March 5, 2012

Mr. Al Richardson
Interim Town Manager
Town of Black Mountain
160 Midland Avenue
Black Mountain, NC 28711

Subject: Structural Engineering Report – Downtown Fire Department – Town of Black Mountain, Black Mountain, NC
Project Number: 226711

Dear Mr. Richardson:

As requested, Medlock & Associates Engineering, PA (MAE) representatives conducted site observations at downtown location of the fire department in Black Mountain, North Carolina to observe and evaluate movement in the concrete floor below the dorm room and bathrooms in the structure.

Based on our site observations, discussions with town officials, and prior experience with similar situations, we recommended performance of geotechnical testing to assist in our assessment. ECS Carolinas, LLP was contracted to perform testing of material below the slab of the existing structure and adjacent to the perimeter of the structure. We have developed the following observations, findings and recommendations:

Observations:

1. Cracks were observed in the floor of dorm rooms of the fire department through the tile in the bathrooms of the structure. It was apparent that patching had been previously attempted along the cracks.
2. Vertical movement in the slab along this observed crack indicating settlement below the slab in these areas.
3. A continuous horizontal crack was noted in the exterior brick veneer of the structure along the north wall. The crack was noted near the interior concrete slab elevation.
4. Horizontal movement was observed along this crack with the bottom veneer noted outwards relative to the brick veneer above.
5. A continuous horizontal crack was also observed along the interior CMU wall in the basement of the structure (opposite the north exterior wall noted in bullet 3).
6. The retaining wall around the exterior patio near the northwest corner of the structure was observed to have rotated outwards.
7. Several individuals on site indicated to us that the original structure was constructed in a low lying drainage area and that the structure was built on wood pilings. This information has been corroborated by others. The fact that ground water was encountered by ECS 24" below the existing basement slab and that we observed a continuously running culvert pipe below the building confirm this situation.
8. Please see geotechnical reports provided by ECS Carolinas, LLP dated July 18, 2011 and September 9, 2011 for additional geotechnical observations.

Address: 53 Asheland Avenue, Suite 101 • Asheville, NC 28801 Phone: (828) 232-4448 • Fax: (828) 232-5224 Web: www.medlockengr.com

Findings:

1. See geotechnical reports provided by ECS Carolinas, LLP dated July 18, 2011 and September 9, 2011 for geotechnical findings.
2. It is our opinion that the horizontal cracks in the walls noted in bullets 3, 4, and 5 in observations have been caused by lateral pressure created by the existing fill material beneath the slab.
3. It is our opinion that the cracking noted in the floor slab in the dorm room and bathrooms has been caused by settlement of uncontrolled (un-compacted) fill material beneath the slab.
4. It is also our opinion that the settlement and consolidation of soil beneath the bathroom areas has been exacerbated by the presence of running water in these areas. Water either entering through the showers, drains, or pipes beneath the slab would definitely effect fill material beneath the slab and could increase lateral pressure on the walls.
5. It is our opinion that the settlement of the fill material and cracking (movement) in the slab do not create a direct life safety issue with the occupancy of the structure.
6. However, we do recommend that consideration should be given for implementing repairs if long term occupancy of the building is desired.

Recommendations:

1. We recommend bracing the exterior (north) wall of the structure against the existing lateral soil pressure. Our recommendation would be to installed vertical steel channels at a spacing between six and eight feet. Channels would extend two feet below finished grade and extend 12" – 24" above the existing interior slab.
2. We recommend bracing the opposite interior CMU wall against the existing lateral soil pressure. Our recommendation would be to installed carbon fiber strips vertically at approximately five feet on center.
3. The existing concrete slab structure may be maintained, but there will be a continuing maintenance issue with associated maintenance costs (such as repairing cracked tile, patching continuing cracking, and dealing with the inconvenience).
4. An option could be considered involving installation of grade beams approximately six to eight feet on center which could be utilized to support the existing slab and to tie the opposite walls together. This would involve removal of existing floor finishes and interior walls in these locations. This would be intrusive, but provide a continuing life span for the structure. This work would be performed in addition to the wall bracing recommended in bullets 1 and 2 in recommendations. This approach would likely extend the usable life of the structure and additional 20 years.
5. Of the two options offered in the geotechnical report dated September 9, 2011, we would recommend the Polyurethane injection approach as it would be less intrusive and would require less time for implementation. The Polyurethane inject repair would require the bracing of the walls and tie system run through the slab prior to implementation. In essence, the wall bracing and grade beam ties from wall to wall would be required in addition to the Polyurethane injection.
6. The approach of only bracing the two wall systems would likely extend the useful life of the structure for another five years, whereas the grade beam approach and the Polyurethane approach would likely extend the usable life for twenty years or more.

Engineering estimates for costs of construction:

The following engineering estimates are based on our experience with similar projects in the past and reflect magnitudes of pricing, more than actual values. Direct quotes from contractors would be required to set budgets and allowances.

1. Bracing of walls only: $25,000.00
2. Bracing of walls and grade beam option: over $100,000
3. Bracing of walls and Polyurethane: over $150,000 and would require a quote from specialty contractor.

The scope of this report is limited to our assessment of the issues regarding the slab in the old portion of the existing structure. No opinion is offered, and none should be inferred, regarding other aspects of this structure or the structure taken as a whole.

We are pleased to be of service. If you have any questions regarding this report or require further assistance, please call.

Sincerely,

Frank Ungert
Project Manager

NORTH CAROLINA PROFESSIONAL SEAL 25950 ENGINEER EDWARD K. MEDLOCK

Edward K. Medlock, PE
Senior Engineer, President

03-05-12

REPORT OF LIMITED GEOTECHNICAL EVALUATION

BLACK MOUNTAIN FIRE STATION
BLACK MOUNTAIN, NORTH CAROLINA
ECS PROJECT No. 31-1830

Prepared For

MEDLOCK & ASSOCIATES ENGINEERING, INC.

Prepared By

ECS LLP
CAROLINAS

SEPTEMBER 9, 2011

ECS CAROLINAS, LLP "Setting the Standard for Service"
Geotechnical • Construction Materials • Environmental • Facilities NC Registered Engineering Firm F-1076

September 9, 2011

Mr. Ed Medlock, P.E.
Medlock & Associates Engineering, Inc.
53 Asheland Avenue, Suite 101
Asheville, North Carolina 28801

Reference: Report of Limited Geotechnical Evaluation
Black Mountain Fire Station
Black Mountain, North Carolina
ECS Project No. 31-1830

Dear Mr. Medlock:

ECS Carolinas, LLP (ECS) has completed a subsurface exploration for the subject site as authorized by acceptance of our Proposal Number 31-1884-P, dated July 1, 2011. The purpose of this exploration were to explore the subsurface conditions at the site to help evaluate the likely contributing factors for the observed distress and to provide geotechnical considerations for repair. This report presents our understanding of the project information, outlines our scope of work, summarizes the results of our exploration, and provides our evaluations and recommendations.

This report is provided for the use of Medlock & Associates Engineering, Inc. This report is not intended to be used or relied upon in connection with other projects or by other unidentified third parties without the written permission of ECS. The unauthorized use of this report by any undesignated third party or parties will be at such party's sole risk, and ECS disclaims liability for any such unauthorized third party use or reliance.

We appreciate the opportunity to provide our geotechnical services to Medlock & Associates Engineering, Inc. during this phase of your project. Please contact us if you have questions concerning this report.

Respectfully,

ECS CAROLINAS, LLP

Richard G. Adams, P.E.
Geotechnical Services Manager

NORTH CAROLINA PROFESSIONAL SEAL 31049 ENGINEER MATTHEW S. FOGLEMAN 9/9/2011

Matthew S. Fogleman, P.E.
Vice President/Branch Manager
NC Registration No. 31049

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1.0 PROJECT INFORMATION

Our understanding of this project is based upon discussions between Edward Medlock, P.E. with Medlock Engineering and Associates, Inc. (MAE) and our Mr. Richard Adams, P.E., as well as from a site meeting with MAE, the Black Mountain Fire Chief, and Mr. Deon Lytle, representative of the Town of Black Mountain. We understand that for at least 10 years there have been noticeable cracks within the tiled floors of the sleeping quarters, hallways, and showers inside the fire station and that these cracks have been re-tiled several times previously, and continue to crack. During our site visit, we also noted horizontal and stair step cracks on the exterior northern walls, as well as in the interior basement walls that appear to retain soil backfill materials.

The interior and exterior walls generally appear to be constructed of masonry block with an exterior brick veneer. A culvert was observed to pipe storm water under the building, entering on the northern side of the building. It was not clear where the pipe daylighted on the other side of the building. We also understand from Mr. Lytle that prior to construction of the fire station, a natural drainage traversed the approximate location of the pipe that was observed entering the building footprint. We understand from the Fire Chief and Mr. Lytle that at least portions of the existing building were constructed on timber piles due to the presence of soft bearing soils underlying the building footprint. Foundation plans and/or construction documents could not be located at the time of this evaluation, therefore it is not clear which portions of the building, if any, are constructed on timber piles or another foundation system.

2.0 EXPLORATION PROCEDURES

2.1 Mechanized Soil Test Borings

Three mechanized soil test borings, denoted B-1 through B-3, were drilled at the approximate locations shown on the Exploration Location Diagram in the Appendix. The boring locations were selected by ECS and MAE, in conjunction with the Town of Black Mountain and Fire Station personnel and were staked in the field by ECS. The boring locations indicated on the Exploration Location Diagram should be considered approximate.

Drilling was performed by a CME 45C trailer-mounted drill rig using continuous-flight hollow stem augers. Soil samples were obtained by means of the split-barrel sampling procedures in general accordance with ASTM D1586. In the split-barrel sampling procedure, a 2-inch O.D., split-barrel sampler is driven into the soil a distance of 18 inches by means of a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler through a 12-inch interval is termed the Standard Penetration Test (SPT) value and is indicated for each sample on the boring logs. This value can be used to provide a qualitative indication of the in-place relative density of cohesionless soils. In a less reliable way, it also indicates the consistency of cohesive soils. This indication is qualitative, since many factors can significantly affect the SPT value and prevent a direct correlation between drill crews, drill rigs, drilling procedures, and hammer-rod-sampler assemblies.

Representative portions of each SPT sample were sealed in airtight containers and returned to our laboratory for classification, testing, and storage. The soil samples were visually classified in general accordance with the Unified Soil Classification System (USCS). The basic elements of the USCS are described on a legend sheet attached in the Appendix and in ASTM D2487. Additional information from each soil boring is provided on the individual soil test boring logs in the Appendix.

2.2 Hand Auger Borings

Four hand auger borings were performed at the approximate locations indicated on the attached Exploration Location Diagram within the interior of the building to evaluate the slab subgrade conditions. Hand auger boring HA-1 was performed within the basement, near an interior foundation wall to evaluate the foundation subgrade conditions for the wall and basement slab support. Three hand auger borings, HA-2 through HA-4 were performed on the first floor to evaluate the subgrade soils beneath the distressed slab-on-grade. The slab was cored at each boring location to provide access to the underlying soils. The cores were cut using standard concrete wet-coring equipment with a diamond tipped core barrel with a nominal diameter of 6 inches. The hand auger borings were extended to depths of up to about 9 feet beneath the existing subgrade surface, unless auger refusal terminated the boring.

The hand auger borings were supplemented with Dynamic Cone Penetrometer (DCP) tests in general accordance with ASTM STP-399. The DCP tests were performed to provide relative density/stiffness of the soils encountered. Because hand auger boring HA-4 terminated with refusal on an apparent cobble or boulder at a depth of around 1 foot, this boring was supplemented with Wildcat Dynamic Cone Penetrometer (wDCP) testing, which can be extended continuously, rather than requiring hand auger boring. The wDCP was extended to the side of the auger refusal material so that an offset slab core was not necessary.

The Wildcat Dynamic Cone Penetrometer utilizes a 35 pound hammer with a 15 inch drop and drives one meter long 1 inch diameter rods into the ground. The end of the rod which is driven into the ground is outfitted with a sacrificial cone point. The rods are marked off in 10 cm increments and the number of blows of the hammer required to drive each 10 cm increment into the ground is recorded. The cone is 1.4 inches in outside diameter. A 3-gallon tank with hand operated pump injects polymer slurry through the rods to fill the annulus between rods and soil, preventing loss of energy of the hammer blows to friction between rods and surrounding soil. At each test completion, the rods are extracted by a leveraged post-popper, with the sacrificial cone point remaining in the bottom of the test hole.

The test locations are shown on the Exploration Location Diagram in the Appendix and should be considered approximate. The individual Hand Auger Records and Wildcat Dynamic Cone Log are included in the Appendix for your reference.

2.3 Laboratory Testing

Representative split-barrel samples and bulk samples obtained during our field exploration were selected and tested in our laboratory to check field classifications and to help determine pertinent index and engineering properties of the site soils. The geotechnical laboratory testing included:

- Visual classification of soil samples (ASTM D 2487),
- Index property testing of select soils samples including natural moisture content determinations (ASTM D 2216), and percent passing the No. 200 sieve (ASTM D 1140),

The laboratory test results are included on the Laboratory Testing Summary.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Observations

The site surrounding the fire station is primarily surfaced in asphaltic and concrete pavements with small landscaped areas. Cracks in the interior first floor slab-on-grade and a horizontal crack in the northern exterior subgrade wall were noted to us by fire station personnel and the Structural Engineer, Edward Medlock, P.E. (Medlock Engineering and Associates, Inc). We also noted a displacement outward of the northern exterior wall, near its proximity to the slab on grade. The displacement was on the order of ¼ to ½ inch.

We observed a French drain in the basement that apparently was constructed to help address previous groundwater issues in the basement. The drain traversed the basement floor generally from the eastern wall toward the northern wall and visually appeared to consist of ¾ inch stone or similar that was placed through a sawcut in the basement slab. The sawcut appeared to be approximately 18 inches in width. The depth of the drain is unknown.

3.2 Area Geology

The site is located in the Blue Ridge Physiographic Province of North Carolina. Located in the western portion of the state, this province is a deeply dissected mountainous area of numerous steep mountain ridges, intermountain basins, and trench valleys that intersect at all angles and give the area its rugged mountain character. The Blue Ridge is primarily composed of complexly folded and faulted igneous and metamorphic rocks. These rocks date to the PreCambrian and Paleozoic and represent parts of the basement rock of the North American continent. Streams and river channels are generally incised and trend along valleys.

Above the bedrock, the soil mantle in this area is typically composed of a thin layer of unconsolidated material consisting of residuum, alluvium, and colluvium. Residual soils (saprolite) are the product of in-place chemical weathering of the parent bedrock. Weathering is facilitated by the presence of fractures, joints, and less resistant minerals in the rock. In natural areas, the typical residual soil profile consists of clay-rich unconsolidated materials near the surface, transitioning to sandy silts and silty sands that generally become more granular and increasingly dense with depth to the transition to rock. Due to the variation of weathering rates within the residual soil strata, rock fragments, lenses, pinnacles, and boulders can remain within the residual soil matrix. Alluvial soils have been transported by stream action and are typically encountered along stream terraces and within floodplains. Colluvial soils have been deposited by gravitational action or slides and are typically encountered on and at the base of slopes, at the heads of drainage ways, and in coves. Rock fragments and boulders can also be present within alluvial and colluvial soils due to the depositional nature of these soil masses.

The boundary between soil and rock may not be sharply defined in some areas. A transitional zone termed "partially weathered rock" (PWR) often is present at the interface between residual soils and the underlying bedrock. For engineering purposes, PWR is defined as material which can be penetrated by a power auger, but which exhibits Standard Penetration Test resistances (N-values) of greater than 100 blows per foot.

In general, the unconfined shallow water table is contained within the near-surface soil mantle and is controlled primarily by topographic gradients. Recharge occurs primarily by infiltration along higher elevations and typically discharges into streams or other surface water bodies. The elevation of the shallow water table is transient and can vary with seasonal fluctuations in precipitation. As the groundwater percolates downward into the bedrock, it becomes controlled by the orientation of the rock fracture systems and can follow complex pathways through the rock. Thus, the direction of deeper groundwater movement may not be consistent with the topography. Actual site groundwater depths and flow direction cannot be determined without long-term monitoring through the installation of groundwater monitoring wells.

It is important to note that the natural geology within portions of the site has been modified in the past by the placement of fill materials. The quality of man-made fills can vary significantly, and it is often difficult to assess the engineering properties of existing fills. Furthermore, there is no specific correlation between N-values from the standard penetration test and the degree of compaction of existing fill soils; however, a qualitative assessment of existing fills can sometimes be made based on the N-values obtained and observations of the materials sampled in the test borings.

3.3 Subsurface Conditions

Mechanized Borings:

The borings initially encountered approximately 2 inches of asphaltic concrete. Underlying the pavement, fill materials were encountered to depths of about 9 feet in Boring B-1 and about 4 feet in Boring B-3. The fill material typically consisted of silty fine to medium sand (SM) with trace gravel. Standard Penetration Test (SPT) N-values obtained in the fill ranged from 4 to 15 bpf. Fill was not encountered in Boring B-2.

Underlying the fill and surficial materials, alluvial soils were encountered to depths ranging between about 7 and 19 feet below the surface. The alluvium typically consisted of silty fine sand (SM). N-Values in the alluvial deposits typically ranged from 3 to 28 bpf. The higher N-values are likely artificially inflated due to the presence of gravel within the alluvium. The alluvium was underlain by residual soils. The residuum typically classified as silty fine to medium sand (SM) with N-values ranging between 5 to 41 bpf with typical values ranging between approximately 15 and 30 bpf.

Partially weathered rock (PWR) was encountered in Boring B-3 at a depth of about 48 feet and extended to the termination depth of 50 feet. The PWR was sampled as silty fine to medium sand (SM).

Groundwater was encountered at a depth of about 9 to 10 feet in all three borings. Fluctuations of the groundwater table will occur due to seasonal variations in the amount of rainfall, surface runoff, and other factors not evident at the time the borings were performed. In addition, perched water may develop within sandy seams that overlie lower permeability layers following periods of heavy or prolonged rainfall. Therefore, groundwater levels during construction or at other times in the life of the project may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and repair plans for the project.

The above paragraphs provide a general summary of the subsurface conditions encountered at the site at the time of our exploration. The Boring Logs included in the Appendix contain detailed information regarding the subsurface conditions encountered at each boring location. These Boring Logs represent our visual interpretation of the subsurface profile based on the samples retrieved during the field exploration. The stratification lines on the Boring Logs indicate approximate boundaries between various strata. The actual in-situ transitions between soil types are usually more gradual.

Hand Auger Borings:

The core removed from the basement slab (HA-1) and the cores removed from the main floor slab on grade (HA-2 through HA-4) had a typical thickness of about 6 to 8 inches with the exception of the core from location HA-4. The concrete at this location was 12 inches thick and consisted of apparently 3 separate concrete slabs, each approximately 4 inches thick. Obvious voids were not observed beneath the concrete slab at the coring locations.

Underlying the concrete slab on grade on the main floor, fill materials were encountered to the termination depth of each boring. The fill typically consisted of silty fine to medium sand and had DCP values ranging between 1 and 9 blows per increment. Underlying the slab in the basement, soft/loose alluvial soils were encountered to a depth of approximately 5½ feet. The alluvium typically consisted of silty fine sand and had DCP values typically ranging between 3 and 7 blows per increment with occasionally higher values which were artificially inflated due to the presence of gravel, cobble and apparent wood debris near the termination depth of the boring. Groundwater was encountered at a depth of approximately 2 feet below the ground surface at the time of exploration, and was observed to have stabilized at a depth of about 4 inches after approximately 5 hours. Water was not encountered in the other hand auger borings.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration results, and observations onsite, the displacement of the exterior foundation wall (around ¼ to ½ inch off vertical in 4 feet) on the north side of the building, to the outside, appears to be indicative of a rotational movement attributed to lateral earth pressures behind the wall, particularly if the wall was not designed to retain the backfill. Because of the coarse-grained nature of the majority of the soils encountered below footing elevation, we would expect settlement or rotation due to soil bearing capacity issues to have occurred relatively soon after construction (within a week or two of final loading of the structure) rather than slowly, like with consolidation settlement of saturated fine-grained soils.

The distress observed within the main floor slabs generally appears to be caused by poor subgrade support of the underlying fill soils. The borings performed through the concrete slab on grade encountered soft/loose fill materials to depths of 8 to 9 feet below the surface. DCP values obtained in the fill soil do not indicate that the fill was placed with sufficient compactive effort. Slabs constructed over poorly compacted fill can be expected to crack and settle because of compression of the underlying soils.

Our considerations for repair regarding the above described exploration results are presented below.

4.1 Concrete Slab Subgrade Repair

Because of apparent insufficient slab support conditions indicated by our subsurface exploration, there are a couple of options for remedial repair of the slabs. One option is to modify the subgrade soils with polyurethane foam to improve the subgrade support for the slabs. An alternative repair is to install underpins through the slab to transfer the floor loads through the loose fill soils into more competent residual soils. We would expect that the option with polyurethane foam may be a more cost effective and less invasive approach than underpinning the slab, and would recommend that be considered the preferred alternative.

Option 1.

The subgrade soils may be modified by using polyurethane foam, to densify the underlying fill soils to depths of at least 4 to 9 feet below the slabs. Polyurethane foam can be injected through small conduits installed though the slab (approximately ½ inch in diameter). The process is generally not as intrusive and disruptive as the installation of underpins through the slab. Because of the potentially compressible nature of the underlying soils, we recommend the polyurethane foam be considered the preferred method of repair, rather than grout or cement injection. Polyurethane will not significantly add weight to the underlying soils. Additional weight could induce further compression of underlying soils, resulting in additional slab settlement.

The polyurethane used for modifying the subgrade soils beneath the slab should consist of a high density, hydro insensitive (meaning it reacts the same in wet environments as it does in dry environments), expandable polyurethane material. Given the potential for shallow groundwater beneath the slab, a polyurethane-forming mixture, having water insoluble dilutants, that permit the formation of polyurethanes in excess water would be preferred. The actual depth and design of treatment should be determined by the geotechnical contractor, using our subsurface data as a guide for development of the subgrade repair.

Option 2.

Alternatively, slab underpins may be installed to transfer the floor loads to deeper more competent soils. They should be installed in a grid pattern below the slab at all locations where cracking or other distress has been observed. Typical spacing of underpin elements is usually on the order of 4 to 5 feet spacing under the slab. However, the specific design of the underpin system, such as spacing, structural connections and load carrying capacity, should be developed specific to the floor loading and subsurface conditions encountered at this site. Underpins should be extended into competent residual soils below the fill and alluvial soils. Based upon the subsurface information obtained during this exploration, we recommend that underpins extend to depths on the order of 20 to 25 feet to reach competent residual soils to develop the capacity needed to support the estimated floor loads.

Various types of elements may be considered for use as underpins. Typically helical piers, resistance piers or micropiles are used to for this application. If large rock fragments, cobble or boulders are present within the alluvial layer, helical piers or resistance piers may refuse on this material and may not reach the depths necessary to achieve proper support. In such cases, drilled micropiles may need to be used instead. Because our borings did not encounter materials causing auger refusal, helical piers or resistance piers may be considered. However debris or rocks could be present between the boring locations, which may have gone undetected in our exploration program. Drilled micropiles should be considered the preferred underpin alternative to reach competent support materials. Additional considerations for micropile design are provided in Section 4.3.

ECS should be retained to review the foundation contractor's proposed design and to monitor and document the installation of the repair selection to check that they are extending to appropriate depths, to check that they're being installed in accordance with the contractor's specifications, and they meet the general intent of our recommendations.

4.2 Northern Perimeter Foundation Wall and Interior Basement Walls

As discussed above, the conditions observed while onsite and those encountered during the subsurface exploration, appear to indicate that the distress observed in the basement walls is moistly likely associated with excessive lateral earth pressure beyond what they were designed for.

The walls should be secured or tied back in some manner to prevent additional outward movement. Soil tiebacks, such as Platipus anchors or similar products installed through the face of the walls and anchored into the fill soils behind the wall may be considered, however, careful consideration of the subgrade soils behind the walls is imperative. It is likely that it will be difficult to develop the passive resistance required to anchor the walls due to the very loose compaction of the backfill.

The walls will likely require additional reinforcement by installing structural elements such as steel C-channels or carbon-fiber reinforcement so that tiebacks or other forms of restraint do not shear through the wall face. ECS can provide non-destructive testing and/or invasive testing of the wall to help characterize the existing wall construction and reinforcement, if requested.

4.3 Micropile Design Considerations

A micropile is a small-diameter, drilled and grouted, non-displacement pile that is typically reinforced with an interior steel rod and/or an exterior steel casing. A micropile is constructed by drilling a borehole, placing steel reinforcement, and grouting the hole as illustrated in Figure 1-1 below. Micropiles can withstand relatively significant axial load and moderate lateral loads, and may be considered a substitute for conventional driven piles or drilled shafts. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soils, and the environment. They can be installed where access is restrictive and in all soil types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for the installation of ground anchors and for grouting projects.

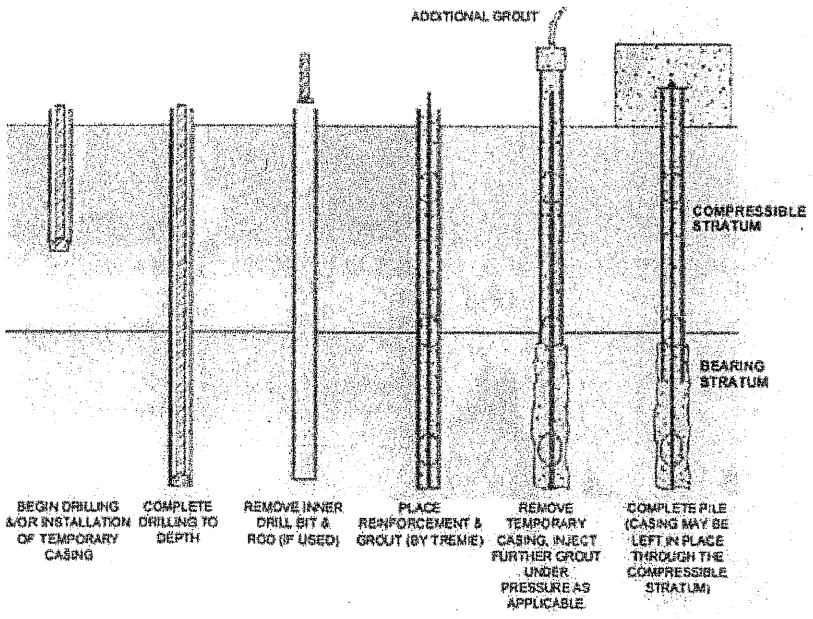


Figure 1-1. Micropile Construction Sequence.

FHWA NHI-05-039
Micropile Design & Construction
1-4
Chapter 1 - Introduction
December 2005

(FHWA NHI-05-039 December 2005)

Micropiles constructed for support of the existing slabs should bear in the medium dense to very dense silty sand residual soils or partially weathered rock at depths of at least 25 feet or greater below the existing grade. Soil parameters listed below for each soil stratum in this area may be used for micropile design.

Typical Depth (ft)	Generalized Soil Type	USCS Classification	Average N-values (bpf)	Friction Angle, phi (deg)	Cohesion (psf)	Unit Weight, γ (pcf)
0 - 5	FILL/ALLUV	SM	5	neglect	neglect	105
5 - 10	SAND	SM	6	28	0	105
10 - 20	SAND	SM	20	30	0	115
20 - 45	SAND	SM	30+	32	0	120
PWR	SAND	SM	100+	36	0	125

5.0 CLOSING

Our evaluation of foundation support conditions has been based on our understanding of the site and project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between the borings. In evaluating the boring data, we have examined previous correlations between penetration resistance values and foundation bearing pressures observed in soil conditions similar to those at your site. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our evaluation. The assessment of site environmental conditions for the presence of pollutants in the soil and groundwater of the site was beyond the scope of this geotechnical exploration.

APPENDIX

Boring Location Diagram

Unified Soil Classification System

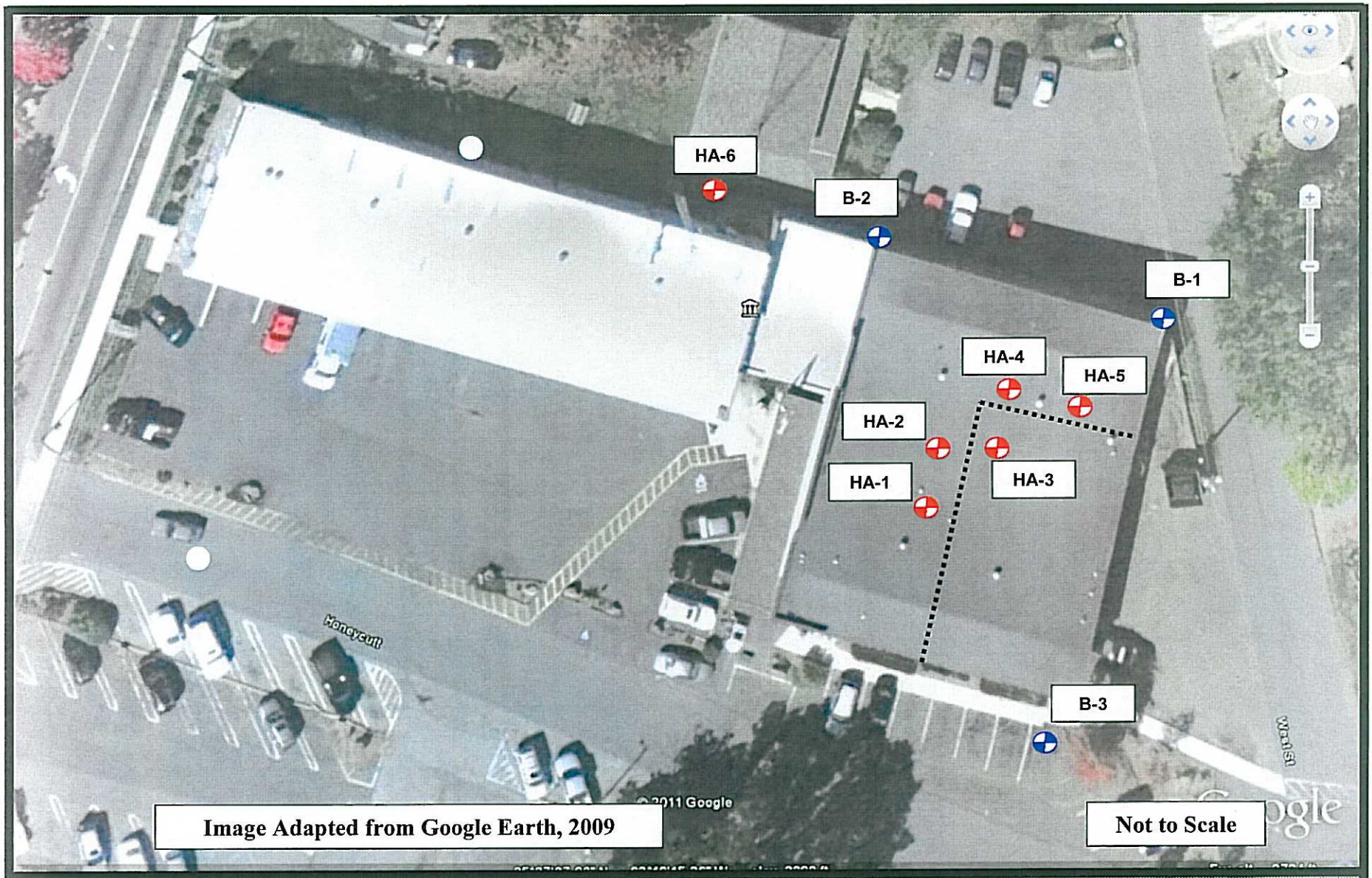
Reference Notes for Boring Logs

Boring Logs B-1 through B-3

Hand Auger Records

Wildcat Dynamic Cone Log

Laboratory Summary Sheet



LEGEND

HA-1 — Coring and Hand Auger Boring/DCP's

B-1 — Boring Locations

..... Interior Basement Walls

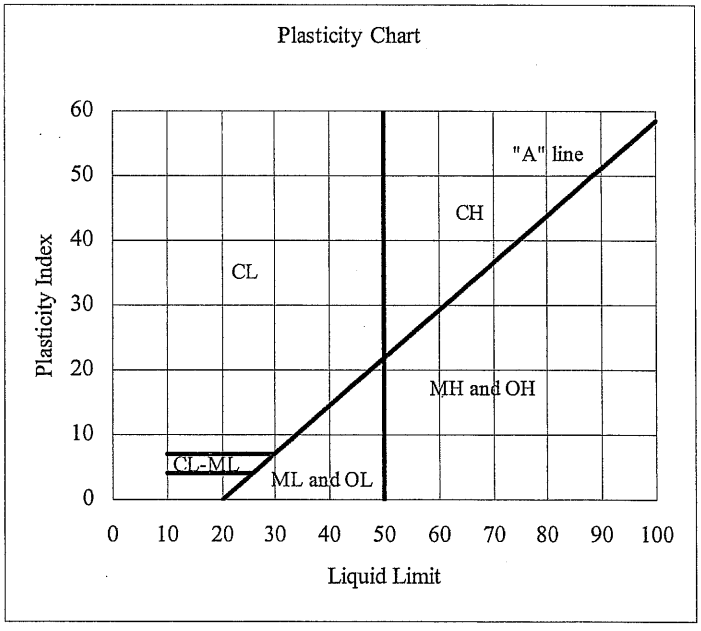
ECS

EXPLORATION LOCATION DIAGRAM
Black Mountain Fire Station
106 Montreat Road
Black Mountain, North Carolina
ECS Project Number 31-1830

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria	
Coarse-grained soils (More than half of material is larger than No. 200 Sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ greater than 4 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand mixtures	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u			
			GC		Clayey gravels, gravel-sand-clay mixtures	Atterberg limits below "A" line or P.I. less than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ greater than 6 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures	Atterberg limits above "A" line or P.I. less than 4	Limits plotting in CL-ML zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u			
			SC		Clayey sands, sand-clay mixtures	Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 Sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			OL		Organic silts and organic silty clays of low plasticity		
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH		Inorganic clays of high plasticity, fat clays		
			OH		Organic clays of medium to high plasticity, organic silts		
	Highly Organic soils		Pt		Peat and other highly organic soils		

Determine percentages of sand and gravel from grain-size curve.
Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:
Less than 5 percent — GW, GP, SW, SP
More than 12 percent — GM, GC, SM, SC
5 to 12 percent — Borderline cases requiring dual symbols[b]



[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC,well-graded gravel-sand mixture with clay binder. (From Table 2.16 - Winterkorn and Fang, 1975)

REFERENCE NOTES FOR BORING LOGS

I. Drilling Sampling Symbols

SS	Split Spoon Sampler	ST	Shelby Tube Sampler
RC	Rock Core, NX, BX, AX	PM	Pressuremeter
DC	Dutch Cone Penetrometer	RD	Rock Bit Drilling
BS	Bulk Sample of Cuttings	PA	Power Auger (no sample)
HSA	Hollow Stem Auger	WS	Wash sample
REC	Rock Sample Recovery %	RQD	Rock Quality Designation %

II. Correlation of Penetration Resistances to Soil Properties

Standard Penetration (blows/ft) refers to the blows per foot of a 140 lb. hammer falling 30 inches on a 2-inch OD split-spoon sampler, as specified in ASTM D 1586. The blow count is commonly referred to as the N-value.

A. Non-Cohesive Soils (Silt, Sand, Gravel and Combinations)

Density		*Relative Properties*	
Under 4 blows/ft	Very Loose	Adjective Form	12% to 49%
5 to 10 blows/ft	Loose	With	5% to 12%
11 to 30 blows/ft	Medium Dense		
31 to 50 blows/ft	Dense		
Over 51 blows/ft	Very Dense		

Particle Size Identification		
Boulders		8 inches or larger
Cobbles		3 to 8 inches
Gravel	Coarse	1 to 3 inches
	Medium	½ to 1 inch
	Fine	¼ to ½ inch
Sand	Coarse	2.00 mm to ¼ inch (dia. of lead pencil)
	Medium	0.42 to 2.00 mm (dia. of broom straw)
	Fine	0.074 to 0.42 mm (dia. of human hair)
Silt and Clay		0.0 to 0.074 mm (particles cannot be seen)

B. Cohesive Soils (Clay, Silt, and Combinations)

Blows/ft	*Consistency*	*Unconfined Comp. Strength Q_p (tsf)*	*Degree of Plasticity*	*Plasticity Index*
Under 2	Very Soft	Under 0.25	None to slight	0 – 4
3 to 4	Soft	0.25-0.49	Slight	5 – 7
5 to 8	Medium Stiff	0.50-0.99	Medium	8 – 22
9 to 15	Stiff	1.00-1.99	High to Very High	Over 22
16 to 30	Very Stiff	2.00-3.00		
31 to 50	Hard	4.00–8.00		
Over 51	Very Hard	Over 8.00		

III. Water Level Measurement Symbols

WL	Water Level	BCR	Before Casing Removal	DCI	Dry Cave-In
WS	While Sampling	ACR	After Casing Removal	WCI	Wet Cave-In
WD	While Drilling	▽	Est. Groundwater Level	▽	Est. Seasonal High GWT

The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in a granular soil. In clay and plastic silts, the accurate determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally applied.

CLIENT	JOB #	BORING #	SHEET
Medlock & Associates Engineering, Inc.	1830	B-1	1 OF 1

PROJECT NAME	ARCHITECT-ENGINEER
Black Mountain Fire Station	

SITE LOCATION
106 Montreat Road, Black Mountain, NC 28711

CALIBRATED PENETROMETER TONS/FT.^{2} 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % Δ

ROCK QUALITY DESIGNATION & RECOVERY
RQD% — — — REC.% ———
20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT.
10 20 30 40 50+

DESCRIPTION OF MATERIAL ENGLISH UNITS
BOTTOM OF CASING ▶ LOSS OF CIRCULATION 100%
SURFACE ELEVATION

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	8	Asphalt Depth 2"		4 (4-2-2)
	2	SS	18	12	FILL: Clayey Fine to Medium SAND, With Trace Gravel, Brown and Dark Brown, Moist, (FILL)		15 (4-4-11)
	3	SS	18	7			9 (8-5-4)
5	4	SS	18	12			4 (2-2-2)
	5	SS	18	16	ALLUVIUM: Silty Fine SAND, Light Gray to White, Moist, Loose, (SM)		5 (4-2-3)
10	6	SS	18	5			6 (4-3-3)
15					ALLUVIUM: Silty Fine SAND, Brown, Moist, Loose to Medium Dense, (SM)		
	7	SS	18	18	RESIDUUM: Silty Fine to Medium SAND, Brown, Moist, Medium Dense, (SM)		16 (6-6-10)
20							
	8	SS	18	17			28 (6-11-17)
25					END OF BORING @ 25.0'		
30							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL (WS) OR WD	BORING STARTED 07/07/11	DRILLER: Jordan Environmental
▽WL(BCR) 9.00 ▽WL(ACR)	BORING COMPLETED 07/07/11	CAVE IN DEPTH @ 16.0'
▽WL	RIG 45C Tk. FOREMAN BJ	DRILLING METHOD HSA

SSAWYER (07-13-11) SSAWYER (07-13-11) SSAWYER (09-07-11) SSAWYER (09-07-11) SSAWYER (09-07-11)

SSoyer

CLIENT	JOB #	BORING #	SHEET
Medlock & Associates Engineering, Inc.	1830	B-2	1 OF 1

PROJECT NAME	ARCHITECT-ENGINEER
Black Mountain Fire Station	

SITE LOCATION
106 Montreat Road, Black Mountain, NC 28711

CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % — LIQUID LIMIT % Δ

ROCK QUALITY DESIGNATION & RECOVERY
RQD% — — — REC.%
20% 40% 60% 80% 100%

STANDARD PENETRATION BLOWS/FT. ⊗
10 20 30 40 50+

DESCRIPTION OF MATERIAL ENGLISH UNITS

BOTTOM OF CASING ▶ LOSS OF CIRCULATION 100%

SURFACE ELEVATION

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	12	Asphalt Depth 2"		3 (1-1-2)
	2	SS	18	8	ALLUVIUM: Silty Fine SAND, Gray and Black, Moist, Very Loose, (SM)		3 (2-1-2)
	3	SS	18	2			3 (5-2-1)
5	4	SS	18	12			(6-41-13) 54
					ALLUVIUM: Silty Fine SAND With Quartz Fragments, Gray and Black, Moist, Very Dense, (SM)		
	5	SS	18	2			12 (4-4-8)
10					ALLUVIUM: Silty Fine to Medium SAND, Gray and Black to Brown, Moist, Medium Dense, (SM)		
	6	SS	18	18	RESIDUUM: Silty Fine to Medium SAND, Reddish Brown, Moist, Medium Dense to Dense, (SM)		18 (5-10-8)
15							
	7	SS	18	17			19 (7-10-9)
20							
	8	SS	18	18			(17-10-25) 35
25					END OF BORING @ 25.0'		
30							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL (WS OR WD)	BORING STARTED 07/07/11	DRILLER: Jordan Environmental
▽WL(BCR) 10.00 ▽WL(ACR)	BORING COMPLETED 07/07/11	CAVE IN DEPTH @ 16.5'
▽WL	RIG 45C Tk. FOREMAN BJ	DRILLING METHOD HSA

SSAWYER (07-13-11) SSAWYER (07-13-11) SSAWYER (09-07-11) SSAWYER (09-07-11) SSAWYER (09-07-11)

(07/11/2011)

SSawyer

CLIENT	JOB #	BORING #	SHEET
Medlock & Associates Engineering, Inc.	1830	B-3	1 OF 2

PROJECT NAME	ARCHITECT-ENGINEER
Black Mountain Fire Station	

SITE LOCATION: 106 Montreat Road, Black Mountain, NC 28711

CALIBRATED PENETROMETER TONS/FT.^{2} (1 2 3 4 5+)

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % Δ

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — REC.% — (20% 40% 60% 80% 100%)

STANDARD PENETRATION BLOWS/FT. (10 20 30 40 50+)

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL — ENGLISH UNITS	WATER LEVELS ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
					BOTTOM OF CASING — LOSS OF CIRCULATION 100%		
					SURFACE ELEVATION		
0	1	SS	18	13	Asphalt Depth 2"		5 (3-2-3)
	2	SS	18	10	FILL: Sandy CLAY, Brown, Moist, (FILL)		5 (3-2-3)
5	3	SS	18	8	ALLUVIUM: Silty Fine to Medium SAND, Brown and Gray and Black, Moist, Loose, (SM)		5 (8-3-2)
	4	SS	18	15	RESIDUUM: Silty Fine to Medium SAND, With Quartz Fragments, Light Gray to White, Moist, Medium Dense, (SM)		28 (7-16-12)
10	5	SS	18	15	Fine Sandy SILT, Light Gray and Gray, Moist, Stiff to Medium Stiff, (ML)	▽	10 (6-5-5)
	6	SS	18	18			5 (2-3-2)
15					Silty Fine to Medium SAND, Light Gray and Gray, Moist, Medium Dense, (SM)		
	7	SS	18	18			12 (4-4-8)
20							
	8	SS	18	18			24 (6-11-13)
25							
	9	SS	18	18			20 (5-8-12)
30					Silty Fine to Medium SAND, Reddish Brown, Moist, Medium Dense to Dense, (SM)		

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL (WS OR WD)	BORING STARTED 07/07/11	DRILLER: Jordan Environmental
▽WL(BCR) 10.00 ▽WL(ACR)	BORING COMPLETED 07/07/11	CAVE IN DEPTH @ 20.0'
▽WL	RIG 45C Tk. FOREMAN BJ	DRILLING METHOD HSA

SSAWYER (07-13-11) SSAWYER (09-07-11) SSAWYER (09-07-11) SSAWYER (09-07-11)

(07/11/2011)

SSawyer

CLIENT	JOB #	BORING #	SHEET
Medlock & Associates Engineering, Inc.	1830	B-3	2 OF 2

PROJECT NAME	ARCHITECT-ENGINEER
Black Mountain Fire Station	

SITE LOCATION: 106 Montreat Road, Black Mountain, NC 28711

CALIBRATED PENETROMETER TONS/FT.^{2}: 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % Δ

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% — 20% 40% 60% 80% 100%

STANDARD PENETRATION BLOWS/FT. ⊗: 10 20 30 40 50+

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL — ENGLISH UNITS
30					Silty Fine to Medium SAND, Reddish Brown, Moist, Medium Dense to Dense, (SM)
35	10	SS	18	16	(10-15-18) ⊗ 33
40	11	SS	18	18	(7-19-22) ⊗ 41
45	12	SS	18	17	⊗ 25 (8-12-13)
50	13	SS	12	12	PARTIALLY WEATHERED ROCK – Sampled As Silty Fine to Medium SAND, Reddish Brown, Moist, (PWR)
					(21-50/6) ⊗ 50/6
					END OF BORING @ 50.0'
55					
60					

WATER LEVELS ELEVATION (FT)

BOTTOM OF CASING ▶ — LOSS OF CIRCULATION 100%

SURFACE ELEVATION

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL (WS OR WD)	BORING STARTED 07/07/11	DRILLER: Jordan Environmental
▽WL(BCR) 10.00 ▽WL(ACR)	BORING COMPLETED 07/07/11	CAVE IN DEPTH @ 20.0'
▽WL	RIG 45C Tk. FOREMAN BJ	DRILLING METHOD HSA

SSAWYER (07-13-11) SSAWYER (09-07-11) SSAWYER (09-07-11) SSAWYER (09-07-11)

(07/11/2011) SSawyer

ECS Carolinas, LLP
200 Ridgefield Court
Asheville, NC 28806
(828) 665-2307 - Phone
(828)665-8128- Fax

HAND AUGER RECORD

ECS Project Name: Black Mountain Fire Station ECS Project No.: 1830
Date: Technician: Dillon Kestner, Justin Biondi

Penetrometer Blow Counts

LOCATION/REMARKS	DEPTH (ft)	1-3/4"	1-3/4"	1-3/4"	SOIL DESCRIPTION
HA-1; 6" CONCRETE	SURFACE	4	3	3	BROWN SILTY SAND, MOIST
GW 4" BELOW GRADE AFTER 5 HRS.	-1	5	5	6	
	-2	7	7	6	
	-3	3	4	5	BROWN SILTY SAND, WET
	-4	3	7	16	GREY SILTY SAND W/ COBBLE
AR -5.5 (WOOD DEBRIS)	-5	17	10	8	
HA-2; 8" CONCRETE	SURFACE	3	2	2	FILL, RED SILTY SAND, MOIST
	-1	2	2	2	
	-2	2	1	2	
SAMPLE 1	-3	2	1	1	
	-4	2	2	3	RED SILTY SAND W/ CLAY, MOIST
	-5	2	2	2	
	-6	3	5	5	
	-7				NO DATA
SAMPLE 2	-8	7	5	3	RESIDUUM -7.5'
EOB, NO GW ENCOUNTERED	-9	-	-	-	GRAY CLAYEY FINE SAND
HA-3; 6" CONCRETE	SURFACE	1	1	2	RED SILTY SAND, ROCK FRAG
	-1	7	8	4	RED SILTY SAND, MOIST
	-2	3	3	3	
	-3	3	3	3	
	-4	7	6	7	
	-5	6	5	5	DK BRN SILTY MED SAND, MOIST
	-6	4	4	5	
	-7	6	9	6	
NO GW ENCOUNTERED	-8	5	5	5	-7.5' GREY CLAYEY FINE SAND
HA-4; 12" CONCRETE	SURFACE	3	6	7	#57 STONE ~8"
AR -1.1'	-1	6	2	2	RED SILTY SAND

WILDCAT DYNAMIC CONE LOG

ECS Carolinas, LLP
8702-A Red Oak Blvd.
Charlotte, NC 28217

PROJECT NUMBER: 31-1830
DATE STARTED:
DATE COMPLETED:

HOLE #: WC #1
CREW: JB/DK
PROJECT: Black Mountain Fire Station
ADDRESS: 106 Montreat Road
LOCATION: Black Mountain, North Carolina

SURFACE ELEVATION:
WATER ON COMPLETION: NA
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	TESTED CONSISTENCY COHESIVE
-	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
-	3	13.3	•••	3	VERY LOOSE	SOFT
- 1 ft	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
-	3	13.3	•••	3	VERY LOOSE	SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
- 2 ft	2	8.9	••	2	VERY LOOSE	SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
-	2	8.9	••	2	VERY LOOSE	SOFT
- 3 ft	2	8.9	••	2	VERY LOOSE	SOFT
- 1 m	2	8.9	••	2	VERY LOOSE	SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
- 4 ft	3	11.6	•••	3	VERY LOOSE	SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 5 ft	2	7.7	••	2	VERY LOOSE	SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
-	3	11.6	•••	3	VERY LOOSE	SOFT
- 6 ft	3	11.6	•••	3	VERY LOOSE	SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 2 m	1	3.9	•	1	VERY LOOSE	VERY SOFT
- 7 ft	1	3.4		0	VERY LOOSE	VERY SOFT
-	1	3.4		0	VERY LOOSE	VERY SOFT
-	1	3.4		0	VERY LOOSE	VERY SOFT
- 8 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	12	41.0	••••••••••	11	MEDIUM DENSE	STIFF
-	10	34.2	•••••••••	9	LOOSE	STIFF
- 9 ft	10	34.2	•••••••••	9	LOOSE	STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	30	102.6	•••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 3 m 10 ft						
-						
-						
-						
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

ECS Carolinas, LLP
Asheville, NC
Laboratory Testing Summary

Printed on (date): September 07, 2011

Project No. 1830 **Project Name.** Black Mountain Fire Station

Project Engineer Richard G. Adams **Principal Engineer** Matthew S. Fogleman **Summary By** Scott W. Sawyer

Boring Number	Sample Number	Depth (feet)	MC[1] (%)	Soil Type[2]	Atterberg Limits[3]			Percent Passing No. 200 Sieve[4]	Moisture - Density (Corr.)[5]		CBR Value[6]	Other
					LL	PL	PI		Maximum Density (pcf)	Optimum Moisture (%)		
B-1	S-6	13.50 - 15.00	26.3	SM				33.1				
B-2	S-3	3.50 - 5.00	48.7	SM				41.3				
B-3	S-6	13.50 - 15.00	26.5	ML				52.4				

Notes: 1. ASTM D 2216, 2. ASTM D 2487, 3. ASTM D 4318, 4. ASTM D 1140, 5. See test reports for test method, 6. See test reports for test method

Definitions: MC: Moisture Content, Soil Type: USCS (Unified Soil Classification System), LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, CBR: California Bearing Ratio, OC: Organic Content (ASTM D 2974)