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CHARACTERIZATION OF PIEDMONT RESIDUAL SOIL AND SAPROLITE IN MARYLAND

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ABSTRACT

Residual soils in the Eastern Piedmont Physiographic province are difficult to characterize because of the unique mineralogy and development of the soils. They are derived in place by weathering of the underlying gneiss and schist bedrock, and are characterized by a gradual transition from soil to decomposed-rock to rock with no clear demarcation between the strata. The soils generally consist of low plasticity micaceous clayey silts, sandy silts and silty sands. It is often difficult to obtain undisturbed samples of these soils and Intermediate Geo-Materials, so most shear strength and compressibility properties are derived from experience or correlations with index parameters such as the SPT N-value and Atterberg limits.

For the State of Maryland's Intercounty Connector (ICC) Project, the General Engineering Consultant (GEC), Intercounty Connector Corridor Partners (ICCCP) Joint Venture working directly for the Maryland State Highway Administration (MSHA), performed a Preliminary Geotechnical Subsurface Exploration (PGSE) during the procurement phase so that the Design-Build (DB) teams would develop preliminary designs on which to base their technical and price proposals. As part of the PGSE performed by the GEC for Contract A of the ICC, several undisturbed samples were obtained so that the shear strength parameters could be determined on relatively undisturbed samples. An attempt was made to correlate the SPT N-values and laboratory testing with seismic refraction geophysical exploration to estimate engineering parameters for design of cut slopes, shrink/swell, a cut/cover tunnel, and several bridges for the three general strata. Not only were undisturbed samples tested to determine the shear strength parameters, remolded samples, compacted to 95% of the modified Proctor maximum dry density, were also tested to determine the remolded shear strength parameters for embankment construction.

INTRODUCTION

Residual soils or saprolite are soils that are derived in place from the weathering of the underlying bedrock. The subsurface profile is characterized by a gradual transition from soil to decomposed rock to unweathered rock with depth. The nomenclatures of these strata have not been standardized and tend to vary from project to project, as the geotechnical engineer tends to see fit. The properties of these materials differ from those derived from sediments and therefore care must be exercised when using correlations and models developed for sedimentary materials (Sowers and Richardson, 1983). In this paper, the properties of the residual materials for a project in the Piedmont region of central Maryland, USA are described.

PROJECT BACKGROUND

The Intercounty Connector (ICC) is an east-west 18.8-mile, limited access, six lane, toll corridor that will link central and eastern Montgomery County, I-270/370, with northwestern Prince George's County, I-95/US 1. The alignment for the ICC is shown in Fig. 1.

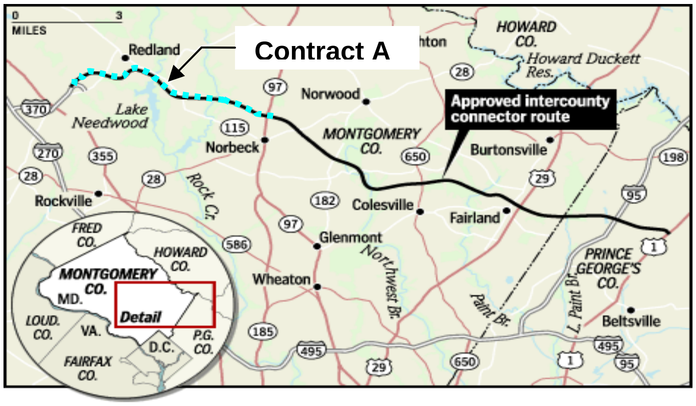

Fig. 1. ICC Alignment (Washington Post - July 12, 2005)

After a careful evaluation of various procurement options (including procurement as a single project), the ICC was divided into five Design-Build (DB) contracts: Contracts A through E. Each DB Contractor will be required to refine the preliminary design, prepared by the GEC, into final construction documents and then construct their portion of ICC. To provide the DB proposers some preliminary information during the procurement phase the GEC performed a preliminary subsurface exploration and released that information in a Geotechnical Data Report (GDR). As a part of the State's risk sharing approach, the State agreed to stand behind the preliminary characterization data; responsibility for evaluations analyses, and design rested with the DB Teams.

This paper discusses the site characterization that was developed based on the Preliminary Geotechnical Subsurface Exploration (PGSE) for the westernmost 7.2-miles of the project: Contract A (I-270/370 to MD 97).

PROJECT DESCRIPTION

The PGSE for the Contract A portion of the ICC extends from I-270/I-370 to approximately 600-feet east of Maryland 97 (approximately 7.2-miles) in Montgomery County, Maryland. Contract A includes the construction of mainline ICC, reconstruction of existing roadways where they will cross over the ICC or need to be re-aligned, and the construction of three interchanges with I-370/MD 355, I-370/Shady Grove Metro Access Road, and MD 97.

The content of the Contract A PGSE was incorporated as part of the Request for Proposals (RFP) documents. The PGSE program, in general, provided about a third of the required subsurface data required for the final design of this project. This program included Standard Penetration Test (SPT) borings with rock core sampling, a seismic refraction study, electrical resistivity testing, installation of groundwater monitoring wells, and a laboratory test program.

A rather large amount of laboratory testing was performed during the preliminary study because it was thought that the DB Team would have very little time to conduct such testing given the compressed DB schedule. This information would prove useful in developing the design-build contractor's bid submittal, reducing the risk to the contractor and in turn reducing the cost to the State. It is expected that the design-build contractor will drill additional borings and develop soil parameters for final design that will reflect the final design prepared by the design-build contractor.

The intent of the Contract A PGSE program was to provide the DB Teams with subsurface data for them to interpret for the detailed design and construction of this project. The PGSE was performed at selected locations along the project alignment; additional information is being obtained by the DB Team for the final design and construction of the project.
At the time the PGSE was in progress not all permits or access agreements were in place. Given the environmental sensitivity of the parks and wetlands; the local overloaded, dense traffic; and the relatively dense suburban residential neighborhoods, the PGSE was carefully developed to minimize impacts to existing wetlands, adjacent residences, parkland, and the traveling public. An environmental compliance inspector was assigned to each drill rig along with the geotechnical drill rig inspector to verify that the drillers and GEC complied with all environmental agreements.

SITE DESCRIPTION

The Contract A alignment for the ICC traverses through varied land uses, including agricultural lands, residential developments, wetlands, parkland, and forests. Elevations in this area range from approximately 300 to 600 feet above sea level.

In the area near I-270, substantial slopes and roadway embankments have been graded for construction of I-370. The project will be primarily constructed within land previously set aside for highway construction and as such, it had not been developed. The roadway will cross through Mill Creek, Rock Creek, and North Branch Parks. Residential development surrounds the project on both sides.

GEOLOGY AND SUBSURFACE EXPLORATION

Regional Geology

The project site is located in the Eastern Section of the Piedmont Physiographic Province. The Piedmont extends from the Fall Zone on the east to the eastern edge of the Frederick Valley on the west and extends from northern New Jersey to Alabama, (Witczak, 1972). The Fall Zone is a region where the sediments of the Coastal Plain Physiographic Province overlay the rock formations of the Piedmont. The western edge is formed by the Triassic Lowland Province. This province is lower in elevation than the Piedmont and consists of Triassic and Ordovician limestones.

The rock formations in the Upland Section of the Piedmont consist of metamorphic and plutonic rocks that include Precambrian and Cambrian granites, gneisses, and schists. There are frequent quartz pegmatite intrusions from the Mesozoic as well as mafic rocks such as gabbros and dikes and sills. Frequent orogenic activity as well as the intrusive materials have created significant metamorphic processes that have severely altered the chemistry and physical structure of the bedrock. The faulting, fractures, and foliations have all been directly controlled by these forces and in turn have a marked affect on the non-isotropic engineering properties of the derived materials.

The geomorphology of the Upland Piedmont is characterized by many small hills cut by streams flowing in a dendritic pattern. Although rock outcrops are not uncommon (especially where streams are migrating laterally), gradual soil slopes predominate within the project area.

Three metamorphic rock mapping types are identified within the area of the alignment. These are believed to date from the early Paleozoic to late Precambrian periods and include schist, gneiss, and mafic rocks.

<u>Schist.</u> This material consists of units previously mapped as the Wissahickon and Marburg Formations, and includes Pelitic schist, mica schist, metagraywacke, and quartz-feldspar-mica schistose gneiss rock types. Schist is heavily foliated with fractures commonly oriented parallel to foliation. There are many small scale folds. Overbreak and rock load depend on the orientation of the excavation to the foliation. Squeezing ground in wet shear zones is probable. This can sometimes create slope instability in unpredictable ways in deep excavations. Scaling is slight to moderate. Schist is susceptible to shearing toward open cut faces. Intrusions of mafic rocks are mapped within this formation. The static modulus of elasticity may range from one to eight million psi (Froelich, 1975).

<u>Gneiss.</u> This material consists of units previously mapped as the Sykesville, Wissahickon, and Laurel Gneiss formations and includes schistose gneiss, granite, granofels, pegmatite, and granodiorite rock types. In this region, gneiss frequently forms deep residual soils with massive bedrock pinnacles. Multiple joint sets frequently split the gneiss into blocks. The static modulus of elasticity may range from four to twelve million psi (Froelich, 1975).

<u>Mafic Rocks.</u> This material consists of units previously mapped as Sam's Creek Metabasalt, Norbeck Quartz Diorite, and the Georgetown Complex and includes meta-igneous, metavolcanic, and volcaniclastic greenstone; epidote-chlorite schist, amphibolite, chlorite-actinolite-talc schist, metagabbro, tonalite, metadiorite, etc. These rocks may be massive or schistose. Mafic rocks may have many fractures commonly filled by veins of quartz, calcite, or other minerals. The static modulus of elasticity may range from one to twelve million psi (Froelich, 1975).

Chemical weathering of all three rock types has created large volumes of residual soils within the project area. Physical weathering has not been a major factor in the development of the residual materials due to the protection from the vegetation and the moderate temperatures. The thickness of overburden within the project ranges from over 50-ft to exposed bedrock at the ground surface (Froelich, 1975). In many areas, the relic rock structure is evident even in areas where the material has completely weathered into soil (Mayne and Brown, 2003). The degree of weathering can vary quite rapidly in both the vertical and horizontal direction due mostly to the variations in the foliations of the underlying rock. In some areas boulder-size unweathered rock fragments can cause sampling and excavation difficulties, and can cause a very irregular contact zone in seismic refraction profiles. In other areas, the weathering may leave pinnacles of relatively unweathered material nearly to the ground surface with relatively softer soil zones between. This is particularly common in areas with intrusive metaigneous pegmatite and dikes. The strata change in an almost random manner, but is actually tied closely to the chemical composition, degree of weathering, fracturing, and thermal, chemical and physical metamorphic history (Sowers and Richardson, 1983). The principal discontinuities in rock and residual material generally are parallel to the foliation banding. This is important in evaluating the stability of excavations (Wirth and Zeigler, 1982).

In many locations, fluvial erosion has stripped away residual soil and deposited the material in stream valleys as river alluvium.

<u>Preliminary Geotechnical Subsurface Exploration (PGSE)</u>

The PGSE for the project consisted of drilling 392 SPT borings, with rock core sampling. The field work within this area was conducted in several phases between May 2004 and August 2006.

For the PGSE all drill rigs had automatic hammers except for one. The drill rig type and hammer type was recorded and tracked during the PGSE.

<u>SPT Sampling</u>. Soil borings were advanced using hollow stem augers or casing. Soil samples were obtained at a maximum 5.0-feet interval in accordance with the SPT procedure. Disturbed soil samples were recovered from the split barrel sampler for visual identification and laboratory index testing.

In addition, bulk samples were obtained from auger cuttings from select borings.

Relatively Undisturbed Samples. Relatively undisturbed samples of fine-grained soils were obtained using either a thin-walled tube sampler or a double/triple core barrel sampler such as a Denison sampler or a Pitcher sampler.

The thin-walled tube sample, or Shelby tube, sampling procedure consists of slowly pushing a 3-inch diameter tube into the soil to minimize disturbance. Generally, this sampling method was suitable only in soils with SPT N-values less than about 20 to 25 blows/ft.

For material that could not be sampled using a Shelby Tube, either the Denison or Pitcher sampling method was used to obtain relatively undisturbed samples of denser soils that could not be adequately sampled using rock core procedures. These methods consist of an inner liner, an inner barrel with a cutting edge, and an outer rotating barrel. The relatively undisturbed sample with these methods was either obtained with or without the use of drilling fluid.

Rock Core Sampling. Bedrock was sampled using NQ II diamond bit with a double tube, swivel type barrel, which provides a 1.875-inch diameter core. Generally, rock coring was used to sample spoon or auger refusal materials. Spoon refusal was defined as material with SPT N-values of more than 50 blows/inch.

Seismic Refraction Study

To supplement the SPT borings, to explore areas of proposed deep excavations, and in areas that were not accessible due to access agreements or environmental permit limitations, seismic refraction techniques were used. Within the Contract A limits, the seismic refraction study consisted of 91 lines, totaling approximately 49,160-ft. The seismic refraction study consisted of setting seismic lines using a 24-channel SmartSeis Seismograph with 24-geophone sensors. An impulse source, consisting of 8 to 10-pound sledgehammer, was used to strike an aluminum plate to produce a shockwave through the ground surface

A seismic refraction survey typically involves the transmission of sound waves into the earth and recording the acoustic responses using a seismograph at set distances from a seismic energy source. The seismograph measures the time it takes for a compression sound wave generated by the seismic energy source to travel down through the layers of the earth and back up to detectors (called geophones) placed on the surface.

Geophones were placed at 5 and 10-ft intervals on the ground surface out to a maximum length of 120-feet away from the point of impact. Five shots were made for each geophone spread: a midpoint shot, two endpoint shots, and two far shots. Far shots were located at least one and a half of the crossover distance to obtain refracted arrivals for the third layer at all geophones. The crossover distance is the distance from the source at which the sound traveling along the top of the third layer replaces the sound traveling along the ground surface as the first arrival. Far shot distance ranged from 30 to 70-feet.

The arrival time of the sound wave at each geophone location indicated on the instrument was recorded. The velocity of the shock wave is dependent on the apparent density of the material encountered by the shockwave. Upon passing through a boundary between subsurface layers of variable densities, (ie; soil, decomposed rock, or rock) the shock wave is partly refracted. Geophones are spaced along the linear direction of the area under study and reflected shockwaves are recorded for analysis. It should be noted that seismic velocities of the waves are dependent on several factors that include depth of overburden, water content, existence of frozen material, porosity, composition, density of materials, and degree of fracturing. It is also possible that a shallow high velocity layer could blind the system to softer materials at greater depth.

The areas for planned seismic exploration were determined based on the location of proposed excavations such as in tunnel areas or where under passes will be built to carry cross traffic over the ICC or other deep road cuts.

LABORATORY TESTING PROGRAM

The following tables summarize the quantity of laboratory testing conducted for the PGSE. The laboratory testing program is further discussed below.

Table 1a. Summary of PSGE Laboratory Testing

Laboratory Test	ASTM Test Method	Number of Tests
Natural Moisture Content	D2216-05	1101
Grain Size Distribution with	D422-63	736
Atterberg Limits	D4318-00	736
Modified Proctor Moisture	D1557-00	55
California Bearing Ratio	D1883-99	11
UU Triaxial	D2850-03	16
UC Rock	D2938-95	103
Field PLT		447

Table 1b. Summary of PSGE Laboratory Testing

Laboratory Test	ASTM Test Method	Number of Samples	Total Number of Points
Direct Shear	D3080-04	37	110
Remolded Direct Shear	D3080-04	13	39
CU Triaxial	D4767-03	18	39
CU K_o Triaxial	D4767-03	4	8

SPT and Bulk Sample Testing

The laboratory index testing consisted of determining the natural moisture content, the grain-size distribution with hydrometer, and the Atterberg limits of selected soil samples recovered from the split barrel sampler. Such index and classification testing does not fully describe the Piedmont residual soils and is seldom used at all for describing the Intermediate Geo-Material. The American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification Systems (USCS) were devised with sedimentary soils in mind (Sowers and Richardson, 1983).

If the USCS is used the residual soil bands seem to alternate between SM and ML when actuality the mean grain size is near the #200 sieve size and the classifications are merely random variations about the D_{50}, (Mayne and Brown, 2003). It is probably more helpful to use the AASHTO classification system, as the boundary between fine grained and coarse grained soils is where the percent minus #200 sieve (0.075-mm) is 35% instead of 50% as in the USCS.

The mica content can frequently interfere with a meaningful classification using either of these two methods. The mica flakes can blind a sieve shifting the grain-size distribution curve to reflect a coarser grained soil than is actually the case. The mica content can also have a significant affect on the engineering properties. In this study, not much mica was encountered except in the highly plastic soils.

For the Intermediate Geo-Material, neither system is suitable. Most of the SPT sample recovery usually consists of pulverized rock dust and gravel-sized, broken rock fragments that will usually be classified as GM, GC or A-1b. None of these classifications suitably describes the behavior of these materials.

The modified Proctor moisture-density relationship, Resilient Modulus, and California Bearing Ratio (CBR), in addition to the soil classification tests, were performed on the bulk samples. In some areas a Shelby tube sample was obtained near a bulk sample to estimate the shrink/swell for earthwork estimates.

Laboratory testing was performed by The Robert B. Balter Company (RBB) of Owings Mills, Maryland, E2CR, Inc. (E2CR) of Baltimore, Maryland, and URS (URS) Corporation of Ft. Washington, Pennsylvania. Previous laboratory testing was completed by Maryland State Highway Administration, EBA Engineering, Inc. of Baltimore, Maryland and Hillis Carnes Engineering Associates of Annapolis Junction, Maryland.

Undisturbed Sample Testing

In addition to performing classification and index testing, the shear strength properties of selected undisturbed samples were determined using the following test methods: Unconsolidated-Undrained (UU) Triaxial, Direct Shear (DS), Isotropically Consolidated Undrained (CIUC) Triaxial with Pore Pressure, and Constant K_o Consolidated Undrained (K_oCUC) Triaxial compression. Shear strength testing consisting of the direct shear test was performed on some remolded sample as well.

Rock Core Testing

Selected rock core samples were tested in the laboratory to determine the unconfined compressive strength (UCC) and the elastic modulus of the parent bedrock samples. The unconfined compression testing of the rock core was performed by E2CR and RBB.

The unconfined compressive strength of selected rock samples was estimated based on the point load strength index (Is).

CORRELATIONS FOR DEVELOPMENT OF SOIL AND ROCK PARAMETERS FROM PGSE

Based on the PGSE data, correlations were developed for the residual, Intermediate Geo-Material, and parent bedrock which was encountered within the project limits.

There is no consensus of how to define or denote the strata in these areas. For this paper, residual soil, Intermediate Geo-Material, and bedrock are defined as the following. This is based on local experience and other studies such as in Smith (1987). For other examples see Wirth and Zeigler (1982), Smith (1987), and Sowers and Richardson (1983).

- Residual soil: SPT N-values less than 80-blows per foot (bpf). Seismic velocity less than 3,000-ft/sec.
- Intermediate Geo-Material: SPT N greater than 80-bpf and less than 50/1-inch (split spoon refusal). Seismic velocity ranging from 3,000 to 6,000-ft/sec.
- Parent Bedrock: Below split spoon or auger refusal. Seismic velocity greater than 6,000-ft/sec.

This paper evaluates the following correlation methods to develop soil parameters:

- SPT N Values
- Laboratory Index Testing
- Laboratory Undisturbed Testing
- Laboratory Testing for Remolded Samples
- Seismic Refraction Study
- Rock Core Testing

<u>SPT N Values</u>

To account for the factors that affect the SPT N value, such as: operator, equipment, and drilling method, we standardized the SPT N values to an efficiency of 60-percent. The SPT N_{60} values accounted for borehole diameter, sampling method, overburden stress, and rod length.

Figure 2 summarizes the normal distribution of the SPT N_{60} obtained from the PGSE. Approximately 66-percent of the soil samples obtained during the subsurface exploration were classified as residual material. For the Intermediate Geo-Material, all SPT N values that were recorded as 50-blows per inch were summarized for this study as 100-bpf.

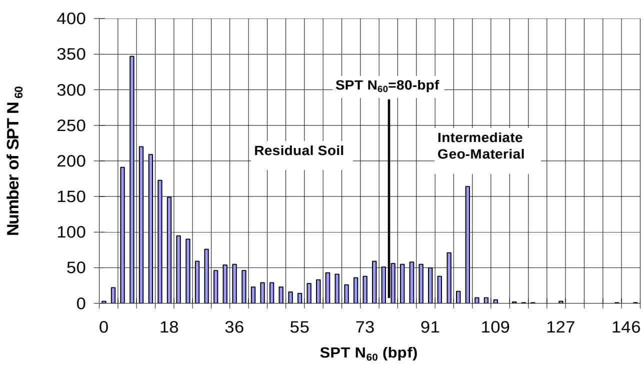

Fig. 2. Histogram of All Contract A PGSE SPT N_{60} (bpf)

Figure 3 summarizes the log normal distribution of the SPT N_{60} obtained from the PGSE.

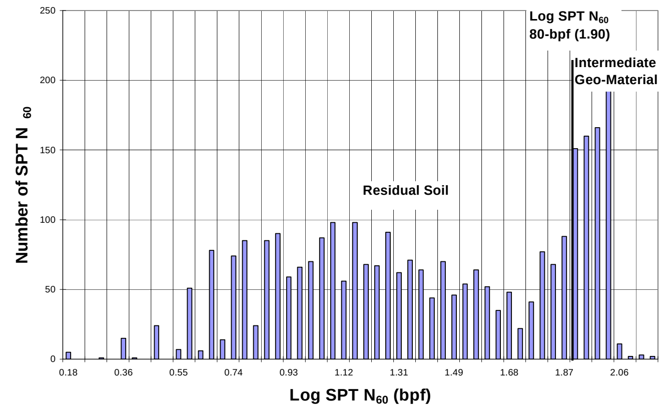

Fig. 3. Histogram of All Contract A PGSE Log SPT N_{60} (bpf)

Based on the SPT N_{60} values obtained from the PGSE, the drained angle of friction and undrained shear strength was estimated and summarized in Tables 2 and 3 for the residual and Intermediate Geo-Material strata, respectively. The drained angle of friction was estimated using the Meyerhof equation (1). The undrained shear strength was estimated using equation (2).

$$\phi = 27 + \frac{10N_{60}}{35} \tag{1}$$

$$S_u = \frac{N_{60}(1000)}{7.5} \tag{2}$$

The residual material was sub divided into three categories:

- Coarse Grained: Gravels and Sand – AASHTO Classification A-2-4 or better. Approximately 34-percent of the SPT samples obtained for the PGSE were from this stratum.

- Fine Grained: Silt and Clay – AASHTO Classification A-4 and A-5. Approximately 58-percent of the SPT samples obtained for the PGSE were from this stratum.

- Fine Grained: Highly Plastic - Silt and Clay – AASHTO Classification A-6, A-7-5, and A-7-6. Approximately 8-percent of the SPT samples obtained for the PGSE were from this stratum

Table 2. Summary of Residual Soil SPT N and N_{60} Values

Statistical Analysis	SPT N (bpf)	SPT N_{60} (bpf)	Drained ϕ (deg)	S_u (psf)
Residual Coarse Grained				
Maximum	79	84	45.2	-
Minimum	1	0	8.5	-
Average	22	21	33.4	-
Std Dev	16.8	15.3	4.3	-
Count	732			
Residual Fine Grained				
Maximum	79	76	-	8,711
Minimum	2	0	-	200
Average	19	18	-	2,319
Std Dev	15.2	14.6	-	1,677
Count	1287			
Residual Fine Grained (Highly Plastic)				
Maximum	56	52	-	6,933
Minimum	2	1.5	-	200
Average	10	9	-	1,266
Std Dev	8.5	8.6	-	1,151
Count	182			
ϕ = Angle of Friction S_u = Undrained Shear Strength				

Table 3. Summary of Intermediate Geo-Material SPT N and N_{60} Values

Statistical Analysis	SPT N (bpf)	SPT N_{60} (bpf)	Drained ϕ (deg)	S_u (psf)
Maximum	158	148	62.6	19,760
Minimum	80	50	35.8	4,000
Average	100	85	45	11,676
Std Dev	4.3	13.3	2.2	2,859
Count	849			
ϕ = Angle of Friction S_u = Undrained Shear Strength				

Laboratory Index Testing

The residual soil index testing from the PGSE laboratory testing is summarized in Table 4.

Table 4. Summary of Residual Soil Index Classification Testing

Statistical Analysis	NMC (%)	LL	PI	LI	% Fines
Residual Coarse Grained					
Maximum	63.2	49	19	1.7	72
Minimum	0.4	17	1	-15.6	5.4
Average	17.3	32	7	-1.3	39
Residual Fine Grained					
Maximum	81.9	58	10	3.4	93
Minimum	0.8	42	8	-4.2	6.4
Average	22.4	52	10	-2.0	61
Residual Fine Grained (Highly Plastic)					
Maximum	55.5	77	40	0.5	93
Minimum	5.4	32	12	-1.2	43
Average	28.6	51	19	-0.2	69
NMC: Natural Moisture Content **LL:** Liquid Limit **PI:** Plasticity Index **LI:** Liquidity Index **% Fines:** Percent Passing No. 200 Sieve					

Figures 4a and 4b summarize the residual soil, fine grained and highly plastic, residual angle of friction versus the liquid limit and plastic index, respectively.

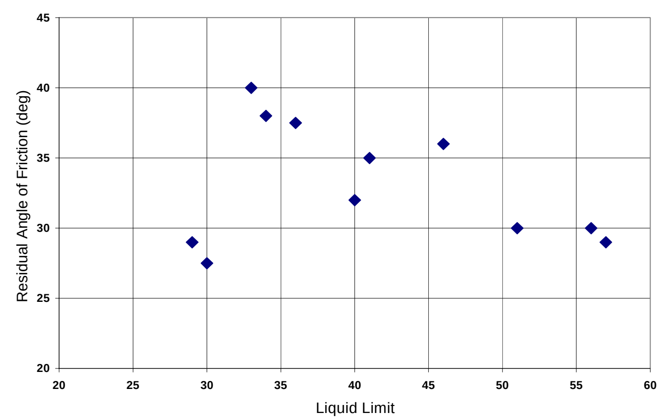

Fig. 4a. Liquid Limit vs Residual Angle of Friction

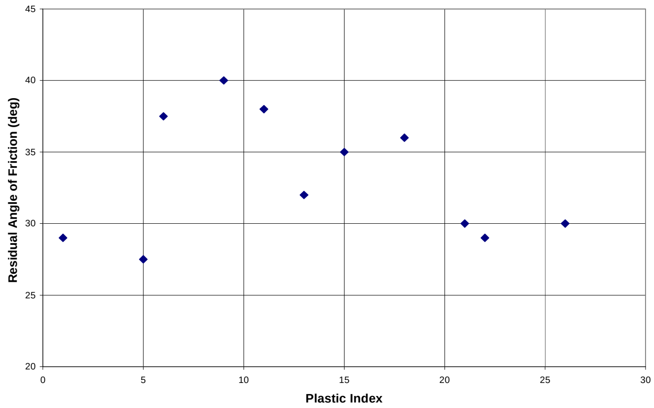

Fig. 4b. Plastic Index vs Residual Angle of Friction

Laboratory Undisturbed Sample Testing

The results of the undisturbed sample testing for all strata from the PGSE is summarized in Table 5 and is based on the results of the direct shear testing. Both the peak and residual soil parameters were recorded.

For the coarse grained material, the angle of friction from the SPT N correlations (33.4-degrees) seems to be an over estimate compared to the direct shear test results from both the peak and residual states (29.1 and 30.7-degrees, respectively).

For the fine grained material modeled in a drained condition, based on an average SPT N-value of 18-bpf, an average angle of friction for this material is 32-degrees. This is an overestimate of the angle of friction compared to the direct shear test results from both the peak and residual states (29.8 and 30.7-degrees, respectively).

Table 5. Summary of All Strata Direct Shear Test Results

Material Type	**Stress**	**ϕ (deg)**	**c (psf)**
Residual Soil	Peak	29.7	749
	Residual	30.8	447
	Residual	34.8	-
Residual Soil - Fine Grained	Peak	29.8	820
	Residual	30.7	468
Residual Soil - Course Grained	Peak	29.1	733
	Residual	30.0	516
Residual Soil - Highly Plastic	Peak	27.5	436
	Residual	29.9	231
Intermediate Geo-Material	Peak	33.0	1,067
	Residual	35.8	434
ϕ = Angle of Friction **c** = Cohesion			

The angle of friction from the SPT N-values maybe overestimated due to the presence of gravel-sized rock fragments. However, the angle of friction calculated from the SPT N-values is within a 90% confidence interval of the direct shear test results

The results of the direct shear testing, summarized in Table 5, are graphed with a 90-percent confidence interval in Figs. 5 through 15.

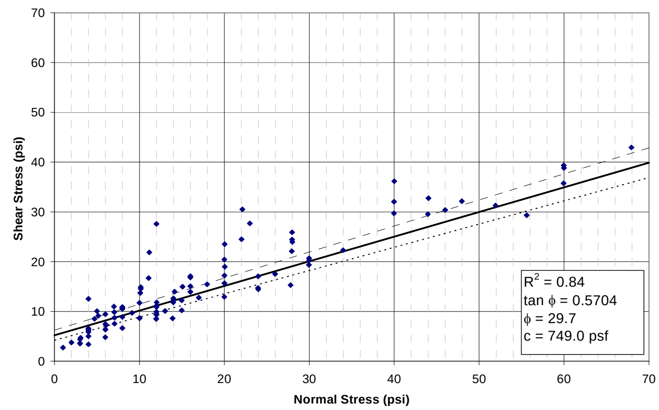

Fig. 5. Direct Shear Test Results for Residual Soil (Peak Stress)

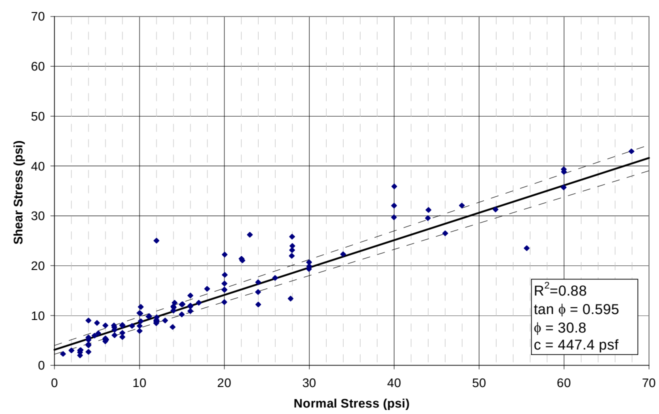

Fig. 6. Direct Shear test Results for Residual Soil (Residual Stress)

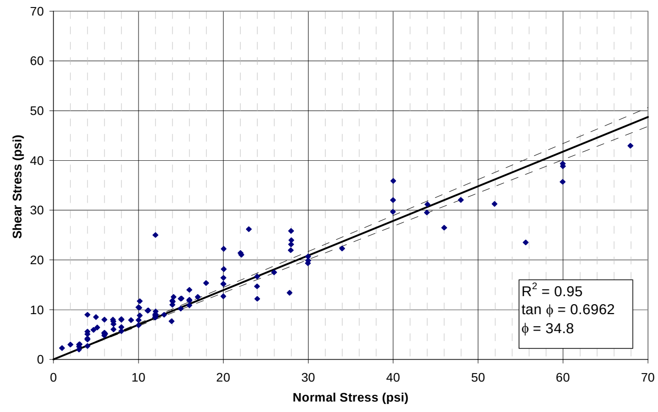

Fig. 7. Direct Shear Test Results for Residual Soil Assuming Zero Cohesion

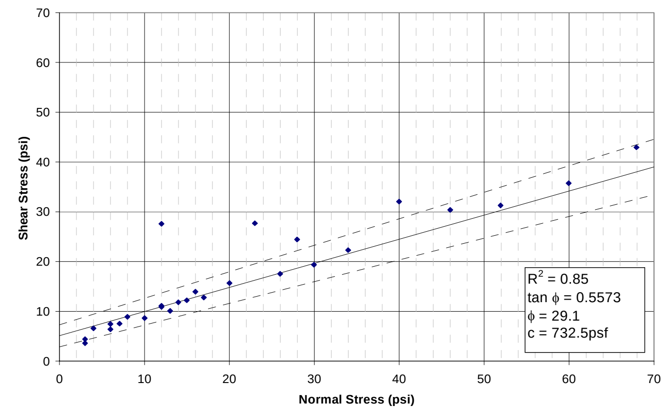

Fig. 10. Direct Shear Test Results for Residual - Coarse Grained (Peak Stress)

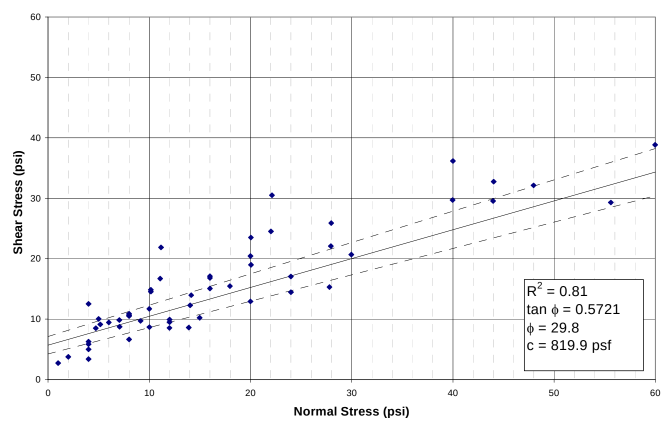

Fig. 8. Direct Shear Test Results for Residual - Fine Grained Soil (Peak Stress)

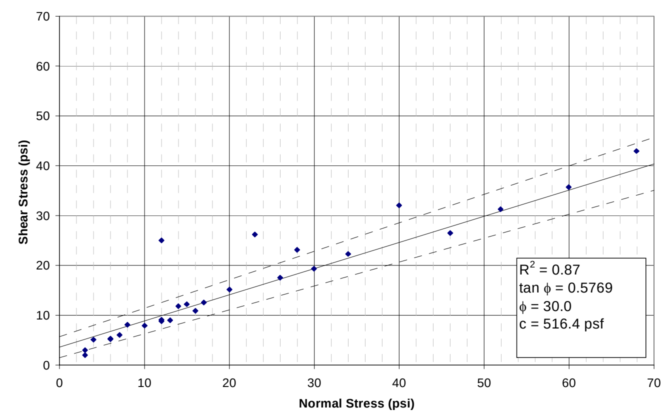

Fig. 11. Direct Shear Test Results for Residual -Coarse Grained Soil (Residual Stress)

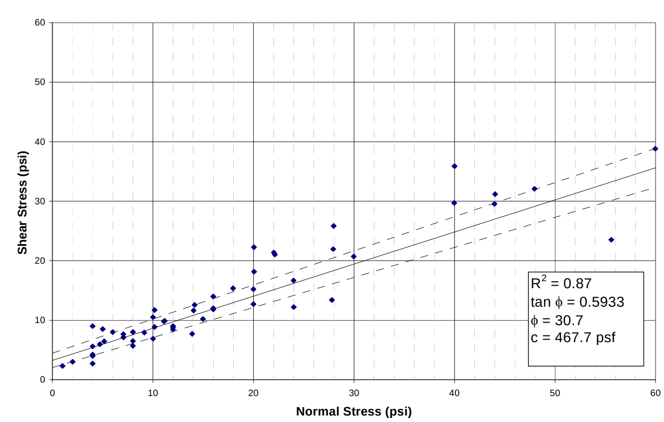

Fig. 9. Direct Shear Test Results for Residual -Fine Grained (Residual Stress)

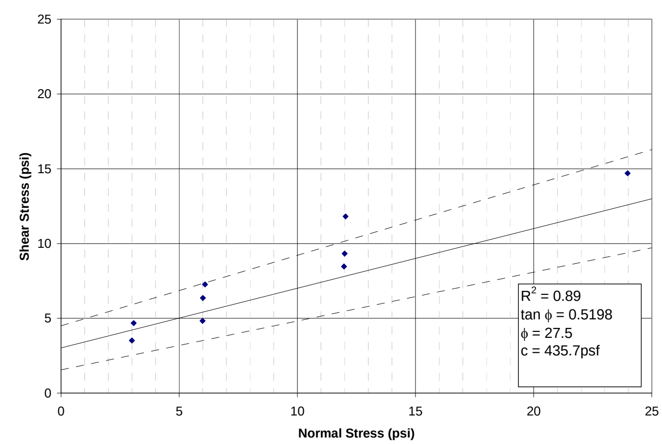

Fig. 12. Direct Shear Test Results for Residual – Highly Plastic Fine Grained Soil (Peak Stress)

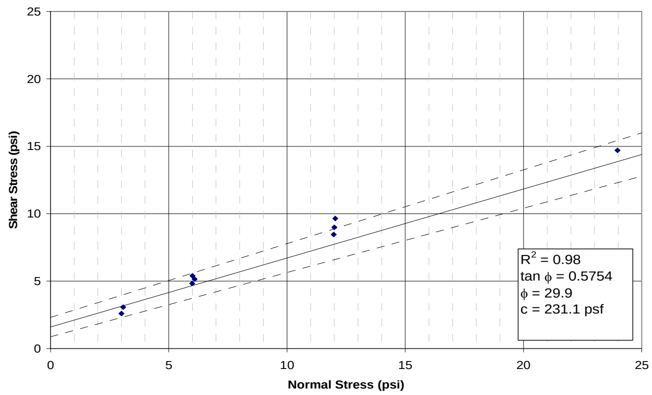

Fig. 13. Direct Shear Test Results for Residual - Highly Plastic Fine Grained Soil (Residual Stress)

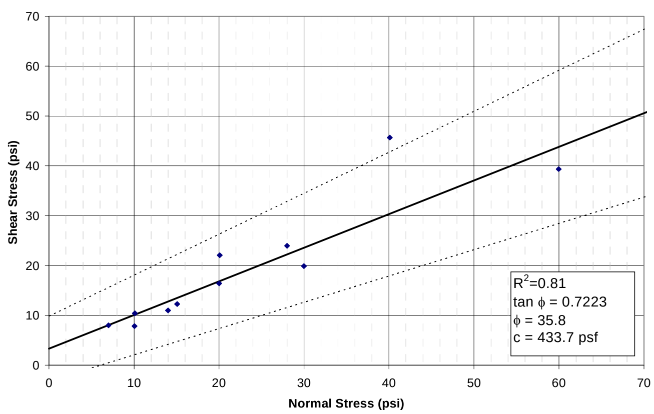

Fig. 14. Direct Shear Test Results for Intermediate Geo-Material (Residual Stress)

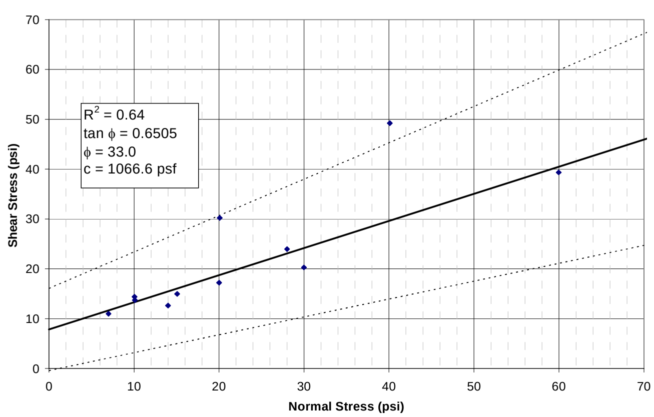

Fig. 15. Direct Shear Test Results for Intermediate Geo-Material (Peak Stress)

The results of the CIUC undisturbed sample testing from the PGSE is summarized in Table 6. The CIUC testing for the PGSE was only conducted for the residual soil material since there was not enough recovery in the Denison or Pitcher samplers to perform a CIUC on even one specimen from the sampler. This was unfortunate since the rock fragments in the Intermediate Geo-Material often made interpretation of the thin direct shear test samples difficult. All laboratory shear strength testing was performed on saturated samples.

Table 6. Summary of CIUC Residual Soil Test Results

Material Type	**Stress**	**Drained ϕ (deg)**	**c (psf)**
Residual Soil	Residual State	33.8	211
ϕ = Angle of Friction **c** = Cohesion			

Figure 16 summarizes the effective stress from the CIUC testing for the residual soil using equations 3a and 3b for p' and q'. The strength of the model indicated in Fig. 16 for the CIUC testing has a coefficient of determination of 95-percent. The CIUC is actually an undrained test, but it is often used in lieu of drained tests to develop drained soil parameters, as it is more economical than a Consolidated-Drained (CD) triaxial test.

Since the residual soils often behave in an undrained manner according to Sowers and Richardson (1983), we also evaluated the undrained shear strength from the CIUC tests as it varied with depth in Fig. 17. The normalized shear strength was 0.58 with the 90% confidence limits ranging from 0.78 to 0.89. That is consistent with 0.66 obtained by Mayne and Brown (2003). The rather large normalized shear strength is usually associated with over consolidated soils in sedimentary areas.

Using the relationship:

$$\left(\frac{S_u}{\sigma_{vo}}\right)_{oc} = OCR^{0.8}\left(\frac{S_u}{\sigma_{vo}}\right)_{nc} \tag{3}$$

$$p' = \left(\frac{\sigma_1 + \sigma_3}{2}\right) \tag{3a}$$

$$q' = \left(\frac{\sigma_1 - \sigma_3}{2}\right) \tag{3b}$$

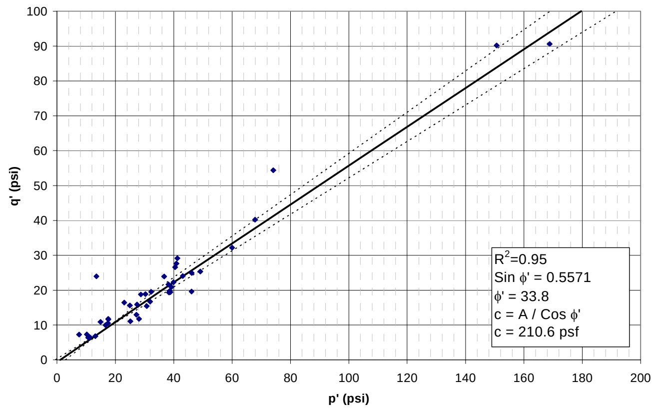

Fig. 16. CIUC Test Results for Residual Soil (Effective Stress)

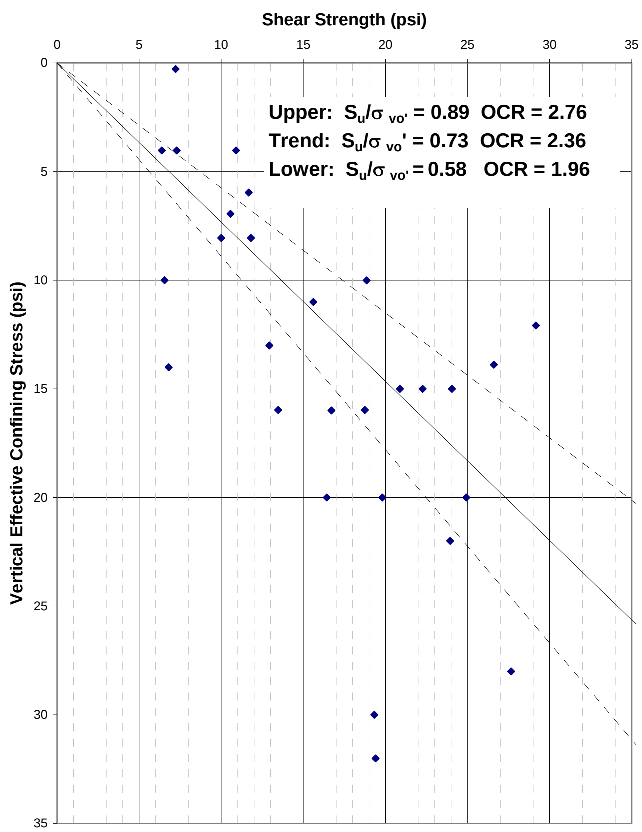

Fig. 17. Shear Strength vs Vertical Effective Stress for Residual Soil

Assuming $s_u/\sigma_{vo(nc)}$ is about 0.25, the Apparent Over Consolidation Ratio (AOCR) is between 2.0 and 2.8. Mayne and Brown (2003) cite several other researchers that generally tend to agree that the Over Consolidation Ratio (OCR) of residual soil should range from about 1 (or No Consolidation - NC) to no more than 5 based on one-dimensional laboratory tests. No one-dimensional consolidation laboratory tests were performed for this project as there were few samples of sufficient quality and there were few very large embankments proposed in the area of the drilling and sampling. These authors cite examples using the Cone Penetration Test (CPT) where the AOCR's range from 6 to 17, but using the Flate Plate Dilatometer Testing (DMT) AOCR's generally range from 1 (NC) to about 6.

<u>Remolded Direct Shear Testing</u>

Thirteen bulk samples of residual material were obtained across the project site for remolded direct shear testing. The residual remolded shear testing is summarized in Table 7.

The portion of shear strength attributable to remnant structure of the material can be estimated by comparing the results of undisturbed testing and remolded testing (Wirth and Zeigler, 1982).

Table 7. Summary of Remolded Residual Soil Direct Shear Testing

Material Type	**Stress**	**ϕ (deg)**	**c (psf)**
Residual	Peak	37.3	377
Remolded	Residual	36.4	50
ϕ = Angle of Friction **c** = Cohesion			

The results of the remolded direct shear testing, which is summarized in Table 7, are graphed with a 90-percent confidence interval in Figs. 18 and 19.

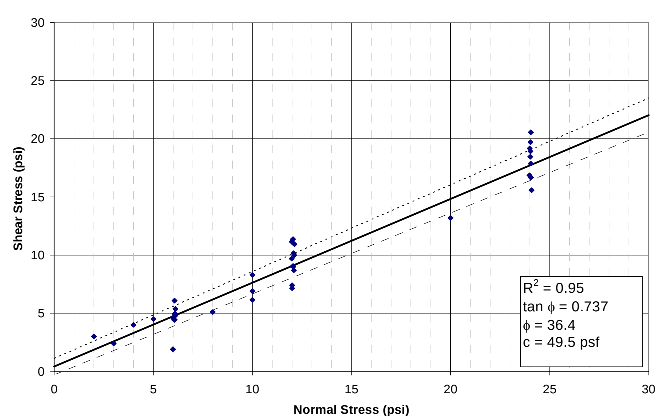

Fig. 18. Remolded Direct Shear Test Results (Residual Stress)

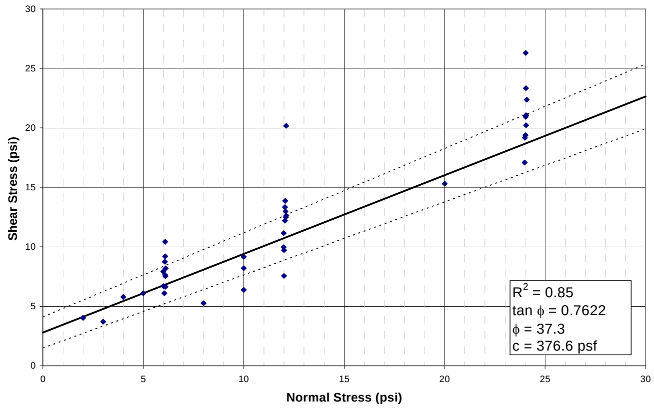

Fig. 19. Remolded Direct Shear Test Results (Peak Stress)

Rock Core UCC

Tables 8 through 10 summarize the UCC of the parent bedrock, per rock type, from the laboratory testing and the correlated UCC from the PLT.

Table 8. Summary of UCC: Gneiss

Rock Type	Maximum (psi)	Minimum (psi)	Average (psi)	Count
Gneiss - UCC	29,797	1,270	10,735	25
Gneiss - PLT	46,729	531	17,882	136

Table 9. Summary of UCC: Schist

Rock Type	Maximum (psi)	Minimum (psi)	Average (psi)	Count
Schist - UCC	30,540	240	6,280	74
Schist - PLT	48,828	110	8,147	305

Table 10. Summary of UCC: Quartz

Rock Type	Maximum (psi)	Minimum (psi)	Average (psi)	Count
Quartz -UCC	26,963	3,062	16,496	3
Quartz - PLT	78	25	57	6

SEISMIC REFRACTION STUDY

When comparing the results of the SPT borings and the seismic refraction study the indicated depths to the boundaries between residual soil/Intermediate Geo-Material and Intermediate Geo-Material/rock were not consistent. However, the seismic refraction did indicate that in some areas, there were significant variations in the depths to these boundaries and that was reflected in the inconsistent results from the SPT borings. Based on the experience of the authors, on other unpublished work and Hiltunen et. al. (2006) seismic tomography is a more reliable method than traditional seismic refraction. The more traditional method is very good at picking up variations the subsurface conditions between borings, and is useful in selection excavation equipment.

The seismic refraction study aiding in minimizing the risk to MSHA by providing continuous data in areas that had difficult access while reducing the time required to obtain tradition SPT borings.

SUMMARY

Residual soils and Intermediate Geo-Materials derived from the underlying rock in the Piedmont Physiographic Province are not nicely behaved and composed of easily predictable material properties. There are significant difficulties in sampling, testing and classifying these materials. The samples are often disturbed and with low recoveries. The rock fragments in the specimen often influence the test results or at the very least contribute noise the test data making interpretation difficult. The most commonly used classification system, the Unified Soil Classification System, is not a reliable method; the AASHTO is slightly better in the demarcation between fine-grained and coarse-grained soils is 35% and not 50%, the D_{50} for most residual soils.

It is often risky to use correlations between index parameters and shear strength or deformation parameters that have been derived for sedimentary soils and extend them to residual materials. This is particular the case when trying to determine the stress history of a site. Actual stress-strain tests or the DMT or Pressuremeter Testing (PMT) should be used instead of relying on correlations derived from sedimentary soils. This paper has presented and data and information that will aid in the future development of such correlations and the experience and judgment of geotechnical engineers.

ACKNOWLEDGEMENTS

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