath a dam site in Jordan (El-Naqa 1996). Note: Dahsed lines represent 95% and 90% confidence limits.



Figure 3.31: Relationship between RQD and V_P based on laboratory testing on a calcarenite block (Leucci and De Giorgi 2006).



Figure 3.32: Relationships between *RQD* and discontinuity frequency (λ) based on Sjogren et al. (1979) *V*_P measurements primarily in Norway (Barton 2002).

A limited number of studies have attempted to develop similar empirical relationships between $V_{\rm S}$ and RQD (Leucci and De Giorgi 2006; Biringen and Davie 2013). However, as noted in Figs. 3.33 – 3.34, the level of fit for these relationships is quite variable depending on the dataset examined.



Figure 3.33: Relationships between RQD and V_s : (a) Igneous rock site in South Carolina; and (b) metamorphic rock site in Virginia (Biringen and Davie 2013).



Figure 3.34: Relationship between RQD and V_s based on laboratory testing on a calcarenite block (Leucci and De Giorgi 2006).

Finally, it is worth noting that correlations exist between *RQD* and other rock mass parameters of interest. Based on these correlations and the aforementioned relationships proposed between seismic velocity and *RQD*, geophysical measurements can be used to estimate these

rock mass properties. For example, *RQD* can be related to volumetric joint count (J_v), which measures the number of joints within a unit volume of rock mass (Palmström 2005):

$$RQD = 110 - 2.5J_{\nu} \tag{3-29}$$

Based on similar relationships, it may be possible to estimate joint related parameters (e.g., joint density, spacing, etc.) indirectly from estimates of *RQD* using geophysical measurements of seismic velocity. Fig. 3.32 provides one such graphical form for the relationship between V_P , *RQD*, and λ proposed in Sjogren et al. (1979). *RQD* can also be correlated to deformation modulus and UCS (Figs. 3.35 – 3.36). However, relationships to *RQD* can be quite crude because of the one-dimensional nature of the *RQD* calculation (Palmström 2005). In any case, geophysical measurements using seismic methods can still provide useful information regarding rock characteristics on a broad scale across a site since the seismic velocities can be directly correlated to *RQD* and indirectly to jointing parameters, UCS, and deformation modulus.



Figure 3.35: Relationships between deformation modulus ratio and RQD (Ebisu et al. 1992).



Figure 3.36: Relationships between UCS ratio and RQD (Zhang 2016).

3.1.7.2 Tunneling Quality Index System (Q-system) Value

The Tunneling Quality Index System (*Q*-system) was developed by Barton et al. (1974) as a way to designate rock quality for design and support recommendations in underground excavations (Table 3.11).

Q	Classification of Rock Quality	
0.001 - 0.01	Exceptionally Poor	
0.01 - 0.1	Extremely Poor	
0.1 - 1	Very Poor	
1-4	Poor	
4 - 10	Fair	
10 - 40	Good	
40 - 100	Very Good	
100 - 400	Extremely Good	
400 - 1000	Exceptionally Good	



In this approach, a Q-value is obtained by using the following relationship:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$
(3-30)

where *RQD* is the rock quality designation, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number (related to friction angle), J_w is the joint water reduction number, and *SRF* represents the stress reduction factor. Table 3.12 provides a discussion of these inputs into the *Q*-system. The first term in Eq. 3-30 relates to the size of the intact rock blocks in the rock mass. The second term represents the shear strength along the discontinuity planes between rock blocks. The third term is related to the stress environment around the underground excavation. The *Q*-system is essentially a classification system for rock masses with respect to stability of underground openings. The *Q*-value obtained from Eq. 3-30 can be used to estimate the ultimate support pressure and recommendations regarding appropriate support design for an underground excavation (Figs. 3.37 – 3.38).



Figure 3.37: Correlation between support pressure and Q-value (from Barton et al. 1974).

1.	Rock Quality Designation	R	QD 5. Joint Water Reduction Factor (kg/cm ² (kg/cm ²)			approx water pres. (kg/cm ²)	Jw	
A B	Very poor Poor	0 - 25 -	· 25 · 50	A Dry excavations or minor inflow, <i>i.e.</i> , <5 l/min <1		<1	1.0	
С	Fair	50 -	- 75	В	Medium inflow or pressure, occasional of joint fillings	outwash	1-2.5	0.66
E	D Good 75 - 5 E Excellent 90 - 1		· 100	c	C Large inflow or high pressure in competent rock with unfilled joints		2.5-10	0.5
Note: i) Where RQD is reported or measured as ≤ 10 nominal value of 10 is used to evaluate Q. ii) ROD intervals of 5, <i>i.e.</i> , 100, 95, 90, <i>etc.</i> , are s		(including 0), a ufficiently accurate.		D	D Large inflow or high pressure, considerable outwash of joint fillings		2.5-10	0.33
				E	Exceptionally high inflow or water pres blasting, decaying with time	sure at	>10	0.2-0.1
Z .	Massive, no or few joints	J n F Exceptionally high inflow or water pressure >10		>10	0.1-0.05			
B	One joint set	0.5	2 Note: i) Factors C to F are crude estimates. Increase J _w if drainage				age	
C	One joint set plus random joints Two joint sets		 3 measures are installed. ii) Special problems caused by ice formation are not considered. 				idered.	
E	Two joint sets plus random joints		6 6 Stress Reduction Factor			SRF		
F	Three joint sets		9	a)	Weakness zones intersecting excavation, which	mav cause	loosenina of	
н	Four or more joint sets, random, heavily jointed,		5	Ľ	rock mass when tunnel is excavated		na davar	
 J	"sugar cube", etc. Crushed rock, earthlike	15		A	A chemically disintegrated rock, very loose surrounding ro			
Note	:: i) For intersections, use (3.0 × J _n) ii) For portals, use (2.0 × J _n)	20		в	(any depth) Single weakness zones containing <i>clay</i> or chemically B disintegrated rock (depth of excavation < 50 m)			5
3.	Joint Roughness Number	1	lr I	с	Single weakness zones containing <i>clay</i>	or chemic r > 50 m	cally	2.5
a) I	Rock-wall contact, and b) rock-wall contact before 10	cm shear			Multiple shear zones in competent rock	k (clay-free	e), loose	7.5
A	Discontinuous joints	4	4	E	Single shear zones in competent rock	(clay-free)	(depth of	5.0
C	Smooth, undulating		2	F	excavation ≤ 50 m) Single shear zones in competent rock ((clav-free)	(depth of	2.5
D	Slickensided, undulating	1.5 F excavation > 50 m)		ate	2.5			
F	Smooth, planar	1	.0	G	(any depth)	sugar cube	e', etc.	5.0
G	Slickensided, planar	0.5		Note	Note: i) Reduce these value of SRF by 25-50% if the relevant sh only influence but did not intersect the excavation			
NOTE	 Descriptions refer to small scale reatures and features, in that order. 	i intermed	liate scale	b)	Competent rock, rock stress problems	σ_c/σ_1	σ_{ϕ}/σ_{c}	SRF
c) I	No rock-wall contact when sheared			н	Low stress, near surface, open joints	>200	< 0.01	2.5
н	Zone containing clay minerals thick enough to prevent rock-wall contact	1	.0	J	Medium stress, favourable stress condition	200-10	0.01-0.3	1
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0		к	favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
 Note: () Add 1.0 if the mean spacing of the relevant joint so ii) J_r = 0.5 can be used for planar slickensided lineations, provided the lineations are oriented. 		joints havi	ing imum	L	Moderate slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
	strength.			м	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
4.	Joint Alteration Number	Φ _r approx.	Ja	N	Heavy rock burst (strain-burst) and immediate dynamic deformations in maccine rock	<2	>1	200-400
a) I	Rock-wall contact (no mineral fillings, only coating	s)		Note	: ii) For strongly anisotropic virgin stress	field (if me	easured): w	/hen
A	filling, <i>i.e.</i> , quartz or epidote	•	0.75		$5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\sigma_c$. σ_c to $0.5\sigma_c$, where $\sigma_c = unconfined$	When σ_1/c	$\sigma_3 > 10$, rec	duce
В	Unaltered joint walls, surface staining only Slightly altered joint walls. Non-softening mineral	25-35°	1.0		σ_3 are the major and minor principal	stresses, a	and $\sigma_{\theta} = m$	aximum
с	coatings, sandy particles, clay-free disintegrated rock, etc.	25-30°	2.0		 iii) Few case records available where depth of crown below surfless that span width. Suggest SRF increase from 2.5 to 5 for 			
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20-25°	3.0	c) (cases (see H). Squeezing rock: plastic flow of incompetent	rock	σ_{Φ}/σ_{c}	SRF
Е	<i>i.e.</i> , kaolinite or mica. Also chlorite, talc, gypsum,	8-16°	4.0	0	Mild squeezing rock pressure		1-5	5-10
-	clays.			Ρ	Heavy squeezing rock pressure		>5	10-20
b) I	Rock-wall contact before 10 cm shear (thin minera	al fillings)		Note	e: iv) Cases of squeezing rock may occur f et al., 1992). Rock mass compressio	or depth H n strength	i > 350 Q ^{1/} can be esti	3 (Singh imated
F	Sandy particles, clay-free disintegrated rock, etc. Strongly over- consolidated non-softening clay	25-30°	4.0		from $\sigma \approx 0.7 \gamma Q^{1/3}$ (MPa) where $\gamma = 1993$).	rock dens	ity in kN/m	³ (Singh,
6	mineral fillings (continuous, but <5 mm thickness)	10-24-	0.0	d) :	Swelling rock: chemical swelling activity depend	ing on prese	nce of water	
н	mineral fillings (continuous, but <5 mm thickness) Swelling-clay fillings, <i>i.e.</i> , montmorillonite	12-16°	8.0	R	Mild swelling rock pressure Heavy swelling rock pressure			5-10 10-15
J	(continuous, but <5 mm thickness). Value of J _a depends on percent of swelling clay-size particles,	6-12°	8-12	Note	e: J _r and J _a classification is applied to the	i joint set (or discontir	nuity that
01	and access to water, etc.	fillinge)			is least favourable for stability both fro orientation and shear resistance, r (wh	m the point of $r \approx \sigma$	nt of view (tan ⁻¹ (),/)	of J _a).
KIM	Zones or bands of disintegrated or crushed rock and	6-249	6, 8, or		Choose the most likely feature to allow	v failure to	initiate.	~
M	clay (see G, H, J for description of clay condition) Zones or bands of silty- or sandy-clay, small clay	-27	8-12		$O = RQD \cup J_r$	Jw		
11	fraction (non-softening) Thick, continuous zones or bands of clay (see G	6.249	10, 13		$Q = \frac{1}{J_n} \times \frac{1}{J_a}$	SRF		
OPR	H, J for description of clay condition)	6-24°	or 13-20					

Table 3.12: Summary of *Q*-system parameters (ASTM D5878).



Figure 3.38: Rock support chart based on Q-value (NGI 2015).

Barton (1991) first proposed a relationship between the *Q*-value and V_P based on data from over 2000 core samples:

$$V_P = 3.5 + \log_{10} Q \tag{3-31}$$

where V_P is expressed in km/s (Fig. 3.39).



Figure 3.39: Integrated Q-V_P relationship including depth and porosity (Barton 2002).

This relationship was based on data from hard rock tunneling projects in several countries (including Sjogren et al. 1979) where V_P was measured using a number of seismic methods, including seismic tomography. However, given the database, Eq. 3-31 is not well suited for rock conditions outside those used in its formulation, particularly for "weaker" rock conditions. To extend this relationship, a modification was proposed by Barton (1995) that normalized the *Q*-value to a nominal hard rock compressive strength value of 100 MPa:

$$Q_c = Q \frac{\sigma_c}{100} \tag{3-32}$$

where σ_c is the uniaxial compressive strength expressed in MPa. The normalized *Q*-value (*Q_c*) is then input for *Q* in Eq. 3-31 to improve the correlation. Additional studies with a wide range of rock conditions (e.g., marls, chalks, sandstones, shales, granites, gneiss, etc.) were used to develop an integrated *V_P-Q* (and modulus) seismic correlation chart, which uses the *Q_c* from Eq. 3-32 (Fig. 3.39). This seismic correlation chart allows an approximate *Q*-value to be selected for preliminary assessment of rock support needs based on a measurement of *V_P* at a depth *H* with estimated porosity and uniaxial compressive strength. Rock mass deformation modulus could also be estimated using the same chart. All of this could be accomplished in a rapid manner using geophysical measurements prior to in situ measurements using coring.

3.1.7.3 Rock Mass Rating (RMR) Value

The Geomechanics Classification or the Rock Mass Rating (*RMR*) system traces its origins to the work of Bieniawski (1973) with shallow tunnels in sedimentary rocks. Since that time, the database used to develop the rock mass classifications with *RMR* has increased in size and the system has been successively refined until its most recent iteration in Bieniawski (1989). In some cases, the refinements were quite significant [e.g., use of ISRM (1978) rock mass descriptions (Bieniawski 1979)], and it is advisable to note which version of the system is used to provide a classification when communicating with other engineers. Generally, the *RMR* value allows an estimate of rock strength parameters. A total of six parameters are used to classify a rock mass using the *RMR* system: (1) *UCS* of intact rock material; (2) *RQD*; (3) joint or discontinuity spacing; (4) joint condition; (5) groundwater condition; and (6) Joint orientation (Bieniawski 1989). Table 3.13 presents the *RMR* system based on Bieniawski (1989). *RMR* classification can help estimate

many aspects related to underground excavations, including unsupported span, stand-up time, bridge action period, support pressure, strength parameters (i.e., cohesion and friction angle), modulus of deformation, and allowable bearing pressure. This information is extremely useful in selecting the method of excavation and the permanent support system.

A. C	LASSIFICAT	ION PARAMETERS AND	THEIR RATINGS						
⊢		Parameter			Range of values				
⊢	Strength Point-load >10 MPa of strength index		4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred			
1	intact ro materi	ck Uniaxial comp. al strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
L		Rating	15	12	7	4	2	1	0
	Dri	Drill core Quality RQD 90% - 100%		75% - 90%	50% - 75%	25% - 50%	< 25%		
2		Rating	20	17	13	8	3		
		Spacing of	> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
3		Rating	20	15	10	8		5	
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		thick nm
L		Rating	30	25	20	10		0	
Γ		Inflow per 10 m tunnel length (I/m)	None	< 10	10 - 25	25 - 125		> 125	
5	Groundwa ter	(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
L		General conditions	Completely dry	Damp	Wet	Dripping		Flowing	
		Rating	15	10	7	4		0	
B. R	ATING ADJ	USTMENT FOR DISCONT	INUITY ORIENTATIONS (See	• F)					
Strik	e and dip ori	entations	Very favourable	Favourable	Fair	Unfavourable	Very	Unfavou	rable
Ratings		Tunnels & mines	0	-2	-5	-10		-12	
		Foundations	0	-2	-7	-15		-25	
⊢		Slopes	0	-5	-25	-50			
C. R	OCK MASS	CLASSES DETERMINED	FROM TOTAL RATINGS						
Rating		100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21			
Class number I		I	1		IV	v			
Des	ription		Very good rock	Good rock	Fair rock	Poor rock	Ve	ry poor ro	xck
D. M	EANING OF	ROCK CLASSES							
Clas	Class number		00	II	III	IV IV	V		
Average stand-up time 20 yrs for 15 m		20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span		span	
Con	esion of rock	mass (KPa)	> 400	300 - 400	200 - 300	100 - 200	< 100		
Frict	ion angle of	rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25		< 15	
E. G	UIDELINES	FOR CLASSIFICATION O	F DISCONTINUITY condition	1.0-	0.40-	40.00-		- 00	
Ratir	Discontinuity length (persistence) Rating		<1m 6	4	2	10 - 20 m	> 20 m 0		
Sepa Ratir	Separation (aperture) Rating		None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0		
Roughness Rating		Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0		d	
Infilling (gouge) None Rating 6		None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0			
Weathering Unweathered Ratings 6		Slightly weathered 5	d Moderately weathered Highly weathered 3 1		Decomposed 0				
F. E	FFECT OF D	ISCONTINUITY STRIKE A	AND DIP ORIENTATION IN T	JNNELLING**					
		Strike perpe	ndicular to tunnel axis		ę	Strike parallel to tunnel axis			
	Drive w	vith dip - Dip 45 - 90°	Drive with dip	- Dip 20 - 45°	Dip 45 - 90°		Dip 20 - 45°		
	V	ery favourable	Favou	rable	Very unfavourable		Fair		
	Drive ag	ainst dip - Dip 45-90°	Drive against d	ip - Dip 20-45°	Dip 0-20 - Irrespective of strike ^o				
		Fair	Unfavo	urable	Fair				
Fair			Untavo	urable	Fair				

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly. ** Modified after Wickham et al (1972).

Table 3.13: Summary of Rock Mass Rating (RMR) system parameters (after Bieniawski 1989).

The inputs necessary to determine a *RMR* value overlap significantly with those necessary to determine the *Q*-value. Given this link, it is unsurprising that *RMR* and Q values are statistically correlated and that they provide similar tunnel support recommendations (Pariseau 2011). For example, one the first correlations between these two parameters is discussed in Bieniawski (1976):

$$RMR = 9\ln Q + 44$$
 (3-33)

Equation 3-33 was developed based on a large database of *RMR* and *Q* measurements [117 total case histories (68 in Scandinavia, 28 in South Africa, and 21 in USA)]. Since the classic Bieniawski (1976) relationship, multiple *RMR-Q* correlations using the same functional form as Eq. 3-33 have been developed based on different sets of case histories (Table 3.14). It is unsurprising that multiple correlations can be developed given that the two ratings systems take into account different rock mass parameters (e.g., uniaxial compressive strength of the intact rock and orientation of the rock fractures in the *RMR* system and stress influence in the *Q* system).

RMR	Reference		
$9 \ln Q + 44$	Bieniawski (1976)		
$5.9 \ln Q + 43$	Rutledge and Preston (1978)		
$5.4 \ln Q + 55.2$	Moreno Tallon (1980)		
$5 \ln Q + 60.8$	Cameron-Clarke and Budavari (1981)		
$10.5 \ln Q + 41.8$	Abad et al. (1984)		
$15\log Q + 50$	Barton (1995)		

Table 3.14: Summary of *RMR-Q* relationships.

Since the *Q*-value can be related empirically to V_P , it follows that *RMR* can also be estimated in a similar manner, as demonstrated below in a relationship proposed by Sunwoo and Hwang (2001) based on data acquired across multiple sites with different geological conditions in Korea:

$$RMR = 6 \times 10^{\frac{V_p - 3.5}{3.96}} + 47 \tag{3-34}$$

where V_P is input in km/s. Moreover, *RMR* can be estimated directly using Eq. 3-33 (among other similar proposed relationships) once *Q* has been estimated using V_P .

3.1.8 MASS DENSITY

The mass density (and, by extension, unit weight) of earth material plays a significant role in estimating stresses. The calculation of stresses proves to be a fundamental step in practically any project involving geotechnical engineering. Mass density is itself affected by the distribution of grains, pore space, and mineralogy. Many of these factors also affect various geophysical measurements. The following sections describe various correlations that exploit this relationship between mass density and geophysical measurements.

3.1.8.1 Seismic Methods

As previously discussed, mass density is inextricably linked to soil moduli. Independent knowledge of the appropriate soil modulus and the corresponding seismic velocity can allow an estimate of ρ . However, it is rare to have these two parameters independently measured when using field-based geophysical approaches. Therefore, a number of researchers have compiled databases where velocities were estimated in the field as part of site subsurface investigation efforts or in the laboratory and ρ values were obtained for the soil profile based on in-situ testing or laboratory testing on undisturbed samples. From these databases, statistical regressions could be performed to empirically relate measured V_s (and/or V_P) with ρ (or γ). Examples of these relationships for rocks were presented previously in the discussion regarding earthwork/grading factor (Figs. 3.26 – 3.27). An example of a general correlation is provided in Fig. 3.40, where the regression is performed with 438 data points from a wide variety of geomaterials including intact rocks, gravels, sands, silts, and clays. The resulting relationship could be expressed as follows:

$$\rho = 0.277 + 0.648 \log V_s \tag{3-35}$$

where V_s is input in m/s and the resulting ρ is in g/cc (Burns and Mayne 1996). Mayne (2007) proposed a similar relationship for the total unit weight that also incorporated the effects of depth:

$$\gamma_t = 8.32 \log V_s - 1.61 \log z \tag{3-36}$$



where V_s is input in m/s, depth (z) is input in meters, and the resulting γ_t has units of kN/m³.

Figure 3.40: Relationship between ρ and V_s (Burns and Mayne 1996).

The database for development of Eq. 3-36 is based on 727 samples of different saturated soils, including soft to stiff clays and silts, loose to dense sands and gravels, and mixed geomaterials (Fig. 3.41). Figure 3.41 also includes data points from intact rocks for comparative purposes (i.e., data not included in the regression for Eq. 3-36). Based on this data, limiting values can be placed on the maximum unit weight (26 kN/m³) and shear wave velocity (V_S = 3300 m/s) of rock for correlating between the two properties.



Figure 3.41: Relationship between γ_t and V_s and depth (Mayne 2007).

The relationship from Eq. 3-36 can also be expressed so that the effects of depth are incorporated into vertical effective stress as a normalization parameter for V_s :

$$\gamma_t = 4.17 \ln V_{s1} - 4.03 \tag{3-37}$$

$$V_{s1} = \frac{V_s}{\left(\frac{\sigma'_{vo}}{\sigma_{atm}}\right)^{0.25}}$$
(3-38)

where γ_t is the total unit weight in kN/m³, V_{SI} is the stress-normalized shear wave velocity in m/s (Eq. 3-38), V_S is the measured shear wave velocity in m/s, σ'_{vo} is the in situ vertical effective stress in kPa, and σ_{atm} is a reference pressure of 1 atm (i.e., 101.3 kPa) (Mayne 2006). Figure 3.42 demonstrates that Eq. 3-37 should allow estimates of γ_t within ± 1 kN/m³. The use of Eq. 3-37 to determine γ_t throughout a soil profile should proceed downward from the ground surface in a stepwise fashion since V_{SI} is itself a function of σ'_{vo} (i.e., depth). Mayne (2006) notes that Eq. 3-37 is for "well-behaved" soils, meaning that there is no observed cementation or unusual structure associated with the soil. Cemented soils and carbonate sands would likely plot to the right of the mean relationships in Fig. 3.42 since the bonding would yield a fast velocity through the soil matrix despite the more open porous structure (and low unit weight).



Figure 3.42: Relationship for γ_t that incorporates the effects of depth based on σ'_{vo} as normalization parameter for V_s (Mayne 2006).

From resonant column testing, Mayne (2006) highlighted a similar relationship for dry unit weight from a much smaller database of reconstituted quartz sands (Fig. 3.43):

$$\gamma_t = 0.06V_{s1} + 2 \tag{3-39}$$

where γ_t is the dry unit weight in kN/m³ and, as before, V_{s1} is the stress-normalized shear wave velocity in m/s (Eq. 3-38).



Figure 3.43: Relationship between γ_d and normalized V_{S1} for reconstituted sands (Mayne 2006).

Care should be exercised when applying any of these equations on partially saturated soils. The capillary forces resulting from partial saturation can drastically change the small strain stiffness (and V_s by extension) depending on the gradation of the soil (Cho and Santamarina 2001). Since Eqs. 3-35 - 3-39 were not calibrated based on partially saturated soils, it is advisable to adjust any field measured V_s to account for differences in interparticle forces from partial saturation. In doing so, field estimates of V_P from seismic geophysical methods can be used to estimate the saturation (e.g., Yang 2005) and the concepts introduced in Cho and Santamarina (2001) can be used to revise the predicted V_s .

3.1.8.2 Electromagnetic Methods

Electromagnetic methods provide a measurement of the relative permittivity (i.e., dielectric constant) of the propagating medium. Since soil and rock are multi-phase materials, the measured dielectric constant represents a composite value affected by each of the phases. Therefore, electromagnetic methods can allow for insight into the relative composition of the tested earth material. In this manner, electromagnetic methods have been used to correlate to bulk mass density, often in conjunction with estimates of water content. The most commonly applied approach has been the TDR method. This method has seen significant development in the recent past primarily related to its use as a quality control tool for verifying compaction in the field (Siddiqui and Drnevich 1995; Lin et al. 2000; Siddiqui et al. 2000; Drnevich et al. 2003; Yu and Drnevich 2004; Sallam et al. 2004; Rathje et al. 2006; Lin et al. 2012). This work has led to the development of ASTM D6780 for this particular purpose.



Figure 3.44: Example of a typical TDR waveform (Lin et al. 2012).

As previously noted, the TDR method relies on evaluations of the reflections from electromagnetic signals traveling along a multi-conductor probe placed in the ground. The initial step pulse generated into the probe is reflected at the soil surface and at the end of the probe. Typically, multiple reflections of the waveform also result within the probe after which a steady state voltage is reached (Fig. 3.44). The difference in time between the reflection at the soil surface and at the probe end allows for a round trip travel time to be determined for the electromagnetic wave. From this information the dielectric constant of the soil can be determined:

$$K_a = \left(\frac{V_c \Delta t}{2L}\right)^2 \tag{3-40}$$

where K_a is the dielectric constant, V_c is the speed of light in air, Δt is the round trip travel time determined from testing, and L is the probe length (Topp et al. 1980). Since small dielectric losses are always present, the TDR-measured relative dielectric permittivity in Eq. 3-40 is referred to as the "apparent" dielectric constant (Topp et al. 1980). The steady state voltage can also be used to measure the electrical conductivity of the soil:

$$EC = K_p \left(\frac{2V_o}{V_{\infty}} - 1\right) \tag{3-41}$$

where *EC* represents the electrical conductivity, K_{ρ} is a constant related to probe geometry and source impedance (Ball 2002; Dallinger 2006) and can be determined experimentally with measurements on electrolytic solutions with known *EC*, V_o is the magnitude of the step input voltage used to generate the electromagnetic wave in the probe, and V_{∞} is the steady state voltage recorded by the TDR sensor (Giese and Tiemann 1975). This equation is derived based on two assumptions: (1) cable resistance is neglected; and (2) the characteristic impedance of the TRD sensor and of the transmission line perfectly match. Modifications to account for these two assumptions have been proposed by Lin et al. (2008) to improve measurements of *EC*. The K_a and *EC* measurements from Eqs. 3-40 and 3-41 can then be used to estimate soil phase properties (including mass density) using the following expressions:

$$\frac{\sqrt{K_a}\rho_w}{\rho_a} = a + bw \tag{3-42}$$

$$\frac{\sqrt{EC}\rho_w}{\rho_d} = c + dw \tag{3-43}$$

where *w* is the gravimetric water content, ρ_w is the mass density of water, ρ_d is the dry density of the tested soil, and *a,b,c*, and *d* are constants to be calibrated using a laboratory standard Proctor compaction test as exemplified in Figs. 3.45 – 3.46 (Yu and Drnevich 2004). Parameters *a* and *c* are primarily related to the tested soil type, *b* does not vary significantly, and *d* represents the effects of the pore fluid *EC* (Lin et al. 2012).

Assuming both K_a and *EC* are known from TDR field measurements, Eqs. 3-42 and 3-43 represent two equations with two unknowns, which means they can be solved simultaneously to obtain wand ρ_d . This method is commonly referred to as the One-Step Method and "Procedure B" in ASTM D6780. In this case, calibration constant d may differ in the field from the laboratoryderived value because there may be differences in pore fluid conductivity between the two settings. Yu and Drnevich (2004) proposed a methodology to adjust the field measurements to allow laboratory calibrations to remain applicable. This method relies on the fact that K_a is relatively insensitive to the *EC* of the pore fluid. In this approach, Eqs. 3-42 and 3-43 are combined to form a relationship between *EC* and K_a :

$$\sqrt{EC} = f + g\sqrt{K_a} \tag{3-44}$$

where f and g are new calibration constants related to the previously defined calibration constants. These f and g calibration constants are derived based on laboratory measurements (e.g., Fig. 3.47) and then applied to Eq. 3-44 so that an adjusted field *EC* can be estimated from the K_a measured in the field using Eq. 3-40. This adjusted *EC* represents the field *EC* as if the pore fluid in the field was replaced by the pore fluid used in the laboratory for calibration.



Figure 3.45: Examples of calibration between normalized apparent dielectric constant and *w* for TDR testing (Lin et al. 2012).



Figure 3.46: Examples of calibration between normalized electrical conductivity and *w* for TDR testing (Lin et al. 2012).



Figure 3.47: Examples of calibration between electrical conductivity and apparent dielectric constant for TDR testing (Lin et al. 2012).

If only measurements of K_a are made in the field, then Eq. 3-42 represents a single equation with two unknowns. This method is commonly referred to as the Two-Step Method and "Procedure A" in ASTM D6780. This highlights the fact that an extra step must be taken in the field to determine gravimetric water content and mass density information. A portion of the soil from where the original TDR measurements were made must be removed, placed in a compaction mold, and weighed so that total mass density (ρ_t) can be estimated. A second K_a measurement is also made on the compaction mold sample with the TDR sensor and probe. Since $\rho_t [= \rho_d (1+w)]$ and K_a in the mold are both known, the field *w* can be estimated based on assuming it is the same as in the mold:

$$w_{field} = w_{mold} = \frac{\sqrt{K_{a,mold}} - \frac{a\rho_{t,mold}}{\rho_w}}{\frac{b\rho_{t,mold}}{\rho_w} - \sqrt{K_{a,mold}}}$$
(3-45)

Once the *w* is obtained, the field ρ_d can then be determined from the two measurements of K_a as follows:

$$\rho_{d,field} = \frac{\sqrt{K_{a,field}}}{\sqrt{K_{a,mold}}} \times \frac{\rho_{t,mold}}{1 + w_{mold}}$$
(3-46)

In this manner, field compaction specifications can be checked rapidly in the field as in Lin et al. (2012).

Measurements of w and ρ_d from TDR have shown reasonable agreement (e.g., w measurements within 1%, ρ_d within 5%) with those estimated from sand cone testing (Fig. 3.48 and 3.49), particularly when a soil-specific calibration is performed rather than reliance on a general calibration from all soils. However, there are a number of limitations with the method in its current form. First, high clay content soils can cause dispersion in electromagnetic waves (e.g., West et al. 2003), which can lead to higher apparent dielectric constants than coarse grained soils (Lin et al. 2012). This means the assumption that dielectric-based TDR measurements of w

and ρ_d are independent of soil type may lead to discrepancies in the results, particularly when general calibration is performed.



Figure 3.48: Comparison of *w* measured in the field using TDR and *w* obtained from oven drying (Lin et al. 2012).



Figure 3.49: Comparison of γ_d measured in the field using TDR and sand cone tests (Lin et al. 2012).

Full scale laboratory evaluations have demonstrated that the One-Step method and the Two-Step method both perform similarly (e.g., Lin et al. 2012). However, the One-Step method is affected by issues with the empirical adjustment process to account for pore fluid conductivity. This leads to a systematic error proportional to the difference between field dry density and mold dry density during calibration (Lin et al. 2012). Both methods can be negatively affected by the penetration disturbance caused by the field TDR probe (Lin et al. 2006a,b; Lin et al. 2012). The amount of error introduced by penetration disturbance can be reduced by using a smaller diameter probe or by predrilling the hole for probe insertion. Finally, it is not possible to measure w and ρ_d solely with TDR without some sort of physical soil sample for calibration. However, recent studies have explored combining TDR with other measurements such as thermal conductivity to remove the need for calibration with a physical sample (e.g., Zhang et al. 2015). It should be noted that though this approach has overwhelmingly focused on compaction quality assurance, it can be used in any situation where estimates of the *w* and ρ of the surficial soils are desired.

3.1.8.3 Gravity Methods

Use of the microgravity technique to establish quantitative measurements of mass density necessitates significant data post-processing (e.g., Sissons 1981). An inversion process must be used to deduce the subsurface model that best represents the surficial gravity observations after corrections for elevation, topography, tidal fluctuations, and regional anomalies. The inversion must be constrained by a priori information regarding the density of subsurface earth materials, either through direct measurements (e.g., sampling and laboratory testing, in situ testing, other geophysical methods, etc.) or indirect assessments (e.g., average values based on anticipated subsurface geologic units). Unfortunately, the inherent non-uniqueness of inversion algorithms and the need for a priori information signifies that empirical relationships between microgravity measurements and density are not practical. However, a significant number of studies have been performed that have evaluated the subsurface spatial distribution of density based on surficial gravity measurements, which demonstrates the method's effectiveness for this application. For example, Rim et al. (2005) discusses the use of microgravity to determine the density variation within the interior of a rock fill dam and Styles et al. (2005) discusses its use for mapping cavity locations based on density (Fig. 3.50).

Hayashi et al. (2005) discusses a case history where microgravity and MASW surveys were performed to delineate a buried channel filled with soft alluvium sediments. In this study, inversion of the gravity data was constrained by using estimated density values derived from empirical relationships between density and the shear wave velocity obtained from the surface wave measurements. The resulting density distribution (Fig. 3.51) was used with the shear wave velocity model to estimate spatial distribution of shear modulus and map the soft alluvial sediments. The Hayashi et al. (2005) study demonstrates how other geophysical methods could

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be used to supplement microgravity surveys and provide useful a priori information to constrain the inversion of microgravity data. A similar concept was proposed by Lines et al. (1988) using the results from sonic logging and borehole gravity measurements. Other similar approaches have been reported by Pilkington (2006), Mochales et al. (2008), Orfanos and Apostolopoulos (2011), and Paine et al. (2012), typically in combination with electrical or magnetic methods (e.g., resistivity, GPR, etc.).



Figure 3.50: Example microgravity results: (a) Residual gravity measurements. Black dots represent gravity stations and white lines represent potential cavities (solid = probable, dashed = possible). (b) Resulting 3D map of cavity location/thickness (Styles et al. 2005).



Figure 3.51: Delineation of soft alluvial sediments using microgravity measurements and constrained inversion with an MASW V_s profile (Hayashi et al. 2005).

In addition to information from other geophysical methods, drilling information (e.g., in situ testing) and laboratory measurements of density can be used to supplement microgravity studies. For example, Whitelaw et al. (2008) demonstrates the use of drilling information as an inversion constraint. Whitelaw et al. (2008) used microgravity results to estimate density variation and image a sinkhole basin at the Gray Fossil Site in Washington County, Tennessee. In

this study, the gravity inversion was constrained by laboratory measurements from borehole samples drilled in the area by the Tennessee Department of Transportation. The resulting density contrast images demonstrated the presence of 11 individual sinkholes within the basin (Fig. 3.52).



Figure 3.52: 3D ρ contrast models estimated using microgravity measurements: (a) Two vertical slices through model (with regions of interest numbered); and (b) Two horizontal slices through model (with regions of interest lettered) (after Whitelaw et al. 2008).

Another study that used existing drilling data was performed by Mankhemthong et al. (2012), where the spatial distribution of density was being studied to map an existing fault zone separating two distinct geological settings. Mankhemthong et al. (2012) used existing well logging data and density measurements from rock samples to serve as constraints for inversion of gravity measurements. The resulting estimates of density highlighted the two distinct geological terrains on either side of the known fault zone in the study (Fig. 3.53).



Figure 3.53: Estimates of near surface bulk ρ across Border Ranges Fault System in Alaska. Vertical lines represent uncertainty in estimated ρ and horizontal lines represent spatial extent of gravity measurements for a given lettered zone (Mankhemthong et al. 2012).

The determination of density from microgravity surveys is more straightforward in cases where the measurements are performed directly on top of each other. In such cases, it is possible to develop a direct expression relating density to the measurements of gravity. For example, if a gravity measurement can be made at the surface and at a depth immediately below that location, the density can be estimated using the following relationships:

$$\rho (g/cm^{3}) = 3.68 - \frac{11.93(\Delta g - \varepsilon_{T})}{\Delta z}$$
(3-47)

$$\rho \left(g/cm^{3} \right) = 3.68 - \frac{39.06(\Delta g - \varepsilon_{T})}{\Delta z'}$$
(3-48)

where ρ is the mass density expressed in units of g/cm³, Δg is the difference in the gravity measurements, Δz is the elevation difference in meters ($\Delta z'$ is the elevation difference in feet), and ε_{T} is the difference in terrain corrections expressed in units of mGal (Telford et al. 1990). However, despite being direct expressions between gravity and density, application of Eqs. 3-47 and 3-48 still involves successive approximations because the ε_{T} term depends on ρ . Gravity measurements at locations directly on top of each other can be accomplished in a number of ways. In some cases, the unique circumstances of the site conditions allow for repeated measurements at different depths over the same location. For example, Harris et al. (2013) estimated the density of waste material at a bioreactor landfill using existing settlement gauge information in combination with multiple microgravity surveys performed over several years. As development of each landfill cell progressed, the microgravity surveys could be repeated over the same locations as the height of the waste material increased. Harris et al. (2013) was able to use this approach to estimate the spatial distribution of density for placement of different landfill waste cells (Fig. 3.54).



Figure 3.54: Estimates of waste ρ between initial survey and final survey in Harris et al. (2013) study.

In other cases, measurements at different elevations at the same location are accomplished using a tripod or tower structure (e.g., Butler 1984). The most direct approach for taking gravity measurements directly on top of each other is with a borehole gravimeter (BHGM). In these cases, a terrain correction is unnecessary since the gravity measurements are made below the surface (Telford et al. 1990). A generalized form can then be developed for the relationship expressed in Eqs. 3-47 and 3-48:

$$\rho = \frac{F}{4\pi G} - \frac{\Delta g / \Delta z}{4\pi G}$$
(3-49)

where ρ is the mass density, Δq is the difference in the gravity measurements, Δz is the elevation difference, F is the free-air gradient (typically 0.3086 mGal/m or 0.09406 mGal/ft), and G is the universal gravitational constant (6.674 x 10^{-11} m³/kg s²) (Smith 1950; Hammer 1950; LaFehr 1983). Eq. 3-49 assumes that the BHGM passes through uniformly thick and laterally homogeneous strata of constant density (LaFehr 1983). The radius of investigation for borehole gravity measurements is directly dependent on the different in elevation levels where the measurements were recorded. Telford et al. (1990) notes that half of the effect on borehole gravity measurements is produced by an area within 0.7 Δz of the borehole, 80% from 2.45 Δz , and 90% from within 5 Δz , where Δz as before represents the difference in elevation between the corresponding gravity measurements. Thus, compared to other borehole density logging methods, borehole gravity measurements can be made with less influence from borehole fluids, rugosity, casings, and drilling operations (Beyer and Clutsom 1978). More accurate density measurements as a function of depth offers tremendous benefits to applications related to exploration for petroleum. Therefore, it is not surprising that the first BHGMs were developed in the late 1950s for oil exploration (e.g., Howell et al. 1966) and that the borehole gravity technique enjoyed a surge of popularity upon its development. A large number of case histories exist in the literature where borehole gravity measurements were made on rock for petroleum engineering purposes, particularly in the time frame immediately following development of the first BHGM (Hammer 1950; Hammer 1965; McCulloh 1965; Jageler 1976; Beyer and Clutsom 1978; Schultz 1989; Popta et al. 1990; MacQueen 2007, Brady et al. 2013). Robbins (1989) provides an annotated bibliography of pertinent literature and documented borehole gravity case histories through the 1980s. Application of borehole gravity for traditional geotechnical engineering purposes has been less prevalent (e.g., Healey et al. 1984), but as equipment costs decrease with time it is expected that borehole gravity measurements can offer a highly useful and accurate approach to estimate in situ mass density without the errors present in traditional geotechnical sampling and borehole density logging techniques.

3.1.8.4 Nuclear Methods

As previously noted in this document, a number of surface and subsurface methods have been developed that detect the presence of radiation in the surrounding soil, including background levels of radiation (e.g., gamma logging) and induced backscatter from active sources of radioactive isotopes (e.g., gamma-gamma logging). One of the most common applications of

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nuclear methods is to estimate the bulk density of the surrounding material in diverse applications ranging from wireline formation logging in petroleum engineering to quality control of compacted soils, cast in drilled hole (CIDH) foundations, and asphalt pavements (Faul and Tittle 1951; Stroup-Gardiner and Newcomb 1988; McCook and Shanklin 2000; Liebich 2004; Sanders et al. 1994). Typically, a gamma-gamma logging tool is used in these applications. The flux of gamma rays that reaches a detector decays exponentially with distance from the source and with the number of electron scatterers in the travel path assuming Compton scattering is the primary mechanism (Tittman and Wahl 1965; Keys 1989; Schlumbeger 1991):

$$G = G_o e^{-n_e C_S x} \tag{3-50}$$

where *G* is the gamma ray flux measured at the detector in counts per second (cps)/cm², *G*_o is the initial gamma ray flux emitted from the source in cps/cm², *n*_e is the number density of electrons in electrons/cm³, *C*_s is the cross section of each scatter center in cm², and *x* is the distance from source to detector in cm. The density of electrons in the material is in turn directly related to bulk density:

$$n_e = \frac{NZ}{A}\rho_b \tag{3-51}$$

where *N* is the Avogadro constant (6.02 x 10^{23} electrons/mole) of the material, *Z* is the atomic number of the material (no units), *A* is the atomic weight of the material in g/mole, and ρ_b is the bulk density of the material in g/cm³. Based on Eqs. 3-50 and 3-51, materials with a smaller electron number density (i.e., smaller bulk density) will result in less attenuation of the gamma rays and a higher gamma ray count. A useful parameter related to the electron density can be defined as follows:

$$\rho_e = \frac{2 n_e}{N} \tag{3-52}$$

where ρ_e is the effective electron number density in units of mole/cm³. Eq. 3-51 can then be modified based on this new parameter:

$$\rho_e = \frac{2Z}{A}\rho_b \tag{3-53}$$

This transformation allows the measured electron density from the gamma-gamma log to transform directly to bulk density because the ratio of Z/A is typically very close to 0.5 for most common elements that make up earth materials (Bertozzi et al. 1981). However, hydrogen (present in water) has a ratio of Z/A closer to 1. This signifies that electron density index of water is 11% larger than its bulk density would imply based on these equations. To account for this, Gaymard and Poupon (1968) proposed that the density inferred from gamma ray scattering be modified as follows:

$$\rho_a = 1.0704\rho_e - 0.188 \tag{3-54}$$

where ρ_e is the "apparent" number density. In modern gamma-gamma logging tools, two detectors are used to measure the gamma ray flux at different distances from the source. The readings taken at the detector farthest from the source is used to estimate the density of the surrounding material by combining Eqs. 3-50 - 3-54 (Fig. 3.55). The difference between the readings at the farthest detector and from the closer detector can be analyzed to correct for the rugosity of the borehole walls and the effects of any drilling fluid caked onto the walls (Tittman 1986; Ellis 1987; Gearhart 1989). A key aspect of gamma-gamma logging is also ensuring proper calibration of the equipment on material of known bulk densities. Calibration can be performed in test pits like the American Petroleum Institute neutron pit in Houston, TX or in commercially available pits across the country. For on-site calibration, test blocks of material with predetermined densities can be employed.



Figure 3.55: Example density log from gamma-gamma testing (Yearsley et al. 1991).

A few items are worthy of discussion regarding operation of a gamma-gamma log. First, the equations presented are necessary in estimating bulk density from older instruments that display gamma measurements in counts per second. Most modern equipment displays its measurements in the standard American Petroleum Institute (API) unit, which is based on a reference standard of an artificially radioactive concrete block at the University of Houston, TX that is defined to have a radioactivity of 200 American Petroleum Institute (API) units. Based on the calibration, the equipment provides an automatic output of the density measurement along with the measured API of the material. It should also be noted that an estimate of porosity is often included with the output based on assumptions regarding the pore fluid and the specific gravity of the minerals present in the tested material. Surface-based gamma-gamma logging equipment (i.e., surface nuclear gauge) relies on the same concepts of the borehole method, but also typically includes instrumentation to perform neutron logging to estimate moisture content. Great care is placed in nuclear gauge documentation to highlight the calibration process (e.g., Rawitz et al. 1982; Ellis et al. 1985; Ward and van Deventer 1993; ASTM

D7759/D7759M – 14). When properly calibrated, the measured densities from both boreholeand surface-based tools agree well with other measurements (Figs. 3.56 – 3.57).



Figure 3.56: Example of comparisons between lab bulk ρ and those obtained using gammagamma logging at two test sites in Alberta, Canada (Hoffman et al. 1991).



Figure 3.57: Example of comparisons between γ obtained using sand cone testing and surface nuclear gauge testing at a test site in Schenectady County, New York (Mintzer 1961).

3.1.9 POROSITY

The amount of pore space in a soil is a useful parameter for a number of engineering applications as it is directly related to several aspects of soil behavior, including relative density, permeability, and strength. The following sections describe the relationships that exist between geophysical measurements and porosity of soils and/or rock. Typically, these relationships have been established for porosity since many are derived from research in the geophysics area (particularly as related to rock porosity). However, porosity can easily be converted to the void ratio parameter more commonly discussed within the context of geotechnical engineering.

3.1.9.1 Seismic Methods

As previously noted, travel times obtained from seismic methods allow an estimate of the elastic wave velocities of earth materials (and therefore their moduli). These moduli are affected by the different phases in a multi-phase material such as soil/rock. For example, the bulk modulus of a saturated soil is a combination of the bulk modulus of the soil skeleton, the fluid in the pore space (typically water), and the bulk modulus of the minerals that make up the soil grains. As a result, the relative amount of void space can affect the measured velocities of porous media, as demonstrated in the Biot-Gassmann low frequency asymptotic solution for P-wave velocity of a saturated porous solid material (Biot 1956a, Biot 1956b):

$$V_{p} = \sqrt{\frac{\left(B_{sk} + \frac{4}{3}G_{sk}\right) + \left(\frac{\eta}{B_{w}} + \frac{1-\eta}{B_{g}}\right)^{-1}}{(1-\eta)\rho_{g} + \eta\rho_{w}}}$$
(3-55)

where B_{sk} is the bulk modulus of the soil skeleton, B_w is the bulk modulus of water, B_g is the bulk modulus of the minerals that make up the soil grains, G_{sk} is the shear modulus of the soil skeleton (same as the saturated soil since the presence of fluids does not affect shear modulus), η is the porosity of the soil, ρ_g is the mass density of the minerals that make up the soil grains, and ρ_w is the mass density of water. Note that both η (geotechnical) and ϕ (geology/petroleum engineering) will be used interchangeably in this document to represent porosity and recall that porosity is directly related to void ratio [i.e., $\eta = e/(1+e)$]. Fig. 3.58 presents the effect of porosity on P-wave velocity based on the Biot-Gassmann solution in Eq. 3-55 for a saturated soil with V_s less than approximately 400 m/s. Relationships such as these allow the estimation of porosity based on measured velocities from geophysical measurements.



Figure 3.58: V_P and η relationship based on Biot-Gassmann solution (adapted from Santamarina et al. 2001). Note: $G_s = 2.65$, $B_g >> B_{sk}$ (appropriate for soils with $V_s \leq 400$ m/s).

Other researchers have utilized the theories developed by Biot to explore the variation of Pwave velocity with porosity. For example, Foti et al. (2002) and Foti and Lancellotta (2004) used Biot theory to develop a direct expression to determine porosity based on measured P-wave and S-wave velocities:

$$\eta = \frac{\rho_g - \sqrt{(\rho_g)^2 - \frac{4(\rho_g - \rho_w)B_w}{V_p^2 - 2\left(\frac{1 - \nu}{1 - 2\nu}\right)V_s^2}}}{2(\rho_g - \rho_w)}$$
(3-56)

Eq. 3-56 assumes that the soil grains are incompressible and that no relative motion occurs between the solid and fluid phases in the soils (valid at low frequencies). The only term in Eq. 3-56 that does not have associated standard values is the Possion's ratio (v) of the soil, which typically varies between 0.1 to 0.4 in most soils depending on stiffness and drainage conditions. However, Foti et al. (2002) demonstrated that the results were relatively insensitive to v. Foti et al. (2002) and Foti and Lancellotta (2004) verified Eq. 3-56 based on laboratory porosity measurements of high quality undisturbed samples and V_P and V_S results using bender elements. Eq. 3-56 was then applied to crosshole and downhole field measurements at a number of field sites in Italy and Canada and compared to laboratory measurements of porosity on high quality undisturbed samples. Foti and Lancellotta (2004) generally found good agreement (typically within an average of 10% difference) between estimates of porosity using Eq. 3-56 and those obtained independently via sampling and laboratory testing (Fig. 3.59).



Figure 3.59: Comparison of results using Foti et al. (2002) relationship between velocity and η : (a) Sample field site in Florence, Italy; and (b) All data in Foti and Lancellotta (2004).

In general, the wave velocities of earth material are dependent on multiple factors not directly highlighted in Eqs. 3-55 and 3-56, including overburden stress, grain size and distribution,

structure, and degree of saturation (Zimmer et al. 2006). Therefore, empirical relationships have been developed from data that incorporate some of these effects directly. Some of these relationships explicitly relate V_P or V_S (or their ratio) directly to porosity (or void ratio). For example, Salem (2000) developed an empirical relationship based on in situ seismic refraction measurements of glacial deposits in northern Germany:

$$\frac{V_p}{V_s} = 4.0665 - 0.042617\phi \tag{3-57}$$

where ϕ represents the porosity expressed as a percentage (Fig. 3.60 presents the raw data and statistical information regarding Eq. 3-57).



Figure 3.60: Relationship between V_P/V_S ratio and porosity proposed by Salem (2000).

Hunter (2003) compiled velocity and porosity data from boreholes logged by the Geological Survey of Canada (GSC) and proposed a series of empirical relationships:

$$\phi = 0.2714 + 4.192e^{-V_p/542.4} \tag{3-58}$$

$$\phi = 1.396 - 0.1600 \ln V_s \tag{3-59}$$

$$\phi = 0.1963 \ln\left(\frac{V_p}{V_s}\right) + 0.1523 \tag{3-60}$$

Hunter (2003) found that the V_s data was better constrained than V_P data. Care should be exercised with these relationships as there is significant scatter in the data (Fig. 3.61).



Figure 3.61: Velocity variations with porosity for Holocene and Pleistocene sediments in Hunter (2003): (a) V_P data; (b) V_S data; (c) V_P/V_S data.

Other studies have incorporated a number of other parameters into the velocity-porosity relationship and can be used to estimate porosity/void ratio assuming information about the other parameters is known or inferred [e.g., Taylor Smith (1986) as demonstrated in Fig. 3.62].


Figure 3.62: Variation of V_P with pressure and *e* for various soils (after Taylor Smith 1986).

One of the simplest approaches has been to relate velocity to both porosity/void ratio and stress. For example, Robertson et al. (1995) proposed an empirical relationship based on fitting V_s data for reconstituted samples of Ottawa sand:

$$V_s = (A - Be) \left(\frac{\sigma'}{p_a}\right)^{\frac{n}{2}}$$
(3-61)

where *A*, *B*, and *n* are empirical coefficients (A = 381, B = 259, and n = 0.52), *e* is the void ratio, σ' is the effective stress, and p_a is atmospheric pressure. Robertson et al. (1995) cautioned that the empirical coefficients determined in their study were limited to clean, uncemented, freshly deposited Ottawa sand and should be established for other sediments through additional laboratory testing (for example see Zimmer 2003). Jarvis and Knight (2002) proposed a similar relationship for sands based on seismic reflection measurements and comparisons to laboratory void ratio measurements on frozen undisturbed samples from an aquifer in British Columbia:

$$e = 2.6 - \frac{V_s}{37\sigma^{\frac{1}{4}}}$$
(3-62)

where σ is the confining pressure. Jarvis and Knight (2002) found excellent agreement using this relationship and were able to evaluate the spatial distribution of hydraulic properties of the aquifer in their study.

Plasticity Index (<i>PI</i>)	k	
0	0.00	
20	0.18	
40	0.30	
60	0.41	
80	0.48	
≥ 100	0.5	

Table 3.15: Variation of *OCR* exponent k with *PI* for Hardin and Black (1968) empirical relationship between σ' , e, and G (after Hardin and Drnevich 1972).

Other relationships have included a number of other factors that characterize soil structure. In such cases, knowledge of various parameters related to soil structure (e.g., OCR, stress state, etc.) in addition to velocity estimates would allow determination of void ratio and porosity information. For example, Hardin and Black (1968) introduced an empirical relationship between effective pressure, porosity, and shear modulus of soil for low pressures (<0.7 MPa) based on resonant column testing:

$$G_{ij}^{e} = \frac{OCR^{k}}{F(e)} \frac{S_{ij}}{2(1+\nu)} p_{a}^{1-n} (\dot{\sigma}_{i} \dot{\sigma}_{j})^{\frac{n}{2}}$$
(3-63)

where $G^{e_{i,j}}$ is the shear modulus on a plane with principal stresses of σ_i' (direction of propagation) and σ_j' (direction of particle motion), v is the Poisson's ratio, S_{ij} is a multiplier that accounts for textural factors and structural anisotropy (can vary from as low as 700 for cohesive soils to larger than 1400 for uniform granular or cemented soils), n accounts for the effects of stress and is typically close to 0.5 for many soils, p_a is the atmospheric pressure, k is a function of plasticity index (PI) and is generally close to 0 for sands and increases as PI increases (Table 3.15), *OCR* is the overconsolidation ratio, and F(e) is a function that accounts for the effects of voids on the shear modulus (Hardin and Black 1968; Hardin and Black 1969; Hardin and Drnevich

1972; Yu and Richart 1984; Hardin and Blandford 1989; Hryciw and Thomann 1993; Zimmer 2003). A number of relationships have been proposed for the functional form of F(*e*), including $F(e) = (1+e)/(2.97 - e)^2$ (Hardin and Drnevich 1972), $F(e) = 0.3 + 0.7e^2$ (Hardin and Blandford 1989), and $F(e) = e^{1.3}$ (Jamiolkowski et al. 1991). Kramer (1996) suggested based on the laboratory data that $S_{ij}/2(1+v) \approx 625$ was a good estimate for most applications of Eq. 3-63. Determination of V_S from seismic reflection/refraction measurements and knowledge of the effective stress (i.e., depth) and stress history of the soil would allow an estimate of void ratio using Eq. 3-63 and 3-2.

The mechanical behavior of rocks can differ significantly from soils. For example, cementation at grain contacts and the presence of fractures play a major role in rock mechanics. Owing to these differences, a number of researchers have attempted to develop rock-specific relationships between velocity and porosity. These studies were driven by the increasing need to evaluate lithology and rock formations during continuous well logging for petroleum exploration. Therefore the velocities in these studies were often obtained by downhole geophysics (e.g., sonic logging) or using pulse techniques with instrumentation such as transducers on laboratory samples subjected to in situ effective stresses. However, the proposed relationships can be applied to the P- and S-wave velocities obtained from other seismic methods to determine porosity variation for bedrock at a site (at a reduced resolution in depth interval relative to sonic logging). Domenico (1984) provides a review of empirical relationships proposed between rock porosity and velocities. One of the earliest was the "time-average" equation proposed by Wyllie et al. (1956, 1958) based on ultrasonic testing of natural and synthetically-created laboratory samples:

$$\frac{1}{V_p} = \frac{\phi}{V_f} + \frac{1 - \phi}{V_{p,m}}$$
(3-64)

where V_P is the measured P-wave velocity, V_f is the P-wave velocity of the fluid in the pore space, $V_{P,m}$ is the P-wave velocity of the solid rock matrix (i.e., velocity of the mineral grains), and ϕ represents porosity. Wyllie et al. (1956, 1958) cautioned that Eq. 3-64 was developed for "clean" water-saturated sandstones and is not suitable for carbonate rocks subject to fractures and large cavities. Additionally, the relationship is less suitable at low confining pressures and when the rock is poorly consolidated (Castagna et al. 1993). Therefore, Eq. 3-64 should not be applied to estimate porosity of soils. Application of Eq. 3-64 assumes a-priori knowledge of the P-wave velocity of the rock matrix (i.e., mineral grains), which is non-trivial, particularly in cases of mixed lithologies. For relatively "pure" rocks of a single mineralogy, Table 3.16 can be utilized to approximate V_m . Eq. 3-64 can be rearranged into a more general expression:

$$\frac{1}{V} = A + B\phi \tag{3-65}$$

where constants *A* and *B* are determined empirically. The *A* constant represents the dependency of porosity on the solid rock matrix, while *B* captures a number of other contributing factors such as consolidation, pore geometry, and effective overburden stress (Domenico 1984).

Mineral	ρ (g/cc)	V _P (km/s)	<i>Vs</i> (km/s)
Calcite	2.71	6.53	3.36
Calcite	2.71	6.26	3.24
Dolomite	2.87	7.05	4.16
Halite	2.16	4.50	2.59
Muscovite	2.79	5.78	3.33
Quartz	2.65	6.06	4.15
Quartz	2.65	6.05	4.09
Anhydrite	2.96	6.01	3.37

Table 3.16: Reported mineral properties (as adapted from Castagna et al. 1993). Velocities are averaged to represent zero-porosity isotropic aggregates.

The functional form of Eq. 3-65 has been shown to apply to either compressional or shear wave velocity (e.g., King and Fatt 1962; Gregory 1963; Pickett 1963; Domenico 1984; Castagna et al. 1985). Based on the data from Pickett (1963), Domenico (1984) performed regression analysis on velocities of sandstone and limestone and estimated the *A* and *B* parameters for V_P as 163 and 365, respectively, and for V_S as 224 and 889, respectively, where the velocities are expressed in m/s (see Figs. 3.63 – 3.64 and Table 3.17).



Figure 3.63: Reciprocal of V_P and V_S as a function of porosity for sandstones (Domenico 1984).



Figure 3.64: Reciprocal of V_P and V_S as a function of porosity for limestones (Domenico 1984).

		Sandsto	one		,	Limesto	one	
Differential	S-Wave							
pressure (psi)	A	В	S.D.	C.C.	A	В	S.D.	C.C.
500 1000 2000 3000 4000 5000 6000 Avg. A	$\begin{array}{cccccccccccccccccccccccccccccccccccc$		333.4 323.3 318.5 314.1 311.5 309.1 <u>307.3</u> <u>316.7</u> (308.4)**	.4 649.0 .3 451.8 .5 374.8 .1 335.5 .5 304.7 .1 286.9 .3 273.3 .7 .4)**		.85 .89 .89 .89 .88 .88 .88 .87		
Differential		P-Wave						
pressure (psi)	A	В	S.D.	C.C.	A	В	S.D.	C.C.
500 1000 2000 3000 4000 5000 6000 Avg. A	163.1 164.7 165.2 164.9 163.7 162.8 <u>162.7</u> 163.9 (165.1)*	573.8 499.8 427.1 390.4 376.9 370.5 364.2	7.2 5.8 4.2 4.1 3.6 3.5 3.3	.97 .98 .98 .98 .98 .98 .99 .99	171.3 168.7 167.3 166.1 165.1 164.2 <u>163.5</u> <u>166.6</u> (159.8)**	370.8 283.1 241.3 215.4 197.9 186.9 178.8	11.2 9.1 8.5 8.2 7.9 7.8 7.8	.91 .90 .89 .87 .86 .85 .84

Note: Dimensions of A and S.D. are μ s/m. Dimensions of B are μ s/m/unit porosity, that is, for ϕ expressed in fractional porosity.

*Quartz reciprocal velocity **Calcite reciprocal velocity

Table 3.17: Summary of regression constants *A* and *B* for use in empirical relationships between porosity and seismic velocity (Domenico 1984).

A number of additional relationships between velocity and porosity have been proposed over the years to improve or expand on the Wyllie et al. (1956) time-average equation. Some of these expressions maintained the functional form in Eq. 3-65, but expanded the database into other rock types (e.g., Rafavich et al. 1984; Wang et al. 1991). Other researchers established new functional forms for the velocity-porosity relationship (e.g., Watkins et al. 1972; Raymer et al. 1980; Tosaya 1982; Castagna 1985) (Table 3.18). Additionally, a number of studies increased the complexity of the empirical models to account for other factors that can affect the relationship between velocity and porosity, including clay content (Tosaya 1982; Han et al. 1986; Castagna et al. 1993) and effective overburden stress (Eberhart-Phillips et al. 1989). This area of research is ongoing with studies continuing to explore the effects of porosity on rock structure and velocity (e.g., Freund 1992; Jones 1995; Khaksar et al. 1999; Khaksar and Griffiths 2000; Berryman et al. 2002; Fabricius et al. 2007; Fournier and Borgomano 2009; Gomez et al. 2010). Given the many factors that affect the velocities of rocks (e.g., mineralogy, clay content, depositional environment, particle size/shape/packing, degree of cementation, stress state/history, presence of fluids, etc.) there is a significant amount of uncertainty in applying any general porosityvelocity relationship to field data. Calibration of the models presented in this section with direct measurements is highly recommended.

Reference	ence Compression She				
Wyllie et al. (1956)	$\frac{1}{V_p} = \frac{\phi}{V_f} + \frac{1-\phi}{V_{p,m}}$	-			
Raymer et al. (1980)	$V_p = \phi V_f + (1 - \phi)^2 V_{p,m}$	-			
Castagna (1985)	-	$V_{s} = (1-\phi)^2 V_{s,m}$			
Tosaya (1982)	$V_p = \phi V_f + (1 - \phi)^2 V_{p,m}$	$V_s = 3.7 - 6.3\phi - 2.1C$			
Domenico (1984)	$\frac{1}{V_p} = 0.163 + 0.365\phi$	$\frac{1}{V_s} = 0.224 + 0.889\phi$			
Watkins et al. (1972)	$V_p = e^{rac{1.56-\phi}{0.175}}$	-			
Castagna et al. (1993)	$V_p = 5.81 - 9.42\phi - 2.21C$	$V_s = 3.89 - 7.07\phi - 2.04C$			
Han et al. (1986)	$V_p = 5.59 - 6.93\phi - 2.18C$	$V_s = 3.52 - 7.07\phi - 1.89C$			
Eberhart-Phillips et al. (1989)	$V_p = 5.77 - 6.94\phi - 1.73C^{\frac{1}{2}} + 0.446(P_e - e^{-16.7P_e})$	$V_s = 3.70 - 4.94\phi - 1.57C^{\frac{1}{2}} + 0.361(P_e - e^{-16.7P_e})$			

Note: $V_{P,m}$ = P-wave velocity of solid mineral (km/s), $V_{S,m}$ = S-wave velocity of solid mineral (km/s), C = fractional clay content, P_e = effective pressure (MPa)

Table 3.18: Summary of velocity-porosity relationships for sandstones (adapted from Batzle et al. 2007).

3.1.9.2 Electromagnetic Methods

Given the three phases (solids, water, and air) present in earth materials, measurements of relative permittivity (i.e., dielectric constant) using electromagnetic methods represent a composite value affected by each of the phases. This interdependence between permittivity and material phases helps to account for the wide range of factors that can affect the permittivity. For example, dielectric properties of geologic materials have been shown to be sensitive to frequency and temperature (e.g., Chung et al. 1970; Saint-Amant and Strangway 1970), presence of water (Topp et al. 1980; Malicki et al. 1996; Roberts and Lin 1997; Mukhlisin and Saputra 2013), mineralogy (e.g., Hansen et al. 1973), fabric (e.g., Tuck and Stacey 1978; Hawton

and Borradaile 1989), and bulk density (e.g., Olhoeft and Strangway 1975). Therefore, a number of correlations have been developed for earth material properties based on successful acquisition of dielectric permittivity. This section specifically discusses geophysical measurements acquired using electromagnetic methods and their relationship to porosity.

To account for the composite nature of earth materials, dielectric models have been developed to predict permittivity based on assumptions regarding the interdependence and properties of its constituents (e.g., see Alharthi and Lange 1987; Knoll 1996; Sihvola 1999; Martinez and Byrnes 2001). One of the most commonly referenced models is a volumetric mixing model known as the Complex Refractive Index Method (CRIM), which can be used to estimate porosity of the material:

$$\varepsilon^{\alpha} = S_w \eta(\varepsilon_w)^{\alpha} + (1 - S_w) \eta(\varepsilon_a)^{\alpha} + (1 - \eta)(\varepsilon_s)^{\alpha}$$
(3-66)

In Eq. 3-66, ε represents the relative composite permittivity of the soil-water-air mixture, ε_w represents the water (i.e., equal to 81 at 100 MHz), ε_a represents air (i.e., equal to 1), ε_s represents the solids, S_w is the degree of saturation of the mixture, η is the porosity, and α is an experimental fitting parameter that accounts for the orientation of the electrical field relative to the soil layering (i.e., often assumed to be equal to 0.5 but varies between -1 for perpendicular and +1 for parallel orientation) (Birchak et al. 1974; Roth et al. 1990; Knoll 1996; West et al. 2003). As presented, Eq. 3-66 is actually often referred to the power-law (Sihvola 1999) and the CRIM equation is a special case where a value of 0.5 is input for α . A few complications arise in application of the CRIM equation. First of all, a key input is the permittivity of the solids phase. This can be measured under ideal conditions using laboratory samples. However, access to such testing is not always feasible and appropriate values must be selected from ranges provided in the literature (e.g., Cassidy 2009; Reynolds 2011). Additionally, the water table must be accounted for since the saturation is a necessary input in Eq. 3-66. If the variation of saturation is not well established at all depths, then at a minimum the depth to the water table is necessary. Using that information, the travel time corresponding to unsaturated zones near the surface must be subtracted from the total travel time using the average direct ground wave

velocity to only account for fully saturated conditions. In that manner, S_w can be set equal to 1 and the measured EM velocity can then be used to estimate a value to input for ε in Eq. 3-66.

A number of studies have demonstrated applicability of the CRIM equation to estimate porosity. For example, Lai et al. (2006) performed laboratory measurements using GPR to estimate porosity of pavements and soils using the CRIM equation. Bradford et al. (2009) used GPR measurements to estimate porosity at the Boise Hydrogeophysical Research Site and demonstrate the advantages of 3D tomographic velocity inversions. Recently, Mount et al. (2014) utilized GPR data from the Everglades National Park in south Florida and the CRIM equation to estimate spatial variability in the porosity of the limestone that forms the upper portion of the Biscayne Aquifer.

There is inherent overlap between equations developed for porosity (e.g., CRIM) and those developed for water content (e.g., Malicki et al. 1996). Essentially, these equations can be used interchangeably in situations where the soil is fully saturated since volumetric water content would be equal to the porosity. So the development and application of these relationships was driven by a subtle difference in the motivation of the researchers (i.e., water content versus porosity). For example, two of the applications described in this section for CRIM were focused on aquifers. In these cases porosity measurements represent the volumetric capabilities of the system, a key aspect in understanding the hydrogeological conditions of the aquifer.

3.1.9.3 Resistivity Methods

Electrical resistivity (ER) testing estimates the electrical properties of the subsurface by utilizing electrodes to inject current into the ground surface and to take measurements of the corresponding voltage potentials. From this information, the subsurface spatial distribution of material resistivity (i.e., how strongly the material opposes flow of electric current) is obtained after an inversion process is performed on the collected field data. The results from ER are useful in a number of applications, including the determination of stratigraphy and geologic structures (e.g., Colella et al. 2004; Slater and Reeve 2002), locating karst features (e.g., Ramakrishna 2011), water salinity studies (e.g., Amidu and Dunbar 2008; Toran et al. 2010), and non-destructive testing purposes for pavements and concrete structures (e.g., Forough et al. 2013; Tucker et al. 2015). As previously mentioned, the primary measurement from ER testing is

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the resistivity of the subsurface materials. Multiple factors influence the resistivity of earthen materials, including presence/salinity of water, soil mineralogy, and properties related to soil structure (e.g., porosity) (Abu-Hassanein et al. 1996). Therefore, correlations have been developed for a number of these earth material properties based on successful acquisition of resistivity. This section focuses specifically on the relationship between resistivity and porosity.



Figure 3.65: Strong correlation between electrical resistivity and η for a soil (Oh et al. 2014).



Figure 3.66: Strong correlation between electrical resistivity and volumetric water content for various soil types (Samouelian et al. 2005).

In granular soils, the individual grains tend to behave as electrical insulators and current is conducted primarily through movement of ions within the electrolytic pore water in the void spaces. As a result, the distribution of voids and water present in that pore space each have a large impact on the measured resistivity of a soil (e.g., Figs. 3.65 - 3.66). A number of researchers have therefore explored the link between these parameters and have proposed equations that relate resistivity and porosity of a soil (Table 3.19).

Name	Equation			
Parallel Model [1]	$\sigma = (1-\eta).\sigma_p + \eta.\sigma_w$			
Series Model [13]	$\frac{1}{\sigma} = \frac{(1-\eta)}{\sigma} + \frac{\eta}{\sigma}$			
Lesseldarde Madal [17]				
Logarithmic Model [16]	$\sigma^{\alpha} = (1-\eta).\sigma_{p}^{\alpha} + \eta.\sigma_{w}^{\alpha}$			
General Effective Medium Model [16]	$\frac{(1-\eta)(\sigma_{p}^{-1/t} - \sigma^{1/t})}{\sigma_{p}^{-1/t} + X\sigma^{1/t}} + \frac{\eta(\sigma_{w}^{-1/t} - \sigma^{1/t})}{\sigma_{w}^{-1/t} + X\sigma^{1/t}} = 0$			
Maxwell Model [15]	$\frac{\sigma}{\sigma_{w}} = \frac{2\sigma_{w} + \sigma_{p} - 2(1 - \eta)(\sigma_{w} - \sigma_{p})}{2\sigma_{w} - \sigma_{p} + (1 - \eta)(\sigma_{w} - \sigma_{p})}$			
Archie's Law [17]	$\sigma = c\sigma_w.\eta^B$			
Modified Archie's Law [14]	$\sigma = \sigma_{\rm p} \cdot (1 - \eta)^{\rm k} + \sigma_{\rm w} \cdot \eta^{\rm s}$			
	where $k = log(1-\eta^s)/log(1-\eta)$			

Table 3.19: Summary of published relationships between conductivity (i.e. inverse of resistivity) and porosity (Shah and Singh 2005).

One of the most well-known of these relationships was proposed by Archie (1942). This empirical relationship was developed using borehole resistivity logs and estimated the bulk resistivity for a single conducting phase (i.e., water) distributed within a non-conducting phase (i.e., soil/rock skeleton). A general form of Archie's Law can be expressed as follows:

$$\rho = a\rho_f \eta^{-m} S^{-n} \tag{3-67}$$

where ρ represents the bulk resistivity, ρ_f the resistivity of the pore fluid, η the porosity, *S* the saturation, *m* is an empirical fitting parameter related to cementation and grain shape, *n* is an empirical fitting parameter related to saturation, and *a* is a fitting parameter related to the tortuosity of the pore space. Some formulations (such as Archie's original presentation) omit the parameter *a* altogether as it can often take on values close to 1. Typically, the cementation exponent increases with a decrease in the connectivity of the pore fluid (Kwader 1985; Glover et al. 2000), and the presence of clay causes higher *m* values (Jackson et al. 1978). As noted in Table 3.20, various researchers have proposed values for *m* depending on soil/rock type, including 1.8 for kaolinite, 2.11 for illite, 3.0 for sodium montmorillonite, 1.6 – 1.7 for silty-sandy-clay mixtures, 1.3 – 1.6 for clean sand, 1.6 – 2.0 for sandstone, 1.09 for porous dolomite, 1.2 – 1.3 for fractured limestone, and 1.5 – 2.3 for irregularly shaped particles (Atkins and Smith

1961; Timur et al. 1972; Jackson et al. 1978; Campanella and Weemees 1990; Salem and Chilingarian 1999; Kim et al. 2011). The saturation parameter *n* typically varies from 1 to 2.5 and is usually assigned a value close to 2. The original formulation for Archie's Law was developed for saturated conditions, indicating that the porosity term (η) in Eq. 3-67 can be replaced with volumetric water content (θ). Eq. 3-67 was originally developed based on measurements in sandstone, but has proven applicable as long as the pore fluid resistivity is low and there are relatively small quantities of conducting clay minerals present in the soil (i.e., clean sands and gravels) (Bryson 2005).

I. Medium	Porosity Range	m, Archie's Exponent	Reference				
clean sand	0.12-0.40	1.3	Archie (1942)				
consolidated sandstones	0.12-0.35	1.8-2.0					
glass spheres	0.37-0.40	1.38	Wyllie and Gregory (1955)				
binary sphere mixtures	0.147-0.29	1.31					
cylinders	0.33-0.43	1.47					
disks	0.34-0.45	1.46					
cubes	0.19-0.43	1.47					
prisms	0.36-0.52	1.63					
8 marine sands	0.35-0.50	1.39-1.58	Jackson et al. (1978)				
glass beads (spheres)	0.33-0.37	1.20					
quartz sand	0.32-0.44	1.43					
rounded quartz sand	0.36-0.44	1.40					
shaley sand	0.41-0.48	1.52					
shell fragments	0.62-0.72	1.85					
fused glass beads	0.02-0.38	1.50	Sen et al. (1981)				
fused glass beads	0.10-0.40	1.7	Schwartz and Kimminau (1987)				
sandstone	0.05-0.22	1.9-3.7	Doyen (1988)				
polydisperse glass beads	0.13-0.40	1.28-1.40	de Kuijper et al. (1996)				
fused glass beads	0.10-0.30	1.6-1.8	Pengra and Wong (1999)				
sandstones	0.07-0.22	1.6-2.0					
limestones	0.15-0.29	1.9-2.3					
Syporex [®]	0.80	3.8	Revil and Cathles III				
			(1999)				
Bulgarian altered tuff	0.15-0.39*	2.4-3.3	Revil et al. (2002)				
Mexican altered tuff	0.50*	4.4					
glass beads	0.38-0.40		Friedman and Robinson				
quartz sand 0.40-0.44		1.45	(2002)				
tuff particles	0.60-0.64	1.66					

*connected (inter-granular) porosity

Table 3.20: Summary of published values for Archie's Law cementation factor, *m*. (Lesmes and Friedman 2005).

In cohesive soils, current flow also involves surface conduction in the diffuse double layer of ions surrounding the clay (Santamarina et al. 2001). Given this knowledge, there is some disagreement in the literature regarding the most appropriate relationship from which to

determine porosity of clayey soils. A number of researchers have proposed use of Archie's Law in its original form or with minimal modifications (Atkins and Smith 1961; Jackson et al. 1978; Campanella and Weemees 1990; Salem and Chilingarian 1999; Shah and Singh 2005). Shah and Singh (2005) specifically note that the effects of surface conduction are inherently built into the cementation fitting parameter *m* in Eq. 3-67. However, others have suggested that Archie's formulation oversimplifies resistivity in clays and have proposed electrical mixing models that specifically contain a term to account for surface conductivity (e.g., Sen et al. 1988; Johnson et al. 1986; Waxman and Smits 1968; Sen and Goode 1992). The Waxman and Smits (1968) relationship, for example, accounts for the additional surface conduction based on modeling the pore-fluid and pore-skeleton system as two electrical resistors in parallel:

$$\rho = \frac{a\rho_f \eta^{-m} S^{-n}}{S + \rho_f BQ} \tag{3-68}$$

The new term *B* accounts for the conductance of opposite charge ions to the surface charge of the diffuse double layer and can be obtained empirically (e.g., Waxman and Thomas 1974). *Q* is the cation exchange capacity per unit pore volume of the clay. The *BQ* terms together describe the surface conductivity along the double layer and the units are Siemens per meter. Efforts have been made to relate the surface conductivity to Atterberg limits of clays to allow an estimate of the *BQ* terms [e.g., Abu-Hassanein et al. (1996) and Bryson (2005)].

Given the wide range of resistivity values and the many parameters that affect these values, it is advisable to calibrate the relationships provided in Eqs. 3-67 and 3-68 for site conditions. This is especially the case if the goal is direct evaluation of porosity. Another approach is to utilize these relationships to track changes in porosity (and/or water content if the soil is assumed saturated) across a site over a specified period of time (e.g., Chambers et al. 2014). In these cases, it is likely that the parameters in Eqs. 3-67 and 3-68 will remain constant over the area of interest throughout testing and any changes in resistivity will be directly a result of changes in porosity.

3.1.9.4 Gravity Methods

The primary application of gravity measurements is to evaluate the density of the underlying earth materials. However, by making inferences regarding the distribution of fluids in the pore space as well as the pore fluid and grain densities, gravity measurements can be used to evaluate porosity for the soil or rock based on the application of basic soil mechanics phase relationships:

$$\rho = G_s \rho_f (1 - \eta) + \eta S \rho_f \tag{3-69}$$

where ρ is the density estimated from gravity measurements, G_s is the specific gravity of the minerals, ρ_f is the density of the pore fluid, η is the porosity of the strata, and S is the saturation within the pore space of the strata. Assuming the pore space is completely saturated with fluid, a simplified expression can be developed:

$$\eta = 100 \frac{\rho_s - \rho}{\rho_s - \rho_f} \tag{3-70}$$

where ρ_s is the density of the minerals and the other factors are as described for Eq. 3-69. Given the robustness of gravity measurements and the wide scale of appreciable gravity measurements, this approach has seen applications ranging from estimating formation porosities for petroleum engineering purposes [e.g., Beyer and Clutsom (1978), Fig. 3.67], to estimating the porosity of crustal rocks for oceanic investigations (e.g., Johnson et al. 2000), and investigating the diagenetic processes associated with changes in carbonate rock porosity (Halley and Schmoker 1983).





3.1.9.5 Nuclear Methods

Of the different nuclear methods previously discussed in this document, neutron logging is the method most closely associated with determination of porosity (and water content). Neutron logging has a long history in the petroleum industry as a wireline tool to characterize rock formations in the search for hydrocarbons (e.g., Dewan and Allaud 1953; Stick et al. 1962). Much of the proceeding discussion on the theoretical basis of neutron logging as related to measurements of porosity has been adapted from numerous sources with similar discussions (Goldberg et al. 1955; Tittle 1961; Tittman et al. 1966). The reader is encouraged to review these sources of information for additional details that are beyond the scope of this document, including the effects of various correction factors.

Neutron logs contain a source of high energy (i.e., fast) neutrons in a probe and two detectors that record the interactions that occur at two distances away from the source. The emitted high energy neutrons (typically americium-beryllium) begin to slow down as they collide with the nuclei of elements composing the propagating medium. Once these fast neutrons have undergone enough collisions, their kinetic energy approaches the average kinetic energy of the atoms in the propagating medium based on the ambient temperature. At this point, the fast neutron is in equilibrium with the surrounding material atoms and is considered a slow (or thermal) neutron. The straight-line distance necessary for a fast neutron to reach this equilibrium state is a characteristic of the propagating medium and is referred to as the slowingdown length. Fast neutrons that have not reached the slowing-down length are also referred to as epithermal neutrons since their temperature is still greater than the average thermal conditions of the other atoms in the medium. Hydrogen most effectively slows down the neutrons since its mass is very similar to the neutrons. Therefore, neutron logging is most sensitive to the amount of hydrogen in the material. Since the hydrogen in most earth material is primarily contained in water in the pore space or bonded to clay minerals, neutron logging can be readily used to estimate porosity (and, by extension, water content). However, other elements do interact with neutrons to a lesser degree (e.g., boron, lithium, cadmium, chlorine and iron), so mineral composition ideally should be estimated when interpreting test results. Additionally, earth materials with high organic content may cause issues with the measurements because the hydrogen content of the organic matter will contribute to the total count rate.

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The manners in which measurements from neutron logging are used to estimate porosity depend significantly on the type of tool employed in the study. Most modern equipment such as the compensated neutron log (CNL) contains two neutron detectors that respond to thermal neutrons. The hydrogen content (and porosity, by extension) is estimated based on a correlation to the ratio of the count rate recorded by the near and far detectors in the CNL tool (Fig. 3.68).



Figure 3.68: Calibration between the ratio of neutron count for a CNL tool and the porosity of the formation (Alger et al. 1972).

Wireline CNL tools for logging well bores in the petroleum industry are typically calibrated at the API test pit in Houston, TX or in other commercially available test pits. Additionally, calibration can be performed based on reconciliation with cored samples from the same well. Most modern neutron logging tools allow the calibration to be automatically applied so that the output in from field efforts is already in estimates of porosity (Fig. 3.69). Surface-based neutron measuring tools also exist in practice, often for the purpose of estimating water content. For example, the surface nuclear gauge has a long history in geotechnical engineering to measure moisture content for compaction quality control based on neutron logging (Mintzer 1961). Calibration for these tools is discussed in ASTM D7759/D7759M – 14 and is typically performed under controlled settings in the laboratory with a set of reference soils whose water contents are well-determined by other methods (e.g., oven drying). Additional information is provided in the discussions on mass density and water content estimates from nuclear methods.



Figure 3.69: Example output of porosity (in limestone equivalent units) based on calibration of a neutron logging tool (Berendsen et al. 1988).

3.1.10 WATER CONTENT

The amount of water present in earth materials affects several aspects of behavior, particularly in clayey soils whose mineralogy makes them particularly susceptible to hydration, flocculation, and/or dispersion. Therefore, the measurement of water content is fundamental to a wide-range of engineering applications, including compaction. Geophysical methods may be particularly useful as they can provide estimates of water content in an efficient nondestructive manner, which can allow for rapid measurements of the temporal and/or spatial variations in water content across a site. The following sections describe the relationships that exist between geophysical measurements and water content.

3.1.10.1 Electromagnetic Methods

In general, permittivity is a measure of polarizability of a medium when subjected to a timevarying electric field. Water molecules in soils possess dipoles that impart this polarizability property to soil (Curtis 2001). As a result, there is a strong correlation between the amount of water in a soil and its dielectric properties (Fig. 3.70).



Figure 3.70: Strong correlation between soil permittivity and volumetric water content: (a) Curtis (2001); and (b) Mukhlisin and Saputra (2013).

A previous discussion was provided in the mass density section of this document regarding the use of TDR to estimate water content (and mass density). The methodology and calibration procedures presented in that section can certainly be applied in a more general context, but is typically relegated to its original intended purpose of compaction quality control since the TDR probe is limited in depth of penetration. For more general applications, a number of

relationships have been proposed to correlate the measured permittivity of a soil to its volumetric water content (θ). The volumetric water content (θ) is related to the gravimetric water content (*w*) as typically used for geotechnical purposes by Eq. 3-71:

$$\theta = w \frac{\rho_d}{\rho_w} = \eta S \tag{3-71}$$

where ρ_d is the dry density (i.e., bulk density, as computed from dry mass of solids divided by total volume), ρ_w is the density of water, η is the porosity, and *S* is the saturation. Note that it is possible to evaluate saturation levels based on determination of volumetric water content, assuming information regarding porosity (or void ratio) is known. Huisman et al. (2003) provides an excellent review that provides significant background on the use of electromagnetic methods for determining water content. One of the relationships most commonly used for geotechnical purposes was proposed by Topp et al. (1980):

$$\theta = (-530 + 292\varepsilon - 5.5\varepsilon^2 + 0.043\varepsilon^3) \times 10^{-4}$$
(3-72)

$$\theta = (-252 + 415\varepsilon - 14.4\varepsilon^2 + 0.22\varepsilon^3) \times 10^{-4}$$
(3-73)

where θ is the volumetric water content and ε is the apparent permittivity (i.e., dielectric constant) of the soil as measured using electromagnetic propagation velocity estimates. Eq. 3-72 is intended for sands and Eq. 3-73 is formulated for organic soils. These empirical equations fit the variability of water content using a direct relationship between θ - ε . Other relationships exist that include other parameters such as porosity and/or density in the statistical regression. Such relationships obviously necessitate additional information relative to the Topp et al. (1980) equations above, but can offer superior performance in estimating water content. A useful example of such a relationship was proposed by Malicki et al. (1996), which can be expressed as both a function of bulk density or porosity:

$$\theta = \frac{\sqrt{\varepsilon} - 0.819 - 0.168\rho_b - 0.159\rho_b^2}{7.17 + 1.18\rho_b}$$
(3-74)

$$\theta = \frac{\sqrt{\varepsilon} - 3.47 + 6.22\eta - 3.82\eta^2}{7.01 + 6.89\eta - 7.83\eta^2}$$
(3-75)

Eqs. 3-74 and 3-75 are preferred for clayey soils over the relationships proposed by Topp et al. (1980). Mukhlisin and Saputra (2013) and Robinson et al. (2003) provide a thorough summary of the proposed relationships between permittivity and water content (for both single parameter and multiple parameter equations) (Table 3.21).

Equations Source		E	C = 11 +	Properties of soil				
Equat	ions Source	Experimental method	Soil type	Porosity (cm ³ cm ⁻³)	Bulk density (g cm ⁻³)	Particle density (g cm ⁻³)		
(1a) (1b)	Topp et al.	ε: using TDR Tektronik Model 7S12 to perform 18 experiments with different	Model with one parameter (i) Mineral soil (ii) Organic soil (iii) Vermiculite (iv) Glass beads	_	(i) 1.04–1.44 (ii) 0.422 (iii) 1.08 (iv) 1.49–1.61	_		
(Ic)	()	treatments	Organic soil	_	0.422	_		
(1d)		e: using gravimetric	450 μ m glass beads	_	1.60-1.61	_		
(2a)	Roth et al.	ε: TDR miniprobe 250 ps	9 Mineral soils	0.418-0.482	1.26-1.55	2.28-2.67		
(2b)	[21]	rise time needle pulse	7 Organic soils	0.527-0.785	0.2-0.77	0.70-1.63		
(3)	Ferré et al. [25]	Using model of inverse averaging for TDR method by analysing the mixing model	-	-	_	-		
(4)	Schaap et al. [29]	ε : TDR Tektronix 1502B θ : gravimetric technique	25 samples of forest floors	_	0.086-0.263	1.3		
(5)	Curtis [18]	Coaxial Transmission/reflection apparatus controlled by a Hewlett-Packard 8510C Vector Network Analyzer system 45 MHz to 26.5 GHz	-	_	_	_		
(6)	Wu et al. [20]	ε: based on capacitance measurement θ: gravimetric technique	Quartz sand	_	_	_		
			Model with two parameters					
(7a) (7b)	Wang and Schmugge [30]	Modelling using data from other studies [34–36]	22 different samples	0.4-0.6	1.1–1.7	2.6-2.75		
(8a) (8b)	Roth et al. [21]	TDR	From 11 different field sites					
(9)	Malicki et al. [23]	TDR CAMI	62 kinds of soil samples	0.33-0.95	0.13-1.66	1.06-2.7		
(10)	Gardner et al. [19]	Capacitance probe 80–150 MHz	(i) Brown earths (ii) Silica materials	_	(i) 1.08–1.49 (ii) 1.24–1.63	_		
(11)	Robinson et al. [14]	TDR Tektronix 1502B	Coarse grained, quartz grain, sandy soil	_	_	_		

Table 3.21: Summary of published relationships between permittivity and water content as illustrated in Figs. 3.71 – 3.72 (Mukhlisin and Saputra 2013).

Figs. 3.71 – 3.72 illustrate the analysis Mukhlisin and Saputra (2013) performed to evaluate the various θ - ε relationships using a common data set. Mukhlisin and Saputra (2013) found that no

single relationship could model the θ - ε behavior across all data ranges. However, at relatively small values of water content, many of the relationships agreed favorably with the data set and provided similar estimates. Of the proposed relationships that take porosity into account, the Malicki et al. (1996) formulation proved superior at capturing water content dependence on permittivity and porosity across multiple ranges. The majority of these relationships between water content and permittivity were calibrated using Time Domain Reflectometry (TDR), which operates at a high frequency range (e.g., 500 MHz – 1000 MHz). Additional discussion regarding water content measurements with TDR was discussed previously with regards to estimates of mass density. As mentioned in that discussion, high clay content soils can cause dispersion in electromagnetic waves (e.g., West et al. 2003). This dispersion leads to frequency variations in permittivity, particularly at the low range of electromagnetic wave frequency (e.g., 100 Hz and lower). Therefore, care must be exercised in applying the θ - ε relationships developed using TDR when the permittivity is obtained using lower frequency GPR antennas for soils with significant clay content. Site-specific calibrations of the proposed θ - ε relationships should be considered to improve accuracy of water content predictions in those situations.



Figure 3.71: Comparison of single parameter relationships between soil permittivity and volumetric water content (Mukhlisin and Saputra 2013). Note: References for each equation in the plot is provided in Table 3.21.



Figure 3.72: Comparison of multi-parameter relationships between soil permittivity and volumetric water content (Mukhlisin and Saputra 2013). Note: References for each equation in the plot is provided in Table 3.21.

3.1.10.2 Resistivity Methods

As previously noted, the resistivity of a soil is drastically affected by the amount of water present in the pore space because water acts as an electrical conductor. Implicit in a number of the relationships proposed between resistivity and porosity is that water tends to be present in that pore space. In fact, the original formulation for Archie's Law (Eq. 3-67) was developed assuming saturated conditions, which means that the porosity term could have also been substituted by the volumetric water content. Given the inherent link between porosity and

water content as well as their combined effects on resistivity, many of the studies in this section are similar to those presented where resistivity was used to estimate porosity. Often, the differences between studies of water content and resistivity result from a subtle difference in application of the results. This is not unlike the case of radar methods where a similar link exists between porosity and water content for permittivity.

A number of relationships have been proposed by various researches over the years to quantify the effects of water content on resistivity. Gupta and Hanks (1972) developed a simple linear empirical relationship between water content and resistivity based on laboratory measurements:

$$\rho = a + b\theta \tag{3-76}$$

where a and b are empirical constants that can be established by fitting temporal data of water content and resistivity. Goyal et al. (1996) suggested values of 50 and -0.1 for a and b, respectively.



Figure 3.73: Example of field calibration curve to estimate volumetric water content from measured resisitivity (adapted from Michot et al. 2003).

A number of researchers have utilized a linear form as in Eq. 3-76 or developed their own field or lab calibrated empirical relationships (e.g., Fig. 3.73) to estimate water content from resistivity and examine temporal fluctuations in water content (e.g., McCarter 1984; Kalinski and

Kelly 1993; Goyal et al. 1996; Fukue et al. 1999; Hymer et al. 2000; Zhou et al. 2001; Binley et al. 2002; Walker and Houser 2002; Michot et al. 2003; Cosenza et al. 2006; Al Hagrey et al. 2004; Shah and Singh 2005; Zhu et al. 2007; Wenninger et al. 2008; Brunet et al. 2009; Kibria and Hossain 2012; Oh et al. 2014). Table 3.22 summarizes the results from many of these studies and catalogs the recommended empirical constants.

Authors	Location	Soil texture	Land use	Size	Depth (m)	Procedure	$\theta \text{-} \rho$ relation	Ν	Parameter values ^a	r
McCarter (1984) Kalinski and Kelly (1993)		Clay Clay				Lab Lab	Non-linear law Second order polynomial			
Goyal et al. (1996)					4		Linear		a = 50 b = -0.1	-0.98
Fukue et al. (1999) Hymer et al. (2000)	US	Clay Sandy loam	Brush		0.3	Lab Field	Non-linear law Power law		b = -0.08 to -0.65	-0.57 to -0.94^{b}
Zhou et al. (2001) Binley et al. (2002)	Japan UK	Loam Fine and medium sand	Grassland	1200 and 1800 m ²	1.5 15	Lab Field	Power law Power law	46 23	b = -0.88	-0.68 to -0.93 -0.97°
Walker and Houser (2002)	US	Sandy loam	Bare soil	250 m	0.7	Field	Power law			~ -0.59
Michot et al. (2003)	France	Loamy clay	Corn crop	6.2 m	0-0.6	Field	Linear	30– 250	a = 28.5 to 37.7 b = -0.05 to -0.36	-0.46 to -0.97
Cosenza et al. (2006)	France	From coarse sand (top) to clayey silt		Borehole	2.5	Field	Power law	20	a = 1.07 b = -0.41	-0.82
Al Hagrey et al. (2004)	Italy	Loam	Olive orchard	36 m ²	0.25	Lab and field	Power law	75	a = 1.30	-0.98
Zhu et al. (2007)	China	Sand	Pine forest	8 Plots	1–1.5	Field	Linear and exponential	20	b = -0.46 a = 0.1496	-0.80 (lin)
									b = -0.0001 (lin) a = 0.1741 b = -0.0015 (exp)	-0.94 (exp)
Wenninger et al. (2008)	South Africa		Forested	200 m	0.5	Field	Power law	52	b = -0.67	-0.87
Brunet et al. (2009)	France	Sand		400 m ²		Lab and field	Power law	20	<i>b</i> = -0.60	-0.99 ^d

^a Relation: $\theta = a + b\rho$ (linear); $\theta = a\rho b$ (power law); $\theta = a \exp(b\rho)$ (exponential). ^b The soil moisture–resistance relation was investigated.

^c Computed from the data extracted by the manuscript.

d For laboratory data.

Table 3.22: Summary of published relationships between resistivity and water content (Calamita et al. 2012).

Based on these results, various alternative functions in addition to the linear function in Eq. 3-76 have been proposed for the form of the resistivity-water content relationship, including multiorder polynomials, exponential, and power law relationships. As an example, Cosenza et al. (2006) found that a power law relationship best fit their data when water content measurements from field samples were compared with inverted resistivity results:

$$\rho = 1.187 w^{-2.444} \tag{3-77}$$

where *w* represents the gravimetric (mass) water content as typically utilized in geotechnical practice. Rhoades et al. (1976) added an additional term to the linearized relationship presented in Eq. 3-76 to account for the electrical current conducted by adsorbed water on the surfaces of clay particles:

$$\frac{1}{\rho} = \frac{1}{\rho_w} (a\theta^2 + b\theta) + \frac{1}{\rho_s}$$
(3-78)

where ρ_s represents the resistivity of the solid matrix, ρ_w represents the resistivity of the porewater, and *a* and *b* are coefficients that depend on texture and mineralogy of the solid phase of the soil. Using Eq. 3-78, Kalinski and Kelley (1993) were able to accurately predict volumetric water content ranging from 0.2 to 0.5 for a soil with a clay fraction of 20%. Finally, Archie's Law has been recently reexamined as applied to the determination of volumetric water content (e.g., Shah and Singh 2005; Grellier et al. 2007). In Shah and Singh (2005) a relationship was proposed specifically for fine-grained soils that did not directly incorporate a soil matrix conductivity term as in Eq. 3-68 because the effects of surface conductivity were included into the Archie's Law cementation factor:

$$\sigma_b = c\sigma_w \theta^m \tag{3-79}$$

where σ_b is the bulk conductivity of the soil, σ_w is the pore-water conductivity, and c and m represent the fitting factor and cementation factor, respectively. Shah and Singh (2005) observed values of c and m equal to 1.45 and 1.25, respectively, for the soil in their study with clay fraction less than 5%. For soils with larger amounts of clay, c and m could be estimated as function of the clay fraction.

$$c = 0.6CL^{0.55} \tag{3-80}$$

$$m = 0.92CL^{0.2} \tag{3-81}$$

where CL is the percentage of clay fraction in the soil. Shah and Singh (2005) found that the generalized form of Archie's Law fit their data better than the relationship proposed by Rhoades et al. (1976).

Based on the previous discussion, it is clear that resistivity is a useful parameter to evaluate the presence of moisture in the pore space of soils. This area continues to receive significant attention with recently published studies continuing to add to the database (e.g., Kibria and Hossain 2012; Oh et al. 2014). However, care should be exercised in relying on a general relationship to estimate water content, particularly when the relationship is an empirical formulation. Site specific calibration of such relationships is recommended for practical use as many factors related to soil conditions can affect ρ - θ behavior in addition to water content.

3.1.10.3 Nuclear Methods

Much of the previous discussion in this document regarding the neutron logging method and estimation of porosity is directly applicable to estimates of water content. The neutron logging method responds to hydrogen contained in the pore-fluid of the material. Estimates of porosity in the recorded logs are actually derived from assumptions regarding the mineralogy of the tested material in relationship to the hydrogen content/ratio obtained during neutron testing. Calcite is commonly chosen as a default mineral, such that limestone is the reference rock in use for typical neutron porosity logs. In material with different mineralogy, the limestone calibrated neutron log is no longer accurate and it must be rescaled based on an appropriate mass density of the minerals making up the tested material. For surface testing, the surface nuclear gauge test uses the neutron method to estimate the water content and the gamma-gamma method to estimate mass density. An appropriate calibration performed on reference materials of known water content (e.g., ASTM D7759/D7759M - 14) ensures that the surface nuclear gauge provides a usable estimate for water content. When used in this manner, previous studies have demonstrated good agreement between the water contents estimated from laboratory or field drying methods and those estimated from the nuclear gauge using the neutron approach (e.g., Fig. 3.74).



Figure 3.74: Example comparison of *w* obtained using a surface nuclear gauge with respect to laboratory measured values using an oven (Berney et al. 2011).

3.1.11 PERMEABILITY

Hydraulic conductivity (i.e., permeability) is an important parameter that reflects the ability of a soil to transmit water. It is a key input into a number of geotechnical and transportation applications, including estimates of infiltration, evaluation of soil drainage conditions, and site dewatering, among others. A number of geophysical measurements are sensitive to the same factors that affect how water flows through the pore spaces in earth materials. The following sections describe relationships that have been developed that exploit this link and allow estimation of permeability using various geophysical measurements.

3.1.11.1 Resistivity Methods

Many of the factors that affect hydraulic conductivity (e.g., saturation/water content, pore continuity, shape, and tortuosity, etc.) also affect resistivity of the soil. Therefore, a logical extension is that a relationship exists between permeability and resistivity, whereby measurements of resistivity can be used to estimate this parameter. Given the substantial range in permeability and the relative uncertainty and issues regarding in situ and laboratory measurements, ERI and other methods that can measure resistivity would present a useful approach to quickly estimate permeability across a site if a unique relationship can be established to resistivity. A number of researchers have investigated such an approach, particularly as related to the quality control of compacted clays and the evaluation of aquifer

hydrogeological properties (Worthington 1977; Kelly 1977; Heigold et al. 1979; Mazac et al. 1985; Huntley 1986; Mazac et al. 1990; Sadek 1993; Abu-Hassanein et al. 1996; Rinaldi and Cuestas 2002; Bryson 2005; Miller et al. 2010). In terms of general trends, Mazac et al. (1990) concluded that an inverse relationship exists between permeability and resistivity for clean sandy soils. For example, a dense saturated clean sand will exhibit a lower value for permeability and larger resistivity than the same clean sand in a loose configuration. However, for clayey soils a direct relationship is expected between permeability and resistivity due to the changes in surface conductance with grain size (Abu-Hassanein et al. 1996). An increase in clay content decreases permeability and increases surface conductivity effects, which in turn decreases resistivity.

Unfortunately, given the discrepancies described so far, it has been difficult to develop a general direct relationship between resistivity and hydraulic conductivity that is applicable across multiple soil types and conditions. Sadek (1993) argues that such a relationship is inherently non-unique because the same electrical resistivity can be measured for soil specimens with completely different structures (and therefore hydraulic conductivities). However, a number of sources over the years have highlighted the ability to develop site-specific empirical relationships. For example, Kelly (1977), Abu-Hassanein et al. (1996), and Niwas and Celik (2012) each found that a unique relationship could be established in their studies (only for a small subset of the soils in Abu-Hassanein et al. 1996) (Figs. 3.75 - 3.77). The functional form of similar site-specific empirical relationships has often been expressed as a power-law expression:

$$k = a\rho^c \tag{3-82}$$

where k is the permeability and the a and c parameters are regression coefficients based on specific site and soil conditions (Cassiani and Medina 1997). Figure 3.78 presents an example of such a correlation based on data presented in Heigold et al. (1979). The exponent c is typically less than 1 for sandy soils and larger than 1 for clayey soils.



Figure 3.75: Relationship between resistivity and hydraulic conductivity (*k*) at sites tested in Kelly (1977).



Figure 3.76: Relationship between resistivity and hydraulic conductivity (*k*) for four soils tested in Abu-Hassanein et al. (1996). Note: The results for soils A and B are non-unique.



Figure 3.77: Relationship between resistivity and hydraulic conductivity (*k*) for aquifer tested in Niwas and Celik (2012).



Figure 3.78: Relationship between resistivity and hydraulic conductivity (*k*) for aquifer tested in Heigold et al. (1979).

Other expressions have been used in addition to a power law functional form. For example, Miller et al. (2010) established an exponential relationship between saturated hydraulic conductivity and resistivity based on field measurements at two flood plain sites using a direct-push permeameter and the United States Bureau of Reclamation gravity permeability method (USBR 1985):

$$k = 0.114e^{0.024\rho} \tag{3-83}$$

where k is the saturated permeability in m/day and ρ is the resistivity expressed in Ω -m (Fig. 3.79). Others have linearized the expression by presenting the permeability-resistivity

correlation and performing regressions on a log-log plot (Frohlich et al. 1996; Niwas and Celik 2012) (Figs. 3.80 and 3.81). No matter the functional form, in each of the preceding cases the empirical constants for the regression were not universal as they were developed for site-specific conditions.



Figure 3.79: Relationship between resistivity and saturated hydraulic conductivity (k) established from direct-push permeameter and the USBR gravity method at two flood plain sites in Miller et al. (2010).



Figure 3.80: Relationship between resistivity and hydraulic conductivity (*k*) for aquifer tested in Frohlich et al. (1996).



Figure 3.81: Relationship between hydraulic conductivity (*k*) and formation factor (Urish 1981).

Other researchers have noted that the relationship between permeability and resistivity is indirect, meaning that there is a stronger correlation between resistivity and another property that ultimately also impacts permeability. For example, Rinaldi and Cuestas (2002) explored the relationship in the laboratory between various geotechnical parameters and conductivity (i.e., inverse of resistivity) for a silty clay soil sampled in Cordoba, Argentina. Rinaldi and Cuestas (2002) noted a stronger correlation between hydraulic conductivity and porosity. The best fit function for the porosity-permeability relationship was then incorporated into Archie's Law to obtain a relationship between resistivity and permeability:

$$k = b \left(\frac{a}{F}\right)^{\frac{g}{m}} \tag{3-84}$$

where *a* and *m* represent the tortuosity and cementation factors, respectively, *b* and *g* are empirical fitting constants, and *F* is the formation factor (ratio of bulk formation/soil resistivity to the electrolyte/fluid resistivity). Based on the laboratory test results, Rinaldi and Cuestas (2002) obtained the following values for the empirical constants: a = 0.66, m = 2.49, $b = 2.0 \times 10^6$, and g = 28.22, where k was expressed in cm/s. Notable is the fact that Eq. 3-84 was derived indirectly after incorporating a more distinct porosity-permeability relationship present in the

Rinaldi and Cuestas (2002) experimental data. Additionally, Eq. 3-84 highlights the fact that permeability is a function of the formation factor and not solely the resistivity of the bulk soil sample. As highlighted in Figs. 3.81 - 3.83, several studies over the years have supported this finding (e.g., Shockley and Garber 1953; Croft 1971; Plotnikov 1972; Worthington 1975; Heigold et al. 1979; Mazac and Landa 1979; Kosinski and Kelly 1981; Urish 1981; Allessandrello and Lemoine 1983; Kwader 1985). Therefore, it is possible for a soil compacted to the same unit weight to have different resistivity values but similar *k* and *F* values (i.e., the relationship is non-unique based solely on soil resistivity). Rinaldi and Cuestas (2002) found that a direct relationship is possible only for samples saturated with an electrolytic at medium to large concentration.



Figure 3.82: Various relationships between hydraulic conductivity (*k*) and formation factor (Mazac et al. 1985).



Figure 3.83: Relationship between hydraulic conductivity (*k*) and formation factor (Kwader 1985).

In view of the preceding discussions, broad use of any of the highlighted relationships between permeability and resistivity is not recommended until additional research provides greater insight and a robust generalized expression that reduces uncertainty in predicting permeability. There is evidence to support that such an expression may not be feasible given the inherent non-uniqueness of the relationship (e.g., Sadek 1993). Therefore, in situations where ERI will be implemented as a rapid, non-invasive evaluation method to estimate permeability, laboratory/field calibration for a given soil is necessary to establish a working functional form and ensure that the empirical relationship is unique based on specific site conditions. Such sitespecific correlations can provide useful estimates within the typical ranges of uncertainty associated with permeability. Moreover, once such a correlation is established, the lower costs associated with surface electrical resistivity methods in relation to additional boreholes can be exploited to allow larger spatial coverage when estimating permeability across a site.

3.1.11.2 Gravity Methods

The gravity method has been increasingly used to monitor aquifers, oil and gas reservoirs, geothermal reservoirs, and carbon sequestration activities over a wide range of sizes and

conditions. In these applications, the focus is typically on using repeated time-lapse gravity measurements to examine reservoir dynamics, including the spatial distribution of migrating fluid, based on density changes (e.g., Allis and Hunt 1986; Pool and Eychaner 1995; Pool and Schmidt 1997; Hare et al. 1999, 2008; Ferguson et al. 2007, 2008; Alnes et al. 2008; Davis et al. 2008; Gasperikova and Hoversten 2008; Vevatne et al. 2012; Dodds et al. 2014). The growing utilization of "four-dimensional" (4D) gravity monitoring (i.e., time-lapse measurements) has sparked increased coverage of the technique with a workshop taking place at the 77th SEG Annual International Meeting in 2007 and a special section of within a recent volume of *Geophysics* (Biegert et al. 2008). The permeability of the underlying earth materials plays an important role in these applications since the ability of porous earth materials to allow fluid flow is controlled by this property. In the context of reservoir dynamics, the term permeability is not used interchangeably with hydraulic conductivity as is routinely the case for geotechnical purposes. Instead, permeability is related to hydraulic conductivity based on the following expression:

$$\kappa = K \frac{\mu}{\rho g} \tag{3-85}$$

where κ is the permeability (in units of length squared), K is the hydraulic conductivity familiar from geotechnical applications (i.e., units of velocity), μ is the dynamic viscosity of the fluid, ρ is the mass density of the fluid, and g is the acceleration due to gravity. Permeability is a complex parameter that can depend on factors outside of those that tend to affect gravity measurements as well. As a result, estimating permeability using gravity measurements is a complicated process that often requires assumptions to be made or complementary information regarding the measured zone (e.g., reservoir, aquifer, etc.). For example, Damiata and Lee (2006) used numerical modeling to simulate drawdown of a shallow unconfined aquifer and examine the gravitational attraction of a drawdown cone (Fig. 3.84). Damiata and Lee (2006) found that high-resolution gravity surveying can augment hydraulic testing by spatially monitoring the development of the drawdown cone in lieu of an extensive system of monitoring wells or piezometers. Therefore, gravity measurements have enough resolution to allow the improvement of permeability estimates in conjunction with drawdown tests.


Figure 3.84: Numerically simulated drawdown and gravitational response due to pumping groundwater from an unconfined aquifer (Damiata and Lee 2006).

Blainey et al. (2007) demonstrated synthetically that, although time-lapse gravity measurements have sufficient signal-to-noise ratio to detect changes in hydrologic conditions, they are incapable of adequately constraining estimates of permeability on their own. Chapman et al. (2008) used repeated high precision gravity measurements to monitor infiltration events for aquifer recharge at a site in Utah. As part of the study, reductions in gravity measurements were simulated by analytical solutions for the decay of a groundwater mound through a saturated porous media. The results from these simulations allowed a relatively accurate prediction of hydraulic conductivity for the alluvial materials that formed the aquifer. Glegola et al. (2012a, 2012b) used a stochastic approach to simulate reservoir behavior and study the feasibility of integrating 4D gravity data to estimate reservoir parameters such as permeability (among others). Again, it was found that gravity measurements perform better in estimating permeability when other information is incorporated regarding the reservoir (e.g., pressure data). Finally, Capriotti and Li (2015) developed a method to directly invert time-lapse gravity data to estimate permeability in conjunction with injection-production data. In their approach, the time-lapse gravity data serves as the input into the inverse problem, the reservoir spatial

distribution of permeability serves as the desired output, and the injection-production data provides boundary conditions for fluid-flow modeling in combination with assumptions regarding the saturation of the reservoir materials. The inversion constructs the permeability distribution so that the gravity and production data are satisfied simultaneously (Fig. 3.85). Despite the efforts in the aforementioned studies, much work remains to reliably utilize gravity measurements to estimate permeability of earth materials. However, given the resolution and potential spatial coverage of gravity measurements, gravity techniques show tremendous promise as a robust and cost effective field technique to characterize the in situ hydrological properties of subsurface earth materials, particularly when compared with more elaborate field methodologies such as drawdown tests.



Figure 3.85: Permeability recovered using inversion of simulated time-lapse gravity measurements at 121 stations and a rate-controlled well with pressure injection data (Capriotti and Li 2015).

3.1.12 CLAY CONTENT

The amount of clay present in a soil can have a large impact in a number of engineering applications. For example, significant clay content in a sand reduces its susceptibility to liquefaction during dynamic loading. Therefore, clay content is an important parameter for use in transportation applications.

3.1.12.1 Electromagnetic Methods

In the case of electromagnetic methods, the clay fraction of a soil can significantly alter the propagation of electromagnetic signals. Electrical conductivity increases as the cation-exchange

capacity (CEC) of a clay increases (Saarenketo 1998). As electrical conductivity increases, more of the energy in the electromagnetic field of a radar signal is consumed during propagation through the medium. Therefore, electromagnetic signals attenuate more rapidly and travel slower in soils with significant clay content, particularly those dominated by high CEC minerals (e.g., montmorillonite). The large effect of clay on electromagnetic signals signifies that electromagnetic methods (GPR in particular) have significant potential as a rapid, non-invasive technique to estimate clay content over a broader scale across a site than point measurements from traditional subsurface investigation techniques. For example, evaluation of sub-asphalt compacted soil layers may be a suitable application of such investigations based on the broader range of coverage GPR offers and the limitations that would exist on the depths of investigation due to significant attenuation of electromagnetic signals in clays.

Despite the widespread awareness that clays affect GPR signals and the potential advantages of using GPR for estimating clay content in roadway applications, only a limited number of studies have attempted to systematically quantify clay content using GPR. Tosti et al. (2013) examined multiple approaches to estimate clay content based on an evaluation of acquired GPR signals in clayey soils. In that study, three soils were mixed with various percentages of bentonite clay (varied from 2 to 25% by weight) and compacted into test boxes. Radar signals were propagated into the soils using two different GPR instruments at 500 MHz and 1–3 GHz frequency range. Tosti et al. (2013) utilized shifts in the peak of the radar signal frequency spectrum and estimates of permittivity using full-waveform inversion and time-domain signal picking techniques to estimate the clay content of the soils. Figure 3.86 demonstrates the results based on analysis of the shifts in the peak of the signal frequency spectrum. Similar results were also noted in Benedetto and Tosti (2013). De Benedetto et al. (2012) statistically estimated clay content across a site based on a hybrid kriging interpolation technique applied to electromagnetic induction (EMI) data and GPR instantaneous amplitude (i.e., envelope) data (Fig. 3.87). In comparison to 36 soil cores taken at the site, De Benedetto et al. (2012) found generally good spatial agreement between measured and predicted clay content. Based on these studies, it is evident that GPR shows promise as a tool to estimate clay content in nearsurface soils. Further development is necessary in order to ensure robustness of the technique for application in practice.



Figure 3.86: Shift in the peak of the frequency spectrum of radar signals based on soil clay content in Tosti et al. (2013). Note: A1, A2, A3 refers to AASHTO classification of tested soil prior to addition of bentonite clay.



Figure 3.87: Map of clay content using GPR and EMI data (adapted from De Benedetto et al. 2012).

3.1.12.2 Resistivity Methods

As previously noted, clay content of a soil is an important parameter for transportation applications as clayey soils tend to present particular challenges in geotechnical design (e.g., settlement, swelling, etc.). Much like the case for radar methods, the presence of clays can significantly impact the electrical properties of soils. Generally, the presence of clay tends to decrease resistivity as the high specific surface area of clays improves surface conductance (Kwader 1985). In fact, various modifications have been proposed that include terms to model the increased surface conductance in clayey soils in the original porosity-resistivity and water content-resistivity relationships referenced in this document.

Given the effects of clay content on resistivity, it is evident that ERI can be used to delineate clayey soils during site investigations. However, a few studies have gone further and have investigated the ability to directly estimate clay content based on measured resistivity values. For example, Abu-Hassanein et al. (1996) explored the ability of ERI to evaluate compacted clay liners for landfill applications. As part of the study, ten clayey soils were tested for various geotechnical index properties, mineralogy, and compaction characteristics. The soils were then compacted in the lab and their resistivity values were measured using a custom built apparatus that essentially doubled as a compaction mold. Abu-Hassanein et al. (1996) found a general trend of decreasing resistivity as the clay fraction of the tested soils increased (Fig. 3.88). For the soils tested in their study, resistivity was relatively insensitive to changes in clay fraction above approximately 35%. Additionally, one of the soils did not follow the observed trend due to the significant presence of coarse particles (i.e., gravels). Once the coarse particles were removed and only the percent passing the #200 sieve was tested, the soil no longer proved to be an exception to the observed trend in Fig. 3.88. These results demonstrate the importance of calibrating resistivity measurements to specific soil conditions.



Figure 3.88: Resistivity as a function of clay fraction for soils in Abu-Hassanein et al. (1996).

More recently, Long et al. (2012) explored the relationship between resistivity and several geotechnical parameters (including clay content) for Norwegian quick clays. The resulting trends in this study complemented the results from Abu-Hassanein et al. (1996) (Fig. 3.89). Long et al. (2012) found that clay contents larger than 50% typically resulted in extremely low resistivity values (e.g., $\approx 5 \Omega$ -m). However, there was larger scatter in their data, particularly at larger clay contents, and the proposed polynomial trend line exhibited a relatively low coefficient of determination (Fig. 3.89). Neither Abu-Hassanein et al. (1996) nor Long et al. (2012) provided the functional form used for their trend lines. In either case, the results were empirical and specific to the site conditions in their studies.



Figure 3.89: Resistivity as a function of clay fraction for soils tested in Long et al. (2012).

Another study where clay content was examined and compared to measured resistivity values was presented in Shevnin et al. (2007). In this case, as part of this research a generalized theoretical resistivity model was developed for sandy-clay soils based on a binary mixing model (for details, see Shevnin et al. 2007). The model depends on inputs of soil porosity, CEC of clay, and estimates of the geometrical structure of the soil. The predicted resistivity-clay content relationship was compared with laboratory samples of sand-clay mixtures that were prepared with various percentages of bentonite and saturated with water of various salinity

concentration levels (Fig. 3.90). Shevnin et al. (2007) found very good agreement between their model and the results obtained in the laboratory. Generally, the clay content was overestimated when it varied between 10% and 40% and underestimated above 60% (Fig. 3.90). The average error in their estimates of clay content was approximately 19%. Moreover, they applied their proposed resistivity model to fit data from real field samples from the Mexican Petroleum Institute and a number of oil-contaminated sites. Based on the quality of the fit, Shevnin et al. (2007) found that practical application of the proposed technique could determine a limiting resistivity that differentiated between contaminated and clean soils. However, the proposed model necessitates knowledge or inference of soil parameters that are not always available or easy to measure (e.g., CEC), which can decrease its accuracy and limit its applicability.



Figure 3.90: Resistivity as a function of clay fraction and salinity (Shevnin et al. 2007).

Based on the aforementioned discussion, it is clear that clay content can have a large effect on measured resistivity. All studies demonstrated that resistivity decreases as the clay content increases (Abu-Hassanein et al. 1996; Shevnin et al. 2007; Long et al. 2012). However, there is difficulty in developed a generalized expression that directly relates clay content to resistivity. The recommendation is that a functional form be established for this expression based on site-

specific empirical correlations using the combined results of laboratory testing for clay content and measured field resistivity using ERI.

3.1.13 ATTERBERG LIMITS

Atterberg Limits provide a wealth of information regarding a soil and help distinguish between non-plastic silts and plastic clays. Establishing these index properties is often among the initial steps in a subsurface investigation since many aspects of geotechnical behavior demonstrate some dependence on liquid limit (LL) and plastic limit (PL) (e.g., swell potential, strength, compressibility, etc.). These limits are essentially special values of water content that signify transitions in soil behavior. Since relationships exist between water content and other geophysical measurements (particularly electrical/electromagnetic), it is unsurprising that research has been performed to investigate the relationship between geophysical measurements and Atterberg Limits.

3.1.13.1 Resistivity Methods

Many of the factors that affect resistivity are also associated with variations in LL and PL [and therefore plasticity index (PI)]. For example, resistivity is affected by a soil's capability to conduct electrical current on water adsorbed on the particle surfaces (e.g., Waxman and Smits 1968; Rhoades et al. 1976; Johnson et al. 1986; Sen et al. 1988; Sen and Goode 1992). As a result, clayey soils tend to exhibit smaller resistivity values relative to sands and gravels. The combined *BQ* terms in Eq. 3-68 provide an estimate of this surface conductivity and efforts have been made to relate the terms to Atterberg Limits (Abu-Hassanein et al. 1996; Bryson 2005):

$$LL = \alpha_1 (BQ)^{\beta_1} \tag{3-86}$$

$$PI = \alpha_2 (BQ)^{\beta_1} \tag{3-87}$$

where *LL* and *PI* are expressed as a decimal, *BQ* is in units of Siemens/m, and the α and β terms each represent empirical regression constants. Based on the data from Abu-Hassanein et al. (1996), Bryson (2005) found that $\alpha_1 = 3.33$, $\beta_1 = 1.59$, $\alpha_2 = 13.93$, $\beta_2 = 3.08$. Eqs. 3-86 and 3-87 imply an inverse proportionality between resistivity and Atterberg limits, one in which increases in *LL* and *PI* lead to decreases in resistivity as a result of the corresponding increases in surface

conductivity. The activity (A) of a soil also relates to the relative amount of surface conductivity and is directly proportional to PI based on the formulation proposed by Skempton (1953):

$$A = \frac{PI}{CF}$$
(3-88)

where A is the activity, PI is the plasticity index, and CF is the clay fraction (percentage by weight of soil less than 2 μ m). Highly active soils (primarily clays of the smectite group) will exhibit values of A larger than 1.25 and are generally more chemically reactive and susceptible to volume changes (Holtz and Kovacs 1981). Table 3.23 provides estimates of the activities of various clay minerals. Again, an inverse relationship is established whereby increases in PI will decrease resistivity due to the increased activity levels and greater surface conductance (Abu-Hassanein et al. 1996).

Mineral	Activity
Na-montmorillonite	4 – 7
Ca- montmorillonite	1.5
Illite	0.5 – 3
Kaolinite	0.3 – 0.5
Halloysite (dehydrated)	0.5
Halloysite (hydrated)	0.1
Attapulgite	0.5 – 1.2
Allophane	0.5 – 1.2
Mica (muscovite)	0.2
Calcite	0.2
Quartz	0

Table 3.23: Activities of various clay minerals (after Skempton 1953 and Mitchell and Soga 2005).

Ultimately, both *A* and *BQ* (and therefore surface conductance) are related to the specific surface area (SSA) of a soil. The SSA represents the ratio of the total surface area of a particle relative to its mass. The liquid limit (LL) and plastic limit (PL) reflect the SSA of the soil, the

thickness of the diffuse double layer of water, and the fabric that tends to form under the prevailing pore fluid conditions (Mitchell 1993; Muhunthan 1991). Therefore it is anticipated that equations exist that directly relate SSA to the LL of a soil (Farrar and Coleman 1967):

$$LL = 19 + 0.56A_s \tag{3-89}$$

where A_s is the SSA of a soil expressed in m²/g. Generally, larger specific surface areas (SSA) improves surface conductance and decreases resistivity (Kwader 1985). Given the direct relationship between SSA and LL as expressed in Eq. 3-89, again an inverse correlation in noted between an Atterberg limit and resistivity.

Considering the preceding discussion regarding the interrelatedness of Atterberg limits and various factors that affect conductance/resistivity (i.e., clay activity, surface conductance, SSA), various researchers have explored the development of direct relationships to estimate LL and PL (and/or PI) from resistivity measurements. However, a generalized relationship has not been established in the literature and efforts to develop empirical relationships have had mixed results despite the strong theoretical basis pointing to an inverse resistivity-plasticity relationship. Abu-Hassanein et al. (1996), Bery (2014), and Kibria and Hossain (2014) each found a distinct relationship for the soils in their study (Figs 3.91 – 3.93).

In the case of Abu-Hassanein (1996), the resistivity values decreased for increasing values of LL and PI. However, the tested soils were less sensitive to increases in LL and PI at higher values of LL and PI (i.e., 35 for LL and 20 for PI) (Fig. 3.91). As was the case with clay content, one of the soils significantly deviated from the measured LL and PI until the coarse fraction was removed.



Figure 3.91: Resistivity as a function of Atterberg limits for soils in Abu-Hassanein et al. (1996).

Bery (2014) compared time-lapse resistivity measurements to geotechnical subsurface testing results for a slope monitoring project in Penang Island, Malaysia. Based on the results of 32 soil samples, an inverse relationship was noted and a linear function was fitted for each of the trends for Atterberg limits:

$$w_I = a\rho + b \tag{3-90}$$

where w_i represented the Atterberg limit (i.e., either *LL*, *PL*, or *PI*), ρ is the measured resistivity, and a and b were empirical regression constants (Fig. 3.92). Bery (2014) established values of aand b for each Atterberg limit: a = -0.06 and b = 91.84 for *LL*, a = -0.018 and b = 45.89 for *PL*, a =-0.041 and b = 45.95 for *PI*. However, there was significant scatter in the data and relatively mediocre fit as a result.



Figure 3.92: Resistivity as a function of Atterberg limits for soils tested in Bery (2014).

Finally, Kibria and Hossain (2014) explored the relationship between resistivity and PI for various artificial soil samples created by mixing two types of commercially available bentonite (i.e., Volclay Na-bentonite and Panther Creek Ca-bentonite) with fine sand at various percentages (from 20% to 100% bentonite by weight). For a given saturation level, the resistivity decreased as *PI* increased (i.e., as more bentonite was added to the sample) (Fig. 3.93). Kibria and Hossain (2014) also noted differences in the *PI*-resistivity relationship based on mineral content (i.e., Na versus Ca in the bentonite). In the case of Na-bentonite, resistivity decreased by as much as 64% as *PI* increased from 40 to 226 (at 40% saturation). For Ca-bentonite, the change was more drastic over a smaller increase in *PI* at the same saturation level (i.e., 190% reduction in resistivity for *PI* increase from 7 to 55). These results demonstrate that the development of resistivity-plasticity empirical relationships may have to account for more factors in addition to *LL* and *PI*.



Figure 3.93: Resistivity as a function of Atterberg limits for artificial soils tested in Kibria and Hossain (2014): (a) samples prepared with Na-bentonite; and (b) samples prepared with Cabentonite.

Not all studies highlighted strong correlations between Atterberg limits and resistivity. Several factors can impact measured soil resistivity levels, including porosity, water content, and conductivity of fluid within the pore space as discussed in previous sections. Changes in some of these parameters do not necessarily reflect changes in *LL*, *PL*, or *PI*. For example, Giao et al. (2003) explored the relationships between various geotechnical parameters (i.e., organic content, water content, *PI*, unit weight, etc.) and resistivity for clays in the Nakdong River plain in South Korea. In this study, the *PI* results exhibited significant scatter and no conclusive trend with the measured resistivity levels (Fig. 3.94).



Figure 3.94: Resistivity as a function of *PI* for soils tested in Giao et al. (2003).

Kibria (2011) was able to establish a consistent inverse trend for the soils tested in that study, but this trend only predicted very small differences in resistivity for changes in *LL* and *PI* (i.e., the results were relatively insensitive to resistivity) (Fig. 3.95). Long et al. (2012) noted a fairly conclusive inverse relationship between resistivity and *PI*. Low *PI* marine clays tended to exhibit much smaller resistivity than corresponding high PI marine clays (Fig. 3.96). However, again there was significant scatter in the data and a clear trend line could not be established. These results demonstrate the difficulty in establishing consistent empirical relationships between Atterberg limits and resistivity. Though the results from Abu-Hassanein et al. (1996), Bery (2014), and Kibria and Hossain (2014), and even Long et al. (2012) support the basic theory that increases in *LL* and *PI* will decrease resistivity, other factors can influence the results and increase the amount of scatter in the data. More studies are necessary that are similar to Kibria and Hossain (2014) where soil plasticity is manipulated in the laboratory on controlled samples.

Such an approach would allow more control of the variables affecting resistivity in addition to *LL* and *PI* and would better elucidate the sensitivity of the results to these index parameters. The results from laboratory studies could then allow the development of more robust empirical models for the field. In the meanwhile, care must be exercised when utilizing field calibrated resistivity-plasticity relationships due to the relatively large scatter in the results.



Figure 3.95: Resistivity as a function of Atterberg Limits for soils tested in Kibria (2011).



Figure 3.96: Resistivity as a function of PI for marine soils tested in Long et al. (2012).

3.1.13.2 Electromagnetic Methods

Much of the preceding discussion related to the factors affecting resistivity measurements is directly relevant to soil electromagnetic properties as well. For example, the propagation of electromagnetic waves in soils is also influenced by clay activity, surface conductance, and the

SSA of a soil. A number of studies have explored the use of electromagnetic methods (particularly GPR) to delineate soils with different material properties, including Atterberg Limits (e.g., Carreon-Feyre et al. 2003; Rogers et al. 2009). Additionally, some researchers have indirectly explored the effects of Atterberg Limits on electromagnetic measurements. For example, Thomas et al. (2010a,b) studied the electromagnetic properties of fine grained soils using TDR to examine how electromagnetic dispersion (i.e., changes in apparent permittivity with frequency) was influenced by differences in Liquid Limit and shrink/swell potential. Thomas et al. (2010a,b) found that electromagnetic dispersion was greater in soils with larger LL. Additionally, electromagnetic dispersion for soils at water contents equal to their LL appear to depend on both LL and linear shrinkage. LL related to the high-frequency values in the electromagnetic dispersion curve and the linear shrinkage affected how much increase occurred in the high-frequency values as the signal frequency was reduced. Other studies have examined the effects of the dielectric constant of the pore fluid on the measured properties of different clay soils, including LL and PI (e.g., Fernandez and Quigley 1985; Fernandez and Quigley 1988; Acar and Olivieri 1989; Kaya and Fang 1997). In many cases, these studies were performed within the context of examining contamination in soils and the permeability of clay liners/barriers. However, despite these studies and the strong link between electromagnetic properties and Atterberg Limits, a direct relationship between the Atterberg Limits of a soil and relative permittivity from geophysical measurements has not proven feasible so far due to the many factors that influence both properties. For example, Spagnoli et al. (2011) found that the effects of the pore fluid dielectric constant on LL were inconsistent depending on mineralogy of the clay (Fig. 3.97). Additional research efforts will be necessary to isolate the effects of Atterberg Limits on electromagnetic properties of soils. Such efforts will improve our capabilities to predict LL and PL from electromagnetic geophysical methods.



Figure 3.97: *LL* as a function of pore fluid dielectric constant for different monomineralic clays (adapted from Spagnoli et al. 2011).

3.2 APPLICATIONS RELATED TO SHEAR WAVE VELOCITY MEASUREMENTS

This section discusses applications that utilize shear wave velocity (V_s) and the time-averaged shear-wave velocity in the upper 30 meters of the Earth's crust (V_{s30}). These applications include site response analysis using site terms in conjunction with a ground motion model, one-dimensional ground response analysis, and liquefaction triggering evaluation. The scale and resolution of various geophysical methods is an important aspect, particularly when measurements of V_s are concerned. Therefore, this section starts with a discussion on how scale and resolution affect wave velocity measurements and then discusses specific applications related to V_s .

3.2.1 DIFFERENCES IN SCALES AND RESOLUTION AMONG GEOPHYSICAL METHODS

Different geophysical methods provide a significant range in resolution that must be considered for a particular application. Different methods mobilize different volumes of soil, and measure the average wave velocity within the mobilized volume. The mobilized volume is a function of wavelength and sensor spacing, as shown in Fig. 3.98. Suspension logging provides an average vertical wave speed of the soil adjacent to the borehole wall over a sensor spacing of about 1m. Cross-hole testing measures the average horizontal shear wave and/or p-wave velocity between adjacent boreholes typically spaced meters apart. Downhole testing measures the vertical shear wave and/or p-wave velocity between the ground surface and the receiver. Enhanced insights into stratigraphy may be achieved by computing differences in travel time between multiple recordings as the receiver is lowered down the borehole. Surface wave methods utilize varying resolution, with short wavelengths and close receiver spacing used to measure the Rayleigh wave velocity of shallow layers, and long wavelengths and sensor spacing used to measure deeper layers.



Figure 3.98: Difference in scale and resolution among various geophysical methods for measuring *V*_s.

Selection of a particular method depends on the application. Borehole methods are well-suited to identifying stratigraphic details. For example, the suspension logging profile would identify the gray shaded layers in the profile in Fig. 3.98. The cross-hole method would accurately identify the horizontally continuous gray shaded layer, but would provide an average wave speed shallower in the profile where a layer intersects only two of the three boreholes. When using these high resolution methods, multiple measurements may be required to characterize

sites with significant horizontal variability. For example, thin gray shaded layers would be missed using a single suspension log, downhole, or cross-hole measurement.

Surface wave measurements, by contrast, average a larger volume of soil, providing an overall picture of the site. However, due to this averaging, surface wave methods may be unable to accurately measure the velocity of a layer whose thickness is small relative to its depth. For example, Fig. 3.99 shows three $V_{\rm S}$ profiles with a constant $V_{\rm P}$ profile, along with dispersion curves for the first four modes of Rayleigh wave propagation. The $V_{\rm S}$ profiles generally exhibit the same trend of increasing velocity with depth, but Profile 1 exhibits significant variation about this trend whereas Profile 3 is smooth, and Profile 2 is intermediate. At a depth of 8m, the V_s value for Profile 1 is only about 100m/s, whereas it is about 200m/s for Profile 3. Surface wave methods are therefore poorly suited to identifying the presence of thin soft layers. For example, the layer at 8m depth might be considered liquefiable for Profile 3, and non-liquefiable for Profile 1. The presence of this layer would be detected using a borehole method. For this reason, surface wave measurements should not be used when high resolution stratigraphic detail is desired. Surface wave methods utilize frequencies that are similar to earthquake ground motions. By averaging a similar volume of soil, surface wave methods provide a better indication of the velocity structure that will be mobilized by earthquake waves compared with borehole methods.



Figure 3.99: Three different V_s profiles produce essentially the same first-mode Rayleigh wave dispersion curve.

Considering that different methods provide different resolutions, the best practice is to combine geophysical measurements with geotechnical site investigation information to obtain a more comprehensive understanding of the site. Thin problematic layers are less likely to be missed using this approach.

3.2.2 UNCERTAINTY IN GEOPHYSICAL METHODS AND VARIOUS PROXIES

Uncertainty in geophysical measurements can be conceptualized as arising from intramethod variability, meaning variability among measurements of a single method at the same site, and inter-method variability, meaning uncertainty between or among different geophysical measurements at the same site. As defined by Moss (2008), intramethod uncertainty is caused by inversion of surface wave dispersion curves, curve-fitting procedures, sensor errors, travel time picks, etc. Inter-method variability is attributed to differences in the scale of the different measurement techniques, difficulties in measuring shallow sediments using invasive methods, and soil-disturbance effects associated with invasive methods.

Method	Coefficient of Variation
Downhole, suspension logging, and seismic cone penetration testing	1% to 3%
Spectral analysis of surface waves	5% to 6%
Correlation with geologic unit	20% to 35%

Table 3.24: Coefficients of variation for measuring V_{s30} using various geophysical methods (Moss 2008).

Moss (2008) quantified intramethod uncertainty in V_{s30} for both non-invasive and invasive geophysical methods, as well as for correlations with surface geology, using the coefficient of variation (COV), i.e., the standard deviation divided by the mean of dataset. Table 3.24 provides a list of COV values of different methods. Invasive methods such as suspension logging or downhole measurements are associated with the least amount of uncertainty, with COV values on the order of 1-3%, while for surface-wave methods (SASW, MASW) COV was found to be on the order of 5-6%. Furthermore, Moss (2008) found that the ratio of V_{s30} measured by a non-invasive method to that measured by an invasive method tends to be higher than 1 at soft sites,

and lower than 1 at stiff sites. Moss (2008) postulated that this trend may be caused by soil disturbance during invasive methods, with strain softening of softer soils causing a decrease in V_{s30} , and strain-hardening in stiffer soils causing an increase in V_{s30} . It was shown that near-surface effects (such as low confining pressure in the upper few meters of soil) did not contribute much to intra-method variability (Moss 2008).

Moss (2008) also studied V_{s30} relations based on surface-geology correlations, and found that the COV is generally about 20-35%, with COV increasing with mean V_{s30} . The reason for such high uncertainty is attributed to the combined errors in measurement, modeling, and spatial variability of V_{5} . One should be wary of using such correlations, and it is stressed that the shear wave velocity at a site should be directly measured, rather than correlated via proxy, whenever possible.

3.2.3 MEASUREMENT OF V530 FOR COMPUTING SITE AMPLIFICATION FACTOR

The primary factors that influence earthquake ground motions are source, path, and site effects. Site effects refer to the characteristics of the near-surface soil and rock that can significantly alter the amplitude and frequency content of seismic waves. Anderson et al. (1996) note that the upper 30 m of a site can significantly alter earthquake ground motion despite the fact that the upper 30 m generally accounts for less than 1% of the distance to the earthquake source. This underscores the importance of the time-averaged shear-wave velocity in the upper 30 meters of soil (V_{s30}), which is computed as:

$$V_{S30} = \frac{30 \, m}{\sum_{i}^{n} \frac{h_{i}}{V_{S_{i}}}} \tag{3-91}$$

where h_i and V_{Si} are the thickness and shear wave velocity of layer *i*, respectively, in a profile with *n* layers in the upper 30 m. Note that while V_{S30} may be computed with British units (by replacing the denominator with 100 ft), SI units are used in several applications related to site amplification based on input of V_{S30} . Therefore, the authors advise consistent use of the SI system when applying methods described within this section. Note that V_{S30} is not defined as the arithmetic mean of the V_s profile in the upper 30 m, but rather is equal to 30 m divided by the travel time of a vertically propagating shear wave through the upper 30 m.

The seismic provisions in building codes are periodically updated based on recommendations within the NEHRP Provisions and Commentary (BSSC 2009). One important aspect of the NEHRP Provisions and Commentary is the specification of design-basis ground motions. These ground motions are derived for rock site conditions at 0.2 sec and 1.0 sec period from probabilistic seismic hazard analysis (PSHA) and then modified by site amplification factors. These site amplification factors are based in large part on the seminal studies of Borcherdt (1994) using a reference shear wave velocity of 1050 m/s for a uniform site condition. Borcherdt (1994) originally showed consistent correlations between site amplification and V_{s30} , leading to its adoption in the NEHRP Provisions and Commentary. The NEHRP *Provisions* originally outline site classes defined by binned ranges of V_{s30} , which are available in the Caltrans Seismic Design Criteria (SDC) (Caltrans 2013) and reproduced in Table 3.25. It is important to note that these site classes are also used in both the current ASCE 7-10 *Standard Minimum Design Loads For Buildings and Other Structures* (ASCE 2013) and the California Building Code (CBSC 2013).

Soil Profile	Sail Brofile Description*		
Туре			
А	Hard rock with measured shear wave velocity $V_{s30} > 1,500 \text{ m/s}$ (5000 ft/s)		
В	Rock with shear wave velocity 760 m/s < V_{s30} < 1,500 m/s (2,500 < V_{s30} < 5000 ft/s)		
С	Very dense soil and soft rock with shear wave velocity 360 m/s < V_{S30} < 760 m/s (1,200 < V_{S30} < 2,500 ft/s) or with either standard penetration resistance $N > 50$ or undrained shear strength $s_u \ge 100$ kPa (2,000 psf)		
D	Stiff soil with shear wave velocity 180 m/s < V_{S30} < 360 m/s (600 < V_{S30} < 1,200 ft/s) or with either standard penetration resistance $15 \le N \le 50$ or undrained shear strength $50 < s_u < 100$ kPa (1,000 < $s_u < 2,000$ psf)		
E	A soil profile with shear wave velocity $V_{s30} < 180 \text{ m/s}$ (600 ft/s) or any profile with more than 3 m (10 ft) of soft clay, defined as soil with plasticity index $Pl > 20$, water content $w \ge 40$ percent, and undrained shear strength $s_u < 25$ kPa (500 psf)		
F	 Soil requiring site-specific evaluation: 1. Soils vulnerable to potential failure or collapse under seismic loading; i.e. liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peat and/or highly organic clay layers more than 3 m (10 ft) thick 3. Very high-plasticity clay (<i>PI</i> > 75) layers more than 8 m (25 ft) thick 4. Soft-to-medium clay layers more than 36 m (120 ft) thick 		

*Note: The soil profile types shall be established through properly substantiated geotechnical data. Key: $V_{s_{30}}$ = time-averaged shear wave velocity through upper 30 m; s_u = undrained strength; PI = plasticity index; N = SPT blowcount.

Table 3.25: NEHRP Site Classes (after Caltrans 2013).

Ground motion models (GMMs, formerly called "Ground Motion Prediction Equations" [GMPEs] and "attenuation relations") are empirical models that consider the effects of seismic source, travel path, and local site conditions on ground motion intensity measures (GMIMs), such as peak ground acceleration, PGA, peak ground velocity, PGV, or pseudo-spectral acceleration, PSa, at a specified oscillator period, among other GMIMs, at a given site. Generally, GMMs output the geometric mean of the two horizontal components of motion (see Bozorgnia et al. 2014 for a list of GMM model developer teams), but some newer models also give the vertical motion (e.g., PEER 2013). The development of GMMs has its origins in the 1980s and 1990s based on seminal works by Campbell (1981), Youngs et al. (1988), Joyner and Boore (1988), Idriss (1991), Boore et al. (1993), Sadigh et al. (1993), Campbell and Bozorgnia (1994), Abrahamson and Silva (1997), Boore et al. (1997), Campbell (1997), and Sadigh et al. (1997). These models have since undergone revisions as part of the "Next Generation of Ground-Motion Attenuation Models" (NGA) phase 1 project concluded in 2008 (see Power et al. 2008 for an overview) and NGA-West2, NGA-East, and NGA-Subduction projects (see Bozorgnia et al. 2014 for an overview). Douglas (2015) provides a holistic review of published GMM models from around the world, including their functional forms, what data are used to derive coefficients used in the model regression, and explanations of various source-, path-, and site-related input parameters. Caltrans' ARS Online software makes use of the average of two NGA GMM models, Campbell and Bozorgnia (2008) and Chiou and Youngs (2008), to compute seismic hazard and response spectra for any location California, based on user-specified latitude/longitude coordinates and the $V_{s_{30}}$ at that location (Caltrans 2013). It is important to note that as part of the NGA-West2 project, the Campell and Bozorgnia and Chiou and Youngs GMMs were updated to apply to a broader database of earthquake ground motion recordings at various magnitudes and distances (Bozorgnia et al. 2014; Campbell & Bozorgnia 2014; and Chiou and Youngs 2014).

Early GMMs (e.g., Abrahamson and Silva 1997) used various qualitative site classes, ranging from as simple as "rock" versus "soil", to including potentially vague descriptors such as "stiff" or "soft", to including geologic ages (such as Holocene, Pleistocene, Quaternary, etc.), as the GMM developer is at liberty to select any site classification scheme desired. However, an effort was made in developing the NGA project to use V_{s30} for computing GMM site terms, which is considered to be more diagnostic in determining site amplification than the broad and ambiguous soil and rock categories previously used (Power et al. 2008). The decision to use V_{s30}

for development of site amplification factors in GMMs stems from a movement towards use of a single representative parameter that captures the signature of a given site. This ideology was upheld in the development of the NGA-West2 site database, which retained V_{S30} as the primary site parameter, as it effectively describes first-order site effects, both linear and nonlinear, and is arguably the most easily-determinable site parameter, when compared with site period or basin depth. Seyhan et al. (2014) provides further justification of use of V_{S30} .

It is evident that the continuous nature of a numeric value such as V_{s30} will eliminate confusion associated with selecting a qualitative descriptor for a given site. It also alleviates issues that potentially arise when the V_{s30} of a given site falls close to one of the NEHRP site class boundaries. For example, Wills et al. (2000) created intermediate site classes for development of a site condition map of California based on surface geology and V_{s30} , because mean values of common types of geology fall near site class borders (e.g., the Franciscan Complex has a distribution of V_{s30} values that cross the B/C site class border [760 m/s]). It has also been shown that site amplification factors from the current NEHRP provisions have discrepancies compared to those used in NGA GMMs (Seyhan and Stewart 2012).

In some cases, a V_s measurement is made to a depth less than 30 m, and must be extrapolated to 30 m to compute V_{s30} . Kwak et al. (2017) summarized five such methods, and assessed their accuracy using a world-wide data set in which V_s was measured to at least 30 m. These profiles were extrapolated to 30 m based on shallow portions of the profile measured to a depth *Z*. The methods generally produced unbiased estimates of V_{s30} , meaning that the average of the error in the extrapolation was close to zero. However, the standard deviation of the error term increased as the depth of the V_s profile decreased. Hence, when V_{s30} is needed for a particular application, the V_s profile should be measured to 30 m to avoid introducing unnecessary uncertainty into the prediction. However, these extrapolation methods are useful when existing V_s profile data is available, and new measurements are impractical or impossible to obtain.

Site amplification factors describe both linear (e.g., Boore et al. 1997, Walling et al. 2008) and nonlinear (e.g., Choi and Stewart 2005, Walling et al. 2008, Seyhan and Stewart 2014) ground response to earthquake shaking. Nonlinearity in site response occurs because strong ground shaking softens the soil and increases its damping, thereby altering the characteristics of

earthquake waves that propagate through the soil to the surface. Nonlinearity is most pronounced for soft sites shaken by strong ground motions, as shown in Fig. 3.100 from Seyhan and Stewart (2014). The strong ground motion database contains very few recordings from soft sites shaken with strong ground motion. Therefore, nonlinear site amplification functions are commonly constrained by one-dimensional ground response analysis.



Figure 3.100: Site amplification models for various spectral periods and various V_{S30} values (Seyhan and Stewart 2014). *PGAr* is the peak horizontal acceleration corresponding to a reference site condition of V_{S30} = 760 m/s, and ln(*F*) is the natural logarithm of the amplification factor.

3.2.4 MEASUREMENT OF VS PROFILES FOR GROUND RESPONSE ANALYSIS

One-dimensional ground response analysis (1D GRA) models the vertical propagation of shear waves through a horizontally layered soil profile. A key input parameter to these models is the distribution of shear wave velocity with depth; knowledge of V_{s30} alone is inadequate for running a 1D GRA. The simulations may be performed using equivalent-linear or nonlinear methods, and involve input of modulus reduction and damping behavior for each soil layer.

Details of the theory and application of 1D GRA is beyond the scope of this report. Rather, this report focuses on use of geophysical methods to obtain a shear wave velocity profile for input into a 1D GRA.

The Caltrans (SDC) calls for site-specific response analysis (e.g., 1D GRA) for sites characterized as NEHRP soil class types E and F (Table 3.25) for final design. Also, site-specific analysis is required for type F sites for preliminary design; with recommendations provided to extend use of these procedures to type E sites as well (Caltrans 2013). Generally, the Type E preliminary design spectra will exceed spectra developed using a 1D GRA, which is the reason why the Caltrans SDC recommends performing a 1D GRA for preliminary spectrum development.

GMMs (conditioned on V_{s30}) provide predictions of site response based on global averages (referred to as ergodic), which can be biased for a particular site (Stewart et al. 2014). The ergodic assumption may be a poor predictor of site response at sites with a strong impedance contrast (e.g., a soil profile resting on shallow rock), or at soft sites that are not well-represented in the empirical ground motion database. A 1D GRA can therefore reduce uncertainty (but not eliminate uncertainty, as discussed later) compared with an ergodic site term. A comprehensive study which navigated the literature and sought to provide guidelines for performing 1D GRA is presented by Stewart et al. (2014).

A key input to a 1D GRA is a V_s profile that defines V_s from the surface to a depth deemed adequate for the analysis. In general, 1D GRA models permit specification of an elastic bedrock condition at the base of the profile. Hence, the depth of exploration should extend into stiff material that will mobilize small strains in response to imposed earthquake ground motions such that the elastic bedrock assumption is reasonable. At sites with shallow rock, this depth is easy to determine. In deep basins, the bottom of a 1D GRA is often determined as the depth where V_s exceeds a threshold value, ideally as 760 m/s, though often lower due to practical considerations. For these reasons, a V_s profile for a 1D GRA may need to extend deeper than 30 m, depending on site geology.

Borehole methods tend to provide a vertical profile of V_s at a point within a site. The degree to which the profile is representative of the site depends on geologic conditions, and the

associated scale of fluctuation. In some cases, scales of fluctuation within a particular geologic unit are available in the literature (e.g., Thompson et al. 2007 for San Francisco Bay area sediments). When such studies are not available, knowledge of site geology can aid the interpretation of the horizontal scale of fluctuation and guide the horizontal sampling interval. Note that the horizontal scale of fluctuation is generally much larger than the vertical scale of fluctuation due to the manner in which soils are deposited. To account for variability within a site, ideally multiple profiles should be measured and analyzed to gain insights into the influence of spatial variability on the resulting ground surface motions. If multiple measurements are impractical or unavailable, there are methods for randomizing a measured V_S profile to account for spatial variability. For example, Toro (1995) presented a method for developing probabilistic models of the site velocity profiles for site response studies in which the depth to the layer contacts and the V_S values are treated as random variables. Values of V_S among the layers are spatially correlated.

Surface wave measurements average a larger volume of soil than borehole methods, and utilize wavelengths that are similar to those mobilized during earthquake shaking. Fewer surface wave measurements may therefore be required to characterize a site because each measurement averages a large volume of soil. However, surface wave inversions are non-unique, meaning that many shear wave velocity profiles may be consistent with a measured dispersion curve. Many computer programs for inverting surface wave dispersion data are capable of providing many profiles that are consistent with the dispersion curve. Griffiths et al. (2016) performed a study in which many profiles were selected to be consistent with a measured surface wave dispersion curve. The dispersion curve was termed the "site signature" and they found that selecting V_s profiles consistent with the site signature produced significantly less variability in surface motion than other methods for randomizing V_s profiles (e.g., Toro 1995). They conclude that the site signature provides important information that should be incorporated into selection of random profiles for ground response analysis.

One-dimensional ground response analysis does not capture all of the physical process that affect site response. Ground motion at a site is affected by a complex interaction of 2D and 3D effects, including basin-edge effects, topographic effects, inclined body waves, surface-waves, and complex geologic conditions that differ significantly from horizontally layered stratification.

Furthermore, low frequency waves mobilize wavelengths that are often significantly longer than the thickness of profile used in a 1D GRA study, and are influenced by deep soil and rock structures that cannot be captured in 1D GRA models. Thompson et al. (2012) utilized multiple earthquake records from 100 KiK-net sites in Japan to study the degree to which site response was one-dimensional at the strong motion sites. They computed transfer functions from the measured ground motions, and also developed theoretical transfer functions consistent with the site velocity profiles. They found that of the 100 sites, 69 sites exhibited low inter-event variability thereby providing a suitable means for separating site effects from source and path effects. Of these 69 sites, only 16 exhibited site response that was consistent with onedimensional wave propagation. Furthermore, they found that surface wave dispersion curves were spatially variable within sites that exhibited non-1D wave propagation, and spatially consistent within sites that were consistent with 1-D wave propagation. Therefore, making multiple surface wave measurements within a site may provide insights into the extent to which ground motions can be accurately modeled by 1D GRA. Afshari and Stewart (2016) performed a similar study for vertical arrays in California, and found that only 4 of the 12 sites examined to date exhibit a reasonably good fit between measured and theoretical amplification functions. They found that 1D GRA was able to reduce uncertainty in the site term compared with ergodic site factors, but only at spectral periods shorter than about 1.0s. At longer periods, the uncertainty reverted to the ergodic values.

Considering the complex processes that influence site response, it is not surprising that 1D GRA often produces biased predictions, particularly at sites with complex geologic conditions. Nevertheless, 1D GRA can improve upon the ergodic predictions provided by empirical site amplification functions, particularly at soft sites and at sites with a strong impedance contrast. Many different geophysical methods can be used to provide the V_S profile required as an input to 1D GRA codes, and engineers are urged to consider analyzing multiple profiles that are consistent with the anticipated scale of horizontal variation.

Small strain damping is an important consideration for input to 1D GRA, and can be directly measured using geophysical methods (e.g., Press and Healy 1957; Mok et al. 1988; Stewart and Campanella 1993; Rix et al. 2000; Wang et al. 2006; Yang et al. 2011). Most commonly, small-strain damping is inferred from the amplitude of the measured waveforms at various distances

from the seismic source. Amplitude decreases with distance from the source due to geometric attenuation, and due to material damping. Separating out the contribution from material damping therefore requires independent knowledge of geometric damping. Wave amplitude attenuates more rapidly with distance in soft soils than in stiff soils. Therefore, geometric damping is a function of the velocity profile, which complicates the estimation of material damping (Foti et al. 2015).

3.2.5 MEASUREMENT OF VS FOR LIQUEFACTION TRIGGERING EVALUATION

Recent notable earthquakes such as the 2010 Mw 7.0 Darfield (Canterbury), 2011 Mw 6.2 Christchurch, and 2015 Mw 7.8 Gorkha (Nepal) earthquakes have continued to remind us of the devastating effects of liquefaction of saturated, loose, granular soils. A significant amount of ongoing research has been devoted to the study of various aspects of liquefaction, especially given the continued influx of data from recent seismic events (e.g., Boulanger and Idriss 2014; Maurer et al. 2014; Robertson 2015).

Liquefaction hazard assessment must address three specific concerns: (1) susceptibility of the soil to liquefaction; (2) initiation or triggering of liquefaction; and (3) effects and damage caused by soil liquefaction. Susceptibility of a soil to liquefaction depends on soil type, with cohesionless "sand-like" soils generally considered susceptible to liquefaction and cohesive "clay-like" soils considered susceptible to cyclic softening (Idriss and Boulanger 2008). An assessment of susceptibility can only be made based on direct observation of a soil sample, either by visual manual classification, or by laboratory testing. Borehole geophysical methods facilitate assessment of liquefaction susceptibility based either on trimmings retrieved as the borehole is advanced, or based on SPT samples taken during drilling. Downhole methods that utilize the cone penetrometer (e.g., SCPT) provide a soil behavior type index that may be used to assess liquefaction susceptibility. Non-invasive methods generally do not provide a means of assessing liquefaction susceptibility, and independent knowledge of the soil type is therefore required to utilize V_s to perform a liquefaction triggering evaluation using non-invasive techniques. Independent knowledge may be obtained by supplemental site investigations, including drilling and sampling, or by knowledge of site geology.

For soils that are deemed susceptible to liquefaction, the factor of safety against liquefaction triggering is computed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). Alternatively, the probability of liquefaction may also be computed from the CRR and CSR. Shear wave velocity is utilized in a number of different methods for computing CRR, and also influences CSR due to its role in site response.

3.2.5.1 Correlations Between Vs and CRR

Although CRR is most commonly correlated with penetration resistance measurements, correlation with V_s provides some fundamental benefits (Table 3.26). Development of excess pore pressure during undrained loading is fundamentally a strain-driven phenomenon (Dobry et al. 1982). When cyclic shear strains exceed a threshold, pore pressures develop and eventually may lead to liquefaction. Mobilized shear strains are fundamentally related to the shaking intensity and to the soil stiffness. In fact, the shear strain amplitude for shear waves propagating through an unbounded elastic medium is equal to PGV / V_s , where PGV is the particle velocity amplitude. The presence of the free surface, where shear strain must be zero even when PGV is non-zero, alters this relationship for application to liquefaction problems, but the relation between shear strain and shear wave velocity is nevertheless fundamental. It is not surprising, therefore, that V_s correlates with CRR.

	Test Type			
Feature	SPT	CPT	V_s	BPT
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

Table 3.26: Advantages and disadvantages of various field tests for assessment of CRR.

Models directly relating V_S to CRR initiated in the 1990's (e.g., Robertson et al. 1992, Kayen et al. 1992, Lodge 1994), though these efforts were preceded by relations between V_S and the threshold acceleration required to develop pore pressure (Dobry et al. 1982), and based on SPT- V_S correlations (Seed et al. 1983). The database of V_S profiles at liquefaction sites has continued to grow with time, resulting in more recent correlations by Andrus and Stokoe (2000) and Kayen et al. (2013). These two more recent relationships are discussed in more detail herein.

3.2.5.1.1 Andrus and Stokoe (2000)

The relationship by Andrus and Stokoe (2000) is provided by Eq. 3-92:

$$CRR = \left[a \left(\frac{K_c V_{S1}}{100} \right)^2 + b \left(\frac{1}{V_{S1}^* - K_c V_{S1}} - \frac{1}{V_{S1}^*} \right) \right] MSF$$
(3-92)

Where $V_{S1} = V_S (P_a / \sigma_v)^{0.25}$ is the overburden-corrected shear wave velocity, V_{S1}^* is the limiting upper value of V_{S1} for cyclic liquefaction occurrence, *a* and *b* are curve-fitting parameters, *MSF* is the magnitude scaling factor, and K_c is a correction factor caused by cementation and ageing. Based on evaluation of case history data, the values of V_{S1}^* were found to be dependent on fines content, *FC*, as illustrated in Eqs. 3-93 below:

$$V_{S1}^* = 215 \text{ m/s}, \text{ for sands with FC} \le 5\%$$
 (3-93a)

$$V_{S1}^* = 215 - 0.5(FC - 5) \text{ m/s}, \text{ for sands with } 5\% < FC < 35\%$$
 (3-93b)

$$V_{S1}^* = 200 \text{ m/s}, \text{ for sands and silts with FC} \ge 35\%$$
 (3-93c)

Furthermore, Andrus and Stokoe found that a = 0.022 and b = 2.8 provided reasonable bounds for the case history data. Furthermore, Andrus and Stokoe suggest using $MSF = (M_w/7.5)^{-2.56}$, where M_w is moment magnitude. The value assigned to K_c should be 1.0 for Holocene soils, and average estimates of K_c are 0.6 to 0.8 for Pleistocene-age soils. Andrus and Stokoe suggest caution and use of engineering judgment in assigning a K_c value lower than 1.0.

3.2.5.1.2 Kayen et al. (2013)

Kayen et al. (2013) defined the cyclic resistance ratio as a function of probability of liquefaction using Eq. 3-94:

$$CRR = \exp\left\{\frac{\left(0.073 \cdot V_{S1}\right)^{2.8011} - 2.6168 \cdot \ln\left(M_{w}\right) - 0.0099 \cdot \ln\left(\sigma_{vo}^{'}\right) + 0.0028 \cdot FC - 0.4809 \cdot \Phi^{-1}(P_{L})}{1.946}\right\}$$
(3-94)

Where $V_{S1} = V_S (P_o/\sigma_v)^{0.25}$ is the overburden corrected shear wave velocity, σ_{vo}' is the in-situ vertical effective stress, *FC* is fines content, and P_L is the probability of liquefaction. For deterministic application, Kayen et al. suggest using $P_L = 15\%$. The Kayen et al. relationship for $P_L = 15\%$ and various M_w values is compared with the Andrus and Stokoe relationship for $M_w = 7.5$ in Fig. 3.101. The Andrus and Stokoe relationship tends to ascend more abruptly as V_{S1} increases above about 200 m/s, and is lower than the Kayen et al. $M_w = 7.5$ relationship at V_{S1} values less than about 210 m/s.



Figure 3.101: Comparison of Andrus and Stokoe (2000) and Kayen et al. (2013) relationships for CSR as a function of V_{s_1} .

3.2.5.2 Influence of Vs on CSR

Seed and Idriss (1971) expressed the CSR as follows:

$$CSR = \frac{\tau_{max}}{\sigma_{v}'} = 0.65 \left(\frac{PGA}{g}\right) \left(\frac{\sigma_{v}}{\sigma_{v}'}\right) r_{d}$$
(3-95)

where *PGA* is the peak horizontal ground acceleration, *g* is the acceleration due to gravity, and r_d is a shear stress reduction factor due to the deformable dynamic response of the soil column (Idriss and Boulanger 2008). Shear wave velocity influences the *PGA* term because a V_{s30} value

must be selected to define an ergodic site term when using a GMM, or because a V_s profile is needed when running a 1D GRA. These site response issues were discussed in the previous section and are not repeated here. The r_d term also depends on V_s because it is related to the dynamic response of the soil column. The degree to which V_s factors in to r_d expressions varies by method. Cetin et al. (2004) explicitly include $V_{S,12m}^*$ (i.e., the average shear wave velocity in the upper 12m) as given in Eq. 3-96:

$$r_{d} = \frac{\left(1 + \frac{-23.013 - 2.949 \cdot a_{\max} + 0.999 \cdot M_{w} + 0.0525 \cdot V_{S,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \left(-d + 0.0785 \cdot V_{S,12m}^{*} + 7.586\right)}}\right)}{\left(1 + \frac{-23.013 - 2.949 \cdot a_{\max} + 0.999 \cdot M_{w} + 0.0525 \cdot V_{S,12m}^{*}}{16.258 + 0.201 \cdot e^{0.341 \left(0.0785 \cdot V_{S,12m}^{*} + 7.586\right)}}\right)} \pm \varepsilon_{rd}$$
(3-96)

Where a_{max} is the peak horizontal acceleration in units of g, M_w is moment magnitude, d is depth, and ε_{rd} is a normally distributed random variable with zero mean and standard deviation defined by Eq. 3-97. The Cetin et al. (2004) relationship for r_d was adopted in the V_s triggering procedure by Kayen et al. (2013).

$$\sigma_{\varepsilon r_d} = d^{0.850} \cdot 0.0198 \quad \text{for } d < 12.2 \text{ m}$$

$$\sigma_{\varepsilon r_d} = 12.2^{0.850} \cdot 0.0198 \quad \text{for } d \ge 12.2 \text{ m}$$
(3-97)

Idriss (1999) formulated an r_d expression in terms of depth and moment magnitude, but not shear wave velocity. This expression forms the basis of the Idriss and Boulanger (2008) SPT- and CPT-based liquefaction triggering procedures, and was based on suites of 1D GRA performed on various soil profiles using a variety of ground motions.

3.2.5.3 Considerations for V_s-based Liquefaction Triggering Evaluation

A key benefit of using V_s field techniques for assessment of liquefaction triggering potential is that it is related to the small-strain shear modulus, G_{max} . V_s may therefore be useful for other engineering evaluation procedures, in addition to liquefaction triggering evaluation, such as settlement analysis, soil structure interaction applications, and others. A second benefit of V_s based liquefaction triggering procedures is that V_s measurements are less sensitive to fines content than penetration resistance measurements. The presence of fines within a matrix of coarser-grained soil particles has little effect on V_s because the shear waves are carried by interparticle contacts and the fines do not significantly participate in the wave propagation mechanism (the effect, of course, becomes large as the fines content increases and the coarse grained particles are floating in a matrix of fine particles). The fines, however, more significantly influence penetration resistance measurements because the soil becomes more compressible, and excess pore pressures dissipate more slowly. The fines corrections commonly applied in liquefaction triggering evaluation procedures generally reflect two distinct phenomena: (1) the influence of fines on penetration resistance or V_s , and (2) the influence of fines on liquefaction more directly corresponds to the influence of fines on liquefaction resistance.

Andrus and Stokoe (2000) outline potential disadvantages in using *V*_s methods for liquefaction evaluation. First, the fact that small-strain shear waves fail to induce liquefaction-inducing excess pore-water pressure buildup (unlike the more destructive penetration tests) renders them more sensitive to weakly-cemented soils or silty soils above the water table (in which negative pore-water pressures can increase effective stresses and thus increase *V*_s). Second, as physical samples cannot be obtained directly from geophysical methods, potentially-non-liquefiable layers with clays or higher non-plastic FC may be missed. Finally, the likelihood of overlooking thin, potentially liquefiable strata increases when using test intervals that are too large. These concerns should be considered when planning a site investigation for purposes of liquefaction resistance analysis, and in such cases having supplementary data from penetration resistance tests is recommended.

3.3 ESTIMATING V_S FROM PENETRATION RESISTANCE MEASUREMENTS AND FROM PROXIES

Shear wave velocity (V_s) occasionally must be estimated at a site where a geophysical measurement is unavailable and cannot reasonably be obtained. In such circumstances, V_s can be estimated based on correlations with penetration resistance, or with various proxies such as surface geology, ground slope, or elevation. This chapter discusses correlations between V_s and penetration resistance, followed by a discussion of proxy-based methods. These methods are shown to provide highly uncertain estimates of V_s , which may significantly influence ground motion predictions. We therefore provide an example in which uncertainty in V_{s30} is propagated

through a GMM to obtain distribution functions representing surface motion for various methods for estimating V_{S30} . The primary conclusion from this section is that V_S should be measured whenever possible, and correlations with penetration resistance or proxy-based methods should only be used when direct measurements cannot reasonably be obtained.

3.3.1 CORRELATIONS BETWEEN V_S AND PENETRATION RESISTANCE

Compared with penetration resistance, V_s is more sensitive to cementation, in situ stresses, and age, and less sensitive to fines content. It is therefore no surprise that the correlation between V_s and penetration resistance tends to be poor. Nevertheless, correlation with penetration resistance does provide an incremental improvement in estimating V_{s30} compared with correlations with surface geology or geomorphology, and can be quite good when calibrated within a particular site, and is therefore valuable in some contexts. This section focuses first on explaining appropriate and inappropriate uses of correlation between V_s and penetration resistance, then presents a number of correlations that have been formulated including one that is specific for Caltrans bridge sites, and finally presents the ground motion uncertainty that arises from prediction errors in various methods of obtaining V_{s30} .

An example of appropriate use of correlations between V_s and penetration resistance is screening a large number of bridges to identify a manageable subset for seismic hazard evaluation. Caltrans owns approximately 13,000 bridges, most of which were constructed before 1970. Traditional geotechnical site investigations including measurements of penetration resistance (SPT blow count and/or CPT tip resistance) were performed at these bridge sites, results of which are available in logs of test borings. However, shear wave velocity profiles were not measured at most of these sites. Measuring V_{s30} at the thousands of bridge sites where geophysical measurements were not made is not reasonable for seismic hazard screening. In this case, utilizing the correlation between V_s and penetration resistance would be more accurate than proxy-based correlations, and could therefore be utilized to improve the screening procedure.

An example of inappropriate use of correlations between V_s and penetration resistance is design of a new bridge or detailed retrofit evaluation of an existing bridge. In these cases, a direct geophysical measurement is generally feasible, and should be performed to significantly reduce

the error in the resulting $V_{s_{30}}$ value. Correlation with penetration resistance should never be used when a direct geophysical measurement is feasible.

3.3.2 PUBLISHED RELATIONS BETWEEN V_S AND PENETRATION RESISTANCE

Many studies have been performed to derive relations between V_s and blow count (N) from various regions around the world. Brandenberg et al. (2010) and Wair et al. (2012) summarized many of these studies, and equations are not reproduced in this report for brevity. Most published relations utilize the functional form $V_s = \beta_0 \cdot N^{\beta_1}$, where the constants β_0 and β_1 were determined by statistical regression of a data set. Significant differences among these relations indicate regional variability. Therefore, relations formulated for a specific region should be utilized when available.

Although the functional form $V_s = \beta_0 \cdot N^{\beta_1}$ is common, it ignores the fact that V_s and penetration resistance scale differently with overburden pressure. Note that $(N_1)_{60} = N_{60}(\sigma_v'/\rho_a)^m$, and $V_{51} = V_s(\sigma_v'/\rho_a)^n$, where $m \neq n$ in general. Therefore a "uniform" soil profile with constant $(N_1)_{60}$ and V_{51} will exhibit a relation between V_s and N_{60} that depends on overburden stress, and directly correlating V_s with N_{60} is problematic. Acknowledging the overburden scaling problem, a number of relations have explored using various combinations of overburden-corrected values. Sykora and Koester (1988) evaluated a relation between V_s and $(N_1)_{60}$, and found the correlation to be poorer than the relation directly between V_s and N_{60} because both V_s and N_{60} vary with overburden stress, whereas $(N_1)_{60}$ does not. Andrus et al. (2004) correlated the overburdencorrected shear wave velocity with overburden-corrected blow count values using a functional form $V_{51} = \beta_0 \cdot (N_1)_{60}^{\beta_1}$ for Holocene clean sands. This functional form is superior because it removes the effect of overburden since both V_{51} and $(N_1)_{60}$ are independent of overburden stress, provided that the exponents m and n are known.

Brandenberg et al. (2010) utilized SPT blow counts and suspension logs at Caltrans bridge sites to develop a relation defining V_s as a function of N_{60} and σ_v because independent knowledge of the *m* and *n* was not available. The functional form adopted by Brandenberg et al. (2010) is rearranged here as shown in Eq. 3-98:

$$V_s = \beta_0 N_{60}^{\beta_1} \sigma_v^{\prime \beta_2} \tag{3-98}$$

where β_0 , β_1 , and β_2 are regression constants that depend on soil type (Table 3.27). The regression also included an inter-boring random effect term, η , with zero mean and standard deviation τ , and an intra-boring variation term, ε , with zero mean and standard deviation σ .

Soil Type	eta_0	eta_1	eta_2	σ	τ
Sand	57.1	0.096	0.236	0.57-0.07·In(σ₂') if σ₂'≤200kPa 0.20 if σ₂'>200kPa	0.217
Silt	43.9	0.178	0.231	0.31-0.03·ln(σ₂') if σ₂'≤200kPa 0.15 if σ₂'>200kPa	0.227
Clay	54.3	0.230	0.164	0.21-0.01·ln(σ₂') if σ₂'≤200kPa 0.16 if σ₂'>200kPa	0.227

Table 3.27: Regression parameters (Brandenberg et al. 2010).

Figure 3.102 shows the relations among V_s , N_{60} , and σ_v ' from Brandenberg et al. (2010). These figures show clearly that the relation between V_s and N_{60} is not very strong, as reflected by the standard deviations in Table 3.27. This observation supports the conclusion that such correlations should not be used when a geophysical measurement can be reasonably obtained.

Soil Type	eta_0	eta_1	β_2
Sands	30	0.23	0.23
Clays & Silts	26	0.17	0.32
Gravels - Holocene	53	0.19	0.18
Gravels - Pleistocene	115	0.17	0.12

Table 3.28: SPT-stress-V_s correlation equations (Wair et al. 2012).

Wair et al. (2012) summarized various relations for relating V_s to SPT blow count and CPT tip resistance. Their recommendations for SPT blow count utilize the same functional form as Brandenberg et al. (2010), and the constants are provided in Table 3.28. These relations produce
different predictions of V_s relative to those by Brandenberg et al. (2010), which is a reflection of the inherent uncertainty in the correlations. Wair et al. (2012) did not quantify prediction errors in the same manner as Brandenberg et al. (2010).



Figure 3.102: Results of regression equations for V_s as a function of N_{60} and σ'_{ν} for (a) sand, (b) silt, and (c) clay, with trend lines corresponding to the mean and $\pm 1\sigma$ for σ_{ν}' and N_{60} (Brandenberg et al. 2010).

In addition to the SPT-based relations, Wair et al. (2010) also provided relations between V_s and CPT tip resistance, q_t or q_c , and sleeve friction, f_s , and overburden stress, σ_v' . They recommend computing V_s as the average value of equations provided by Mayne (2006), Andrus (2007), and

Robertson (2009). Details of these methods are not reproduced in this report for brevity, but are summarized by Wair et al. (2010).

Wair et al. (2012) suggest that site-specific relations between V_s and CPT data may be developed according to the functional form in Eq. 3-99, in which the constants β_0 , β_1 , β_2 , and β_3 must be obtained by regression of a known dataset containing both geophysical measurements and CPT data.

$$V_{s} = \beta_{0} q_{t}^{\beta_{1}} f_{s}^{\beta_{3}} \sigma_{v}^{\prime \beta_{2}}$$
(3-99)

Benefits of this procedure are that (i) much of the uncertainty in the correlation between V_s and penetration resistance is eliminated due to the site-specific calibration of the regression constants, and (ii) CPT soundings can be obtained rather quickly at many locations, permitting V_s to be estimated at multiple locations within a geological unit without having to make multiple geophysical measurements. Although this procedure may be reasonable at short separation distances within a single geological stratum, errors may arise at larger distances, or when geological conditions change within a site. More research is required to quantify these errors.

3.3.3 PROXIES FOR V₅₃₀

In many cases, geophysical measurements and penetration resistance measurements are not available at a site where V_{530} must be estimated. This is common for mapping applications, and is also the case at many strong ground motion recording stations. In these cases, V_{530} may be approximated from proxies that include geologic mapping, topographic slope, and/or terrain classes. These methods involve significant uncertainty, and should only be used when geophysical measurements and cannot reasonably be obtained at a particular site. Recent examples of proxy-based methods for estimating V_{530} are Yong (2016) for California, Seyhan et al. (2014) for the NGA-West 2 project, Parker et al. (2017) for Central and Eastern North America, and Ahdi et al. (2017) for the Pacific Northwest and Alaska. This report focuses on application of geophysical measurements to geotechnical problems. Therefore details of the proxy-based methods are omitted from the report, and readers are referred to other sources of literature for details of these models.

3.4 ESTIMATING V_S FROM LABORATORY TESTING

Under certain circumstances, field-based geophysical methods may provide less than ideal coverage or may suffer from limitations that prevent adequate estimates of V_s . For example, in surface wave methods such as MASW and SASW, the amount of uncertainty in $V_{\rm S}$ increases and the resolution capabilities decrease as the depth of investigation increases. Seismic refraction is incapable of resolving the V_s of a layer where a stiffness inversion exists (i.e., stiff over soft strata). Finally, the recorded wavefields from seismic reflection may be complex due to overlapping reflections from multiple soil layers and/or may suffer from poor signal to noise ratio. Borehole-based geophysical methods may be useful in such circumstances. However, as described in previous sections, borehole methods are not without their own limitations, particularly related to costs, coupling between the borehole wall and casing, and logistical constraints. Therefore, in a number of cases, laboratory testing may prove quite useful in overcoming these issues and estimating V_s and/or V_P . In the case of material damping, laboratory methods are well-suited as they allow testing in a controlled environment that better address the inherent difficulties in measuring damping in dry, saturated, or cemented soils (e.g., Toksoz et al. 1979). For example, laboratory conditions can reduce uncertainty in the effects of geometric attenuation and reflections/scattering due to heterogeneities present in the wave path.

3.4.1 RESONANT COLUMN TESTING

Resonant column testing (ASTM D4015) has been used for over 50 years to determine the relationship between shear modulus, material damping, and shear strain in soils. Richart et al. (1970) provides a good discussion of the early history regarding development of resonant column testing. It is the most commonly used laboratory apparatus for measuring the small-strain properties of soils (Kramer 1996). Much of the current ASTM standard for this test method is derived from the seminal work of Drnevich et al. (1978). In this test, a solid or hollow cylindrical soil sample is placed into what is typically a fixed-free apparatus (Fig. 3.103). The bottom of the specimen is affixed to a rigidly fixed base and the top of the specimen is affixed to a driving plate that applies a torsional input excitation. A sinusoidal torque with a range of excitation frequencies is typically applied to the specimen, though random noise (Amini et al. 1988) and impulse loading (Tawfiq et al. 1988) have also been employed previously. By measuring the response of the specimen to this input excitation in the time and frequency

domain, the resonant (i.e., fundamental) frequency is obtained (Fig. 3.104). The V_s of the specimen can then be estimated by solving the equation of wave motion in a prismatic rod (see Richart et al. 1970 or Kramer 1996 for details):

$$\frac{I}{I_o} = \frac{\omega_n h}{V_s} \tan \frac{\omega_n h}{V_s}$$
(3-100)

where *I* is the mass polar moment of inertia of the specimen, I_o the mass polar moment of inertia of the torsional loading system attached to the top of the specimen, *h* the height of the specimen, ω_n the fundamental angular frequency ($\omega_n = 2\pi f_n$), and V_s the shear wave velocity.



Figure 3.103: Schematic of a resonant column device (Drnevich et al. 2015).



Figure 3.104: Example of a soil specimen's frequency response from resonant column testing (Khan et al. 2008).

Assuming that testing is performed at very small strains, the V_s as computed from Eq. 3-100 can be used to represent the in-situ V_s and is related to the small strain shear modulus (G_{max}) based on Eq. 3-2. Damping is estimated from the frequency response curve using a half-power bandwidth approach or from a logarithmic decrement approach after subjecting the sample to free vibration (see Kramer 1996 for details on these methods). Once resonant response has been obtain at one value of torque, the torque is then adjusted (which changes the level of applied shear strain) and the specimen is again excited through a range sinusoidal excitations with different frequencies. Testing in this manner continues and allows estimates of shear modulus and damping at different shear strain levels (Fig. 3.105).



Figure 3.105: Example of shear modulus and damping ratio results from resonant column testing (Werden et al. 2013).

The resonant column test is capable of exciting the specimen at strain levels ranging from 10^{-5} % to 0.5%, which allows it to develop modulus reduction and damping curves for use in soil dynamics applications (e.g., earthquake engineering, traffic vibrations, machine foundations, etc.). It should also be noted that the apparatus can apply axial loading in a similar manner as the torsional loading, which yields estimates of the constrained modulus. However, resonant column testing is used so rarely in this manner that ASTM D4015 removed this discussion in the latest revision to the standard. The apparatus is often housed in a pressure chamber that allows testing to be performed at a range of confining stresses representative of various depths within a soil profile. A significant amount of literature exists regarding resonant column testing, including references related to developments in the methodology and/or available testing systems (e.g., Drnevich et al. 1978; Drnevich 1985; Avramidis and Saxena 1990; Cascante et al. 2003; Kumar and Clayton 2007), interpretation of results (e.g., Amini et al. 1988; Tawfiq et al. 1988; Cascante and Santamarina 1997; Ashlock et al. 2013; Werden et al. 2013), and different applications of the method (e.g., Prange 1981; Acar and El-Thahir 1986; Macari and Hoyos 1996; Kramer 2000; Hardin and Kalinski 2005; Senetakis et al. 2012; Castelli and Lentini 2017). Despite the strong history of this test method, its use is largely confined to academic settings where researchers explore the fundamental behavior of soils or to large critical projects (e.g., nuclear power plant project). Much of this can be attributed to the large costs and highly specialized nature of the equipment. Other, even more specialized testing equipment exists to measure shear modulus and damping [e.g., the Dual Specimen Direct Simple Shear (DSDSS) device as discussed in Doroudian and Vucetic (1995)]. However, the availability and costs of such equipment even further limit their applications to academic settings.

3.4.2 TRANSDUCERS AND BENDER ELEMENTS

In addition to resonant column testing, wave velocity has also been measured in the laboratory using transducers and/or bender elements (Fig. 3.106) in a through transmission setup (typically referred to ultrasonic pulse testing) (Fig. 3.107). In such cases, an incident wave is input into one side of a specimen with known geometry and the arrival time is recorded on the other side of the specimen. Care must be exercised regarding data interpretation as there is some ambiguity in which part of the received signal best represents the arrival time of the wave, particularly for shear waves (e.g., see Lee and Santamarina 2005a for discussion) (Fig. 3.108). The use of multiple reflections can aid this determination and allow for consistency in how travel time is

defined (Lee and Santamarina 2005a). Additionally, transducers and bender elements both exhibit a near field response, where the waves are not fully developed and signal amplitudes fluctuate spatially instead of decaying from geometric spreading (e.g., Lee and Santamarina 2005a; Lee and Santamarina 2005b) (Fig. 3.109).



Figure 3.106: Sensors for estimating V_s in the laboratory: (a) Transducers (www.olympusims.com); and (b) Bender elements (www.piezo.com).



Figure 3.107: Schematic and associated electronics for through transmission ultrasonic pulse testing with transducers/bender elements (Brignoli et al. 1996).



Figure 3.108: Typical S-wave signal and potential interpretations for arrival time: (A) first deflection; (B) first bump maximum; (C) zero after first bump; and (D) first major peak (Lee and Santamarina 2005a).



Figure 3.109: Near field effect on signal amplitude along the centerline of a typical P-wave transducer (Lee and Santamarina 2005b).

Once the arrival time is determined, the wave velocity (*V_P* or *V_s*) is then estimated by simply dividing the length of the specimen by the arrival time of the corresponding wave. Estimates of velocity using transducers and/or bender elements can be as much as 3% - 10% larger than those obtained in resonant column testing due to the different frequencies used during testing (Stokoe et al. 1994). Transducers and bender elements both rely on the phenomenon of piezoelectricity to accomplish these measurements. Piezoelectricity was first observed by Curie and Curie (1880) and refers to the voltage potential that occurs in certain ceramic materials due to an applied mechanical stress. It results from lack of symmetry in the crystalline structure of the ceramic or from the electrically polar nature of crystals (Lee and Santamarina 2005a). As the amount of crystal asymmetry increases, the piezoelectric effect increases as well, which leads to increasingly larger voltage output for a given applied mechanical stress. Moreover, this process works in the inverse direction, whereby an application of a voltage potential across the crystalline structure of the ceramic causes it to distort. The polarization direction of the crystal

changes the sign of the voltage output and, by extension, the direction of mechanical deformation.



Figure 3.110: Schematic of typical ultrasonic transducer (Lee and Santamarina 2005b).



Figure 3.111: Example of P-wave signal generated by S-wave transducers (Brignoli et al. 1996).

Transducers are composed of a piezoelectric element [e.g., lead zirconate titanate (PZT)], a backing block that controls the extent with which the piezoelectric element vibrates after excitation, and a matching layer that optimizes the energy transferred from the piezoelectric element to the medium (Fig. 3.110). Both V_S and V_P can be measured for the soil by employing transducers that generate shear waves and compression waves, respectively. Care must be exercised when interpreting the first time of arrival because spurious motions can be present, particularly for shear wave transducers where it is practically impossible to prevent some amount of compressive wave energy from being generated (Brignoli et al. 1996) (Fig. 3.111).

Additionally, the source and receiver transducers should be separated by at least two wavelengths to reduce near-field effects, particularly in V_s measurements (Sanchez-Salinero et al. 1986).

Some of the earliest instances of measuring V_s using an ultrasonic pulse arrangement with shear wave transducers occurred in the 1960s (e.g., Lawrence 1963; Nacci and Taylor 1967; Sheeran et al. 1967). Since that time, its usage has been more widely adopted (e.g., Stephenson 1978; Cockaerts and De Cooman 1994; Brignoli et al. 1996; Nakagawa et al. 1996; Fioravante 2000; Inci et al. 2003). In many cases, suppliers of geotechnical testing equipment can provide modules for their equipment that contain transducer-based systems, including sensors, automated data acquisition systems, and other peripheral hardware for operation. However, based on recent publications, it appears that bender elements have become more popular than transducers for estimating V_s in soils.



Figure 3.112: Schematic representation of bender elements: (a) Typical components; (b) Series type wiring; and (c) Parallel type wiring (Brandenberg et al. 2006).

Bender elements consist of two conductive outer electrodes, two piezoceramic sheets between the electrodes, and a conductive metal shim at the center (Fig. 3.112). Depending on how the bender elements are wired, they can behave in "parallel" or "series" operational modes. For series type bender elements, the two piezoelectric layers are connected at the outer electrodes, which results in their poling directions being opposite to one another (Fig. 3.112b) Parallel type

bender elements have the voltage wire connected to the conductive metal shim and the outer electrodes share a common ground (Fig. 3.112c). This wiring results in the piezoelectric sheets sharing the same poling direction and twice the level of displacement for the same applied voltage. Given this behavior, parallel type bender elements are well suited to behave as a source of shear waves and series type bender elements as the receiver. Bender element testing to estimate wave velocity of laboratory samples was first introduced in the 1970s (Shirley and Anderson 1975; Shirley and Hampton 1978; Shirley 1978). In some ways, bender elements are preferable for this application as they tend to be smaller, operate at lower frequencies, and can more readily be adapted to interface with a typical triaxial, direct shear, and/or oedometer testing apparatus. Bender elements also develop larger deformations (and stronger input signals) for a given voltage excitation when compared to shear wave transducers (Brignoli et al. 1996). Moreover, shear wave transducers can suffer from limitations related to poor coupling, weak directivity, high operating frequency [typically in the ultrasonic range (>20 kHz)], and/or impedance mismatch (Lee and Santamarina 2005a). However, bender elements must be installed so that they penetrate the sample being tested, which may cause issues with stiffer soils and soils with large particles/aggregates (Brignoli et al. 1996). Since its inception in the 1970s, bender element testing has experienced increased rates of usage for estimating V_s in small-scale (e.g., triaxial, oedometer) and/or large-scale (e.g., centrifuge model) laboratory samples (e.g., Dyvik and Madshus 1985; Fam and Santamarina 1995; Jovičić et al. 1996; Pennington et al. 1997; Lee and Santamarina 2005a; Brandenberg et al. 2006; DeJong et al. 2006; Zhou and Chen 2007; Montoya et al. 2012; El-Sekelly et al. 2014). As with transducers, many geotechnical testing equipment suppliers offer bender element systems as add-on modules for testing equipment [typically for their strength testing equipment (e.g., triaxial apparatus)].

3.4.3 APPLICABILITY OF LABORATORY TESTING

Laboratory measurements of V_{s} , V_{P} , and/or damping may be desirable under certain circumstances related to the feasibility of field geophysical measurements. Generally speaking, measuring the in situ geophysical properties of a soil is preferable with field-based methods that are non-destructive since testing occurs at the same stress and drainage conditions. These approaches are more efficient and they avoid the limitations of sample disturbance that are inevitably present whenever drilling and sampling occur to procure laboratory specimens. In

fact, in many cases, laboratory samples may be reconstituted or subjected to remolding in an attempt to better replicate field density conditions at the expense of maintaining soil fabric/structure (Kramer 1996). Moreover, laboratory samples only represent a distinct depth and/or location within the site. This implies that laboratory measurements of geophysical parameters are likely to provide less overall information regarding general site conditions as the samples may only cover a limited area of the site. There is also always concern as to whether a given sample is truly representative of the strata of interest. Field based geophysical methods sample a much larger area and are likely to "average" local variations in geophysical properties that may highly affect a particular sample. Additionally, laboratory testing systems must be appropriately assembled. In the case of resonant column testing, this means that the apparatus must be appropriately calibrated and the specimen must be placed in a membrane while minimizing disturbance. For bender element or transducer testing, adequate coupling must be ensured between sensor and specimen and electrical coupling issues must be addressed with adequate shielding. Despite these issues, laboratory measurements do provide some advantages. For instance, conditions in the laboratory are much more controlled and data interpretation is often more straight forward. The desired dimensions for laboratory investigations are drastically reduced in scale to centimeters from the tens of meters necessary in the field, which leads to a corresponding increase in resolution. These effects often considerably reduce the amount of uncertainty present in geophysical laboratory measurements. However, this must always be balanced against the limitations related to sample disturbance and limited spatial coverage of the measurements as previously described.

3.5 APPLICATIONS RELATED TO NON-DESTRUCTIVE TESTING (NDT)

As previously noted in this document, there is significant overlap between geophysical methods as applied to earth materials and for non-destructive testing (NDT) purposes. Transportation agencies acknowledge the similarities between geophysical methods and NDT in practice (e.g., Fig. 1.4). In many cases, there is no formal distinction between geophysics and NDT. For the purposes of this document, geophysical methods measure the properties of natural earth materials and NDT evaluate properties of engineered materials (e.g., pavement, concrete, structural fills, etc.) as suggested by Wightman et al. (2003). The following sections briefly describe important applications of NDT for transportation infrastructure and their corresponding geophysical methods. A comprehensive discussion regarding NDT is beyond the

scope of this document and the reader is encouraged to review the references described in each section and other extensive literature on the topic (e.g., Wightman et al. 2003; Von Quintus et al. 2009).

3.5.1 QUALITY ASSURANCE OF PAVEMENT CONSTRUCTION

Nearly all state DOTs perform quality assurance on pavement construction by measuring density with coring samples and smoothness with profilographs (Cominsky et al. 1998; Von Quintus et al. 2009). NDT methods have seen increased usage for assessing the quality of hot mix asphalt (HMA) overlays and flexible pavement construction due to their ability to evaluate pavement material properties in a time-efficient manner with limited disruptions to traffic operations. Additionally, NDT methods can potentially address some of the challenges associated with properly evaluating the influx of new technologies in asphalt pavements (e.g., recycled products, binder additives, stone matrix asphalts, warm mix asphalts, etc.). Finally, NDT methods can be used to monitor long-term condition of existing pavements as a form of early stage deterioration detection (Wightman et al. 2003). In this manner, NDT can serve as a component of network-level pavement management efforts.

Typical NDT technologies for evaluation of pavement include deflectometers, GPR, impact echo, ultrasonic pulse velocity, infrared thermography, intelligent compactors, lasers, non-nuclear and nuclear density gauges, permeameters, and ultrasonic seismic devices (Tables 3.29 – 3.30). A detailed discussion of the operation of all these NDT technologies is outside the scope of this study on geophysics and earth material properties and the reader is encouraged to refer to multiple references available regarding NDT in pavements (e.g., Wightman et al. 2003; Von Quintus et al. 2009). However, one particular item of note is that many of the NDT methods share similar characteristics with geophysical testing methods previously highlighted in this document. For example, GPR as used for pavement assessment relies on higher frequency antenna to focus on the immediate near surface, but is otherwise identical in practice to GPR as used in geophysical studies. Many of the seismic wave systems (e.g., ultrasonic pulse velocity) rely on testing techniques that share many of its fundamental features with seismic reflection/refraction approaches. In fact, the overlap between the two areas has continuously increased as many of the recent development in seismic geophysical methods (e.g., SASW, MASW, etc.) have been simultaneously studied within the context of earth materials and

pavements. For example, since its inception in the 1990's, there has been growing interest in using MASW to evaluate the stiffness of pavements and underlying subgrades as an investigative quality assurance tool (e.g., Park et al. 2001; Ryden and Lowe 2004; Alzate-Diaz and Popovics 2009; Lin and Ashlock 2015). Given these links, it is expected that future new developments in geophysical methods will continue to improve the current state of practice for quality assurance of pavement materials.

		NDT Technologies and Methods				
Type of Property or Feature		HMA Layers	Unbound Aggregate Base and Soil Layers			
	Density	 GPR Non-Nuclear Gauges: PQI, PaveTracker 	 GPR Non-Nuclear Gauges; EDG, Purdue TDR 			
	Air Voids or Percent Compaction	 GPR Infrared Tomography Acoustic Emissions Roller-Mounted Density Devices 	 GPR Roller-Mounted Density Devices 			
Volumetric	Fluids Content	• GPR	 GPR Non-Nuclear Gauges; EDG, Purdue TDR 			
	Gradation; Segregation	GPRInfrared TomographyROSAN	NA			
	Voids in Mineral Aggregate	• GPR (Proprietary Method)	NA			
	Thickness	 GPR Ultrasonic; Impact Echo, SPA, SASW Magnetic Tomography 	GPRUltrasonic; SASW, SPA			
Structural	Modulus; Dynamic or Resilient	 Ultrasonic; PSPA, SASW Deflection-Based; FWD, LWD, Roller-Mounted Response Systems; Asphalt Manager 	 Impact/Penetration; DCP, Clegg Hammer Ultrasonic; DSPA, SPA, SASW Deflection-Based; FWD, LWD Steady-State Vibratory; GeoGauge Roller-Mounted Response Systems 			
	Bond/Adhesion Between Lifts	 Ultrasonic; SASW, Impulse Response Infrared Tomography 	NA			
Functional	Profile; IRI	 Profilograph, Profilometer, Inertial Profilers 	NA			
Functional	Noise	Noise Trailers	NA			
CDA Cult	Friction	CT Meter, ROSAN	NA			
SPA – Seism PSPA – Portt SASW – Spe LWD – Ligh ROSAN - RC EDG – Electr TDR – Time DSPA – Dirt PQI – Pavem DCP – Dynar CT – Circula	ic Pavement Analyzer able Seismic Pavement ctral Analysis of Surfa t Weight Deflectomete Dad Surface ANalyzer rical Density Gauge Domain Reflectometry Seismic Pavement An ent Quality Indicator mic Cone Penetromete r Texture	Analyzer ce Waves r d alyzer				

Table 3.29: Summary of NDT methods used to evaluate pavement properties for quality control (Von Quintus et al. 2009).

Attribute	Falling	Light	Ground	Impact	Infrared	Intelli-gent	Portable
	Weight	Weight	Penetrating	Echo	Thermo-	Compa-	Seismic
	Deflecto-	Deflecto-	Radar		graphy	ctor	Pavement
	meter	meter			015	(7)	Analyzer
(1)	(2)	(3)	(4)	(5)	(6)		(8)
Operational	Layer	Stiffness	Electro-	Stress	Rate of	Vibration	Ultrasonic
Principle	stiffness	under	magnetic	waves	heat	adjusted to	waves
	from static,	controlled	waves	propagate	radiation	measured	radiate
	vibratory, or	impulse	reflect to	material to	and emissi-	material	and detect
	impulse	loading.	measure	establish	vity.	stiffness.	material
	loading	-	material	resonance			properties.
Measures and	Moduli	Moduli	Thickness,	Layer	Temp.	Stiffness,	Moduli,
Indicators			density,	thickness,	segrega-	index,	thickness,
			moisture	delamina-	tion	compact-	moisture,
			content	tion		tion meter	delamin-
						value	ation
Test Equipment	Trailer- or	Hand-held	Vehicle,	Scanning	Infrared	Roller	Seismic
	vehicle-	device,	pulse,	unit,	camera,	compactors	source,
	mounted,	computer.	antenna(e),	computer.	sensor bar	ensing	receivers,
	computer.	_	computer.	_		equipment	computer
Portability	Fair	Excellent	Good	Excellent	Excellent	Good	Excellent
Complexity	Fair	Good	Fair	Good	Good	Good	Fair
Test Time	2 min	1 to 5 min	Cont.	1 min	Cont.	Cont.	45 sec
Environ-mental	Sensitive to	Sensitive	Sensitive to	Not	High temp.	Not	Pavement
limitations	pavement	to	wet surface	advisable	gradients	affected by	temp. of
	temp.	pavement	and/or layer	on elevated	are	surface	32 to 120
	_	temp.		temps	required.	moisture or	°F.
				_	_	temp.	
Reliability	Good	Fair	Good	Poor	Excellent	Excellent	Good
(see Table 2)							
ASTM test	D4695-08	E2583-07	D4748-06	C1383-04	D4788-07	None	STP 1375,
protocols					E1543-06		2000
Degree of	Poor	Good	Poor	Good	Good	Fair	Fair
training							
Purchase Price	\$100,000 -	\$20,000	\$50,000	\$30,000	\$4,000 -	\$175,000 -	\$30,000
	\$150,000				\$50,000	\$280,000	

Table 3.30: Summary of NDT methods for pavement evaluation, including information regarding costs, training needs, portability, etc. (Schmitt et al. 2013).

In addition to density and smoothness, critical field construction-related characteristics that influence flexible pavement quality, stability, and durability include mix segregation, in-place compaction, layer thickness, temperature segregation, layer interface bonding, and layer moduli (Schmitt et al. 2013) (Table 3.31). As previously noted, many state DOTs rely on laboratory tests on coring samples or on field tests that provide information regarding volumetric properties (e.g., density, presence of voids, thickness, etc.) and smoothness of the asphalt layer. The current Mechanistic-Empirical Pavement Design Guide (MEPDG) uses layer modulus as a key material property for structural design of flexible pavements. Layer modulus has also been demonstrated to be more objective for characterization of asphalt layers because it incorporates the effects of temperature and loading frequency on pavement performance (Nazarian et al. 2005). Therefore, there has been increasing interest in procedures to measure

the modulus of each pavement layer shortly after placement during construction (Celaya and Nazarian 2006). This shift from an empirical to a performance-based mechanistic design has partially driven the increased interest in NDT technologies as many can be used to estimate modulus (Table 3.29). However, it should be noted that Von Quintus et al. (2009) demonstrated significant differences in the field values of moduli measured using NDT when compared to results from laboratory tests on coring samples (Tables 3.32 – 3.33). However, the results generally correlated well with increases in laboratory measured moduli exhibiting similar increases in NDT measured moduli. It is expected that efforts will continue in the future to address the challenges in applying NDT methods and corresponding moduli measurements for quality assurance of asphalt pavements.

			Property Needed for:			
Pavement Layer	Ma	terial-Layer Property	Structural	Mixture	Acceptance	
	Dansity	Ain Voids at Construction	Design	Design		
	Density –	Air voids at Construction	Yes	res	N N	
	Voids in N	lineral Aggregate	Yes	Yes	N	
	Effective A	Asphalt Binder Content	Yes	Yes		
	Voids Fille	ed with Asphalt		Yes		
HMA Layers;	Gradation		Yes	Yes	\checkmark	
Mixtures	Asphalt Bi	nder Properties	Yes	Yes		
Witxtures	IDT Strength and Creep Compliance		Yes	Yes		
	Dynamic Modulus		Yes	Yes		
	Flow Time or Flow Number			Yes		
	Smoothness, Initial		Yes		\checkmark	
	Density		Yes	Yes	\checkmark	
	Water Content		Yes	Yes		
Unbound Layers;	Gradation		Yes	Yes	\checkmark	
Dense Graded	Minus 200 Material		Yes	Yes	\checkmark	
Granular Base,	Plasticity Index (Atterberg Limits)		Yes	Yes		
Embankment Soils	Resilient Modulus		Yes	Yes		
	Strength	CBR or R-Value	Yes	Yes		
		DCP; Penetration Rate	Yes			
IDT – Indirect Tensil	IDT – Indirect Tensile					
CBR – California Bearing Ratio						
DCP – Dynamic Cone Penetrometer						

Table 3.31: Summary of material properties used for design and acceptance of flexible pavements and HMA overlays (Von Quintus et al. 2009).

Project Identification		Resilient Moduli, ksi		Adjustment Ratios Relating Laboratory Moduli to NDT Values			
		Laboratory Measured Value	Predicted with LTPP Equations	Geo Gauge	DSPA	DCP	LWD
		Fine-Grain	ed Clay Soils				
I-85 Low-	Before IC Rolling	2.5	10.5	0.154	.0751	0.446	0.39
Plastic Soil	After IC Rolling	4.0	13.1	0.223	0.113	0.606	0.39
NCAT; OK	High Plastic Clay	6.9	19.7	0.266	0.166	0.802	
SH-21, TX	High Plastic Clay	26.8	19.6	1.170	0.989	3.045	2.78
	Average Rat	tios for Fine-Gra	ined Clay Soils	0.454	0.336	1.225	
	Emban	kment Materials	; Soil-Aggregate	Mixtures			
TH 22 MN	South Embankment	16.0	15.7	0.696	0.367	1.053	3.13
1 H -23, MIN	North Embankment	16.4	16.3	0.735	0.459	0.863	3.13
US-2, ND	Embankment	19.0	19.5	1.450	0.574	0.856	
SH-130, TX	Improved Soil	35.3	21.9	1.337	1.029	1.657	1.43
Average Ratios for Soil-Aggregate Mixtures;			Embankments	1.055	0.607	1.107	
		Aggregate I	Base Materials				
Co. 103, TX	Caliche Base		32.3	1.214		1.436	
NCAT, SC	Crushed Granite	14.3	36.1	0.947	0.156		
NCAT, MO	Crushed Limestone	19.2	40.9	0.747	0.198		
TH-23 MN	Crushed Stone, Middle	24.0	29.9	0.851	0.303	0.725	1.69
111-25, 1011	Crushed Stone, South	26.0	35.6	0.788	0.235	0.560	1.69
US-53, OH	Crushed Stone	27.5	38.3	1.170	0.449	0.862	
NCAT, FL	Limerock	28.6	28.1	0.574	0.324	0.619	
US-2, ND	Crushed Aggregate	32.4	39.8	1.884	0.623	1.129	
US-280, AL	Crushed Stone	48.4	49.3	1.010	0.244	0.962	1.04
Average Ratios for Aggregate Base Materials			1.021	0.316	0.899		
Overall Average Ratios for Processed Materials			0.942	0.422	1.084		
NOTES:							
1. The adjust	ment ratio is determined by d	lividing the resilien	t modulus measure	ed in the labor	ratory at a sp	ecific stress	state by
the NDT estimated modulus.							

2. The overall average values listed above exclude those for the fine-grained clay soils.

Table 3.32: Adjustments between moduli measured using laboratory methods and field NDT methods for unbound layers (Von Quintus et al. 2009).

Duciest/Mintune	Dynamic	Ratio or Adjustment Factor		
Project/whxture	Modulus, ksi	PSPA	FWD	
I-85 AL, SMA Overlay	250	1.055	0.556	
TH-23 MN, HMA Base	810	1.688	NA	
US-280 AL, HMA Base; Initial Area	650	1.407	3.939	
US-280 AL, HMA Base; Supplemental Area	780	1.398	2.516	
I-35/SH-130 TX, HMA Base	1,750	5.117	3.253	
I-75 MI, Dense-Graded Type 3-C	400	0.919	NA	
I-75 MI, Dense-Graded Type E-10	590	0.756	NA	
US-47 MO, Fine-Graded Surface	530	1.158	NA	
US-47 MO, Coarse-Graded Base Mix	420	0.694	NA	
I-20 TX, HMA Base, CMHB	340	0.799	NA	
US-53 OH, Coarse-Graded Base	850	1.275	NA	
US-2 ND, Coarse-Graded Base, PG58-28	510	1.482	NA	
NCAT AL, PG67 Base Mix	410	0.828	NA	
NCAT FL, PG67 Base Mix	390	0.872	NA	
NCAT FL, PG76 Base Mix	590	1.240	NA	
NCAT AL, PG76 with RAP and Sasobit	610	1.3760	NA	
NCAT AL, PG76 with RAP and SBS	640	1.352	NA	
NCAT AL, PG67 with RAP	450	0.881	NA	
Overa	1.128	2.566		

NOTES:

- 1. The adjustment factor or ratio was determined by dividing the dynamic modulus measured in the laboratory for the in-place temperature and at a loading frequency of 5 Hz by the modulus estimated with the NDT device.
- 2. The laboratory dynamic modulus values listed above are for a test temperature of a loading frequency of 5 Hz at the temperature of the mixture when the NDT was performed.
- 3. The overall average adjustment factor excludes the SH-130 mixture because it was found to be significantly different than any other mixture tested in the laboratory; which has been shaded.

Table 3.33: Adjustments between moduli measured using laboratory methods and field NDT methods for HMA layers (Von Quintus et al. 2009).

3.5.2 DETERMINATION OF CONCRETE CONDITION/INTEGRITY

The use of reinforced concrete is ubiquitous on transportation projects across a wide range of scales and applications (e.g., bridge decks, retaining walls, concrete pavements, foundations, etc.). This signifies that measuring the properties of concrete is an important aspect of quality assurance for many transportation related projects. The compressive strength of concrete is typically the property of interest when evaluating concrete structures, though other concrete properties such as air void content, surface roughness, density, and chloride content among others may be relevant depending on application. For new structures, a common approach for quality assurance is to simultaneously cast samples of the concrete used in the structure for future evaluation of compressive, flexural, and tensile strengths. There are a number of disadvantages to this approach, including the delay in availability of results (i.e., at least 28 days), differences in the concrete samples relative to the actual structure, and dependence of

concrete strength properties on sample size and shape (Bungey et al. 2006). In a similar manner to asphalt pavements, many DOTs also employ coring methods to retrieve samples of concrete as a quality assurance method during construction and as an investigative method on potentially problematic in-service concrete structures. However, this approach suffers from similar limitations related to inefficiency and the point source nature of such a measurement.

Given the aforementioned limitations, NDT methods have long been utilized to evaluate concrete properties. Though somewhat dated, Carino (1994) thoroughly reviews the historical development of NDT methods for evaluating concrete. The relationship between NDT results and the desired concrete property is typically indirect and a reliable correlation must be established. Discussion on all the available correlations between NDT measurements and concrete properties is outside the scope of this document, but the reader is referred to existing references that highlight these relationships (e.g., Malhotra and Carino 2004; Bungey et al. 2006). Since NDT methods only indirectly evaluate concrete properties, it has typically been advised that multiple NDT methods be used so that secondary measurements can mitigate the influence of uncertainty in a primary measurement (Breysse 2012; Sbartai et al. 2012). Additionally, NDT methods have often been combined with "semi-destructive" tests that may cause localized surface zone damage or require the removal of surface finishes (Bungey et al. 2006). Table 3.34 provides a list of various testing methods (including NDT, partially destructive, and destructive methods) available to evaluate concrete properties for quality assurance purposes. Generally, this list includes penetration tests, rebound tests, pull out tests, dynamic tests, and radioactive methods.

Property under investigation	Test	Equipment type
Corrosion of embedded	Half-cell potential Resistivity	Electrochemical Electrical
steel	Linear polarization resistance	Electrochemical
	AC Impedance	Electrochemical
	Cover depth	Electromagnetic
	Carbonation depth	Chemical/microscopic
	Chloride concentration	Chemical/electrical
Concrete quality,	Surface hardness	Mechanical
durability and	Ultrasonic pulse velocity	Electromechanical
deterioration	Radiography	Radioactive
	Radiometry	Radioactive
	Neutron absorption	Radioactive
	Relative humidity	Chemical/electronic
	Permeability	Hydraulic
	Absorption	Hydraulic
	Petrographic	Microscopic
	Sulfate content	Chemical
	Expansion	Mechanical
	Air content	Microscopic
	Cement type and content	Chemical/microscopic
	Abrasion resistance	Mechanical
Concrete strength	Cores	Mechanical
	Pull-out	Mechanical
	Pull-off	Mechanical
	Break-off	Mechanical
	Internal fracture	Mechanical
	Penetration resistance	Mechanical
	Maturity	Chemical/electrical
	Temperature-matched curing	Electrical/electronic
Integrity and	Tapping	Mechanical
performance	Pulse-echo	Mechanical/electronic
	Dynamic response	Mechanical/electronic
	Acoustic emission	Electronic
	Thermoluminescence	Chemical
	Thermography	Infrared
	Radar	Electromagnetic
	Reinforcement location	Electromagnetic
	Strain or crack measurement	Optical/mechanical/electrical
	Load test	Mechanical/electronic/ electrical

Table 3.34: List of principal testing methods to evaluate concrete properties (Bungey et al. 2006).



Figure 3.113: Example of correlations between concrete compressive strength and: (a) rebound hammer index; and (b) seismic velocity (Mikulic et al. 1992).

Many of the NDT methods highlighted as useful for pavement testing purposes in the previous section are either directly applicable to concrete or share many similarities with comparable concrete evaluation methods (e.g., GPR, impact echo, infrared thermography, etc.). Similar to previous discussions related to earth materials, electromagnetic and electrical NDT methods in concrete (e.g., GPR, ERI, etc.) are often used to evaluate the presence of anomalous features. They can establish the presence of voids, cracks, and delamination of steel for damage detection (Hugenschmidt and Mastrangelo 2006; Barnes et al. 2008) or determine water or chloride content for corrosion evaluation (Saleem et al. 1996; Sbartai et al. 2006; Sbartai et al. 2007). Radiation-based methods have been in use since at least the 1960s to assess density as either a proxy for strength or to locate the presence of voids and defects in concrete construction (Preiss 1965; Preiss and Caiserman 1975; Rucker 1990; Liebich 2004; Winters 2014). Given that the unconfined compressive strength of weaker rocks fall within range of typical concrete compressive strength, it is unsurprising that there is significant overlap between NDT/geophysical methods to measure these properties. The most common methods to evaluate concrete compressive strength include seismic NDT methods such as the ultrasonic pulse velocity (UPV) or pulse-echo techniques or partially destructive methods such as the Schmidt hammer (i.e., rebound hammer) (Breysse 2012) (Fig. 3.113). The results from these methods

(i.e., seismic velocity and rebound hammer index) correlate very well with concrete compressive strength. The correlations between seismic velocity and compressive strength of concrete use similar functional forms to those presented for UCS of rock. Breysse (2012) provides a good summary of these relationships, including a useful discussion on variability and uncertainty of the velocity measurements in typical seismic-based NDT methods for concrete integrity testing.

From a geotechnical perspective, the most likely encounter with concrete integrity testing using NDT is in quality assurance of cast-in-drilled-hole (CIDH) shaft foundations. Given the "blind" process and potential for construction defects, many state DOTs specify nondestructive testing of newly constructed CIDH foundations, particularly for shafts drilled and placed under wet construction conditions. A key step in acceptance of NDT techniques can be traced back to the Baker et al. (1993) FHWA report on cast in place foundations. Since that time, the field of NDT for foundation integrity has undergone tremendous technological advances. Many of the aforementioned NDT techniques from Table 3.34 can be used to evaluate the integrity of drilled shaft foundations. Generally, NDT for this application can be applied either at the ground surface or within inspection tubes installed with the rebar cage during shaft construction. Surface techniques such as the sonic echo (SE) test (i.e., impact echo test, pile integrity test) or impulse response (IR) test rely on inputs of stress waves applied to the top of the shaft and measurements of reflected wave energy in the time or frequency domain. However, these techniques can suffer from limitations related to uncertainty in size and location of any anomalies, particularly any toe defects since excessive attenuation of the stress wave may prevent reflections from the toe of long shafts (Iskander et al. 2001; Hertlein and Davis 2007). Down-hole methods such as cross-hole sonic logging (CSL), single-hole sonic logging (SSL), crosshole tomography (CT), gamma density logging (GDL) [also known as gamma-gamma logging (GGL)], sonic caliper, parallel seismic integrity testing (PSIT), and thermal integrity profiling (TIP) can address these limitations. CSL, SSL, and CT are essentially seismic-based tests with origins in geophysical borehole logging that have been repurposed for NDT applications. The PSIT method is similar to CSL except it relies on signals received in a borehole adjacent to the constructed shaft. Testing equipment and instrumentation for these methods can be slightly different than the geophysics counterparts to account for the testing conditions within a drilled shaft, but the methodologies are essentially unchanged. The sonic caliper approach uses acoustic waves generated within the excavation to estimate the shape of the excavation, assess verticality, and

estimate concrete volume prior to concrete placement. As previously highlighted, GDL measures the backscatter of gamma rays a set distance away from the emitter and the gamma counts are calibrated to material density. Significant reductions in density can then be used to identify anomalies from shaft construction. Caltrans specifies GDL as the primary NDT approach used in quality assurance of drilled shafts. The most recent addition to the suite of available NDT test is the TIP method, which relies on measuring the heat developed by the shaft during the concrete hydration period. Differences in the temperature profile relate to the shape of the shaft and alignment of the rebar cage (Mullins 2010). When combined with construction logs, the thermal results can be converted into effective radius measurements to detect anomalies across the entire cross section and evaluate alignment of the rebar cage (Winters and Mullins 2012). This is a consistent issue with both CSL and GDL as these methods cannot provide as extensive spatial coverage with their measurements of anomalous features. CSL can only really acquire information regarding the concrete in between the two access tubes and GDL only investigates a limited zone in the immediate vicinity of the access tube (Olson et al. 1998; Hertlein and Davis 2007).

	Length	Diameter	Strength	Durability	Serviceability Score
SET	2	1	2	1	1.50
IRT	2	1	2	1	1.50
CSL	3	1	2	1	1.75
SSL	3	-	1	1	1.25
СТ	3	1	2	1	1.75
GDL	3	-	2	2	1.75
TIP	3	2	1	2	2.00
Sonic Caliper*	3	2	-	-	1.25
PSIT	3	-	-	-	0.75

*Note: Measurements made prior to concrete placement, therefore not a direct measure of the as-built shaft.

Code: 3 = Direct Measurement; 2 = Indirect Measurement; 1 = Least Applicable Measurement; - = Not Applicable

Table 3.35: List of principal NDT testing methods to evaluate CIDH foundations and general assessments of their capabilities (Winters 2014).

As with pavement evaluation, each of these NDT method offer different advantages and limitations, which may affect selection of the appropriate technology based on anticipated shaft concerns (Table 3.35). The use of multiple methods is advised, particularly since uncertainty in the NDT measurements can lead to "false positives" where anomalous areas are detected in a well-constructed drilled shaft (Iskander et al. 2001; Hertlein and Davis 2007). Moreover, a single

NDT method may not perform well enough to adequately determine the extent of any anomalous features and additional testing may be necessary to evaluate whether the anomaly will have a negative effect on shaft capacity.

Continued on Next Page

4. CONCLUSIONS

The current state of geophysical practice allows a large amount of information to be determined regarding conditions at a site, including stratigraphy, ground water conditions, and the presence of subsurface anomalies. The preceding sections of this document also demonstrated the wide range of capabilities to estimate earth material properties using geophysical methods. As such, geophysical methods can provide a wealth of information to guide efforts in transportation projects, including foundation design, construction of earth retaining systems, and placement of embankments. In the case of seismic design and site characterization, seismic geophysical methods such as seismic refraction, SASW, MASW, and borehole methods provide higher quality information regarding *V*_s when compared to correlations with blowcounts. Additionally, many of the concepts central to various geophysical methods form the basis for a number of NDT techniques that are applicable to a number of highway related issues.

Though geophysical methods offer tremendous value for estimating earth material properties in transportation projects, they are by no means a magic bullet that can address all problems encountered in practice. Moreover, they are not meant to entirely replace standard drilling, sampling, and laboratory testing efforts on geo-related projects. The goal of any geoprofessional involved with subsurface characterization should be judicious application of geophysics as a cost effective approach to augment other exploration efforts. With that in mind, the purpose of this document was not to dwell on any one particular methodology and its limitations, but instead present in a concise manner the different relationships that exist in the literature between geophysical measurements and earth material properties. The reader is encouraged to seek further documentation on particular methods in appropriate general reference documents as previously described in this document (e.g., Sharma 1997; Wightman et al. 2003; Butler 2005; Dal Moro 2014). Without focusing on one particular method, however, it is important to discuss future needs and developments to address current limitations in various geophysical methods. The following sections provide this discussion. The goal is to provide the reader with general ideas regarding how the current state of practice can stand to improve and what research avenues are likely to be pursued to the benefit of the geophysical and transportation community.

4.1 FUTURE NEEDS AND DEVELOPMENTS

Though earth material properties can be measured using geophysical methods, the preceding sections also demonstrated that there are a number of limitations in all geophysical methods and that care must be exercised in their use. In many cases, these limitations present an area of need for academic research as well as the development of appropriate guidance documents. The incorporation of new technologies can also potentially address areas of weakness in the current state of geophysical practice. A detailed discussion of all potential research avenues is outside the scope of this document. However, the following sections discuss general trends related to geophysical research and developments as well as the role of guidance documents in increasing the prevalence of geophysical measurements for transportation projects. In addition to this discussion, it is interesting to review previous assessments in the literature regarding the state of practice in near surface geophysical measurements. The reader is encouraged to review such sources (e.g., Dobecki and Romig 1985; Steeples 2001) to highlight the rapid pace in advancements over the last several decades.

4.1.1 GUIDANCE DOCUMENTATION AND TRAINING

Putting aside the limitations of various geophysical methods, one of the key areas of need regarding the use of geophysics to benefit transportation projects is additional guidance documentation and training. In many cases, there is a general disconnect between geophysics and geotechnical engineering since geophysical methods are not routinely addressed in typical civil engineering curricula or in professional development opportunities. This means that those responsible for making engineering decisions regarding geotechnical aspects of transportation projects have not developed an appropriate level of comfort with geophysical methods. As previously highlighted in the rationale for this document, very little exists in the literature that is explicitly written for an audience with a cursory understanding of most geophysical methods but a particular need for guidance regarding their measurements. Most sources of information are either too introductory in nature or aimed at experienced geophysical practitioners looking to explore special topics not routinely incorporated into typical engineering problems. Guidance documents such as ASTM standards provide general information regarding best practices. However, information about different geophysical methods are scattered across multiple references and they often do not provide the level of detail necessary to understand how the measurements are applied in engineering practice. This document is intended to provide a

snapshot of this current state of practice. It is anticipated that future efforts will need to update or develop similar reference documents as new technologies develop and/or new capabilities are acquired. There is also no universal approach that works for all applications of geophysics in transportation projects because each deployment of a geophysical technique and each site condition are unique. This highlights the need for reliable sources that document case histories and discuss successful implementation of geophysics under a wide range of conditions in transportation projects. However, is also important that researchers and practitioners document unsuccessful applications of geophysical methods in the literature. Those who work with geophysical methods must understand not only the circumstances that favor their use but also those that may hinder their use. Unsuccessful case studies can often prove to be just as valuable in that context because they help to establish limits on the various geophysical techniques. Finally, continued efforts must be made to ensure that the aforementioned products reach the practicing community via the development of webinars and in-person seminars/trainings/professional development opportunities.

4.1.2 IMPROVEMENTS IN ANALYTICAL AND DATA INTERPRETATION METHODS

Future efforts should also focus on research to address current limitations in many geophysical methods. This will likely require new developments in both analytical and interpretation capabilities as well as advances in technology. In many cases, the uncertainty in geophysical measurements can stem from issues related to how the data is analyzed or interpreted. For example, measurements in seismic refraction tomography, SASW, MASW, and ERI (and even GPR in some cases) are subjected to inversion algorithms that attempt to match theoretical models of the subsurface to the measured signals. These inversion procedures are often inherently ill-posed, nonlinear, and mix-determined, which means that a unique solution is not possible with the acquired data. A-priori information can help constrain the inversion and improve the potential for a unique accurate solution. However, a-priori information is not always available. Issues with non-uniqueness of the inversion are also possible even when more complex approaches are used to match the data, such as the full waveform inversion technique for MASW, SASW, and GPR (e.g., Virieux and Operto 2009; Busch et al. 2012). The inversion is typically performed using an optimization algorithm (often a linearized least-squares approach) to locate the best fit between theoretical and experimental data. The initial starting model heavily influences the results In the case of linearized local approaches (e.g., Socco et al. 2010).

As a result recent research efforts have concentrated on global search methods such as uniform Monte-Carlo, genetic algorithm, simulated annealing, and neighborhood algorithms (e.g., Foti et al. 2009; Godio 2016; Jiang et al. 2016) that search a broad parameter space and avoid the problem of getting stuck in local minima. These algorithms must still search for models within a predetermined inversion parameter space. This means that the search parameters and the entire space of possible solution profiles must be defined in advance. This task is not trivial because the parameters cannot be overly broad or else the inversion will pursue entirely unrealistic models. However, too many constraints on the global inversion will neglect potentially viable solutions. Therefore, appropriately defining the parameter space for global inversion algorithms represents an area of need related to data analysis that warrants future research efforts. Some work in this area has been initiated within the context of surface wave inversions (e.g., Cox and Teague 2016). Other studies have explored the use of novel inversion techniques with other geophysical data such as electrical resistivity and seismic data (e.g., Zhou et al. 2014; Sabeti et al. 2017). Since so many geophysical methods inherently rely on the solution of an inversion problem, it is expected and necessary that future continual refinements occur in this broad area.

Other topics with similar research needs related to data analysis and interpretation include the following:

• Combining multiple datasets:

One manner to counteract the issues of non-uniqueness during inversion is to simultaneously invert multiple datasets [e.g., simultaneous inversion of P- and Rayleigh wave data (Boiero and Socco 2014), P-wave and microgravity measurements (Coutant et al. 2012); P-wave and GPR data (Al-Shuhail and Adetunji 2016); P-wave and resistivity data (Gallardo and Meju 2003); resistivity and radar data (Linde et al. 2006); etc.]. The purpose of joint inversion is to develop one objective function for optimization based on the individual objective functions representing each of the data sets. In this manner, joint inversion can reduce the number of acceptable models and can produce mutually consistent estimates of the various unknown parameters because the results must explain all data simultaneously (Julia et al. 2000). Different measurements have different capabilities (e.g., resolution, sensitivity, etc.) and the incompatibilities for one type of data can often be resolved by

another (Julia et al. 2000). Additionally, noise sources and their impacts on data quality often differ between methods so that adding another method can improve the results more than adding more data of the same method. However, additional research is needed to aid in identifying the most appropriate coupling strategies for joint inversion of various geophysical data types (e.g., direct parameter relationship, cross-gradient approach, etc.). In addition to using multiple geophysical datasets in joint inversion, geotechnical site investigation information (i.e., stratigraphy, penetration resistance) can also be incorporated into inversions of surface measurements. Penetration resistance measurements do not suffer a reduction in resolution with depth, and therefore present an interesting possibility for enhancing the resolution of joint inversions.

• Use of higher dimensional studies:

As computational power has increased and instrumentation/deployment costs have decreased, there has been a growing shift towards implementing higher dimensional geophysical surveys, including fully 3D surveys (e.g., Friedel et al. 2006; Radzevicius 2008; Loke et al. 2014; Wang et al. 2015) and/or incorporating time as a fourth dimension (e.g., Abdelwahab et al. 2011; Chambers et al. 2014). Future studies will likely continue to exploit the additional information offered about site conditions from these surveys. However, the analytical techniques currently employed for simpler surveys must be revised to account for the multi-dimensionality of the problem. This is a non-trivial task as the extra dimension(s) can introduce another layer of uncertainty in data processing and considerably increase the computation time. Additional efforts are necessary to continue optimizing analytical efforts when geophysical testing is extended into higher dimensions.

Geospatial representation:

Geophysical studies can generate a large amount of spatially variable data. This is actually an important advantage of geophysics over the standard laboratory/drilling approach to subsurface characterization. Even though geophysical measurements may contain more uncertainty and are less direct than other measurement types, the bulk amount of data obtained allows better quantification regarding the level of uncertainty. Statistically, there is also increased likelihood that the quantity of measurements from geophysics may allow estimates of earth material properties to regress towards their means. However, accurate interpretation of this data is affected by the manner in which it is represented and any statistical analyses employed for that purpose. Given this link, interest in geospatial representation and analysis of geophysical data has increased. In more instances, geophysical measurements are processed within a geospatial analytical framework using computer-based geographic information system (GIS) tools either in isolation (e.g., Chik and Taohidul Islam 2013) or in combination with other sources of information [e.g., remote sensing data (Rashid et al. 2012), geotechnical data (Ball et al. 2015), geochemical data (Moura et al. 2012), etc.]. Additional work is required in this area to explore the most effective geospatial analysis techniques for processing geophysical data and to develop documentation regarding best practices. This is particularly the case when attempting to account for uncertainty and variability in site conditions as reflected in the geophysical measurements.

Automation efforts to aid in data interpretation:

Geophysical data must of course be interpreted to provide information regarding site conditions and earth material properties. Similar to geotechnical boring data, this step is not trivial as there is significant subjectivity involved at multiple stages. Decisions must be made regarding intermediate steps in the data post-processing that can highly influence the corresponding results from the analysis. For example, the acquired dispersion image in MASW must be interpreted to determine what constitutes the fundamental mode and what represents higher mode partitioning in order to identify appropriate phase velocities and obtain accurate $V_{\rm S}$ profiles in MASW method. This may be quite complicated in situations where "mode-kissing" can occur due to interference of different modes near certain frequency points (Gao et al. 2016). Similar discrepancies can occur in the analytical procedures for other geophysical methods (e.g., picking first arrivals in seismic refraction and borehole seismic/radar methods). Additionally, once the final results from data processing have been derived, there can be ambiguity with what conclusions are supported by these results. As computational power has improved, additional support from approaches using intelligent systems has been pursued to potentially improve data interpretation and remove some of the subjectivity. For example, a number of studies have explored the use of machine learning, artificial neural networks, and genetic algorithms to classify objects in results from GPR, seismic methods, and well logging methods (e.g., Pasolli

et al. 2008; Pasolli et al. 2009; Ayala-Cabrera et al. 2011; Singh 2011; Braeuer and Bauer 2015; Sabeti et al. 2017). These methods and similar concepts may see further refinements and be applied to other geophysical techniques in future studies. Of course, care must be exercised with these automation efforts to ensure they are used to complement and not eliminate the role of user experience.

4.1.3 TECHNOLOGICAL IMPROVEMENTS

Many limitations in geophysical results arise from issues related to data analysis and interpretation as previously described. These often hinder the motivation to use geophysical methods for situations in which they may prove useful. However, even if all analytical issues related to geophysical methods were to be addressed in the future, there are always limitations related to hardware, testing methodology, and deployment capabilities that can be addressed. Technological advances in these areas have the potential to improve the capabilities of various geophysical methods and increase their appropriate usage in transportation projects. For example, improvements in speed of data acquisition, equipment and deployment costs, and sophistication of sensors can all remove impediments to successful usage of geophysics for engineering purposes.

One area that has seen constant technological improvements has been in the area of hardware, equipment, and instrumentation. Breakthroughs in instrumentation and computer-processing techniques have greatly improved the capabilities of various geophysical methods. For example, the increase in computational power for computers has driven the rise in the more complex analytical approaches previously described. Additionally, sensors are now capable of much higher resolution as analog-to-digital converters have improved. New sensor technologies have been developed, including non-planted sensors for different geophysical testing methods [e.g., seismic methods (Pugin et al. 2004); electrical resistivity (Kuras et al. 2006)] and fiber optic sensors (e.g., Daley et al. 2013; Munn et al. 2017). Multicomponent instrumentation has become more cost effective and readily available. Data acquisition systems have correspondingly enjoyed the benefits of greater number of channels, storing larger amounts of data, and/or wirelessly streaming data. It is anticipated that instrumentation technology will continue experiencing breakthroughs in computational capabilities, sensor miniaturization, reductions in power requirements, improvements in battery technologies, data telemetry

capabilities, and developments of novel sensor technologies. The effects of these efforts are very likely to include improvements in data quality, increased productivity, and reductions in costs, which should continue to make geophysics an attractive approach for transportation projects. Technological hardware improvements in the future may also allow geophysics to provide useful information in previously uncharted transportation applications.

Another technological area that is currently receiving research interest in the field of geophysics and will likely continue to do so in the future is the application of robotics such as unmanned aerial vehicles (UAV) (i.e., drones), unmanned ground vehicles (UGV), and remotely operated underwater vehicles (ROV). For example, a recent SAGEEP conference included an entire session devoted to the use of UAVs in geophysics given their rise in prominence (Zamudio et al. 2016). Generally, geophysical applications with robotics have focused on integrating instrumentation such as magnetic, electromagnetic, and radar sensors (e.g., Arcone et al. 2016; Gavazzi et al. 2016; Eröss et al. 2017). However, there is has been recent interest in also automating the acquisition of seismic data using UAVs in particular (e.g., Sudarshan et al. 2016). Finally, it should be noted that remote sensing techniques as described in this document and the use of unmanned robotics are complementary and a number of studies have begun to merge these two technologies together (e.g., Watts et al. 2012; Colomina and Molina 2014; Lee et al. 2016). The use of unmanned robotics presents a number of advantages over traditional survey methods. For example, the need for manual labor in acquiring data is reduced, which can potentially reduce safety issues and costs, particularly for remote and difficult to access sites. Moreover, the automation capabilities allow for temporal consistency in repeated measurements at the same site. In some engineering applications, such temporal measurements are necessary to evaluate how earth material properties change with time. Given these advantages, the use of robotics presents a potentially viable alternative to address future needs in geophysical data acquisition for engineering purposes.

4.1.4 EFFORTS RELATED TO EARTH MATERIAL PROPERTIES

Previous sections in this document highlighted several relationships between the measurements from various geophysical methods and earth material properties. However, in a number of cases, these correlations were highly empirical and suffered from a large amount of scatter. Additionally, there are some earth material properties relevant to transportation projects that

should affect geophysical measurements for which relationships have not been directly proposed. Future research efforts should explore refinements in existing relationships and the development of new correlations to earth material properties. In that manner, future geophysical testing may be able to contribute to transportation projects in an even more effective manner.

In terms of existing relationships discussed in this document, improvements may be realized by continued development of case histories and/or laboratory studies where the corresponding earth properties are correlated to their respective geophysical measurement. For example, the effects of clay content/mineralogy on electrical and electromagnetic geophysical measurements were previously noted in this document (e.g., Abu-Hassanein et al. 1996; Shevnin et al. 2007; De Benedetto et al. 2012; Long et al. 2012; Tosti et al. 2013). This link was also exploited to explore the use of these methods to predict Atterberg Limits of soils (Abu-Hassanein et al. 1996; Giao et al. 2003; Bryson 2005; Kibria 2011; Long et al. 2012; Bery 2014). However, in many cases, a distinct functional form for these relationships was either elusive or significantly affected by scatter in the data. Some of this can be partly explained by the emphasis on field geophysical measurements and the general lack of comprehensive testing under highly controlled laboratory environments. Alternatively, improvements may be realized by technological advancements that fundamentally enhance measurement capabilities in electrical and electromagnetic geophysical measurements. These efforts may lead to more effective correlations between clay content information and electrical/electromagnetic measurements, as well as other relationships between geophysical measurements and earth material properties. However, even with technological improvements and additional case studies, some scatter will always exist in the data because geophysical measurements are influenced by multiple factors related to the earth materials. Therefore, future research efforts should consider the natural variability in earth material properties and provide some information regarding confidence in the proposed relationships. An example of such an approach was previously noted in Uzielli et al. (2013) where the proposed ϕ_{ρ} -V_s relationship was examined within a probabilistic framework, allowing for the development of probabilities of non-exceedance when applying the relationship.

It is also expected that future efforts may explore the development of new relationships between earth material properties and geophysical measurements. Generally speaking,

geophysical measurements will respond to factors related to the manner in which individual particles are in contact (i.e., soil/rock structure). Some of these factors have either been explicitly discussed in this document (e.g., porosity) or implicitly incorporated in some relationships (e.g., texture). However, in a number of cases, relationships that are reasonably expected to exist between geophysical measurements and earth material properties have not been developed. For example, links between various geophysical measurements and the following earth material properties have not been formally developed: soil type; gradation; angularity; relative density; stress history (i.e., OCR); sensitivity; swell/shrink potential; and undrained shear strength ratio (i.e., S_{u}/σ'_{vo}). As before, refinements may be necessary in either the available technologies or in data analysis/interpretation to improve confidence in the geophysical measurements necessary to develop such relationships. For example, given the current state of practice, there may just be too much uncertainty in seismic measurements to develop a link between seismic velocity and stress history as articulated in the OCR of a soil. In some cases, the appropriate datasets may be available but have yet to be analyzed to specifically isolate the effects of some of the aforementioned soil/rock properties. Finally, the difficulty in isolating the effects of particular soil/rock properties may necessitate the development of dedicated datasets from field case studies or specialized laboratory testing.

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