recommended]. It is crucial to properly backfill the borehole with cementitious grout after the drilling phase is complete.

On some projects, the designer may need to modify the auger boring invert during the design phase to avoid specific ground conditions. Extending the test boring depth to 15 ft below the shaft subgrade provides the designer the opportunity to select a more ideal alignment. Consequently, if the pipe invert is lowered, new borings (along the proposed alignment) should be drilled to the modified invert plus at least one pipe diameter or 5 ft (1.5 m), whichever is deeper. It is recommended that a geotechnical engineering professional make the final decisions on the number and depth of borings on the basis of the complexity of the project and the local ground conditions. The more difficult and costly the proposed project and the more complex the ground conditions, the more extensive the subsurface investigation program should be.

Important decisions that must be made about the test borings include the sampling interval and the need for laboratory and field testing. The frequency and types of samples that must be obtained are a function of the project type and ground conditions. Samples should typically be taken at intervals no greater than 5 ft (1.5 m) and at changes in strata to identify thin strata that may have significant effects on auger bore tool selection. In addition, continuous sampling from one diameter above the pipe to one diameter below the pipe is prudent. Disturbed sampling procedures can be used to classify the soil and rock. For laboratory strength and compressibility tests, more-costly undisturbed samples are required. Field tests that might be used are in situ vane testing, pocket penetrometer testing, point load testing on rock, pressure meter testing, and pump testing for groundwater hydraulic conductivity.

4.3.3.1 Soil Borings Soil borings are typically advanced using either a rotary-wash casing advance system or a hollow stem auger (HSA). Each drilling method is dependent on the anticipated ground conditions and subject to different cost constraints. Both methods allow for the insertion of internal sampling rods for SPTs. SPTs are one of the most widely used methods of sample collection and allow for the generation of a blow count number (N). A standard sampler is driven into the ground by a 140-lb hammer, dropped from a height of 30 in. (760 mm). The number of blows required to drive the sampler every 6 in. (150 mm) is counted and recorded. The first 6 in. of each sample in each interval is typically excluded because it is considered disturbed, but the number of blows for the second and third 6-in. drives is used as the N-value. The total drive lengths are typically 18–24 in. (455–610 mm); however, continuous sampling is also possible. The blow count number (subject to certain corrections related to sampling/drilling equipment and geostatic stress state) is frequently used for geotechnical engineering correlations to in situ soil properties and is known as

Table 4-1. Soil Classification Based on SPT N-Value

Sand and Gravel (Cohesionless Soil)		Clay and Silt (Cohesive Soils)	
SPT N-value	Relative Density	SPT N-value	Consistency
0–4	Very loose	0–2	Very soft
4–10	Loose	2–4	Soft
10–30	Medium	4–8	Medium stiff
30–50	Dense	8–15	Stiff
>50	Very dense	15–30	Very stiff
—	—	>30	Hard

Source: Florida DOT (2004).

the SPT N-value. The SPT is a relatively simple test but a useful index of soil characteristics. If an auto-hammer (usually at N85 to 95) is used, calibration (through an equation) is needed to convert to N60. Table 4-1 provides soil classifications based on the N-value.

4.3.3.2 Rock Coring Rock coring (or core borings) can also be advanced through casings in overburdened soils and seated down to the bedrock. Such methods of casing advance include conventional wash boring, casing hammers (if the overburden is shallow), and more recently, Symmetrix or ODEX-style down-the-hole (DTH) hammers affixed to a casing bit. Coring is frequently completed in 5-ft (1.5-m) run lengths and can be double-tube or triple-tube sampled, depending on the budget and sample disturbance requirements. Each core is generated through either a Christiansen-style (X) or wireline (Q) core retrieval system, where X-style coring requires the removal of all the rods for a sample return and Q-style coring allows for the retrieval of only the sampling interval. Coring bits can be appropriately sized to cut an NQ [1.9-in. (50-mm)] or HQ [2.5-in. (65-mm)] diameter core; however, larger and smaller core sizes are available, depending on the project demands. Each core is inspected on site by an engineering geologist and collected in a box for subsequent laboratory testing. Most HSAs will not accommodate rock coring because of casing size limitations.

Core holes can be vertical (which is frequently the case) or inclined, depending on access limitations and project-specific requirements. Furthermore, rock cores can be postoriented through the use of a downhole acoustic/optical televiewer (A/O TV) logging sonde or through conventional core orientation methods. An oriented rock core is helpful in understanding the three-dimensional attributes of discontinuities (e.g., joints or bedding planes) within the rock mass fabric. For example, with projects that need to advance through terrain that has faults/shears, groundwater, or unstable rock slabs, the orientation of significant fractures in the rock will be important for project viability.

Rock cores should be inspected for general character and for the presence of discontinuities. The core log must, at a minimum, include the time per foot of core drilled; the loss or gain of drill fluid or water; discontinuities during drilling, such as drill string drops; descriptions of the rock type and basic mineralogy; a weathering/alteration index; the field-estimated compressive strength; the core length recovered versus the length drilled; and the rock-quality designation (RQD). A more detailed discontinuity log should also be prepared and should include the attributes of the joints or bedding planes, such as the set spacing, the aperture (i.e., opening) width, the presence and type of infilling material, the joint condition, and the joint roughness.

The user is referred to industry-accepted standards of soil and bedrock drilling, sampling, and identification, as provided for in ASTM D2113, *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration*, and those referenced within these standards.

4.3.4 Test Pits/Trenches and Surface Mapping

At some sites, it is good practice to supplement the test boring program with other forms of subsurface investigation, such as test pits or large-diameter drill holes. Test pits and large-diameter drill holes provide the following:

- Large-volume samples,
- Exposure of undisturbed ground for observation,
- Direct evidence of the type and distribution of objects that exist in the ground, and
- Important facts about ground behavior and groundwater inflows.

For ground suspected to contain cobbles and boulders or other forms of obstructions, test pits and trenches can be an effective exploration technique. Test pits and trenches can be excavated by hand or with a backhoe to provide an accurate assessment of the size and likely frequency of boulders and cobbles, the grain size distribution, and the nature and size of potential obstructions (URS Greiner Woodward Clyde 2000). OSHA requires that pits deeper than 5 ft be adequately supported or shored unless the excavation is sloped or made entirely in stable rock; local codes may be more stringent.

If adjacent or local surface excavations and natural slopes present themselves, it is wise to gather additional information on geologic materials. Such information from field outcrops can be correlated to the boring logs and can help build a three-dimensional picture of rock (and soil) distribution.

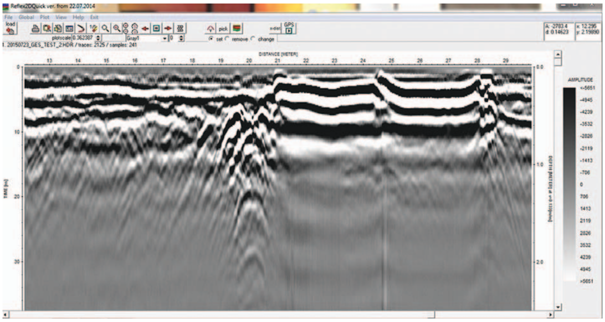

Fig. 4-1. Example image from GPR
Source: Geomatrix Earth Science Ltd. (2017)

4.3.5 Geophysical Investigation

Geophysical techniques and other emerging methods of nondestructive testing (NDT) may be effective in interpolating the subsurface conditions between the test borings, which are usually spaced along the bore alignment. These methods have been shown to be effective in some types of subsurface profiles. The extent of the investigation may vary depending on the local geological conditions and the risk management plan for the project. Subsurface investigation programs should identify general subsurface conditions and target the specific properties that could cause the project to fail, as identified by the initial desktop studies.

The geophysical subsurface investigation of soil deposits can be carried out by the following methods:

- Seismic refraction, which is good for shallow top-of-rock applications [typically <100 ft (30 m)];
- Seismic reflection, which is good for distinct units, faults, and increased depth penetration;
- Ground-penetrating radar (GPR) (Fig. 4-1), which is good for buried obstructions in soil;
- Cross-hole sonic logging;
- Electrical resistivity studies; and
- A/O TV logging.

4.4 GEOTECHNICAL PROPERTIES

Soil and rock characteristics that can significantly impact trenchless construction should be addressed in the geotechnical report. The following sections cover these characteristics and methods to obtain the required data. The relative importance of individual characteristics will vary from project to project, and no particular significance should be placed on the order in which the characteristics are listed and discussed. The following geotechnical properties should be considered during design for auger boring projects:

- Soil conditions;
- Potential for boulders, including their potential sizes and frequency;
- Rock conditions;
- Mixed-face conditions;
- Potential buried objects;
- Groundwater conditions;
- Potential for naturally occurring gas, such as methane and hydrogen sulfide;
- Contaminated groundwater or ground; and
- Artificial and environmentally sensitive features.

4.4.1 Soil Characteristics

It is important to understand the soil condition and potential reaction to the excavation process because soil condition will affect most, if not all, aspects of the boring process. Some ground can include coarse fraction (i.e., gravel and rock fragments) that significantly affects the ground behavior and boring performance. The soil type and properties impact the selection of the auger bore tooling, the production rate, and the ground stability. Laboratory testing to determine soil properties should be done in accordance with ASTM standards.

For cohesionless soils (predominantly sands and gravels), the important characteristics that should be identified include

- Grain size distribution,
- Coarse fraction,
- Cobbles and boulders (determined from test pits or large-diameter borings),
- Unit weight,
- Hydraulic conductivity, and
- Density (typically in terms of SPT N-values).

For cohesive soils (clays and silts), the important characteristics that should be identified include

- Moisture content,
- Plasticity index,
- Unit weight,
- Shear strength,
- Compressibility,
- Consistency (typically in terms of SPT N-values),
- Grain size distribution and hydrometer analysis,

- Coarse fraction,
- Stickiness/adhesiveness, and
- Swollenness.

Tunnelman's Ground Classification system, first described by Terzaghi (1950) and slightly refined by Heuer (1974), is designed to describe ground behaviors resulting from the excavation process and their impact on larger, conventionally constructed soft ground tunnels. The system is also a useful tool for the evaluation of soft ground conditions as they relate to auger boring. Descriptions of various ground behaviors, in terms of the Tunnelman system, and their impacts on boring are presented elsewhere (Bennett et al. 1995). General ground behaviors include:

- Raveling,
- Running,
- Flowing,
- Squeezing, and
- Swelling.

Table 4-2 provides a general description of different ground behaviors as they relate to stand-up times for conventional tunneling methods.

The clogging potential, also known as the stickiness/adhesiveness potential, is the tendency of plastic clay to adhere to the cutterhead and the augers, reducing productivity and/or halting production. Therefore, to prevent the clay from sticking to the cutterhead, the augers, and the outside surface of the casing, additional measures are needed if sticky clays are expected. The clogging potential is evaluated on the basis of the material's in situ moisture content and does not consider the effect of lubricants or other fluids introduced into the subsurface that could change the moisture content of the soil.

If clay soils are expected, it is helpful to measure the level of adhesion or stickiness to the cutterhead, the augers, and the outside surface of the casing. The plasticity index, the liquid limit, the water content, and the chemical composition and structure are the factors that affect the clay adhesion. The plastic limit, the liquid limit, and the moisture content are usually measured during the geotechnical investigation. Fig. 4-2 presents the low, medium, and high zones of clay adhesiveness. If the geotechnical engineer calculates the plasticity index (PI = liquid limit – plastic limit) and the consistency index [Eq. (4-1)] of the clay and plots them as shown in Fig. 4-2, he or she can make an informed recommendation of the levels of adhesion/stickiness.

$$\text{consistency index} = \frac{\text{liquid limit} - \text{water content}}{\text{liquid limit} - \text{plastic limit}} \tag{4-1}$$

The potential for clay soils to swell is a critical factor in the selection of the type of lubricant. Some clay soils swell when in contact with water,

Table 4-2. Tunnelman's Ground Classification for Soils

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support. Final lining can be constructed before ground starts to move.	Loess above the water table; hard clay, marl, cemented sand, and gravel when not highly overstressed.
Raveling	Slow raveling Fast raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed. This behavior is due to loosening or to overstress and brittle fracture (ground separates or breaks along distinct surfaces, as opposed to squeezing ground). In fast-raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table and slow raveling above. Stiff fissured clays may be slow or fast raveling, depending on the degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity and without a perceptible increase in water content. Ground has ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination with raveling at excavation surface and squeezing at depth behind surface.

(Continued)

Table 4.2. Tunnelman's Ground Classification for Soils *(Continued)*

Classification		Behavior	Typical Soil Types
Running	Cohesive running Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (from ±30 to 35 degrees). When exposed at steeper slopes, the materials run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before the material breaks down and runs. Such behavior is cohesive running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls and can flow for great distances, completely filling the tunnel, in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with a plasticity index in excess of about 30, generally containing significant percentages of montmorillonite clay.

Source: Heuer (1974).

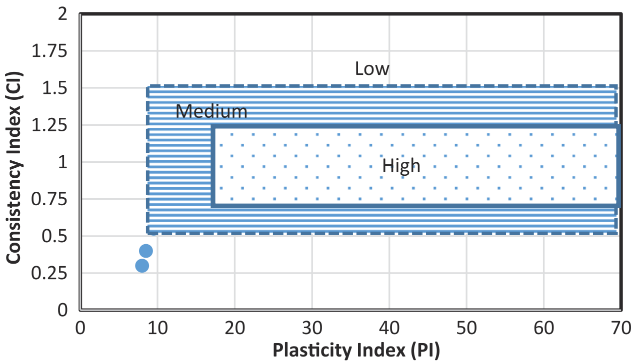

Fig. 4-2. Clogging potential of clay
Source: Thewes and Burger (2005)

reducing the overcut and increasing the friction between the soil and the casing, and consequently, the jacking force. Proper lubricant with hydration inhibitors reduces and prevents the swollenness and stickiness of clay.

4.4.2 Rock Conditions

On the basis of appropriate descriptive criteria (i.e., from ASTM or ISRM), the field subsurface and surface site investigation strategy for projects in which rock is anticipated should determine the following:

- Depth and extent of bedrock,
- Rock type (e.g., lithology and classification),
- RQD,
- Weathering/alteration index,
- Discontinuity set frequency/spacing,
- Discontinuity surface characteristics (e.g., aperture, infill material, roughness, shape, and joint roughness coefficient),
- Presence of fracture-controlled groundwater,
- Discontinuity orientation (e.g., dip and dip direction), and
- Estimated intact rock UCS (via field index and point load strength).

Conventional rock coring accompanied by DTH A/O TV logging can be a cost-effective way to arrive at the field data outlined above. This coring process also allows the user to collect data on discontinuity orientation.

After the field investigation phase, laboratory testing should be completed on select samples obtained in the field to arrive at reasonable estimates of the following:

- Hardness (e.g., Shore),
- UCS,

- Direct shear strength,
- Tensile strength (e.g., Brazilian),
- Punch penetration,
- Abrasiveness (e.g., Cerchar and Taber),
- Mineralogical (thin sections), and
- Slake durability and swelling tendencies (for shale).

Depending on the depth of burial, the geologic region of the proposed work, and the project risk, additional field testing may be necessary and may include the determination of the in situ rock mass modulus of deformation or horizontal stresses through pressure meter testing. In addition, packer testing (e.g., the Houlsby method) can be very useful in determining fracture-controlled hydraulic conductivity and the behavior of discontinuities under controlled head conditions.

Reliable exploration and test methods are available for determining the rock mass characteristics listed above. Drilling and sampling procedures that both maximize core recovery and minimize core damage are essential to the development of a reliable rock mass characterization program. Depending on the quality of the rock, in less massive rock formations triple-tube (e.g., HQ and NQ) coring generally provides the least sample disturbance. When collecting rock cores, it is wise to take photographs with a scale and color guide immediately after opening the sample. Once the sample is opened or moved, it will begin to deviate from its in situ condition because of exposure to the atmosphere and vibration.

Developed by Deere (1963), RQD is widely used as an indicator of overall rock mass quality and was the first rock mass classification method. RQD is one of the most widely used and referenced indexes for construction in bedrock and is calculated as the sum of the lengths of the intact cores greater than 4 in. (100 mm), divided by the total core run length (expressed as a percentage). Rock quality determinations can be incorporated within empirical rock mass classification schemes (e.g., the rock mass rating (RMR) and Q) that were developed for the construction of larger tunnels (Bieniawski 1974; Barton et al. 1974). Rock mass classifications can be used to develop estimates for rock mass properties and behavior, like the modulus of deformation and the stand-up time, respectively. RQD and the subsequent rock mass classification schemes can vary with respect to orientation. When drilling only vertical holes, engineers must be aware that the recorded RQD may not be the lowest RQD, unless it is based on the volumetric joint count method presented by Palmstrom (2005).

The weathering index, the fracture frequency, the joint characteristics, and the overall rock mass quality are also important for estimating borability (the capacity of the rock in front of the cutterhead to be ripped, chipped, and broken into smaller pieces to allow the advance of the cutterhead). The estimation of block sizes can be an important factor in evaluating the