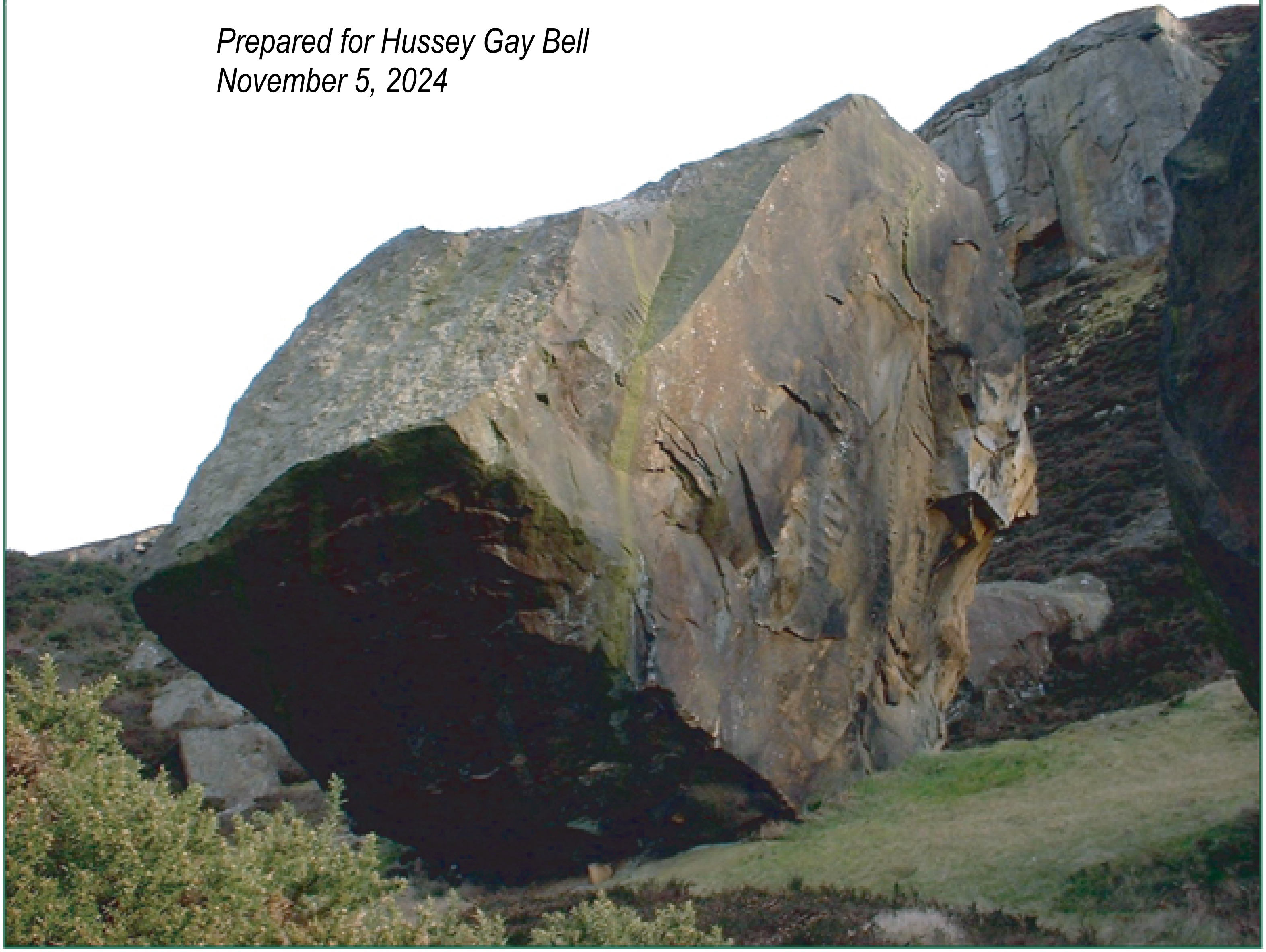
APPENDIX F

GEO HYDRO ENGINEERS

Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Union County 911 Center
Shoe Factory Road
Blairsville, Georgia
Geo-Hydro Project Number 242482.20

Prepared for Hussey Gay Bell
November 5, 2024



Mr. Reid Dyer, PLA
c/o Mr. Kevin Hamby
Hussey Gay Bell
322 West Main Street, Suite 2E
Blue Ridge, Georgia 30513

November 5, 2024

Report of Subsurface Exploration
and Geotechnical Engineering Evaluation
Union County 911 Center
Shoe Factory Road
Blairsville, Georgia
Project Number 242482.20

Dear Mr. Dyer:

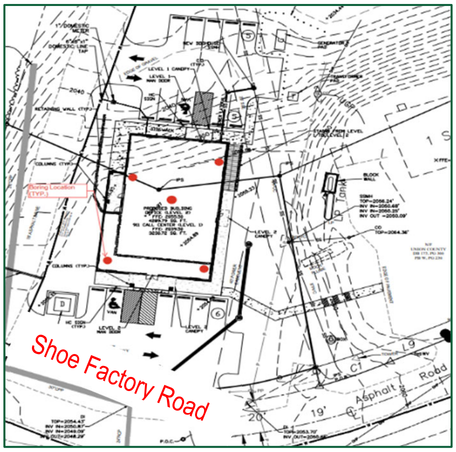
Geo-Hydro Engineers, Inc. has completed the authorized geotechnical engineering services for the above referenced project. The scope of services for this project was outlined in our proposal number 242482.P0 dated September 23, 2024.

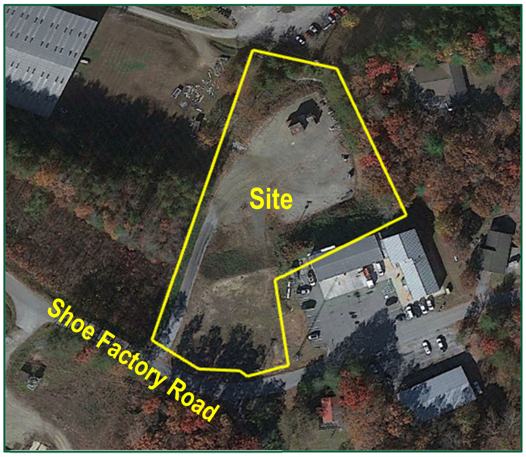
PROJECT INFORMATION

We understand that Union County is in the planning phase for a new 911 center next door to the existing Union County Fire Department Station 1 on Shoe Factory Road in Blairsville, Georgia. Figure 1 in the Appendix shows the approximate site location.

The project consists of a new two-story, 3,200-square-foot 911 center building which we have assumed will have a structural steel frame with masonry walls and a lower level concrete slab-on-grade floor system. Based on our experience with similar projects we have assumed that column loads will not exceed about 150 kips with maximum wall loads no greater than 6 kips per lineal foot.

The project site is currently a combination of grassed and graveled areas which slope down to the north of site. Based on the project plans provided to us, the building will have a walk-out basement level with a finished floor elevation of 2039.5. We expect site grading to involve as much as 15 to 20 feet of cut to reach final grades for the building. The site plan excerpt below left shows the planned building layout and the annotated aerial photo below right shows the approximate project area and current site conditions.





EXPLORATORY PROCEDURES

Soil Test Borings

The subsurface exploration consisted of seven machine-drilled soil test borings performed at the approximate locations shown on Figure 2 in the Appendix. The test borings were located in the field using a hand-held GPS unit with pre-loaded boring coordinates. The elevations shown on the test boring records were obtained from the *Boundary and Topographic Survey for Union County 911 Center* dated August 27, 2024 by Hussey Gay Bell and the information is not certified as correct by this engineer. Users of this information do so at their own risk. In general, the boring locations and elevations should be considered approximate.

Standard penetration testing, as provided for in ASTM D1586, was performed at select depth intervals in the soil test borings. Soil samples obtained from the drilling operation were examined and classified in general accordance with ASTM D2488 (Visual-Manual Procedure for Description of Soils). Soil classifications include the use of the Unified Soil Classification System described in ASTM D2487 (Classification of Soils for Engineering Purposes). The soil classifications also include our evaluation of the geologic origin of the soils. Evaluations of geologic origin are based on our experience and interpretation and may be subject to some degree of error.

Descriptions of the soils encountered, groundwater conditions, standard penetration resistances, and other pertinent information are provided in the test boring records included in the Appendix.

Shear Wave Velocity Profile Analysis (SWVPA)

We used multi-channel analysis of surface waves (MASW) to develop a profile of shear wave velocity for the site to a depth of 100 feet. This method consists of recording source seismic energy (active) and environmental source noise (background) utilizing an L-shaped array of multiple high-resonant-frequency geophones. The resulting data was combined and processed using proprietary, third-party software to produce a shear wave velocity profile. The shear wave velocity profile location is presented on Figure 2 and the results are presented in Figure 3 in the Appendix.

REGIONAL GEOLOGY

The project site is located in the Blue Ridge Geologic Province of Georgia. Soils in this area have been formed by the in-place weathering of the underlying crystalline rock, which accounts for their classification as "residual" soils. Residual soils near the ground surface, which have experienced advanced weathering, frequently consist of red brown clayey silt (ML) or silty clay (CL). The thickness of this surficial clayey zone may range up to roughly 6 feet. For various reasons, such as erosion or local variation of mineralization, the upper clayey zone is not always present.

With increased depth, the soil becomes less weathered, coarser grained, and the structural character of the underlying parent rock becomes more evident. These residual soils are typically classified as sandy micaceous silt (ML) or silty micaceous sand (SM). With a further increase in depth, the soils eventually become quite hard and take on an increasing resemblance to the underlying parent rock. When these materials have a standard penetration resistance of 100 blows per foot or greater, they are referred to as partially weathered rock. The transition from soil to partially weathered rock is usually a gradual one, and may occur at a wide range of depths. Lenses or layers of partially weathered rock are not unusual in the soil profile.

Partially weathered rock represents the zone of transition between the soil and the indurated metamorphic rocks from which the soils are derived. The subsurface profile is, in fact, a history of the weathering process which the crystalline rock has undergone. The degree of weathering is most advanced at the ground surface, where fine grained soil may be present. And the weathering process is in its early stages immediately above the surface of relatively sound rock, where partially weathered rock may be found.

The thickness of the zone of partially weathered rock and the depth to the rock surface have both been found to vary considerably over relatively short distances. The depth to the rock surface may frequently range from the ground surface to 80 feet or more. The thickness of partially weathered rock, which overlies the rock surface, may vary from only a few inches to as much as 40 feet or more.

SOIL TEST BORING SUMMARY

Starting at the ground surface, boring U-1 encountered approximately 7 inches of gravel. The remaining borings did not encounter any measurable surface materials. Detailed measurements necessary for quantity estimation were not performed for this project. The thickness of surface materials should be expected to vary across the site. For planning purposes, we suggest a thickness of 8 inches to account for surface materials such as gravel and topsoil.

Below the gravel or starting at the ground surface, all borings encountered fill materials extending to depths ranging from about 12 to 17 feet. The fill was classified as silty sand or clayey sand with varying amounts of organics. Standard penetration test resistances recorded in the fill ranged from 7 to 23 blows per foot.

Beneath the fill materials, all borings encountered residual soils or partially weathered rock typical of the Blue Ridge Region. The residuum was classified as silty sand, clayey silt, and clayey sand. Standard penetration test resistances recorded in the residuum ranged from 15 to 46 blows per foot.

Partially weathered rock was encountered in borings U-3, U-4, and U-6 at depths ranging from about 12 to 22 feet. Partially weathered rock is locally defined as residual material having a standard penetration test resistance of 100 blows per foot or greater.

Materials causing auger refusal were encountered in borings U-2 through U-6 at depths ranging from 20 to 28 feet. Auger refusal is the condition that prevents further advancement of the rig using conventional soil drilling techniques. In most cases, auger refusal in residual materials is indicative of large boulders or mass rock.

Groundwater was not encountered in the test borings at the time of drilling. It is important to note that stabilized groundwater levels are typically higher than those measured at the time of drilling. It should also be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors and may rise in the future.

For more detailed descriptions of subsurface soil conditions, please refer to the test boring records and included in the Appendix.

Summary of Subsurface Conditions

Boring	Approx. Current Ground Elevation	Approx. Planned Elevation	Bottom of Fill)		Groundwater*		Top of PWR		Auger Refusal		Boring Termination	
			Depth (feet)	Elev.	Depth (feet)	Elev.	Depth (feet)	Elev.	Depth (feet)	Elev.	Depth (feet)	Elev.
U-1	2037	2039.5	12	2025	NE	---	NE	---	NE	---	15	2022
U-2	2054	2039.5	17	2036	NE	---	NE	---	28	2026	28	2026
U-3	2054	2039.5	17	2037	NE	---	**17**	**2037**	20	2034	20	2034
U-4	2054	2039.5	12	2042	NE	---	22	2032	25	2029	25	2029
U-5	2054	2039.5	17	2037	NE	---	NE	---	27	2027	27	2027
U-6	2054	2039.5	12	2042	NE	---	**12**	**2042**	20	2034	20	2034
U-7	2054	2039.5	17	2037	NE	---	NE	---	NE	---	20	2034

All Depths and Elevations in this Summary Table are Approximate
NE: Not Encountered
PWR: Partially weathered rock
*Groundwater level measured at time of drilling
Red: Partially weathered rock above or within 5 feet of finished floor elevation

EVALUATIONS AND RECOMMENDATIONS

The following evaluations and recommendations are based on the information available on the proposed construction, the data obtained from the test borings, and our experience with soils and subsurface conditions similar to those encountered at this site. Because the test borings represent a very small statistical sampling of subsurface conditions, it is possible that conditions different from those indicated by the test borings could be encountered during supplemental exploration and during construction.

Geotechnical Considerations

The following geotechnical characteristics of the site should be considered for planning and design:

- Fill materials were encountered in all borings extending to depths ranging from about 12 to 17 feet. Some of the recovered fill material samples contained varying amounts of organic matter fragments. We expect any fill encountered on site during construction to be variable, and it is possible that some management of poor-quality or loose fill will be necessary during construction. Any loose, unstable fill material that cannot be readily densified in place should be removed and replaced with well compacted structural fill.

- Based on the results of the test borings and our understanding of site grades, difficult excavation conditions should be expected within the planned building footprint. Partially weathered rock was encountered in borings U-3, U-4, and U-6 at depths ranging from about 12 to 22 feet (approximate elevations 2032 to 2042). The partially weathered rock encountered in borings U-3 and U-6 was above or within 5 feet of the planned finished floor elevation. Excavation of partially weathered rock typically requires large equipment capable of ripping. Due to the leverage required to pre-loosen partially weathered rock, it is often impractical to rip partially weathered rock in trench excavations, on sloping terrain, or in wet conditions.

- Materials causing auger refusal were encountered in borings U-2 through U-6 at depths ranging from 20 to 28 feet. For planning purposes, we recommend assuming that blasting will be necessary to remove material below the depth of auger refusal. Additionally, partially weathered rock with standard penetration resistances of 50/2", 50/1", or 50/0" may be difficult to rip and may require blasting to remove.

- Based on the results of the soil test borings, residual soils and fill materials should be reusable as structural fill. Routine adjustments of moisture content will be required. It should be noted that fill materials are inherently variable, and some fill materials on site may not be suitable for reuse as structural fill. Geo-Hydro should evaluate fill material encountered on site to evaluate its suitability for reuse as structural fill.

- At the time of drilling, groundwater was not encountered in the test borings. Based on the results of the borings, we do not expect groundwater to be a major hindrance to design or construction. It is important to note that the groundwater levels recorded in the borings were measured at the time of drilling. Stabilized groundwater levels are typically higher than those measured at the time of drilling.

Regardless of groundwater conditions, the contractor should be prepared to manage runoff during wet weather conditions, and subsurface drainage will be necessary behind all below-grade structures including foundation walls.

- Based on the results of the shear wave velocity profile analysis performed for the project, and following the calculation procedure in the 2018 International Building Code (Chapter 20, ASCE 7-16), the seismic *Site Class* for the site is *C*. The seismic design parameters are as follows: S_S=0.345, S_1=0.106, S_{DS}=0.299, S_{D1}=0.106.

- Based on the results of the soil test borings, it is our opinion that the planned 911 center building can be supported using conventional shallow foundations. For planning and design purposes, we recommend using an allowable bearing pressure of 3,000 psf for column loads no greater than 150 kips and wall loads not exceeding 6 kips per lineal foot.

The following sections provide recommendations regarding these issues and other geotechnical aspects of the project.

Existing Fill Materials

Fill materials were encountered in all borings extending to depths ranging from about 12 to 17 feet. The fill materials contained varying amounts of organics fragments. We expect any fill encountered on site during construction to be variable, and it is possible that some management of poor-quality or loose fill will be necessary during construction. Any loose, unstable fill material that cannot be readily densified in place should be removed and replaced with well compacted structural fill.

- The quality of existing fill materials can be highly variable, and test borings are often not able to detect all of the zones or layers of poor quality fill materials.

- Layers of poor quality fill materials that are less than about 2.5 to 5 feet thick may often remain undetected by soil test borings due to the discrete-interval sampling method used in this exploration.

- The interface between existing fill materials and the original ground surface may include a layer of organic material that was not properly stripped off during the original grading. Depending on its relationship to the foundation and floor slab bearing surfaces, an organic layer might adversely affect support of footings and floor slabs. If such organic layers are encountered during construction, it may be necessary to “chase out” the organic layer by excavating the layer along with overlying soils.

- Subsurface exploration is simply not capable of disclosing all conditions that may require remediation.

General Site Preparation

Brush, topsoil, roots, and other deleterious materials should be removed from the proposed construction area. Any existing utilities should be excavated and removed from the building footprint. Additionally, site clearing, grubbing, and stripping should be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive rutting and mixing of topsoil and debris with underlying soils. All excavations resulting from rerouting of underground utilities or demolition of below-grade structures should be backfilled in accordance with the Structural Fill section of this report.

We recommend, wherever possible, that areas to receive structural fill be proofrolled prior to placement of structural fill. Areas of proposed excavation should be proofrolled after rough finished subgrade is achieved. Proofrolling should be performed with multiple passes in at least two directions using a fully loaded tandem axle dump truck weighing at least 18 tons. Proofrolling must be avoided within 10 feet of existing structures, walls, hardscapes, and utilities that will remain. If low consistency soils are encountered that cannot be adequately densified in place, such soils should be removed and replaced with well compacted fill material placed in accordance with the Structural Fill section of this report. Proofrolling should be observed by Geo-Hydro to determine if remedial measures are necessary.

For budgeting purposes, we suggest considering that approximately 30 percent of the aggregate building and pavement areas will require undercutting and recompaction or replacement extending to a depth of about 2 feet below current grades (fill areas) or planned finished grades (cut areas). *The suggested stabilization approach is intended only as a tool to estimate a cost associated with ground stabilization. The need for, extent of, location, and optimal method of ground stabilization should be determined by Geo-Hydro at the time of construction based on actual site conditions. The extent and cost of ground stabilization may exceed the suggested budgetary estimate.*

Items related to old homesteads which can be of concern for site development include domestic water wells and septic system tanks and drain fields. Water wells, if encountered, must be abandoned in accordance with the requirements of the Georgia Water Well Standards Act of 1985. The owner of the property is responsible for plugging the well in accordance with the requirements outlined in Circular 13, "Grouting and Plugging of Domestic Water Wells in Georgia" published by the Georgia Department of Natural Resources, Environmental Protection Division and the Georgia Geologic Survey. A water well contractor licensed to practice in Georgia must perform the actual work of plugging the well. Additionally, any existing septic systems and drain fields must be removed, and the resulting excavation should be backfilled in accordance with the recommendations in the Structural Fill section of this report.

During site preparation, burn pits or trash pits may be encountered. All too frequently such buried material occurs in isolated areas which are not detected by the soil test borings. Any buried debris or trash found during the construction operation should be thoroughly excavated and removed from the site.

Groundwater

At the time of drilling, groundwater was not encountered in any of the borings. Based on our understanding of the planned construction, we do not expect groundwater to be a major hindrance for design or construction.

Although groundwater is not expected to be a concern for site preparation and building construction, we must point out that groundwater levels vary and may rise in the future. Regardless of the groundwater conditions encountered in the borings, waterproofing and subsurface drainage is required for all retaining walls and building walls below grade.

Earth Slopes

Temporary construction slopes should be designed in strict compliance with OSHA regulations. The exploratory borings indicate that most soils at the site are Type B and Type C as defined in 29 CFR 1926 Subpart P. This dictates that temporary construction slopes in residual soils above the groundwater level for excavation depths of 20 feet or less should be no steeper than 1H:1V. Excavation slopes in fill materials, or in any soil type below the groundwater level, should be no steeper than 1.5H:1V. Temporary construction slopes should be closely observed on a daily basis by the contractor's "competent person" for signs of mass movement: tension cracks near the crest, bulging at the toe of the slope, etc. The responsibility for excavation safety and stability of construction slopes should lie solely with the contractor.

We recommend that extreme caution be observed in trench excavations. Several cases of loss of life due to trench collapses in Georgia point out the lack of attention given to excavation safety on some projects. We recommend that applicable local and federal regulations regarding temporary slopes, and shoring and bracing of trench excavations be closely followed.

Formal analysis of slope stability was beyond the scope of work for this project. Based on our experience, permanent cut or fill slopes should be no steeper than 2H:1V to maintain long term stability and to provide ease of maintenance. The crest or toe of cut or fill slopes should be no closer than 10 feet to any foundation or to the edge of any pavement that will support truck traffic. The crest or toe should be no closer than 5 feet to the edge of any pavements supporting cars or light truck traffic or parking. Erosion protection of slopes during construction and during establishment of vegetation should be considered an essential part of construction.

Excavation Characteristics

Based on the results of the test borings and our understanding of site grades, difficult excavation conditions should be expected within the planned building footprint. Partially weathered rock was encountered in borings U-3, U-4, and U-6 at depths ranging from about 12 to 22 feet (approximate elevations 2032 to 2042). The partially weathered rock encountered in borings U-3 and U-6 was above or within 5 feet of the planned finished floor elevation. Excavation of partially weathered rock typically requires large equipment capable of ripping. Due to the leverage required to pre-loosen partially weathered rock, it is often impractical to rip partially weathered rock in trench excavations, on sloping terrain, or in wet conditions.

Materials causing auger refusal were encountered in borings U-2 through U-6 at depths ranging from 20 to 28 feet. For planning purposes, we recommend assuming that blasting will be necessary to remove material below the depth of auger refusal. Additionally, partially weathered rock with standard penetration resistances of 50/2", 50/1", or 50/0" may be difficult to rip and may require blasting to remove.

It is important to note that the depth to rock or partially weathered rock can vary quite drastically over relatively short distances. It would not be unusual for rock or partially weathered rock to occur at higher elevations between or around some of the soil test borings.

For construction bidding and field verification purposes it is common to provide a verifiable definition of rock in the project specifications. The following are typical definitions of mass rock and trench rock:

- Mass Rock: Material which cannot be excavated with a single-tooth ripper drawn by a crawler tractor having a minimum draw bar pull rated at 56,000 pounds (Caterpillar D-8K or equivalent), and occupying an original volume of at least one cubic yard.

- Trench Rock: Material occupying an original volume of at least one-half cubic yard which cannot be excavated with a hydraulic excavator having a minimum flywheel power rating of 123 kW (165 hp); such as a Caterpillar 322C L, John Deere 230C LC, or a Komatsu PC220LC-7; equipped with a short tip radius bucket not wider than 42 inches.

The foregoing definitions are based on large equipment typically utilized for mass grading. Subsequent excavations for building foundations, retaining walls, and underground utilities are often performed with smaller equipment such as rubber-tired backhoe/loaders or even mini-excavators. If difficult excavation in dense soils or partially weathered rock is encountered, contractors will often request additional payment for mobilizing larger equipment than that which was anticipated during preparation of their construction bid. The amount of additional compensation, if any, and the minimum equipment size necessary to qualify for any additional compensation should be defined before the start of construction.

Reuse of Excavated Materials

Based on the results of the test borings and our observations, residual soils and fill materials appear to be suitable for reuse as structural fill. However, it is possible that some fill materials will not be suitable for reuse. Geo-Hydro should observe the excavation of materials to evaluate their suitability for reuse. Routine adjustment of moisture content will be necessary to allow proper placement and compaction of excavated soils.

It is important to establish as part of the construction contract whether soils having elevated moisture content will be considered suitable for reuse. We often find this issue to be a point of contention and a source of delays and change orders. From a technical standpoint, soils with moisture contents wet of optimum as determined by the standard Proctor test (ASTM D698) can be reused provided that the moisture is properly adjusted to within the workable range. From a practical standpoint, wet soils can be very difficult to dry in small or congested sites and such difficulties should be considered during planning and budgeting. A clear understanding by the general contractor and grading subcontractor regarding the reuse of excavated soils will be important to avoid delays and unexpected cost overruns.

Structural Fill

Materials selected for use as structural fill should be free of organic matter, waste construction debris, and other deleterious materials. In general, the material should not contain rocks having diameters over 4 inches. It is our opinion that the following soils represented by their USCS group symbols will typically be suitable for use as structural fill and are commonly found in abundance in the Blue Ridge region: (CL), (SM), and (ML). The following soil types are typically suitable but are not abundant in the Blue Ridge region: (SW), (SP), (SC), (SP-SM), and (SP-SC). The following soil types are considered unsuitable: (MH), (CH), (OL), (OH), and (Pt).

Laboratory Proctor compaction tests should be performed on representative samples of proposed fill materials to provide data necessary to determine acceptability and for quality control. Soils having a standard Proctor maximum dry density of less than 90 pcf should be considered unsuitable unless laboratory evaluations of their stress-strain characteristics indicate that they will perform acceptably. The moisture content of suitable borrow soils should generally be no more than 3 percentage points above or below their optimum moisture content at the time of compaction. Tighter moisture limits may be necessary with certain soils.

Suitable fill material should be placed in thin lifts. Lift thickness depends on the type of compaction equipment; but in general lifts of 8 inches loose measurement are recommended. The soil should be compacted by heavy compaction equipment such as a self-propelled sheepsfoot roller. If highly micaceous soils exist at finished subgrade elevation, a smooth-drum, steel-wheeled roller can often be used to compact loose surface soils. Within small excavations, such as those in utility trenches or around manholes, we recommend the use of “wacker packers” or “Rammax” compactors to achieve the specified compaction. Loose lift thicknesses of 4 to 6 inches are recommended in small area fills.

We recommend that structural fill be compacted to at least 95 percent of the standard Proctor maximum dry density (ASTM D698). The upper 12 inches of floor slab subgrade soils should be compacted to at least 98 percent of the standard Proctor maximum dry density (ASTM D698). Following Georgia DOT guidelines, the upper 12 inches of pavement subgrade soils should be compacted to at least 100 percent of the standard Proctor maximum dry density. Geo-Hydro should perform density tests during fill placement.

Earth Pressure (Cast-In-Place Walls)

Three earth pressure conditions are generally considered for retaining wall design: "at rest", "active", and "passive" stress conditions. Retaining walls which are rigidly restrained at the top and will be essentially unable to rotate under the action of earth pressure (such loading dock walls) should be designed for "at rest" conditions. Retaining walls which can move outward at the top as much as 0.5 percent of the wall height (such as free-standing walls) should be designed for "active" conditions. For the evaluation of the resistance of soil to lateral loads the "passive" earth pressure must be calculated. It should be noted that full development of passive pressure requires deflections toward the soil mass on the order of 1.0 percent to 4.0 percent of total wall height.

Earth pressure may be evaluated using the following equation:

$$p_h = K (D_w Z + q_s) + W_w(Z-d)$$

where: p_h = horizontal earth pressure at any depth below the ground surface (Z).
W_w = unit weight of water
Z = depth to any point below the ground surface
d = depth to groundwater surface
D_w = wet unit weight of the soil backfill (depending on borrow sources). The wet unit weight of most residual soils may be expected to range from approximately 115 to 125 pcf. Below the groundwater level, D_w must be the buoyant weight.
q_s = uniform surcharge load (add equivalent uniform surcharge to account for construction equipment loads)
K = earth pressure coefficient as follows:

Earth Pressure Condition	Coefficient
At Rest (K_o)	0.53
Active (K_a)	0.36
Passive (K_p)	2.8

The groundwater term, $W_w(Z-d)$, should be used if no drainage system is incorporated behind retaining walls. If a drainage system is included which will not allow the development of any water pressure behind the wall, then the groundwater term may be omitted. The development of excessive water pressure is a common cause of retaining wall failures. Drainage systems should be carefully designed to ensure that long term permanent drainage is accomplished.

The above design recommendations are based on the following assumptions:

- Horizontal backfill
- 95 percent standard Proctor compactive effort on backfill (ASTM D698)
- No safety factor is included

For convenience, equivalent fluid densities are frequently used for the calculation of lateral earth pressures. For "at rest" stress conditions, an equivalent fluid density of 66 pcf may be used. For the "active" state of stress an equivalent fluid density of 45 pcf may be used. These equivalent fluid densities are based on the assumptions that drainage behind the retaining wall will allow *no* development of hydrostatic pressure; that native sandy silts or silty sands will be used as backfill; that the backfill soils will be compacted to 95 percent of standard Proctor maximum dry density; that backfill will be horizontal; and that no surcharge loads will be applied.

For analysis of sliding resistance of the base of a cast-in-place concrete retaining wall, the coefficient of friction may be taken as 0.4 for the soils at the project site. This is an ultimate value, and an adequate factor of safety should be used in design. Customarily, retaining wall design includes a factor of safety which affects the global design. Using that design approach, it is not necessary to reduce the coefficient of friction as a design input. Such a reduction, coupled with the global factor of safety applied to the wall design, would place an unreasonable reduction in the calculation of the frictional resistance. The force that resists base sliding is calculated by multiplying the normal force on the base by the coefficient of friction. Full development of the frictional force could require deflection of the base of roughly 0.1 to 0.3 inches.

Foundation Design

After general site preparation and site grading have been completed in accordance with the recommendations of this report, it is our opinion that the planned 911 center building can be supported using conventional shallow foundations. For planning and design purposes, we recommend using an allowable bearing pressure of 3,000 psf for column loads no greater than 150 kips and wall loads not exceeding 6 kips per lineal foot. In addition, we recommend a minimum width of 24 inches for column footings and 18 inches for continuous wall footings to prevent general bearing capacity failure. Footings should bear at a minimum depth of 18 inches below the prevailing exterior ground surface elevation to avoid potential problems due to frost heave.

The recommended allowable soil bearing pressure is based on an estimated maximum total foundation settlement no greater than approximately 1 inch, with anticipated differential settlement between adjacent columns not exceeding about ½ inch. If the architect or structural engineer determine that the estimated total or differential settlement cannot be accommodated by the proposed structure, please contact us.

Foundation bearing surface evaluations should be performed in all footing excavations prior to placement of reinforcing steel. Geo-Hydro should perform these evaluations to confirm that the design allowable soil bearing pressure is available. Foundation bearing surface evaluations should be performed using a combination of visual observation, hand augering, and portable dynamic cone penetrometer testing (ASTM STP-399).

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, and replacement with well-compacted graded aggregate base (GAB) meeting Georgia Department of Transportation specifications for gradation. Stone replacement is generally performed to depths ranging from a few inches to as much as 2 times the footing width, depending on the actual conditions. For budgeting purposes, we suggest considering a contingency to treat approximately 20 percent of the foundation excavations using stone replacement extending to a depth of 3½ feet below bearing elevation. The actual quantity of stone replacement will be different and may exceed the suggested estimate.

Seismic Design

Based on the results of the shear wave velocity profile analysis performed for the project, and following the calculation procedure in the 2018 International Building Code (Chapter 20, ASCE 7-16), the seismic *Site Class* for the site is *C*. The seismic design parameters are as follows: S_S=0.345, S_1=0.106, S_{DS}=0.299, S_{D1}=0.106.

Based on the information obtained from the soil test borings, it is our opinion that the potential for liquefaction of the residual soils at the site due to earthquake activity is relatively low.

Floor Slab Subgrade Preparation

The soil subgrade in the area of concrete slab-on-grade support is often disturbed during foundation and superstructure construction. We recommend that the floor slab subgrade be evaluated by Geo-Hydro immediately prior to beginning floor slab construction. If low consistency soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted fill material placed in accordance with the *Structural Fill* section of this report or with well-compacted graded aggregate base (GAB).

Assuming that the top 12 inches of floor slab subgrade soils are compacted to at least 98 percent of the standard Proctor maximum dry density, we recommend that a modulus of subgrade reaction of 120 pci be used for design. This value is suitable only for light floor loads (no greater than 150 psf) and transient loads, and should not be used for designing thickened slab sections or floors supporting permanent or semi-permanent loads such as those from equipment and storage racks. For floor areas supporting permanent or semi-permanent loads from floor storage, storage racks, equipment, etc., we recommend using a modulus of subgrade reaction of 70 pci for design purposes.

Moisture Control for Concrete Slabs

To prevent the capillary rise of groundwater from adversely affecting the concrete slab-on-grade floor system, we recommend that all slab-on-grade construction in areas other than the apparatus bay be underlain by a minimum 4-inch thickness of open-graded stone. Use of #57 crushed stone meeting Georgia DOT specifications for gradation is suggested. The stone should be covered by a vapor retarder consisting of polyethylene sheeting at least 10 mils thick.

For any floor areas that may be subjected to relatively heavy wheel loads from vehicles, lift platforms, or other similar equipment, we recommend that slab-on-grade floors be underlain by a minimum 5-inch thickness of GDOT compliant graded aggregate base (GAB) compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557). The GAB must be covered by a vapor retarder as suggested above.

Flexible Pavement Design

Based on our experience with similar projects, assuming standard pavement design parameters, and contingent upon proper pavement subgrade preparation, we recommend the following pavement sections:

Entrance/Exit Driveways and Truck Traffic Areas

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave Type II	2
Asphaltic Concrete 19mm Superpave	2
Graded Aggregate Base (GAB) (Base Course)	8
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D698)	12

Automobile Parking and Automobile Traffic Only

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave Type II	2
Graded Aggregate Base (GAB) (Base Course)	6
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D-698)	12

A concrete thickness of 7 inches is recommended for the approach and collection zone in front of the dumpster, in loading/unloading zones, and in any designated truck turn-around areas. Please refer to the *Concrete Pavement* section of this report for concrete pavement recommendations.

The top 12 inches of pavement subgrade soils should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698). Scarification and moisture adjustment will likely be required to achieve the recommended subgrade compaction level. Allowances for pavement subgrade preparation should be considered for budgeting and scheduling.

GAB must be compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557).

All pavement construction should be performed in general accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications, will be critical to the performance of the constructed pavement.

Concrete Pavement

A rigid Portland cement concrete pavement may be considered. Although usually more costly, a Portland cement concrete pavement is typically more durable and requires less maintenance throughout the life cycle of the facility. Concrete thicknesses of 5 inches in automobile parking areas and 6 inches in driveways and truck traffic areas are recommended for this project. A concrete thickness of 7 inches is recommended for the approach and collection zone in front of the dumpster, in loading/unloading zones, and in any designated truck turn-around areas. A 600-psi flexural strength concrete mix with 4 to 6 percent air entrainment should be used. The concrete pavement should be underlain by no less than 5 inches of compacted graded aggregate base (GAB). GAB should be compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557). The top 12 inches of soil subgrade should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698).

The concrete pavement may be designed as a "plain concrete pavement" with no reinforcing steel, or reinforcing steel may be used at joints. Construction joints and other design details should be in accordance with guidelines provided by the Portland Cement Association and the American Concrete Institute.

In general, all pavement construction should be in accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications will be critical to the performance of the constructed pavement.

Pavement Design Limitations

The pavement sections discussed above are based on our experience with similar type facilities. After traffic information has been developed, we recommend that you allow us to review the traffic data and revise our recommendations as necessary.

Pavement Materials Testing

To aid in verifying that the pavement system is installed in general accordance with the design considerations, the following materials testing services are recommended:

- Density testing of subgrade materials.

- Proofrolling of pavement subgrade materials immediately prior to placement of graded aggregate base (GAB). This proofrolling should be performed the same day GAB is installed.

- Density testing of GAB and verification of GAB thickness. In-place density should be verified using the sand cone (ASTM D1556) or Nuclear Density Gauge method (ASTM D6938).

- Coring of the pavement to verify thickness and density (asphalt pavement only).

- Preparation and testing of beams and cylinders for flexural and compressive strength testing (Portland cement concrete only). The total number of test specimens required will depend on the number of concrete placement events necessary to construct the pavement.

* * * * * *

We appreciate the opportunity to serve as your geotechnical consultant for this project and are prepared to provide any additional services you may require. If you have any questions concerning this report or any of our services, please call us.

Sincerely,

GEO-HYDRO ENGINEERS, INC.

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GEORGIA
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KDJ/LEB/242482.20 - Union County 911 Center - Report leb.docx

APPENDIX

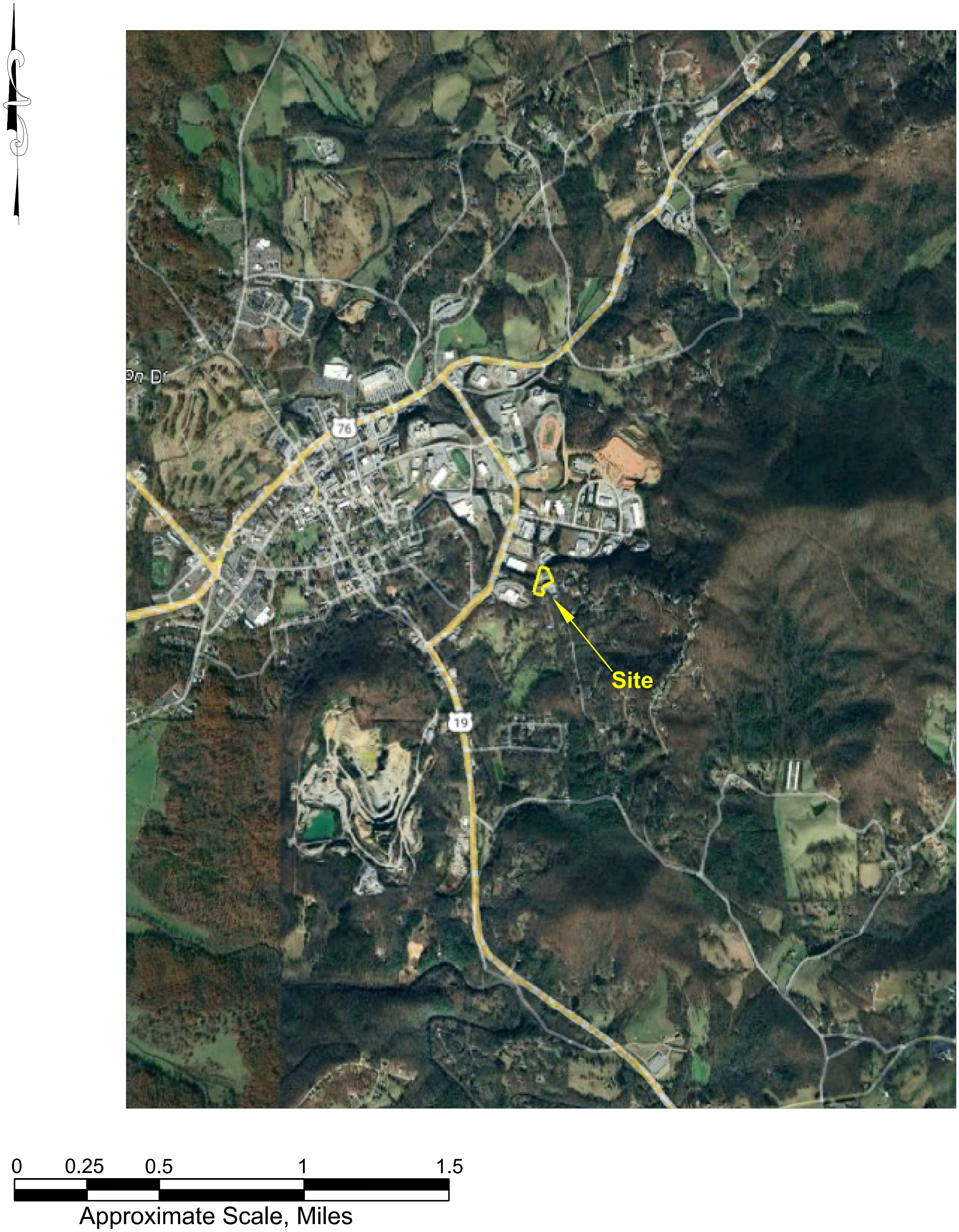


Figure 1: Site Location Plan

Union County 911 Center
Shoe Factory Road
Blairsville, Georgia
Geo-Hydro Project Number 242482.20

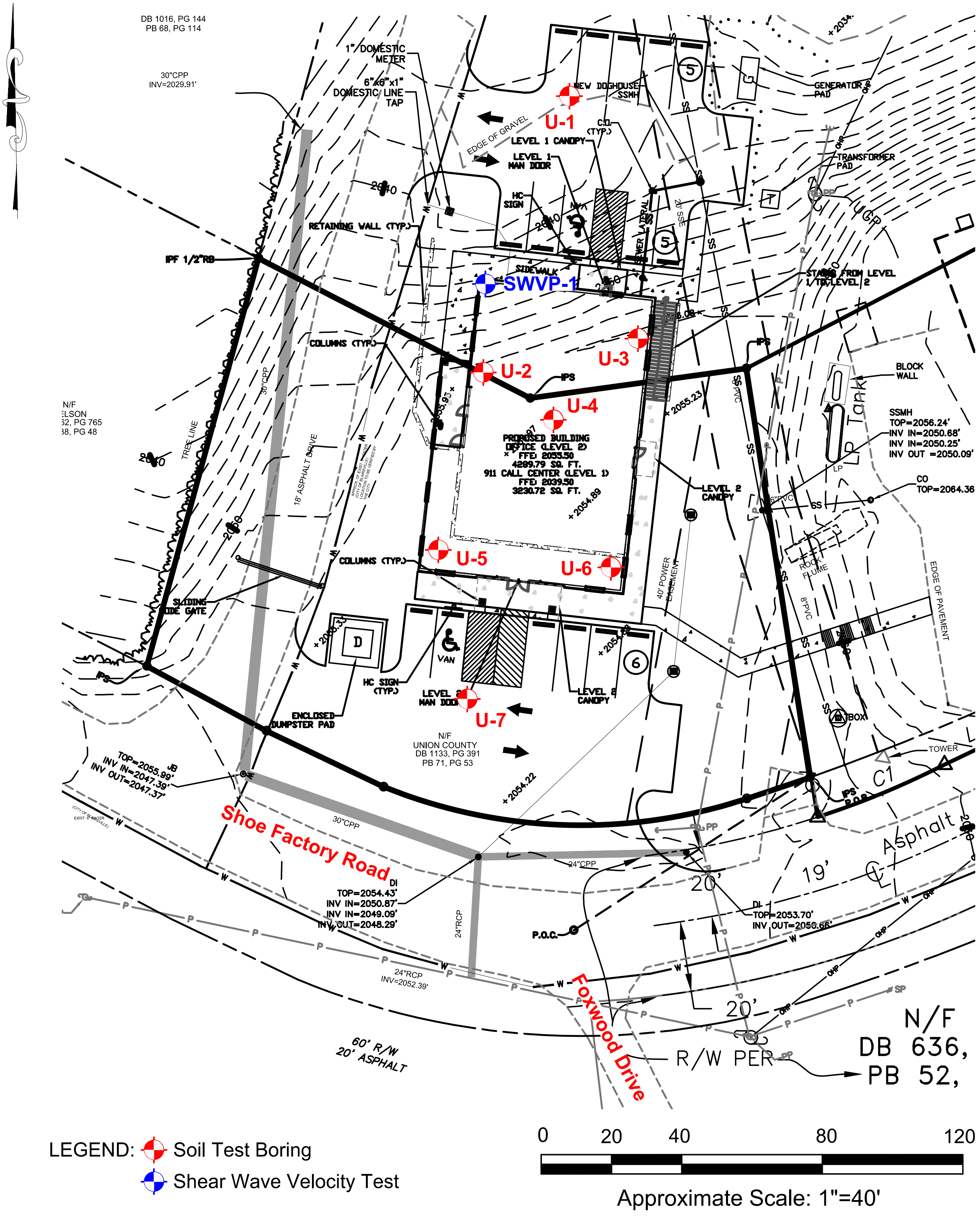
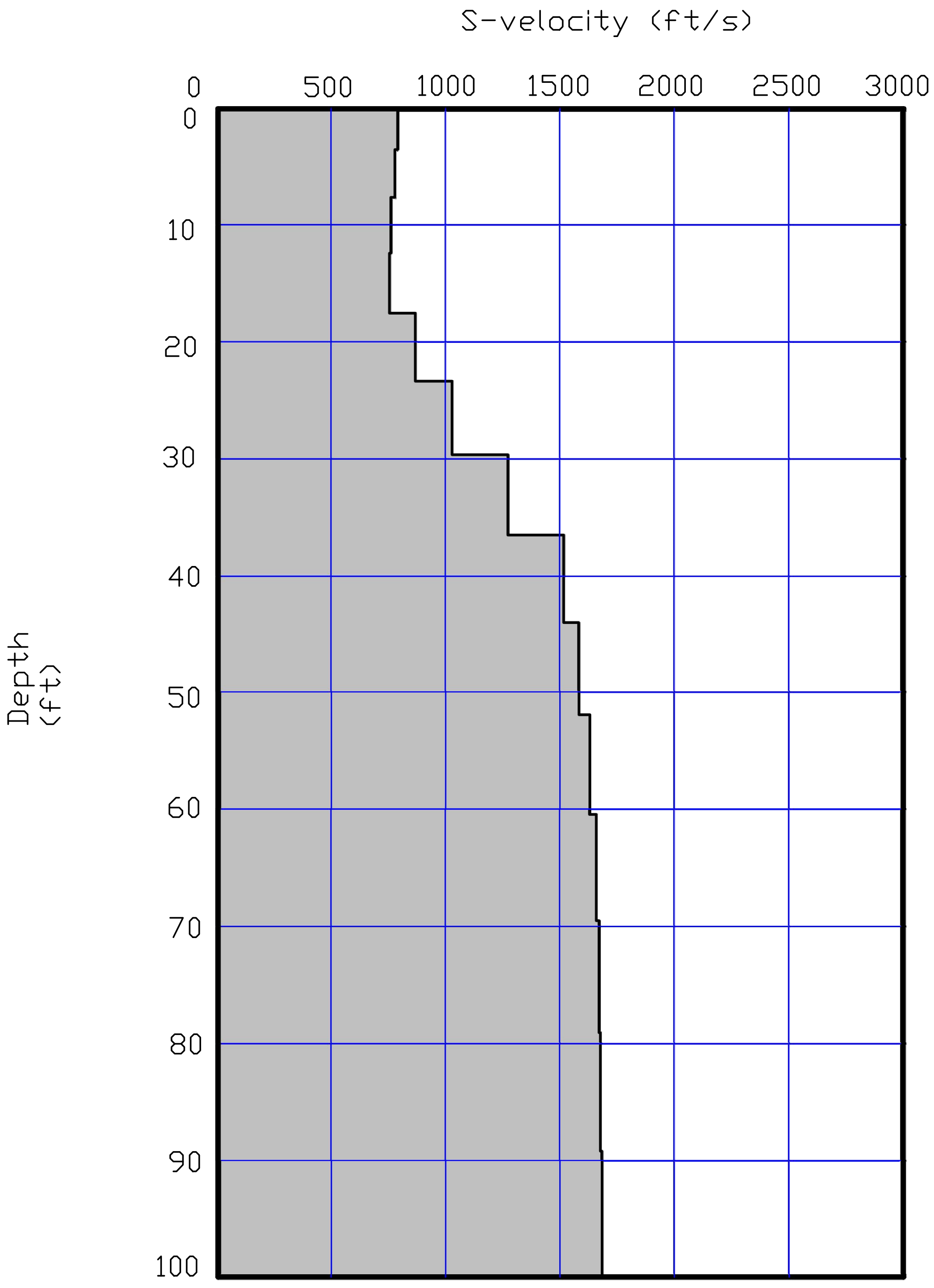


Figure 2: Boring Location Plan

Union County 911 Center
Shoe Factory Road
Blairsville, Georgia
Geo-Hydro Project Number 242482.20



$\overline{V}s$ = 1,250 ft/sec

Figure 3: Shear Wave Velocity Profile
(SWVP-1)

Union County 911 Center
Shoe Factory Road
Blairsville, Georgia
Geo-Hydro Project Number 242482.20

Symbols and Nomenclature

Symbols

▌	Thin-walled tube (TWT) sample recovered
▯	Thin-walled tube (TWT) sample not recovered
●	Standard penetration resistance (ASTM D1586)
50/2”	Number of blows (50) to drive the split-spoon a number of inches (2)
65%	Percentage of rock core recovered
RQD	Rock quality designation - % of recovered core sample which is 4 or more inches long
GW	Groundwater
▼	Water level at least 24 hours after drilling
▽	Water level one hour or less after drilling
ALLUV	Alluvium
TOP	Topsoil
PM	Pavement Materials
CONC	Concrete
FILL	Fill Material
RES	Residual Soil
PWR	Partially Weathered Rock
SPT	Standard Penetration Testing

Penetration Resistance Results

	Number of Blows, N	Approximate Relative Density
Sands	0-4	very loose
	5-10	loose
	11-20	firm
	21-30	very firm
	31-50	dense
	Over 50	very dense

	Number of Blows, N	Approximate Consistency
Silts and Clays	0-1	very soft
	2-4	soft
	5-8	firm
	9-15	stiff
	16-30	very stiff
	31-50	hard
	Over 50	very hard

Drilling Procedures

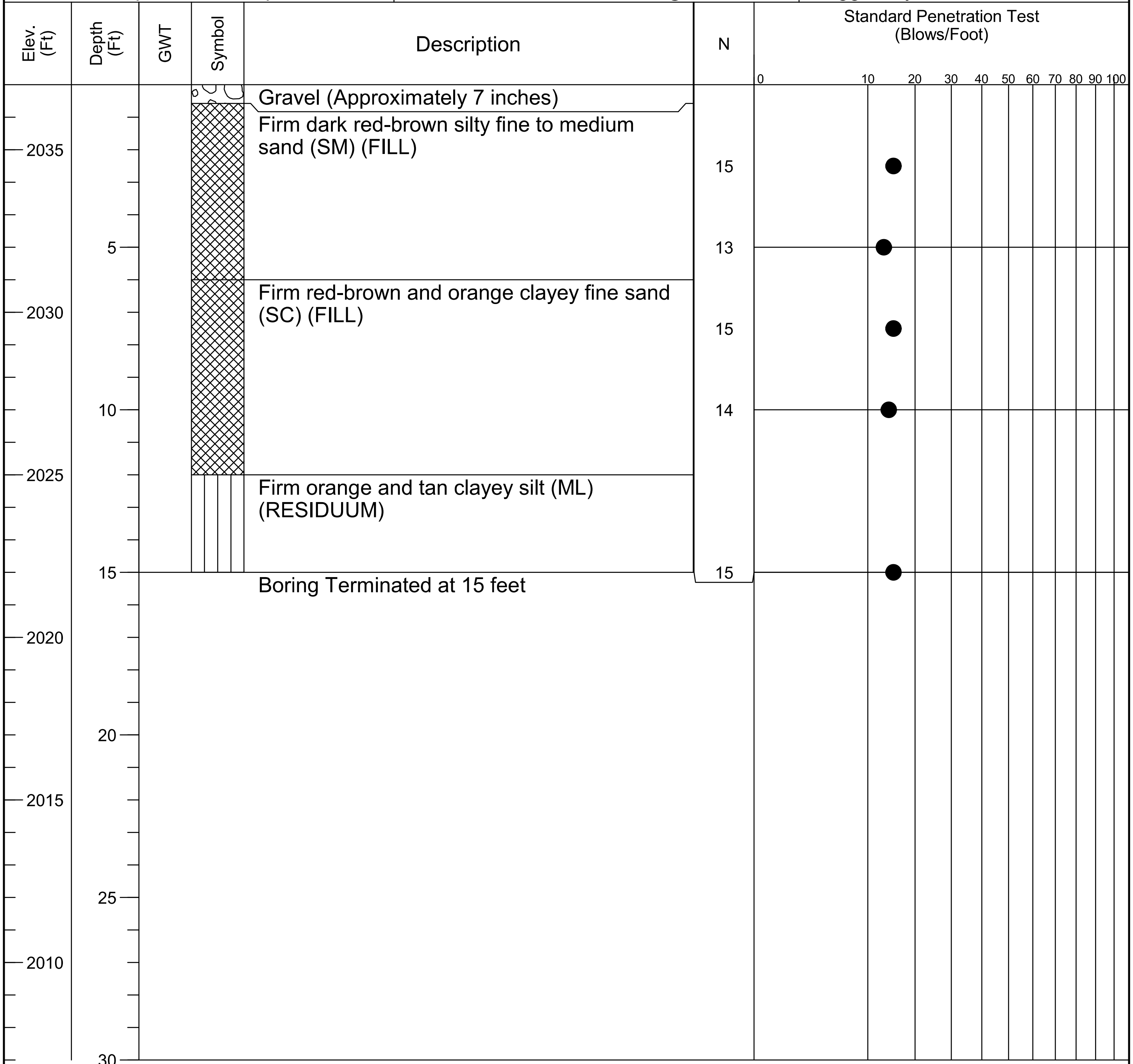
Soil sampling and standard penetration testing performed in accordance with ASTM D 1586. The standard penetration resistance is the number of blows of a 140-pound hammer falling 30 inches to drive a 2-inch O.D., 1.4-inch I.D. split-spoon sampler one foot. Rock coring is performed in accordance with ASTM D 2113. Thin-walled tube sampling is performed in accordance with ASTM D 1587.

U-1

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Union County 911 Center**		Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**		Date: **10/10/24**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **2037**
Driller: **GCD (Auto-Hammer)**	GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**



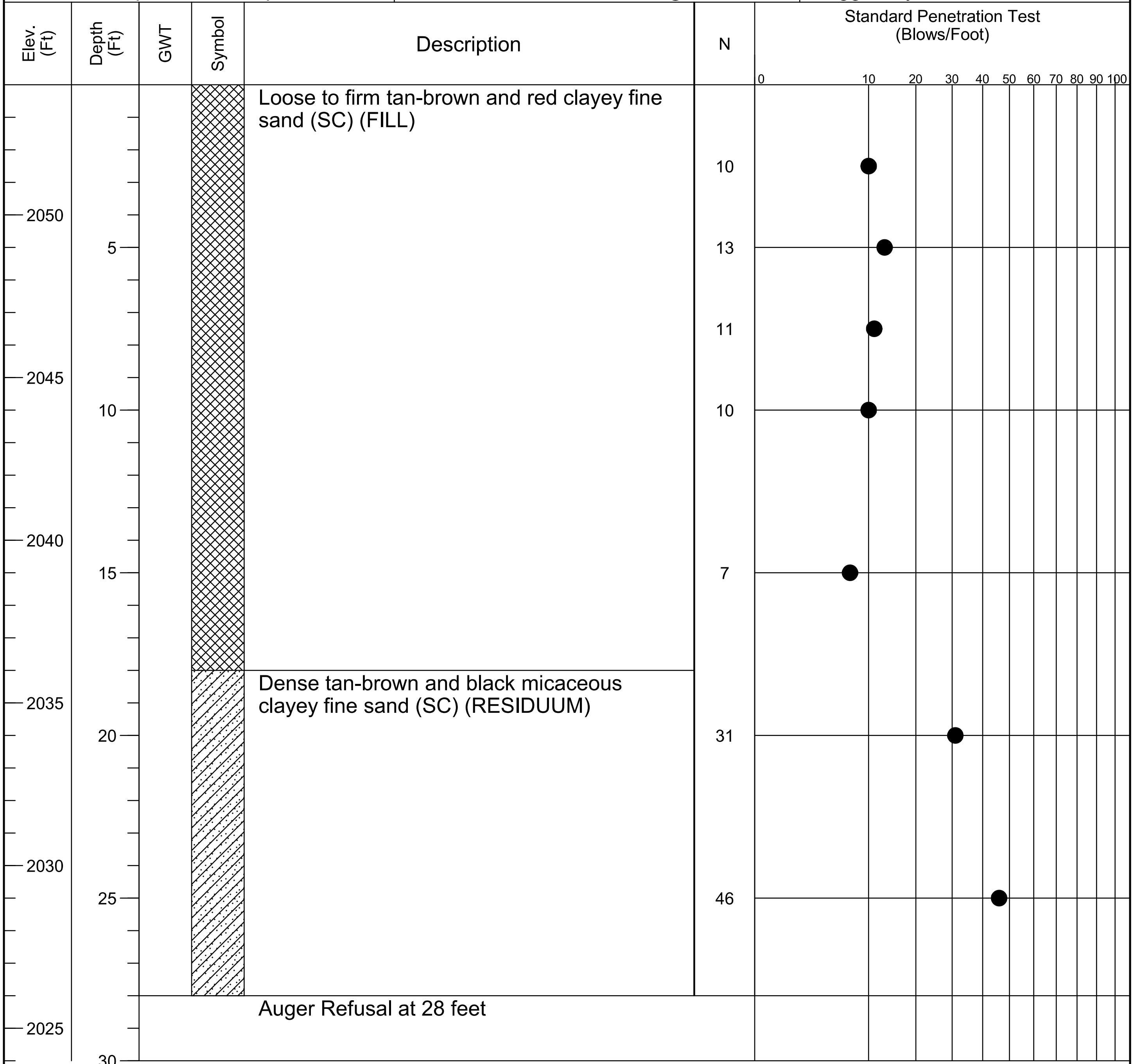
Description	N
Gravel (Approximately 7 inches)	
Firm dark red-brown silty fine to medium sand (SM) (FILL)	15 13
Firm red-brown and orange clayey fine sand (SC) (FILL)	15 14
Firm orange and tan clayey silt (ML) (RESIDUUM)	15
Boring Terminated at 15 feet	

Remarks:

U-2

Test Boring Record

Project: **Union County 911 Center**		Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**		Date: **10/10/24**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **2054**
Driller: **GCD (Auto-Hammer)**	GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**



Loose to firm tan-brown and red clayey fine sand (SC) (FILL)

Dense tan-brown and black micaceous clayey fine sand (SC) (RESIDUUM)

Auger Refusal at 28 feet

Remarks:

U-3

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Union County 911 Center**

Project No: **242482.20**

Location: **Shoe Factory Road, Blairsville, Georgia**

Date: **10/10/24**

Method: **HSA- ASTM D1586**

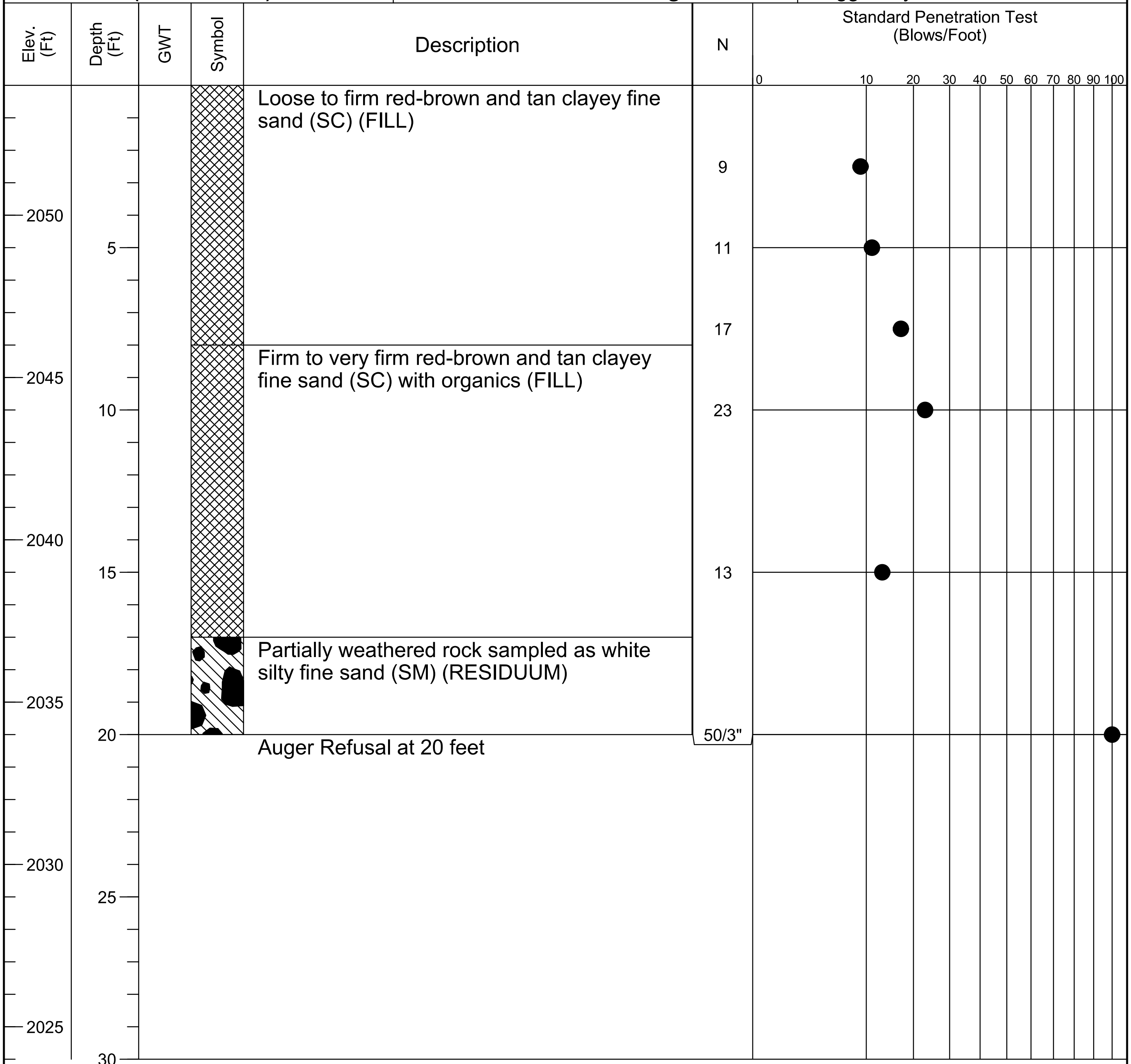
GWT at Drilling: **Not Encountered**

G.S. Elev: **2054**

Driller: **GCD (Auto-Hammer)**

GWT at 24 hrs: **N/A: Boring Backfilled**

Logged By: **BGS**



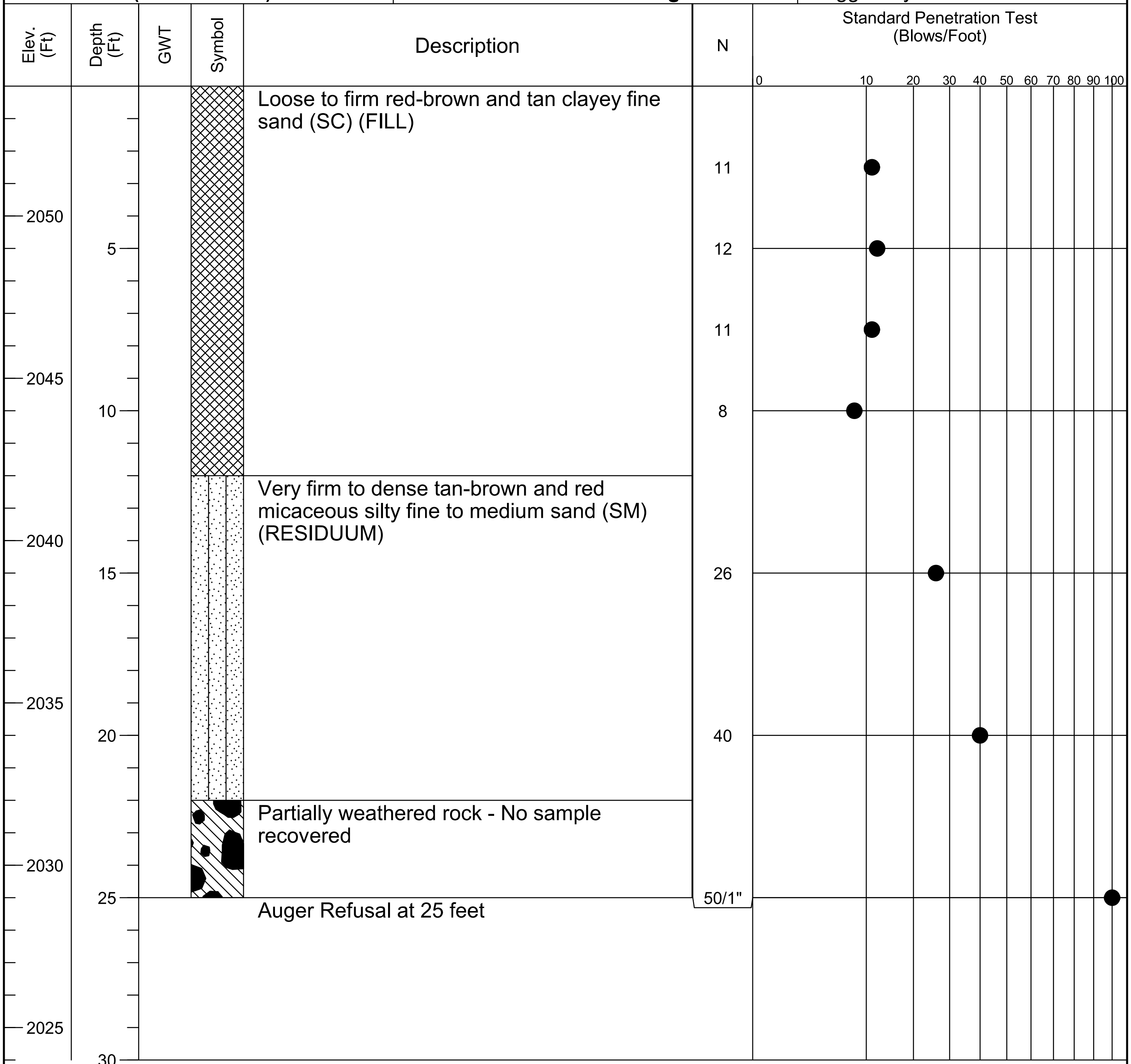
Description	N
Loose to firm red-brown and tan clayey fine sand (SC) (FILL)	9 11 17
Firm to very firm red-brown and tan clayey fine sand (SC) with organics (FILL)	23 13
Partially weathered rock sampled as white silty fine sand (SM) (RESIDUUM)	50/3"
Auger Refusal at 20 feet	

Remarks:

U-4

Test Boring Record

Project: **Union County 911 Center**		Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**		Date: **10/10/24**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **2054**
Driller: **GCD (Auto-Hammer)**	GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**



Description	N
Loose to firm red-brown and tan clayey fine sand (SC) (FILL)	11 12 11 8
Very firm to dense tan-brown and red micaceous silty fine to medium sand (SM) (RESIDUUM)	26 40
Partially weathered rock - No sample recovered	50/1"
Auger Refusal at 25 feet	

Remarks:

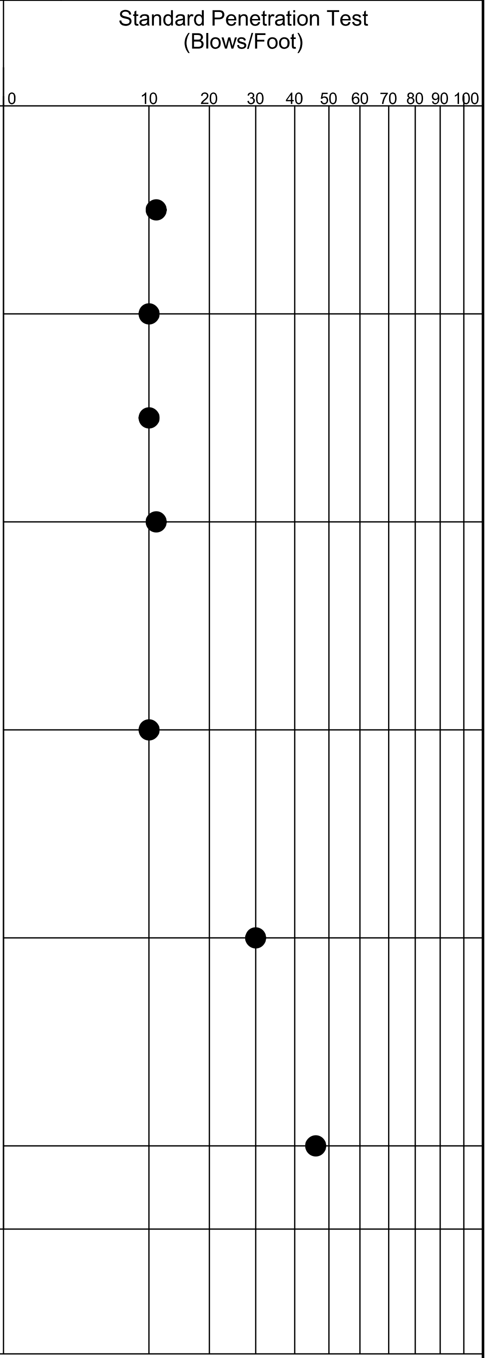
U-5

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Union County 911 Center**	Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**	Date: **10/10/24**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev: **2054**
Driller: **GCD (Auto-Hammer)** / GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
			▨	Loose to firm red-brown and tan clayey fine sand (SC) (FILL)	11
2050	5		▨		10
			▨		10
2045	10		▨	Loose to firm tan-brown and red silty fine sand (SM) (FILL)	11
2040	15		▨		10
2035	20		⁙	Very firm to dense tan-brown and black micaceous silty fine sand (SM) (RESIDUUM)	30
2030	25		⁙		46
2025	30			Auger Refusal at 27 feet	



Remarks:

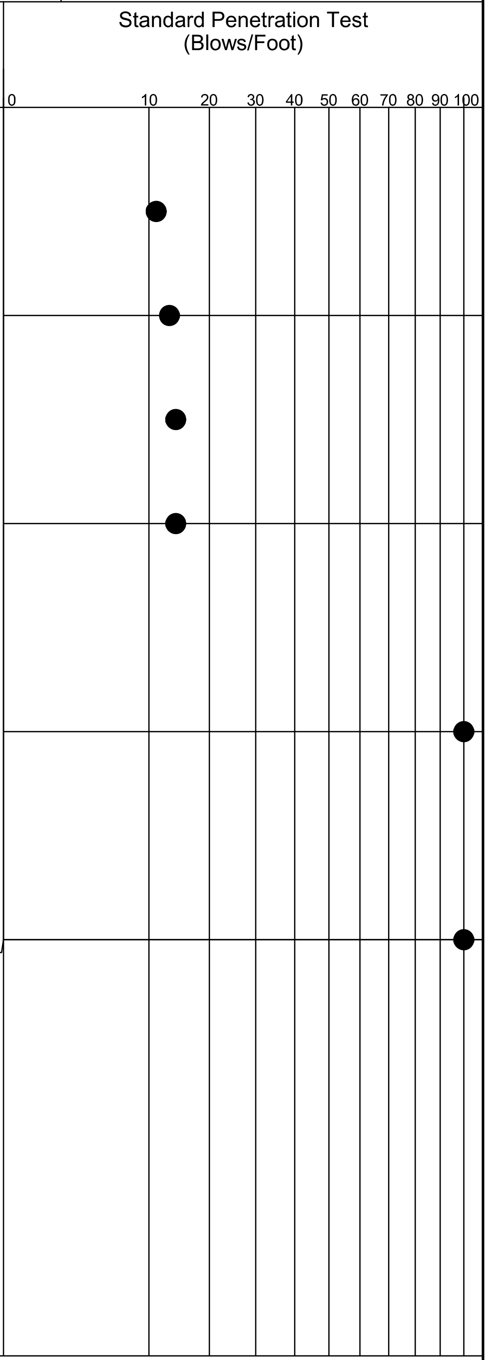
U-6

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Union County 911 Center**	Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**	Date: **10/10/24**
Method: **HSA- ASTM D1586** GWT at Drilling: **Not Encountered**	G.S. Elev: **2054**
Driller: **GCD (Auto-Hammer)** GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
			▩	Firm tan-brown to red-brown clayey fine sand (SC) (FILL)	
					11
2050					
	5				13
					14
2045					
	10				14
			▧	Partially weathered rock sampled as tan, red, and white silty fine sand (SM) (RESIDUUM)	
2040					
	15				50/3"
				No Sample Recovery at 20 Feet	
2035					
	20			Auger Refusal at 20 feet	50/1"
2030					
	25				
2025					
	30				



Remarks:

U-7

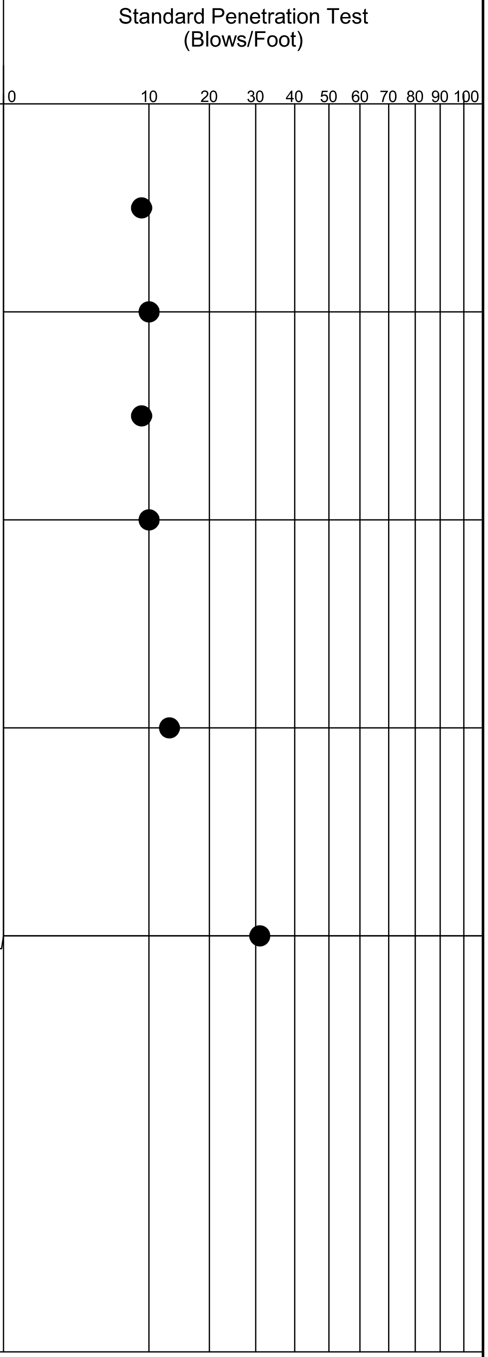
Test Boring Record

GEOHYDRO ENGINEERS

Project: **Union County 911 Center**		Project No: **242482.20**
Location: **Shoe Factory Road, Blairsville, Georgia**		Date: **10/10/24**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **2054**
Driller: **GCD (Auto-Hammer)**	GWT at 24 hrs: **N/A: Boring Backfilled**	Logged By: **BGS**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
			▨	Loose red-brown and tan clayey fine sand (SC) (FILL)	9
	5		▨		10
			▨		9
			▨	Stiff red-brown and tan fine sandy clay (CL) with organics (FILL)	
	10		▨		10
	15		▨		13
			⁘	Dense tan-brown silty fine sand (SM) (RESIDUUM)	
	20		⁘		31
				Boring Terminated at 20 feet	
	25				
	30				

Elev. (Ft): 2050, 2045, 2040, 2035, 2030, 2025



Remarks: