CHARACTERIZING COMPLICATED NEAR-SURFACE GEOLOGIC PROFILES USING NOVEL IN-SITU TESTING AND DATA PROCESSING TECHNIQUES

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Abstract

There are several shortcomings associated with current characterization methods of near-surface (shallow, tens of meters in depth) geologic profiles. Cone penetration testing (CPT) is relied on as an indicator of soil stiffness and strength, but data smearing at boundaries between layers in "complicated" soil profiles obscures the relationship between measured CPT indices and the true properties of the soil at a given depth. Furthermore, CPTs do not collect soil samples. When soil samples are collected, it is typically done using primitive methods of extraction such as with the standard split-spoon sampler (a cylindrical steel tube with an outer diameter of 2 inches) that is driven into the ground with a hammer at (typically) 5-foot intervals. This type of sampling not only destroys the soil's fabric, which has a significant influence on the engineering behavior of the soil, due to disturbance caused by the small sampler size, but is representative of the soil only at a discrete point, leaving the interpretation of much of the geologic profile up to the engineer. Accurately characterizing the engineering properties of subsurface profiles is essential for effective predictions of soil behavior for applications including pavement, foundation, and earthquake engineering. This paper discusses a new method of in-situ testing and data interpretation to address several of these subsurface characterization limitations.

Introduction

"Complicated" soil profiles have many thin, interbedded layers of different soil types. Limitations in our current in-situ testing and sampling procedures prevent the accurate characterization of these soil profiles and negatively impact the accuracy of subsequent geotechnical analyses performed using these data. For example, the misprediction of the occurrence of soil liquefaction as a result of the 2010-2011 Christchurch, New Zealand earthquakes is partially attributed to the widespread prevalence of complicated soil sites across Christchurch (e.g., as discussed by Cox et al. 2017, McLaughlin 2017).

We are using a two-pronged approach to address this problem using novel techniques to sample and process geotechnical data. First, the shortcomings of a very common in-situ geotechnical testing method (cone penetration testing) are discussed and an alternative data-processing methodology is proposed. To develop this methodology, numerical modeling tools are employed. Second, a novel in-situ testing method, geo-slicing, is introduced. Findings from laboratory tests that have been performed in order to scale-up this testing method to the field are discussed. Ultimately, both the numerical modeling and the geo-slicing techniques will be implemented to collect and process data from a large field study performed in New Zealand to help better understand soil response during earthquakes.

Cone Penetration Testing

Background

The cone penetration test (CPT) is a widely used method of in-situ geotechnical testing and consists of hydraulically pushing a cone-shaped probe (cone) into the ground. Both the "tip resistance" and "sleeve friction" are measured nearly continuously as the cone advances with depth. These measurements can be correlated to a variety of geotechnical parameters, including soil strength and stiffness. The resistance to the cone advancement is due to a stress bulb that extends both beneath and above the cone's tip. Consequently, the data measured at a given depth is influenced by soil up to 10 to 30 cone diameters (typically 36 to 72 cm) beneath the tip of the cone and to a lesser extent by the soil above the cone tip (Ahmadi and Robertson 2005). As a result, the CPT data do not clearly show thin soil layers in complicated soil profiles, and the measurement process "smears" the data near layer boundaries such that the measurement at a discrete point may not actually be representative of the properties of the soil at that depth (often referred to as thin-layer, transition-zone, or multiple thin-layer effects).

Many researchers have proposed methods to correct the smearing of data (e.g. Youd et al. 2001, Ahmadi and Robertson 2005, Boulanger et al. 2016). However, these methods are difficult to implement and can be subjective. Recently, Boulanger and DeJong (2018) proposed treating this as a mathematical inverse problem in which the measured tip resistance is presumed to be a convolution of the true tip resistance and a depth-dependent spatial filter that smears the data. The "true" cone tip resistance is extracted from the measured cone tip resistance by solving an optimization problem. Boulanger and DeJong (2018) provide several examples of the application of their method to field data. Since we do not know the "true" tip resistance in the field, it is impossible to evaluate the efficacy of the procedure. Furthermore, the procedure has some limitations in its application to very thin soil layers, less than 2 cm in thickness (Yost et al. 2021b).

This project aims to modify the inversion procedure proposed by Boulanger and DeJong (2018) to enhance the identification of very thin layers and fine-scale stiffness contrasts. To test and verify the new method, a numerical model has been developed and validated that can be used to create a suite of synthetic CPT soundings. This allows the "measured" tip resistance to be computed by the simulated cone penetration, and the "true" tip resistance is known from the model input parameters and layer stratigraphy. Thus, the efficacy of the revised inversion procedure can be evaluated prior to application on field data.

Methodology

Material Point Method

The Material Point Method (MPM) is a numerical method particularly well suited for problems with large deformations (e.g., landslide runout) and multi-body contacts (e.g., cone penetration) because it avoids the mesh-tangling problems that arise in other numerical procedures like the Finite Element and Finite Difference Methods. MPM has been used to model cone penetrometer testing by several researchers (e.g., Beuth 2012, Ceccato et al. 2015) but has not yet been used to examine the influence of multiple thin layers on CPT tip resistance.

Calibration Chamber Tests

A series of laboratory calibration chamber experiments were performed at Deltares by de Lange (2018) to better understand CPT penetration in soil with multiple thin layers. These experiments were used to calibrate and validate the numerical model. The following paragraph provides a brief description of the tests so that the geometry of the numerical model can be understood. Much more detailed information regarding these tests is provided in de Lange (2018).

Soil profiles consisting of saturated Baskarp B15 sand (of varying relative densities) with interbedded layers of Vingerling K147 clay were constructed in a cylindrical chamber having a diameter of 0.9 m and a height of 1 m.

Profiles consisting only of Baskarp B15 sand were also constructed as reference models. To simulate field conditions much deeper in a soil profile, a flexible cushion applied a constant vertical stress at the top of the model and the model was pressurized on its sides by a fluid-filled membrane. 25-mm-diameter cones were advanced hydraulically at 4 mm/sec through holes in the cushion to a maximum penetration depth of 0.75 m. Cone tip resistance and sleeve friction were measured. In this study, three of the layered soil profiles, designated as SM4 CPT2, SM2 CPT2, and SM8 CPT1 were selected to be replicated with numerical models.

Numerical Model

A numerical model was developed using the Anura3D platform, a software created by the Anura3D MPM Research Community (http://www.anura3d.com/). The geometry of the model was chosen so that it replicated that of the calibration chamber used in the de Lange (2018) experiments, as shown in Figure 1. Cone penetration was modeled as a 2D-axisymmetric problem. The cone was modeled as a rigid (incompressible) body that was advanced at a prescribed constant velocity into the soil. The diameter (D) of the cone was 25 mm with a 60° apex angle. The tip of the cone was slightly rounded to avoid numerical issues. A frictional contact algorithm after Bardenhagen et al. (2001) was employed to describe the interaction between the cone and the soil. The contact friction angle between cone and soil (δ) can be expressed as a fraction of the soil's effective friction angle (ϕ') as $\delta = \alpha\,\phi'$. Durgunoglu and Mitchell (1973) reported values of α ranging from 0.28 (for polished aluminum) to 0.9 (for sanded aluminum). In this study, α was assumed to be 0.5.

A typical model (with a different model being created for each different soil profile) had a mesh comprised of 5,410 triangular elements and contained 63,753 material points. The mesh extended 40D below the cone's tip at its initial position and extended 10D radially. The bottom and top of the mesh were fixed in the horizontal and vertical directions. The left

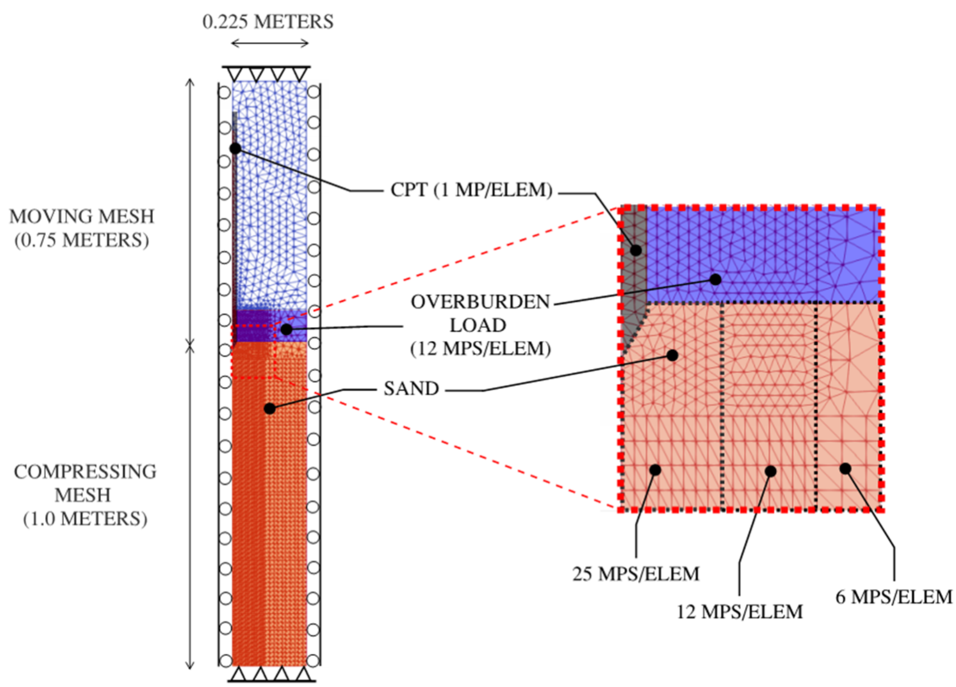


Figure 1. Cone penetration test Material Point Method model configuration (Yost et al. 2021c).

and right boundaries were fixed in the horizontal direction. A refined mesh was used in the region through which the cone penetrates and a higher density of material points (MPs) was assigned in this area (see Figure 1) to enhance the accuracy of the solution. A moving mesh technique was employed to ensure the accurate geometry of the cone throughout the calculation and improve the efficiency of the contact algorithm. As the simulation progressed, the zone of mesh above the cone tip moved downward at the same velocity as the cone and the mesh elements retained their shape. At the same time, the zone beneath the cone remained in place, but the mesh elements vertically compressed.

The soil material properties were inferred from data reported by de Lange (2018), as well as from additional laboratory tests performed on Baskarp sand (Ibsen and Bødker, 1994 and Borup and Hedegaard 1995). The vertical overburden pressure imparted on the soil by the cushion was modeled as a single layer of material with a density and height that result in a pressure identical to the overburden pressure applied in the calibration chamber. The strength parameters used as inputs to the soil constitutive models are summarized in Table 1. A drained, strain softening Mohr Coulomb constitutive model was selected for the Baskarp B15 sand based on the dilative tendency observed in triaxial test results. An undrained Tresca constitutive model was selected for the Vingerling K147 clay.

Results and Discussion

As the CPT penetrates into the soil, a stress bulb develops around the tip of the cone and causes a reaction force to develop on the lateral face of the cone. CPT tip resistance versus penetration depth as measured in the MPM simulation can then be compared to the tip resistance recorded in the corresponding calibration chamber test experiments. The results from this exercise are presented in Figure 2.

As shown in Figure 2, the experimental measured tip resistance (shown in black) matches very well with the synthetic measured tip resistance (shown in red) from the MPM simulations in the layered soil profiles, within an accuracy of about 200 kPa. Furthermore, the true tip resistance of the sand and clay layers can be determined by performing MPM simulations on uniform profiles consisting only of sand or clay under identical conditions to that of the layered models. These true tip resistance profiles are

Table 1. Material Parameters used for Numerical Simulation of CPT in Layered Profiles

Parameter		SM4 CPT2	SM2 CPT2	SM8 CPT1
Soil Model	Vertical Stress (kPa)	50	50	25
	Relative Density of Sand (%)	54	29	61
	Layer Thickness Ratio	1.6	1.6	0.8
Baskarp B15 Sand	Peak Friction Angle (°)	38	36	38
	Residual Friction Angle (°)	36	34	37
	Peak Dilatancy Angle (°)	12	7	12
	Shape Factor	5	2	10
	Young's Modulus (kPa)	20000	7000	10000
	Poisson's Ratio	0.33	0.33	0.33
Vingerling K147 Clay	Undrained Shear Strength (kPa)	27	27	23
	Young's Modulus (kPa)	25000	25000	25000

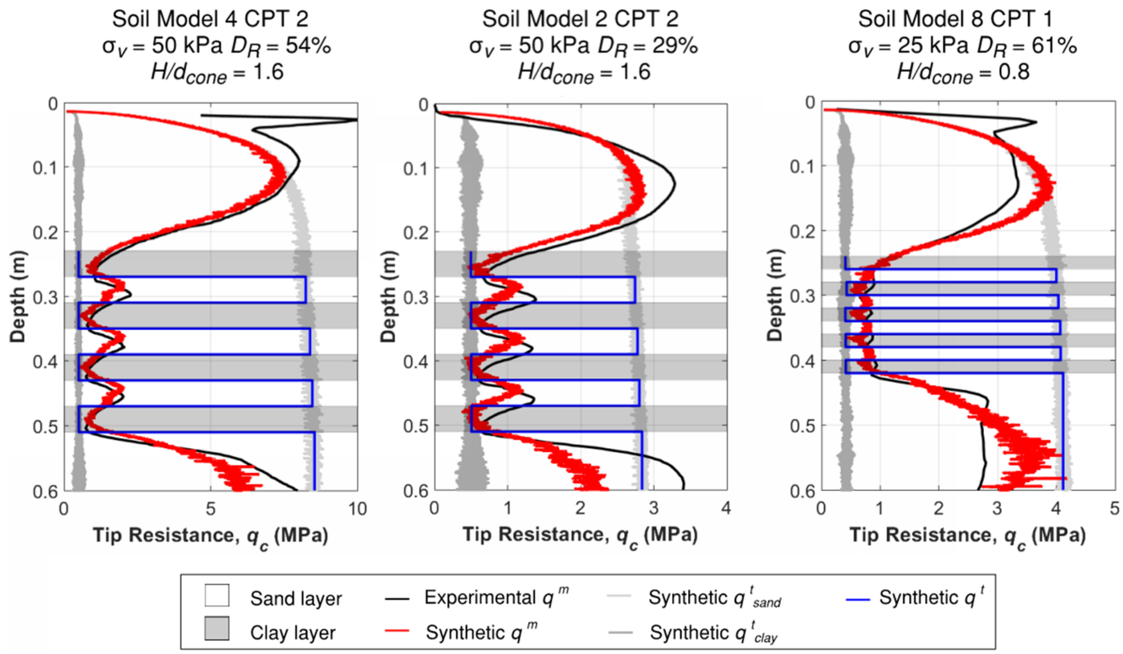


Figure 2. Comparison of experimental measured tip resistance (q^m) obtained during three calibration chamber tests performed by de Lange (2018) with synthetic q^m and synthetic true tip resistance (q^t) obtained from MPM simulations (Yost et al. 2021c).

shown in gray. Finally, because the true tip resistance of the sand and clay layers can be numerically determined, and the locations of the layers are known, a true tip resistance profile for the entire layered model can be developed (as shown in blue). You can observe that the measured tip resistance is significantly less than the true tip resistance in the thin sand layers, while the measured tip resistance in the thin clay layers is only slightly larger than the true tip resistance. Correction/inversion procedures for multiple thin-layer effects aim to use the measured tip resistance as input and produce the true tip resistance as output.

In summary, it has been shown that MPM can be used to develop true tip resistance profiles from measured tip resistance profiles and known multi-layer stratigraphy. The combination of measured and true tip resistance for a single profile allows for the validation of correction/inversion procedures for multiple thin-layer effects.

Geo-Slicing

Background

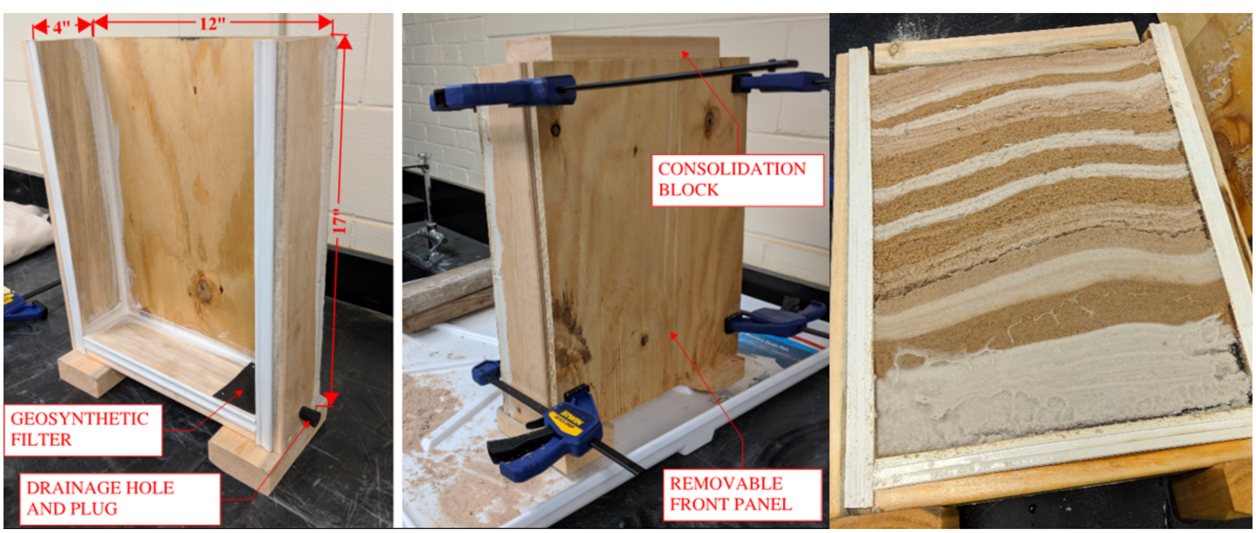
Geo-slicing is a soil sampling technique that consists of driving a sheet pile (trapezoidal-shaped steel element) into the ground, followed by a “shutter” plate that slides along the edge of the pile. The sheet pile and shutter plate are then extracted together with a soil sample encased in the annular space. Geo-slicing allows for the extraction of a high-quality sample that is approximately 0.5 m wide and 9 m deep. Performing several geo-slices in a line can allow for a near-continuous picture to be developed of the near-surface geologic profile across an entire site. Samples of this nature can provide invaluable insights into the subsurface profile, allowing the examiner to see intricate bedding structures and depositional trends across a site – something virtually impossible to do with more common soil sampling techniques that only sample at discrete locations and depths.

Once the geo-slice sample is extracted, it is laid horizontally on the ground. The shutter plate is removed and the disturbed soil on the surface is scraped clean to reveal detailed soil stratigraphy. To enhance and preserve the bedding features in the sample, a "peel" is created by covering the soil sample with a reinforcing cloth and pouring permeation grout over the surface. Once the grout has set, the peel is removed from the sample. Because the grout permeates more deeply into coarser-grained soils, the resulting peel shows a relief of the soil bedding in more detail than the soil sample itself.

While widely used in Japan, geo-slicing has only been used in the US once (Takada and Atwater, 2004). The Japanese grout OH-4 used in the Takada and Atwater (2004) study to create the geo-slice peels is highly toxic and there are concerns about importation to other countries. Since we aim to use this technique in New Zealand, we performed bench-scale tests with several alternative adhesive products to see if there is a suitable alternative.

Methodology

To conduct bench-scale tests, a small-scale geo-slicer was constructed; see Figure 3. Instead of constructing a true scaled version of the geo-slicer, we created a wood frame into which we could water-pluviate sand to simulate the natural bedding structure of a soil deposit. Soil layering was created manually by water-pluviating coarse sand and fine sand in alternating patterns. Some profiles were also created to include thin clay layers. The geo-slicer has a drainage hole near the bottom which remains plugged during water pluviation. After the geo-slicer is filled with soil, the drain plug is removed and a weight is added to the top of the consolidation block which sits on top of the sample. The sample is allowed to consolidate for at least 24 hours. Once consolidation is complete, the geo-slicer is placed horizontally on the lab bench



(a) (b) (c)

Figure 3. a) Bench-scale geoslicer with front panel removed. b) Bench-scale geoslicer with front panel clamped in place (Yost et al. 2021a). c) Completed geo-slice sample after consolidation and removal of front panel.

and the front panel is removed. The exposed soil surface is scraped clean with a spatula. A completed geo-slice sample after consolidation and removal of the front panel is shown in Figure 3c.

So far, eleven different adhesive products have been tested for this application (for brevity in this paper, we will not discuss details of all eleven, however, complete details can be found in Yost et al. 2021a). The products generally fall into one of three classification categories (permeation grouts, liquid rubber, or erosion control products) and have never been tested for making geo-slice peels before. A series of feasibility tests were performed on each of the eleven products to determine whether they were able to create an acceptable geo-slice peel. Of the eleven, only three adhesives were able to produce a peel that was both flexible and firmly adhered the soil to the cloth: MG Gel Foam, DirtGlue Dry, and FlexSeal Liquid. These three adhesives, whose descriptions are provided in Table 2, advanced to the next round of testing.

In the next round of testing, six peels were created on the same geo-slice sample using the MG Gel Foam (MG-1 and MG-2), DirtGlue Dry (DG-1 and DG-2), and Flex Seal Liquid (FS-1 and FS-2), as shown in Figure 4. The resulting peels were evaluated based on the five criteria outlined in Table 3.

Results and Discussion

The results of the second phase of testing are presented in Table 4. In summary, the Flex Seal Liquid peels had the most desirable characteristics. They both dried relatively quickly and enhanced the stratification features well. They also met the thickness and flexibility requirements of Table 3. The DirtGlue Dry peels were the thinnest and most flexible, however, the amount of time required for them to cure was well outside the desired criteria. The MG Gel Foam peels cured the fastest and enhanced the stratification features very well, however, they were much thicker and less flexible than desired.

Based on the criteria used to evaluate the peels in this study, the FlexSeal liquid is recommended for making geo-slice peels. While this adhesive does not have identical properties to the OH-4 grout used by Takada and Atwater (2004), the resulting peels were considered acceptable. Furthermore, the FlexSeal liquid product does not present concerns with environmental regulations and can easily be found in local hardware stores.

Table 2. Grout descriptions (modified from Yost et al. 2021a)

Type	Manufacturer	Product	Description	Treatment
Permeation Grout	Mountain Grout	MG Gel Foam	Single component, low viscosity hydrophilic polyurethane resin	1:3 grout to water ratio; pour directly on sample
Erosion Control Product	Global Environmental Solutions	DirtGlue Dry	Acrylate-based fine granulated powdered polymer	spread directly on sample, thoroughly expose to water to activate
Liquid Rubber	Swift Response	FlexSeal Liquid	Liquid rubber (clear)	undiluted; pour directly on sample

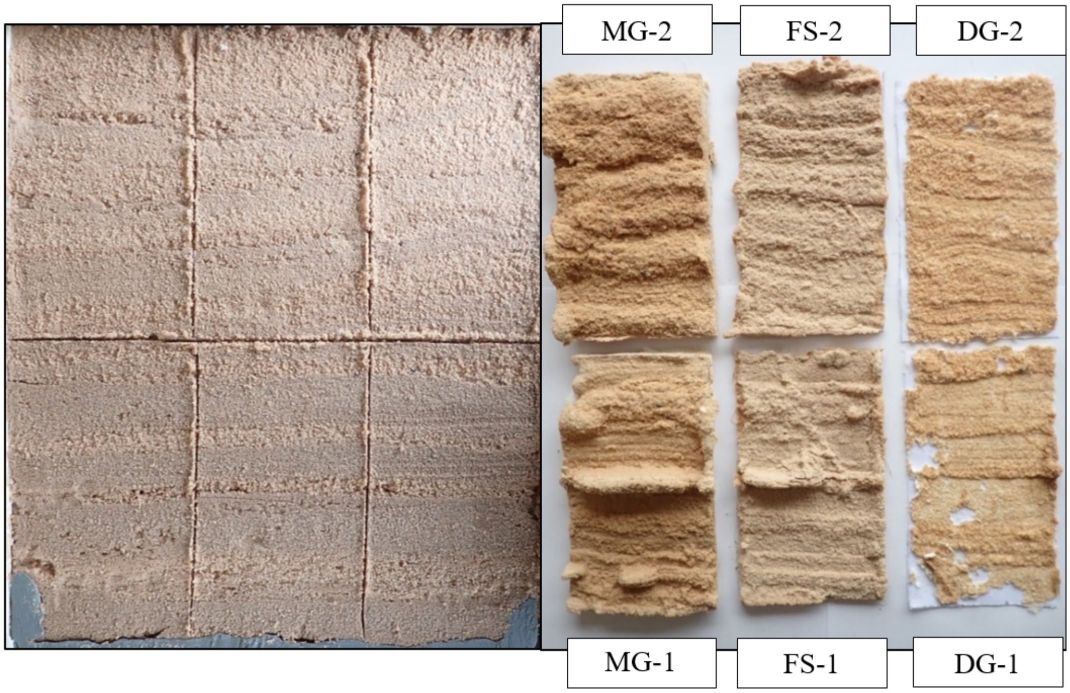


Figure 4. a) Geo-slice sample divided into six 4”×8” sections; and b) Corresponding peels created using MG Gel Foam [MG-1, MG-2], Flex Seal Liquid [FS-1, FS-2], and DirtGlue Dry [DG-1, DG-2] (Yost et al. 2021a).

Table 3. Geo-slice Peel Evaluation Criteria for Phase 2 of Testing (Yost et al. 2021a)

Characteristic	Target
Thickness of Peel	Less than 2.0 cm
Flexibility of Peel	Able to roll for storage and transport (as evaluated by bending tests consisting of draping the center of the peel over a 2-cm-thick metal rod and measuring the angle of bending with respect to horizontal)
Time to Cure	Less than 24 hours, 1 to 2 hours preferred
Enhancement of Stratification Features	Grout permeates more deeply in coarser-grained layers than in finer-grained layers
Hazard Level of Adhesive	Non-toxic, non-hazardous preferred

Table 4. Testing Results of Geo-slice Peels (modified from Yost et al. 2021a)

Product	Peel ID	Peel Thickness (cm)	Angle of Bending (°)	Time to Cure (hrs)	Enhancement of Stratification Features	Relative Hazard Level
MG Gel Foam	MG-1	0.9 to 3.8	7	1 to 2	Yes	High
	MG-2	1.6 to 2.9	17			
DirtGlue Dry	DG-1	0.2 to 1.0	90	34 to 36	Yes	Low
	DG-2	0.2 to 0.4	90			
Flex Seal Liquid	FS-1	0.5 to 2.0	16	2 to 4	Yes	Medium
	FS-2	0.4 to 1.6	43			

Conclusions and Future Work

Shortcomings in our current characterization methods of near-surface geologic profiles have important consequences on the engineering analyses that rely on accurate subsurface data. This research specifically focuses on the development of better characterization and analysis techniques for "complicated" soil profiles that consist of multiple thin, interbedded layers of varying soil types. It was shown that the Material Point Method (MPM) is a very effective tool for numerically modeling CPT penetration in layered soil profiles. This method can be used to generate complementary measured and true tip resistance data in layered soil profiles for use in development and validation of multiple thin-layer correction/inversion procedures. In future work, a suite of synthetic data using the MPM models will be created for this purpose.

It was also shown that geo-slicing is a promising soil sampling technique that can be used in conjunction with more traditional characterization methods to obtain an undisturbed and highly detailed understanding of the subsurface stratigraphy. Adhesives readily available in the United States are capable of creating acceptable geo-slice peels and can serve as less hazardous options to the previously used Japanese OH-4 grout. Future work will explore alternative liquid rubber products in comparison to the FlexSeal liquid recommended by this study. Furthermore, full-scale testing of both the geo-slicing and the geo-slice peel techniques will be performed.

Both the numerical modeling of CPT and the geo-slicing sampling technique are novel ways to improve the characterization of complicated near-surface soil profiles. The implementation of these methods will improve the accuracy of engineering predictions at sites with complicated soil profiles.

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