Ohio River Valley Soils Seminar XLIX

Tools for Assessing Geotechnical Site Conditions

Thursday, November 29, 2018
Embassy Suites by Hilton
801 Newtown Pike
Lexington, Kentucky

49th Annual
Ohio River Valley Soils Seminar

Thursday, November 29, 2018
Lexington, KY 40511

PROGRAM

6:30–7:20 am	Exhibitor Registration and Setup
7:20–8:00 am	Registration
8:00–8:10 am	**Opening Remarks**
8:10–8:50 am	Michael J Marasa, PE – Hayward Baker *"Bridge the Gap"*
8:50–9:30 am	Enrique Farfan, PhD, PE – Stantec *"To Code or Not to Code"*
9:30–10:10 am	Mark T. Bowers, PhD, PE – University of Cincinnati *"The Who, What, When, Where, Why and How of Teaching Students About Innovations in Geotechnical Site Characterization"*
10:10–10:25 am	**Break (Visit Exhibitor's Hall)**
10:25–11:05 am	Raghava A. Bhamidipati – Consulting Services Inc. *"Estimating the Collapse Potential of Gypsum Sands Using Shear Wave Velocity Testing"*
11:05–11:45 am	Zhenming Wang, PhD, PE, et al. – KGS, University of Kentucky "*Shear-Wave Velocity Database for Communities along the Ohio River Valley"*
11:45 am–12:00 pm	**Break (Visit Exhibitor's Hall)**
12:00–12:55 pm	**Lunch and Door Prize Drawings**
12:55–1:05 pm	Keynote Introduction: Ben Webster, PE – Stantec
1:05–2:00 pm	Keynote: George Filz, PhD, PE, Dist.M. ASCE – Virginia Tech University *"Geotechnical Engineering at the Kennedy Space Center"*
2:00–2:40 pm	Michael Kalinski, PhD, PE – University of Kentucky *"Use of Geophysical Methods for Geotechnical Site Characterization"*
2:40–3:20 pm	Jordan Keeney – Stantec *"Using Unmanned Aerial Systems (UAS) to Remotely Assess Landslide Events in Near Real-Time"*
3:20–3:35 pm	**Break (Visit Exhibitor's Hall)**
3:35–4:15 pm	Mohammadhasan Sasar – Purdue University *"Geotechnical site characterization of a talus slope for an un-conventional deep excavation project around a 21 Story Building"*
4:15–4:55 pm	Michael Bartosek and Don Fuller, II, PE – Stantec *"Innovative Applications of Digital Kriging Software Solutions to Challenging Geotechnical Site Conditions"*
4:55-5:00 pm	**Closing Remarks and Door Prize Drawing**

Proceedings of the 49th Annual Ohio River Valley Soils Seminar

Tools for Assessing Geotechnical Site Conditions

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1801 Newtown Pike
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With Support From:

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BRIDGE THE GAP[1]

A Look at *Typical* Geotechnical Report Content and Geotechnical Construction Design Requirements

Over the last quarter of a century, the geotechnical contracting industry has experienced explosive growth. Many options for ground improvement, earth retention and deep foundations have been introduced and now enjoy widespread use. And new technologies are continuing to be developed. The rapidly expanding geotechnical specialty techniques are outpacing the educational system leaving a gap between full understanding of many of the techniques and, consequently, the preferred subsurface characterization information to enable an efficient, economical design. This presentation will discuss the content of 'typical' reports, the design requirements of more common specialty techniques and thoughts on "bridging the gap". A few brief case studies may be included to illustrate the issues.

The presentation will be interactive and, therefore, the content cannot be completely predicted in advance. However, during discussions across the country, the issue seems to remain the same in a majority of geotechnical construction projects. That is that often, geotechnical studies will recommend a ground improvement technique or a specialty foundation system with including all of the engineering parameters needed for design of the technique. For example, while it might seem simple and logical that vibro densification could be effective in loose sands, the ability to achieve adequate densification is heavily influenced by the percentage of fines smaller than the #200 sieve. Therefore, laboratory grain size analyses are essential to the determination of the applicability of that technique. Similarly, ground improvement options such as aggregate piers are designed based on the ratio of the soil modulus to stone modulus. In the majority of reports reviewed by the presenter, there is a distinct lack of any data that could be used to calculate a soil modulus value.

It is reasonable to assume that responsibility for "the gap" falls on both the engineer and the contractor. To look a little more closely at these reasons, or perhaps excuses, a few thoughts will be presented for each discipline below.

Geotechnical firms are challenged with the task of evaluating a site for proposed construction very early in the design phase. Many times, the proposed construction is conceptual and will change several times before construction. Buildings can be shifted on the site, stories can be added or deleted, floor elevations can be raised or lowered, building uses can be modified and so on. All of these things mean that the recommendations are aimed at an obscure and moving target. Further, because the studies are performed so early in the project development, often the early design fees are out-of-pocket expenses to the developer and hence strongly encouraged to be as low as possible and usually

[1]Michael J. Marasa, PE, Senior Engineer, Hayward Baker

competitively solicited. Hence, budgets are restricted, and both exploration and laboratory testing is minimized.

As the report writing time arrives, the engineer must develop recommendations for the safe and economical construction of the planned project. Often, the economies derived in the geotechnical aspects of the project involve implementation of “modern” technologies. It isn’t until this point, that preliminary identification of specialty techniques is identified and suggested. It is also at this point the gap begins to widen. Every specialty geotechnical technique is designed using the engineering properties of the subsurface materials. Each technique uses variations of the characteristics of the underlying soil and rock that are the most influential in the performance of the specialty system. In many cases, the required parameters are not included in the geotechnical report.

The geotechnical specialty contractor faces a number of challenges in the process as well including price pressure during competitive bidding. The plans, specifications and subsurface information are generally received only a few weeks prior to final pricing. Designs are performed that are nearly final in order to properly price the project. While the project details are much more highly developed, the geotechnical data has not been updated. It is at this point that the project information and the bid price are resolved through risk management. The specific geotechnical parameters applicable to a specific technique are often assumed from the skeletal data contained in the report balanced with construction experience in similar settings. During the bid design and pricing phase, risk management involves evaluating as many of the potential changes from report to reality and applying price factors to the scenarios. Ultimately, the bid amount is determined as a balance between the anticipated low bid number and the cushion needed to cover the downside risk. Usually, the conservatism in the design and the price for the work increases as the quality of the information decreases.

Clearly, the “gap” has causes created by both the engineer and the contractor. The challenge ahead is finding ways to bridge the gap. These could include such things as participating in continuing education classes in the specialty techniques that provide sufficient detail to train consultants about the detailed design considerations of each technique. Engineering schools should introduce more specialty design in the classroom. Contractors should be available for in-house training such as lunch and learns. Changes in the procurement of construction services could also help to bridge the gap. Early involvement of the geotechnical contractor can aid in the vetting of specialty techniques and identify deficiencies in the exploration and laboratory data very early in the design process. Design-build and other teaming approaches also involve the consultants and the contractors resulting in better information and more efficient designs.

Perhaps the discussion during the presentation will identify the components to begin to build the bridge.

NOTES

TO CODE OR NOT TO CODE[1]

Abstract:
The development of computer programs such as programming languages, spreadsheets, CAD, and word processors promised more efficient ways to analyze and resolve numerical problems. In the early years, several users were required to write a computer program to perform numerical and scientific computations; commercial software were limited.

The 21st Century has brought the era of Big Data and our society has been overwhelmed by the task of processing all the information that is generated. The civil engineers are not exempt from dealing with large amounts of data that many times can not fit in a single spreadsheet. From monitoring data to field data, civil engineers can take advantage of tools and methods that are generated in other fields to process and evaluate data.

This paper provides an overview of different examples where codes were developed to analyze soil data and solve different geotechnical engineering problems. Also, recommendations are provided for tools that are available for the use in the geotechnical engineering field.

1. Introduction

The use of personal computers in the office became affordable during the 1980s, and engineers were very eager to assimilate them in their everyday tasks. The development of computer programs such as programming languages, spreadsheets, CAD, and word processors promised more efficient ways to analyze and solve numerical problems. The generation of reports and computer-generated image data were also impacted by this new technology. In the early years, users were required to write a computer program to perform numerical and scientific computations.

Computers have evolved dramatically in the last 20 years following Moore's law. Computer popularity and dependency have increased to the point to be essential to the most important aspects of our lives. As the computer's capacity increased, commercial computer applications become more available to facilitate the numerical modeling and data processing of different problems in the civil engineering field.

For civil engineers, the necessity of writing a computer program (*coding*) has decreased with the years, and they have limited themselves mainly to a set of tools such as Microsoft Excel, Mathcad, and MATLAB to solve interactive problems.
The 21st Century has brought the era of the Big Data and our society has been overwhelmed by the task of processing all the information that is generated. The civil engineers are not exempt from dealing with a large amount of data that in many instances cannot fit in a single spreadsheet. From monitoring data

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to field data, civil engineers can take advantage of tools and methods that are applied in other fields to process and evaluate large data sets.

Missed opportunities arise when we become only end-users of the commercial computer applications, which could limit our capacity to fully use one of the most important tools in human history to analyze and solve the present engineering challenges.

Managing engineers that opt to develop their solution through *coding*, requires implementing strategies to ensure that the analysis reaches the objectives and time constraints. Civil engineers can use the current strategies employed in the computer science field to manage the process of developing computer programs.

The advancement of Artificial Intelligent (AI) systems and its applications in solving engineering problems will force future generations to assimilate the art of *coding*. Redundant tasks will be assigned to AI systems, which potentially will replace entirely many of the tasks by which our younger engineers and other professionals learn the details of our profession. The new generation will be required to be involved in the developing and maintenance of these systems to gain some of the experience of our trades.

The question to *code* or not to *code* is not a question to be resolved in the near future, but rather a question that requires to be answered in our present.

2. Developing Computer Programs in Civil Engineering

A computer program (or *code*) is a set of instructions that can be executed on a computer, while an application is a computer program with a user-interface that directly helps a user perform different tasks. In the civil engineering field, the programmers have a mission, to develop a tool to solve a determined problem. The client's demands that shape the development of computer programs in computer science, morph in civil engineering into the supervisor's demands for accuracy of the solution and mechanism to check the calculations. The quality of a user interface with the computer program is placed in a secondary priority.

The development of computer software requires an organized strategy for carrying out the steps in the life cycle of the computer program development. The strategy is required to be predictable, efficient, and repeatable.

In computer science, several strategies are applied in the development of computer software. Some of these strategies reflect the relationship between programmer and client (end-user). A summary of these strategies is presented below:

Build-and-fix
This methodology relies on the end-user feedback to debug and improve the computer application. The result of this methodology is numerous free upgrades and a rapid proliferation of new versions. Requires a faithful and

patient end-user. This methodology is not suitable to develop computer programs.

Waterfall
The waterfall design methodology was developed in the 1970's and was a great step forward in software development. The model relies on a single-level feedback path and requires that the requirements are perfectly addressed before following to the design creation. All the steps, prior to the creation of the computer program, are also required to be perfectly implemented. This method is more suitable for small computer programs.

Rapid prototyping
This development process produces a program that performs some essential or perhaps typical set of functions for the final product. The computer program develops based on feedback and requirements. This methodology does not require pre-planning which expedites writing the code. This method has been used in the development of one-off programs.

Extreme programming
This process is based on the end-user requirements and the development is made by incremental steps based on the selected features envisioned by the client. This model is akin to the rapid prototyping that generates continuously a new prototype with certain functionality in place for critical business reasons.

Spiral
The spiral model could be considered as an enhancement of the waterfall and rapid prototype models. A risk analysis is performed at each phase of the cascade. It is similar to the rapid prototyping model represented in the form of a spiral. This method allows software to be reused and depends on controlling all factors and eliminating or minimizing the influence of external factors.

Incremental
The incremental model is a variant of the waterfall and rapid prototyping models. The objective is to provide an operational-quality system at each development stage.

OOP (Object-Oriented Programming)
The main idea of this methodology is encapsulation and polymorphism, with the main objective to reduce complexity and increase program re-usability. The style of object-oriented computer programming differs from the traditional procedural programming because the development of computer programs is performed by slices rather than by layers.

Iterative
This methodology reflects a more realistic aspect of the computer programming process. This method is based on a continuous feedback between the beginning and the end of each stage. Also, feedback is provided between several stages when the version is well-developed.

In civil engineering, a combination between an OOP (Figure 1.a) and Iterative (Figure 1.b) methodologies could be considered appropriate strategies. Working with an OOP strategy the team could ensure that different parts of the program can be reused in the future. The creation of classes should be preferred over the creation of functions. The Iterative methodology will provide the framework for the civil engineer to develop parts of the program that can be tested. As the program evolves over time, new features are added from a partially complete system to a complete computer program. Each incremental development allows performing a quality control of the program, using intermediate outputs.

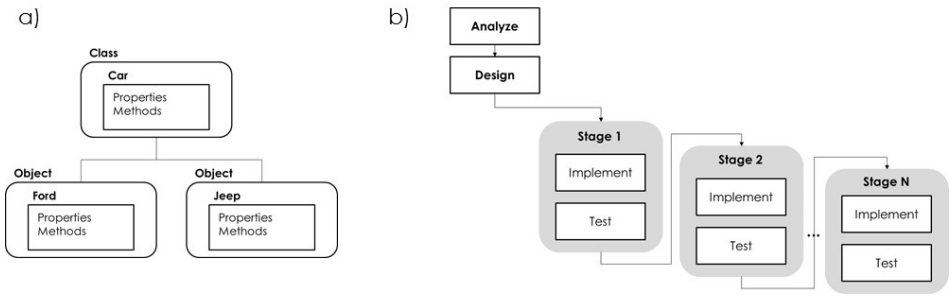

Figure 1. *a) Representation of a Class in OOP b) Iterative Methodology*

3. Available tools

There are several available programming languages to be used by civil engineers. The most popular programming languages in 2018 per IEEE Spectrum's fifth annual ranking are:

1. Python
2. C++
3. C
4. Java
5. C#
6. R
9. Go

Python has been the preferred programming language for many applications such as YouTube, Dropbox, Google, and some of the most recognized programs used in the civil engineering field such as Abaqus, ArcGIS, and Rhinoceros 3.0. The computer program R is one of the most common programs for statistical computing used by engineers and scientists.

While R can be downloaded directly from the its own webpage (https://www.r-project.org/), it is advised to download Python from some of the distributions that are available on the internet such as Anaconda, Enthought, ActiveState, and Python(X,Y). Nevertheless, for the most advanced computer user, python can be downloaded and compiled with the desired modules from different sites on the web (https://www.python.org/).

The advantage of using a python distribution is that most useful modules are already compiled and tested. There is a degree of stability in the installation, but like everything that is ready-made, limitations arise.

4. Licenses

The number of programing languages and applications available online are countless. Many of them operate under very restrictive licenses and others operate under free software concepts. In the early 1980s, Richard Stallman initiated the free software movement to counter the move of big computer corporations to protect their software under copyright licenses. Civil engineers have at their disposition several computer programs that operate under the concept of free software. In this section a summary on license types are provided.

Public Domain
Under this license there is no ownership or patent and the software can be modified, distributed or sold without the need of attribution by anyone.

Non-Protective Free and Open-Source Software (FOSS) License
Also called BSD-style license, provides the framework for free software licenses, where redistribution of the software can be done with minimum restrictions. MIT Software Licensing is one of the best-known licenses under this category.

Protective FOSS Licenses
Also known as a copyleft license, this license is a copyright license where the author surrenders some but not all the rights of the software. The main objective of using this type of license is to control different aspects such as software distribution, documentation, and art. This license allows the software reproduction and modification. GNU/GPL General Public License is an example of this popular license. Under the GNU/GPL license programs such as Python and R are distributed.

Freeware / Shareware / Freemium
Freeware allows the use of the software without a charge, examples of software under this license are Skype and Adobe Acrobat Reader.

The Shareware license provides the framework that allows sharing the software without restrictions but the source code is restricted to the user.

Freemium provides the license for the free distribution of a software with limited capabilities, but if additional features are required, they can be purchased.

Proprietary Licenses
The software copyrights are protected. This license includes all commercial licenses. Examples include AutoCAD, SAP2000, ArcGIS, and MATLAB.

5. Recommended Programs

There are several programming languages available under the GNU/GPL General Public License that civil engineers can use in their everyday computing and data processing.

Python
Python is one of the most popular programming languages. Python scientific modules make this programming languages a very efficient tool to compute and analyze data.

Mysql/SQLite
Mysql is a database management system. A version of this software named Community Edition is distributed under a GPL license. An alternative to Mysql is SQLite which is also a relational database management system with the big difference being, it is not a client-server database. Both databases can be linked to Python, which provides great capability in dealing with big data sets. SQLite is distributed under a public domain license.

R
A powerful free software for statistical computing and graphics. This software is distributed under a GPL license.

Grasshopper 3D
This software requires Rhinoceros 3D to be installed, which is a commercial 3D computer graphics program. Grasshopper is a visual programming language that runs along Rhinoceros 3D application and it is used to build parametric analysis by developing generative algorithms. This software is distributed under a freeware license.

6. Managing the Programmer

Many managers in the civil engineering field see the task of analyzing or processing data as a continuous process, where the data is manipulated along with the development of the analysis. This traditional analysis is supported by tools such as spreadsheets. Although spreadsheets provide a visual and intuitive manner to analyze data, spreadsheets present limitations when operating with big data sets, complex equations with many variables, and most of all challenges, copying and pasting data or specifying big sets of data.

Traditional analysis can provide managers with a sense of control over the data processing, especially for those managers that are not familiar with developing computer programs. Unfamiliar territory can discourage managers from opting for solutions that could involve *coding*. One of the factors that affect many managers is the sense that data is not processed and the resources are redirected to *coding* efforts. As deadlines approach and no data has been processed, an unfamiliar manager with *coding* can experience an antagonistic feeling toward the team that opted for this type of analysis.

Figure 2 illustrates the development of data processing over time for the traditional and the *coding* options. While in the traditional analysis the data processing progresses with time. In the *coding* option the data is processed in a fraction of time. In many cases, the *coding* option could represent savings in time and therefore in resources. The anxiety of a manager arises when the process of *coding* could require more time to arrive at a solution, which represents a loss of resources.

During *coding*, losses in resources often happen when "scientific curiosity" dominates the team. The programmer relies on past experiences and the review of current references. The internet provides a great source of information on the latest tools and trends. Review of new material could lead programmers to try new modules, which could affect productivity.

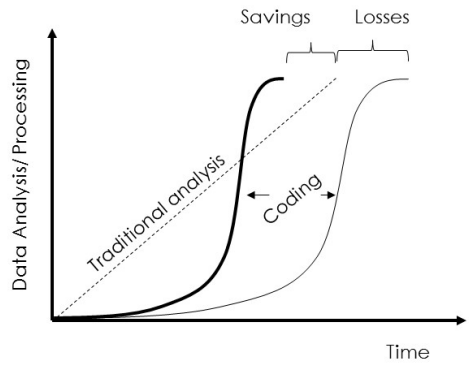

***Figure 2**. Traditional data analysis vs programming (coding).*

The process of developing a computer program for data analysis consist of several stages. The developing of a computer program can be summarized in six stages as follows:

1. Problem Analysis
2. Conception
3. Specification
4. Programming
5. Test
6. Implementation

The manager could control the process of developing a solution by writing a computer program (*coding*) establishing the time frames for every stage of the process. Rauterberg and Strohm (1992) provide in general the mean effort required at different stages of the process. It is important that the initial stages be extensively discussed by the manager, so the programmer can establish a clear path to move forward. Giving time for innovation when possible is also important and can be shared between the time dedicated to developing the specifications and the programming.

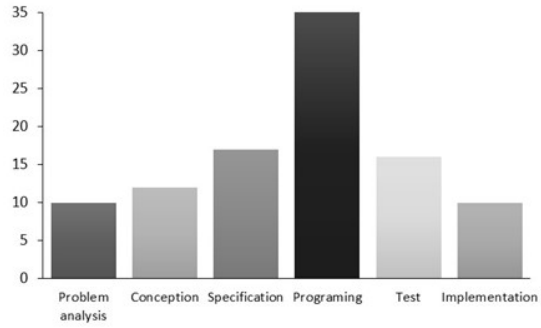

***Figure 3.** Mean effort in the different steps of software development process (Rauterberg and Strohm, 1992)*

The programming strategy based on an iterative methodology (see Figure 1) allows the manager to follow the progress of the programmer. The manager is

not required to be familiar with the *coding* process if meaningful intermediate outputs are set at every iteration. The objective should be that there is enough intermediate data that the data operations can be clearly reviewed. Also, the use of graphical outputs can help to give a general sense of the value range of the outputs.

For testing the computer software, it is recommended that a defined data set with known simple values and outcomes be used for testing the computer program. Also, a subset of the data can also be used to check the accuracy of the program when the solution can be compared to hand calculations.

One important advantage of python over a spreadsheet is that equations in python can be easily reviewed, compared with equations in a spreadsheet that requires a major effort to follow the links and operations. The following equation and its implementation in a spreadsheet, and in python are shown below for comparison.

$$I_8 = \frac{2}{\pi}\left[\frac{m'_1 n'_1}{\sqrt{1+m'^2_1+n'^2_1}}\,\frac{1+m'^2_1+2n'^2_1}{(1+n'^2_1)(m'^2_1+n'^2_1)} + sin^{-1}\frac{m'_1}{\sqrt{m'^2_1+n'^2_1}\sqrt{1+n'^2_1}}\right]$$

Equation in a spreadsheet:
=2/PI()*((($K$16*I26)/(1+$K$16^2+I26^2)^0.5)*((1+$K$16^2+2*I26^2)/((1+I26^2)*($K$16^2+I26^2)))+ASIN($K$16/(($K$16^2+I26^2)^0.5*(1+I26^2)^0.5)))

Equation in Python:
I8 = 2./math.pi * (((m1*n1) / (1+m1**2+n1**2)**0.5) * (1+m1**2+2*n1**2) / ((1+n**2) * (m1**2+n1**2)) + math.asin ((m1 / (m1**2+n1**2)**0.5 * (1+n1**2)**0.5)

7. Examples

In the following section, a brief description is provided to illustrate the potential of coding in civil engineering.

Plotting soil data

At the initial phase of analyzing the soil data for a design-build (DB) proposal, project data was provided as pdf documents and spreadsheets. Soil data in gINT format was not readily available. Something that is not unusual in Request for Proposals (RFPs) for DB projects. Soil data was extracted by digitizing the data from the reference documentation provided to the proponents.

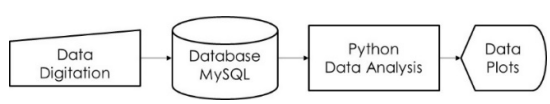

Figure 4. *Flow diagram soil data analysis*

Digitized data was uploaded to a database (SQL). A computer program using Python was developed to access and query the database. The data was organized using different tables that contain data such as boring name, location (coordinates), Standard Penetration Test (SPT) blow-counts data, SPT depth, soil description, soil laboratory data by sample location, cone penetration test data and field test data such as pocket penetration tests. The computer program was able to retrieve, process and plot the analyzed data (see Figure 4 **Error! Reference source not found.**).

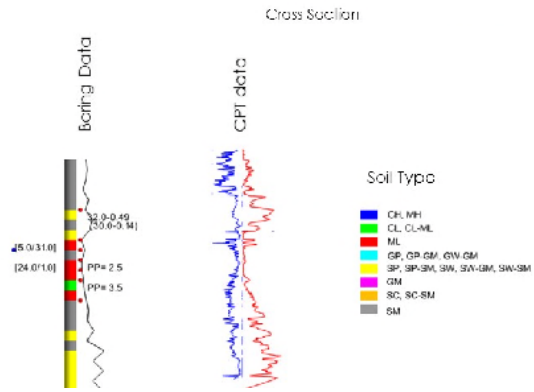

Figure 5. *Soil data analysis and data plots, cross section generation.*

The computer program was able to generate plots of the soil data effectively that supported the design during the bid preparation. Some of these plots generated by Python are illustrated in Figure 5.

Pile uplift calculations

According to AASHTO, uplift resistance of pile groups shall be evaluated when the foundation is subject to uplift loads. The pile group is considered as a block in the calculations of the uplift resistance. The geometry of the block for cohesionless soil is described in AASHTO (see Figure 6). The calculation of the volume of the block is only a geometric problem. A program using Grasshopper in the Rhinoceros 3D platform was created to handle the calculation of the weight of the block also considering a groundwater table. The program was able to generate the weight of block for a series of foundations with different configurations. Due to the evolution of the design several pile foundation configurations were developed. The program saved a considerable amount of time compared to other methods to calculate the weight of the block. The input parameters were the pile diameters, pile length, pile spacing and groundwater table location. In addition to providing the volume and weight of the block, the *code* generated an isometric image showing the piles and the block of soil, and a wire view of the block with the dimensions for easy reference and reporting.

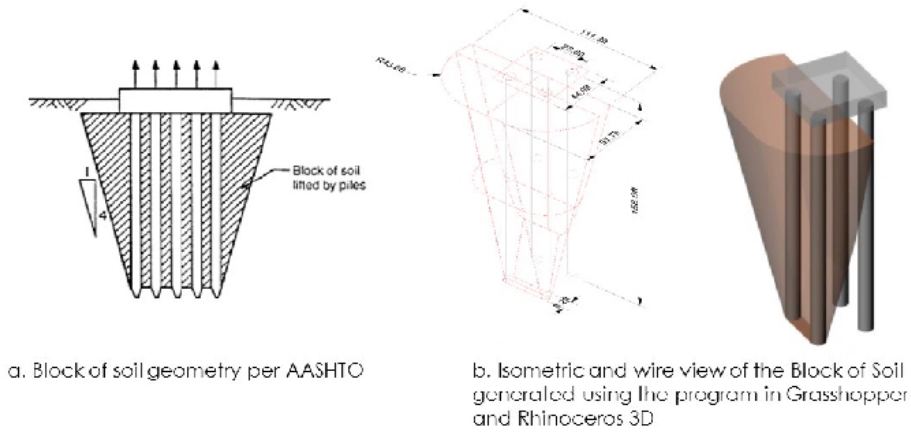

***Figure 6**. Pile group uplift calculation using Grasshopper (Rhino 3D)*

Pile analysis and design

This project required the design of more than 1,500 piles that supported a very complex building structure. The design not only comprised the calculation of the pile capacity but also the structural design of the piles. The time for developing the pile design became very limited since most of the available design time was utilized to calculate the pile loads by using a very intricate finite element model for the structure.
The pile loads were stored in a database to be retrieved by the computer program to compute the pile load combinations. ASCE-7 provides seven basic combinations for the allowable strength design (ASD) and eight combinations for load and resistance factor design (LRFD). This resulted in 32 load cases for ASD and 20 load cases for LRFD. Also, ASCE-7 requires that effects of one or more loads not acting shall be investigated. Part of the computer program evaluated all possible load combinations. The maximum and minimum loads for each pile were stored in the database.

***Figure 7.** Pile class and piling 3D representation*

The program was able to generate statistical plots on the pile loads, calculated required pile lengths and several other plots that assisted with the design of the piles. Using the plots generated with the computer program, it was possible to find and correct discrepancies in the pile loads.

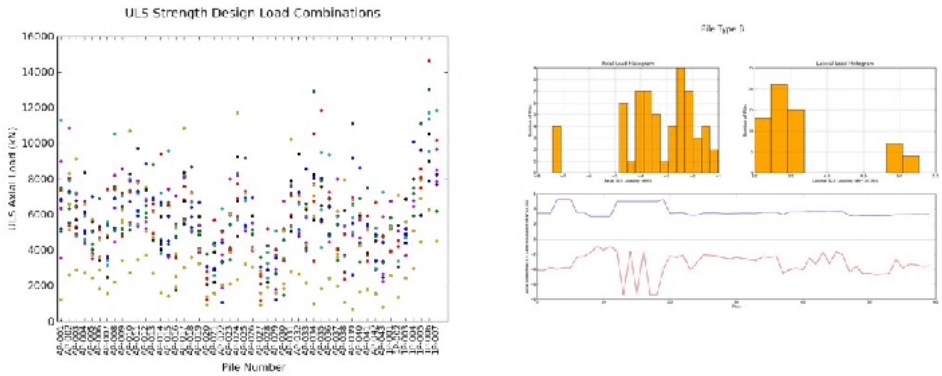

Figure 8. *Pile Load Analysis*

The commercial computer program L-Pile (Ensoft) was used to develop the relationships between lateral load and maximum shear, maximum moment, and maximum displacement for different pile lengths. A polynomial equation was used to curve fit the data points for each relationship. The set of polynomial equations was programmed into the computer program to determine the maximum forces and maximum displacement for different load conditions. The computer program provided a plot (pile number vs load) where all the load combinations per pile were represented for easy inspection.

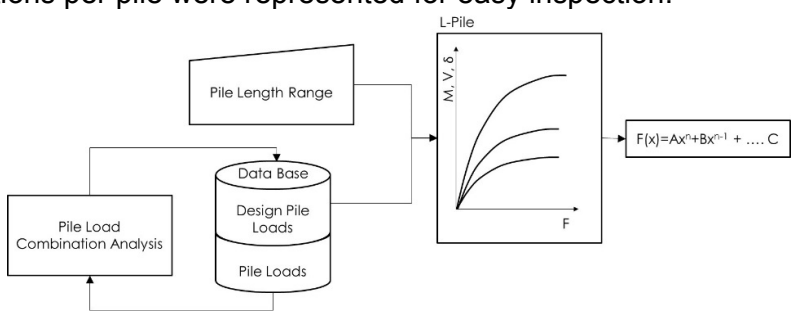

Figure 9. *Flow Diagram, Pile Lateral Response Parametrization*

The computer program was able to output in a spreadsheet format, several intermediate outputs to allow an independent review.

Plot soil strength

In some cases, the stability analysis of a critical slope, requires the modeling of several soil layers and the use of bilinear strength models. Visualizing the location of zones with lower strengths and the compatibility of assigned strengths is not easy without a graphic representation (see Figure 10). A computer program was developed to plot the stresses developed in the cross section and the strengths of the soil using a heat map allowing the inspection of soil properties along the cross section.

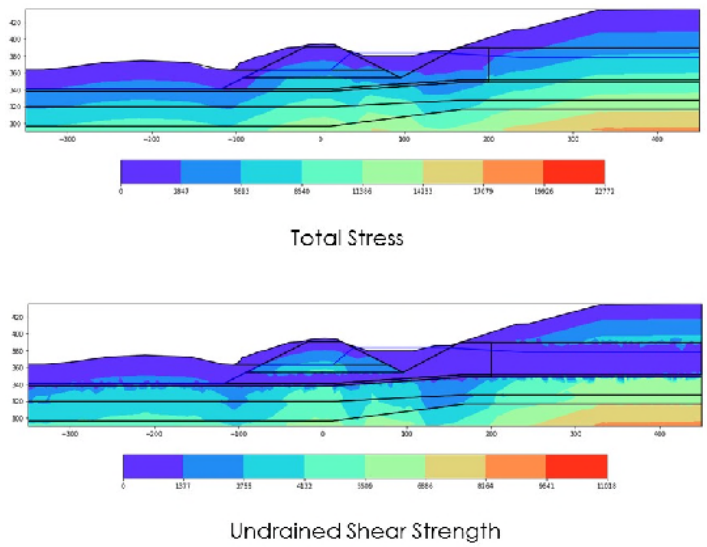

Figure 10. *Soil Strength Analysis*

Analyzing weather data for water balance modeling

The developing of water balance models requires statistical analysis of weather data. The computer program R was used to calculate the parameters for the probability density function to represent weather data for the analysis. A program using R was created to extract the relevant information from a text file and calculate the Weibull density parameters. In addition, the program generates two plots to illustrate the accuracy of the calculated distribution as shown in Figure 11.

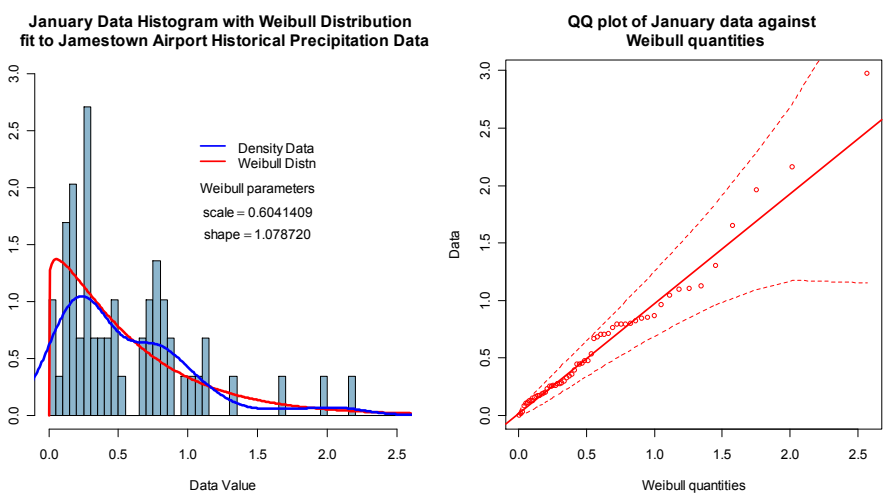

Figure 11. *Weather Data Analysis*

7. Conclusions

Computer programming has been paralleled to literacy, since it is considered a generalizable skill that should be taught to new generations. Universities and schools, in all levels have incorporated computer programing classes in their curricula. In civil engineering courses, programing also is taught, and is mainly related to graduate and research activities. Only a few civil engineers can use their coding skills in the professional world.

Knowing how to code is like knowing a second language, it needs to be practiced regularly to be proficient and efficient in that language. It cannot be expected that the team or engineer can jump into coding occasionally. Rather it is important to provide continuous opportunities for the programmer to maintain and develop their skills.

The application of AI and Machine Learning in civil engineering are at the door steps and it will require that we use this resource to solve the present and future challenges in the geotechnical engineering field.

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The Who, What, When, Where, Why and How of Teaching Students about Innovations in Geotechnical Site Characterization

Mark T. Bowers, PhD, PE

Abstract: This paper discusses many questions about teaching students about innovations in geotechnical site characterization. It is a personal reflection of a professor who has been working in the geotechnical field for nearly 42 years and teaching at the university level for 35 years. It discusses the author's reflections and recommendations on who to teach, who should teach, what to teach, when to teach the material in the curriculum, where the material could be taught (classroom, field, lab), why this subject is important, and how to continue learning after the degree has been earned.

Introduction: The theme of this seminar is "Tools for Assessing Geotechnical Site Conditions." The program is broad and of great use to practicing Geotechnical Engineers. As a professor I want to share this information in an appropriate way with my undergraduate and graduate students. This effort led me to the topic of my presentation, that is, "The Who, What, When, Where, Why and How of Teaching Students about Innovations in Geotechnical Site Characterization." This is my personal reflection on the matter after working in the geotechnical field for the past 42 years and teaching for 35 years.

Who? I believe this question is two-fold. First, "who" should be taught and second, "who" should teach this material? I will address the first question on "who should be taught" now and I will address the second question "who should teach this material" later in my presentation.

It is important that our graduating Civil Engineering students have an appreciation for a Soils Exploration Report. Information about this report is usually discussed in a first-level course on Soil Mechanics taken in the Junior year. The coverage of soil exploration is usually brief (perhaps two hours) as a broad swath of the subject of soil mechanics is covered in just one semester. The majority of our seniors in Civil Engineering follow the taking of Soil Mechanics with a one semester course in Foundation Engineering. The majority of the students (I have fifty students enrolled this Fall) take this class to meet an ABET Design Proficiency in Geotechnical Engineering, not that they are going to pursue a career in Geotechnical Engineering. Whereas I end the instruction of Soil Mechanics with a presentation on soil exploration, I begin Foundation Engineering with a review of soil exploration and then extend that conversation.

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What? What material should we teach as we introduce soil exploration? My instruction is aimed at the development of a soils report and its use by structural engineers, architects, and construction engineers in the creation of geotechnical works be they earth dams, foundations for buildings, earth retention systems, or ground improvement projects. As listed by Das and Sobhan (Principles of Geotechnical Engineering, 9th edition, Cengage Learning, 2018),

> Any soil exploration report should contain the following information:
>
> 1. Scope of investigation.
> 2. General description of the proposed structure for which the exploration has been conducted.
> 3) Geologic conditions of the site.
> 4) Drainage facilities at the site.
> 5) Details of boring [I would amend to say "and/or cone penetration tests"].
> 6) Description of subsoil conditions as determined from the soil and rock samples collected.
> 7) Groundwater table as observed from the boreholes.
> 8) Details of foundation recommendations and alternatives.
> 9) Any anticipated construction problems.
> 10) Limitations of the investigation.

In summarizing their chapter on subsoil exploration, Das and Sobhan (Principles of Geotechnical Engineering, 9th edition, Cengage Learning, 2018) note the following:

> 1) Soil exploration planning involves compilation of existing information, reconnaissance, and detailed site investigation.
> 2) Borings are generally made with continuous flight augers. Rotary drilling, wash boring, and percussion drilling are other methods of advancing a bore hole.
> 3) Soil samples during boring can be obtained by standard split-spoon sampler, thin-wall tube, and piston sampler.
> 4) Standard penetration resistance can be correlated with unconfined compression strength of cohesive soils. In granular soil, it can be correlated to relative density and friction angle.
> 5) Other in situ tests are vane shear test, pressuremeter test, and cone penetration test.
> 6) Rock coring is done by attaching a core barrel to the drilling rod. A coring bit is attached to the bottom of the core barrel. Recovery ratio and rock quality designation are parameters to evaluate the quality of rock.

For the students who go on to take Foundation Engineering, Das and Sivakugan (Principles of Foundation Engineering, 9th edition, Cengage

Learning, 2019) add the following tools in the soil exploration work:

1) Dilatometer test.
2) Iowa Borehole Shear Test.
3) K_o Stepped-blade Test.
4) Geophysical Exploration
    a) Seismic Refraction Surveys.
    b) Cross-hole Seismic Surveys.
    c) Resistivity Surveys,

Again, our lecture time is limited on this subject matter to about three hours in the course on Foundation Engineering.

When? I believe it is appropriate and effective to end the first course in Soil Mechanics with two to three hours of presentation on field investigation and the writing of the soil exploration report and then continue that discussion in the subsequent undergraduate course on Foundation Engineering. I prefer to make these presentations using my prior consulting projects as case histories.

Who should receive further instruction on soil characterization? I believe it would be a better use of time to focus this discussion on those who have chosen Geotechnical Engineering for their graduate studies. For example, a graduate course in advanced field and laboratory testing of soils would be useful. Useful references for such a course might include the following:

1) Geotechnical Investigations, EM 1110-1-1804, U.S. Army Corps of Engineers Design Manual.

2) Laboratory Soils Testing, EM 1110-2-1906, U.S. Army Corps of Engineers Design Manual.

How might a graduate course be structured? I believe in hands-on instruction. **Where might we teach**? I think it would be most useful to introduce advanced topics in site characterization in the classroom and then get into the field. I believe it would be advantageous to develop partnerships with geotechnical consulting firms where we could visit their labs to see the best of modern technology and testing equipment and, where possible, take graduate students in Geotechnical Engineering to field testing sites to see state-of-the-art equipment in action. I would further propose that, if possible, data sets be shared that the students can take back to campus to digest, contemplate over, and determine field properties. The classes wherein I learned the most were those that incorporated field trips and assignments that used actual field and laboratory from real jobs. I was blessed in that my professor for undergraduate Soil Mechanics had his own consulting firm. From him I took the following courses during my BS and MS studies:

1) Soil Mechanics (used Soil Mechanics by Lambe and Whitman)
2) Soil Mechanics Laboratory (used Soil Properties and Their Measurement by Bowles)
3) Foundation Engineering (used Foundation Analysis and Design by Bowles)
4) Earth and Rock-fill Dams (used Earth and Earth-rock Dams by Sherard et al.)
5) Advanced Soil Mechanics (used Principles of Soil Mechanics by Scott and Fundamentals of Theoretical Soil Mechanics by Harr)
6) Advanced Foundation Engineering (used Foundation Engineering edited by Leonards)

Our homework assignments involved data from recent jobs at his firm. We were expected to take what we had learned from our reading and his lectures and develop soil exploration plans, laboratory testing plans, bearing capacity charts, and recommended foundation schemes. We examined site data from proposed schools, warehouses, churches, hospitals, and manufacturing facilities. We completed settlement analyses for additions to a steel mill, a paint factory, and a La-Z-Boy furniture manufacturing facility. Most memorably we designed an earth dam including a grout curtain. Those classes prepared me for a lifetime of work in the Geotechnical Engineering field.

So, back to "**who**" but this time **who should teach this material**? Ideally the professor has some practical experience. If not, then the professor should be encouraged to develop strong relationships with his/her peers in the geotechnical consulting arena. Again, I have been blessed as a professor in having strong support from the local geotechnical community. These practicing engineers have been willing to give guest lectures on the following topics:

1) Slope stability problems and remedial action
2) Deep foundation design and implementation
3) Pile-supported mat foundations
4) Soil nailing
5) Braced excavation design
6) Earth retention systems
7) Seepage problems in dam foundations and their repair
8) Earthquake damage to building foundations

I acknowledge the recent invitation we received to attend the ADSC (Association of Drilled Shaft Contractors) Drill Rig Operator School held in mid-September in Cincinnati at Goettle's facility. We were told that approximately 40 operators from around the country would be learning and honing their skills on drill rigs from five different manufacturers. We were invited to participate in demonstrations of different drilling techniques–open hole, cased holes, and slurry holes. We were also invited to participate in classroom activities and receive a presentation especially directed to students touching on

design, installation methods, tooling, and constructability. Having this "school" in our own backyard and being invited to participate was wonderful.

These personal relationships with local consulting firms (Terracon, Stantec, Thelen Associates–a Division of Geotechnology, Inc., and others) have now developed over 34 years. These firms offer samples that we use in our Soil Mechanics Lab. They also offer meaningful COOP jobs to some of our undergraduates. These COOP jobs have been the springboard to graduate study and employment thereafter with these and other firms.

Why? Why is it important that our undergraduate students be introduced to site characterization techniques? Whether you want to become a structural engineer, a construction manager, a highway engineer, or an environmental engineer, the structures you will be involved with will be founded on or within the soil or be constructed of soil. Having an appreciation for how the soils exploration report was developed is important as you read for the details you need.

How? In addition to those items we have already discussed, I would encourage all of us to be lifelong learners. How important it is to keep up with the state-of-the-art even as we work in the state-of-the-practice. How can this be done? First, I would strongly suggest that we subscribe to a journal. I know these subscriptions can be expensive but reading of new research, new technology, case histories, and remedial measures is important. Some of the journals that I have found to be of benefit to me include:

1) Journal of Geotechnical and Geoenvironmental Engineering from ASCE. This journal was formerly entitled Journal of Geotechnical Engineering, Journal of the Geotechnical Engineering Division, and Journal of the Soil Mechanics and Foundations Division.

2) Canadian Geotechnical Journal.

3) Civil Engineering Practice, the Journal of the Boston Society of Civil Engineers, ASCE.

4) Geotechnique, Proceedings of the Institution of Civil Engineers, London.

5) Foundation Drilling, the magazine of the International Association of Foundation Drilling (ADSC).

Attendance at conferences, short courses and seminars such as this Ohio River Valley Soils Seminar are beneficial. Looking through my personal library I found the following volume:

Site Investigations: Geotechnical and Environmental, Proceedings of the XXVI Ohio River Valley Soils Seminar, Clarksville, Indiana, October 20, 1995.

I think it is appropriate that we are here today to again reflect on this important topic. I am grateful for those exhibitors who have brought literature, videos, equipment and other items today. I encourage all of us to take advantage of our breaks and visit these company representatives.

The proceedings of conferences can be very instructive. As I prepared for this talk I went to the Engineering Library at the University of Cincinnati and perused the geotechnical holdings. Here is a short list of recent relevant conference proceedings and texts available regarding geotechnical site characterization:

1) Lutenegger, A.J. and D.J. DeGroot (editors), Performance Confirmation of Constructed Geotechnical Facilities, Proceedings of sessions of ASCE Specialty Conference on Performance Confirmation of Constructed Geotechnical Facilities, April 9-12, 2000, Amherst, Massachusetts. ASCE Geotechnical Special Publication 94.

Abstract: Geotechnical engineering involves the evaluation of site conditions, the determination of soil and rock properties, and the application of design principles to produce solutions to ground engineering problems. Performance confirmation of the geotechnical aspects of a project provide a means for geotechnical engineers to validate design approaches, reinterpret property assessment, and improve the design process. The papers include performance and evaluation of shallow and deep foundations; roadway and railways; tunnels, excavations, and retaining structures; and fills, embankments, and slopes. Grouting in geotechnical engineering is also included, as are aspects of the statistical analysis involved in the evaluation.

2) Mayne, P.W. and R. Hryciw (editors), Innovations and Applications in Geotechnical Site Characterization, Proceedings of sessions of Geo-Denver 2000, August 5-8, 2000, Denver, Colorado. ASCE Geotechnical Special Publication 97.

Abstract: In situ testing provides a direct means for evaluating natural and man-made ground conditions for geotechnical site characterization. Soils and rocks can be subjected to loading under ambient anisotropic stress states, complete with effects of inherent fabric, aging, cementation, sensitivity, fines content, and structure. The variety of field tests are particularly useful for designing different types of foundation systems, assessing soil liquefaction potential, and verifying soil improvement works. Many in situ tests now provide continuous and real time data with

multiple independent readings taken within a single sounding.... Recent advances have developed in the interpretation of in situ penetration test data from cone soundings (CPT, CPTu), flat plate dilatometer (DMT), pressuremeter tests (PMT), soil borings (SPT), as well as geophysical techniques (CHT, DHT). In addition, new devices have been invented to provide higher stratigraphic resolution and to quantify specific soil parameters and properties.

3) Proceedings of the Fifteenth International Conference on Soil Mechanics and Geotechnical Engineering, August 27-31, 2001, Istanbul, A.A. Balkema Publishers, Lisse, The Netherlands.

Section 1.2 is entitled "Soil Property Characterization by Means of Field Tests." Within this section are 47 papers. Some of the topics include:

*evaluation of consolidation parameters from piezocone penetration tests
*correlation between penetration resistance and relative density of sandy soils
*in situ and laboratory shear wave velocity measurements in clay
*characterization of non-linear elastic soil behavior from field plate load tests
*undrained shear strength of clay estimated by neural networks
*evaluation of undrained shear strength of cohesive soil from in situ tests
*in situ measurement of local saturated permeability of compacted clayey soils by means of minipiezometers
*in situ direct shear testing method for rockfill materials, sands and clays
*engineering properties and slake durability of weak Triassic Basin rock
*liquefaction prediction using fuzzy neural network model based on SPT
*an in situ permeability measurement technique for cut-off walls using the Cambridge self-boring pressuremeter
*estimating dynamic shear modulus in cohesive soils
*interpretation of pressuremeter tests using non-linear elasto-plastic analysis
*estimation method of the bearing capacities of shallow and pile foundations from dynamic probings

4) Puppala, A.J. et al. (editors), Site and Geomaterial Characterization, Proceedings of sessions of Geo-Shanghai, June 6-8, 2006, Shanghai, China. ASCE Geotechnical Special Publication 149.

Abstract: [This publication] contains more than 40 technical papers that address issues ranging from the microcharacterization of processes to the in situ evaluation of engineering properties of soils.

5) Alshawabkeh, A.N. et al. (editors), GeoCongress 2008–Characterization, Monitoring, and Modelling of Geosystems,

Proceedings of selected sessions of GeoCongress 2008, March 9-12, 2008, New Orleans, Louisiana. ASCE Geotechnical Special Publication 179.

Abstract: [This publication] covers mechanical and chemical soil behavior, testing, and modeling. [It] presents innovations on subsurface characterization and monitoring, characterization of rocks, problematic soils and waste materials; and sensor technologies. Recent developments in numerical and computational geotechnics, emerging technologies, fate and transport modeling, uncertainty modeling, and micro-and environmental geomechanics are also covered in this [publication].

6) Rollins, K. and D. Zekkos (editors), Geotechnical Engineering State of the Art and Practice, Keynote lectures from GeoCongress 2012, March 25-29, 2012, Oakland, California. ASCE Geotechnical Special Publication 226. Notable papers include the following:

a) Mayne, P.W. "Geotechnical Site Characterization in the Year 2012 and Beyond."

b) DeGroot, D.J. and C.C. Ladd, "Site Characterization for Cohesive Soil Deposits Using Combined In Situ and Laboratory Testing."

c) Houston, W.N. and J.D. Nelson, "The State of the Practice in Foundation Engineering on Expansive and Collapsible Soils."

Other publications that are very useful are the following:

1) Simons, N., B. Menzies, and M. Matthews (2002), A Short Course in Geotechnical Site Investigation. Thomas Telford Publishing, London, 353 pages.

Overview: "When planning this book, we sought advice from site investigation practitioners. –In particular, we asked them to tell us what young civil engineers and engineering geologists would want from our book; the following reply from Andrew Bowden (Mouchel Consulting Ltd., UK) was adopted as our theme:"

> What all young engineering geologists ask me is: how do you plan a site investigation? How many holes, where, how deep, what samples and why? What to test them for and how many tests should be done? How do you know if the tests are correct? My reply to them is that you first have to learn some site investigation. Then learn some design and then some more site investigation until the whole picture emerges. You cannot plan a site

investigation until you understand what is needed for design and you cannot understand what is needed for design until you know about soil properties which you understand by doing site investigation and getting dirty....

2) Anderson, N., N. Croxton, R. Hoover, and P. Sirles, (2008), Geophysical Methods Commonly Employed for Geotechnical Site Characterization, Transportation Research Circular -C130, Transportation Research Board, Washington, DC, 35 pages.

3) ICE Manual of Geotechnical Engineering, Volume I Geotechnical Engineering Principles, Problematic Soils and Site Investigation, edited by J. Burland, T, Chapman, H. Skinner, and M. Brown, Institution of Civil Engineers, ICE Publishing, London, 2012, 727 pages.

4) ICE Manual of Geotechnical Engineering, Volume II Geotechnical Engineering Design, Construction and Verification, edited by J. Burland, T. Chapman, H. Skinner, and M. Brown, Institution of Civil Engineers, ICE Publishing, London, 2012, 806 pages.

5) Loehr, J.E., A. Lutenegger, B. Rosenblad, and A. Boeckmann, Geotechnical Site Characterization, Geotechnical Engineering Circular No. 5, National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Washington, DC, 2016.

Conclusions and Recommendations: Teaching Innovations in Geotechnical Site Characterization:

Who? Introduce this topic to undergraduate students in Soil Mechanics and Foundation Engineering classes. Offer advanced site characterization classes to graduate students in Geotechnical Engineering.

Who should teach this material? A dedicated professor with experience plus presentations and field demonstrations by qualified consultants who can bring in the state-of-the-art as well as state-of-the-practice.

What? Present the material that is essential to the audience. Undergraduate Civil Engineering students need to understand how soil parameters are determined both in the field and the laboratory. They need to understand how to put together a site exploration plan and how to write and interpret a soils report. Leave more advanced presentations for those who have decided to attend graduate school where they can receive instruction in advanced topics of Geotechnical Engineering.

When should such material be presented? Introduce site characterization in

Soil Mechanics to undergraduates in their junior year; expand upon site exploration and testing in Foundation Engineering to students in the senior year; and specialized and in-depth geotechnical studies during graduate programs.

Where? A combination of classroom lecture, laboratory hands-on testing, and field demonstrations of equipment.

Why? Because it matters that we understand our field of choice. We cannot survive using only the tools we were taught in our undergraduate programs. They were designed to get us through the first five years of our professional careers. It was expected that ideas, tools, and our understanding of processes would change. We need to teach our graduating civil engineering students how to keep abreast of this knowledge, laying out for them ideas for continued growth and learning.

How? As an appendage to "what should we teach," the "how" should explore the idea of being a life-long learner. Individually we need to continue to read, study and advance our knowledge. We can do this and encourage our young engineers to participate in seminars, short courses, conferences, and field demonstrations. Subscribing to and reading journal and discipline-specific magazines is essential. Using the internet to explore articles to keep ourselves current is important.

This paper is dedicated to the memory of Ralph Rollins, PhD, PE, who taught me Soil Mechanics at Brigham Young University and who offered me my first job with his firm (1977-1981) as a fresh graduate in Civil Engineering.

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Estimating the Collapse Potential of Gypsum Sands Using Shear Wave Velocity Testing

Raghava A. Bhamidipati[1], Michael E. Kalinski[2] and L. Sebastian Bryson[3]

ABSTRACT

This paper provides a description of results of shear wave velocity and collapse potential testing conducted on reconstituted gypsum-sand specimens. Mixtures of quartz sand and ground gypsum were used to reconstitute cylindrical specimens in the laboratory. Six different specimens were subjected to mean effective confining stresses (σ_o') ranging from 34-300 kPa. Shear wave velocity (v_s) of the specimens was measured in the laboratory using the free-free resonant column method. By plotting v_s versus σ_o' it was observed that v_s increased with increasing σ_o'. Shear wave velocity also increased as gypsum content increased from 0% to 30%, but decreased when gypsum content exceeded 30%. Next, a set of collapse tests was performed on gypsum sand mixtures with varying gypsum content. Collapse potential was observed to increase linearly with gypsum content. Based on these two sets of observations, an approach was developed to predict collapse potential in gypsum soils as a function of v_s. In regions where the engineering properties of gypsum-rich soils are of concern, these results may be used as a basis for field testing where surface geophysical methods can be employed to measure in v_s to nondestructively estimate collapse potential.

Introduction

Gypsum sands across the world are susceptible to numerous engineering hazards, annually incurring losses of the order of millions of dollars. The unique properties of the mineral gypsum, $CaSO_4 \cdot 2H_2O$, including its relative softness, moderate solubility and reactivity, are responsible for rendering soils hazardous for new or existing construction (Cooper & Calow, 1998). Gypsum is often found in arid regions of the world along with calcite and dolomite in the form of evaporites. Gypsum soils have a high permeability, low unit weight and are predisposed to settlement. The problem of collapsibility is especially widespread and catastrophic. Structures like irrigation canals and dams have been reported to show major deformations and failures. Many such instances have been reported in Iraq. The deformation of irrigation canals in Ebro Valley, Spain,

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constructed on gypsiferous loess soils is another prominent example (Alphen & Rios Romero, 1971). Fourteen dam sites in the United States alone have been affected by dissolution of gypsum karst as of 1998 (Cooper & Calow, 1998) .

Fig 1. Subsidence on gypsum soil in Ripon, England

A number of researchers from Iraq studied the collapse potential and engineering properties of gypsiferous soils. They found that factors such as gypsum content, unit weight, moisture content, void ratio and soil plasticity influence collapse potential of these soils (Seleam, 2006). Fattah et al. (2008) noted an increase in collapse potential with gypsum content. Fattah et al. (2007) also noted that the collapse potential of gypsiferous soils increases with increase in gypsum content and thickness of collapsible layer but was not dependent on the footing dimensions. They used the computer program Settle3D in this analysis. Nashat (1990), found that collapse potential increased from gypsum content of 20% to 60% and decreased thereafter.

There is a need to develop non-destructive and non-intrusive tests to rapidly screen sites with gypsum soils. Geophysical seismic methods are attractive candidates for such tests. They can be used to measure in situ stiffness, which in turn can be correlated to gypsum content. Fixed–free resonant column testing and bender element testing methods have traditionally been used to perform small-strain (less than 0.001%) dynamic tests on soils in the laboratory. The technique presented herein is known as free-free resonant column testing (Kalinski & Thummaluru, 2005). It is a simpler alternative to the conventional fixed-free resonant column test.

Kalinski & Thummaluru (2005), used free-free resonant column testing to measure the variation in stiffness of Ottawa sand over a range of confining pressures. Mazari et al. (2013), used Free-Free Resonant Column testing method to develop correlations between seismic modulus and resilient modulus of subgrade materials. Verastegui-Flores et al. (2014) used the free-free resonant frequency method to determine the small-strain stiffness moduli of cement-treated soil (kaolin).

The collapse potential of gypsum soils is studied using the methodology proposed by Jennings and Knight (Al-Rawas, 2000). In this study, collapse potential is measured using the Single Collapse Test (Knight, 1963). The objective of the study presented herein is to find a relationship between collapse potential, gypsum content and stiffness so that geophysical seismic testing can be used to estimate gypsum content and rapidly screen sites for collapse potential.

The term “gypsiferous sand” is broadly used to describe sandy soils having a significant amount of gypsum, but not exceeding 50% by weight (Herrero & Porta, 1999; Al-Marsoumi et. al, 2008). Gypsum sands usually occur in arid regions, where soils generally have substantial coarse-grained fractions. Considering these aspect, mixtures of fine quartz sand (passing the # 40 sieve) and ground gypsum were reconstituted in the laboratory with different percentages of gypsum by weight. The quartz sand was classified according to the Unified Soil Classification System (USCS) as SP. Stiffness and collapsibility of gypsum soils are based on a number of factors such as gypsum content, moisture content, initial void ratio, dry density and fines content (Seleam, 2006). The research presented herein focuses on the effect of gypsum content.

Free-Free Resonant Column Testing

Mixtures of quartz sand and gypsum were reconstituted into cylindrical specimens with a 2:1 (length:diameter) aspect ratio using a latex membrane and two plastic end caps. A Ledex 500 model No. H-1079-032 rotary solenoid was attached to one endcap and a pair of PCB 353B16 accelerometers was attached to the other endcap across the diameter (Fig. 2). The accelerometers have a sensitivity of 10 mv/g and an operating frequency of 0.7–20,000 Hz (Kalinski & Thummaluru, 2005). The usage of two accelerometers ensures that the recorded motion is predominantly torsional rather than flexural. The resonant frequency of the specimen (f_n) is identified as the frequency that produces the maximum torsional acceleration. The solenoid, which is driven by a function generator, excites the specimen in the torsional mode by imparting a transient pulse. The time-domain torsional excitation of the specimen measured by the accelerometers is converted into the frequency domain and f_n is identified as the frequency corresponding to the peak amplitude. The summed voltage output

from the accelerometers was passed through a PCB signal conditioner (Model no. 482A22) and recorded using a Coco-80 dynamic signal analyzer.

Vacuum was applied to the pore spaces in the specimen to provide confinement. The specimen is suspended horizontally with both ends of the cylinder free to oscillate. The specimen is placed in a pressure cell (Fig. 2) and the effective confining stress (σ_o') is taken as the sum of the vacuum and the cell pressure.

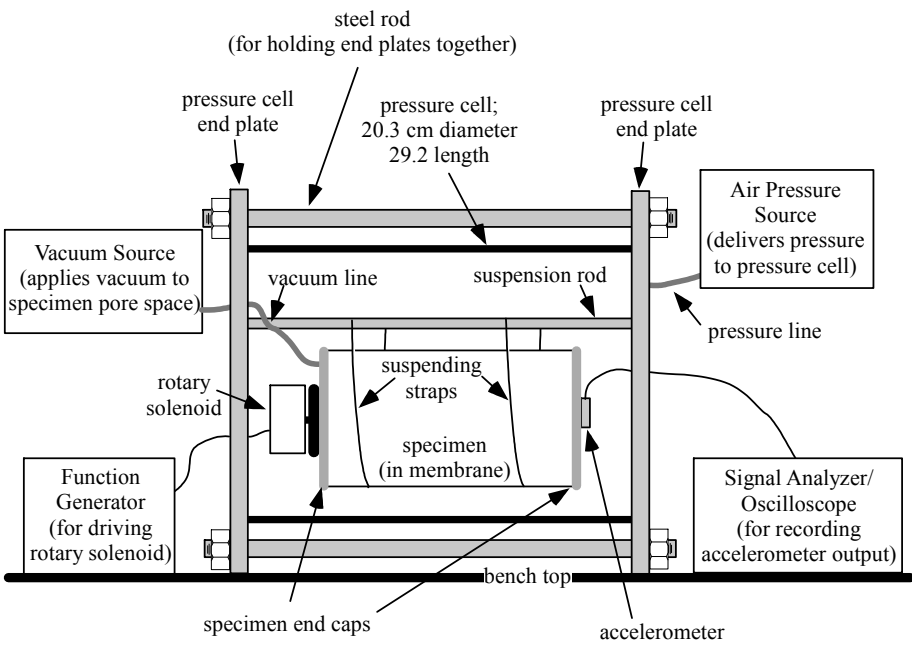

Figure 2. Free-Free Resonant column Testing Arrangement

The stiffness of the specimen is quantified in terms of shear wave velocity (v_s) and small-strain shear modulus (G_{max}). The relationship between these parameters is given by:

$$G_{max} = \rho\, v_s^2, \qquad (2)$$

where ρ is the mass density of the specimen. Kalinski and Thummaluru (2005) present the following relationships between the length of the specimen (L) and an intermediate variable □:

$$\beta = 2\pi f_n L / v_s. \qquad (3)$$

For the free-free condition,

$$\mathrm{Tan}\beta = (\mu_1 + \mu_2)\beta / (\mu_1\mu_2\beta^2 - 1), \qquad (4)$$

where

$$\mu_1 = I_1 / I \qquad (5)$$

and

$$\mu_2 = I_2 / I. \quad (6)$$

In Eqns. 5 & 6, I_1 and I_2 are the polar moments of inertia of the masses attached to each end of the specimen and I is the polar moment of inertia of the specimen about its longitudinal axis.

A total of six specimens were reconstituted with gypsum contents ranging from 0-100%. The specimens were prepared using a split mold. Gypsum sand mixtures were poured into the mold and compacted using a tamper. The void ratio of all the specimens prepared in this manner was approximately 0.55. Specimens were 10 cm in diameter, 22 cm in length and weighed around 3.2 kg. Shear wave velocities were measured at effective confining stresses ranging from 34-300 kPa.

Collapse Potential Testing

The collapse settlement of sand is estimated based on a parameter called collapse potential (*CP*). In this study, *CP* was measured using the Single Collapse Test (Knight, 1963) in a one-dimensional consolidation load frame. Samples of gypsum sand were placed in the consolidation cell and the mass and volume of the sample were noted. This information was used to calculate the initial void ratio of the specimen. Dry gypsum sand mixtures were poured into the cell using a funnel. The specimens used in this research had a diameter of 6.35 cm and a height of 1.88 cm. The specimens prepared in this manner had a void ratio of about 0.62. Next, the consolidation cell was mounted in the load frame. The soil sample was then gradually loaded up to 200 kPa. At this point the sample was inundated with water and left for 24 hours. Loading was thereafter continued until a final load of 312 kPa was achieved and then the test was discontinued. The deformation of the sample in the 24-hour time period (ΔH) is recorded. Collapse potential of the soil is defined as:

$$CP = \Delta H / H_0 = (\Delta e / 1 + e_0) * 100\%, \quad (7)$$

where H_0 is the initial height of the sample, e_0 is the initial void ratio and Δe is the change in void ratio of the specimen. Collapse potential is a measure of vertical strain which in practice would be multiplied by the thickness of a given layer of gypsum sand to estimate vertical settlement at the ground surface. Figure 3 is a schematic illustration of results from a single collapse test. Five different samples were tested with gypsum contents ranging from 5-75% and their collapse potential was plotted against gypsum content. The degree of severity of collapse potential is based on a classification system suggested by Jennings and Knight (1975). It is based on the magnitude of collapse potential presented in Table 1 (Al-Rawas, 2000).

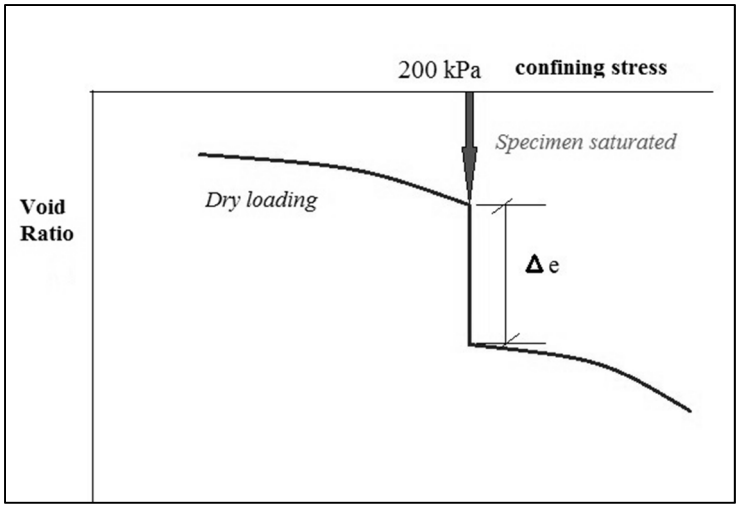

Figure 3. Schematic illustration of a collapse test

Table 1. Severity of collapse based on *CP* values

Collapse Potential, *CP* (%)	**Severity**
0-1	No problem
1-5	Moderate trouble
5-10	Trouble
10-20	Severe trouble
>20	Very severe trouble

Results

Variations of v_s with σ_o' were plotted for each specimen as a result of free-free resonant column testing. Figure 4 shows v_s versus σ_o' for each specimen. A power curve was fit through the different data points. The tests on dry soils showed that stiffness of the gypsum sands (v_s) increases with increasing confining stress. The increase in v_s was roughly proportional to the fourth root of confining pressure.

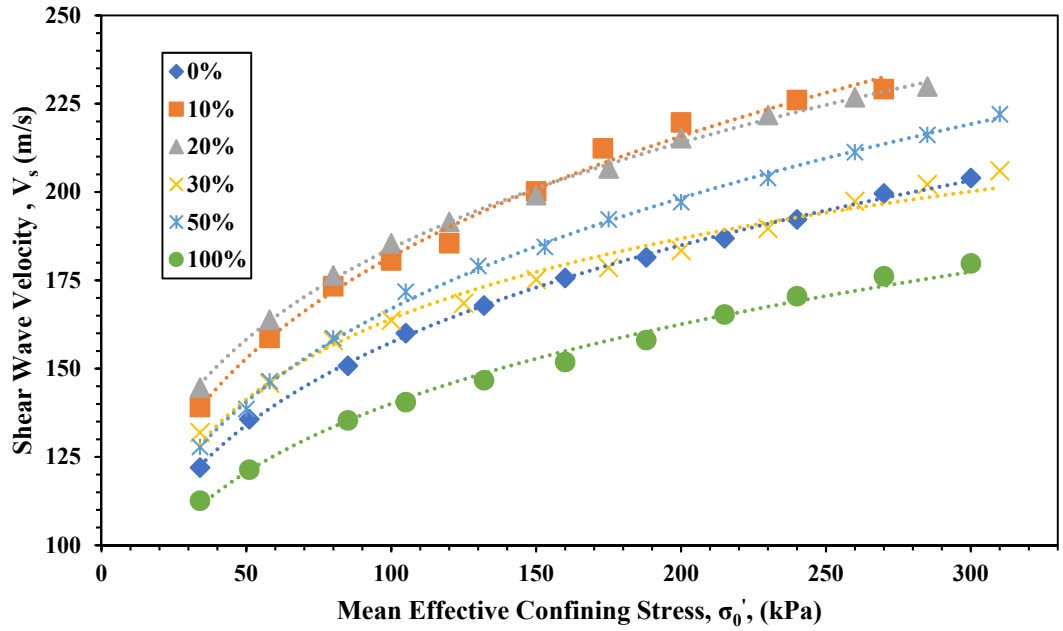

Figure 4. Plot showing v_s versus σ_o' curves for reconstituted gypsum-sand specimens

Shear wave velocity varied with σ_o' according to the relationship

$$v_s = C(\sigma_o')^n , \qquad (8)$$

where C and n are regression coefficients (Table 2).

Shear wave velocity for the different specimens was measured as a function of effective stress and followed the relationship described in Eqn. 8. Corrected shear wave velocities were calculated by normalizing v_s with respect to stress as follows:

$$v_{scorr} = v_s/(\sigma_o')^n, \qquad (9)$$

where σ_o' is in units of kPa. Corrected v_s was plotted against gypsum content as shown in Figure 8. The plot showed that v_{scorr} is a maximum when the gypsum content is around 20%.

Table 2. Regression coefficients of the v_s versus σ_o' plots for specimens tested using cell pressure

Gypsum Content (%)	*C*	*n*
0	54.0	0.233
10	51.2	0.252
20	61.4	0.237
30	63.9	0.223
50	53.2	0.248
100	52.1	0.215

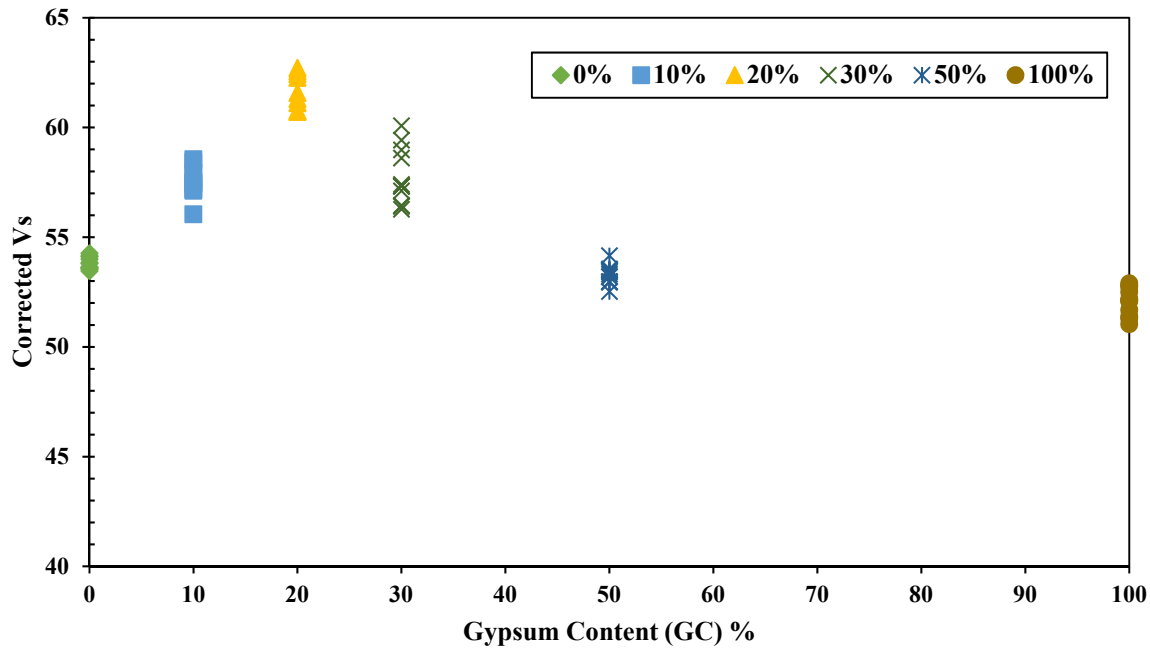

Figure 5. Stress-corrected shear wave velocity versus gypsum content

The stiffness of the soil specimens initially increased with increasing gypsum content. Mixtures with 10% and 20% gypsum exhibited the highest stiffness. Stiffness decerased with further increase in gypsum content and remained

constant after around 50%. This behavior is attributed to the softness of gypsum and the contact between the quartz and gypsum particles. At lower gypsum contents, the quartz particles are still in contact with each other and gypsum occupies the interstitial void spaces and increases the overall stiffness of the material. As the gypsum content increases beyond a certain level (in this case, 30 %), the quartz particle contact starts reducing leading to a decrease in the material stiffness. Gypsum has a lower specific gravity (2.32) than quartz (2.65). As such, when the proportion of gypsum in soils is high, the lower specific gravity of gypsum becomes a dominant factor and the soil stiffness is lower. This aspect has been confirmed by testing a pure gypsum specimen which yielded the lowest v_s profiles among all the mixtures tested.

Al-Marsoumi et al. (2008) studied the influence of gypsum content on the shear strength parameters of six different gypsiferous soil samples colleted near Basrah, Iraq. They performed triaxial compression and unconfined compressive strength tests and found that the angle of internal friction and cohesion of the soil samples increased with gypsum conetnt up to 20% and decreased thereatfter.

From the collapse potential tests it was observed that collapse potential increased with increasing gypsum content in a linear manner. This behaviour could be attributed to the higher solubility of gypsum. Figure 6 shows vertical strain versus normal load for the different specimens and Fig. 7 shows collapse potential versus gypsum content. The results are in agreement with Fattah et al. (2008), who studied the settlement behavior of gypsum sands in Iraq with natural gypsum content ranging from 15-66%. The collapse potential of these samples also increased with increasing gypsum content (Fattah et al., 2008). The linear relationship between collpase potential and gypsum content shown in Eqn. 10 can be fit to a straight line using linear regression to yield the following relationship:

$$CP = 0.0635(GC) + 0.28. \quad (10)$$

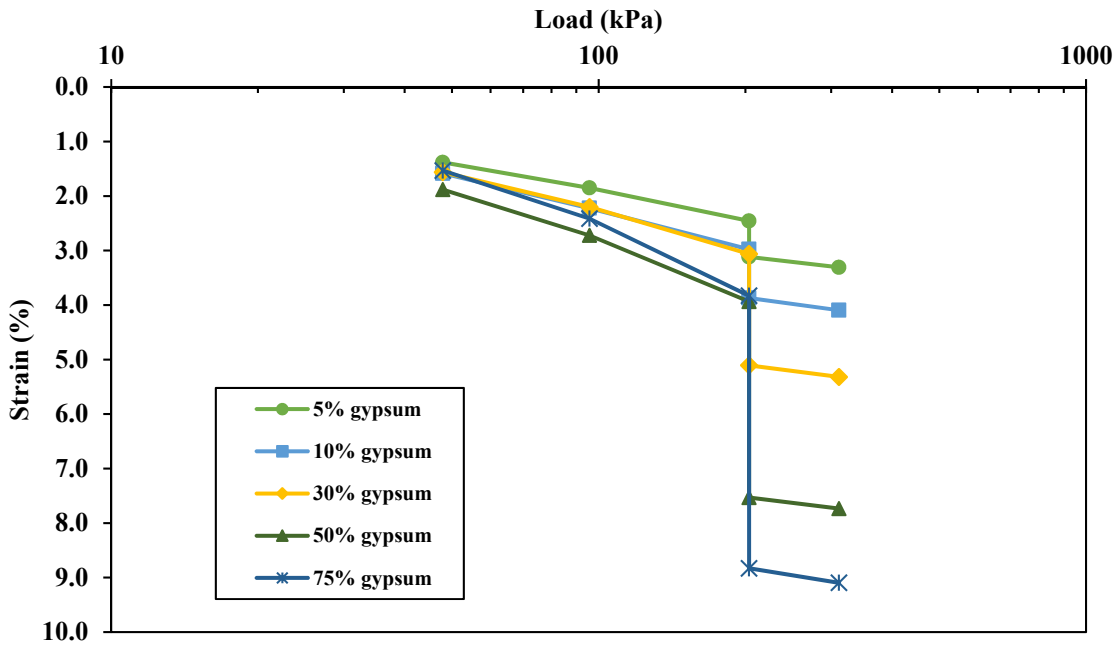

Figure 6. Vertical strain versus confining stress for the different gypsum soils in the collapse potential test.

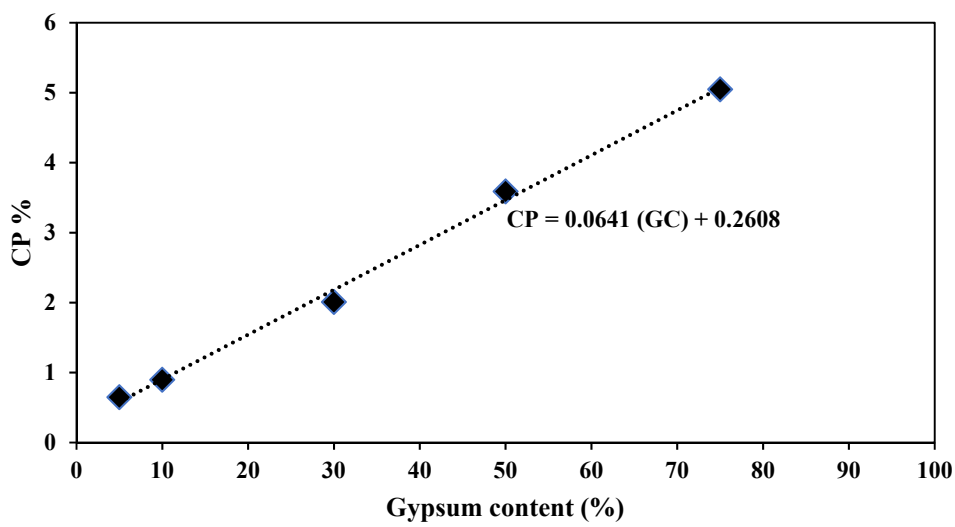

Figure 7. Collapse potential versus gypsum content

Conclusions

Using the results from the laboratory testing presented herein, an approach can be developed to use nondestructive, nonintrusive geophysical seismic testing to estimate collapse potential in gyspum soils. For a given site of interest in a region prone to gypsum soils, nonintrusive geophysical seismic testing can be performed. There are numerous methods available, including reflection seismic,

refraction seismic and surface wave methods. For the purpose of measuring v_s, the surface wave methods are the simplest, most direct option. Common surface wave methods include the Spectral-Analysis-of-Surface-Waves (SASW) Method (Stokoe et al., 1994) and the Multichannel-Anaysis-of-Surface-Waves (MASW) Method (Park et al., 1999). Done properly, both method produce reliable results and allow the user to quantify variations in v_s with depth.

Once v_s has been calculated, it can be corrected using Eqn. 9 to remove the effect of confining stress. In Eqn. 9, the stress term $\boldsymbol{\sigma_o'}$ is a mean effective term, which can be calculated using vertical effective stress, $\boldsymbol{\sigma_v'}$, using the following relationship:

$$\boldsymbol{\sigma_0' = \frac{\sigma_v'}{3}(1 + 2K_o),} \qquad (11)$$

where K_o is the coefficient of earth pressure at rest. By superimposing v_{scorr} onto Fig. 11, gypsum content can be estimated. Note that the interpretataton of Fig. 8 may not be unique and one value of v_{scorr} may yield two values for gypsum content. In this instance, judgement is required to select a lower or higher value for gypsum content depending on other factors such as the appearance of the soil. The estimated value for gypsum content can then be used with Eqn. 10 to estimate collapse potential. By testing several prospective sites, the sites can be screened and compared for collapse potential and estimates for collapse potential can be taken into account in subequent design.

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NOTES

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Shear-Wave Velocity Database for Communities Along the Ohio River Valley

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Abstract

Shear-wave velocity of near-surface sediment and bedrock is of geotechnical engineering interest. For example, the site coefficient for seismic engineering design is determined by the time-weighted average shear-wave velocity for the top 30 m of soils and rock. Thus, obtaining the shear-wave velocities of sediments and bedrocks is a key component of a geotechnical engineering site investigation. Researchers at the University of Kentucky have been collecting in-situ shear-wave velocities for sediments and bedrocks in many Kentucky communities, such as Maysville, Louisville, Owensboro, and Paducah, along the Ohio River Valley using SH-wave refraction/reflection methods. The SH-wave is a horizontally polarized S-wave that can be generated by horizontally striking a section of steel I-beam with a sledgehammer. The SH-wave is more effective than the P-wave in determining shear-wave velocities and subsurfaces of water-saturated sediments because the S-wave can only propagate through the soil matrix, not water. A database of shear-wave velocities derived from in-situ measurements has been compiled and used for geologic mapping and seismic-hazard assessment in communities along the Ohio River.

Introduction

Near-surface sediment and bedrock is of environmental and engineering interest. For example, earthquake ground motion can be modified by near-surface soft sediments, the so-called site-effect, resulting in significant damage to the built environment. A classic example of such damage is in Mexico City, where amplified ground motions of near-surface lake sediments caused significant damage during the 1985 Michoacán earthquake (M 8.1) (Seed and others, 1988). As shown in Figure 1, Quaternary soft sediments concentrate along the

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river valleys, particularly the Ohio and Mississippi Rivers. As demonstrated by Woolery and others (2008, 2012), site-effect has also been widely observed in many communities along the Mississippi and Ohio Rivers in the central United States. For example, ground-motion amplification by the alluvial sediments caused about $3 million in damage in Maysville during the 1980 Sharpsburg earthquake (M 5.3) (Woolery and others, 2008). Therefore, site-effect is a major concern in earthquake engineering in the central United States.

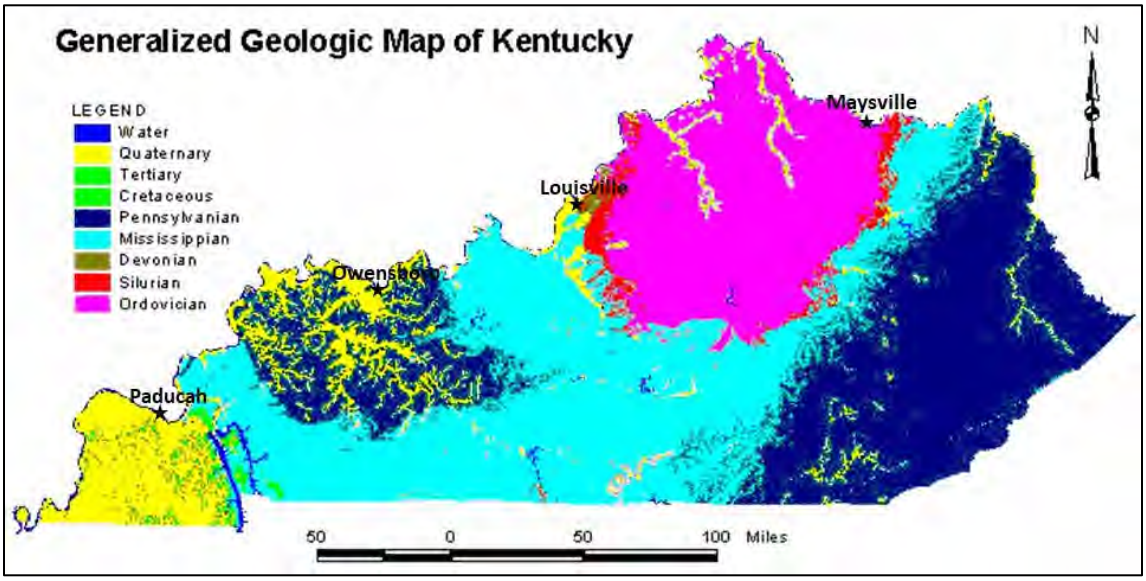

Figure 1. Generalized geology of Kentucky.

Site-effect is influenced by many factors, including lateral and/or vertical velocity gradients in the sediment and/or bedrock, impedance contrasts within the sediment overburden and at the sediment-bedrock interface, sediment thickness, sediment-bedrock interface geometry (i.e., horizontal, irregular, dipping, etc.), incoming ground-motion amplitude (i.e., linear vs. nonlinear), and surface topography. Shear-wave velocity of the sediments and bedrock is the key parameter that is considered in current engineering practice. For example, the site coefficient for seismic design is determined by the time-weighted average shear-wave velocity for the top 30 m of soils and rock in the United States (e.g., BSSC, 2009). Therefore, determination of the shear-wave velocity of sediments and bedrock is of engineering interest.

Researchers at the University of Kentucky (UK) have measured shear-wave velocity for sediment and bedrock throughout Kentucky, particularly for communities along the Ohio River such as Louisville, Owensboro, and Paducah. Specifically, SH-wave reflection and refraction methods are being utilized to obtain in-situ shear-wave velocities for the soft sediments and bedrock. Included is a shear-wave velocity database for sediments and rocks and how it is being used for geologic and seismic hazard mapping.

SH-Wave Reflection and Refraction

An SH-wave is an S-wave whose particle motion is in the horizontal plane and can be easily generated at the surface by a horizontal impact source (e.g., Wang and others, 2000). Figure 2 shows the sources that have been used by the UK researchers. The signal receivers are horizontal component geophones (Fig. 2c). The data were collected with a 24- or 48-channel seismograph. Depending on the local geology and site conditions, different steel I-beams and hammers, geophone spacing, and shot intervals were used in order to optimize data acquisition. For each shotpoint, signals were normally stacked two to five times on each direction (reversed) of the energy source. A single or multiple spreads of reflection and refraction profiles were collected at each site. The data were processed on a PC using the commercial software package VISTA 7.0 by Seismic Image and SeisImager/2D by Geometrics.

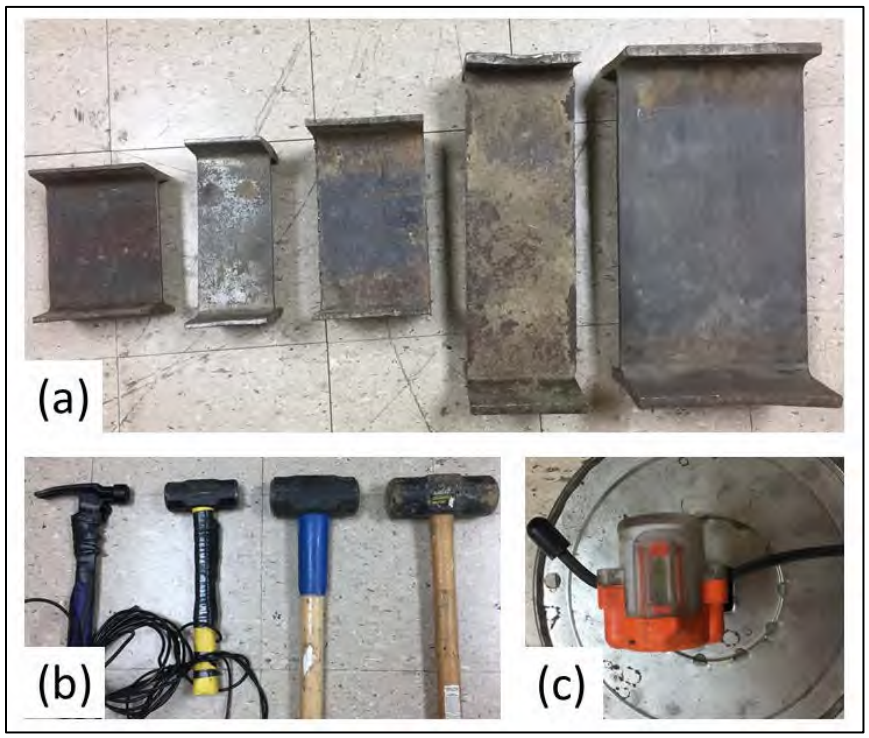

Figure 2. SH-wave seismic sources: steel I-beams (a) and hammers (b), and geophone (c).

Figure 3 shows a comparison between P-wave and SH-wave refractions, which were collected with 48 geophones at 2 m spacing at a site near the Kentucky River in Frankfort. As shown in Figure 3, only two refractors, one from the water-table (**RA_W**) and one from bedrock (**RA_B**), were recorded on the P-wave profile, whereas three refractors, two from the interfaces within sediments (**RA_{S1}** and **RA_{S2}**) and one from bedrock (**RA_B**), on the SH-wave profile. As also shown in

Figure 3, the time window for the SH-wave is about three times larger than that for the P-wave. Thus, SH-wave has the advantages in determining shear-wave velocities and subsurfaces of water-saturated sediments because the S-wave can only propagate through the soil matrix, not water.

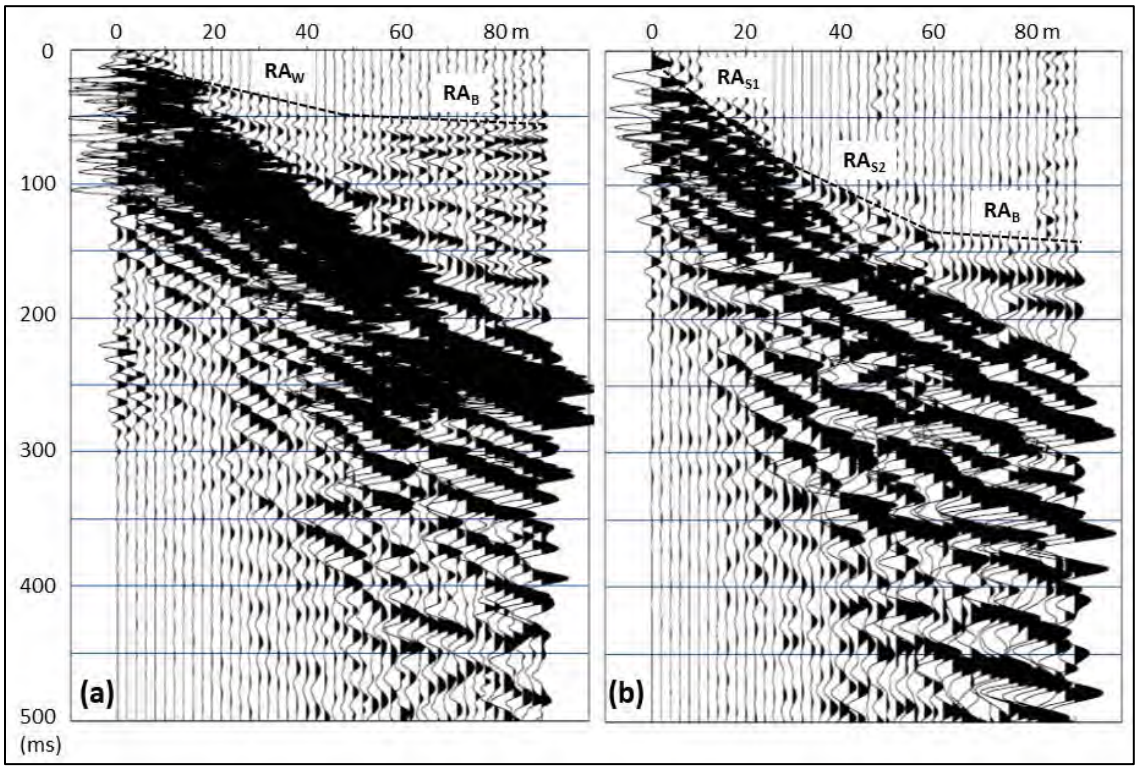

Figure 3. Comparison between P- and SH-wave refraction profiles recorded at a site near the Kentucky River in Frankfort. **RAw** – refractor along the water table; **RAB** – refractor along bedrock; **RAs** – refractor along interface between sediments.

Figures 4a–c show a single-spread SH-wave refraction/reflection recording with 24 geophones at 3.05 m spacing and shots at the first, 12th, and last geophones, collected at a site near the Ohio River in Louisville. As shown in Figures 4a–c, a direct wave (**D**), one refractor (**Ra**), and one strong reflection from bedrock (**Re**) were imaged. Figure 4d is the shear-wave velocity profile derived from the refractions and reflection. As shown in Figure 3d, a thin (~1 m) layer of surficial, very soft sediment is underlain by a thick (~21 m) alluvium. The depth to the bedrock is about 22 m. Figures 5a and b show the SH-wave recordings from a single-spread SH-wave refraction profile with 48 geophones at 2 m spacing and shots at the first and last geophones, collected at Owensboro Hospital in downtown Owensboro. The figures show a direct wave (**D**) and two refractors (**Ra1** and **Ra2**). Figure 5c is the shear-wave velocity profile derived from the refraction arrival times.

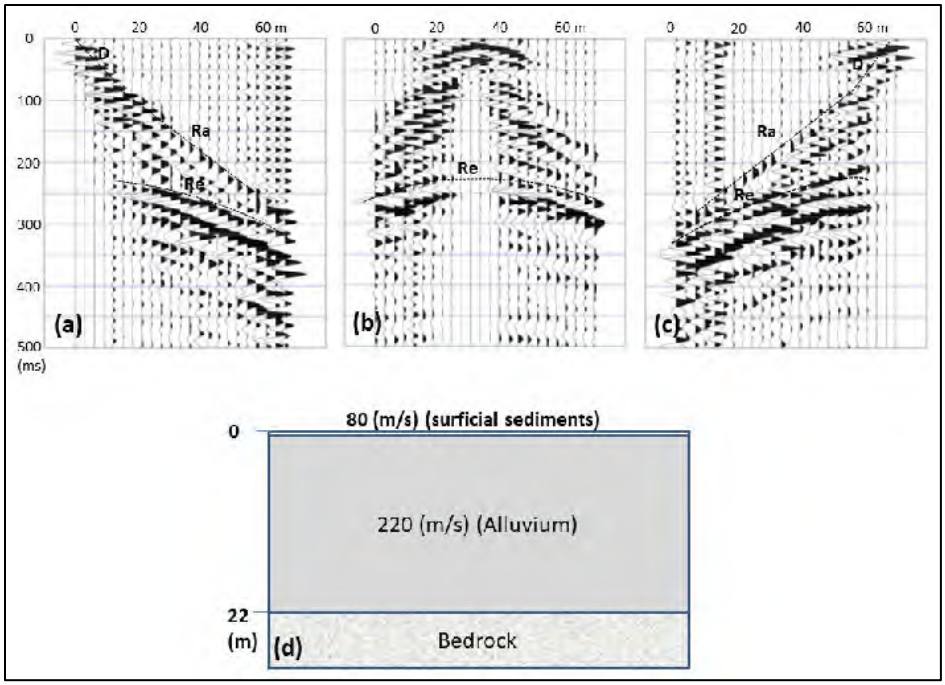

Figure 4. SH-wave refraction/reflection recordings (a–c) and interpreted shear-wave velocity profile (d) at a site near the Ohio River in Louisville.

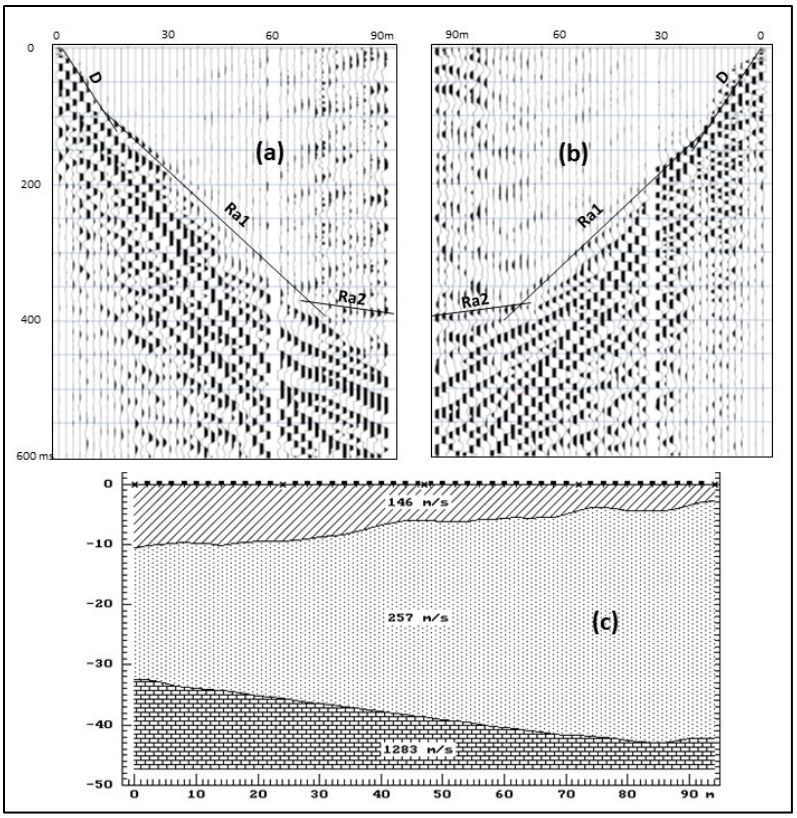

Figure 5. SH-wave refraction profiles (a, b) and interpreted shear-wave velocity profile (c) at Owensboro Hospital in downtown Owensboro.

Shear-Wave Velocity Database and its Utilization

Researchers at the University of Kentucky have been collecting SH-wave reflection and refraction data to derive shear-wave velocity profiles of the sediments and bedrock throughout the central United States, particularly in the Mississippi and Ohio River Valleys, for more than 20 years (Harris and others, 1994; Street, 1997; Street and others, 1995, 1997, 2005; Higgins, 1997; Woolery and others, 2009). An ArcGIS geodatabase has been created to archive these data at the Kentucky Geological Survey (Vance, 2006; Li and others, 2013). The database is flexible so that new data can be added easily.

Woolery and others (2008, 2012) indicated that earthquake ground-motion amplification (i.e., site-effect) has been widely observed in many communities along the Mississippi and Ohio Rivers in the central United States. One of the uses of the shear-wave velocity database is to characterize potential ground-motion amplification. Harris and others (1994) analyzed ground motion based on measured shear-wave velocity profiles and a 1-D site response model and produced peak spectral amplification and dynamic site period maps for Paducah. As shown in Table 1, the ground-motion amplification can be classified based on the average shear-wave velocity of the soils and rock in the top 30 m (Borcherdt, 1994). This site classification has been adopted in engineering design for buildings and other structures: the so-called NEHRP site classification (BSSC, 2009). Street and others (1997) applied the NEHRP site classification to map the amplification potential for the Jackson Purchase Region in western Kentucky based on shear-wave velocity data. This methodology was also applied to map the amplification potential in the Louisville metropolitan area (Fig. 6) (Wang, 2008).

Table 1. NEHRP site classification.

Site Class	Soil Profile Name	Top 30 m Average Shear-Wave Velocity (m/s)
A	Hard rock	$V_{S30} > 1{,}500$
B	Rock	$760 < V_{S30} \leqslant 1{,}500$
C	Very dense soil/soft rock	$360 < V_{S30} \leqslant 760$
D	Stiff soil	$180 \leqslant V_{S30} \leqslant 360$
E	Soft soil	$V_{S30} < 180$

The shear-wave velocity data derived from the SH-wave reflection and refraction profiles at 32 sites throughout the urban and suburban areas of Owensboro were also used to delineate the geologic units. Figure 7a shows the shear-wave velocity cross section derived from a 15-spread SH-wave refraction profile along Field Road in Owensboro. As shown in the figure, two layers of sediment with shear-wave velocities of 99–113 and 162–211 m/s, respectively, overlying

bedrock with a shear-wave velocity of 1,095–1,266 m/s, were derived from the SH-wave refractions. Figure 7b is the interpreted geologic cross section: Pleistocene-Holocene loess and alluvium (Qel-Qal) and Pleistocene lacustrine terrace (Qlt) over the bedrock (Andrews, 2005). Table 2 lists the shear-wave velocities and thicknesses for the sediments and bedrock that were mapped in the Owensboro area (Andrews, 2005).

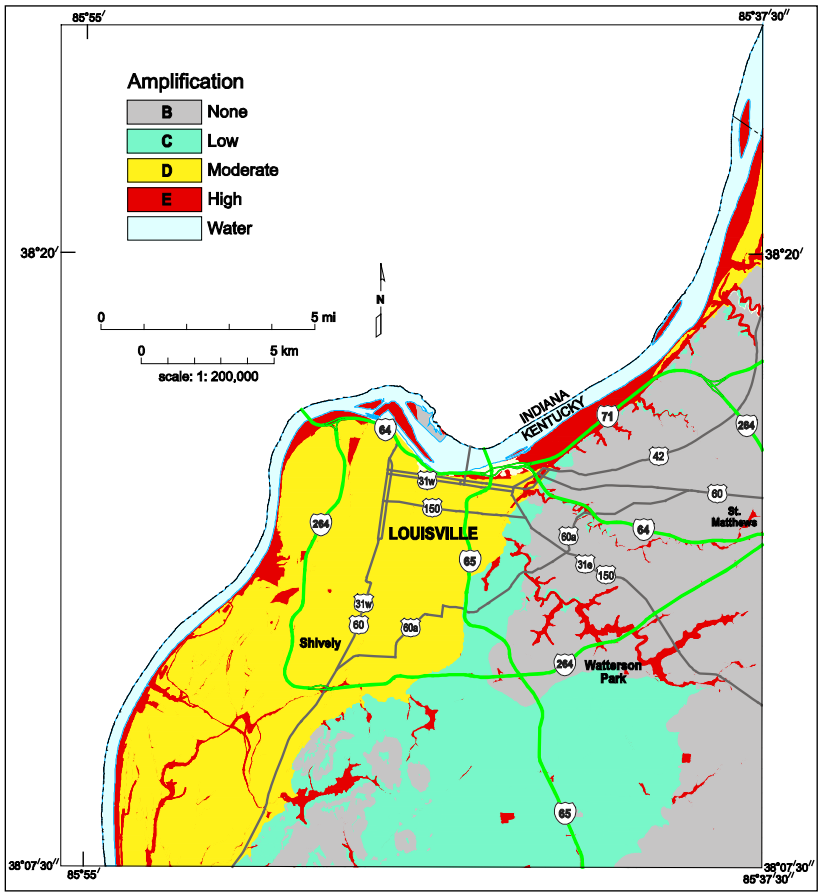

Figure 6. Earthquake ground-motion amplification potential in Louisville.

Table 2. Quaternary sediments and their shear-wave velocities in the Owensboro area.

Sediment	Shear-wave velocity (m/s)	Age
Modern alluvium	100–216	Holocene
Glacier outwash	218–312	Pleistocene-Holocene
Sand dune	122	Pleistocene
Loess	129–275	Pleistocene-Holocene
Lacustrine	153–224	Pleistocene
Bedrock	1,000–1,568	Pennsylvanian

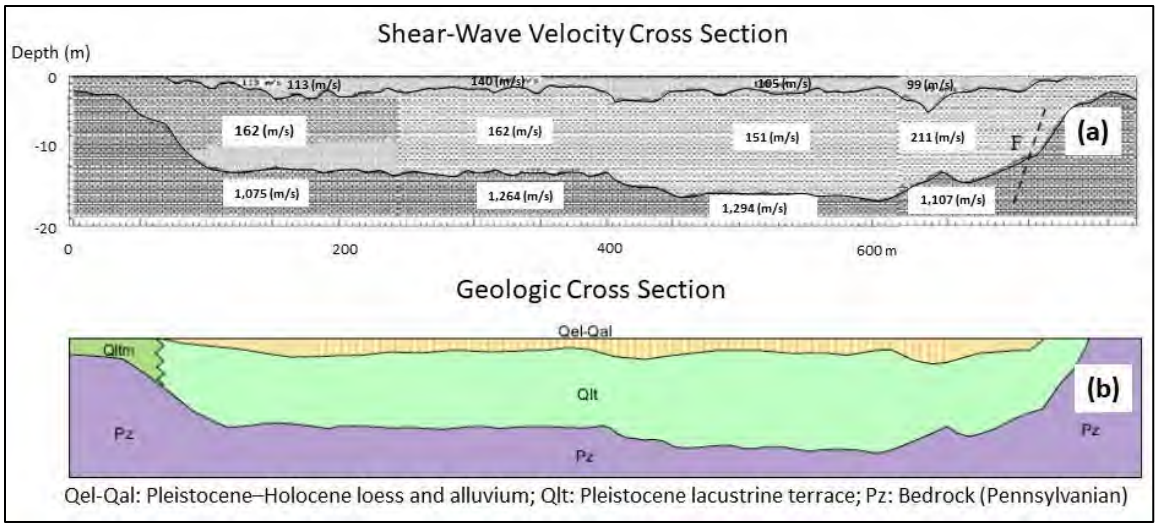

Figure 7. Shear-wave velocity cross section (a) and interpreted geologic cross section (b).

Summary

SH-wave reflection and refraction methods have been used by researchers at the University of Kentucky to obtain in-situ shear-wave velocities of sediments and bedrock in the central United States, the Ohio and Mississippi River Valleys in particular. The SH-wave is generated by horizontally striking a section of steel I-beam with a sledgehammer; it is recorded by horizontal component geophones. Different steel I-beams and hammers, geophone spacing, and shot intervals were optimized to acquire data based on the local geology and site conditions. In comparison with a conventional P-wave, which can propagate in water, an SH-wave can only propagate through the soil matrix of sediments. Thus, the SH-wave has the advantage in determining shear-wave velocities and sub-surfaces of water-saturated sediments. A database of shear-wave velocities of sediments and bedrock has been compiled at the Kentucky Geological Survey. This database has been used to delineate the major sediments and their distribution and potential ground-motion amplification in many communities in Kentucky. This database is also of interest for environmental and engineering purposes.

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Use of Geophysical Methods for Geotechnical Site Characterization

Michael E. Kalinski, Ph.D., P.E.[1]

Abstract

Geotechnical site characterization is necessary to measure the mechanical, deformational, dynamic and hydraulic properties of soil for engineering analysis, and to predict the behavior and performance of soil under various design conditions. Traditional methods of geotechnical surveying include soil sampling, mapping of soil and rock structure and stratigraphy, in situ soil testing and laboratory testing of remolded and undisturbed samples of soil. These traditional methods have been successfully developed, refined and applied over the past decades as means to acquire the necessary parameters for engineering analyses. However, there are instances where physical or economic constraints restrict the breadth over which a traditional geotechnical survey can be performed. In these instances, geophysical methods may be useful as a supplement to traditional geotechnical methods. Geophysical techniques are methods where the physical properties of the earth are measured, analyzed and interpreted to infer subsurface conditions and characteristics. Geophysical surveys performed in conjunction with traditional geotechnical surveys can reduce exploration time and cost associated with geotechnical testing and serve as a more direct measure for soil properties that cannot be directly measured using traditional geotechnical methods. Herein, a summary of the various geophysical methods that are commonly applied to geotechnical site characterization are provided.

Introduction

Geotechnical site characterization is the practice where the geotechnical properties of soil and rock are measured at a site. Geotechnical engineers measure these properties to predict the behavior of the soil under design conditions. Design conditions may consist of loading of soil with a surface structure, exposing the soil to earthquake motion to measure the dynamic and cyclic behavior of the soil, constructing an embankment where large shear stresses are imparted to the soil, or assessing hydraulic properties in the soil in instances where fluids are expected to move through the soil under a gradient. Information regarding soil and rock stratigraphy can also be obtained using a traditional geotechnical survey.

A geotechnical survey may consist of multiple soil borings per acre, with soil borings extending up to 100 ft or more in depth. Soil borings locations may be determined based on the location of structures to be built within the footprint of the site. They may also be controlled by access restrictions and situated to

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provide uniform coverage over a site. Soil borings typically include well logging, in situ standard penetration testing (SPT), undisturbed and disturbed sampling, and laboratory testing of recovered soil samples. For a typical site, the cost associated with the survey may easily run into the 10s of thousands of dollars.

Geophysical methods can be used as a supplement to geotechnical methods in many instances (USACE, 1995; Reynolds, 1997; ASCE, 1998; FHWA, 2003). Geophysical methods cannot replace geotechnical methods, but can be applied to reduce costs by reducing the number of borings needed. They can also be used as an effective interpolation tool to provide insight and information regarding soil conditions between soil borings. In some instances, geophysics can be used to provide soil characteristics that cannot be obtained using traditional geotechnical methods. In this paper, geophysical methods that are commonly used for geotechnical site characterization are discussed, along with their applicability and limitations.

Common Geophysical Methods for Geotechnical Site Characterization

Soil and rock possess physical properties related to its stiffness, density, electrical conductivity, magnetic susceptibility, and dielectric behavior. Geophysics is the measurement of these properties remotely from the ground surface. Geophysical data provides information about these physical properties that is interpreted to infer spatial variations in the subsurface characteristics of the soil and rock. In this manner, geophysical measurements can be used to aid in geotechnical site characterization.

Geophysical methods are traditionally categorized based on the nature of the physical property being measured. Geophysical categories and methods that are commonly used for geotechnical engineering include 1) seismic, 2) electrical, 3) electromagnetic and 4) potential field methods. These methods are described in the following paragraphs, including a discussion of the physical principles, applicability and limitations.

Seismic Methods. Seismic methods are methods where seismic stress waves are generated and measured at the ground surface. Seismic waves may be generated using impulsive sources such as explosives, weight drops, or hammers. Steady-state sources of seismic waves, including specially designed vibratory trucks, may also be used.

When a seismic wave is generated at the ground surface, its energy partitions into surface wave energy that propagates along the ground surface and body wave (P- and S-wave) energy that travels into the ground, where it reflects and refracts off of soil and rock layers before returning to the ground surface. At the ground surface, the seismic wave energy is measured using an array of geophones, which convert ground motion into voltage using a magnet and electrical coil.

Common seismic surface wave testing methods include the Spectral-Analysis-of-Surface-Waves (SASW) method, the Multichannel-Analysis-of-Surface-Waves (MASW) method, and the Refraction Microtremor (ReMi) method. These methods differ in the amount of equipment deployed on the ground surface and the nature of the seismic source. The SASW and MASW methods use seismic waves that are actively generated by the geophysicist, while the ReMi method exploits ambient surface waves that exist as background vibrations. The SASW method utilizes two geophones (Fig. 1), while MASW utilizes larger arrays consisting of dozens of geophones (Fig. 2). Data acquisition using the ReMi method uses an array similar to the geophone array shown in Fig. 2

Seismic surface wave dispersion curves derived in the field are inverted to develop one-dimensional soundings that quantify variations in shear stiffness with Depth (Fig. 3). Regardless of the method, the resulting shear wave velocity sounding is similar, although advanced methods of processing MASW data also allow two-dimensional shear wave velocity profiles to be generated. Seismic surface wave methods are a useful tool for quantifying soil and rock layering, estimating seismic site class and estimating soil and rock stiffness in terms of shear modulus. This information is particularly valuable for calculating seismic site response and assessing the susceptibility to soil to liquefaction.

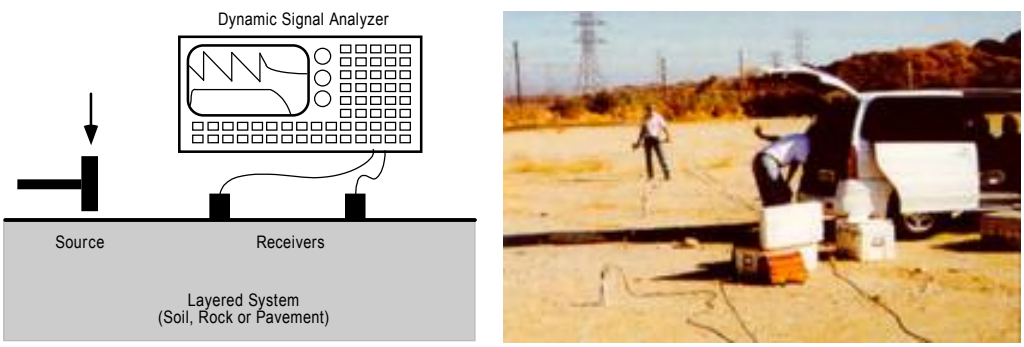

Figure 1. Typical array for performing SASW testing in the field (left) and SASW data acquisition at the OII Landfill in California (right).

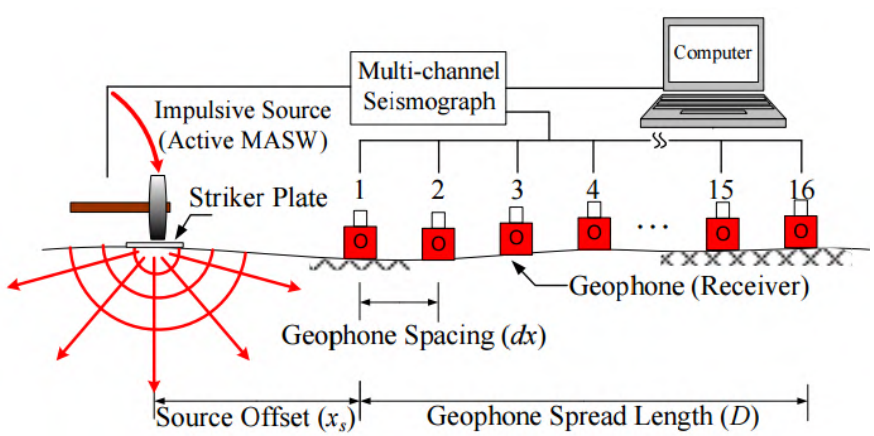

Figure 2. Typical array for performing MASW testing in the field.

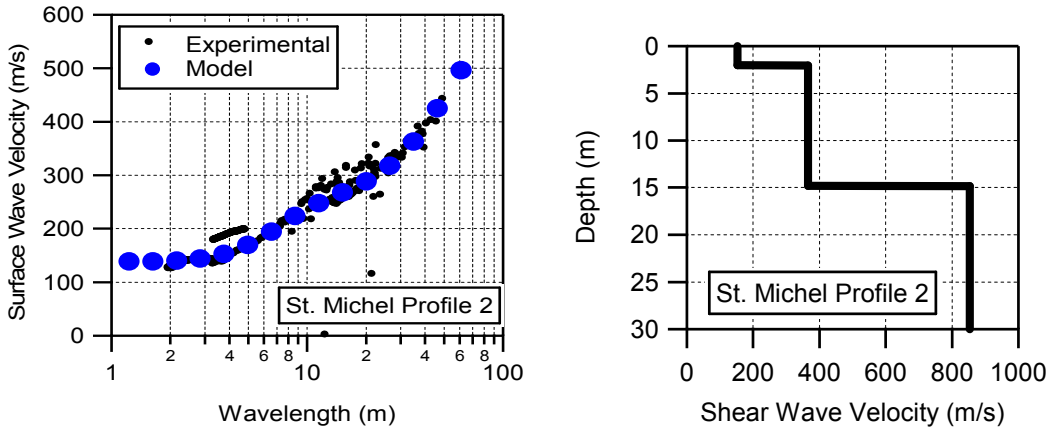

Figure 3. Typical surface wave data, including a surface wave dispersion curve (left) and inverted shear wave velocity sounding (right).

Seismic refraction methods were originally developed in the mid-20th century to explore for oil and gas, but have gained in popularity as an engineering tool in the past 20 years due to the proliferation of multiple-channel seismic recording systems that can be operated using a laptop computer. These methods typically involve the deployment of dozens of sensors placed in the ground at regular intervals similar to the array depicted in Fig. 2. Most surveys rely on the measurement of P-waves, but some techniques allow for S-waves to be measured. Refraction surveys rely on the measurement of travel times of seismic waves as they refract off of deeper subsurface layers and return to the ground surface. The resulting data are processed to generate two-dimensional profiles that allow for soil stiffness to be quantified (Fig. 4). Processing methods range from simple back-of-the-envelope type calculations to sophisticated computer tomography inversion methods. The resulting profiles can be interpreted to identify features such as faults, pinnacles, depth to bedrock and undulating strata.

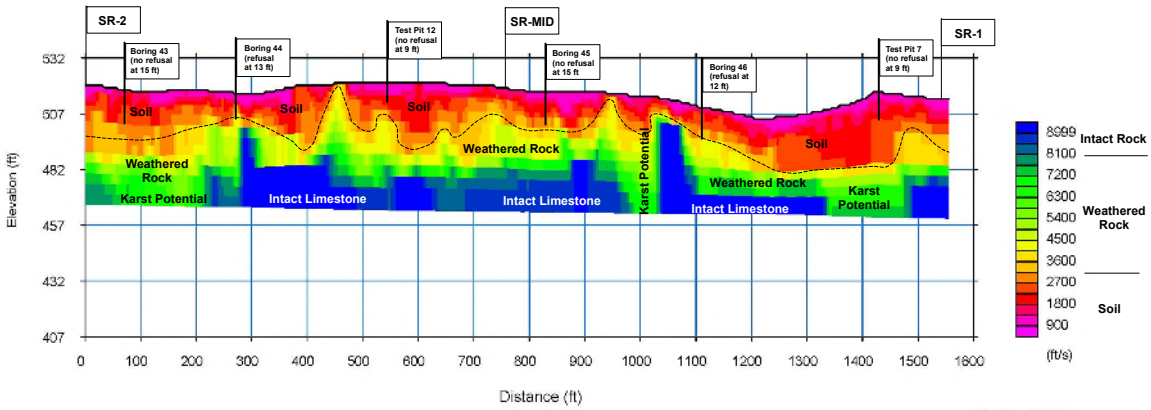

Figure 4. Two-dimensional profile showing variations in P-wave velocity derived from inversion of refraction seismic data at a site in Bowling Green.

Electrical Methods. Electrical direct-current (DC) resistivity sounding is an established method for estimating variations in electrical resistivity with depth in a layered soil or rock system. The method has traditionally employed the use of four electrodes placed in a straight line (Fig. 5). Current is passed through two outer electrodes while voltage is measured across two inner electrodes. Voltage measurements taken with small electrode spacings are affected by the electrical resistivity of the shallower strata, while measurements taken with larger spacings are affected by the properties of the deeper strata. By varying the position of the electrodes, the measured voltage also changes and apparent resistivity (a function of measured voltage divided by applied current), ρ_a, is plotted versus electrode spacing.

Apparent resistivity is a qualitative indicator of variations in resistivity with depth. Through inversion, these variations are quantified to develop a sounding of true electrical resistivity versus depth (Fig. 6). The one-dimensional sounding may be interpreted to infer subsurface conditions. Since the bulk electrical conductivity of soil and rock is mostly controlled by the presence and salinity of pore water, DC resistivity soundings are a valuable tool for ascertaining subsurface groundwater conditions, including the presence and quality of groundwater.

In the past 20 years, traditional four-electrode systems have given way to sophisticated systems consisting of dozens of electrodes where measurements are automated and data are inverted to create two-dimensional resistivity profiles (Fig. 7). In the field, these state-of-the-art systems are relatively equipment-intensive and typically consist of large batteries, a control box, a switch box, cables and dozens of electrodes.

A simple 4-electrode DC resistivity sounding can be completed at a site in less than one hour and the data can be inverted more or less instantly to derive an electrical resistivity sounding. Use of a multi-channel system may require half a day to deploy the electrodes and complete the field measurement, but inversion of the data can be performed in a matter of minutes.

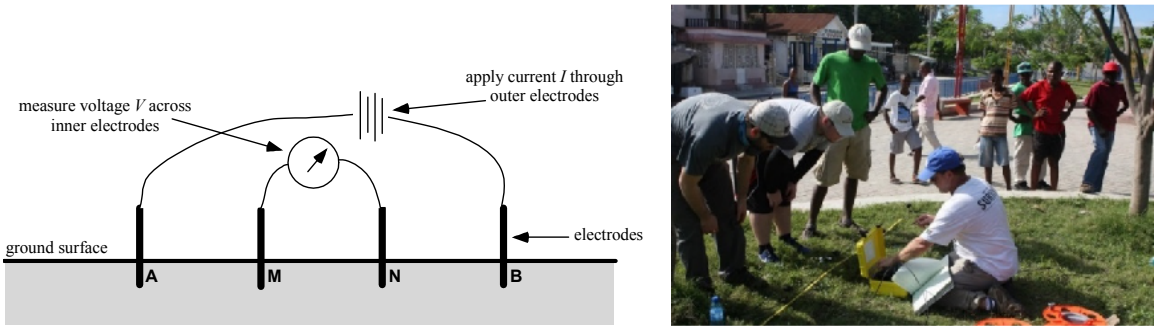

Figure 5. Wenner array configuration used in a traditional four-electrode DC resistivity survey (left) and acquisition of resistivity data in Haiti (right).

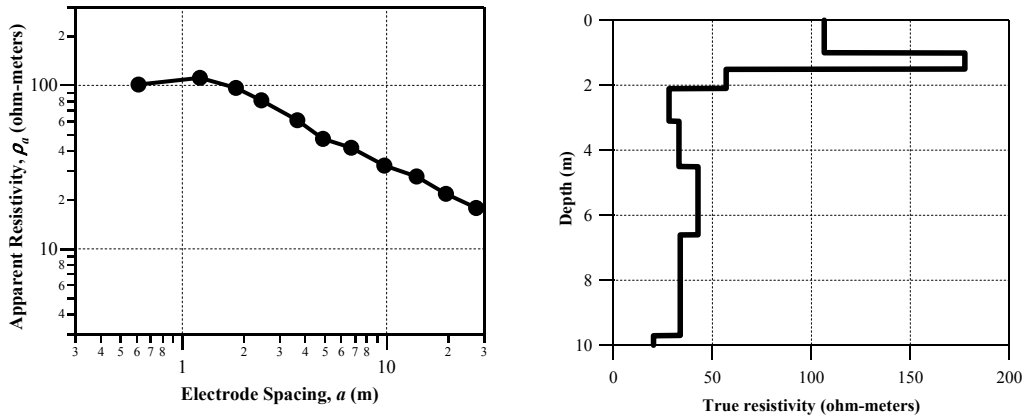

Figure 6. Typical DC resistivity data, including variations in apparent resistivity with electrode spacing (left) and the inverted electrical resistivity sounding (right).

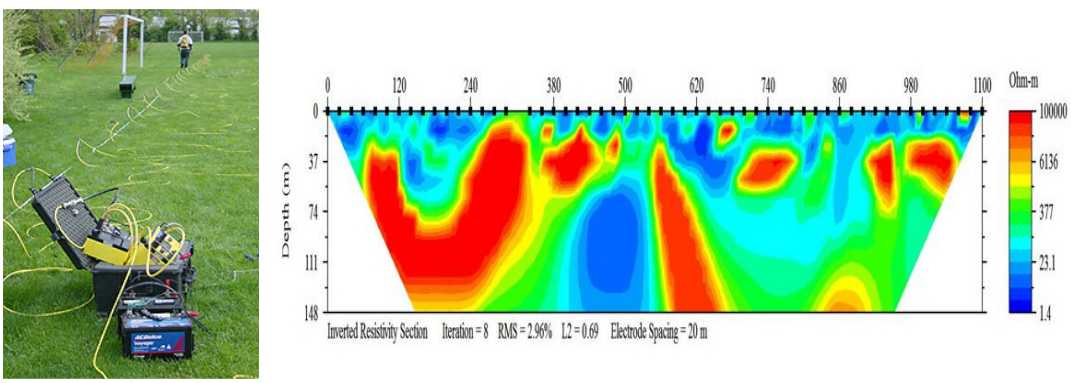

Figure 7. Multiple-channel DC resistivity geophysical system (left) and electrical resistivity profile derived from multiple-channel testing at a site in Thailand (right).

Electromagnetic Methods. Electromagnetic methods are methods where the response of the earth to an external electromagnetic field is measured. Earth response to an electromagnetic field is primarily dependent upon the electrical conductivity of the near-surface soil or rock, so electromagnetic methods are used to quantify variations in electrical conductivity in the subsurface. Electromagnetic methods have been successfully used for a number of geotechnical applications, including:

- Estimation of pore water salinity;
- Detection of subsurface voids and karst features;
- Characterization of soil stratigraphy;
- Delineation of landfills;
- Assessment of borrow materials;
- Estimation of depth to bedrock;
- Characterization of bedrock fracture patterns; and
- Contaminant plume mapping.

Frequency-domain electromagnetic (FDEM) testing is a method where response of the earth to electromagnetic waves with frequencies from 100s of Hz to 10s of

kHz is measured. It is performed using a transmitter and receiver coil spaced a distance *s* apart. The coils are coplanar, and can be oriented either horizontally (vertical dipole mode) or vertically (horizontal dipole mode) as shown in Fig. 8. The transmitter coil is excited with a harmonic electrical signal. The oscillating signal induces an electromagnetic field in the subsurface which is detected by the receiver coil. Depth of penetration of an electromagnetic wave is inversely proportional to the frequency of the wave and the strength of the received electromagnetic signal is proportional to the electrical conductivity of the underlying soil. Using these two relationships, the relationship between conductivity and depth can be derived, and FDEM testing can be used as a sounding tool to quantify variations in electrical conductivity with depth.

Frequency-domain electromagnetic testing can also be used as a tool to identify buried conductive objects (e.g. drums). By gradually decreasing the operating frequency, the frequency at which an anomaly appears can be associated to the depth to the anomaly using the concept of skin depth.

Historically, FDEM testing has been performed using coils at varying spacings. Coils were tuned to a specific frequency. Each spacing corresponded to a specific frequency and a different set of coils was used for each spacing. However, improvements to equipment have led to systems that consist of a pair of coils spaced a few meters apart. Increased dynamic range, improved primary-field rejection algorithms, and use of coils with very high tuning frequencies, have allowed such instrumentation to be developed. This new approach to FDEM testing is an improvement because equipment is more portable (weighing around 10 pounds) and data acquisition is much faster (10,000 field measurements per hour, which each measurement containing a full bandwidth of information for derivation of a sounding). One limitation of the newer systems is that they can only characterize σ to a depth of around 50 m, while older systems can characterize σ to a depth on the order of hundreds of meters.

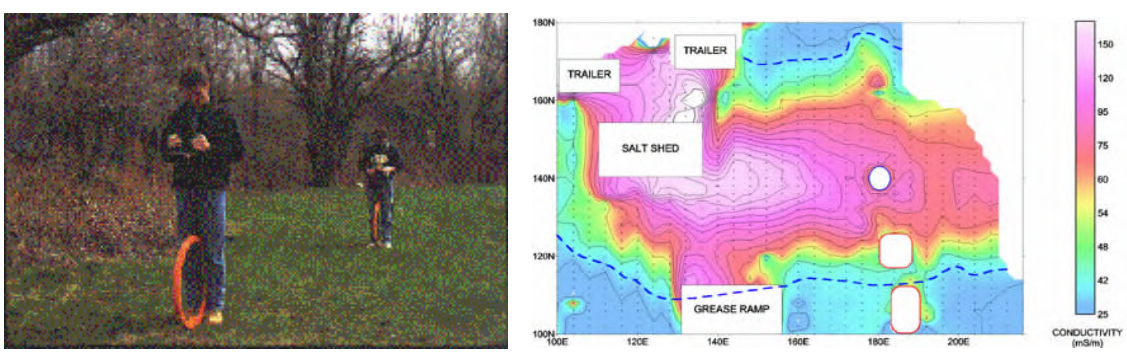

Figure 8. Frequency-domain electromagnetic (FDEM) testing (left) and typical data showing a plume of brackish groundwater in pink (right).

Terrain conductivity testing is a special type of FDEM testing that involves the use of a transmitter and receiver coil with a fixed spacing. The field acquisition configuration is similar to that used for FDEM testing. By driving the transmitter

coil at a low frequency (on the order of 1-10 kHz) such that the skin depth is much greater than the coil spacing, the voltage generated in the receiver coil is proportional to the average electrical conductivity of the near-surface material. Under these conditions electrical conductivity can be estimated based on the strength of the electromagnetic signal detected in the receiver coil. The electrical conductivity measured using the terrain conductivity method represents the average conductivity of the near-surface material to a depth that is approximately equal to the coil spacing of the instrument, which is typically around 2 m.

Terrain conductivity is a rapid method for acquiring large amounts of data with little data reduction effort (Fig. 9). Field equipment is calibrated to directly read in units of conductivity, and measurements are made instantaneously at the push of a button. Field equipment is highly portable and typically resembles PVC pipe a few meters in length. However, the method does not allow variations in conductivity with depth to be quantified.

Ground penetrating radar is a method that employs the use of high-frequency (on the order of MHz) electromagnetic energy. A typical GPR acquisition system consists of a controlling device, a transmitter antenna and a receiver antenna, which are typically mounted in a rolling cart (Fig. 10). To perform GPR testing, a transient electromagnetic pulse is transmitted down into the ground by the transmitter antenna. When the pulse encounters an interface between two materials possessing different dielectric constants, part of the pulse is reflected back to the ground surface where it is detected by a receiver antenna. By assessing variations in two-way travel time (i.e. the time it takes for the pulse to travel from the transmitter antenna to the reflector and back to the receiver antenna) and reflection magnitude (i.e. absolute amplitude) along a profile, anomalous zones indicative of subsurface features are readily identifiable.

Figure 9. Acquisition of terrain conductivity data in Bowling Green (left) and a typical terrain conductivity map showing anomalies at a landfill.

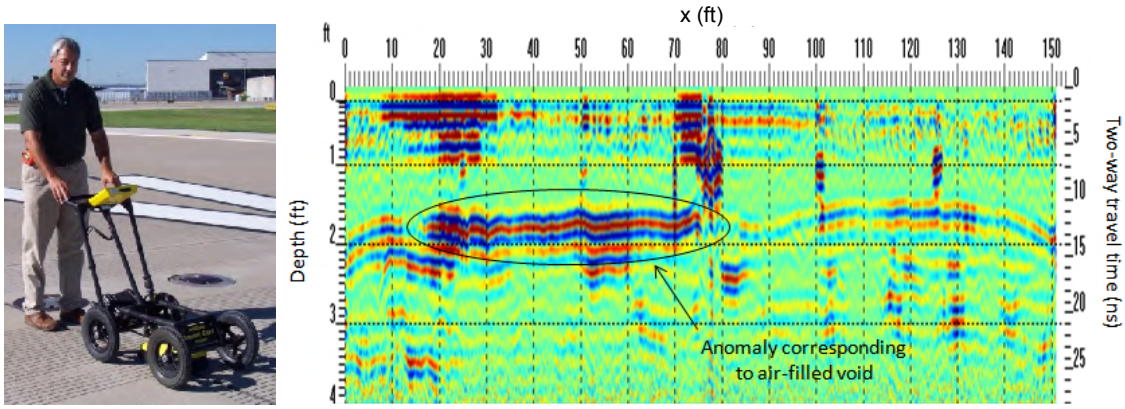

Figure 10. Acquisition of GPR data at the Louisville Airport (left) and a typical GPR profile revealing a hidden void beneath the runway (right).

Potential Field Methods. Potential field methods are methods where a value within a potential field, either gravitational or magnetic, is measured. Potential field measurements are influenced by anomalies in the earth near the point of measurement. The strength of the measured gravitational field is affected by variations in density, so subsurface void features such as karst may be detectable using microgravity geophysical surveying. Microgravity surveying is performed using a gravimeter (Fig. 11). A gravimeter contains a weight suspended by a sensitive spring. When placed on the ground to making a reading, the gravitational force imposed on the weight is related to the density of the material beneath the gravimeter. Minor changes in the amount of deflection of the spring are measured, and are used to calculate the strength of the earth's gravitational field. There are numerous other factors that affect the measured strength of the gravitational field, including latitude, elevation, tides, topography and instrument drift, so analysis of gravity data is highly specialized. When properly analyzed, the result is a map showing localized areas of high and low gravity that can be interpreted to infer subsurface conditions, including the presence of subsurface voids.

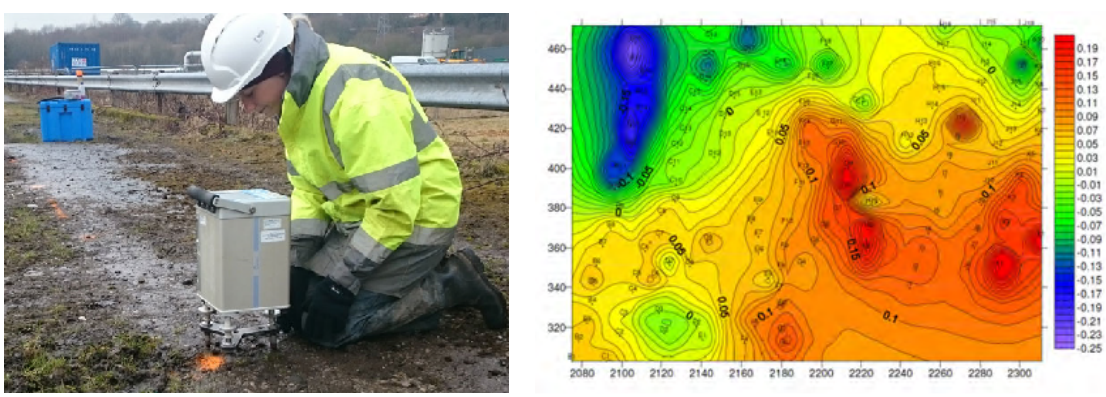

Figure 11. Acquisition of gravity data using a gravimeter (left) and a typical map showing gravity anomalies indicative of variations in the subsurface (right).

Magnetometer surveys allow for variations in the earth's magnetic field to be quantified. When ferrous objects exist in the magnetic field, they distort and concentrate the flow lines of the field. These distortions appear as zones of high magnetic susceptibility, which can be detected using a magnetometer. Most magnetometers operate on the principle of proton precession. A magnetometer contains a hydrogen-rich liquid such as kerosene. The liquid is subjected to an artificial magnetic field generated in the magnetometer. The protons in the hydrogen align themselves in the artificial field. When the artificial field is switched off, the protons begin to precess like a spinning top as they go out of alignment. The precession generates a secondary magnetic field that is measured in the magnetometer. The frequency of the precession is controlled by the strength of the earth's magnetic field, so detection of the precession frequency in the instrument is related to magnetic susceptibility.

Magnetometer surveys are typically performed by walking over a grid (Fig. 12). Readings and GPS locations are constantly logged as the surveyor walks so that a grid of very dense readings can be generated. The readings can then be mapped to identify anomalies indicative of subsurface features.

Summary

Geophysical methods can be used to provide information that is useful for geotechnical design. Geophysics should not be considered as a replacement to traditional geotechnical surveys, but as a supplement that may help to reduce the size and cost of the geotechnical survey. Geophysics can also provide information that cannot be directly obtained using geotechnical measurement. A number of geophysical methods were discussed in this paper. These represent the more commonly applied methods for geotechnical characterization, but do not represent all of the geophysical methods that are in use today.
The methods described herein are summarized in Table 1, along with their applicability, depth limitations and practical limitations.

Before performing geophysical testing, it is always helpful to perform some sort of modeling to verify that the method will work. Modeling can range in sophistication to back-of-the-envelope calculations to numerical analysis to assess whether or not it is feasible to assess a site using a particular geophysical technique.

Keep in mind that resolution almost always decreases with depth. You may be able to resolve a 5-ft layer if it is 15 ft below the ground surface, but it is not likely that you will be able to resolve the layer if it is 100 ft deep. Be realistic in your expectations and perform modeling to assess the feasibility of resolving deeper, thinner layers.

Costs for a geophysical survey are typically around a few thousand dollars per day, which includes field testing, analysis of the day's data and reporting. Small

projects can typically be completed in one day, while larger projects may take longer. Costs may be affected by the availability of equipment, particularly when expensive, specialized equipment (e.g. gravimeters) must be secured to complete the field testing.

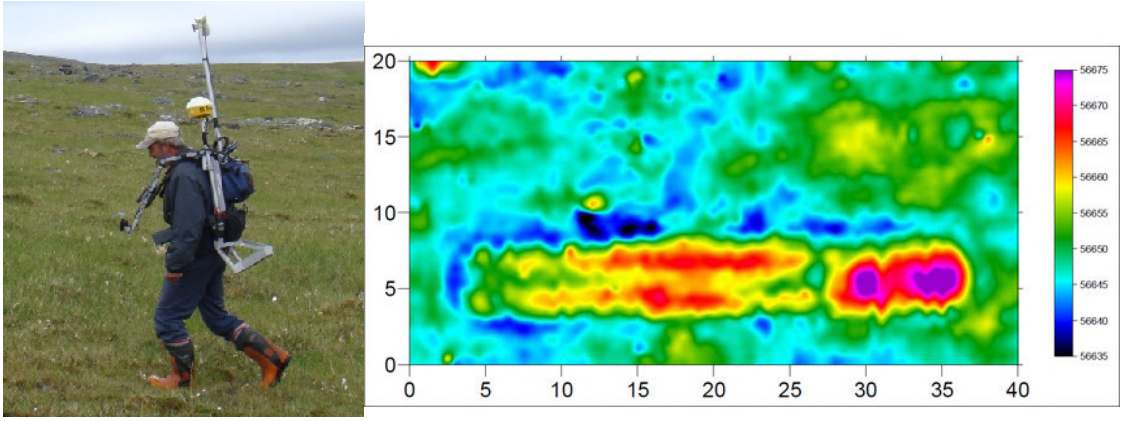

Figure 12. Magnetometer surveys conducted on foot (left) and a typical map showing magnetic anomalies (right).

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Table 1. Geophysical methods, their typical applicability and limitations

Geophysical Method	Site Characterization Objective							Typical Depth Limit	Limitations
	Depth to bedrock	Soil/rock stiffness	Depth to water table	Groundwater contamination	Void detection	Metal detection	Buried object detection		
Surface Wave Seismic	X	X						<200 ft	Need large source for depths >50 ft
Refraction Seismic	X	X						<200 ft	Need long array with lots of geophones
DC Resistivity	X		X	X				<200 ft	Need long array for depths >50 ft
Frequency-Domain EM				X		X		<100 ft	Inversion software needs improvement
Terrain Conductivity				X		X	X	<20 ft	Measures average of upper 20 ft
GPR					X		X	<50 ft	Ineffective in clay and water
Microgravity					X		X	<100 ft	Fragile, expensive and easy to generate bad results
Magnetometer						X	X	<50 ft	Small, deep anomalies undetectable

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USING UNMANNED AERIAL SYSTEMS AND PHOTOGRAMMETRY TO REMOTELY ASSESS LANDSLIDE EVENTS IN NEAR REAL-TIME[1,2]

Abstract:
Commercially-available unmanned aerial systems (UAS) and photogrammetry software have undergone rapid advancements in recent years. However, the use of UAS and photogrammetry techniques for monitoring surface landform deformation has not been adopted for the most part due to complicated workflows and complex UAS systems. This study demonstrates the ability to monitor landslides in near-real time with commercially-available UAS and photogrammetry software using direct georeferencing and co-registration techniques. The results of this research were then assessed to develop an optimal workflow for the rapid assessment of surface deformations with direct georeferenced UAS obtained imagery and photogrammetry software.

1 Introduction

Close range photogrammetry using an unmanned aerial system (UAS) platform has been shown to be effective for slope assessments and surface monitoring of unstable slopes. (Niethammer et al. 2012; Lucieer et al. 2013; Rothmund et al. 2013; Turner et al. 2015) However, current photogrammetry practices require that ground control points (GCPs) be placed and surveyed around or within the slope failure to obtain accurate and georeferenced measurements. Fortunately, recent UAS advancements and the miniaturization of electronic sensors such as GPS, inertial measurements units (IMU), and lightweight high-resolution imagers have shown potential in allowing for the acquisition of georeferenced photogrammetric models, without the need of GCPs. (Tsai et al. 2010; Turner et al. 2014; Yildiz and Oturanc 2014)

The ability to georeference photogrammetric models without the use of GCPs is termed direct georeferencing while georeferencing with GCPs is termed indirect georeferencing. Turner et al. (2014) demonstrated the potential of direct georeferencing for producing accurate measurements without physically installing GCPs using high precision GPS equipment onboard a UAS. However, the methodology and equipment used for the known ways of direct georeferencing photogrammetric models, which has been demonstrated by Turner et al. (2014), rely on expensive UAS that are normally custom built to each specific application and are not intuitive enough for widespread adoption.

The general concept of this research was to use direct georeferencing with commercially-available UAS and photogrammetry software to effectively monitor and assess the stability and characteristics of a landslide without the use of GCPs. In this study, the relative accuracy of photogrammetric models was

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assessed using a UAS manufactured by DJI and Pix4D photogrammetry and flight logging software. To assess the relative accuracy of the computed direct georeferenced photogrammetric models, relative measurements were compared to indirect photogrammetric models and surveyed measurements in a controlled environment that simulated a real-world slope failure setting.

The relative accuracy of a photogrammetric model is defined in this study as the ability to obtain measurements within direct georeferenced models in relation to known survey measurements. Once the relative accuracy of the photogrammetric models has been determined, the viability of using commercially-available UASs to directly estimate that accuracy of important slope stability assessment parameters such as displacement, areas, and volumes can be assessed.

2 Site Description, Instrumentation, and Equipment

The project site used for controlled experimentation was chosen based upon: accessibility, minimal surface change, and the resemblance to a natural slope. The location that best satisfied these needs was the University of Kentucky's public amphitheater located behind Memorial Hall, shown in Figure 1.

Figure 1: Ground Based Imagery Depicting the Project Site Used for Controlled Experimentation

To check the relative accuracy of the direct georeferenced photogrammetric models, known features that are visible from UAS photography were surveyed using high precision equipment. GCPs were placed throughout the area of interest, and afterwards were surveyed with a Nikon DTM 322 total station in an assumed coordinate system.

In order to track surface changes of a slope failure over time, a feature, or set of features that are moving must be easily identifiable in the original and subsequent flights from the aerial imagery. In natural landscapes, these features can be anything from tree stumps to uniquely identifiable rocks. However, in the controlled experimentation simulated features and displacements were

necessary. The simulated displacements were performed by moving eight wooden blocks within the study area. The average dimensions of the wooden blocks were 4 cm x 9 cm x 30 cm and each block contained a black and orange target with a diameter of 5 cm.

The UAS used for this study was a DJI PHANTOM 3 Professional (P3P) multi-rotor quadcopter. The P3P UAS utilizes both GPS and GNSS positioning units for flight tracking and automation. The P3P UAS has a 12.4-megapixel camera with a field of view (FOV) of 94 degrees. Flight planning was completed using a cell phone running the Pix4D Capture application. The Capture application was used to program the desired flight path and monitor the progress of each flight in real-time. The Capture application was also used for automated collection of direct georeferenced UAS imagery.

3 Methodology

The locations of features within this controlled site were surveyed to determine the precise relative locations of GCPs for comparisons with UAS obtained measurements. This was accomplished by creating an arbitrary coordinate system using temporary benchmarks. A total of 23 ground control points were placed throughout the area of interest for quality control and accuracy checking.

Wooden blocks were used for simulated displacements within the slope. Each block was randomly placed throughout the amphitheater. This random location will be referred to as the upslope block position. Once the upslope block positions were determined, the block locations were surveyed, and the site was flown for photogrammetric data by the UAS. The wooden blocks were then moved downslope to simulate displacement, the downslope block locations were surveyed, and the site was flown by the UAS again.

A total of four flights were performed for the controlled experimentation. Two of the flights were flown during the initial positions of the blocks and the other two flights were flown after the simulated displacement of the wooden blocks. All four of the flights completed for this experimentation used the same amount of overlap (90%), speed (medium fast – approximate 20 kilometers per hour), and approximate coverage area (approximately .75 hectares). Therefore, the flight paths for the remaining three flights were similar to the flight path shown in Figure 2.

Figure 2: Screenshot of the Pix4D Capture Android Application Showing the Grid and Flight Parameters used.

Once the programmed flight paths were completed, the images and file created using the Pix4D Capture application were transferred from the cell phone and the micro SD card to the processing computer. Each flight was processed individually using the default settings within Pix4D. Processing of UAS captured imagery using Pix4D software enables the generation of three main geospatial data products; orthomosaic, Digital Surface Model (DSM), and 3D geo-positional data of terrain features (i.e. point cloud).

In order to assess the relative accuracies of direct georeferenced measurements such as distances, areas, and volumes, the “true” measurements must be known. While the distance and area measurements can be directly compared to the surveyed GCPs, the volume of the models cannot be assessed by simply using the survey points. Therefore, the indirect georeferencing of a single flight was completed using six GCPs to compare the results found from direct georeferencing. The measurements found in the indirect georeferenced model were considered the base measurements and were used to quantify accuracy parameters of the direct georeferenced models.

When consecutive direct georeferenced photogrammetric models were computed, the errors in the elevation, rotation, and translation restricted the ability to compare the same feature or measurement in multiple models. An example of the misalignment of two combined direct georeferenced flights can be seen in Figure 3. The misalignment shown in Figure 3 shows two separate point clouds of the same area that are not in the same relative coordinate system. Thus, to compare multiple models, the secondary models must be co-registered and aligned to the same feature points. Co-registration involves identifying features or “tie points” that are common in each model.

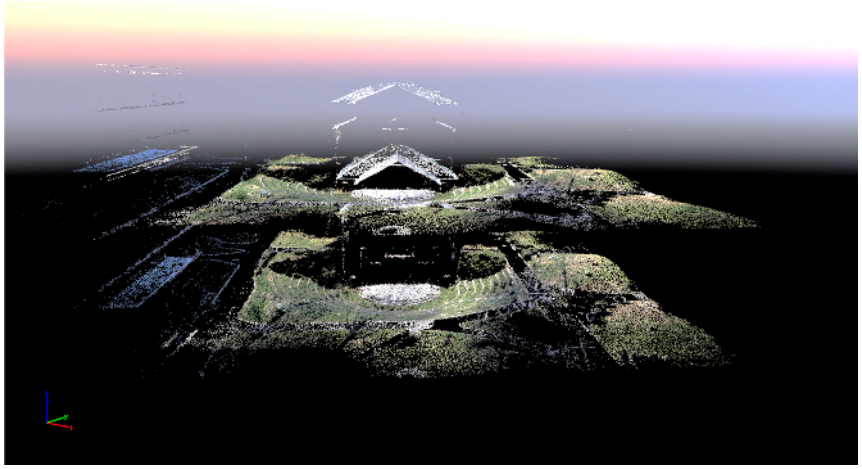

Figure 3: Misalignment of Two Direct Georeferenced Point Clouds

Factors that can affect the ability to co-register each flight are the amount of sunlight on the surface (time of day and weather), vegetation height and color, flight height, and more. For this experimentation, the flights occurred within a short time period and, therefore, minimized the variation of conditions within the area of interest. The area of interest was also considered static besides the simulated displacements. The co-registration of each individual model was accomplished using six (6) of the 23 GCPs at the site.

Although this study used “known” control points to georeference the two separate flights, the GCPs used for co-registering could have been be any unique feature that was identifiable in all of the flights being compared. Examples of other features that could be used to co-register two models include sidewalk control joints or the corner of a roof.

After the GCPs were identified, the GCP coordinates from one of the direct georeferenced models (reference model) were imported to each comparison model. The GCPs in the comparison models were marked and assigned the coordinates from the reference model. The comparison models were then indirectly georeferenced to the reference model coordinates. This process was completed for the three co-registration flights and the resulting accuracies of the co-registration produced by this method was quantified. The co-registration accuracies of each flight were found by comparing various measurements such as distance, area, and volumes.

4 Results

To quantify the accuracies found from direct georeferencing, measurement comparisons were completed. Distance, area, and volumetric comparisons were completed to provide an estimate of the expected accuracy of relative measurements from direct georeferenced models. A co-registration accuracy quantification was also completed after co-registering the four direct georeferenced flights. The co-registration accuracy assessment estimated the

additional inaccuracies that are introduced from the co-registration of multiple flights. Lastly, a surface monitoring accuracy quantification analysis was completed which included the comparison of measurements in co-registered photogrammetric models to the results of the total station measurements.

4.1 Direct Georeferencing Relative Accuracy Assessment

Volume and area measurements developed from 10 outer perimeter GCPs at the top of the amphitheater was used to quantify the relative accuracy of direct georeferenced models. Relative measurements of the four direct georeferenced models were then compared to the relative measurements found from the indirect georeferenced model.

Area measurements were quantified to assess the X and Y planar accuracies of the direct georeferenced models. The results of the enclosed areas computed from direct georeferenced models and the indirect georeferenced model are shown in Table 1.

Table 1: Direct Georeferencing Accuracy Quantification

Model	Total Volume (m^3)	Volume Percent Difference	Enclosed 3D Area (m^2)	Area Percent Difference	$D_{average}$ (m)	$D_{average}$ Percent Difference
Model 2 Indirect	-727.2	N/A	582.1	N/A	-1.25	N/A
Model 1	-772.3	6.2%	562.3	3.39%	-1.37	9.9%
Model 2	-896.1	23.2%	569.7	2.13%	-1.57	25.9%
Model 3	-762.7	4.9%	592.6	1.81%	-1.29	3.0%
Model 4	-701.3	3.6%	551.9	5.17%	-1.27	1.7%

Vertical accuracy of a photogrammetric model is inherently less than the horizontal accuracy (Bernhard Draeyer, 2014 and Strecha et al., 2012). Typically, the horizontal accuracy is considred to be 1 to 2 times the ground sampling distance (GSD) while the vertical accuracy is approximately 2 to 3 times the GSD. The additional uncertainty in the Z-axis is difficult to quantify for relative measurements but can partially be assessed by normalizing the volume by the volume base area. If the volume and area are known, the average volume height can be found by Equation 1 where V is the volume and A is the base area of the volume.

$$D_{average} = \frac{\mathrm{V}}{\mathrm{A}} \tag{1}$$

The computed volume, area, and the average volume height ($D_{average}$) of each model is shown in Table 1. The results of this comparison show that the volume and $D_{average}$ percent differences of each model computed vary more than the area quantifications. The percent difference of the volume ranged from 3.6 percent to 23.2 percent, with a mean absolute deviation of ± 9.5 percent with respect to the

indirect reference model. The percent difference of $D_{average}$ ranged from 1.7 percent to 25.9 percent, with a mean absolute deviation of ± 10.1 percent. The similar discrepancies observed between the $D_{average}$ and the volume calculated for each flight indicate that the inaccuracies were in the Z-axis instead of the X-axis and Y-axis. Since the UAS based nadir photogrammetry is inherently less accurate when computing the Z-axis coordinate of an automatic tie point, the larger error in the Z-axis was not unexpected.

4.2 Co-Registration Accuracy

The previous accuracy assessment demonstrated that while three of the direct georeferenced models had relative measurement accuracies similar to the actual measurement, there is still the possibility for large scale measurement errors when no corrections are applied. This makes it difficult to quantify change over time. However, as previous researchers (Lucieer et al. 2013; Immerzeel et al. 2014; Turner et al. 2015) have shown, co-registration can eliminate much of the variability when comparing two or more models that have large inaccuracies. The hypothesis of the co-registration accuracy assessment assumes that if a direct georeferenced model has an acceptable relative accuracy, then the location of identifiable points can be determined and used as 3D GCPs to co-register subsequent models for comparative analysis. For this assessment, the Flight 3 model was chosen as the reference model since Flight 3 had the highest relative accuracy.

4.2.1 Relative Measurement Accuracy Assessment

A relative measurement accuracy assessment was used to assess the co-registration of each of the models. However, instead of using an indirect georeferenced model, this assessment uses the co-registration reference flight to quantify the accuracy of the relative measurements. To complete the relative measurement accuracy assessment, the volumetric analysis of the amphitheater and the $D_{average}$ value beneath the volume base surface was calculated for the three co-registered models and compared to the reference model. The results of volume measurements beneath the base surface surrounding the perimeter of the amphitheater after the co-registration can be seen in Table 2.

Table 2: Volume and $D_{average}$ measurements after co-registration to Flight 3

Model	RMS Error (cm)	Total Volume (m^3)	Volume Percent Difference	Enclosed 3D Area (m^2)	$D_{average}$ (m)	$D_{average}$ Percent Difference
Flight 3 (reference)	N/A	-759.4	N/A	594.7	-1.28	N/A
Flight 1	1.2	-761.2	0.23%	594.9	-1.28	0.20%
Flight 2	2.1	-763.4	0.52%	594.8	-1.28	0.50%
Flight 4	1.2	-750.2	-1.21%	594.6	-1.26	-1.20%

The average percent difference of the volumetric measurements was observed to be ± 0.65 percent. This percent difference is much lower than the differences computed for the direct georeferenced flights which included an average difference of ± 9.5 percent. Furthermore, the volume of the model with the highest direct georeferencing Z-axis error (Flight 2) exhibited a high co-registration accuracy to Flight 3 and the Z-axis error was virtually eliminated when compared to the reference flight.

4.2.2 DSM Accuracy Assessment

Further analysis of the surface difference of the co-registered models was completed by importing the DSM outputs of each model into ArcMap 10.4. Once the co-registered DSMs were imported into ArcMap™ 10.4, the spatial analysis tool Cut Fill was used to determine the volume differences between each co-registration surface and the reference surface. The Cut Fill tool allowed for the normalization of the volume within the area of each cell to obtain the average distance between surfaces.

The cut and fill co-registration analyses were analyzed using the attribute table and the select by attribute function in order to quantitatively compare the results. The statistics provided by the cut and fill analysis such as the mean, standard deviation, maximum, and minimum were recorded and are presented in Table 3.

Table 3: DSM differences quantified by ArcMap 10.4 and a normalized cut and fill analysis.

	Classification Statistics			
Co-Registration Model	Mean (cm)	Standard Deviation (cm)	Maximum Difference (m)	Minimum Difference (m)
Flight 1	-0.7	2.4	0.67	-0.26
Flight 2	1.3	2.2	0.65	-0.2
Flight 4	-2.2	1.9	0.61	-0.19

Visual interpretation of the Cut Fill analysis shows the maximum surface displacements within the outer perimeter GCPs are near the terraces of the amphitheater. For the visual interpretation, the co-registration surface difference threshold was set as ± 3 cm. The small amount of error observed in the surface difference visualization can likely be attributed to the varying and limited resolutions of each DSM and the sudden height difference between each of the terraces. An example of this example of this maximum displacement is shown in Figure 4.

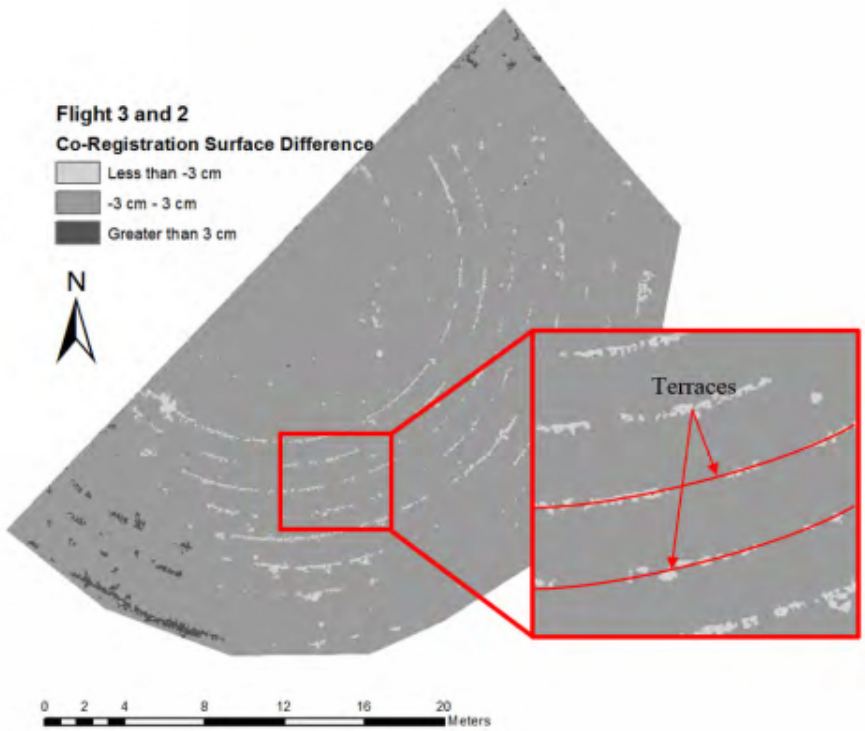

Figure 4: Surface differences of models produced from Flights 3 and 2 after co-registration

4.3 Surface Monitoring Analysis

Once the co-registration accuracy had been quantified, measurements of the simulated displacements were taken between the models produced by the co-registering of Flight 2 with Flight 3.

After an accurate co-registration between each model is completed, one way to measure the displacement across time is by marking the feature locations using a manual tie point (MTP) in each of the models being compared and then merging two or more of the models. After marking the location of the surface feature with an MTP and merging the two models, displacements between two photogrammetric models can be obtained. Otherwise, the displacements can be measured manually by using MTP coordinates and the Pythagorean theorem.

The location of each feature can also be identified after the merging of the projects, but for small displacements, it is hard to differentiate each feature location in the model. Therefore, when marking feature locations after the project is merged, careful consideration of which photos belong to which model must be made.

In this study, the distance between the upslope and downslope location within the direct georeferenced and co-registered model of Flights 2 and 3 was found.

The displacements found were then compared to the displacement measurements collected using total station surveying equipment. The direct georeferenced and co-registration based relative locations and displacements of the wooden blocks can be seen in Figure 5.

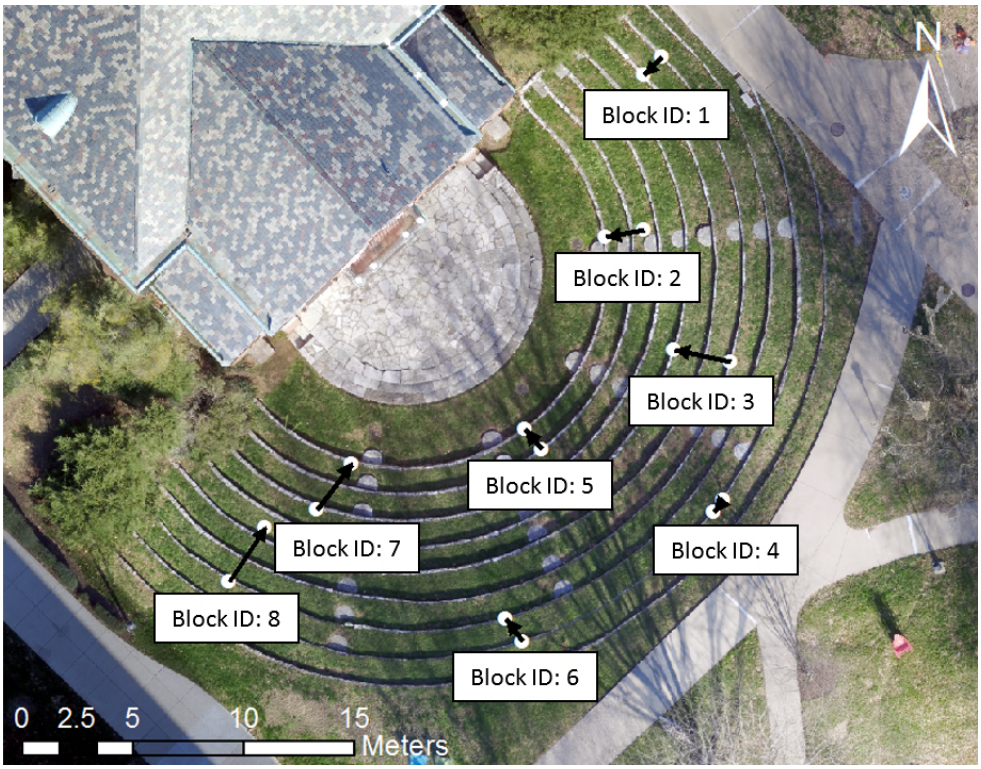

Figure 5: Orthomosaic of merged projects showing the simulated displacements identified in photogrammetric models and exported to ArcMap 10.4.

A full comparison between the UAS based displacements and the total station measured displacements can be found in Table 4.

Table 4: Simulated displacements of total station and UAS derived measurements.

Displacement	Total Station 3D Length (m)	Photogrammetric Terrain 3D Length (m)	Difference Between Photogrammetric and Total Station 3D Length (cm)	Percent Difference (%)
Block ID: 1	1.24	1.26	2.38	1.92
Block ID: 2	1.86	1.85	-1.31	0.70
Block ID: 3	2.71	2.71	-0.15	0.05
Block ID: 4	0.72	0.71	-1.06	1.48
Block ID: 5	1.24	1.24	-0.25	0.20
Block ID: 6	1.36	1.35	-1.12	0.82
Block ID: 7	2.66	2.69	3.13	1.18
Block ID: 8	3.10	3.12	1.84	0.59

The percent differences of each observed simulated displacement ranged from 0.05 percent to 1.92 percent. Additionally, the root mean square error (RMSE) of the observed photogrammetric model measured displacements is 1.7 cm. Some variation in the measurements could be from human error when identifying the MTPs. However, since the direct georeferenced co-registration reference model had X, Y, and Z scalar inaccuracies, the inaccuracies were most likely from the reference model.

5 Discussion

This study shows that direct georeferencing can be used to obtain relative measurements at an acceptable level of engineering certainty in ideal conditions. The direct georeferenced relative accuracies were generally consistent with the indirect georeferenced model in the X and Y-axis. While one of the controlled experimentation models had scalar inaccuracies of approximately 25 percent in the Z-axis. This large percent error in the Z-axis directly affected slope stability relevant parameters and measurements. However, after an accurate co-registration, relative measurements can be accurately obtained in relation to the reference model. While this approach is primarily a qualitative analysis for a navigation grade GPS, higher GPS accuracies could theoretically be able to perform highly accurate quantitative comparative analyses.

The accurate co-registration can allow for further analysis to be conducted in GIS, CAD, and other software with minimal noise or variance. The implemented co-registration technique directly within the photogrammetry software inherently allows for increased co-registration accuracies compared to other software packages due to the ability to mark the co-registration points using the images obtained by the UAS. Additionally, the pseudo-automatic co-registration can be completed in the early stages of photogrammetric processing which, significantly reduces the time needed to quantify change when compared to other methods. Therefore, the pseudo-automatic co-registration of subsequent models indicated that there is potential for near-real time slope stability monitoring with a commercially-available UAS and without the use of GCPs.

6 Near Real-Time Surface Monitoring Workflow

After assessing the results of the experimentation, the processes and procedure used for this research were used to develop an optimal workflow for the near real-time assessment of surface deformations with direct georeferencing and co-registration techniques. This workflow was developed with the intent of repeatability in a variety of project areas and environments but mainly focused on monitoring landslide and landform deformations. The equipment and methodology used for this workflow are commercially-available and do not require extensive knowledge in surveying, UAS piloting, or photogrammetry. Additionally, this workflow could be adapted to work with a variety of UAS platforms and photogrammetry software with similar features. The full workflow developed from the work presented in this research to assess landforms over

time using direct georeferencing with commercially-available UAS and photogrammetry software is presented in Figure 6.

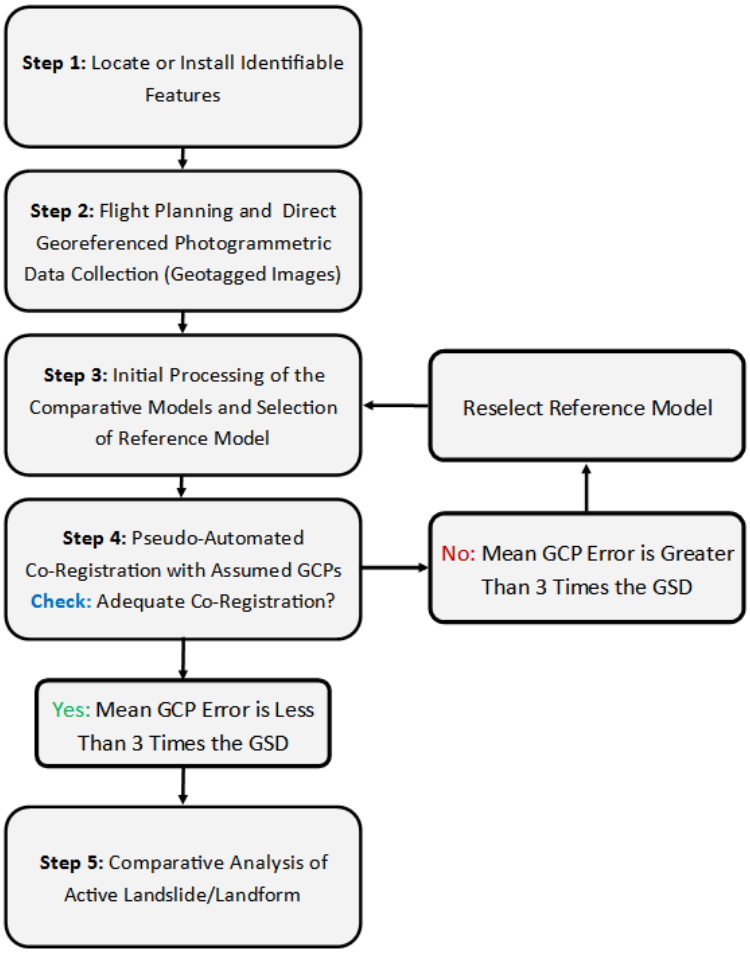

Figure 6: Workflow to assess landforms over time using direct georeferencing with commercially-available UAS and photogrammetry software.

7 Conclusions

With the recent emergence of commercially-available UAS technologies, the ability to obtain UAS based photogrammetric data has become a viable option for remote sensing. The results of this research showed that the use of commercially-available ready to fly UAS can provide valuable information such as displacement magnitude, direction, and other relative measurements without the use of survey equipment as long as there are identifiable points evenly distributed throughout each of the monitoring period models. By assuming that a single direct georeferenced flight is of acceptable quality (reference model), the co-registration techniques used in this research were able to minimize the variance of sequential time-series analysis. Thus, accurate displacement measurements can be obtained with a high relative accuracy without the need and risk of surveyors traversing an active landslide or landform and in areas

where the installation of other monitoring equipment such as inclinometers are not feasible.

While the scope of this research mainly explored the applications of direct georeferencing and co-registration in regards to surface tracking, the ability to accurately co-register two or more time-series photogrammetric models can be used in other engineering applications. Additionally, since the direct georeferencing photogrammetric models had high relative accuracies, the results of each comparison could be used for quantitative analysis.

Please note that while relative measurements can be performed with the aforementioned procedures and techniques, the workflow and procedures presented herein should be used with caution. Since the co-registration models can at best only be as accurate as the reference model, the reference model's relative accuracy becomes the limiting factor for other important slope stability parameters such as slope angle. If slope stability calculations are needed from direct georeferenced photogrammetric models, indirect georeferencing using GCPs with survey-grade precision and accuracy is recommended. Nevertheless, as the miniaturization of highly accurate positioning equipment continues to become commercially-available, the accuracy of slope stability relevant parameters such as slope angle and elevation change from direct georeferenced UAS imagery will continue to increase.

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NOTES

NOTES

Geotechnical site characterization of an un-conventional deep excavation project around a 21 Story Building [1,2,3]

This work focuses on the site investigation, design and construction aspects of a successfully completed deep excavation project in a densely developed urban area in the north west of Tehran, capital of Iran. The excavation was conducted to provide space for 4 basement levels for multiple buildings around the already functional 21 story Tooba tower. The excavation was done simultaneously on 3 sides of the roughly rectangular plan of Tooba tower to the depth of 16.5 meters below its foundation level or 26 meters below the ground surface. The retaining structure necessity for limiting the deformations of the tower consisted of contiguous bored reinforced concrete piles around the building supported at 4 levels with pre-tensioned tieback and wailing system. A monitoring program for measuring the deformations of the tower and supporting system was also enforced during and after excavation. The site consisted of a highly cemented soil. Besides the challenges of investigating soil properties of this cemented soil, working in a ground that the design team had to account for the possibility of encountering ancient water tunnels (Qanats) and boulders with diameters up to few meters. This manuscript is a revision of two papers already published at the 7th international conference on case histories in geotechnical engineering, 2013. In this manuscript the project is revisited with a focus on geotechnical exploration aspect of the project and an additional section on soils in the US with similar properties; some other parts of the manuscript are simply reproduced from the mentioned papers.

Introduction (Project Definition and Geometry)

The excavation project discussed in this paper is located at 35° 45’ 58.41” N, 51° 22’ 12.58” E in a densely developed urban environment in North West of Tehran, capital of Iran, where 4 levels of parking spaces are to be accommodated under ground level.

Figure 1 shows the excavation site and the adjacent structures: A 5-story school and a 2-story residential building adjoin the excavation boundary from south and west, respectively. Also, the office building block, referred to as Tooba tower, is located inside the site boundary. Hence, this project involved excavation on 3 sides of the roughly rectangular plan of TOOBA tower to the depth of 16.5 meters

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below its foundation level or 26 meters below the ground surface. Other sides of the excavation boundary, on the other hand, were excavated to the depths varying from 9 to 28 meters depending on the sloping ground condition.

The Building itself is irregular both in plan and in vertical cross section. Figure 2 shows the 3D view of the building: the tower is about 64 meters tall on the east side and about 42 meters tall on the west side. The vertical load exerted by the building is estimated to be 200 kN/m2 on the taller side and 150 kN/m2 on the other side. The excavation procedure was ensued with no excessive deformations occurring in the building during or after the excavation. This paper considers some of the design and construction aspects of this successfully completed project.

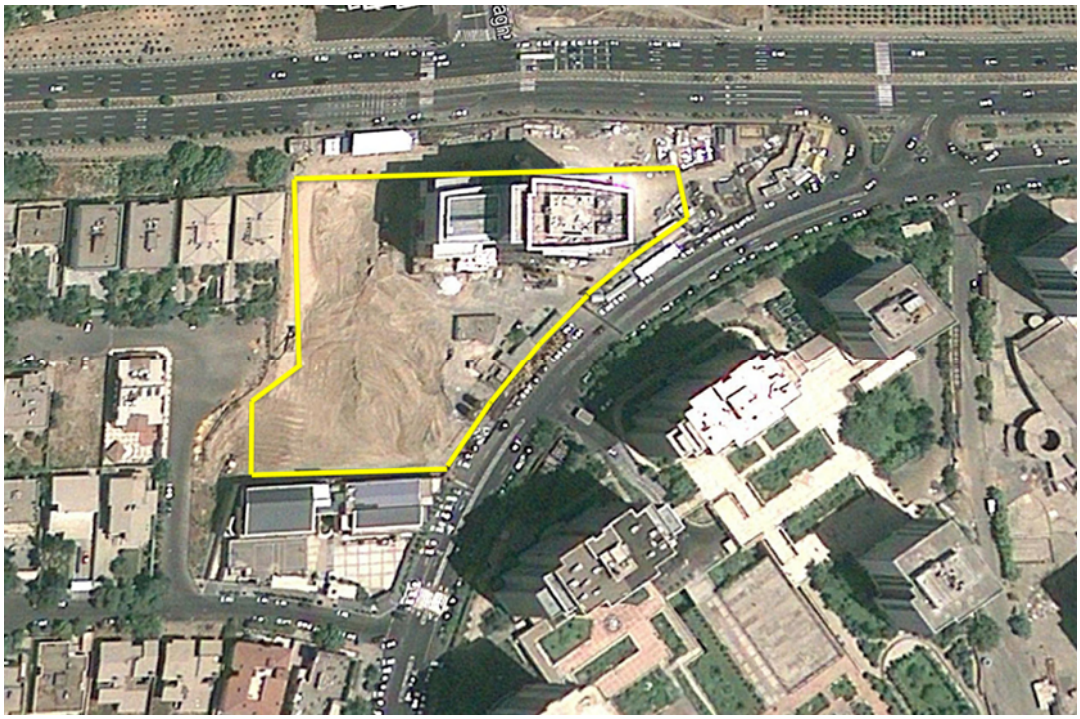

Figure 1. Google Earth view of the excavation site (Source: "Tehran." 35° 45' 58.41" N and 51° 22' 12.58" E. Google Earth. June 30 2009. Project boundary is highlighted by yellow line.

Figure 2. 3D view of the TOOBA tower: the location of the tilt meters (on the right) and the location of prism targets on the west side of the tower (on the left)

Method of Excavation

Given the importance of constraining the deformations of the Tooba tower, excavation face on three sides of the tower was protected by contiguous bored reinforced concrete piles supported at 4 different levels with pre-tensioned tiebacks and wailing system.

The construction sequence and technique for construction of such retaining structures is well described in the literature (i.e. FHWA 1998, 1999). In short, the top-down sequence of construction of this retaining structure consisted of pile installation followed by excavation and installation of support in lifts. A total of 134 cast-in-place reinforced concrete piles were installed prior to excavation. Figure 3, shows in plan view, the reinforced piles and the layout of pre-tensioned tiebacks for the third level of support below the tower. As it can be seen, the tiebacks for each level of support consist of tiebacks perpendicular to the wall and tiebacks installed at sharp angles toward the wall. Two samples of the details of anchor heads designed for this retaining structure are shown in Figure 4.

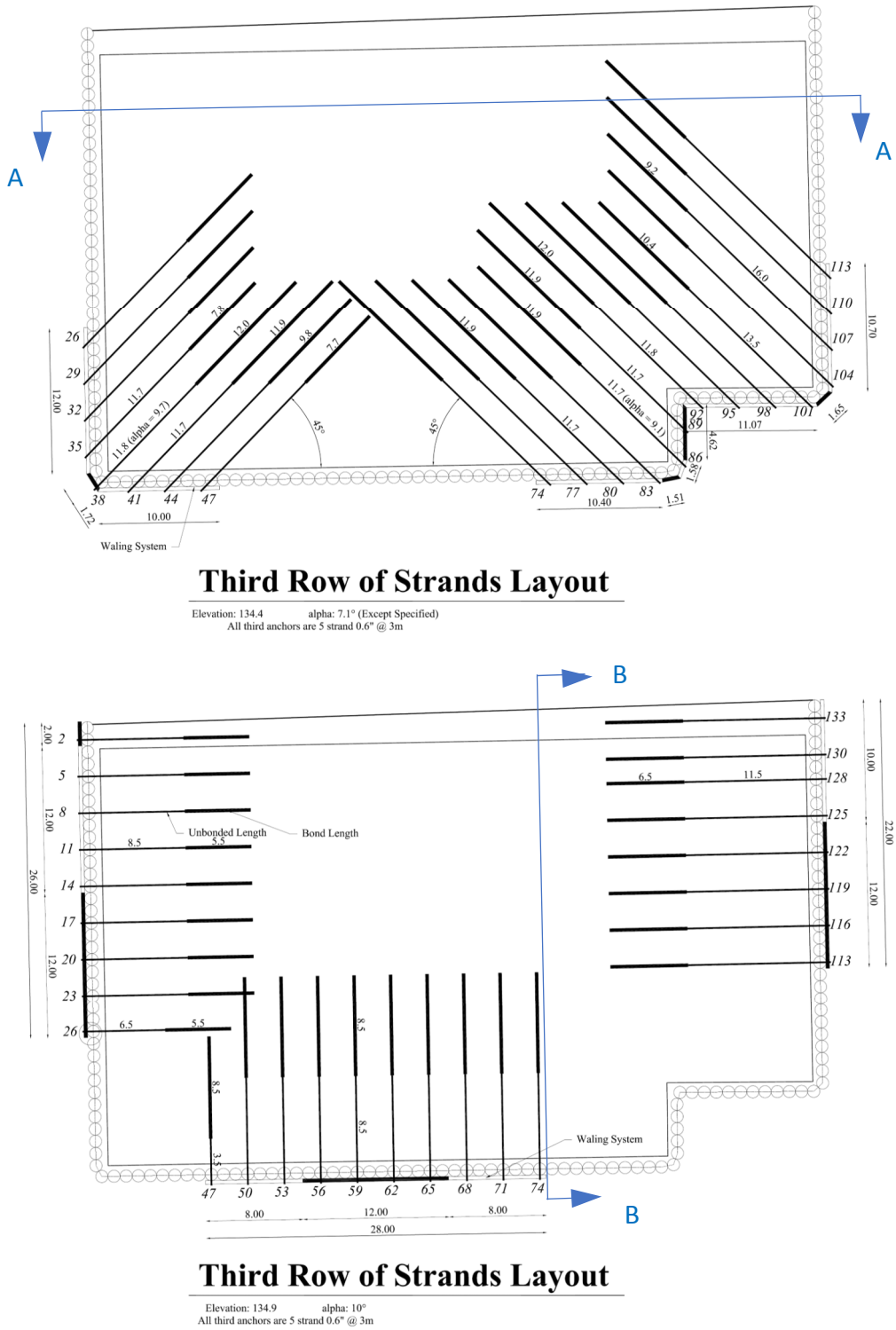

Figure 3. The layout of anchors installed for the third level of wailing

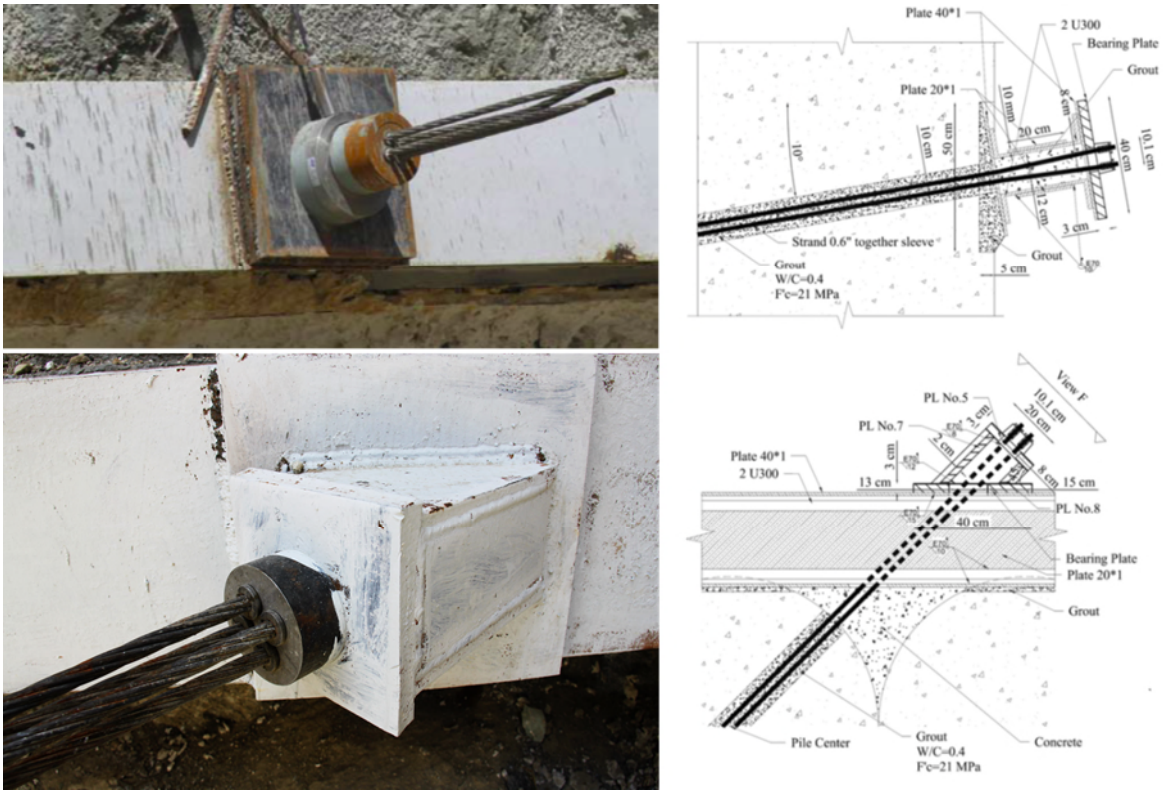

Figure 4. Two samples of the details of anchor heads designed for the retaining structure

Figure 5. Excavation of one of the blocks along the southern leg of the Tooba Tower during the 2nd lift of excavation.

Excavation around the building was carried out in sections to ensure symmetric deformations of the wall. For each lift of excavation soil is usually removed until 1 meter below the anchor level of that excavation lift. Figure 5 shows how one section (or block) of soil is excavated for the installation of the 3rd row of tiebacks. The wailing system and tie backs are installed for each block of soil excavated. Once an entire lift of excavation is completed in this manner, tiebacks are pre-tensioned similarly in a symmetrical pattern.

Figure 6 shows the view of the Tooba tower and the retaining system red dashed line 1 shows the ground level before excavation; red dashed line 2 shows the foundation of Tooba tower where the first level of pre-tensioned tiebacks is installed; three other levels of pre-tensioned tiebacks are highlighted with yellow dashed lines.

Subsurface Conditions

This site is located on a highly cemented gravelly alluvial deposit of Northern Tehran, at the foot of Alborz mountain range. Therefore, it is not unusual to find boulders few feet in diameter in this formation. Another feature typically found in the ground in North Tehran are qanats. Qanats are underground water tunnels excavated from about 5000 years ago in Iran to lead water from interior of a hill to the villages in foothill. Geotechnical Engineers working in these formations need to be aware of these features as they can be challenging during construction phase.

SPT N-values and groundwater data were found from previous geotechnical explorations for 10 borings excavated to the depth of 75 meters. PLT test data

was also found from another 3 observation wells to the depth of 11 meters. Two more borings to the depths of 80 meters and 5 more observation wells to the depth of 25 meters were made to complete the available geotechnical data on the site. In-situ direct shear tests, density tests and PLT tests were performed in these observation wells. Soil samples were obtained using core barrel method which were used for laboratory tests including Atterberg limits, sieve test, density, and direct shear tests.

Figure 6. A panorama view of the tower and the retaining system. In this image, all 4 levels of wailing have been installed.

Geotechnical explorations indicate that the site consists of highly cemented layers of silty and clayey sand and gravel (SC, SM, SW, GC, GM, and GP) along with some boulders (see Figure 7). Soil to the east of the site is generally sandy, whereas the west of the site tends to contain more gravel. Soil is generally brown or light brown and rarely light green or grey. The mechanical behavior of the soil is heavily affected by the cementation bonds between soil particles. Considerably lower shear strength parameters of remolded and disturbed specimens in direct shear test at laboratory is an indicator of high cementation in the soil. As a result, soil parameters were estimated by interpreting the in-situ test results. Detailed information on the characteristics of the cemented soil of Tehran has been published by a number of researchers (Haeri et. al. [2003, 2004, 2005a, 2005b, 2006], Asghari et. al. [2003, 2004], Hamidi and Haeri [2005, 2008] and Haeri and Hamidi [2009]).

Based on the SPT test results the soil to the west and south of the site is very dense and has corrected SPT value of higher than 50. However, a fill material was detected in the vicinity of the TOOBA tower down to the depth of 8.5 meters below the ground surface. The foundation of the existing tower is located 8.5 meters below the ground surface on the dense soil. Fill material was also present on the east side of the site down to a depth of 7 meters. To the authors' knowledge, this could be as the result cut and fill activity done before this urban development activities began in this area. In other words, at this site, which is located near the Alborz mountain range, the soil on the higher ground was cut and filled in the shallower areas without sufficient compaction effort before the area turns into the heavily developed urban environment. Aerial photos of the site from 1969 (before any construction development occur) corroborate our knowledge about the geological formation of the site.

Table 1. Mechanical Properties of the disturbed layer (L1) and the intact deposit (L2)

Soil Parameters	**Disturbed Fill (L1)**	**Intact Cemented Layer (L2)**
Elastic Modulus, E_s (kN/m^2)	50000	140000
Poisson's Ratio, v	0.3	0.3
Internal Friction Angle, ϕ (°)	36	38
Cohesion Intercept, c (kN/m^2)	10	40

The authors of this manuscript were very interested to see if soils with similar properties/origin might be found in United States. Beckwith and Hansen, 1982 reviewed the distribution, and engineering properties of various calcareous soils of the southwestern United States. In that study, Beckwith and Hansen, list a variety of soils with calcium carbonate cementation from collapsible clays to cemented sands. It was very interesting to find, in the list of calcareous soils, two

candidate that resembled the cemented gravely alluvium of Tehran both formed in very similar geological condition:

a) Soil deposit found in the talus slopes of Franklin mountain range in El Paso, Texas
b) 2 to 3 million year old terraces of Salt river in central Arizona with that are formed in the talus slopes of Mazatzal (MAH-zaht-ZAL, locally Ma-tuh-ZEL) mountain range.

Besides being a talus slope with calcium carbonate cementation, these soil deposits resemble Tehran's cemented gravely alluvium because they show similar Elastic moduli and STP-N values.

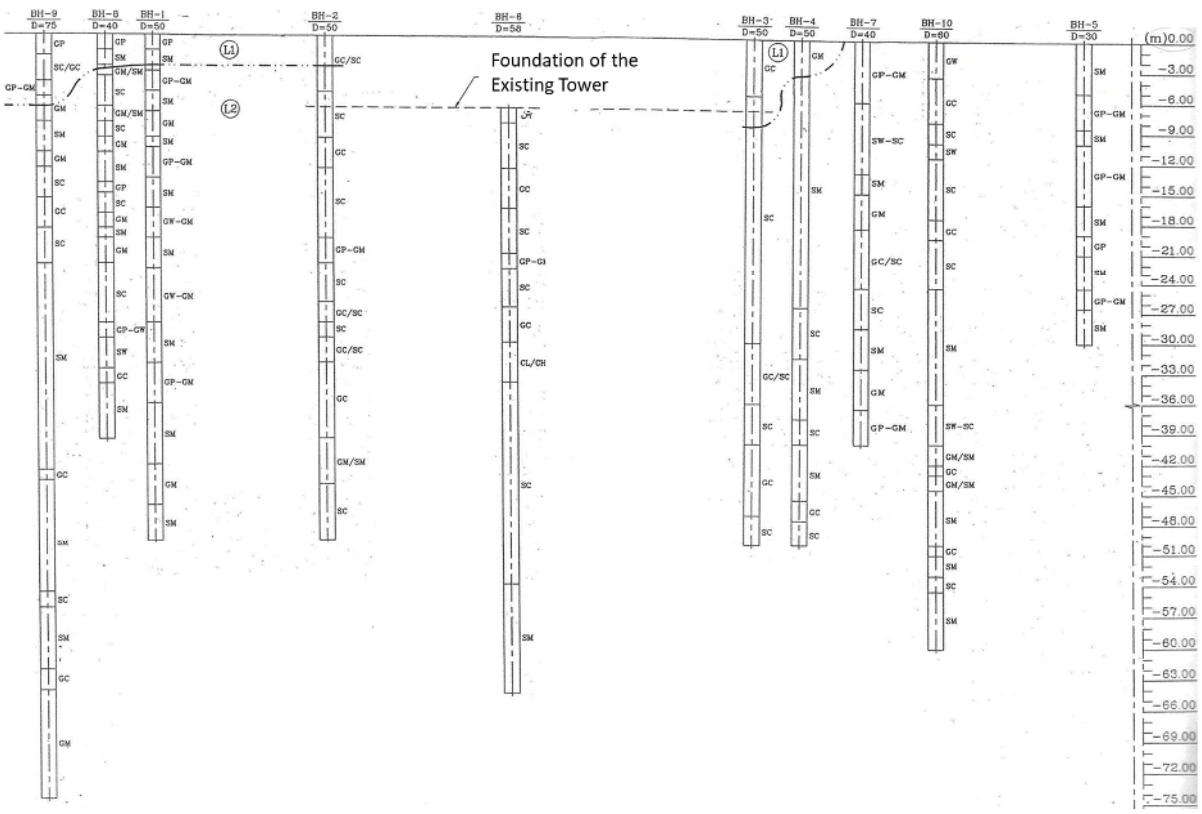

Figure 7. The disturbed layer (L1), the intact deposit (L2) and the alternating sand and gravel sub layers from the bore holes

Even though the site is very heterogeneous, from the mechanical point of view the site stratigraphy can be divided into two distinct layers, both consisting of sandy and gravelly sub layers. The mechanical properties of these 2 layers are shown in Table 1 (the disturbed layer (L1) and the intact deposit (L2)). These two layers can be described as follows: A very dense cemented layer which is intact, and, a disturbed fill layer which has similar grain size distribution of the

underlying soil but the broken cement bonds and lower compaction in this layer gives the soil lower elastic modulus, lower shear resistance and probably higher permeability. Therefore, it can be described as a medium dense soil. The presumed boundary of the disturbed layer has been sketched with a dash double-dot line in Figure 7.

Groundwater table

During the geotechnical investigations a number of abandoned drainage wells (drains) were discovered to the north and east of the TOOBA tower. There is a perched water present in the deposit with unknown origin and is mainly present in the north east of the site. The water level in the borings and observation wells were extremely different in each boring and well. In addition to the effect of perched water, the water level in borings is affected by penetration of water from abandoned drains and flumes. For example, one of the test pits was engulfed by water when it reached a sand lens 9m below ground surface. The rate of water entering the well was so high that efforts for pumping the water out for continuing the boring of the well were abandoned. Hence, initial design the possibility of localized groundwater intrusion into the pit during excavation was considered. Therefore, the design considered drain boreholes near the bottom of the excavation. Drains were also installed to conduct the waste water from TOOBA tower to the drainage network and avoid accumulation behind the retaining structure.

Monitoring Program

A monitoring program for measuring the deformations of the tower and supporting system was enforced during and after excavation. Instrumentations on the tower included 17 prism targets (PT) and 4 tilt meters (TM). The locations of the tilt meters and west side prism targets are shown in Figure 2. EAN-90M tilt meters are used in 2 perpendicular directions to measure tilt in both north-south and east-west directions. ERT-20P-MT mini prism targets, on the other hand, were installed on all 4 sides of the tower. These targets can yield displacements in x, y and z directions. A TS09 total station was used to record the displacements from the pillars installed outside the excavation pit.

Prism targets and tilt meters were used to monitor the deformations of the tower. During the early stages of excavation deformations were very small and the data records from prism targets and tilt meters sometimes produced contradictory results. However, this was not a very unusual observation since the readings from the prism targets at this stage were very close to or within the reading tolerance of the optical reading instrument.

Contradictory readings from the prism targets which sometimes indicated that opposite sides of the tower were deforming in opposite directions continued during entire excavation process. These readings were regarded very seriously

because the assumption that the tower might lose its structural integrity was a serious concern. Therefore, visual inspection of the building was performed regularly for any signs such as cracks that may corroborate the readings from prism targets. Since no visual problems were observed, these differential deformations deemed to be as the result of error in reading the targets including operator and device error. It might have also been because of the temperature gradient between two sides of the building; sunny side and shadow side.

During the excavation of the first 3 lifts general pattern of the movement of tower indicated a rotation about 0.02 degrees toward North (outward the excavation) and 0.02 degrees toward east. Total deformations from the prism targets at this point rarely exceeded 10 mm. After this point (with installation and pre-stressing of the 4th level of anchors and continuation of the excavation), general pattern of tower deformation showed tilting about 0.02 degrees toward south (inward the excavation pit) and continued tilting toward east (about 0.05 degrees) at the end of excavation.

The general pattern of movement suggested by the prism targets corroborates the rotational like deformation of the building towards south and east. During the first 3 lifts of excavation total deformations yielded from the prism target readings rarely exceed 5 mm and were always less than 10 mm. Readings at this stage, as stated before, yield contradictory results; after this point on, however, general pattern of displacements obtained from the prism targets corroborate the rotational like deformation of the building towards south and east.

The prism targets installed near the top of the building had maximum deformations; however, deformations of these targets were less than 20 mm towards south and less than 15 mm towards east during and after the excavation. Small tilting of the tower towards the north at the beginning of the excavation could be as the result of pre-stressing of anchors and the grouting pressure exerted on the tower foundation.

Also, six ELC-30S load cells were installed on the tieback wall in order to measure the changes in tieback pre-stress loads. Normally a reduction in tieback load could signal relaxation in the tieback bond length; in a tieback wall designed to minimize deformations. On the other hand, an increase in tieback load normally indicates that the anchor lock-off load could not impede further deformations and that the wall deformations have occurred at the tieback location and have further stretched the tieback.

The load cell on the second row of tiebacks and one of the 2 load cells on the 4th level showed consistent anchor load throughout the excavation process. Two of the load cells installed on the 3rd row of the tiebacks and one other load cell on the 4th row showed a very small load increase (less than 1 percent of the pre-stress load). The only load cell on the eastern side of the building was installed on an anchor on the 3rd level of tiebacks which showed a reduction in the pre-stress load about 1 percent of the tieback lock off load.

Summary & Final Remarks

This contribution introduced a deep excavation work in north-west of Tehran which included excavation of a deep underground level exactly adjacent to 3 sides of a 21-story office block and two other structures. The excavation procedure succeeded with the monitoring program showing that no excessive deformations occurred in the building during or after the excavation.

The project site consists of layers of highly cemented silty and clayey sand and gravel with some boulders. The difference between the in-situ and laboratory tests corroborate the fact that the soil is highly cemented. As a result, soil parameters were estimated by interpreting the insitu test results. The presence of large boulders and cemented soil meant that insitu tests such as PLT and direct shear tests had to be performed inside observation wells excavated onsite which can be very different from conventional in situ tests such as dilatometer or SPT.

Other unusual geologic features of the site (i.e. presence of disturbed fill, boulders and qanats) and 3D nature of the excavation added to the complexity of the project.

This manuscript is a revision of two papers already published at the 7th international conference on case histories in geotechnical engineering, 2013. In this manuscript the project is revisited with a focus on geotechnical exploration aspect of the project and an additional section on soils in the US with similar properties.

The design and consultation team for this project consists of Prof. S.M. Haeri as the leader of the project, Dr R. Shakeri as the head of design team and Mr. K. Afshari, Mr. M. Sasar and Mrs. P. Ghahremani as members of the design team. All of the figures used in this manuscript is the result of their contribution to this project and is greatly acknowledged. The readers are encouraged to see Haeri et al, 2012 papers for more information on this deep excavation project.

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CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS

ORVSS I	BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, KY
ORVSS II	EARTHWORK ENGINEERING, START TO FINISH, October 15, 1971, Louisville, KY
ORVSS III	LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, KY
ORVSS IV	GEOTECHNICS IN TRANSPORTATION ENGINEERING, October 5, 1973, Lexington, KY
ORVSS V	ROCK ENGINEERING, October 18, 1974, Clarksville, IN
ORVSS VI	SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, KY
ORVSS VII	SHALE AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION, October 8, 1976, Lexington, KY
ORVSS VIII	EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 14, 1977, Louisville, KY
ORVSS IX	DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, KY
ORVSS X	GEOTECHNICS OF MINING, October 5, 1979, Lexington, KY
ORVSS XI	EARTH PRESSURE AND RETAINING STRUCTURES, October 10, 1980, Clarksville, IN
ORVSS XII	GROUNDWATER: MONITORING, EVALUATION AND CONTROL, October 9, 1981, Fort Mitchell, KY
ORVSS XIII	RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, KY
ORVSS XIV	FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, IN
ORVSS XV	PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, KY
ORVSS XVI	APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, KY
ORVSS XVII	NATURAL SLOPE STABILITY AND INTRUMENTATION, October 17, 1986, Clarksville, IN
ORVSS XVIII	LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Fort Mitchell, KY
ORVSS XIX	CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES, October 21, 1988, Lexington, KY
ORVSS XX	CONSTRUCTION IN AND ON ROCK, October 27, 1989, Louisville, KY
ORVSS XXI	ENVIRONMENTAL ASPECTS OF GEOTECHNICAL ENGINEERING, October 26, 1990, Fort Mitchell, KY
ORVSS XXII	DESIGN AND CONSTRUCTION WITH SYNTHETICS, October 18, 1991, Lexington, KY
ORVSS XXIII	IN-SITU SOIL MODIFICATION, October 16, 1992, Louisville, KY
ORVSS XXIV	GEOTECHNICAL ASPECTS OF INFRASTUCTURE RECONSTRUCTION, October 15, 1993, Fort Mitchell, KY
ORVSS XXV	RECENT ADVANCES IN DEEP FOUNDATIONS, October 24, 1994, Lexington, KY

CHRONOLOGY OF OHIO RIVER VALLEY SOIL SEMINARS (CONTINUED)

ORVSS XXVI	SITE INVESTIGATIONS: GEOTECHNICAL AND ENVIRONMENTAL, October 20, 1995, Clarksville, IN
ORVSS XXVII	FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING, October 11, 1996, Cincinnati, OH
ORVSS XXVIII	UNCONVENTIONAL FILLS: DESIGN, CONSTRUCTION AND PERFORMANCE, October 10, 1997, Lexington, KY
ORVSS XXIX	PROBLEMATIC GEOTECHNCIAL MATERIALS, October 16, 1998, Louisville, KY
ORVSS XXX	VALUE ENGINEERING IN GEOTECHNICAL CONSULTING AND CONSTRUCTION, October 1, 1999, Cincinnati, OH
ORVSS XXXI	INSTRUMENTATION, September 15, 2000, Lexington, KY
ORVSS XXXII	REGIONAL SEISMICITY AND GROUND VIBRATIONS, October 24, 2001, Louisville, KY
ORVSS XXXIII	GROUND STABILIZATION AND MODIFICATION, October 18, 2002, Covington, KY
ORVSS XXXIV	APPLICATIONS OF EARTH RETAINING SYSTEMS AND GEOSYNTHETIC MATERIALS, September 19, 2003, Lexington, KY
ORVSS XXXV	ROCK ENGINEERING AND TUNNELING, October 20, 2004, Louisville, KY
ORVSS XXXVI	GEOTECHNICAL INNOVATIONS IN TRANSPORTATION ENGINEERING, October 14, 2005, Covington, KY
ORVSS XXXVII	INNOVATIONS IN EXPLORATION OF SUBSURFACE VOIDS, October 27, 2006, Lexington, KY
ORVSS XXXVIII	CIVIL INFRASTRUCTURE AND THE ROLE OF GEOTECHNICAL ENGINEERING, November 14, 2007, Louisville, KY
ORVSS XXXIX	URBAN CONSTRUCTION, October 17, 2008, Covington, KY
ORVSS XL	GEOTECHNICAL ENGINEERING AND ENERGY INFRASTRUCTURE, November 13, 2009, Lexington, KY
ORVSS XLI	NATIONAL INFRASTRUCTURE: DAM AND LEVEE SAFETY, October 20, 2011, Louisville, KY
ORVSS XLII	LESSONS LEARNED: FAILURES AND FORENSICS, October 21, 2011, Cincinnati, OH
ORVSS XLIII	WALLS: ABOVE AND BELOW GRADE, November 19, 2012, Lexington, KY
ORVSS XLIV	THE APPLICATION OF GEOLOGY TO GEOTECHNICAL ENGINEERING PRACTICE, November 15, 2013, Louisville, KY
ORVSS XLV	GEOTECHNICAL ASPECTS OF WATERFRONT DEVELOPMENT, October 17, 2014, Cincinnati, OH
ORVSS XLVI	GROUTING SOLUTIONS TO GEOTECHNICAL PROBLEMS, December 16, 2015, Lexington, KY
ORVSS XLVII	GEOTECHNICAL ASPECTS OF THE LOUISVILLE-SOUTHERN INDIANA OHIO RIVER BRIDGES PROJECT, November 16, 2016, Louisville, KY
ORVSS XLVIII	INFRASTRUCTURE INNOVATION IN GEOTECHNICAL DESIGN, November 17, 2017, Cincinnati, OH
ORVSS XLIV	TOOLS FOR ASSESSING GEOTECHNICAL SITE CONDITIONS, November 29, 2018, Lexington, KY