

**Geotechnical Engineering Report
ARRC Bald Mountain
Telecommunication Site
12 Miles East of Talkeetna, Alaska**

November 2014

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GEOTECHNICAL REPORT
ARRC BALD MOUNTAIN TELECOMMUNICATION SITE
12 MILES EAST OF TALKEETNA, ALASKA

1.0 INTRODUCTION

This report presents the results of surface reconnaissance, limited subsurface explorations, and geotechnical engineering studies conducted to develop geotechnical engineering recommendations for development of a new telecommunications site on Bald Mountain east of Talkeetna, Alaska. The purpose of the field work was to evaluate the near surface soil and rock conditions in the areas to receive a new tower and other support structures. To accomplish this, we visited the site and made observations of nearby rock outcrops, advanced shallow test holes, and conducted in-situ resistivity testing. Included in this report are descriptions of the site and project, surface observations, interpretation of the likely subsurface conditions, and conclusions and recommendations from our engineering studies.

Authorization to proceed with this work was received in the form of a signed agreement by Michael Pochop, P.E. of Hanson Alaska Professional Services, Inc. (Hanson) on September 26, 2014. The work was generally performed according to our September 5, 2014 proposal. Fieldwork was generally performed in accordance with our proposed “Alternative 2.”

2.0 SITE AND PROJECT DESCRIPTION

The proposed telecommunications site is situated on the west end of Bald Mountain approximately 12 miles east of Talkeetna, Alaska. According to the USGS Talkeetna Mountains B-6 63K Quadrangle map, this project is located in the southwest 1/4, Section 25, Township 26 North, Range 3 West, Seward Meridian. The site is sparsely vegetated with moss and lichen and the ground surface is generally covered with a rocky rubble typically made up of cobble and small boulder-sized particles. A primitive airstrip runs in an east-west direction along the mountaintop south and east of the proposed site. Access to the site was only available via helicopter, small fixed wing aircraft, or on foot (local trails may provide access via all-terrain vehicle). A vicinity map depicting the general project area is included as Figure 1.

At the time of our visit the site was developed with an existing communications tower and two sheds containing support equipment. The existing communications tower consists of an approximately 20 to 25-foot tall, four legged, tower structure. The existing tower appears to be resting on steel grade beams that are resting on the ground surface and appears to be attached to an adjacent connex shed. Rocks have been piled on the beams and two guy wires are attached to the tower and a deadman anchor (a section of a triangular lattice structure with rocks piled

around it) on the north side of the tower. Another existing communications facility is also located on the mountaintop several hundred feet east of the ARRC site.

We understand that the project generally consists of the construction of a 30-foot tall, self-supported, four legged tower (Microflect D4-30) that will likely be supported on a shallow concrete foundation. The project will also incorporate other hardware that includes an equipment shelter (which is already on-site), propane tanks, and other appurtenances. According to available drawings, the tower has a square footprint with approximately 12-foot spacing between each support leg. Ultimately, the foundations of the towers and other improvements will need to be designed to resist relatively significant overturning, shear, and vertical forces (tension and compression).

3.0 FIELD ACTIVITIES

Field activities consisted of drilling several test holes with a hand-operated drill, observing local rock outcrops, and conducting in-situ soil resistivity tests on October 2, 2014. Our activities were focused in the area of the proposed improvements indicated during our initial site visit with the design team on September 18, 2014 and the proposed tower location shown on a preliminary site sketch provided to us by SurvBase after their October 2014 sitework. The locations of the test holes, resistivity lines, and other pertinent exploration points were recorded with a handheld GPS unit that is generally considered accurate to within 20 feet horizontally and should therefore be considered approximate.

Four test holes, designated Test Holes TH-1 through TH-4, were advanced at the site during our site visit. Drilling services were provided by Denali Drilling of Anchorage, Alaska using an Atlas Copco Cobra Combi drill equipped with a star bit. The Cobra Combi drill is a gas operated, portable, handheld drill/breaker similar to a jack hammer but with added capability to drill shallow holes into soil or rock. Drill bits ranged in size from 1 to 2 inches in diameter. It should be noted that this unit is not capable of discrete soil or rock sampling and does not produce soil cuttings suitable for soil classification, collection, or laboratory testing. The intent of these drilling operations was to estimate the general nature and thickness of soil deposits at the site through observation of drill action and penetration rates.

Other activities at the site included general observations of the local terrain and rock exposures and electrical resistivity testing. Observations made at nearby rock outcrops generally consisted of identification of rock type, structure, and included measurements of the orientations of observed structural features. Electrical resistivity testing consisted of three lines situated near

the proposed tower site. Two of the lines were conducted nearly perpendicular to one another and centered within several feet of the proposed tower location marked by SurvBase. The third transect was run in an area between the airstrip and the proposed tower. The testing was performed in accordance with ASTM International (ASTM) G57-95A *Standard Test Method for Field Measurement of Soil Resistivity using the Wenner 4-Pin Method*. The tests were performed using a Nilsson Model 400 soil resistance meter and four pins generally spaced equally at 5 and 10-foot pin spacings for each test array. Additional tests were conducted at 2-foot pin spacings in lines R1 and R2 and 20-foot pin spacings at line R2. Tabulated results of the resistivity testing are presented in Section 4.0. The approximate locations of the proposed site features, rock outcrops, and resistivity testing are shown on Figure 2.

4.0 OBSERVED SURFACE CONDITIONS

General site conditions are shown graphically in Figure 2. The figure includes photographs of the site and general descriptions of the terrain and geology, as well as rock structure measurements as observed by our representative. It is important to note that the conditions below the ground surface as described on the figures and discussed in the report text are inferred from observations of the conditions exposed at the ground surface at the time of our site visit. Conditions below the ground surface could be different from what was observable during our site visit. As such, we recommend that a contingency be maintained to account for potentially different conditions and that we be retained to observe conditions during construction to verify that actual conditions are consistent with what is described in this report.

In general, the ground surface in the project area was largely comprised of gravelly soil and loose, rocky rubble with a thin mantle of moss or lichen. Based on these surface observations and observations made during drilling it appears that a gravelly soil may persist to depths ranging between about 5 to 7 feet bgs. This soil likely contains a relatively large amount of cobbles and boulders and a matrix of loose, silty material in places. Due to the nature of our explorations it is unknown whether some of this soil column is actually weathered bedrock. Since the drill was also near its assumed maximum depth limit at refusal the full depth of the soil and/or weathered bedrock is uncertain. The approximate location of our test holes is shown on Figure 2.

Large scale geologic maps of the project area indicate that at least a portion of Bald Mountain is made up of volcanic rocks that may consist of lava flows, tuffs, pyroclastic rocks, and hypabyssal intrusions. The maps show that the rock surrounding the Bald Mountain volcanics generally consist of granitic plutonic rocks. No bedrock is exposed on the ground surface in the

immediate project area and bedrock exposures at the site were limited to relatively small outcrops on the west slope of Bald Mountain below the proposed tower site. Larger outcrops were visible from the air on the south facing slope of Bald Mountain. Based on visual classification of rock fragments collected from the ground surface most of the rock rubble on top of Bald Mountain in the project area appears to consist of a gray to dark gray, fine-grained rock that may be basalt. Several granitic boulders were also observed on the ground surface near the existing tower site. Outcrop 1 was located approximately 160 feet southwest of the proposed new tower site. The rock visible at the outcrop generally appeared to match what may be basaltic rock observed in the mountaintop rubble. In general the rock appeared relatively massive with randomly oriented fractures spaced 1 to 2 feet apart. Other outcrops were observable further southwest and downhill from the tower site. Rock in these outcrops consisted of a relatively massive, black, crystalline, intrusive igneous rock. Outcrop 2, shown on Figure 2, shows the location of one of these igneous outcrops. Two primary joint sets were observable at Outcrop 2. One set dips at about 70 degrees to the northwest (typical measured dip direction of 310°). Joint spacings on the order of 18 inches to greater than 8 feet were estimated based on our observations of Outcrop 2. The second joint set dips toward the south (typical measured dip direction of 90°) at an angle of about 15 degrees. Exposures of this second joint set were limited and we estimated joint spacing in this set was greater than about 4 feet. In general, the rock observed in the outcrops appeared to be relatively resistive to weathering; however the “rubbly” appearance of the ground surface may indicate that the rock is somewhat susceptible to mechanical and/or chemical breakdown.

Electrical resistivity testing was conducted during our site visit. The locations of the three test transects are indicated on Figure 2. The estimated results are summarized in Table 1 below.

**TABLE 1
ELECTRICAL RESISTIVITY TEST RESULTS**

Location and Pin Spacing	Meter Reading (ohms)	Apparent Resistivity (ohm-m)
Line R1		
2-ft	28,000	107,240
5-ft	19,000	184,925
10-ft	7,100	135,965
Line R2		
2-ft	18,000	68,940
5-ft	22,000	210,650
10-ft	6,600	126,390
20-ft	1,800	68,940
Line R3		
5-ft	100,000	957,500
10-ft	8,000	153,200

5.0 ENGINEERING RECOMMENDATIONS

Design of the foundation for the proposed tower should consider the bearing capacity of the soils, expected settlements, uplift or overturning forces, lateral forces due to wind and seismic loads, possible frost conditions within the subsurface soils, and constructability issues. The following subsections provide our recommendations for foundation type and sizing, soil parameters, settlement, lateral earth pressures, drainage, utility trenches and excavation slopes, and structural fill and compaction. We understand that the preferred foundation type for the proposed tower is a shallow, cast-in-place concrete foundation that will be buried below the ground surface. We believe this foundation system is appropriate to the site conditions if the site is prepared in accordance with the recommendations provided in the following sections.

Note that the recommendations provided herein are based on surface observations and drilling techniques that did not allow for detailed characterization of the subsurface soil and rock. As such, we recommend that a qualified geotechnical engineer be retained to observe the actual soil and rock conditions during construction to confirm the assumptions used to formulate our recommendations and evaluate the appropriateness of the foundation design. We also recommend that a contingency be established to plan for potential design and scheduling impacts that may arise during construction due to site conditions that may differ from the conditions assumed in our analyses.

5.1 Structure Foundations

Based on our limited drilling and surface observations it appears that there is at least 5 to 7 feet of gravelly soil or rubble, some of which may be weathered bedrock, in the area of the proposed improvements. It is our opinion that these materials, below about 2 to 3 feet, will provide adequate support for the proposed tower. At the time of this report, we understand that the preferred foundation is a shallow concrete foundation that may consist of individual elements at each tower leg or a cast-in-place concrete pad with pedestals. We believe this foundation system is appropriate to the site conditions. We recommend that the foundation bear on undisturbed native soils or on compacted structural fill as detailed herein. Uplift loads would appear to control the size and depth of the foundation (i.e., a foundation sized to resist uplift and overturning loads should likewise be sufficient to resist the applied downward loads). Figure 8 presents typical foundation element details for a pedestal resting on a buried concrete block.

Excavation in the native soils beyond several feet below the ground surface may be difficult given the coarse, angular nature of the rocky materials. It is likely that excavations will require mechanical earthmoving equipment to complete. In preparing the bottom of excavation for foundation elements, we recommend working the material such that it is as flat as practicable with hand and mechanical equipment and compacting to a firm state with vibratory equipment. The surface should be firm, relatively uniform, not contain voids more than approximately 6 inches in diameter, and not able to deflect under compaction equipment. If additional fill is needed to fill surface voids or level the excavation surface, we recommend importing Selected Material Type A or B. Placement and compaction recommendations are provided in Section 5.9.

5.2 Foundation Embedment and Frost Considerations

Based on our experience, seasonal frost in the project vicinity may extend to depths of 10 to 12 feet below the ground surface. The native soil materials observed at the site generally consisted of boulder rubble or cobbles surrounded by a silty sand matrix. In our opinion, these materials will provide adequate frost protection beneath and around the tower foundation given the assumed large percentage of gravels/cobbles and the lack of available ground moisture, provided the proper drainage is maintained around foundation elements. Minor seasonal deflections may be experienced depending on the actual soil conditions encountered. We assume that it would be preferable to utilize onsite soils for structural backfill at the site. Prior to reuse we recommend that the gradation of the materials be confirmed for conformance to the material properties

assumed in our analyses. In general, a gravelly soil with less than about 10 to 15 percent fines should be appropriate.

If conditions prevent excavation for the minimum recommended embedment below the ground surface recommended in Section 5.4, additional fill can be placed above the foundation to effectively raise the elevation of the ground surface around the tower. The fill may be salvaged from the ground surface on site and placed as recommended in Section 5.9. We recommend limiting the amount of fill that can be added to no more than 2 feet above the ground surface. The fill section should cover the entire tower footprint and should extend beyond the footprint of the foundation a minimum of 10 feet in all directions. Fill should not be removed from the ground surface from any areas within 25 feet of the proposed tower or any other existing site features.

5.3 Uplift Resistance

In our experience, uplift loads generally control the size and depth of the foundation (i.e., a foundation sized to resist uplift and overturning loads should likewise be sufficient to resist the applied tower loads). Figure 3 presents typical foundation element details for a four-legged tower resting on a shallow concrete foundation. The uplift resistance of the foundation can be estimated by summing the dead weight of the footing, the weight of the soil within a zone described by a vertical surface extending upward from the horizontal limits of the footing, and the shearing resistance of the soil across this surface. Additional resistance gained from friction between the foundation and embedment soils should be negligible in comparison, and in our opinion should not be used in estimating uplift resistance for the foundations. Assuming a gravelly soil with less than about 10 to 15 percent fines is used to backfill around and above the footing using the placement and compaction requirements outlined in Section 5.9, the density of the soil resisting uplift should be at approximately 125 pounds per cubic foot (pcf). Lastly, shearing resistance can be calculated using a frictional resistance of about 35 degrees for densely compacted gravel materials. Using these parameters, we created a graph representing ultimate uplift resistance shown on Figure 8. This graph represents the calculated ultimate (no factor of safety included) uplift resistance over a range of footing area and foundation depths, but does not include the weight of the footing in the calculation.

5.4 Bearing Capacity

After sizing the footing, the foundation should be checked for allowable bearing pressure. We recommend that the footing be designed using an allowable bearing capacity of 4,500 pounds per

square foot (psf) for a footing embedded a minimum of 5 feet bgs and bearing on compacted structural fill or native soils. This bearing value may be increased by 1/3 for short-term wind and seismic loading. Before constructing forms for the footings, we recommend that a qualified geotechnical engineer be retained to probe the bottom of the excavation to look for loose or disturbed zones beneath the foundation. If disturbed or loosened soils are encountered by probing, they should be replaced and re-compacted to a dense state.

Moments created by the structure may create eccentric loads on the foundation element. We recommend using an effective footing width calculated by subtracting two times the eccentricity in each direction, in calculations for allowable loading. The above bearing capacity is applicable only if the eccentricity is less than $\frac{1}{6}$ the width of the foundation element. If the eccentricity exceeds $\frac{1}{6}$ the foundation width, we recommend increasing the footing width until the eccentricity criteria is met. If this is not economically feasible, more complex analysis of bearing capacity may be required.

5.5 Foundation Settlements

The magnitudes of the settlements that will develop at the communications tower foundation are dependent upon the applied loads, the density of the support material, and the care with which structural fills are placed and compacted. Compaction recommendations and procedures described in Section 5.9 should be strictly followed. We estimate that total maximum settlements will be about 1 inch or less with differential settlements being about 1/2 of the total settlements over the width of the footing. The greatest amount of settlement should occur during construction, essentially as fast as the loads are applied, such that long term settlements should be relatively small and well within tolerable limits.

5.6 Lateral Earth Pressures

Foundation structures below the ground surface should be designed to resist horizontal earth pressures. The magnitude of the pressure is dependent on the method of backfill placement, the type of backfill material, drainage provisions, and whether the vertical structure is permitted to deflect after or during placement of backfill.

If the structure is allowed to deflect laterally or rotate an amount equal to about 0.001 times the height of the structure, an active earth pressure condition under static loading would prevail and an ultimate equivalent fluid weight of 34 pcf is recommended for design of vertical structures. For rigid structures that are restrained from deflecting at the top, an at-rest earth pressure condition would prevail and an equivalent fluid weight of 53 pcf is recommended. To simulate

seismic loading (from soils adjacent to the foundation) a rectangular pressure prism with a magnitude of 13 psf per foot of height should be applied to the footing. Note that these values reflect free-draining, compact, granular backfill and no hydrostatic forces acting on the structure.

Lateral forces from wind or seismic loading may be resisted by passive earth pressures against the sides of footings. These resisting pressures can be estimated using an equivalent fluid weight of 250 pcf. This value includes a factor of safety of at least 1.5 on the full passive earth pressure. Along with this value, it is also recommended that backfill around the footings be compacted per Section 5.9.

Lateral resistance may also be developed in friction against sliding along the base of the foundation placed on grade. These forces may be computed using a friction coefficient of 0.4 between concrete and soil.

5.7 Drainage

Groundwater is not expected to be encountered during construction activities. However, there is the potential for surface runoff to infiltrate open excavations during construction. Therefore, the contractor should be responsible for maintaining site grade to prevent surface water from entering excavations during periods of high rain or snow melting. The contractor should be prepared to dewater the excavation with sumps and pumps during construction as needed to remove water that collects in the excavation.

We recommend that the area around the tower be contoured to provide positive surface drainage away from the structure and off the site. Contouring the site for positive drainage should minimize ponding of surface waters during periods of rainfall or rapid snow melting. This in turn will limit moisture contents in the site soils, which should reduce the potential for frost wedging.

5.8 Excavation Slopes and Utility Trenches

An excavation will be needed to construct the tower foundation and buried cables may be needed to tie the new tower into support facilities. Excavations for utilities and tower foundations should extend down to expose firm, consistent, native mineral soils. The bedding and structural fill material around utility lines and the foundation should be densely compacted to support and hold them firmly in place. Soil slopes in these materials will tend to stand steeply initially, but as they dry they will soften and slump in time to their natural angle of repose, which for planning purposes is estimated at about 1.5 horizontal to 1 vertical. If excavations are allowed to stay

open to inclement weather, the soil slopes could slough to shallower angles. The slope and excavation bottom conditions should be made the responsibility of the contractor who will be present on a day to day basis and can adjust efforts to obtain the needed stability, and meet the applicable Alaska and Federal (OSHA) safety regulations.

Backfill for the foundation excavations should consist of compacted structural fill as described in Section 5.9. Above the bedding in underground utilities, excavated native, granular soils should be used to backfill the trenches, to the extent practicable, up to the original ground surface.

5.9 Structural Fill

Backfill will be required under and around the tower foundation. We anticipate that the existing soils will consist of sand, gravel, and cobbles with about 10 to 15 percent fines. Minor amounts of soil fills may need to be imported into the site for bedding cables or levelling foundation pads and it should consist of clean, granular soil free of organic material to provide drainage and frost protection. Selected Material Type A or B as Alaska Department of Transportation & Public Facilities meets these requirements. Gradation requirements for Type II/IIA structural fill are provided in Figure 5.

The existing native soils observed are expected to be predominantly granular but may not meet the gradation requirements for Selected Material Type A or B structural fill, as shown on Figure 5. Existing soils that contain more than about 10 to 15 percent fines should not be used as structural fill below or around foundation elements but may be used as unclassified fill to contour the site or as utility trench backfill under areas that will not support structures. Likewise, materials that contain organic or deleterious matter should not be used in structural applications.

Selected Material Types A and B placed as backfill around, above, and below foundation elements or around buried utilities should be placed in lifts not to exceed 10 to 12 inches loose thickness and compacted to 95 percent of the maximum density as determined by the Modified Proctor compaction procedure (ASTM D 1557). During fill placement, we recommend that large cobbles or boulders with dimensions in excess of 8 inches be removed. Because of the coarse nature of the existing fills it will be difficult to use moisture/density control to compact these materials. As such, we recommend using subjective compaction criteria such that vibratory equipment is used to compact the materials to a visually firm, consistent state where the fill surface contains minimal voids and does not deflect under compaction equipment loading.

Bulking of backfill into the excavations should be discouraged as this can cause voids and lead to large future surface settlements. To avoid damage to foundation elements, fill material within 18 inches of the pedestal and/or block foundation should be placed in layers not to exceed six inches loose thickness and compacted with hand operated equipment. Heavy equipment should not be used as it could cause increased lateral pressures and possibly damage the structure.

6.0 CLOSURE/LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions and recommendations contained in this report are based on information provided from the observed site conditions and other conditions described herein. The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist. It is assumed that the observed surface soil and rock conditions are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If, during construction, conditions different from those encountered in these explorations are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to review those portions of the plans and specifications pertaining to earthwork and foundations to determine if they are consistent with our recommendations.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or excavating test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachments in Appendix A *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.

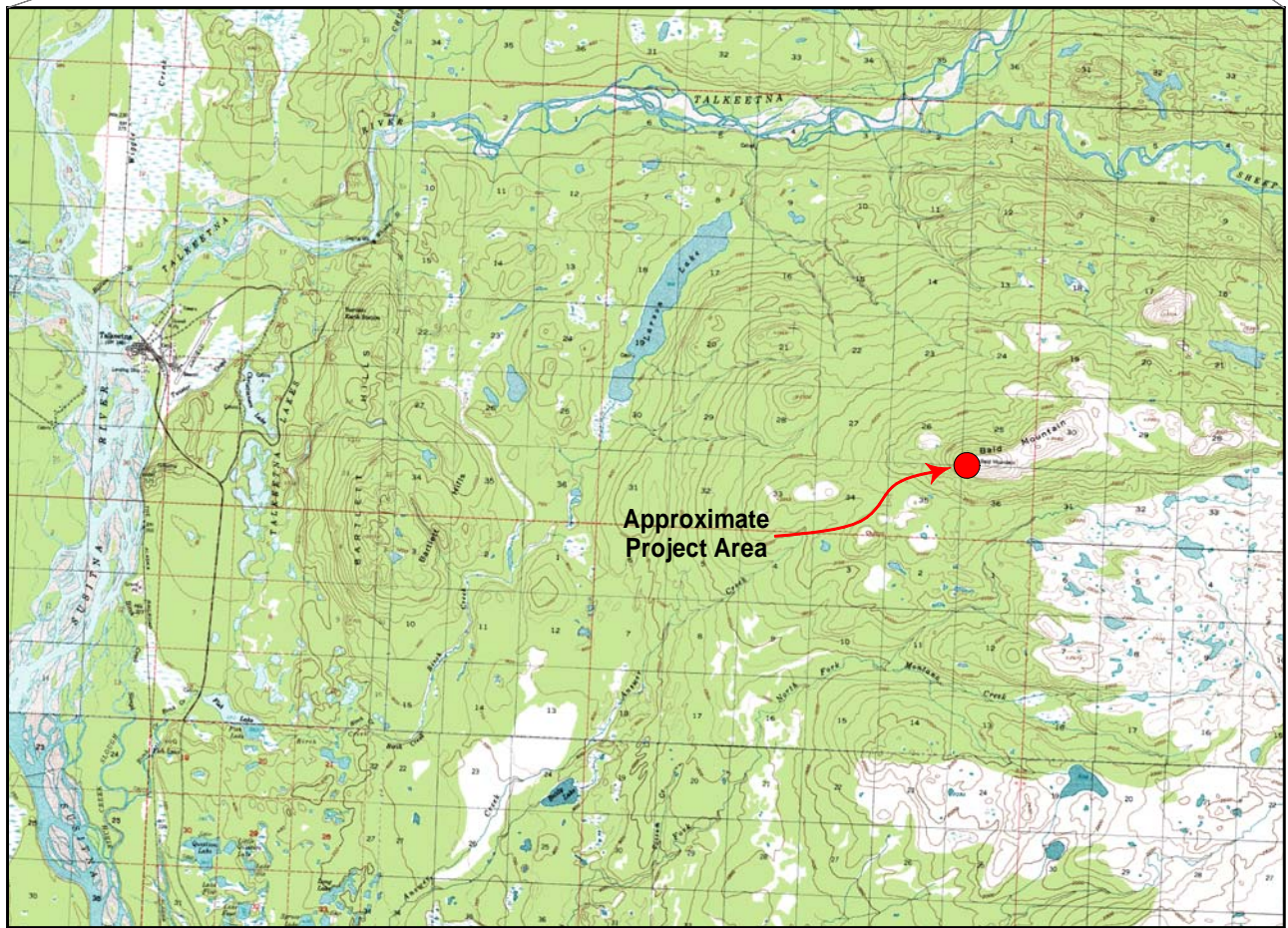
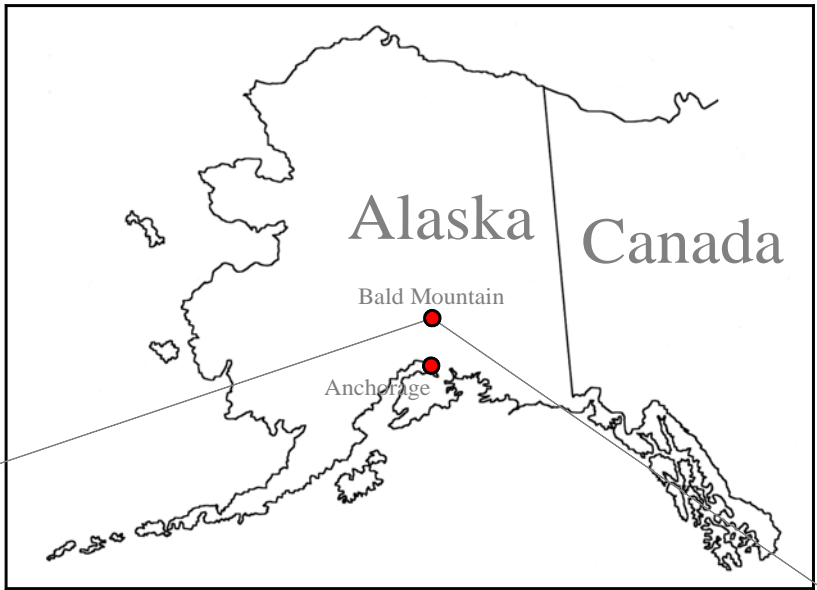
SHANNON & WILSON, INC.

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for Ryan Collins
Senior Geotechnical Staff

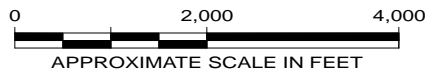
RDC:KLB



Kyle Brennan, P.E.
Vice President



Map adapted from All Topo Maps USGS Talkeetna A-1 and B-1 63K, and Talkeetna Mountains A-6 and B-6 63K Quadrangle maps



ARRC Bald Mountain Telecommunication Site
12 Miles East of Talkeetna, Alaska

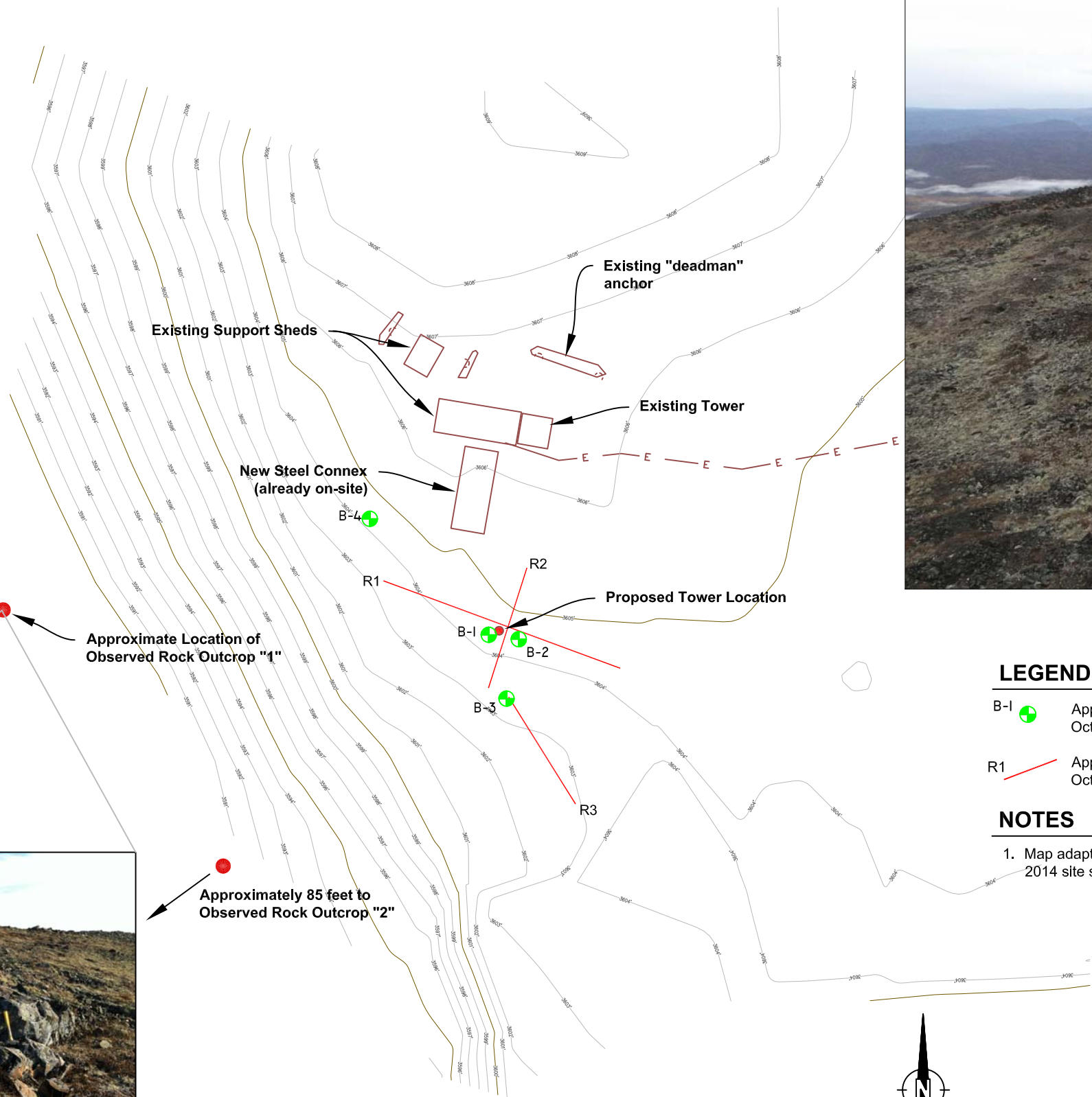
VICINITY MAP

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FIG. 1



Photograph of site. Looking north.



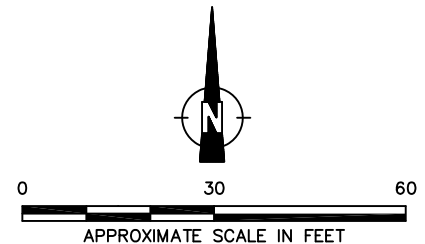
Photograph of possible exposed bedrock (outcrop "2") near the proposed tower location. Looking northeast.

LEGEND

- B-1 Approximate Location of Cobra Drill Boring B-1, Advanced by Shannon & Wilson, October 2014
- R1 Approximate Location of Resistivity Transect R1, conducted by Shannon & Wilson, October 2014

NOTES

1. Map adapted from drawing provided by Hanson Alaska and based on October 2, 2014 site survey performed by SurvBase, LLC.



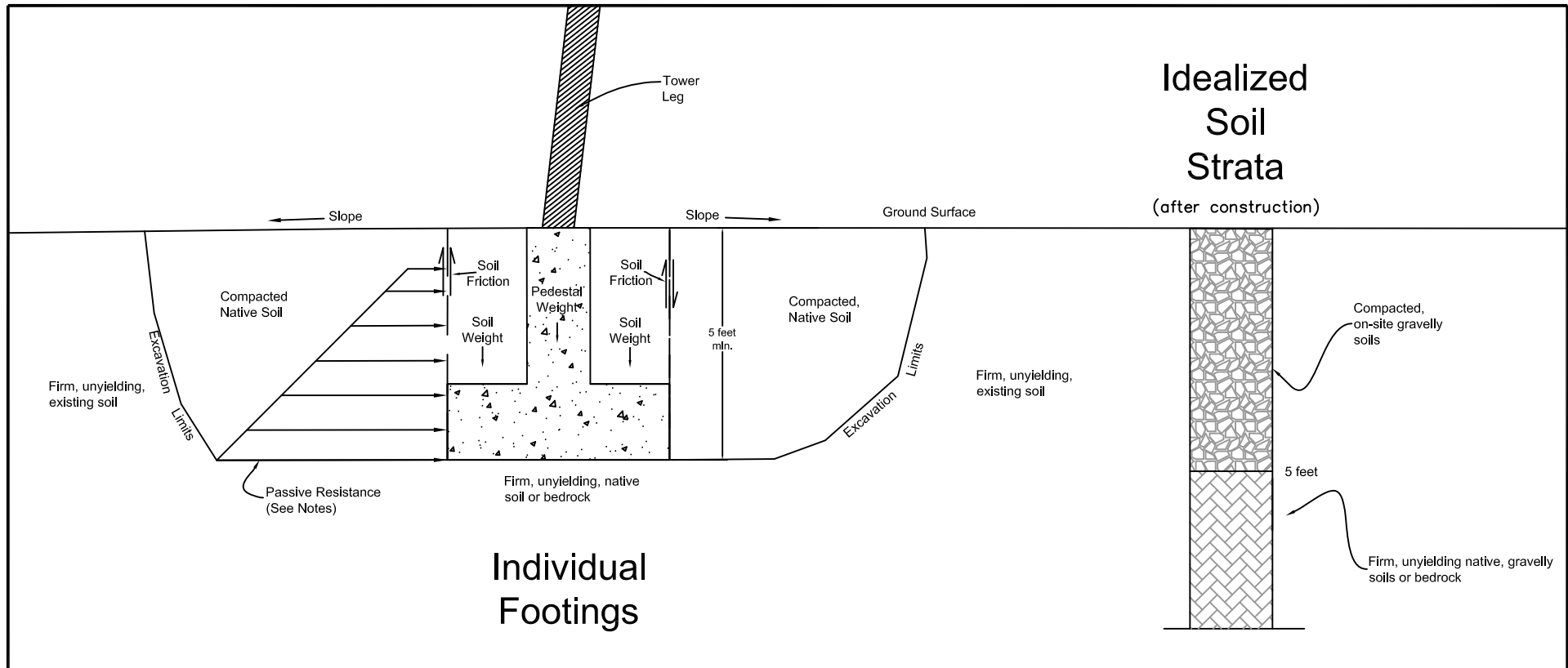
ARRC Bald Mountain Telecommunication Site
Talkeetna, Alaska

SITE PLAN

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FIG. 2



Individual Footings

DRAWING NOT TO SCALE

NOTES

1. Passive resistance to lateral forces can be estimated by multiplying the passive equivalent fluid unit weight for the soil (250 pounds per cubic foot for compact granular fill) by the depth of embedment. The resultant pressure prism should only be applied to subsurface foundation elements in direct contact with soil, hence the triangular prism for the foundation elements.

2. The weight of the soil column above the anchor can be approximated by multiplying the approximate density of the soil (125 pounds per cubic foot) by the volume of soil above the anchor. The frictional resistance to uplift around the soil column can be approximated using a friction angle of 35 degrees along the vertical shear plane (all four sides of the soil column). See Figure 4 for uplift resistance curves. Unit weight and friction angle assume densely compacted granular fill with water table at least 5 feet below bottom of foundation.

3. Neglect passive resistance within 2 feet of ground surface.

4. OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as they are at the project site on a day to day basis and are aware of changing conditions.

ARRC Bald Mountain Telecommunication Site
Talkeetna Area, Alaska

FOUNDATION ELEMENTS DETAIL

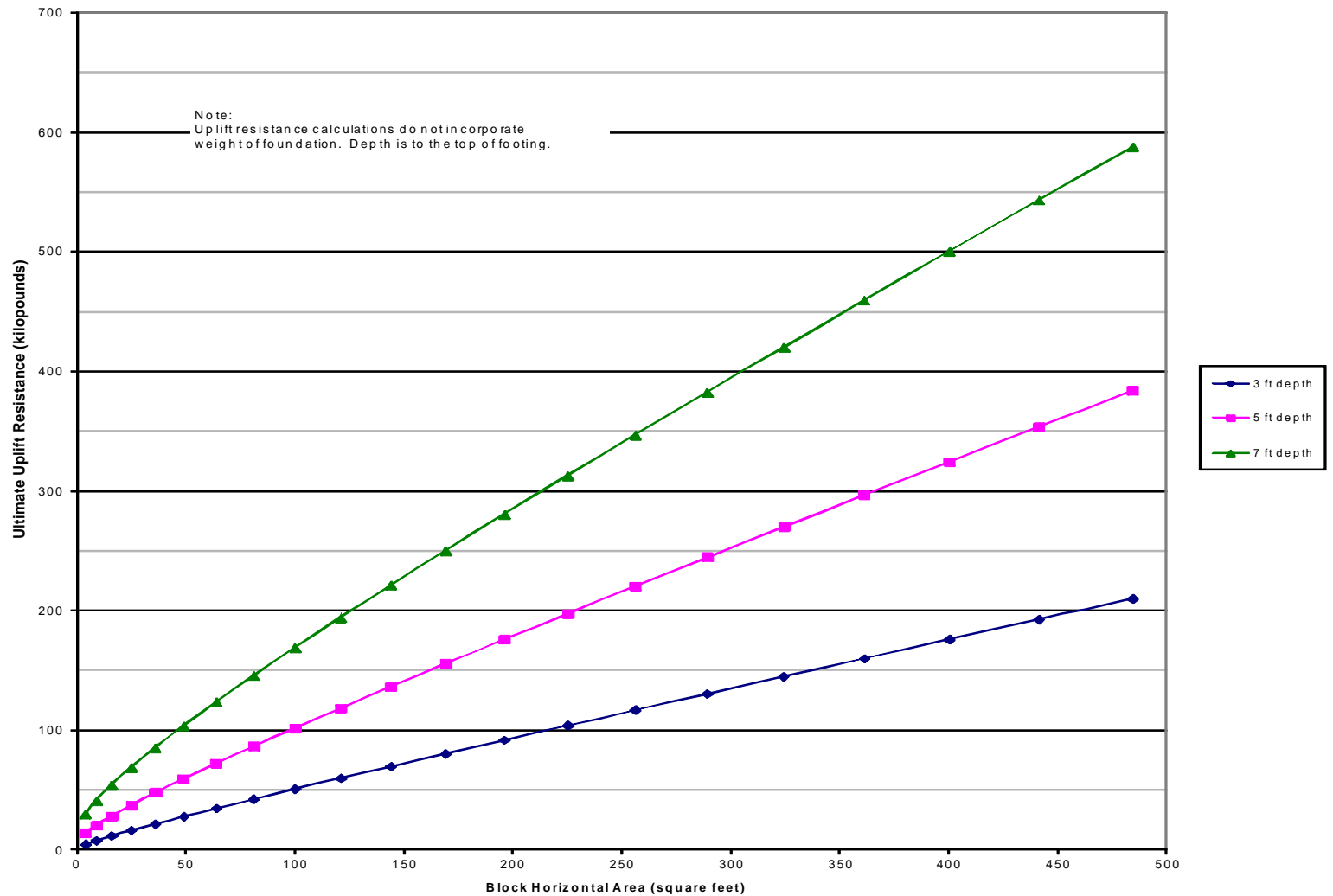
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FIG. 3



Note: Uplift resistance is stated in terms of ultimate capacity for the case where the foundation element is lifted vertically. Assume the foundation element is embedded in compact gravel fill (unit weight at least 125 pcf and friction angle at least 35 degrees). Size the foundation footprint to provide an appropriate factor of safety (Shannon & Wilson recommends FS = 3 to limit deflections). Footing length should not exceed 1.5 times width.

ARRC Bald Mountain Telecommunication Site
Talkeetna Area, Alaska

**UPLIFT RESISTANCE VS.
FOOTING SIZE**

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GRADATION AND DURABILITY REQUIREMENTS

After: Alaska Department of Transportation
Standard Specifications for Highway Construction

Selected Material Type A

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
English	Metric	
No. 4	4.75 mm	20 - 55
No. 200	0.075 mm	6 Max. on minus 3-in. portion
Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plasticity index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T 11.		

Selected Material Type B

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
English	Metric	
No. 200	0.075 mm	10 Max. on minus 3-in. portion
Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plasticity index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T 11.		

Coarse Aggregate Durability

Retained on #4 Sieve

Test Type	Percent Loss
L.A. Abrasion	45 - 50 max. *
Sulfate Soundness	9 max.

* Asphalt and Surface Course = 45% max
Base Course = 50% max

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GRADATION REQUIREMENTS

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FIG. 5

APPENDIX A

**IMPORTANT INFORMATION ABOUT YOUR
GEOTECHNICAL/ENVIRONMENTAL REPORT**



Date: November 2014
To: Hanson Alaska
Re: ARRC Bald Mountain Telecommunication
Site, Talkeetna Area, Alaska

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland