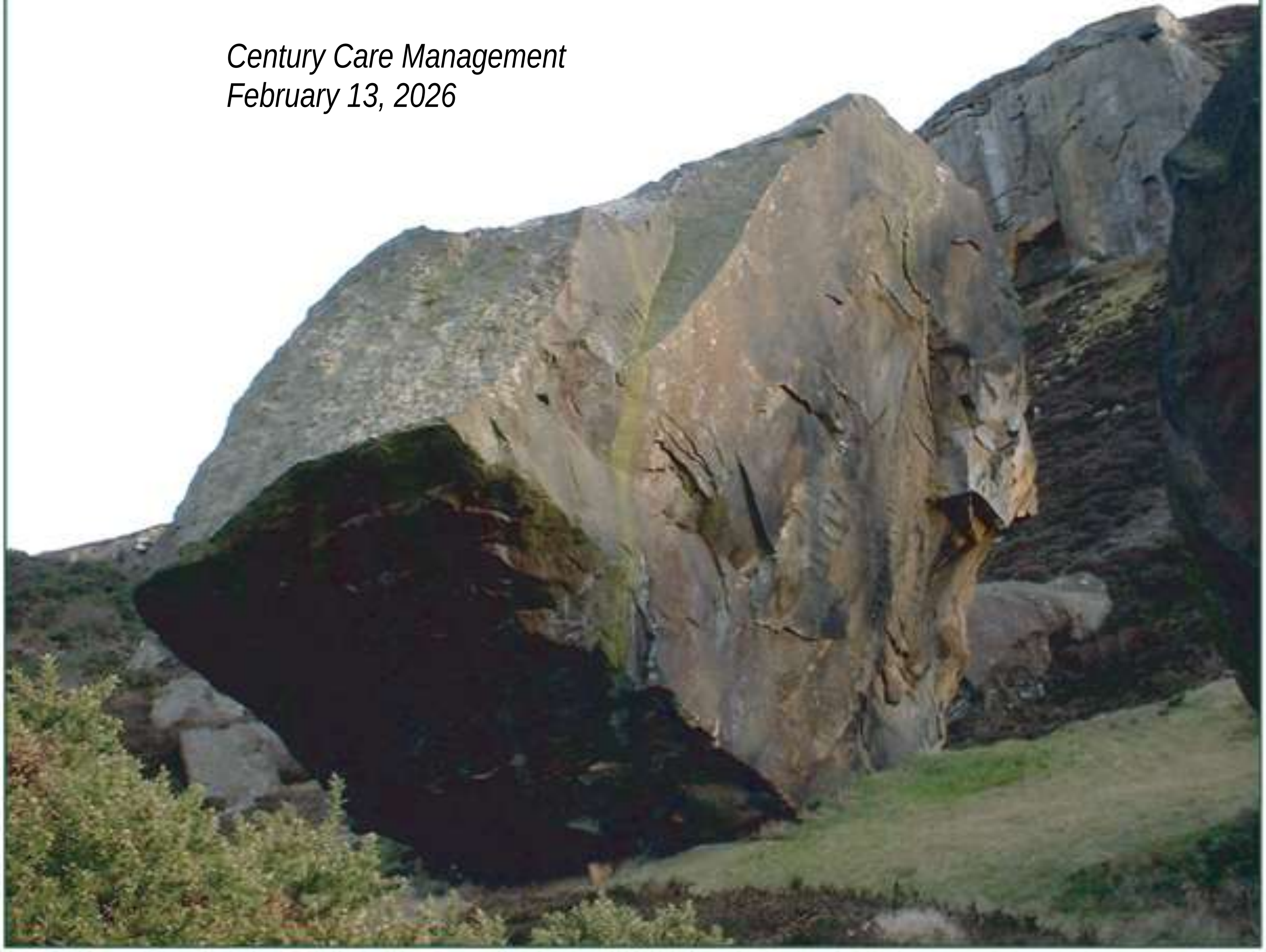
Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Heartland Nursing Home
Colfax, North Carolina
Geo-Hydro Project Number 252953.20

Century Care Management
February 13, 2026



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February 13, 2026

Report of Subsurface Exploration and
Geotechnical Engineering Evaluation
Heartland Nursing Home
Colfax, Guilford County, North Carolina
Geo-Hydro Project Number 252953.20

Dear Mr. Schmidlin:

Geo-Hydro Engineers, Inc. has completed the authorized subsurface exploration for the above referenced project. The scope of services for this project was outlined in our proposal number 252953.P0R1 dated January 8, 2026. Our understanding of the project is based on our correspondence with you, our review of the conceptual site plan provided by you and our experience with similar projects.

PROJECT INFORMATION

Planning is underway for the construction of a new nursing home facility located east of the intersection of Kidd Road and West Market Street in Colfax, Guilford County, North Carolina. Figure 1 in the Appendix shows the approximate site location.

The project site encompasses approximately 21.46 acres and is comprised of Guilford County Parcel ID#s 7806107946 and 7806101797. The planned development will include an approximate 74,982 square-foot, 1-story structure, asphalt access drives and pavements, a borrow pit area, and utilities. Currently, the site is mostly wooded with two open fields on the western and eastern portions of the site and several residential structures with detached garages and/or sheds. The annotated aerial photo below illustrates current site conditions and the site plan excerpt below illustrates the proposed development.

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We expect the nursing home building to have a structural steel frame and masonry walls and a concrete slab-on-grade floor system. Structural loading information for the project was not available at the time of this report. Based on our experience with similar projects, we have assumed that column loads for the new building will not exceed 150 kips, and wall loads will not exceed approximately 5 kips per lineal foot.

If the architect or structural engineer for the project determines that the actual loads are greater than our estimates, please allow us to review the recommendations in this report. Additionally, we should be allowed to review the final site grading plan for finished floor elevation information as well as the relationship of the final building location to our boring locations, and adjust our recommendations as needed.

EXPLORATORY PROCEDURES

The subsurface exploration consisted of 19 machine-drilled soil test borings performed at the approximate locations shown on Figures 2 and 3 included in the Appendix. The test borings were located in the field by Geo-Hydro using a hand-held GPS unit with preloaded coordinates. In general, the locations of the test borings 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.

REGIONAL GEOLOGY

The project site is located in the Charlotte and Milton Belts region of the Piedmont Geologic Province of North Carolina. 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 that 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 soil eventually becomes 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 that the crystalline rock has undergone. The degree of weathering is most advanced at the ground surface, where fine-grained soil may be present. Conversely, 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, all borings encountered about 2 to 3 inches of topsoil. Topsoil thickness at the site should be expected to vary. Many of the borings were performed along freshly cut access trails from which most of the topsoil was removed during the clearing process. On wooded or overgrown sites, it is not unusual for the grading contractor to report an average topsoil thickness of 10 to 12 inches following the intermixing of topsoil, leaves, and branches during tree removal. Topsoil thicknesses will be greater in or near low-lying areas and drainage features. For planning purposes, we suggest a topsoil thickness of 10 inches for wooded areas and four inches for cleared areas of the site.

Beneath the surface materials, all borings encountered residual soils typical of the Piedmont Region. The residual soils were generally classified as sandy clay, silty sand, and sandy silt with varying amounts of rock fragments. Standard penetration resistances recorded in the residual soils ranged from 7 to 46 blows per foot.

Partially weathered rock was encountered in boring B-8 at a depth of about 17 feet. Partially weathered rock is locally defined as residual material having a standard penetration test resistance value greater than 100 blows per foot.

At the time of drilling, groundwater was not encountered in the test borings. Except for borings SW-1 and SW-2, the borings were backfilled with soil cuttings upon completion of drilling operations. Twenty-four hours after drilling, groundwater was not encountered in borings SW-1 and SW-2. The boreholes were backfilled with soil cuttings after the final groundwater level check. It should 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 conditions, please refer to the test boring records included in the Appendix.

Summary of Subsurface Conditions

Boring	Ground Elevation	Bottom of Fill Materials		Top of PWR		Auger Refusal		Groundwater Level at Time of Drilling		Groundwater Level 24 Hours After Drilling	
		Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation
B-1	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-2	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-3	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-4	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-5	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-6	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-7	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-8	--	NE	--	17	--	NE	--	NE	--	N/A	--
B-9	--	NE	--	NE	--	NE	--	NE	--	N/A	--
B-10	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-1	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-2	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-3	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-4	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-5	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-6	--	NE	--	NE	--	NE	--	NE	--	N/A	--
P-7	--	NE	--	NE	--	NE	--	NE	--	N/A	--
SW-1	--	NE	--	NE	--	NE	--	NE	--	NE	--
SW-2	--	NE	--	NE	--	NE	--	NE	--	NE	--

All Depths and Elevations in this Summary Table are Approximate

NE: Not Encountered

N/A: Not Applicable

PWR: Partially Weathered Rock

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 may be encountered during supplemental explorations or during construction that are substantially different from those indicated by the test borings.

Geotechnical Considerations

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

- Partially weathered rock was encountered in boring B-8 at a depth of about 17 feet. Excavation of partially weathered rock typically requires large equipment capable of ripping. If smaller equipment is used for the project, the use of impact hammers will be necessary to remove partially weathered rock where encountered. The partially weathered rock encountered in the borings should be considered when developing final grading and utility plans. It is important to note that the depth to rock or partially weathered rock may vary drastically over relatively short distances. It would not be unusual to encounter partially weathered rock, rock lenses, rock pinnacles, or boulders between or around the test borings.

- The test borings indicate generally favorable excavation conditions within the anticipated excavation limits. In general, the residual soils should be readily removable using conventional soil excavation equipment such as loaders and backhoes.

- Borings SW-1 through SW-2 were performed in the area of the planned stormwater management pond. Twenty-four hours after drilling completion, no groundwater was encountered in the borings. Redoximorphic features were observed in borings SW-1 and SW-2 at depths of about 14 and 11.5 feet, respectively.

- It is our opinion that the test borings performed for the project were not extended to a depth sufficient to characterize the upper 100 feet of the soil subsurface for the purposes of determining the *Site Class* in accordance with the 2018 North Carolina Building Code (2015 International Building Code – Chapter 20, ASCE 7-10). We recommend using a default *Site Class* of *D* as suggested in the code. The mapped and design spectral response accelerations are as follow: S_S=0.173, S_1=0.083, S_{DS}=0.184, S_{D1}=0.132.

- Contingent upon proper site preparation and thorough evaluation of the foundation excavations, it is our opinion that the proposed structures can be supported using conventional shallow foundations and concrete slab-on-grade floors. For design purposes, we recommend using an allowable soil bearing pressure of 3,000 psf or less. Once final grading plans and structural loading information for the buildings have been developed, please allow us to review that information.

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

Existing Fill Materials

Fill materials were not encountered in the borings; however, it is possible that fill materials are present in other localized areas throughout the planned construction footprint. Any loose or unstable fill material that cannot be adequately densified in place should be removed and replaced with well-compacted structural fill.

There are several important facts that should be considered regarding existing fill materials and the limitations of subsurface exploration.

- 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 pavement subgrades and foundation and floor slab bearing surfaces, an organic layer might adversely affect support of pavements, 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

Landscaping, trees, roots, topsoil, demolition debris, and other deleterious materials should be removed from the proposed construction area. All existing utilities should be excavated and removed unless they are to be incorporated into the new construction. 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 subgrade degradation. All excavations resulting from rerouting of underground utilities and from demolition of below grade structures such as foundations should be backfilled in accordance with the *Structural Fill* section of this report.

We recommend 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. 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 10 percent of the aggregate building and pavement areas will require undercutting and recompaction or replacement extending to an average depth of 2 feet. *The suggested site preparation approach is intended only as a tool to estimate a cost associated with managing unstable subgrade conditions. Ground stabilization may be accomplished by in-place densification, treatment with geosynthetics (grid or fabric) and crushed stone, or a combination of methods. The need for, extent of, location, and optimal method of stabilization should be determined by Geo-Hydro at the time of construction based on actual site conditions.*

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.

Seasonal High Water Table (SHWT)

Twenty-four hours after drilling, groundwater was not encountered in boring SW-1 and SW-2. We consider the presence of redoximorphic features in the soil samples to be representative of the SHWT. The following summarizes our estimation of the SHWT based on redoximorphic features:

SW-1 SHWT: 14 feet below ground surface. Redoximorphic features observed.
SW-1 Planned Bottom of Pond: To be determined.

SW-2 SHWT: 11½ feet below ground surface. Redoximorphic features observed.
SW-2 Planned Bottom of Pond: To be determined.

Excavation Characteristics

Partially weathered rock was encountered in boring B-8 at a depth of about 17 feet. It is important to note that the depth to rock or partially weathered rock can vary drastically over relatively short distances. It would not be unusual for rock or partially weathered rock to occur at higher elevations at unexpected locations between or around the test borings.

Backhoes capable of ripping will likely be required to effectively achieve excavation in partially weathered rock materials. In some instances, partially weathered rock having very high standard penetration resistances (50/0" to 50/2") may require the use of hydraulic impact hammers or blasting to achieve excavation. Materials below the depth of auger refusal will require blasting to achieve excavation.

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. 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 test borings and our observations, residual soils at the site appear suitable for reuse as structural fill. Geo-Hydro should observe the excavation of existing materials to evaluate their suitability for reuse. Routine adjustment of moisture content will be necessary to allow proper placement and compaction of excavated soils. Highly organic soils and debris-laden soils will not be suitable for reuse.

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.

Partially weathered rock materials will be suitable for reuse as structural fill only if they break down into a reasonably well-graded material that can be satisfactorily compacted. The presence of cobble size or boulder size material, which does not break down under the action of compaction equipment, will limit the suitability of partially weathered rock materials. Engineering judgment will be required in the field to evaluate the acceptability of partially weathered rock materials for reuse as structural fill.

For planning purposes, we recommend assuming that blast rock will not be suitable for reuse as structural fill.

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 Piedmont region: (CL), (SM), and (ML). The following soil types are typically suitable but are not abundant in the Piedmont region: (SW), (SP), (SC), (SP-SM), and (SP-SC). The following soil types are considered unsuitable: (OL), (OH), and (Pt).

If encountered, elastic silt (MH) or fat clay (CH) soils should be used with extreme caution. Such soils will require protection against desiccation or inundation during the construction process. Soils which have a liquid limit greater than 60 and a plasticity index greater than 35 will require blending with less plastic materials to result in lower Atterberg limits prior to use as structural fill. Another approach to allow the reuse of high plasticity soils is to place such soils in deeper structural fill sections, maintaining a minimum 3-foot separation/buffer between the high plasticity soils and all structural elements and pavements.

Laboratory Proctor compaction tests and classification tests should be performed on representative samples obtained from the proposed borrow material to provide data necessary to determine acceptability and for quality control. The moisture content of suitable borrow soils should generally be no more than 3 percentage points below or above optimum 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 a maximum loose-lift thickness of 8 inches is generally recommended. The soil should be compacted by a self-propelled sheepsfoot roller. Within small excavations such as in utility trenches, around manholes, above foundations, or behind retaining walls, we recommend the use of “wacker packers” or “trench roller” 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 24 inches of floor slab subgrade soils should be compacted to at least 98 percent of the standard Proctor maximum dry density. Additionally, the maximum dry density of structural fill should be no less than 90 pcf. Following North Carolina DOT guidelines, the upper 8 inches of pavement subgrade soils should be compacted to at least 100 percent of the maximum dry density per AASHTO T99. Geo-Hydro should perform density tests during fill placement.

Earth Slopes

Temporary construction slopes should be designed in strict compliance with current OSHA regulations. The exploratory borings indicate that within the likely excavation depths for this project, soil type B as defined in 29 CFR 1926 Subpart P will be encountered. Based on OSHA rules, temporary slopes for materials that may be encountered during grading or during installation of underground utilities are as follows:

Material Classification	Maximum Slope Gradient for Excavations less than 20 feet deep
Type A - Partially Weathered Rock	¾H:1V
Type B - Residual Soil	1H:1V
Type C - Fill Materials, or Any Soil Type Below the Groundwater Level	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 North Carolina 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, light truck traffic, or parking. Erosion protection of slopes during construction and during establishment of vegetation should be considered an essential part of construction.

Earth Pressure (Cast-in-Place Structures)

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 as basement or foundation 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 would place an unreasonable reduction in the calculation of the frictional resistance.

The force which 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 proposed structures can be supported using conventional shallow foundations. We recommend that footings be designed for an allowable soil bearing pressure of 3,000 psf or less. In addition, we recommend a minimum width of 24 inches for column footings and 18 inches for continuous wall footings or turndowns 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 maximum column loads of 150 kips and wall loads no greater than 5 kips per lineal foot for the building. The recommendations above are based on a 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 determines that the maximum anticipated loads exceed the values of those provided, or if it is determined 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. These evaluations should be performed by Geo-Hydro 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 augers and portable dynamic cone penetrometer testing (ASTM STP-399).

Because of natural variation, it is possible that some of the soils at the project site may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations.

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 aggregate base course (ABC) meeting North Carolina 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 budgetary purposes, we suggest considering that as much as 10 percent of the foundation excavations will require overexcavation and stone replacement extending to a depth of 2 feet below bearing elevation. The actual quantity of stone replacement will be different and may exceed the provided estimate.

Seismic Design

It is our opinion that the test borings performed for the project were not extended to a depth sufficient to characterize the upper 100 feet of the soil subsurface for the purposes of determining the *Site Class* in accordance with the 2018 North Carolina Building Code (2015 International Building Code – Chapter 20, ASCE 7-10). We recommend using a default *Site Class* of *D* as suggested in the code. The mapped and design spectral response accelerations are as follow: S_S=0.173, S_1=0.083, S_{DS}=0.184, S_{D1}=0.132.

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.

Moisture Control for Concrete Slabs

To prevent the capillary rise of groundwater from adversely affecting the concrete slab-on-grade floor, we recommend that slab-on-grade floors be underlain by a minimum 4-inch thickness of open-graded stone. Use of #57 crushed stone meeting North Carolina DOT specifications for gradation is suggested. The crushed stone course must be covered by a vapor retarder. We suggest polyethylene sheeting at least 10 mils thick as a minimum vapor retarder.

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:

Heavy-Duty Traffic Areas

Material	Thickness (inches)
Asphaltic Concrete S9.5B Superpave	1.5
Asphaltic Concrete I19B Superpave	2.5
Aggregate Base Course (ABC)	8
Subgrade compacted to at least 100% of AASHTO T99	8

Automobile Parking and Automobile Traffic Only

Material	Thickness (inches)
Asphaltic Concrete S9.5B Superpave	2
Aggregate Base Course (ABC)	6
Subgrade compacted to at least 100% of AASHTO T99	8

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

The top 8 inches of pavement subgrade soils should be compacted to at least 100 percent of the maximum dry density as determined by AASHTO T99. 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. ABC should be compacted to at least 100 percent of the maximum dry density determined by AASHTO T180.

All pavement construction should be performed in general accordance with North Carolina DOT specifications. Proper subgrade compaction, adherence to North Carolina 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. A concrete thickness of 7 inches is recommended for the approach and collection zone in front of the dumpster, in service areas, and in any designated truck turn-around areas.

A 600-psi flexural strength concrete mix (approximate compressive strength of 4,000 psi) with 4 to 6 percent air entrainment should be used. The concrete pavement must be underlain by no less than 5 inches of compacted aggregate base course (ABC). ABC should be compacted to at least 100 percent of the maximum dry density as determined by AASHTO T180. The top 8 inches of soil subgrade should be compacted to at least 100 percent of the maximum dry density as determined by AASHTO T99.

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 North Carolina DOT specifications. Proper subgrade compaction, adherence to North Carolina 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 surfaces immediately prior to placement of aggregate base course (ABC). This proofrolling should be performed the same day ABC is installed.
- Proofrolling of ABC, density testing of ABC and verification of ABC thickness. In-pace density should be verified using the sand cone method (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.
NC Registered Engineering Firm C-3649

Jonathan M. Krawchuk
Staff Professional
jkrawchuk@geohydro.com

2-13-26
028423
RICKY O. COKER

Rick O. Coker, P.E.
NC Geotechnical Manager
rcokerr@geohydro.com

JMK/ROC /252953.20 Heartland Nursing Home Geo Report 2-13-26

APPENDIX

FIGURES

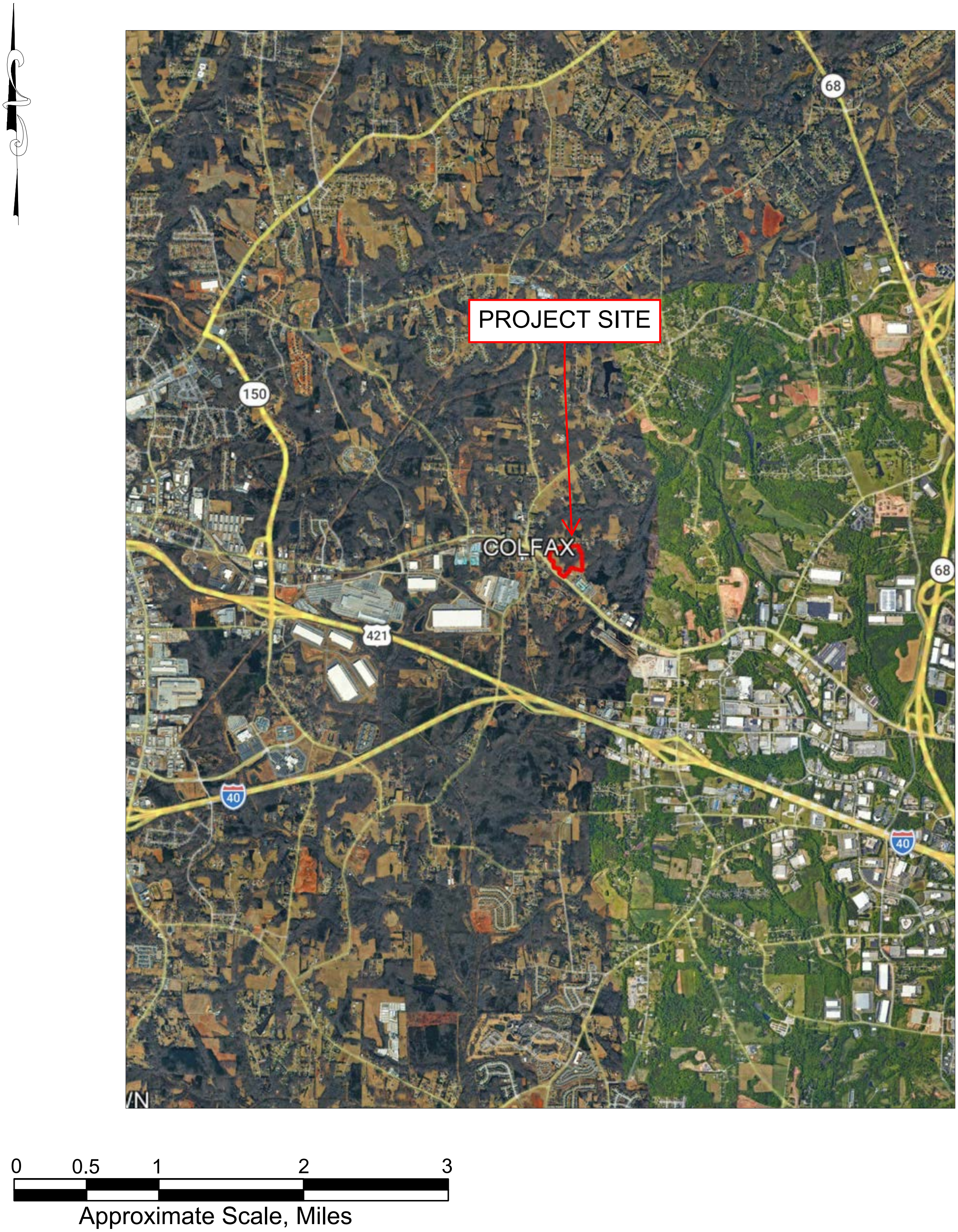
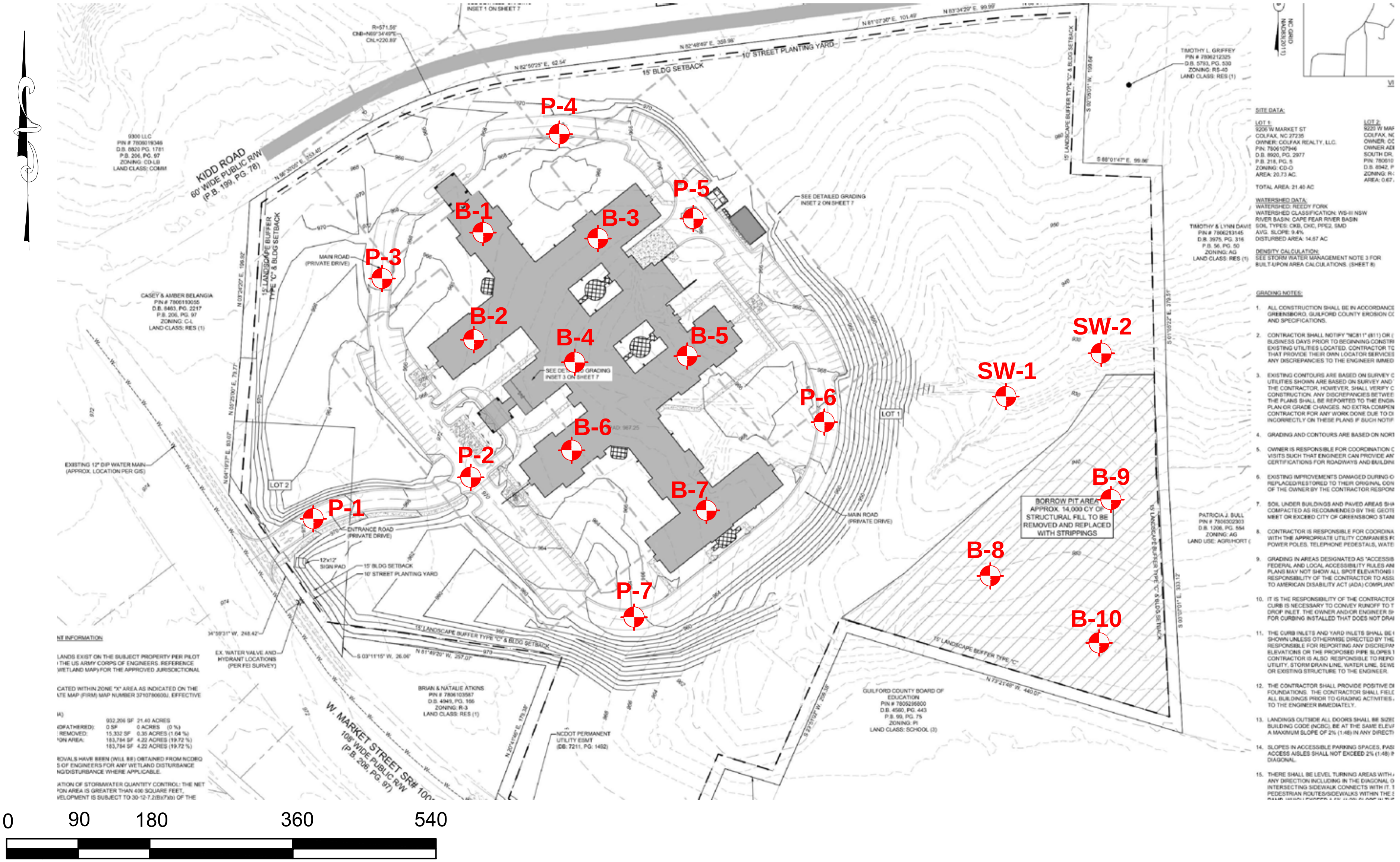


Figure 1: Site Location Plan

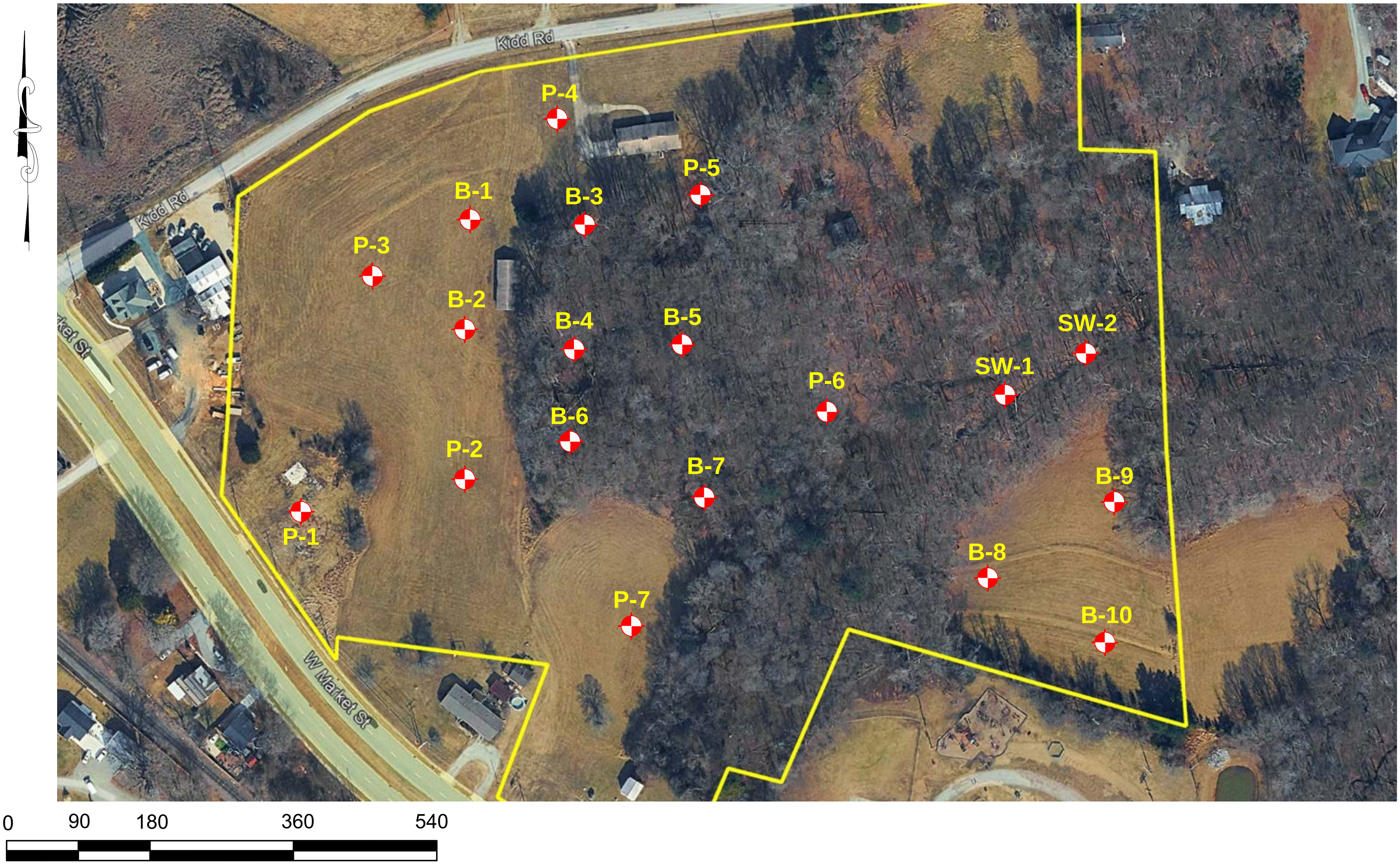
Heartland Nursing Home
Colfax, North Carolina
Geo-Hydro Project Number 252953.20



LEGEND: ⊕ Soil Test Boring

Figure 2: Boring Location Plan

Heartland Nursing Home
Colfax, North Carolina
Geo-Hydro Project Number 252953.20



Approximate Scale: 1"=180'

LEGEND: ⌖ Soil Test Boring

Figure 3: Boring Location Plan

Heartland Nursing Home
Colfax, North Carolina
Geo-Hydro Project Number 252953.20

TEST BORINGS RECORDS

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

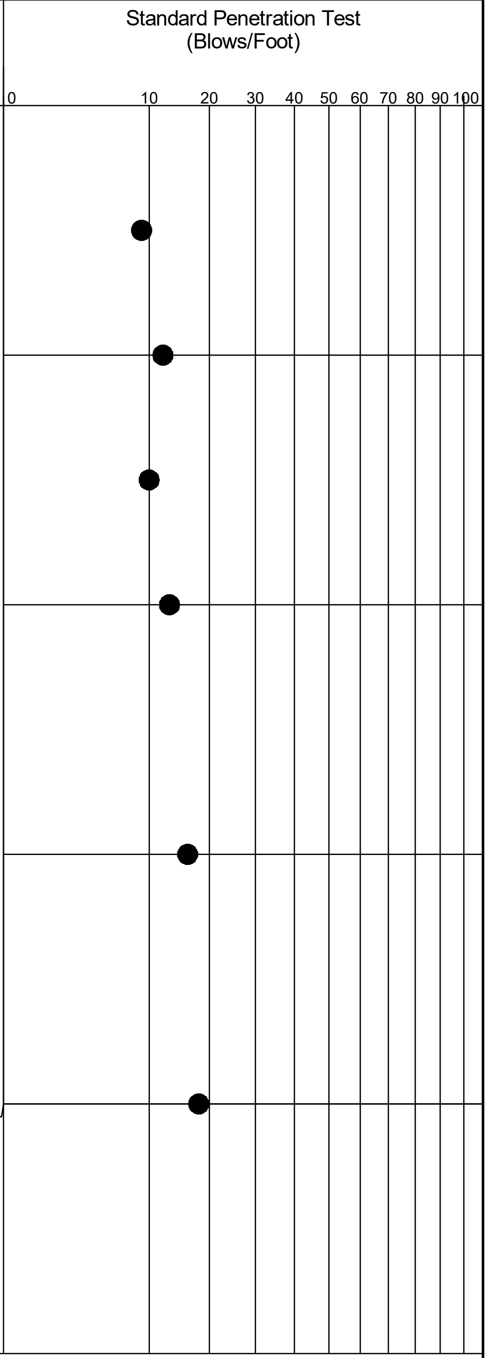
Soil sampling and standard penetration testing performed in accordance with ASTM D 1586. The standard penetration resistance is the number of blows of a 140-pound hammer falling 30 inches to drive a 2-inch O.D., 1.4-inch I.D. split-spoon sampler one foot. Rock coring is performed in accordance with ASTM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

B-1

Test Boring Record

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** — GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** — GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Topsoil (Approximately 2 inches)	
				Stiff brown, red, and tan micaceous fine to medium sandy silt (ML) (RESIDUUM)	
					9
	5				12
					10
	10				13
				Firm tan silty fine to coarse sand (SM)	
	15				16
				Very stiff brown and black micaceous fine sandy silt (ML)	
	20			Boring Terminated at 20 feet	18
	25				



Remarks:

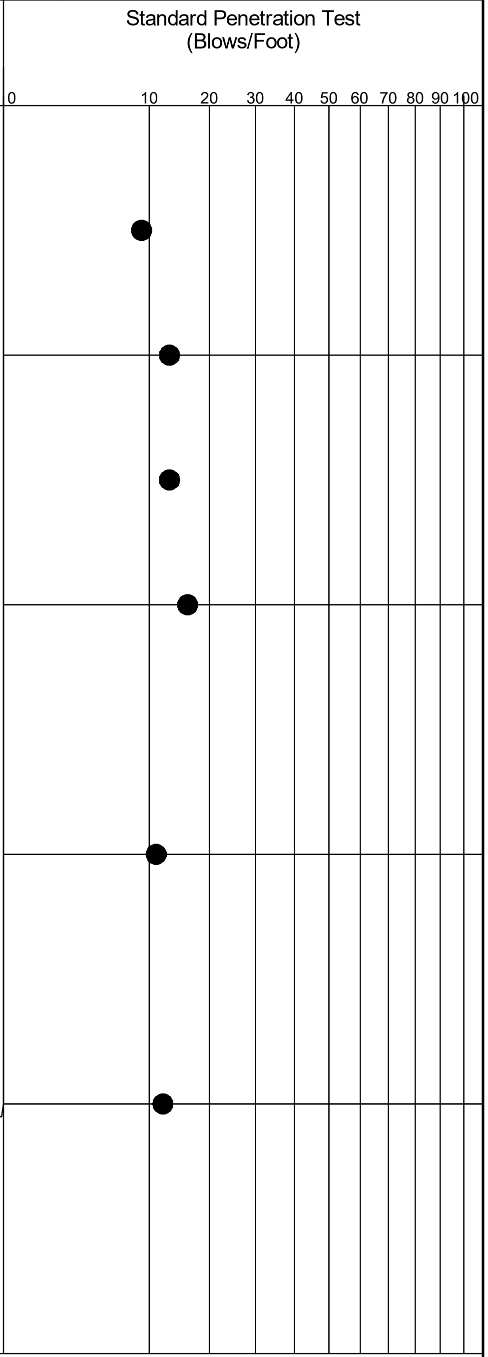
B-2

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 3 inches)	
				Stiff to very stiff brown, red, and tan micaceous fine to coarse sandy silt (ML) (RESIDUUM)	
					9
	5				13
					13
	10				16
	15				11
				Stiff brown, white, and black micaceous fine sandy silt (ML)	
	20			Boring Terminated at 20 feet	12
	25				



Remarks:

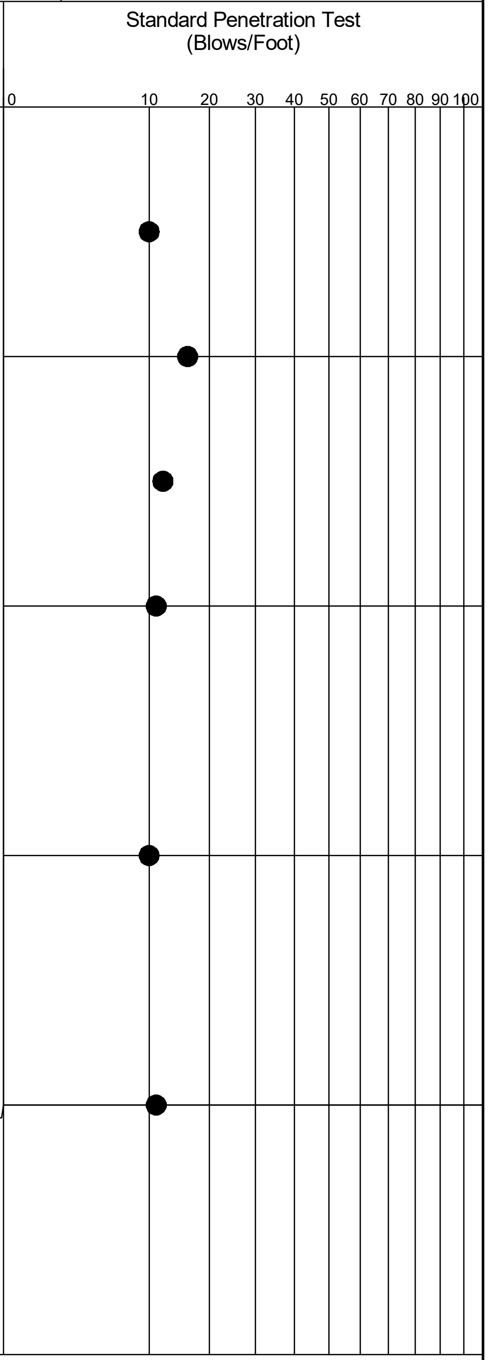
B-3

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **2/4/26**
Method: **HSA- ASTM D1586** GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown and red micaceous fine to medium sandy silt (ML) (RESIDUUM)	10
				Very stiff brown and red micaceous fine to medium sandy clay (CL)	
	5				16
				Stiff brown, tan, and gray micaceous fine sandy silt (ML)	12
	10				11
	15				10
				Stiff gray and white fine to coarse sandy silt (ML)	
	20				11
				Boring Terminated at 20 feet	
	25				



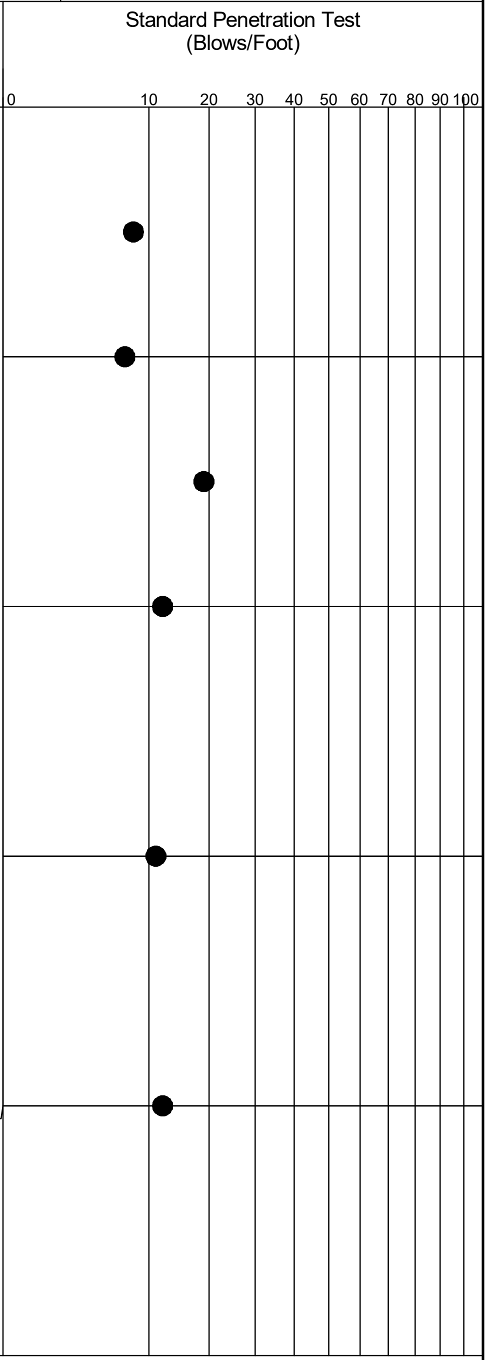
Remarks:

B-4

Test Boring Record

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **2/4/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Firm brown and red micaceous fine to medium sandy silt (ML) with clay seams (RESIDUUM)	8
				Firm brown and red micaceous fine to medium sandy silt (ML)	
	5				7
				Very stiff brown, red, and tan micaceous fine to medium sandy silt (ML)	19
				Stiff brown micaceous fine sandy silt (ML)	
	10				12
				Stiff gray, brown, and white micaceous fine to coarse sandy silt (ML)	
	15				11
	20				12
				Boring Terminated at 20 feet	
	25				



Remarks:

B-5

Test Boring Record

Project: **Heartland Nursing Home**

Project No: **252953.20**

Location: **Colfax, North Carolina**

Date: **2/4/26**

Method: **HSA- ASTM D1586**

GWT at Drilling: **Not Encountered**

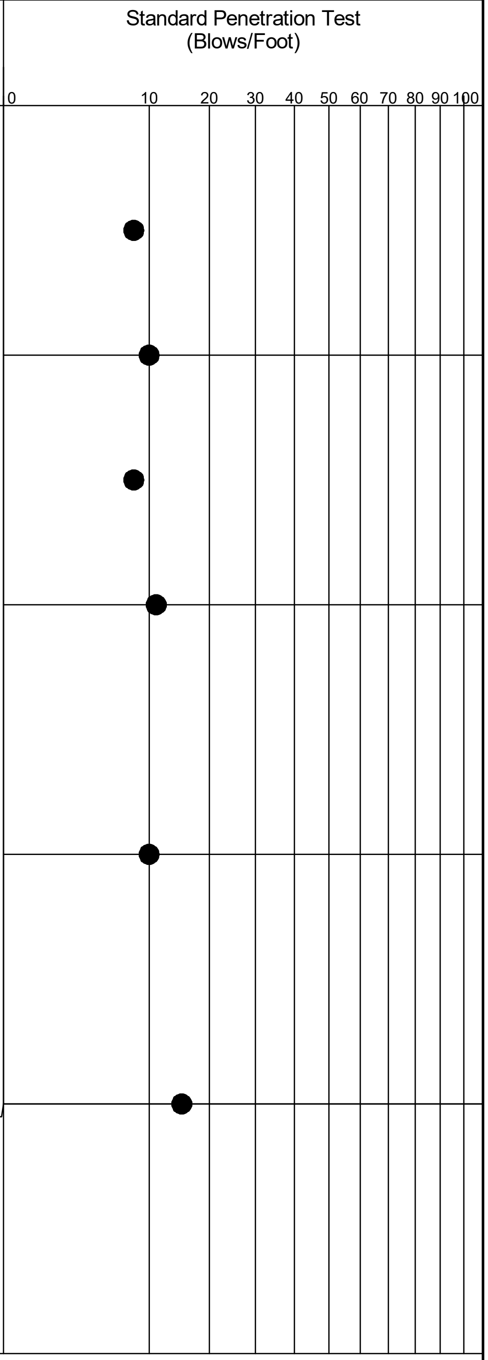
G.S. Elev:

Driller: **Capital Drilling (Auto-Hammer)**

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

Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Firm brown and red micaceous fine to medium sandy silt (ML) (RESIDUUM)	
					8
				Firm to stiff tan, brown, and red micaceous fine to coarse sandy silt (ML)	
	5				10
					8
	10				11
				Stiff brown micaceous fine to medium sandy silt (ML)	
	15				10
				Stiff gray and white micaceous fine to coarse sandy silt (ML)	
	20				15
				Boring Terminated at 20 feet	
	25				



Remarks:

B-6

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**

Project No: **252953.20**

Location: **Colfax, North Carolina**

Date: **2/4/26**

Method: **HSA- ASTM D1586**

GWT at Drilling: **Not Encountered**

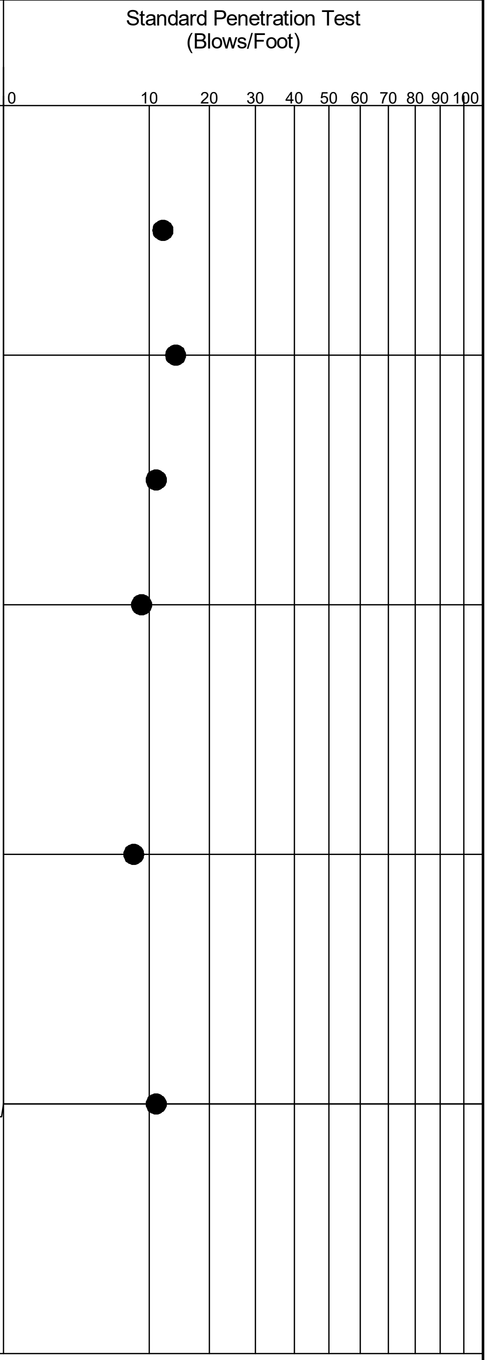
G.S. Elev:

Driller: **Capital Drilling (Auto-Hammer)**

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

Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown and red micaceous fine to medium sandy silt (ML) with clay seams (RESIDUUM)	12
	5			Stiff brown and red micaceous fine to medium sandy silt (ML)	14
				Firm to stiff gray and white micaceous fine to coarse sandy silt (ML)	11
	10				9
	15				8
	20				11
				Boring Terminated at 20 feet	
	25				



Remarks:

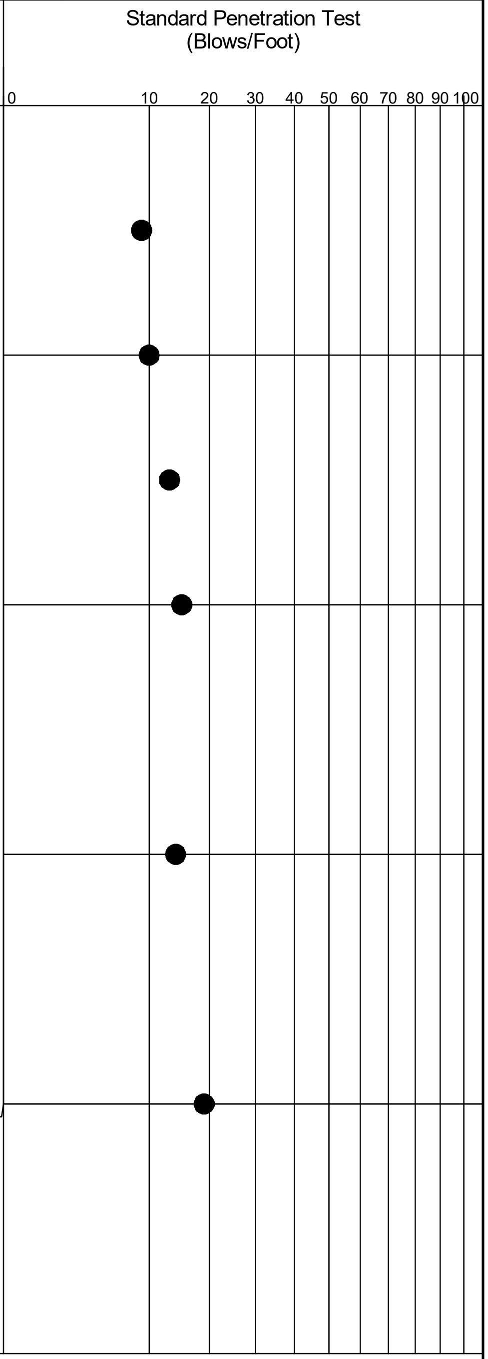
B-7

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown, red, and tan micaceous fine to medium sandy silt (ML) (RESIDUUM)	
					9
	5				10
				Stiff tan and brown fine to coarse sandy silt (ML)	
					13
				Stiff to very stiff gray, brown, and white micaceous fine to coarse sandy silt (ML)	
	10				15
	15				14
	20				19
				Boring Terminated at 20 feet	
	25				



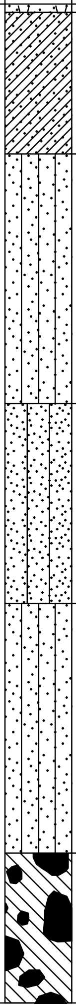
Remarks:

B-8

Test Boring Record

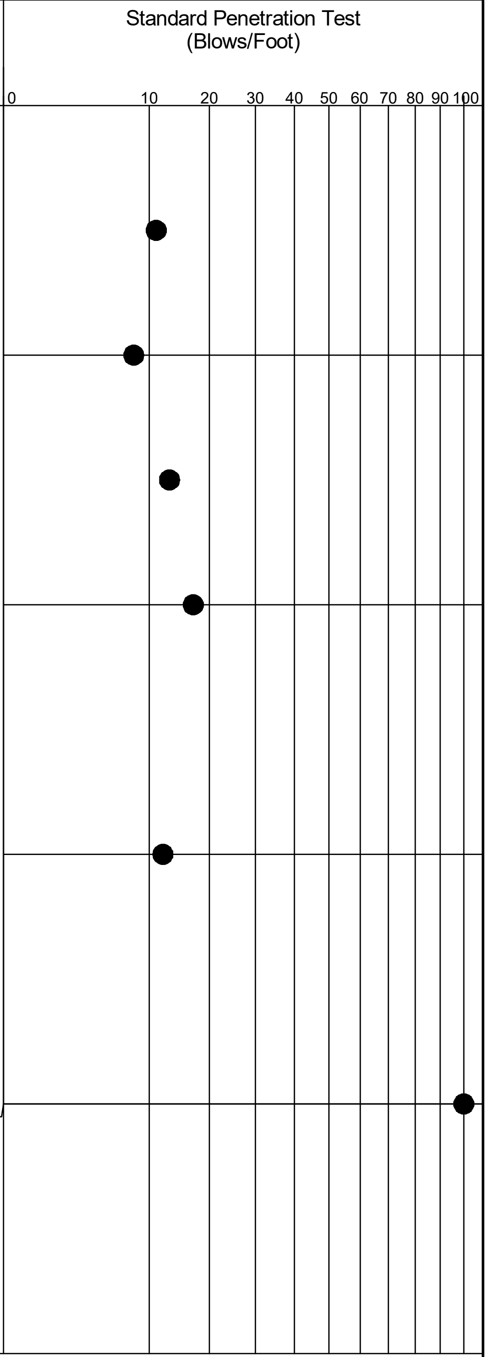
GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**



Elev. (Ft)	Depth (Ft)	GWT	Description	N
			Topsoil (Approximately 2 inches)	
			Stiff brown and red micaceous fine sandy clay (CL) (RESIDUUM)	11
			Firm to stiff brown and red micaceous fine to medium sandy silt (ML)	
	5			8
				13
			Firm black, brown, and red micaceous silty fine to medium sand (SM)	
	10			17
			Stiff brown and red micaceous fine to medium sandy silt (ML)	
	15			12
			Partially weathered rock sampled as tan, black, and red silty fine to medium sand (SM)	
	20		Boring Terminated at 20 feet	50/3"
	25			

Standard Penetration Test (Blows/Foot)



Remarks:

B-9

Test Boring Record

GEOHYDRO ENGINEERS

Project: **Heartland Nursing Home**

Project No: **252953.20**

Location: **Colfax, North Carolina**

Date: **1/28/26**

Method: **HSA- ASTM D1586**

GWT at Drilling: **Not Encountered**

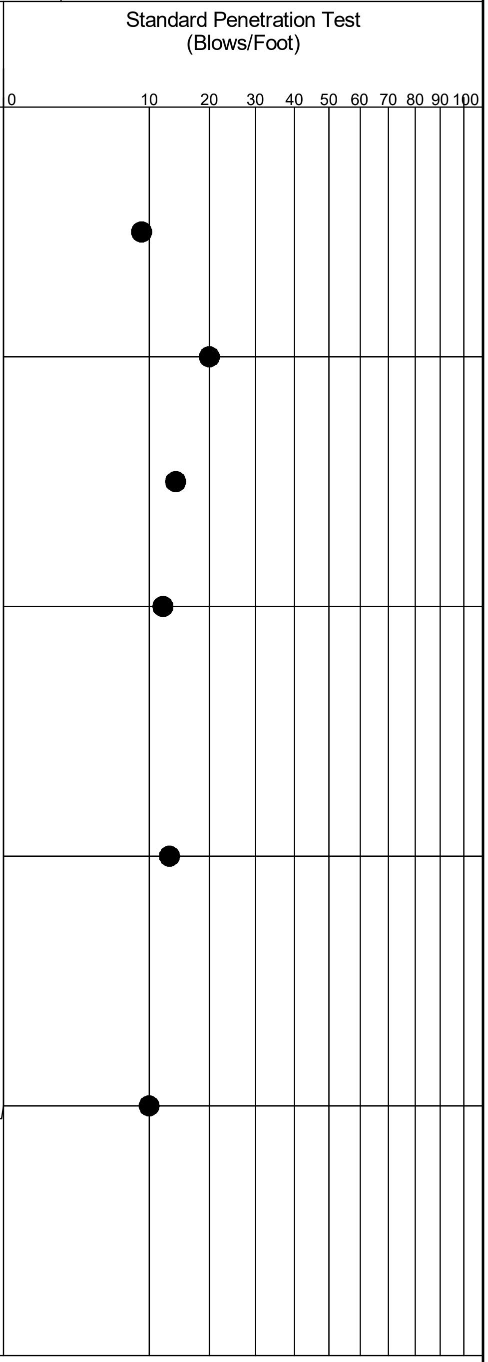
G.S. Elev:

Driller: **Capital Drilling (Auto-Hammer)**

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

Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown, red, and tan micaceous fine to coarse sandy silt (ML) (RESIDUUM)	9
				Stiff to very stiff brown and red micaceous fine sandy silt (ML)	
	5				20
					14
				Stiff brown and red micaceous fine to coarse sandy silt (ML)	
	10				12
	15				13
				Stiff brown and white fine sandy silt (ML)	
	20			Boring Terminated at 20 feet	10
	25				



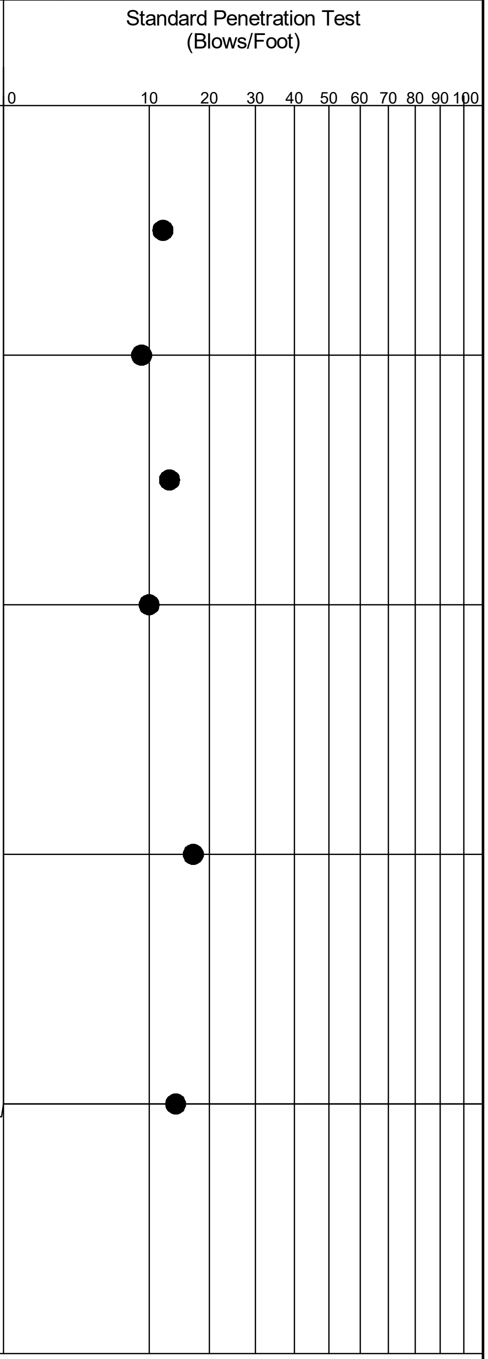
Remarks:

B-10

Test Boring Record

Project: **Heartland Nursing Home**		Project No: **252953.20**
Location: **Colfax, North Carolina**		Date: **1/28/26**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown, red, and tan micaceous fine to coarse sandy silt (ML) with clay seams (RESIDUUM)	12
				Stiff brown, red, and white micaceous fine to medium sandy silt (ML)	
	5				9
					13
	10				10
				Stiff to very stiff tan and brown fine to coarse sandy silt (ML)	
	15				17
	20			Boring Terminated at 20 feet	14
	25				



Remarks:

P-1

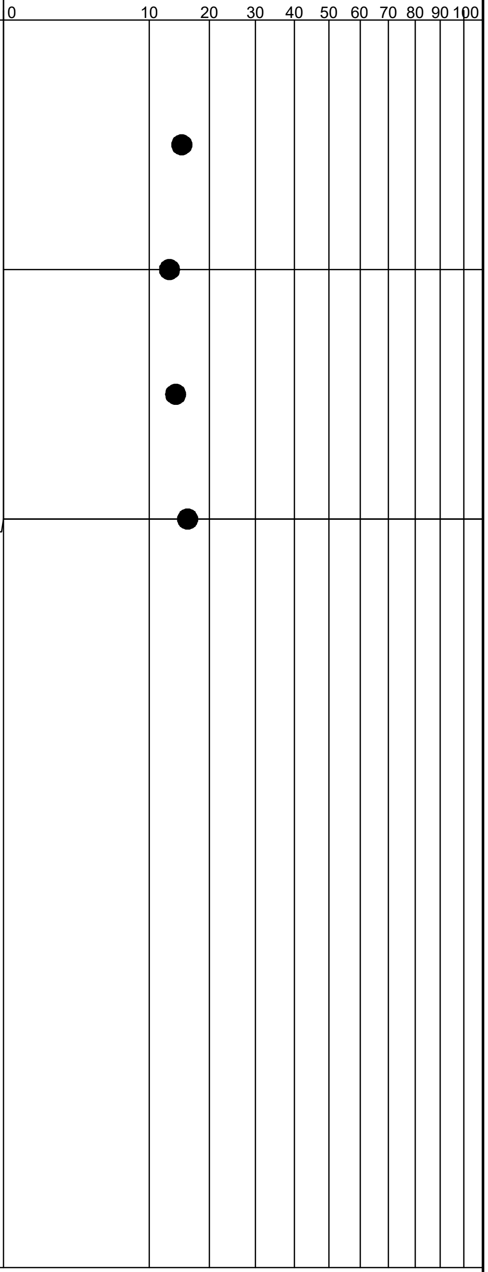
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**		Project No: **252953.20**
Location: **Colfax, North Carolina**		Date: **1/28/26**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 3 inches)	
				Stiff brown and gray micaceous fine to coarse sandy clay (CL) (RESIDUUM)	15
	5			Stiff brown, red, and tan micaceous fine to medium sandy silt (ML)	13 14
	10			Very stiff gray and white micaceous fine to coarse sandy silt (ML)	16
				Boring Terminated at 10 feet	
	15				
	20				
	25				

Standard Penetration Test
(Blows/Foot)



Remarks:

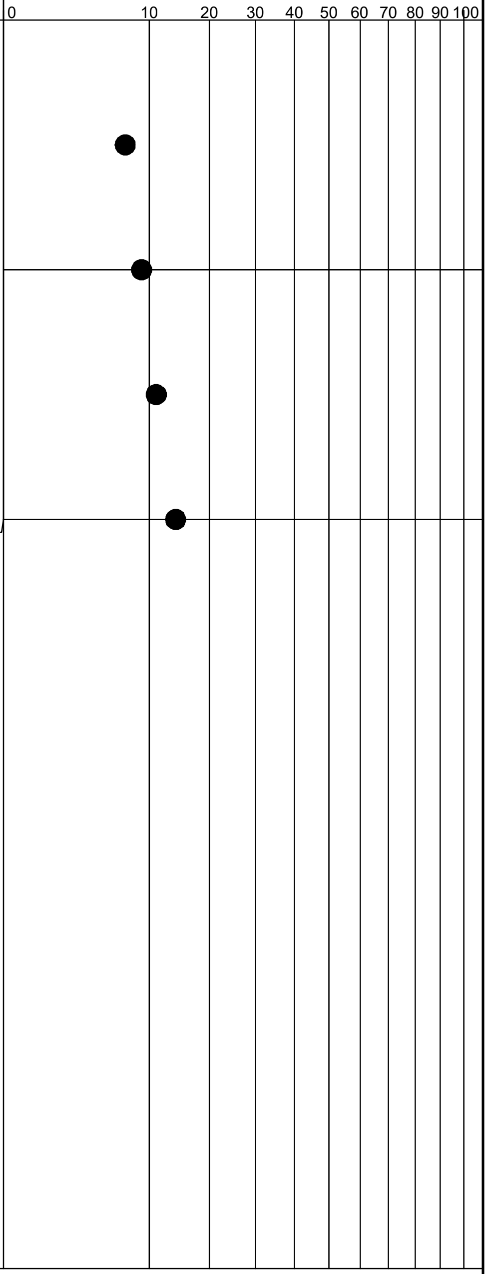
P-2

Test Boring Record

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 3 inches)	
				Firm brown, gray, and red micaceous fine to medium sandy clay (CL) (RESIDUUM)	7
	5			Stiff brown and red micaceous fine to medium sandy silt (ML)	9
				Stiff tan and brown fine sandy silt (ML)	11
				Stiff gray and white micaceous fine to coarse sand silt (ML)	
	10			Boring Terminated at 10 feet	14
	15				
	20				
	25				

Standard Penetration Test (Blows/Foot)



Remarks:

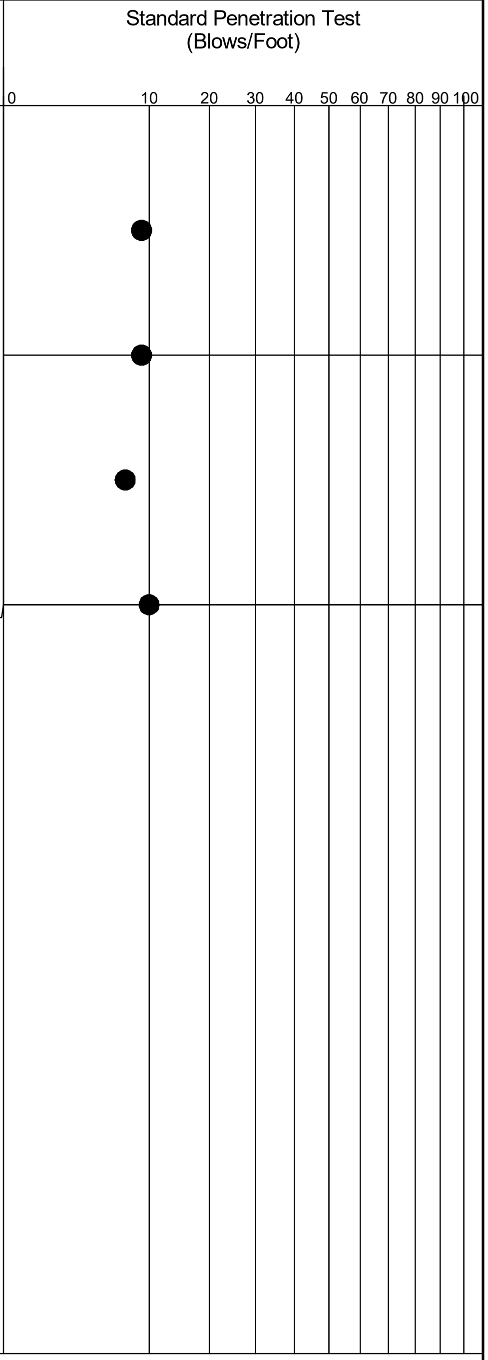
P-3

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**		Project No: **252953.20**
Location: **Colfax, North Carolina**		Date: **1/28/26**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown, red, and gray micaceous fine to medium sandy silt (ML) with clay seams (RESIDUUM)	9
				Firm to stiff brown, red, and tan micaceous fine to medium sandy silt (ML)	
	5				9
					7
	10			Boring Terminated at 20 feet	10
	15				
	20				
	25				



Remarks:

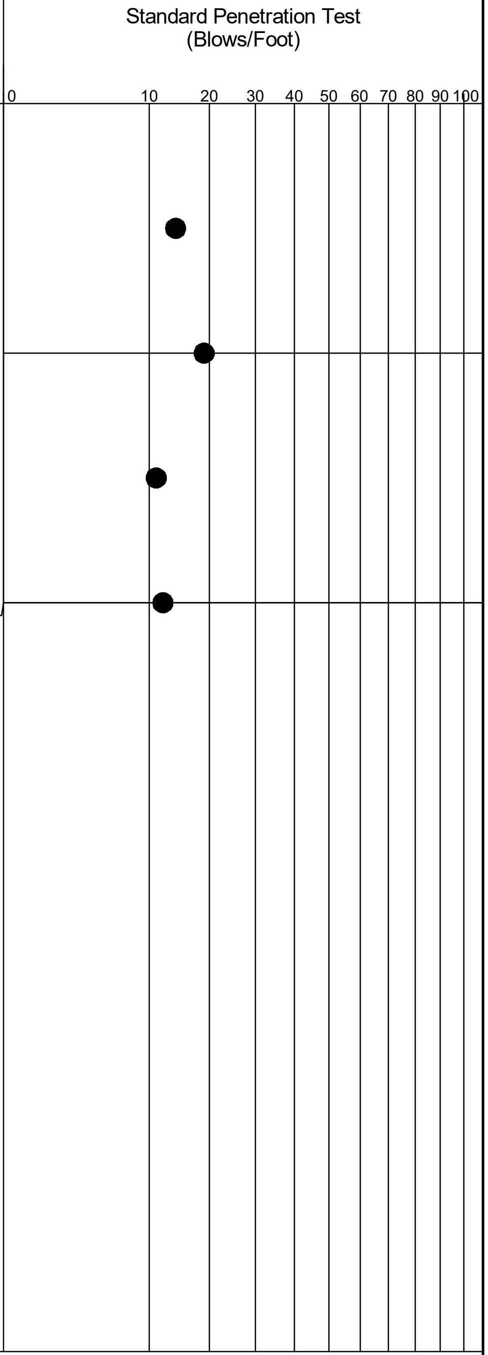
P-4

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**		Project No: **252953.20**
Location: **Colfax, North Carolina**		Date: **1/28/26**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown and red micaceous fine to medium sandy silt (ML) (RESIDUUM)	14
	5			Very stiff brown and red micaceous fine to coarse sandy silt (ML) with rock fragments	19
	10			Stiff brown and red micaceous fine to medium sandy silt (ML)	11 12
				Boring Terminated at 10 feet	
	15				
	20				
	25				



Remarks:

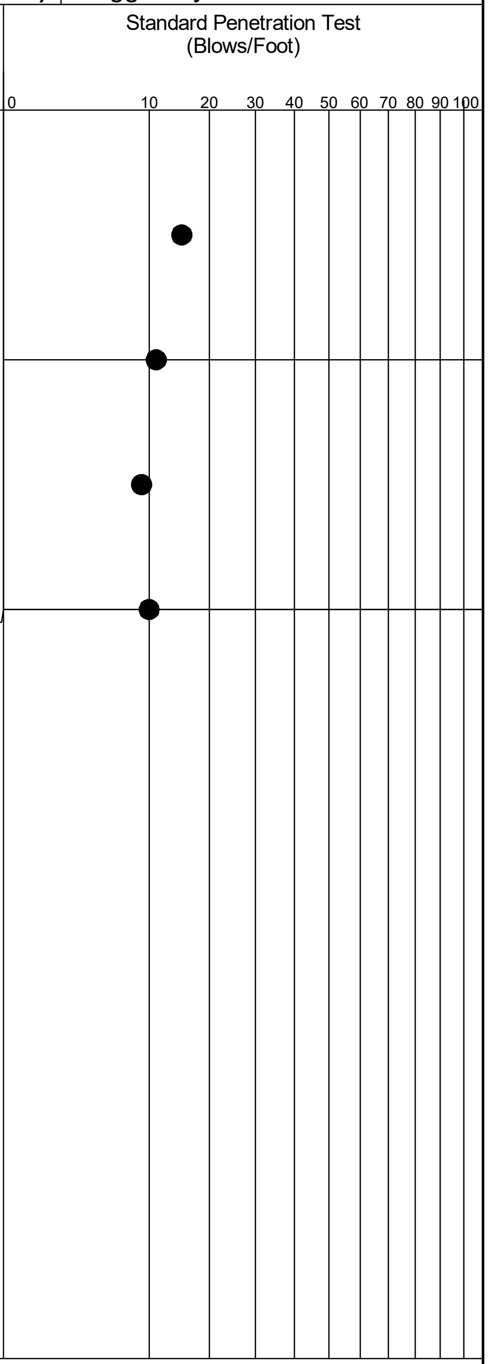
P-5

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **2/4/26**
Method: **HSA- ASTM D1586** GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff brown and red micaceous fine to medium sandy silt (ML) (RESIDUUM)	15
	5			Stiff brown and red micaceous fine to medium sandy silt (ML) with clay seams	11
				Stiff brown micaceous fine to coarse sandy silt (ML)	9
	10			Stiff brown micaceous fine sandy silt (ML)	10
				Boring Terminated at 10 feet	
	15				
	20				
	25				



Remarks:

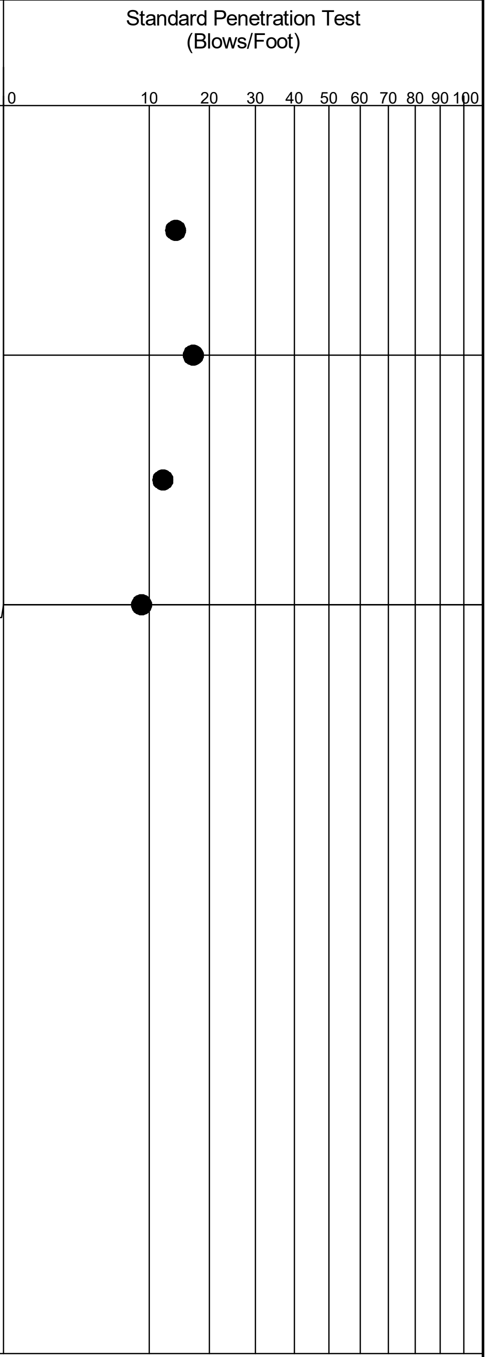
P-6

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**		Project No: **252953.20**
Location: **Colfax, North Carolina**		Date: **2/4/26**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches)	
				Stiff to very stiff brown, red, and tan micaceous fine to medium sandy silt (ML) (RESIDUUM)	14
	5				17
				Stiff gray and white micaceous fine to coarse sandy silt (ML)	12
	10			Boring Terminated at 10 feet	9
	15				
	20				
	25				



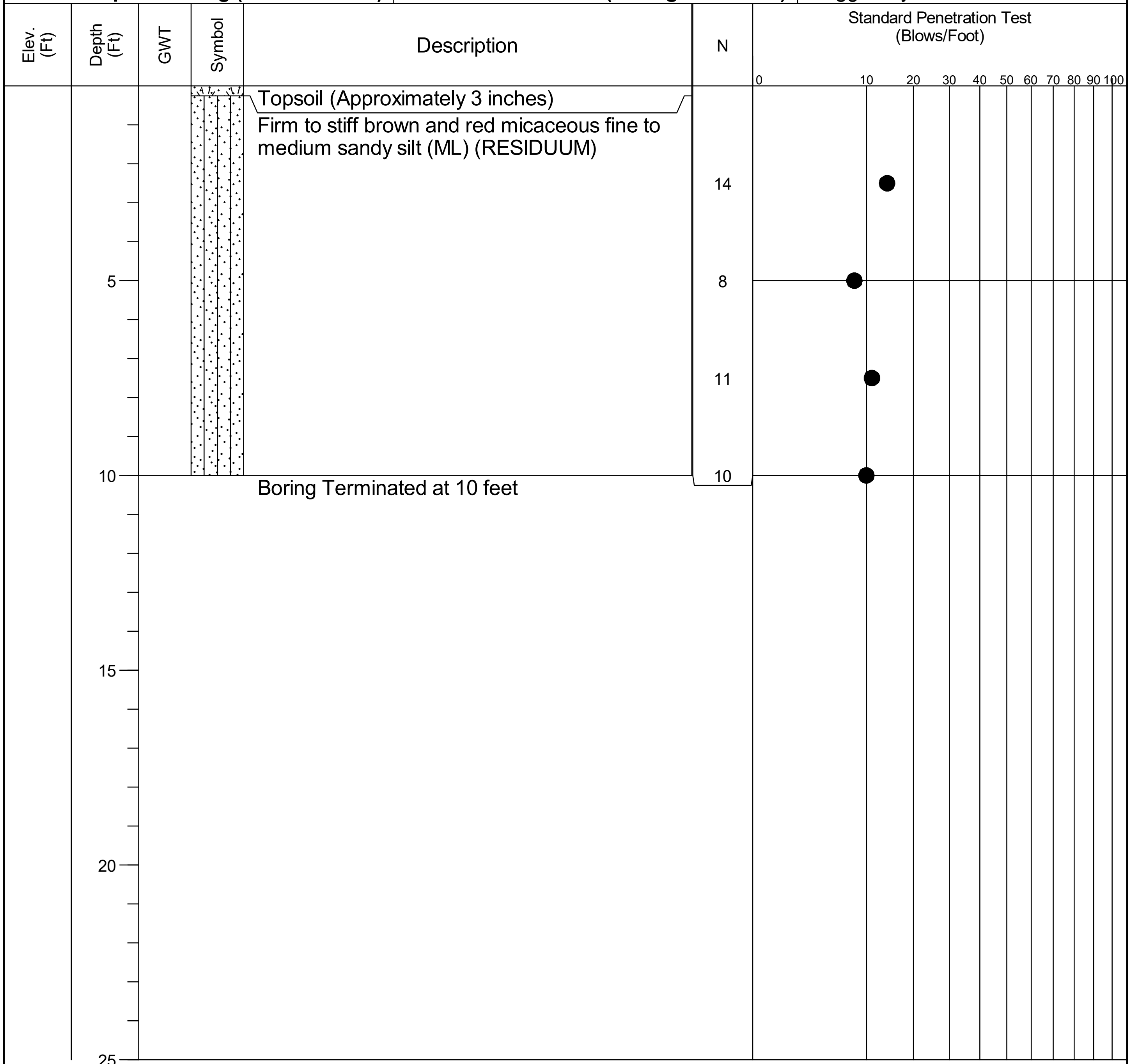
Remarks:

P-7

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/28/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **JMK**



Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 3 inches)	
				Firm to stiff brown and red micaceous fine to medium sandy silt (ML) (RESIDUUM)	14
	5				8
					11
	10				10
				Boring Terminated at 10 feet	

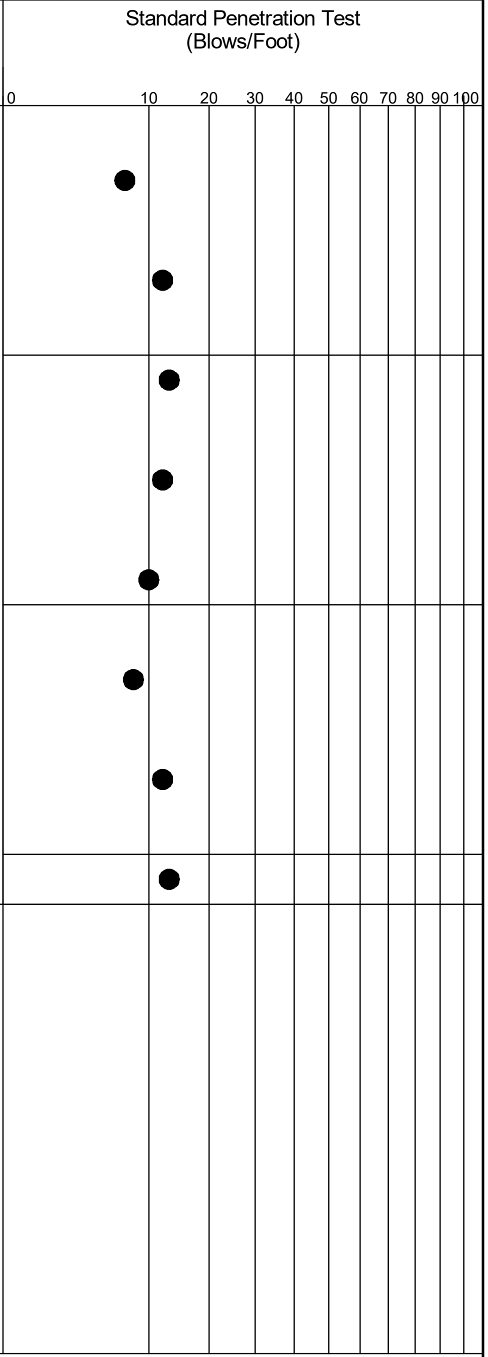
Remarks:

SW-1

Test Boring Record

Project: **Heartland Nursing Home**	Project No: **252953.20**
Location: **Colfax, North Carolina**	Date: **1/29/26**
Method: **HSA- ASTM D1586** / GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)** / GWT at 24 hrs: **Not Encountered**	Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 2 inches) Firm to stiff brown micaceous fine to medium sandy silt (ML) (RESIDUUM)	7 12
	5			Firm to stiff brown, red, and white micaceous fine to coarse sandy silt (ML)	13 12 10 8 12
	15			Firm white and gray silty fine to coarse sand (SM)	13
				Boring Terminated at 16 feet	
	20				
	25				



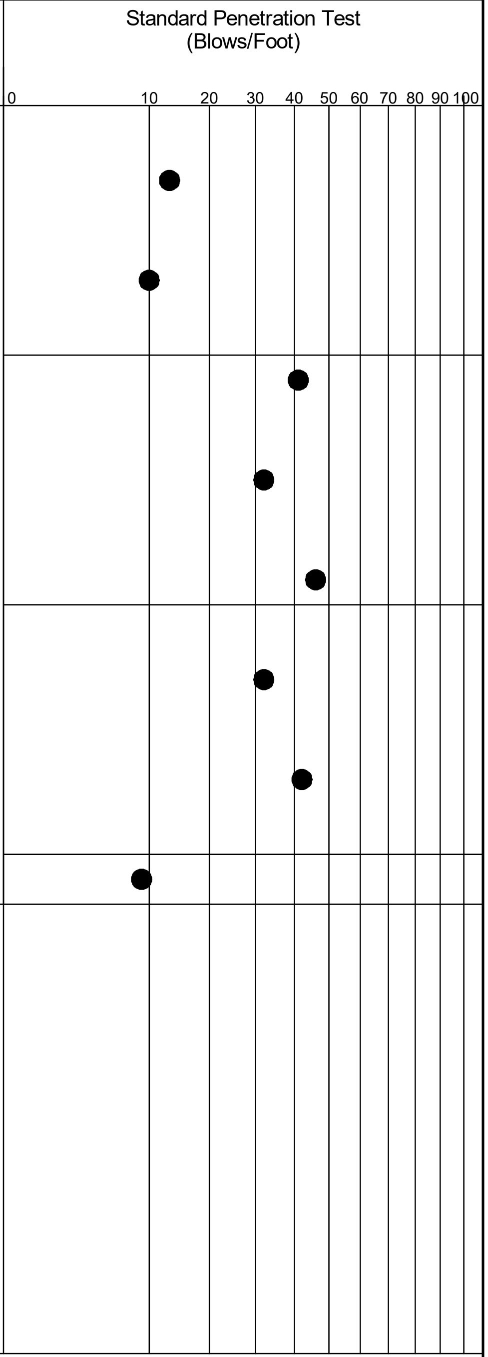
Remarks:

SW-2

Test Boring Record

Project: **Heartland Nursing Home**
Project No: **252953.20**
Location: **Colfax, North Carolina**
Date: **1/29/26**
Method: **HSA- ASTM D1586**
GWT at Drilling: **Not Encountered**
G.S. Elev:
Driller: **Capital Drilling (Auto-Hammer)**
GWT at 24 hrs: **Not Encountered**
Logged By: **JMK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
				Topsoil (Approximately 3 inches)	
				Stiff brown and red fine to medium sandy silt (ML) (RESIDUUM)	13
				Stiff brown, red, and white micaceous fine to coarse sandy silt (ML)	10
	5			Hard brown, red, and white micaceous fine to coarse sandy silt (ML)	41
					32
	10				46
				Hard gray and white fine to coarse sandy silt (ML)	32
					42
	15			Stiff brown, red, and black fine sandy silt (ML)	9
				Boring Terminated at 16 feet	
	20				
	25				



Remarks: