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March 19, 2021

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Supplemental Report of Geotechnical Exploration and Slope Stability Analyses
Allen Street Slope Failure – South Section
Sylva, North Carolina
KEG Project No. JA20-4021-02

Mr. Scott:

Kessel Engineering Group, PLLC (KEG) is pleased to present this supplemental report of geotechnical exploration and slope stability analyses of the southern slope failure section located along a portion of Allen Street in Sylva, North Carolina. The following executive summary provides an overview of the significant geotechnical issues for this project. The summary should not be relied upon exclusively. Specific geotechnical design parameters with a detailed discussion of applications and limitations are provided in the attached report.

EXECUTIVE SUMMARY

The project site is located on Allen Street immediately downhill of the properties listed as 2 Bobwhite Lane and 11 Bobwhite Lane. Two areas of tension cracking have developed in the Allen Street pavement sections in the second half of 2020. A large bowl-shaped slope failure is located within the adjacent property slope located between Allen Street and Chipper Curve Road (owned by others). This failure exhibits a scarp on the order of 10 to 15 feet high approximately mid-slope and extends downhill to Chipper Curve Road, with seepage observed at the toe of the slope failure area at the Chipper Curve Road elevation.

A total of five soil test borings (B-10 to B-14) were performed at the site and typically encountered surficial fill soils underlain by residual soils and partially weathered rock (PWR). The existing fill soils typically consisted of loose silty sands. Residual soils consisted of loose to very dense silty sands and soft to stiff sandy silts. PWR was encountered in two of the five borings. Groundwater was encountered in each of the soil test borings performed during this exploration. Laboratory testing on undisturbed samples and split spoon samples was performed and included consolidated undrained (CU) triaxial shear testing, grain size distribution testing, natural moisture contents, and Atterberg limits testing.

Slope stability analyses indicate that the current observed tension cracking is likely due to localized slope instability associated with fill slope construction supporting Allen Street in these areas. Remedial measures for the existing fill slope should include the installation of soil nails or ground anchors to permanently stabilize the already mobilized materials. However, additional movement of the adjacent property failure area could impact future serviceability of Allen Street in this area. Remedial repair of the adjacent property failure area could likely include a combination of slope dewatering, slope regrading, and the installation of soil nailing or ground anchoring systems.

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We appreciate the opportunity to offer our professional geotechnical services on this project. Please contact us with questions regarding this report or if we may be of further assistance.

Sincerely,
KESSEL ENGINEERING GROUP, PLLC (NC Firm License No. P-0420)

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SCOPE OF EXPLORATION

The purpose of this exploration was to determine general subsurface conditions in the tension crack areas described below in order to develop general geotechnical recommendations for future slope remediation. This geotechnical exploration was performed in general accordance with our *Proposal for Supplemental Geotechnical Exploration*, KEG Proposal No. PA20-3389-02.

PROJECT INFORMATION AND SITE OBSERVATIONS

Previous information for the nearby slope failure was gathered as documented in our *Report of Geotechnical Exploration and Slope Stability Analyses*, KEG Project No. JA20-4021-01, dated July 14, 2020. Updated information was provided during email and telephone correspondences between Mr. Scott and our Mr. Ian Johnson, P.E., as well as during multiple visits to the project site by Mr. Johnson in November and December 2020. We have also been provided with the following digital document: *Topographic Survey prepared for: Town of Sylva*, by Alliance Land Surveying, P.C., dated 01/20/2021, and showing the approximate property boundaries, existing topographic contours, and reissued 03/04/201 with soil test boring locations.

The project site is located within Allen Street in Sylva, North Carolina (see Figure 1). In addition to the primary failure area documented in the aforementioned *Report*, two additional tension crack areas have formed within the Allen Street asphalt section in the second half of 2020. These two areas are located to the southwest of the primary Allen Street failure area noted above and below/southeast of the residences located at 2 Bobwhite Lane and 11 Bobwhite Lane. The tension cracks are on the order of 15 to 25 feet in length and comprised of multiple cracks forming roughly parallel to the slope face (see Photos 1, 2). Significant vertical displacements were not observed at the time of our site visits. The fill slopes supporting Allen Street in these areas are on the order of 6 to 10 feet high and have inclinations varying from approximately 1.3H:1V to 1.8H:1V.

The adjacent property slope located below/southeast of Allen Street is bounded to the southeast by Chipper Curve Road. The property comprising this slope (owned by others) is heavily-vegetated with underbrush/creeping vines. Chipper Curve Road is a two-lane NCDOT maintained road, and Allen Street is a single-lane paved road maintained by the Town of Sylva. A water line owned by TWSA is located within Allen Street.

A large bowl-shaped slope failure is located within the adjacent property slope located between Allen Street and Chipper Curve Road. This failure exhibits a scarp on the order of 10 to 15 feet high approximately mid-slope and extends downhill to Chipper Curve Road, where the lower portion is retained by concrete Jersey barriers adjacent the roadway (see Photo 3). Grades above the scarp are as shallow as approximately 9H:1V in some locations. Some seepage was observed at the lower portion of the slope failure area during our multiple visits to the project site.

Based on our review of the provided topographic survey, the slope in this general area has an overall height of approximately 38 to 46 feet from Chipper Curve Road to Allen Street, with an overall inclination of approximately 2.3H:1V (horizontal:vertical) to 3.0H:1V. Sections of the slope on the northeast and southwest sides of the bowl-shaped slope failure area are inclined approximately 1.7H:1V to 2H:1V adjacent Chipper Curve Road for approximately 20 feet in height, after which the slope inclination shallows to approximately 4H:1V until the Allen Street fill slope is encountered. We understand from anecdotal evidence that slope failures have occurred along sections of Allen Street for several decades.

Photo 1: Tension cracking within Allen Street, approx. 30 feet NE of Bobwhite Lane, vicinity of B-11, photo facing southwest. Photo dated 11/25/2020.

Photo 2: Tension cracking within Allen Street, approx. 180 feet SW of Bobwhite Lane, vicinity of B-14, photo facing northeast. Photo dated 11/25/2020.

Photo 3: Base of adjacent property failure area, viewed from Chipper Curve Road, photo facing northwest. Photo dated 11/25/2020.

SITE GEOLOGY

The project site is located in the Blue Ridge Physiographic Province. The bedrock in this region is a complex crystalline formation that has been faulted and contorted by past tectonic movements. The rock has weathered to residual soils which form the mantle for the hillsides and hilltops. The typical residual soil profile in areas not disturbed by erosion or grading consists of clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands.

The boundary between soil and rock is not sharply defined and there is often a transitional zone, termed "partially weathered rock" overlying the parent bedrock. Partially weathered rock (PWR) is defined, for engineering purposes, as residual material with a standard penetration resistance in excess of 100 blows per foot. Weathering is facilitated by fractures, joints, and the presence of less resistant rock types. Consequently, the profile of the partially weathered rock is irregular even over short horizontal distances. Also, it is not unusual to find lenses and boulders of hard rock and/or zones of partially weathered rock within the soil mantle, well above the general bedrock level.

FIELD EXPLORATION

The site was explored by performing five (5) soil test borings (B-10 to B-14) at the approximate locations as indicated on the attached Field Exploration Plan (see Figure 2). Borings were located in the field by our Mr. Ian Johnson, P.E. Borings were located within Allen Street. Boring B-12 encountered shallow auger refusal and was offset as boring B-12A. Data from these borings was combined as boring B-12/12A.

The soil test borings were performed by utilizing a Mobile B-57 truck-mounted drill rig. Borings were advanced by mechanically twisting a continuous flight steel auger into the soil. Soil sampling and penetration testing were performed in general accordance with ASTM D 1586. At assigned intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final 12 inches was recorded and is designated the “penetration resistance.” The penetration resistance, once properly evaluated, is an index to the strength of the soil. Representative portions of the soil samples, thus obtained, were placed in plastic bags and transported to the laboratory. In the laboratory, the samples were examined to verify the field classifications made by the driller. The boreholes were backfilled with the soil cuttings brought to the surface by the augers. Asphalt was patched with a cold patch asphalt mixture. In addition to bag samples, undisturbed samples were also gathered in order to perform additional laboratory testing.

Soil descriptions and penetration resistances are tabulated on the attached Boring Logs. Top of boring elevations were estimated by referencing the topographic information from the provided plan. The boring locations and elevations shown in the appendix should be considered approximate.

LABORATORY TESTING

A total of three (3) undisturbed specimens (Shelby tubes) were gathered from the project site. Triaxial shear testing (consolidated undrained conditions) was performed on two (2) selected undisturbed specimens by our subcontractor testing agency. In-house laboratory testing was performed on a series of four (4) selected split spoon samples to help determine soil properties. Natural moisture content (ASTM D2216) and particle size distribution (ASTM D6913) testing was performed on each selected sample. Atterberg limits testing (ASTM D4318) was also performed on select samples. A summary of laboratory testing data is provided below in Table 1. Detailed results are provided in the appendix of this report.

TABLE 1
LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft)	Sample Type* (ft)	Sample Strata	Effective Friction Angle, ϕ' (degrees)	Effective Cohesion, C', (psf)	Natural Moisture Content (%)	Percent Passing No. 200 Sieve (%)	Atterberg Limits* (LL/PL/PI)	USCS Classi-ficiation**
B-10	3 to 5	SS	Residuum	-	-	25.7	43.4	NP	SM
B-10	5 to 7	UD	Residuum	36.4	120	15.9	-	NP	SM
B-12	3.5 to 5	SS	Residuum	-	-	20.2	57.7	34 / 27 / 7	ML
B-13	10 to 12	UD	Residuum	30.0	120	29.2	-	NP	SM
B-13	13.5 to 15	SS	Residuum	-	-	28.8	23.1	NP	SM
B-13	28.5 to 30	SS	Residuum	-	-	34.5	57.1	38 / 30 / 8	ML

- Test not performed
* UD = Undisturbed sample (Shelby Tube); SS = Split-spoon sample
** LL = Liquid Limits; PL = Plastic Limits; PI = Plasticity Index; NP = non-plastic
*** SM = Silty Sand; ML = sandy silt

SUBSURFACE CONDITIONS

Soil test borings performed during this exploration were performed through the existing asphalt/stonebase section. Asphalt pavements ranged from approximately 4 to 7 inches thick and were underlain by aggregate base course (ABC stonebase) varying from approximately 8 to 18 inches thick.

Borings B-11, B-13, and B-14 encountered existing fill below the asphalt/stonebase section. The existing fill extended to depths of approximately 5.5 to 7 feet below the existing ground surface and typically consisted of very loose to loose, silty, fine to coarse sands (SM) with some gravel. Crushed asphalt and glass was encountered in some of the existing fill. Existing fill was noted as being slightly moist, and contained trace to moderate mica content. Existing fill was underlain by residual soils at these borings. The remaining borings (B-10 and B-12/12A encountered residual soils directly below the asphalt/stonebase section.

Residual soils encountered by these borings typically consisted of soft to stiff silts (ML) within the upper 3 to 17 feet underlain by loose to very dense silty sands (SM). Residual soils contained varying amounts of mica. Loose sands (N-values between 5 and 10) were encountered in borings B-10 to B-13 to depths of 17 to 37 feet. Partially weathered rock (PWR) was encountered in borings B-10 and B-14 at depths of approximately 37 and 27 feet below the existing ground surface, respectively. Borings were extended to their assigned termination depths of 40 to 45 feet below the existing ground surface

Groundwater measurements were taken by the drillers at the time of boring, and groundwater was encountered in each of the borings. Piezometer standpipes were not installed due to the active status of the roadway; however, end-of-day groundwater measurements were taken in four of the five boring locations. Groundwater levels may fluctuate several feet with seasonal and rainfall variations. Normally, the highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall. A summary of subsurface conditions as well as groundwater conditions encountered by the soil test borings is provided below in Table 2.

TABLE 2
SUMMARY OF SUBSURFACE CONDITIONS ENCOUNTERED BY SOIL TEST BORINGS
(MEASURED IN FEET BELOW THE EXISTING GROUND SURFACE)

Soil Test Boring No.*	**Asphalt & ABC Stone** (feet)	**Existing Fill**		**Residuum**					**Ground-water Depths** (feet)	**Termination Depth** (feet)
		V. Loose to Loose SANDS (feet)	Firm SANDS (feet)	Soft SILTS (feet)	Firm to Stiff SILTS (feet)	Loose SANDS (feet)	Firm to Very Dense SANDS (feet)	PWR (feet)		
B-10	0 to 1		-	-	1 to 3	3 to 17	17 to 37	37 to 40	37.4 / -	40
B-11	0 to 1.5	1.5 to 7	-	7 to 12	-	12 to 32	32 to 40	-	34.8 / 29.7	40
B-12/ 12A	0 to 1.3	-	-	-	1.3 to 5.5	5.5 to 37	37 to 45	-	30.3 / 20.7	45
B-13	0 to 1.5	1.5 to 5.5	-	-	27 to 32	5.5 to 27	32 to 40	-	28.8 / 19.5	40
B-14	0 to 2	3 to 6.8	2 to 3	6.8 to 12	12 to 17	-	17 to 27, 32 to 37	27 to 32, 37 to 40	23.3 / 20.7	40

- Not encountered in soil test boring / not applicable.
*See Figure 2 for approximate locations.
** Groundwater depths shown at time of boring and end of day (except B-10).

The above descriptions provide a summary of the subsurface conditions encountered by the soil test borings. The attached logs contain information recorded at each boring location. The logs represent our interpretation of the field logs based on examination of the field samples. The lines designating the interfaces between various strata represent approximate boundaries. The transition between strata may be gradual. It should be noted the soil conditions may vary between boring locations, and that elevations should be considered approximate.

ANALYSES AND DESIGN RECOMMENDATIONS

Tension Crack Locations

Slope stability analyses were conducted by Spencer's limit equilibrium method using SLOPE/W software developed by Geo-Slope International. Slope stability analyses were utilized to approximate the development of the observed tension crack locations and to develop general geotechnical recommendations for slope stabilization and repair. The soil strength parameters used in the analyses were estimated based on the results of laboratory testing and our experience on similar projects. Our analyses utilized groundwater phreatic surface conditions observed during our exploration and noted in Table 2, as well as the observance of seepage water exiting at the toe of the slope at Chipper Curve Road.

The following values were utilized for existing fill soils: effective cohesion c' = 10 psf, effective friction angle φ' = 27 degrees, total unit weight, γ_t = 125 pcf. The following values were utilized for soft/loose residual soils: effective cohesion c' = 10 psf, effective friction angle φ' = 30 degrees, total unit weight, γ_t = 110 pcf. The following values were utilized for firm or better residual soils: effective cohesion c' = 100 psf, effective friction angle φ' = 32 degrees, total unit weight, γ_t = 120 pcf. The following values were utilized for PWR: effective cohesion c' = 250 psf, effective friction angle φ' = 40 degrees, total unit weight, γ_t = 135 pcf. Shear strength parameters utilized for PWR were estimated based on our experience with similar materials.

Based on our experience, the most likely type of slope failure for these conditions would be a circular failure arc. The minimum factor of safety (FS) generally recommended for long-term slope stability is 1.3 or greater. Generally, a $FS \geq 1.5$ is required for critical slopes retaining infrastructure (such as roadways). A factor of safety $FS \leq 1.0$ is indicative of failure.

Based on our analyses of the slope cross-sections at the observed tension cracking in Allen Street, it is our opinion that the current tension cracking observed approximately 30 feet northeast of Bobwhite Lane (reference Photo 1) and approximately 180 feet southwest of Bobwhite Lane (reference Photo 2) is at this time due to localized instability within the fill slope. Slope stability analyses at these areas indicate a factor of safety $FS \approx 1.0$, indicative of failure or near-failure. It is our experience that unreinforced, permanent fill slopes (such as those supporting Allen Street in these areas) that are benched into a suitable foundation and adequately compacted should be constructed at 2H:1V slope configurations, or flatter, in order to maintain long-term stability and to have an industry standard factor of safety with respect to global slope stability. The existing fill slopes in these areas have inclinations as steep as 1.3H:1V to 1.8H:1V.

Because of the presence of weaker near-surface residual soils (N-values < 7) and the presence of the existing adjacent property slope failure, the use of conventional means to stabilize the existing fill slope localized failures is not recommended. Conventional means such as site retaining walls or rock buttresses would increase loading on these weaker residual soils and could accelerate or trigger additional movement of the adjacent property slope failure area.

It is therefore our opinion that localized repair of these fill slopes should likely include incorporation of the soil nail / ground anchor system previously recommended in our aforementioned *Report*. Soil nails are used to create a mass of soil which then behaves as a gravity system to resist lateral earth pressures. The soil nail system should take the global stability of the site and potential traffic loading into account. Soil nailing will have the advantage of being able to be constructed without removing large sections of the existing roadway; however, care must be taken to avoid existing utilities during installation thereof. Easements may be required where soil nails extend beyond adjacent property lines.

As noted in our aforementioned *Report*, given the existing fill slope inclinations, we anticipate that soil nail systems can be installed such that a shotcrete facing is not required, and vegetation can be grown on the slope face. A steel Tecco mesh facing may be required. The soil nail specialty contractor should determine a final design inclination and whether a vegetative facing is feasible given the site and soil conditions.

Soil nail / ground anchor designs must take the global stability of the site into account. We recommend a minimum global stability factor of safety of FS = 1.5 for the proposed soil nail system. Soil nail systems should be designed by a Professional Engineer licensed in the state of North Carolina and constructed by a specialty contractor specializing in soil nailing.

After remedial repair work associated with soil nail construction has been completed, the roadway stonebase and asphalt sections at Allen Street could then be reconstructed. Shallow ground improvement measures (such as utilization of biaxial geogrid for stabilization) may be required where marginal existing fill soils were encountered within approximately 3 to 4 feet of final pavement elevation.

As noted above, at this time it is our opinion that the tension cracking observed in this portion of Allen Street is related to localized fill slope instability. However, the development of additional tension cracking or vertical displacement of tension cracking within Allen Street in this area should be reported to KEG immediately for further evaluation, as this may be indicative that the adjacent property failure area has extended to Allen Street in this area.

Adjacent Property Failure Area

Subsurface exploration of the adjacent property failure area bounded by Allen Street and Chipper Curve Road was beyond the scope of this report; however, borings located roughly uphill of the main scarp location (B-11, B-12/12-A, and B-13) encountered loose residual soils to depths of approximately 27 to 37 feet in this area, with groundwater measurements ranging from approximately 20 to 30 feet deep. It is likely that similar subsurface conditions are present within the adjacent property failure area, and that the failure has been initiated by a combination of high groundwater and a relatively thick strata of weaker residual soils.

An expansion of the existing failure in this general area could negatively impact the future serviceability of Allen Street. Soil nailing or ground anchor systems described above to stabilize the existing fill slopes will not improve the stability of the adjacent property failure area. Additionally, these systems will not function as intended and could be compromised if the adjacent property slope failure expands significantly uphill. Remedial measures will be required to permanently stabilize the adjacent property failure area. This could likely include some combination of 1.) dewatering the failure area with horizontal drains, 2.) regrading of the property to a shallower overall slope inclination to remove excess driving force, and 3.) the possible installation of soil nail / ground anchor systems. Detailed remedial recommendations should be developed in conjunction with a subsurface investigation and additional analyses of the adjacent property.

Until remedial recommendations are implemented, the adjacent property failure area will likely continue to undergo some degree of slope instability, especially during or after periods of sustained and significant precipitation events. Access to areas below the failure area should be prevented as much as possible to reduce the likelihood of loss of life and/or property in the event of a subsequent failure prior to implementation of remedial recommendations.

SPECIFICATIONS REVIEW

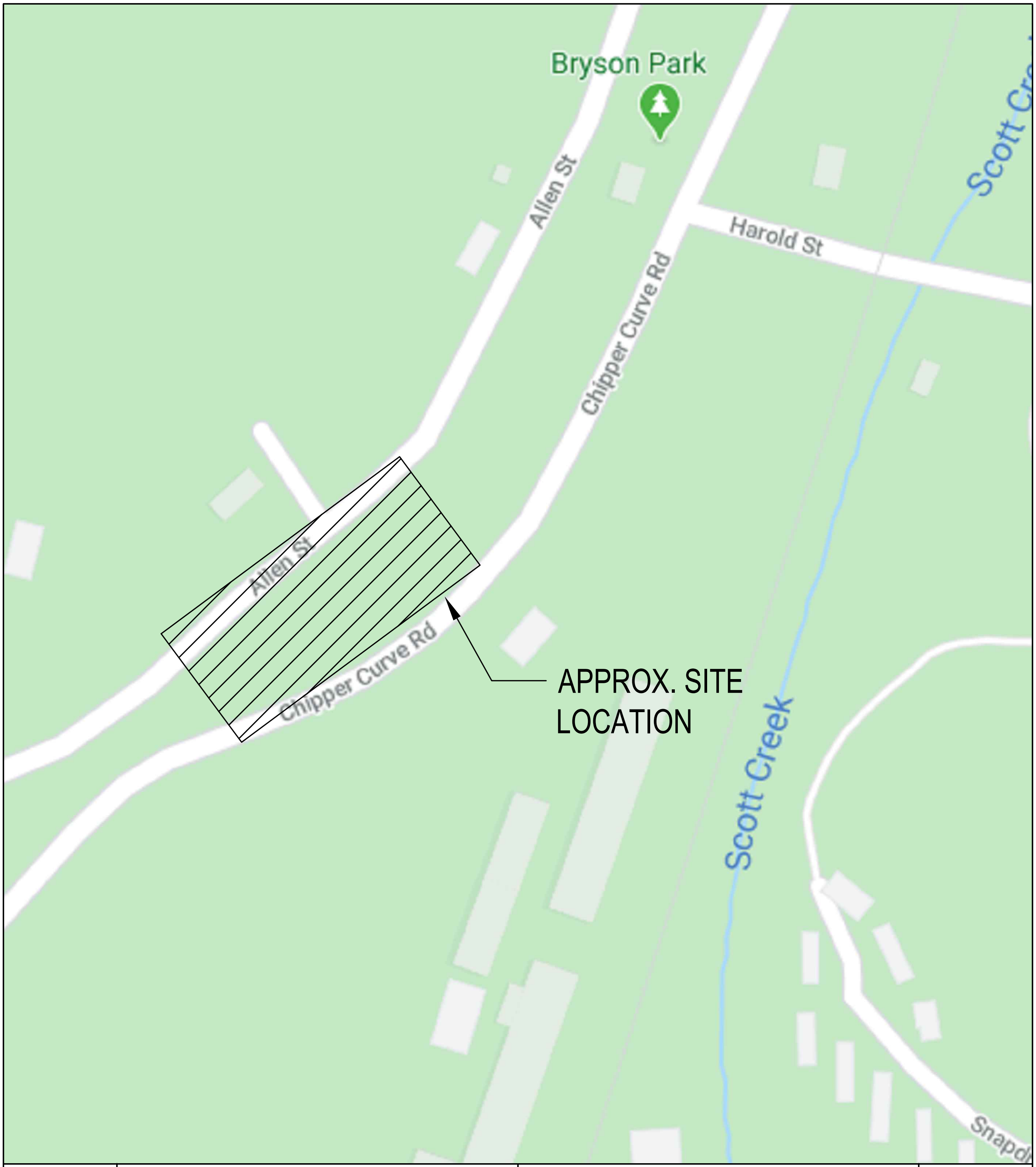
We recommend that we be retained to make a review of the earthwork plans and specifications prepared from the recommendations presented in this report. We would then suggest any modifications so that our recommendations are properly interpreted and implemented. An additional fee would apply for review of plans and specifications.

BASIS OF RECOMMENDATIONS AND LIMITATIONS

Our evaluation of the existing slope conditions has been based on our understanding of the project information, the provided topographic information, and data obtained in our exploration as well as our experience on similar projects. The general subsurface conditions utilized in our slope stability analyses have been based on interpolation of the subsurface data between the widely spaced soil test borings. Subsurface conditions between the borings may differ. If the project information is incorrect, please contact us so that our recommendations can be reviewed. The discovery of site or subsurface conditions during construction which deviate from the data obtained in this exploration should be reported to us for our evaluation.

The assessment of site environmental conditions for presence or absence of pollutants in the soil, rock and ground water of the site was beyond the scope of this exploration. We note that our current scope of services is limited to the existing failure areas located within Allen Street described above and does not extend to slopes present on the adjacent properties on Bobwhite Lane or to other sloped areas located between Allen Street and Chipper Curve Road not associated with the subject slope failure.

APPENDIX



SITE LOCATION PLAN
KEG PROJECT NO. JA20-4021-02
DATE: 03-19-2021

ALLEN STREET SLOPE FAILURE - S. SECTION
SYLVA, NORTH CAROLINA

FIGURE
1

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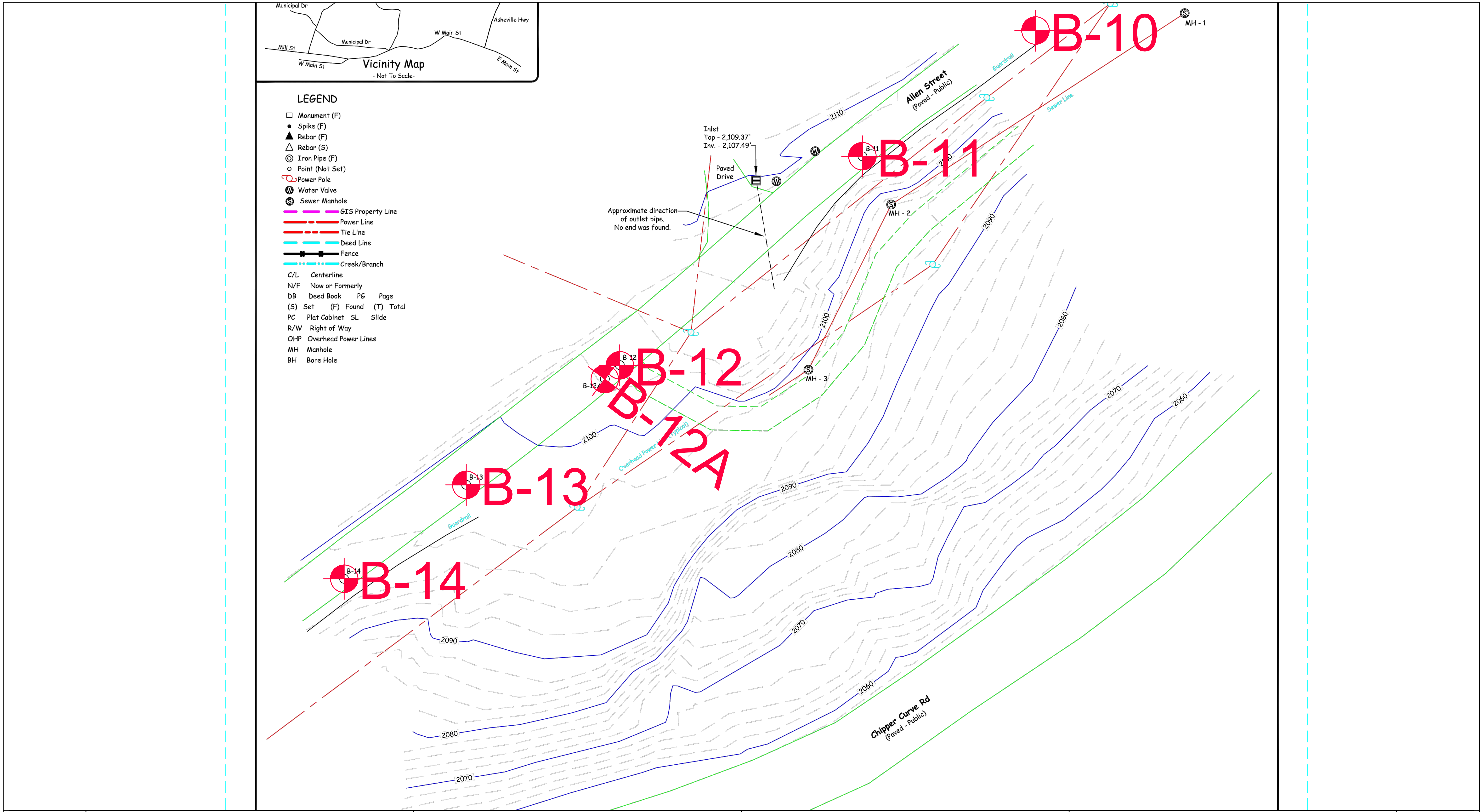

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ALLEN STREET SLOPE FAILURE - S. SECTION
SYLVA, NORTH CAROLINA

KEG PROJECT NO. JA20-4021-02

FIELD EXPLORATION PLAN DATE: 03-19-2021

REFERENCE: Topographic Survey prepared for: Town of Sylva, by Alliance Land Surveying, P.C., dated 01/20/2021.

Note: Boring B-10 location estimated from field measurement by KEG. Other boring locations by surveyor.

0' 15' 30' 60'

SCALE: 1" = 30'-0"

FIGURE
2

FIELD EXPLORATION AND LABORATORY TEST PROCEDURES

SOIL TEST BORINGS

The soil test borings were advanced by mechanically twisting a continuous flight steel auger into the soil. Soil sampling and penetration testing were performed in general accordance with ASTM D 1586. At assigned intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final 12 inches was recorded and is designated the “penetration resistance.” The penetration resistance, once properly evaluated, is an index to the strength of the soil and foundation supporting capability. Representative portions of the soil samples, thus obtained, were placed in plastic bags and transported to the laboratory. In the laboratory, the samples were examined to verify the field classifications made by the driller. Soil descriptions and penetration resistances are tabulated on the attached Boring Logs.

UNDISTURBED SAMPLING

Split-barrel samples are suitable for visual examination and classification tests but are not sufficiently intact for quantitative laboratory tests. Therefore, relatively undisturbed samples were obtained in selected soil test borings by drilling to the desired depth and hydraulically forcing a section of 3-inch O.D., 16-gauge steel tubing into the soil. The sampling procedure is described by ASTM D 1587. Together with the encased soil, each tube was carefully removed from the ground, made airtight and transported to the laboratory. The location of gathered undisturbed samples is indicated on the attached Boring Logs.

CONSOLIDATED – UNDRAINED TRIAXIAL SHEAR TESTING

Consolidated undrained triaxial compression tests with pore pressure measurements were performed on three undisturbed soil samples. The extruded samples were encased in rubber membranes. Each was then placed in a compression chamber and confined by all-around water pressure. An increasing axial load was then applied until the sample failed in shear. The test results are presented in the form of Stress-Strain Curves and Mohr Diagrams on the accompanying Triaxial Shear Test Reports. The results are presented in the Table 1 of the attached Report.

NATURAL MOISTURE CONTENT

The natural moisture content of selected samples was determined in accordance with ASTM D 2216. The moisture content of the soil is the ratio, expressed as a percentage, of the weight of water in a given mass of soil to the weight of the soil particles. The results are presented in the Table 1 of the attached Reports.

PARTICLE/GRAIN SIZE ANALYSES

The distribution of particle sizes present in selected samples was determined by passing the sample through a series of sieves with increasing fine mesh openings. The percent by weight retained on each sieve is the utilized to help determine the soil classification as well as the percent of soils within the silt/clay size range (finer than #200 sieve). These tests were conducted in accordance with ASTM D 6913. The results are on the attached Particle Size Distribution Reports and Table 1 of the attached Reports.

SOIL PLASTICITY

Representative samples of the silty soils were selected for Atterberg Limits testing to determine their soil plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). These characteristics are determined in accordance with ASTM D 4318. The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. The data obtained are presented on the attached Table 1 and the Liquid and Plastic Limit Test Reports.

SOIL TEST BORING LOGS (B-10 TO B-14)

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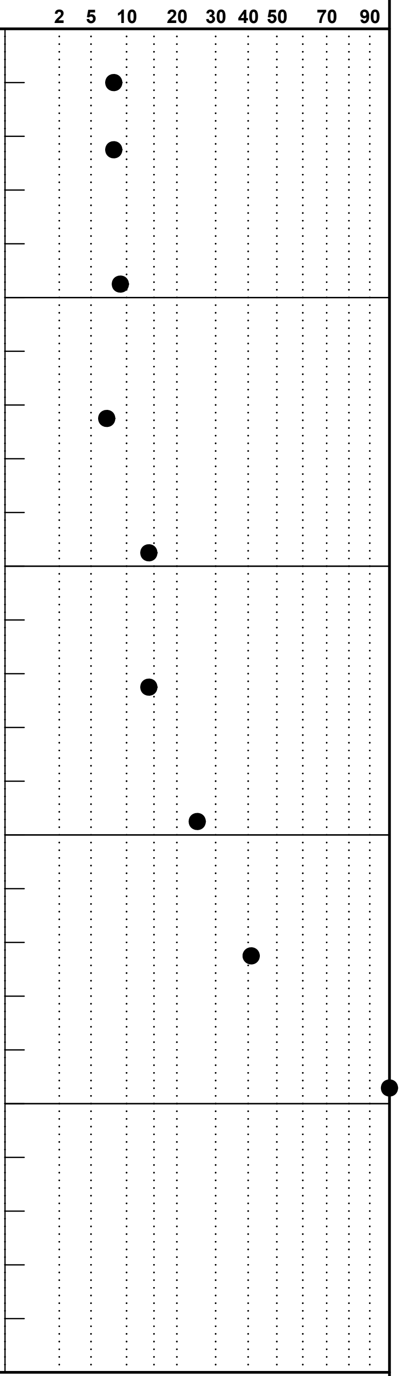
SOIL TEST BORING NO. B-10

PROJECT: Allen Street Slope Failure - S. Section
PROJECT NO.: JA20-4021-02
CLIENT: Town of Sylva
DATE START: 12-7-20 END: 12-7-20
LOCATION: See Figure 2
ELEVATION: 2107 (feet)
DRILLER: M&M Drilling
LOGGED BY: I. Johnson
DRILLING METHOD: Hollow Stem Auger
DEPTH TO - WATER> INITIAL∇ 37.4 ft AFTER 24 HOURS:▼ ________ CAVING>▨ 31.3 ft

ELEVATION/ DEPTH (FT)	DESCRIPTION	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT
	4-Inches ASPHALT		
	8-Inches ABC STONE		
	Firm, Brown, Sandy SILT (Residuum)	3 3 5	N = 8
	Loose, Brown with Light Tan, Slightly Moist, Silty, Fine to Medium SAND	3 3 5	N = 8
	*Undisturbed sample between 5 and 7 feet.		
2100			
	Loose, Grayish Brown to Reddish Brown, Very Slightly Micaceous, Silty, Fine SAND	3 4 5	N = 9
10			
		3 4 3	N = 7
2090	Firm to Very Firm, Brown with Dark Gray and Trace White, Very Slightly Micaceous to Micaceous, Silty, Fine to Medium SAND		
20		6 6 8	N = 14
		5 7 7	N = 14
2080			
30		6 11 14	N = 25
▨			
	Dense, Brown, Very Micaceous, Silty, Fine to Medium SAND	14 22 19	N = 41
2070 ∇	PARTIALLY WEATHERED ROCK which sampled as Brown and Black, Very Micaceous, Silty, Fine to Medium SAND		
40		42 31 50/5	N = 50/5
	Boring terminated at 40.0 feet. Groundwater encountered at 37.4 feet at time of boring. Borehole caved at 31.3 feet at time of boring.		
2060			



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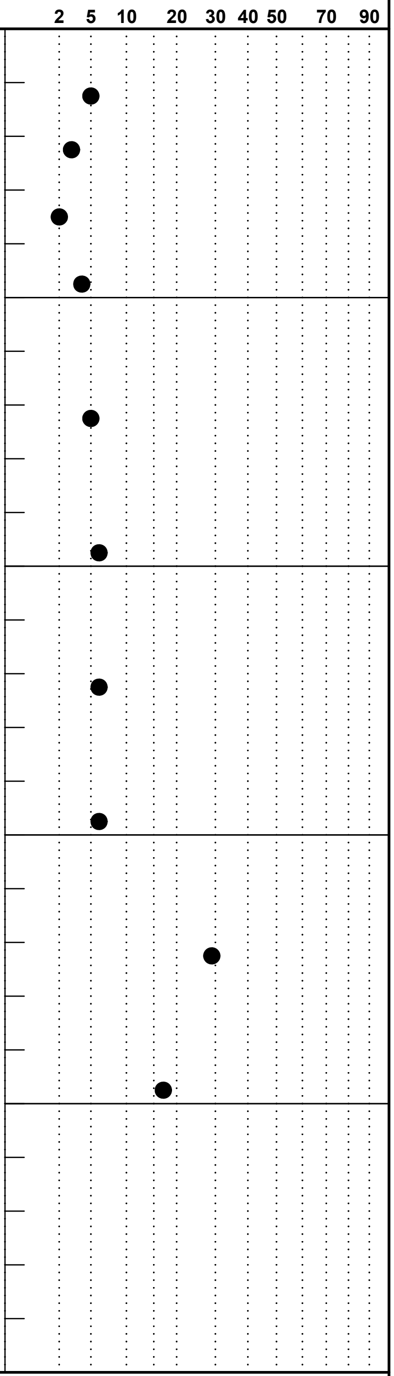
SOIL TEST BORING NO. B-11

PROJECT: Allen Street Slope Failure - S. Section
PROJECT NO.: JA20-4021-02
CLIENT: Town of Sylva
DATE START: 12-8-20 END: 12-8-20
LOCATION: See Figure 2
ELEVATION: 2107 (feet)
DRILLER: M&M Drilling
LOGGED BY: I. Johnson
DRILLING METHOD: Hollow Stem Auger
DEPTH TO - WATER> INITIAL ▽ 34.8 ft AFTER 1 HOURS: ▼ 29.7 ft CAVING> ▩ 29.8 ft

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES		
	6-Inches ASPHALT	●			
	12-Inches ABC STONE	●			
	Loose to Very Loose, Dark Brown to Reddish Brown, Moist, Micaceous, Silty, Fine SAND with Trace Gravel (Fill)	▨	☒	3 2 3	N = 5
			☒	1 1 2	N = 3
2100	Soft, Reddish Brown, Slightly Moist, Sandy SILT (Residuum)		☒	1 1 1	N = 2
10			☒	1 2 2	N = 4
	Loose, Reddish Brown to Brown with Orange and White, Slightly Moist, Slightly Micaceous, Silty, Fine to Medium SAND	▨	☒	3 2 3	N = 5
2090 / 20			☒	3 3 3	N = 6
			☒	3 3 3	N = 6
2080 / 30 ▼ ▩			☒	1 3 3	N = 6
▽	Very Firm, Dark Gray, Moist, Silty, Fine to Medium SAND	▨	☒	9 10 19	N = 29
2070 / 40	Firm, Brown, Very Slightly Micaceous, Silty, Fine SAND	▨	☒	7 8 9	N = 17
2060	Boring terminated at 40.0 feet. Groundwater encountered at 34.8 feet at time of boring and 29.7 feet 1 hour after drilling. Borehole caved at 29.8 feet at time of boring.				

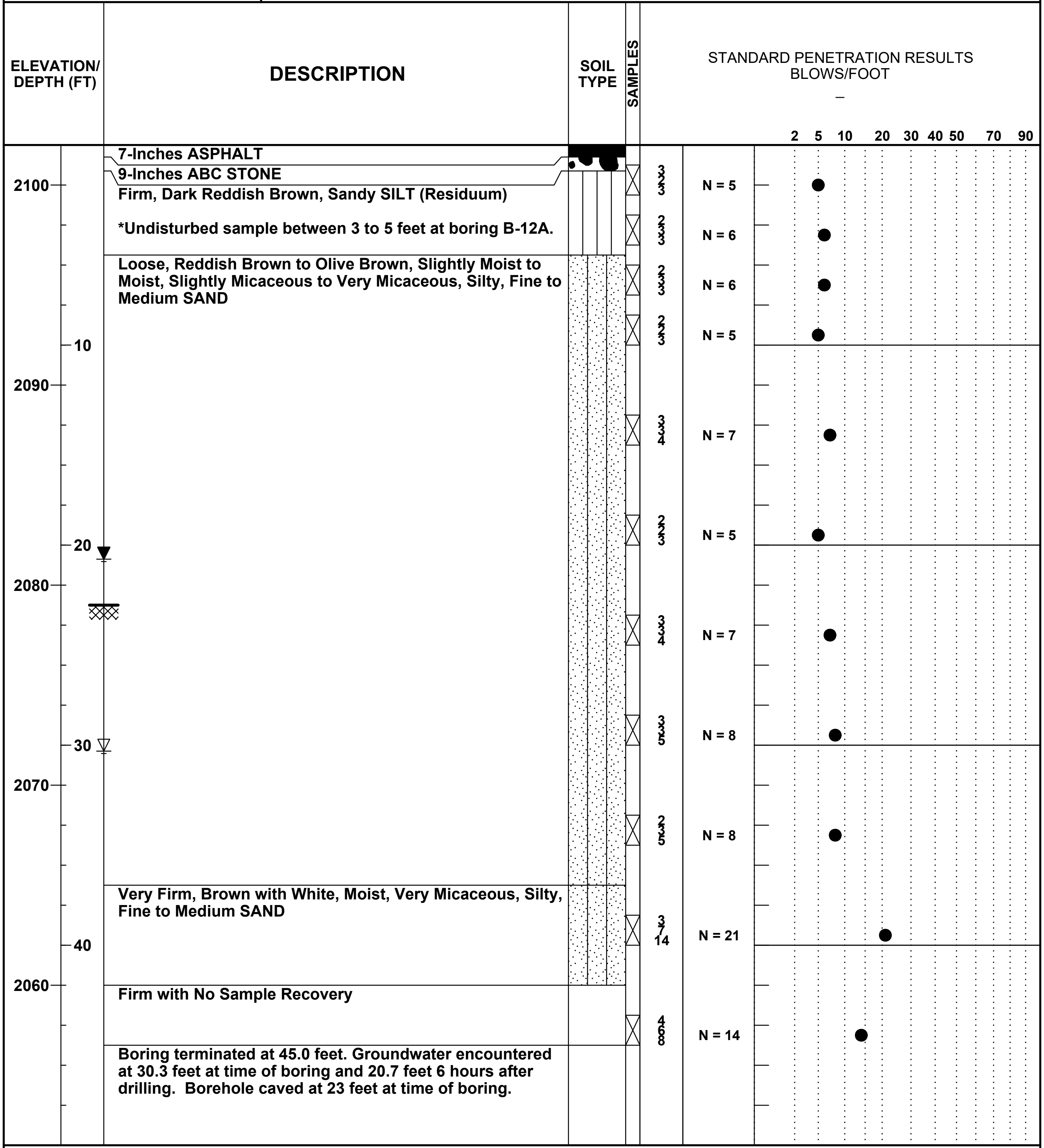
STANDARD PENETRATION RESULTS
BLOWS/FOOT



KESSEL ENGINEERING GROUP

SOIL TEST BORING NO. B-12/12A

PROJECT: Allen Street Slope Failure - S. Section
PROJECT NO.: JA20-4021-02
CLIENT: Town of Sylva
DATE START: 12-7-20 END: 12-7-20
LOCATION: See Figure 2
ELEVATION: 2102 (feet)
DRILLER: M&M Drilling
LOGGED BY: I. Johnson
DRILLING METHOD: Hollow Stem Auger
DEPTH TO - WATER> INITIAL ▽ 30.3 ft AFTER 6 HOURS: ▼ 20.7 ft CAVING> ▨ 23.0 ft



7-Inches ASPHALT

9-Inches ABC STONE

Firm, Dark Reddish Brown, Sandy SILT (Residuum)

*Undisturbed sample between 3 to 5 feet at boring B-12A.

Loose, Reddish Brown to Olive Brown, Slightly Moist to Moist, Slightly Micaceous to Very Micaceous, Silty, Fine to Medium SAND

Very Firm, Brown with White, Moist, Very Micaceous, Silty, Fine to Medium SAND

Firm with No Sample Recovery

Boring terminated at 45.0 feet. Groundwater encountered at 30.3 feet at time of boring and 20.7 feet 6 hours after drilling. Borehole caved at 23 feet at time of boring.

3 2 3	N = 5
2 3 3	N = 6
2 3 3	N = 6
2 2 3	N = 5
3 3 4	N = 7
2 2 3	N = 5
3 3 4	N = 7
3 3 5	N = 8
2 3 5	N = 8
3 7 14	N = 21
4 6 8	N = 14

Note: B-12 encountered shallow refusal at 6 feet due to angled augers. Offset +/- 7 feet southwest and continued as B-12A.

SOIL TEST BORING NO. B-12/12A
Sheet 1 of 1

SOIL TEST BORING NO. B-13

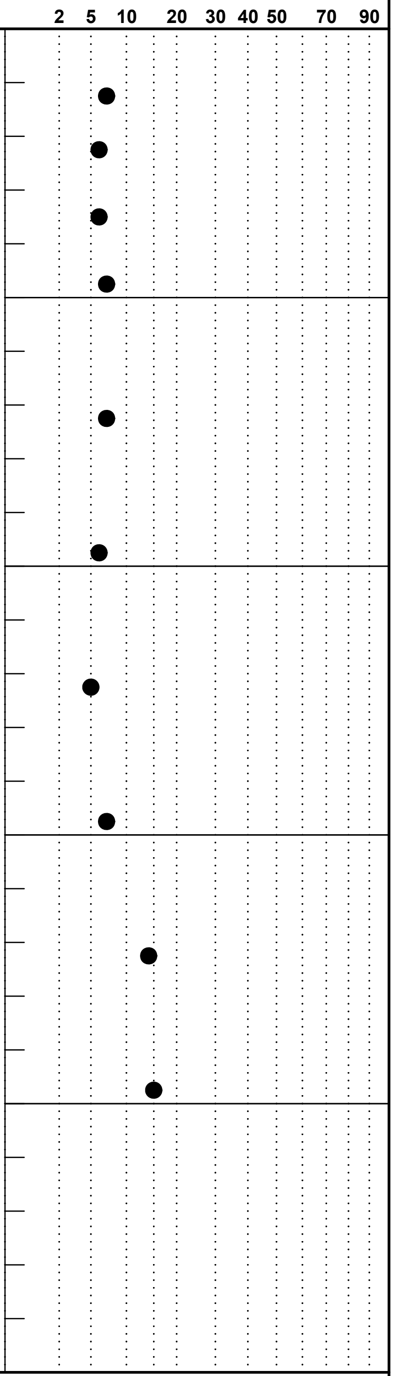
KESSEL ENGINEERING GROUP

PROJECT: Allen Street Slope Failure - S. Section
PROJECT NO.: JA20-4021-02
CLIENT: Town of Sylva
DATE START: 12-7-20 END: 12-7-20
LOCATION: See Figure 2
ELEVATION: 2099 (feet)
DRILLER: M&M Drilling
LOGGED BY: I. Johnson
DRILLING METHOD: Hollow Stem Auger
DEPTH TO - WATER> INITIAL▽ 28.8 ft AFTER 8 HOURS:▼ 19.5 ft CAVING> 32.2 ft

ELEVATION/ DEPTH (FT)	DESCRIPTION
	6-Inches ASPHALT
	12-Inches ABC STONE
	Loose, Brown with Black, Moist, Silty, Fine to Coarse SAND with Gravel, Asphalt and Glass Fragments (Fill)
2090 / 10	Loose, Light Tan with White to Reddish Brown, Moist, Slightly Micaceous to Micaceous, Silty, Fine to Coarse SAND (Residuum) *Undisturbed sample between 10 and 12 feet.
2080 / 20	
2070 / 30	Firm, Reddish Brown, Moist, Sandy SILT
	Firm, Brown, Moist, Micaceous, Silty, Fine to Coarse SAND with Trace Gravel
2060 / 40	Boring terminated at 40.0 feet. Groundwater encountered at 28.8 feet at time of boring and 19.5 feet 8 hours after drilling. Borehole caved at 32.2 feet at time of boring.
2050	

SAMPLES	
6-5-2	N = 7
3-3-3	N = 6
2-3-3	N = 6
2-3-4	N = 7
3-4-3	N = 7
2-2-4	N = 6
1-2-3	N = 5
2-3-4	N = 7
3-6-8	N = 14
4-6-9	N = 15

STANDARD PENETRATION RESULTS
BLOWS/FOOT

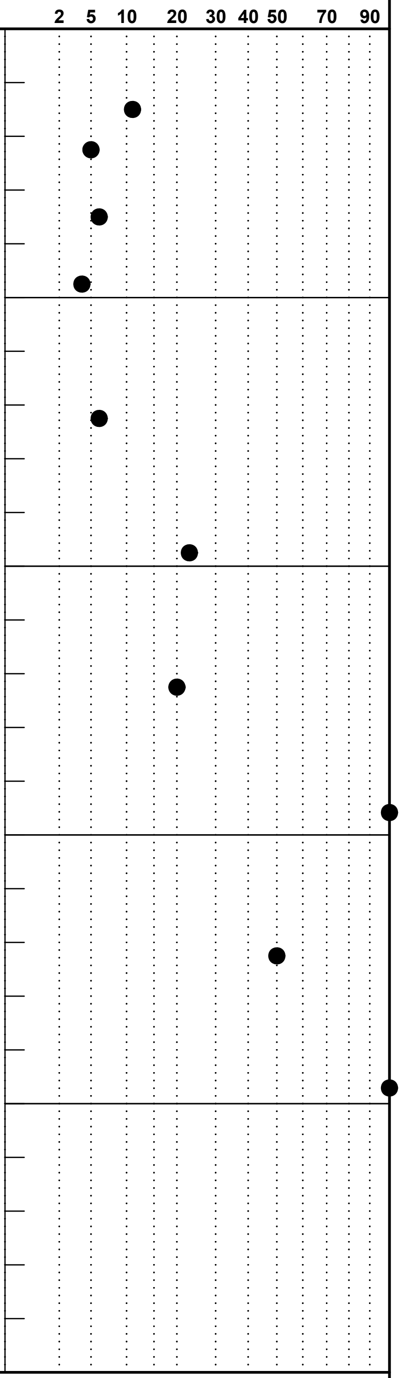


KESSEL ENGINEERING GROUP

SOIL TEST BORING NO. B-14

PROJECT: Allen Street Slope Failure - S. Section
PROJECT NO.: JA20-4021-02
CLIENT: Town of Sylva
DATE START: 12-7-20 **END:** 12-7-20
LOCATION: See Figure 2
ELEVATION: 2097 (feet)
DRILLER: M&M Drilling
LOGGED BY: I. Johnson
DRILLING METHOD: Hollow Stem Auger
DEPTH TO - WATER> INITIAL: ▽ 23.3 ft **AFTER 10 HOURS:** ▼ 20.7 ft **CAVING>** ▨ 22.6 ft

ELEVATION/ DEPTH (FT)	DESCRIPTION	SOIL TYPE	SAMPLES	STANDARD PENETRATION RESULTS BLOWS/FOOT	
	6-Inches ASPHALT				
	Approximately 1.5 feet ABC STONE				
	Firm to Loose, Reddish Brown and Black, Moist, Slightly Micaceous, Silty, Fine to Coarse SAND with Trace Gravel (Fill)		☒	6 5 6	N = 11
			☒	4 2 3	N = 5
2090	Soft to Firm, Reddish Brown, Sandy SILT (Residuum)		☒	4 2 4	N = 6
10			☒	2 2 2	N = 4
			☒	2 3 3	N = 6
2080	Very Firm, Brown, Silty, Fine to Coarse SAND with Gravel		☒	16 12 11	N = 23
20 ▼					
▨	Firm, Tan, Moist, Micaceous, Silty, Fine to Medium SAND		☒	6 9 11	N = 20
2070	PARTIALLY WEATHERED ROCK which sampled as Tan, Moist, Micaceous, Silty, Fine to Medium SAND		☒	16 46 50/2	N = 50/2
30					
	Dense, Black with Orange, Micaceous, Silty, Fine to Coarse SAND		☒	20 20 30	N = 50
2060	PARTIALLY WEATHERED ROCK which sampled as Black with Tan, Silty, Fine to Coarse SAND		☒	28 28 50/5	N = 50/5
40	Boring terminated at 40.0 feet. Groundwater encountered at 23.3 feet at time of boring and 20.7 feet 10 hours after drilling. Borehole caved at 22.6 feet at time of boring.				
2050					



SOIL TEST BORING NO. B-14
Sheet 1 of 1

KEY TO SOIL CLASSIFICATIONS AND CONSISTENCY DESCRIPTIONS

SANDS

Penetration Resistance* Blows per Foot	Relative Density
0 to 4	Very Loose
5 to 10	Loose
11 to 20	Firm
21 to 30	Very Firm
31 to 50	Dense
over 50	Very Dense

Particle Size Identification

Boulder: Greater than 300 mm
Cobble: 75 to 300 mm
Gravel:
Coarse - 19 to 75 mm
Fine - 4.75 to 19 mm
Sand:
Coarse - 2 to 75 mm
Medium - 0.425 to 2 mm
Fine - 0.075 to 0.425 mm
Silts & Clay: Less than 0.075 mm

SILTS and CLAYS

Penetration Resistance* Blows per Foot	Consistency
0 to 2	Very Soft
3 to 4	Soft
5 to 8	Firm
9 to 15	Stiff
16 to 30	Very Stiff
31 to 50	Hard
over 50	Very Hard

* ASTM D 1586

KEY TO DRILLING SYMBOLS

✋ Grab Sample

☒ Split Spoon Sample

☐ Undisturbed Sample

▽ Groundwater Table at Time of Drilling

▼ Groundwater Table 24 Hours after Completion of Drilling

KEY TO SOIL CLASSIFICATIONS

▦ Well-graded Gravel
GW

○ Poorly-graded Gravel
GP

■ Partially Weathered
Rock
BLDRCBBL

▨ High Plasticity Clay
CH

▨ Low Plasticity Clay
CL

▨ Sandy Clay
CLS

▥ Silty Clay
CL-ML

▥ Silt
ML

▥ Clayey Silt
MH

▥ Sandy Silt
MLS

░ Sand
SW

▨ Clayey Sand
SC

░ Silty Sand
SM

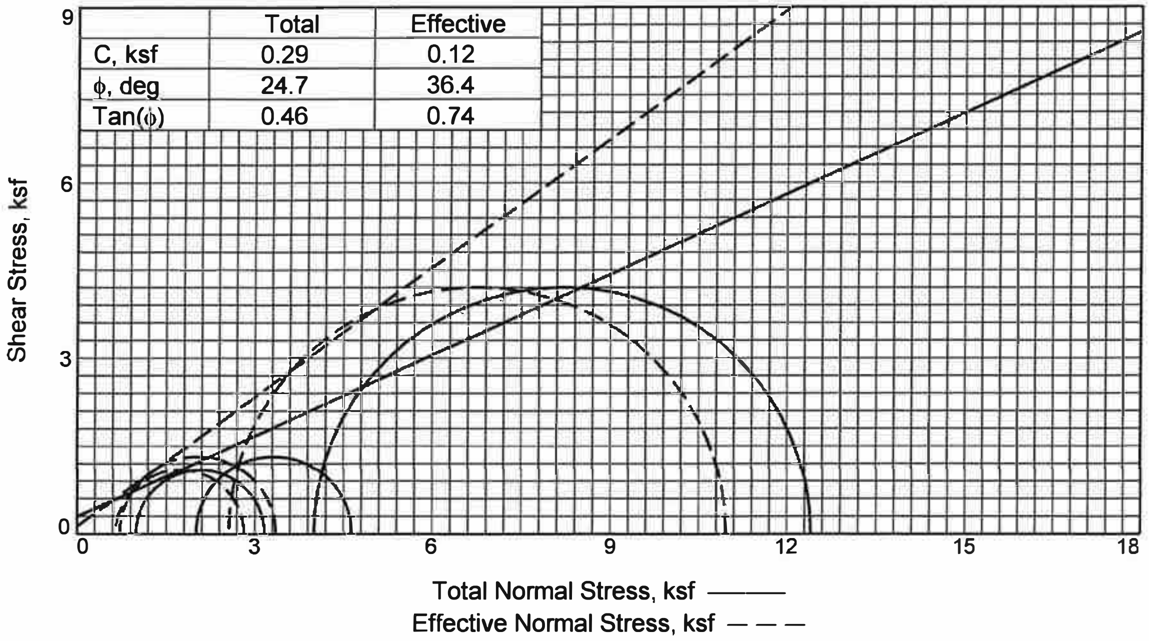
░ Topsoil
TOPSOIL

▧ Bedrock
BEDROCK

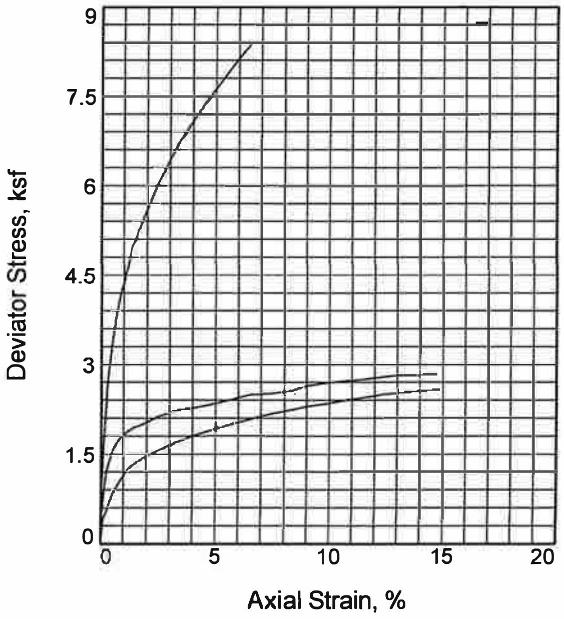
░ Concrete
AS

LABORATORY TEST RESULTS

1. Undisturbed Sample Nos. B-10, B-13
    a. Triaxial Shear Test Report No. TX B-10
    b. Triaxial Shear Test Report No. TX B-13
2. Particle Size Distribution Reports
    a. Particle Size Distribution Report PS B-10
    b. Particle Size Distribution Report PS B-12
    c. Particle Size Distribution Report PS B-13a
    d. Particle Size Distribution Report PS B-13b
3. Liquid Limits and Plastic Limits Test Reports
    a. Liquid Limits and Plastic Limits Test Report PI B-12
    b. Liquid Limits and Plastic Limits Test Report PI B-13b



	Total	Effective
C, ksf	0.29	0.12
ϕ, deg	24.7	36.4
Tan(ϕ)	0.46	0.74



Sample No.		1	2	3
Initial	Water Content, %	17.2	17.1	13.3
	Dry Density, pcf	87.8	81.9	96.1
	Saturation, %	50.6	43.6	47.5
	Void Ratio	0.9197	1.0579	0.7537
	Diameter, in.	2.866	2.865	2.867
	Height, in.	6.107	6.104	6.178
At Test	Water Content, %	32.2	32.2	24.4
	Dry Density, pcf	90.2	90.1	101.6
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8696	0.8707	0.6598
	Diameter, in.	2.837	2.729	2.813
	Height, in.	6.071	6.117	6.076
Strain rate, %/min.		0.07	0.07	0.07
Back Pressure, psi		70.00	70.00	70.00
Cell Pressure, psi		76.90	83.90	97.80
Fail. Stress, ksf		2.2	2.6	8.4
Total Pore Pr., ksf		10.4	11.4	11.5
Ult. Stress, ksf				
Total Pore Pr., ksf				
$\bar{\sigma}_1$ Failure, ksf		2.8	3.4	11.0
$\bar{\sigma}_3$ Failure, ksf		0.6	0.7	2.6

Type of Test:
CU with Pore Pressures

Sample Type: Shelby Tube

Description: Grey-Brown Silty Sand

Assumed Specific Gravity= 2.70

Remarks:

Figure TX B-10

Client: Kessel Engineering Group

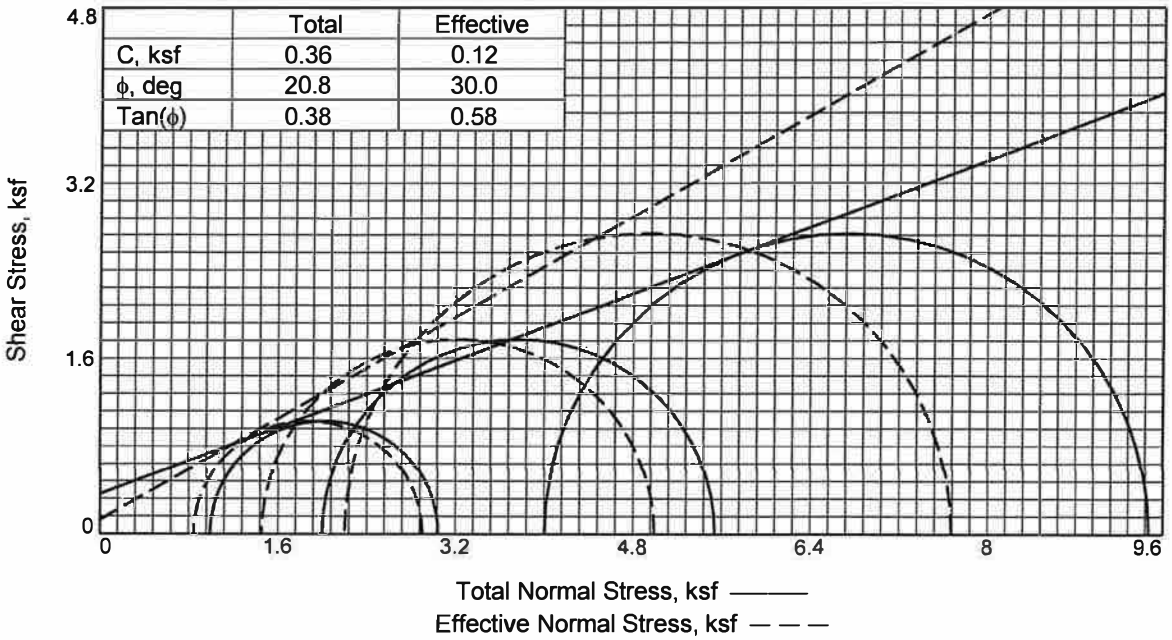
Project: Allen Street Slope Failure
Town of Sylva

Location: B-10 UD @ 5'-7'

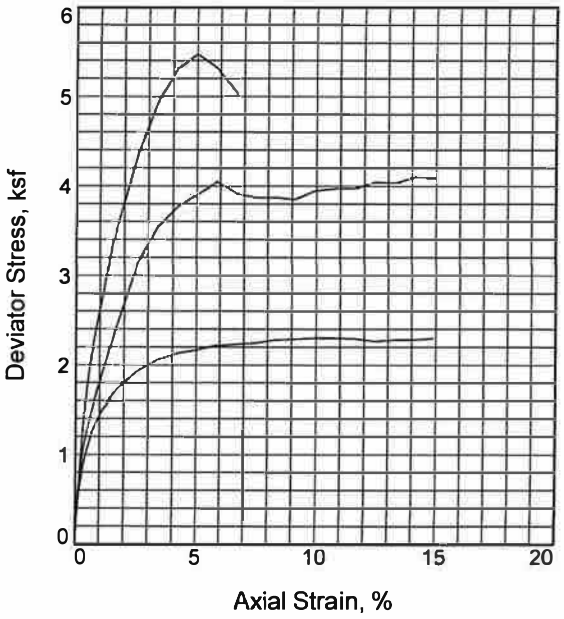
Proj. No.: JA20-4021-02 **Date Sampled:** 12-23-20

TRIAXIAL SHEAR TEST REPORT
Summit Engineering
Ft. Mill, South Carolina

Tested By: FG



	Total	Effective
C, ksf	0.36	0.12
ϕ, deg	20.8	30.0
Tan(ϕ)	0.38	0.58



	Sample No.	1	2	3
Initial	Water Content, %	28.4	28.2	30.9
	Dry Density, pcf	81.9	84.2	81.2
	Saturation, %	72.5	75.9	77.7
	Void Ratio	1.0583	1.0030	1.0748
	Diameter, in.	2.860	2.862	2.866
	Height, in.	6.043	6.081	6.124
At Test	Water Content, %	37.8	35.3	38.9
	Dry Density, pcf	83.4	86.3	82.2
	Saturation, %	100.0	100.0	100.0
	Void Ratio	1.0215	0.9539	1.0512
	Diameter, in.	2.833	2.839	2.876
	Height, in.	6.048	6.030	6.014
	Strain rate, %/min.	0.07	0.07	0.07
	Back Pressure, psi	70.00	70.00	70.00
	Cell Pressure, psi	76.90	83.90	97.80
	Fail. Stress, ksf	2.05	3.54	5.47
	Total Pore Pr., ksf	10.22	10.63	11.88
	Ult. Stress, ksf			
	Total Pore Pr., ksf			
	$\bar{\sigma}_1$ Failure, ksf	2.90	4.99	7.67
	$\bar{\sigma}_3$ Failure, ksf	0.85	1.45	2.20

Type of Test:
CU with Pore Pressures

Sample Type: ShelbyTube

Description: Light-Brown Micaceous Sandy Silt

Assumed Specific Gravity= 2.70

Remarks:

Figure TX B-13

Client: Kessel Engineering Group

Project: Allen Street Slope Failure
Town of Sylva

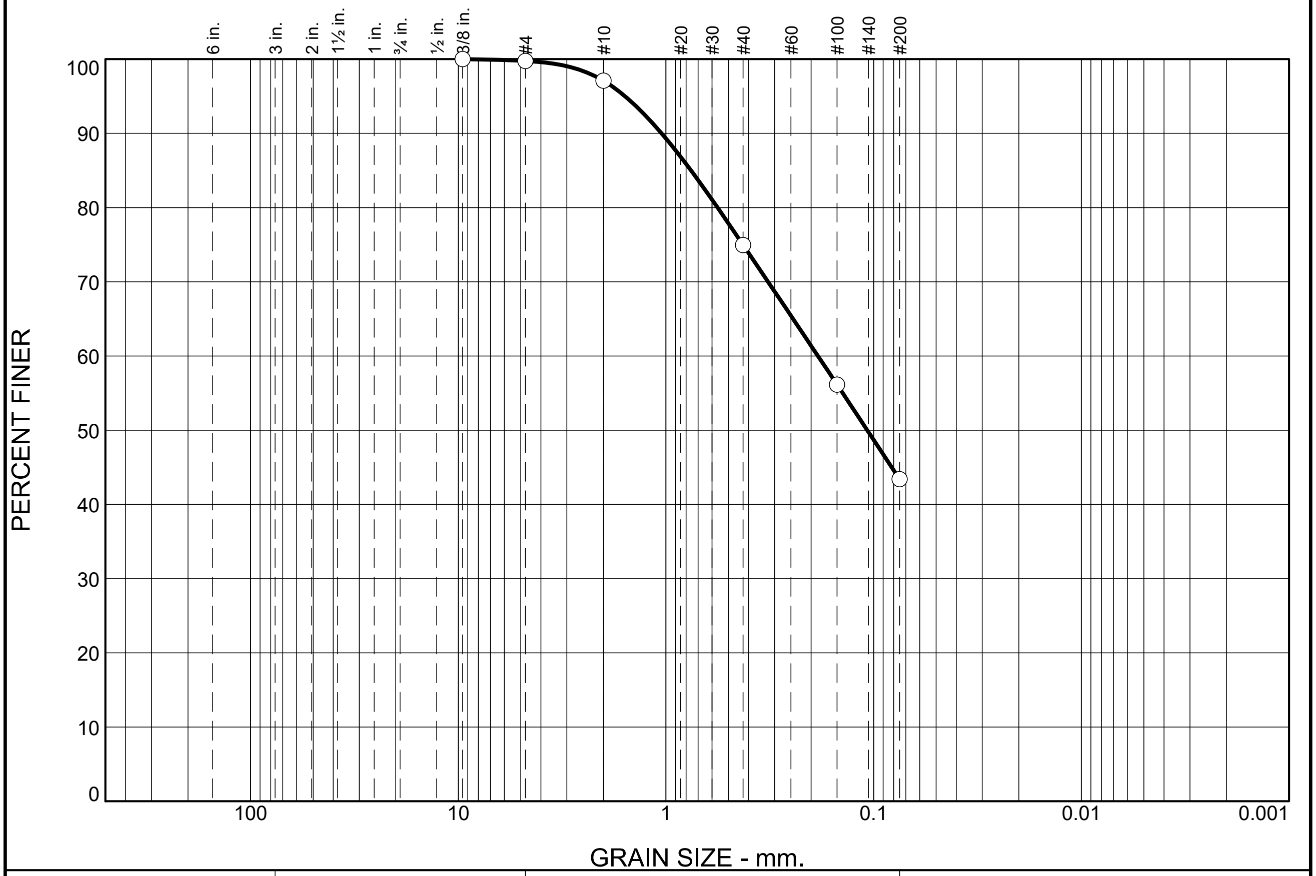
Location: B-13 UD @ 10'-12'

Proj. No.: JA20-4021-02 **Date Sampled:** 12-29-20-

TRIAXIAL SHEAR TEST REPORT
Summit Engineering
Ft. Mill, South Carolina

Tested By: FG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	2.6	22.1	31.6	43.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.7		
#10	97.1		
#40	75.0		
#100	56.1		
#200	43.4		

* (no specification provided)

Material Description

Brown with Light Tan, Silty, Fine to Medium SAND

Atterberg Limits

PL= LL= PI= NP

Coefficients

D_{90}= 1.0493 D_{85}= 0.7576 D_{60}= 0.1853
D_{50}= 0.1073 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sampled at Boring B-10

Source of Sample: Onsite **Depth:** 3.5' - 5.0'
Sample Number: 1 **Date:** 1/13/21

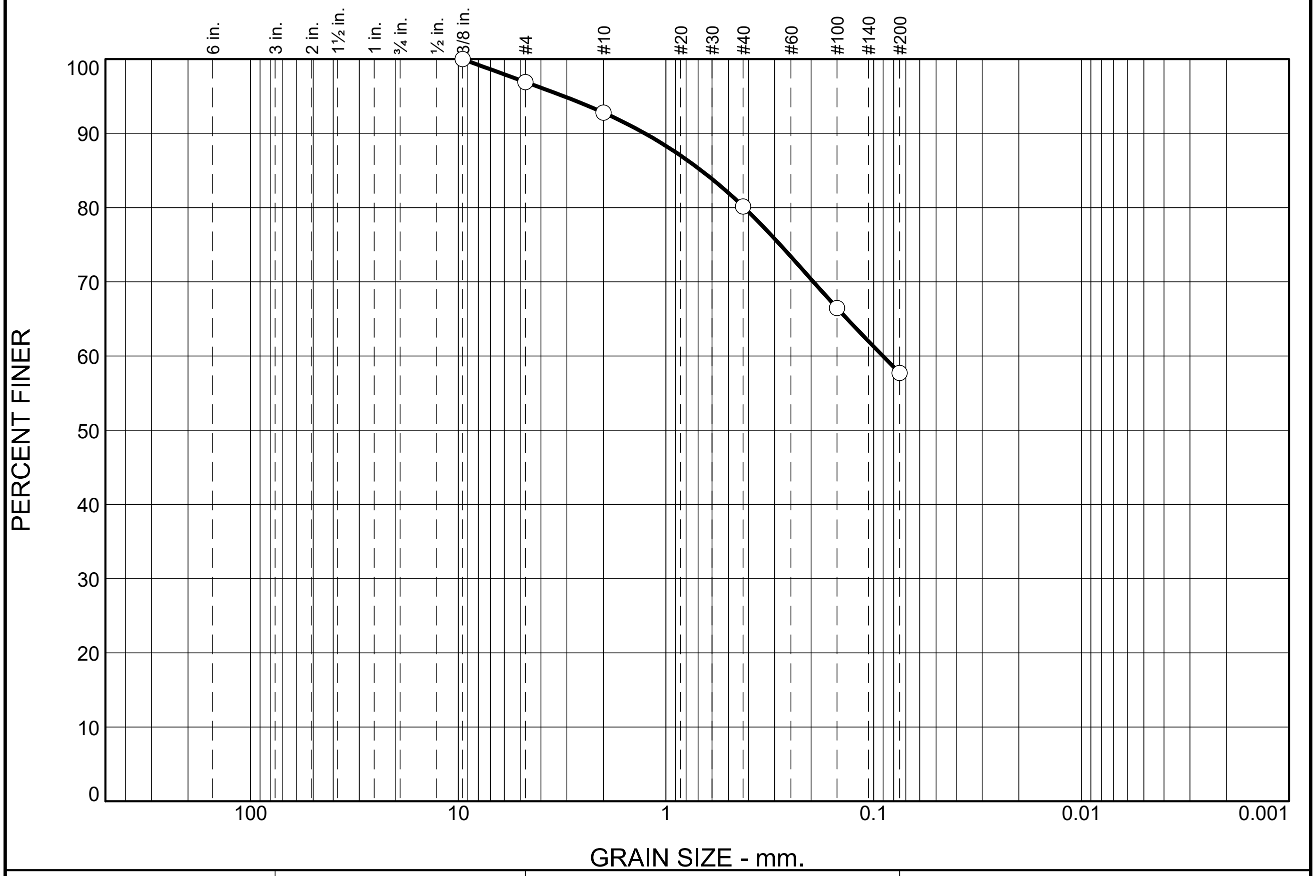
Kessel Engineering Group

Asheville, NC

Client: Town of Sylva
Project: Allen Street Slope Failure 2.0
Project No: JA20-4021-02 **Figure** PS B-10

Tested By: LB **Checked By:** IJ

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.1	4.1	12.7	22.4	57.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	96.9		
#10	92.8		
#40	80.1		
#100	66.5		
#200	57.7		

* (no specification provided)

Material Description

Dark Reddish Brown, Sandy SILT

Atterberg Limits

PL= 27 LL= 34 PI= 7

Coefficients

D_{90}= 1.2646 D_{85}= 0.6764 D_{60}= 0.0902
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO= A-4(3)

Remarks

Sampled at B-12

Source of Sample: Onsite **Depth:** 3.5' - 5.0'
Sample Number: 2 **Date:** 1/13/21

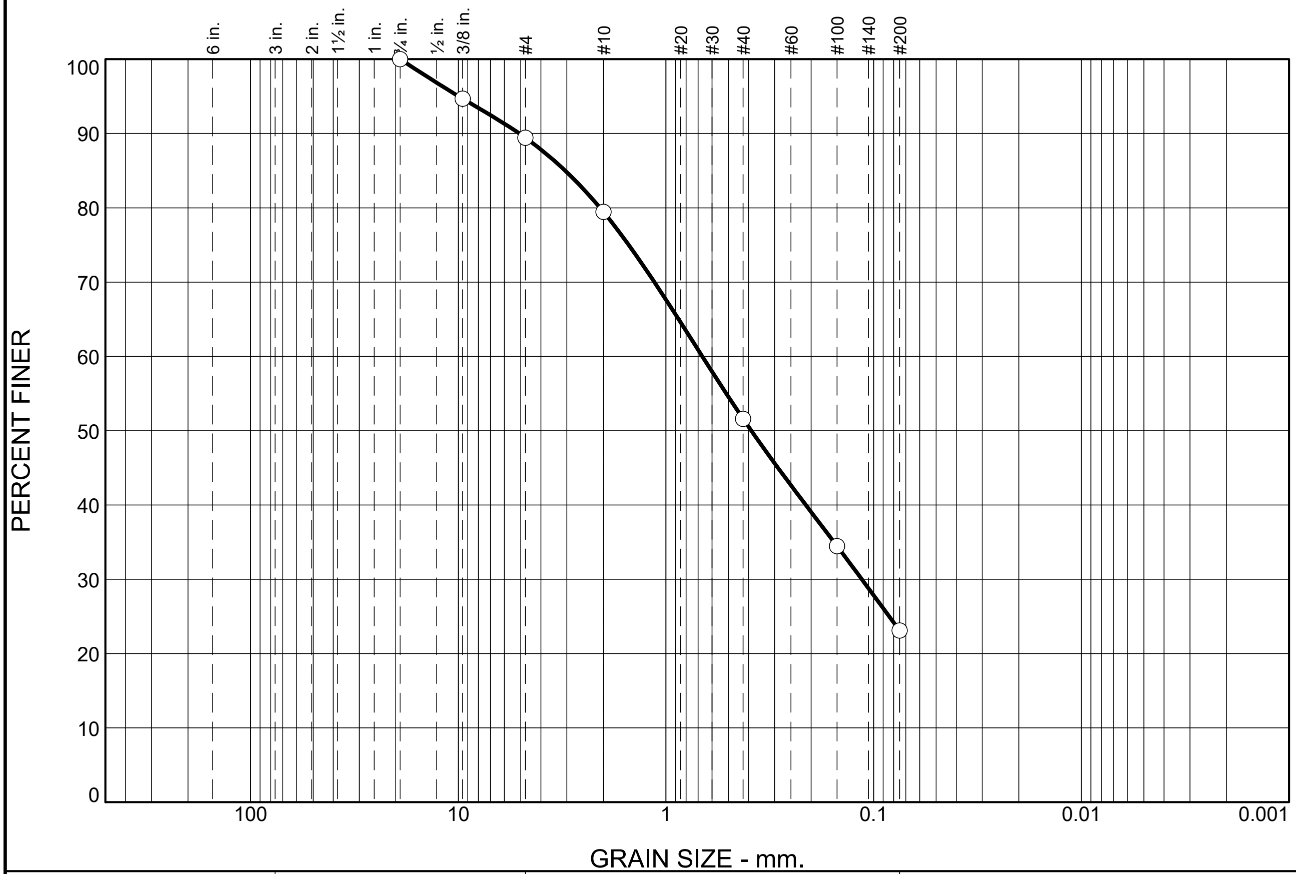
Kessel Engineering Group

Asheville, NC

Client: Town of Sylva
Project: Allen Street Slope Failure 2.0
Project No: JA20-4021-02 **Figure** PS B-12

Tested By: LB **Checked By:** IJ

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	10.6	10.0	27.8	28.5	23.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	94.7		
#4	89.4		
#10	79.4		
#40	51.6		
#100	34.5		
#200	23.1		

* (no specification provided)

Material Description

Reddish Brown, Micaceous, Silty, Fine to Coarse SAND with Gravel

Atterberg Limits

PL= LL= PI= NP

Coefficients

D_{90}= 5.0844 D_{85}= 3.0572 D_{60}= 0.6683
D_{50}= 0.3882 D_{30}= 0.1140 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sampled at B-13

Source of Sample: Onsite **Depth:** 13.5' - 15'
Sample Number: 3 **Date:** 1/13/21

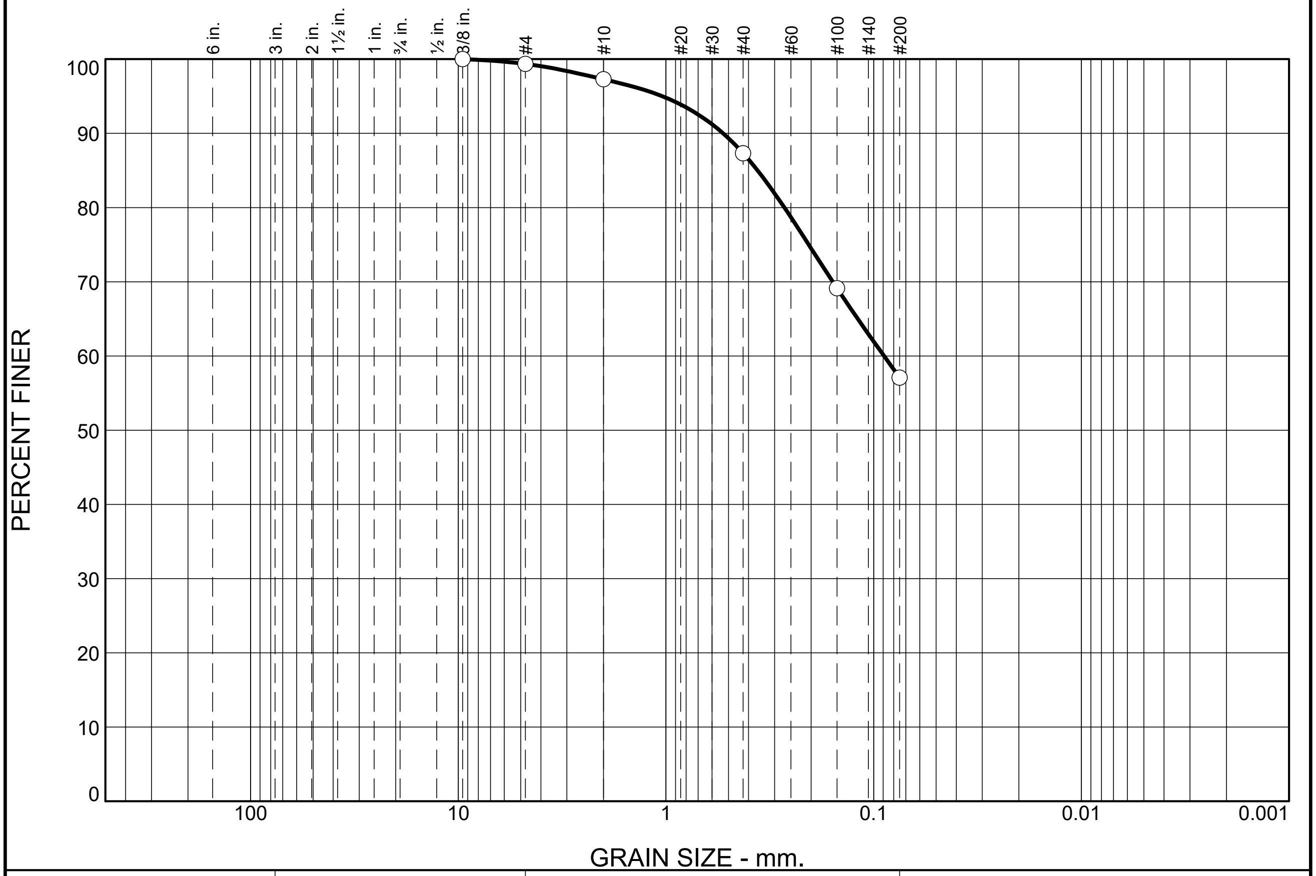
Kessel Engineering Group

Asheville, NC

Client: Town of Sylva
Project: Allen Street Slope Failure 2.0
Project No: JA20-4021-02 **Figure** PS B-13a

Tested By: LB **Checked By:** IJ

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	2.1	10.0	30.2	57.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.4		
#10	97.3		
#40	87.3		
#100	69.1		
#200	57.1		

* (no specification provided)

Material Description

Reddish Brown, Sandy SILT

Atterberg Limits

PL= 30 LL= 38 PI= 8

Coefficients

D_{90}= 0.5304 D_{85}= 0.3620 D_{60}= 0.0891
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO= A-4(3)

Remarks

Sampled at B-13

Source of Sample: Onsite **Depth:** 28.5' - 30'
Sample Number: 4 **Date:** 1/13/21

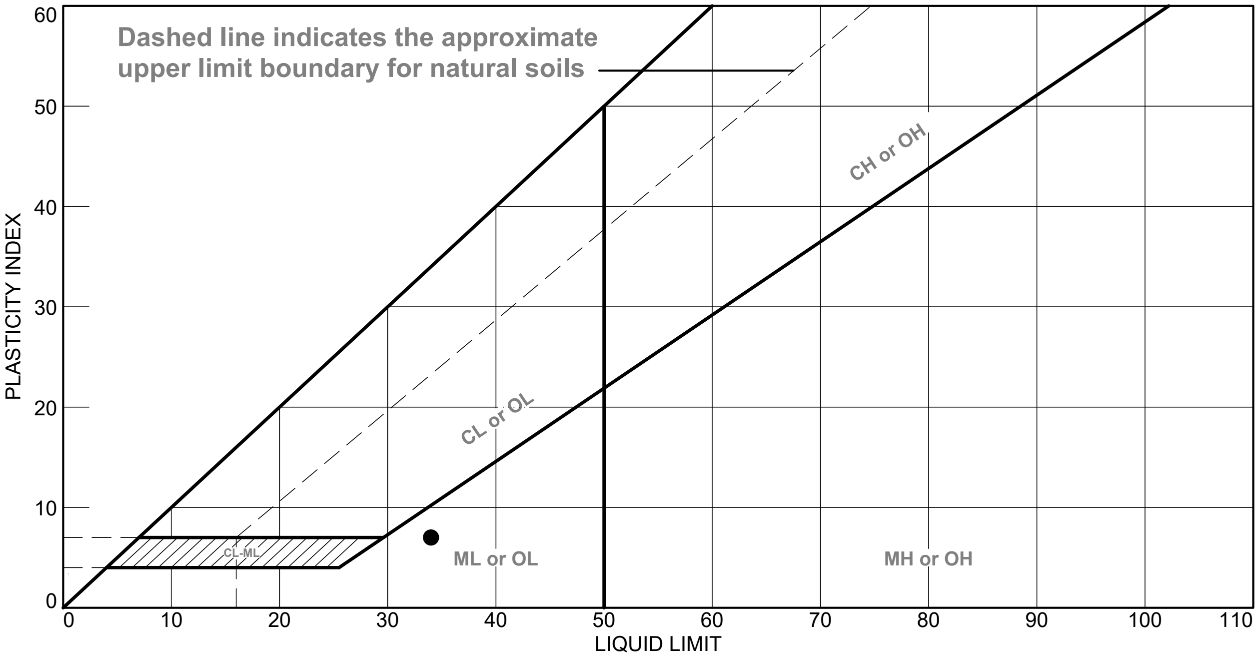
Kessel Engineering Group

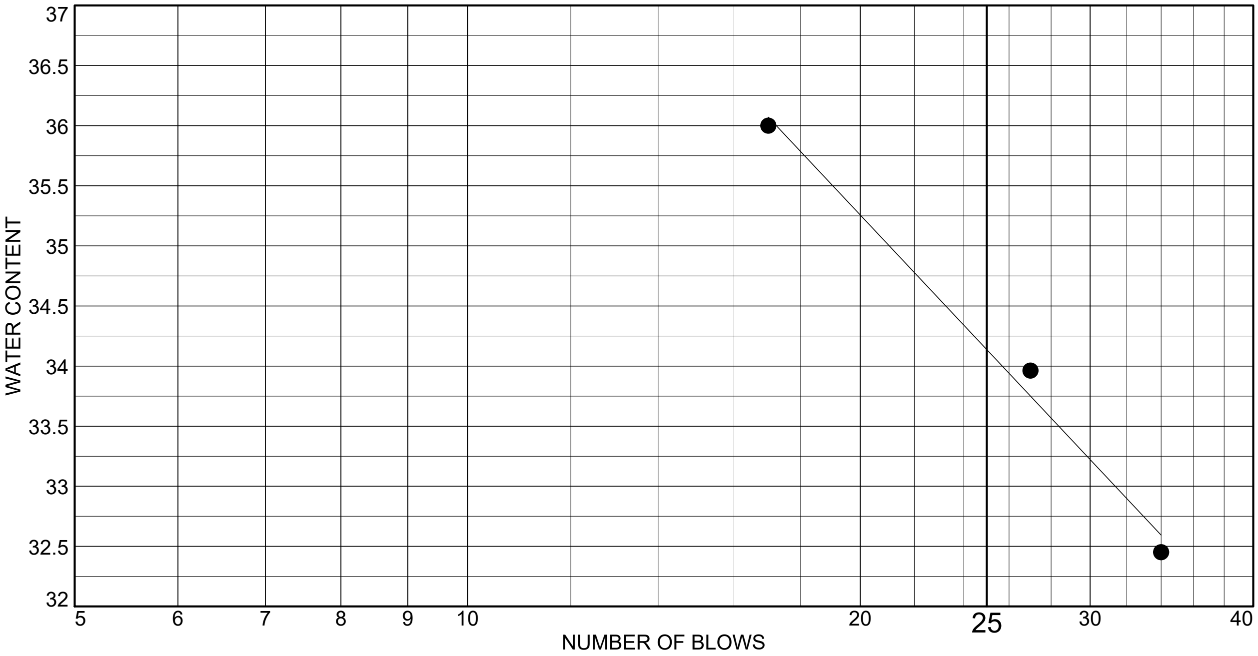
Asheville, NC

Client: Town of Sylva
Project: Allen Street Slope Failure 2.0
Project No: JA20-4021-02 **Figure** PS B-13b

Tested By: LB **Checked By:** IJ

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Reddish Brown, Sandy SILT	34	27	7	80.1	57.7	ML

Project No. JA20-4021- **Client:** Town of Sylva

Project: Allen Street Slope Failure 2.0

Source of Sample: Onsite **Depth:** 3.5' - 5.0'
Sample Number: 2

Kessel Engineering Group

Asheville, NC

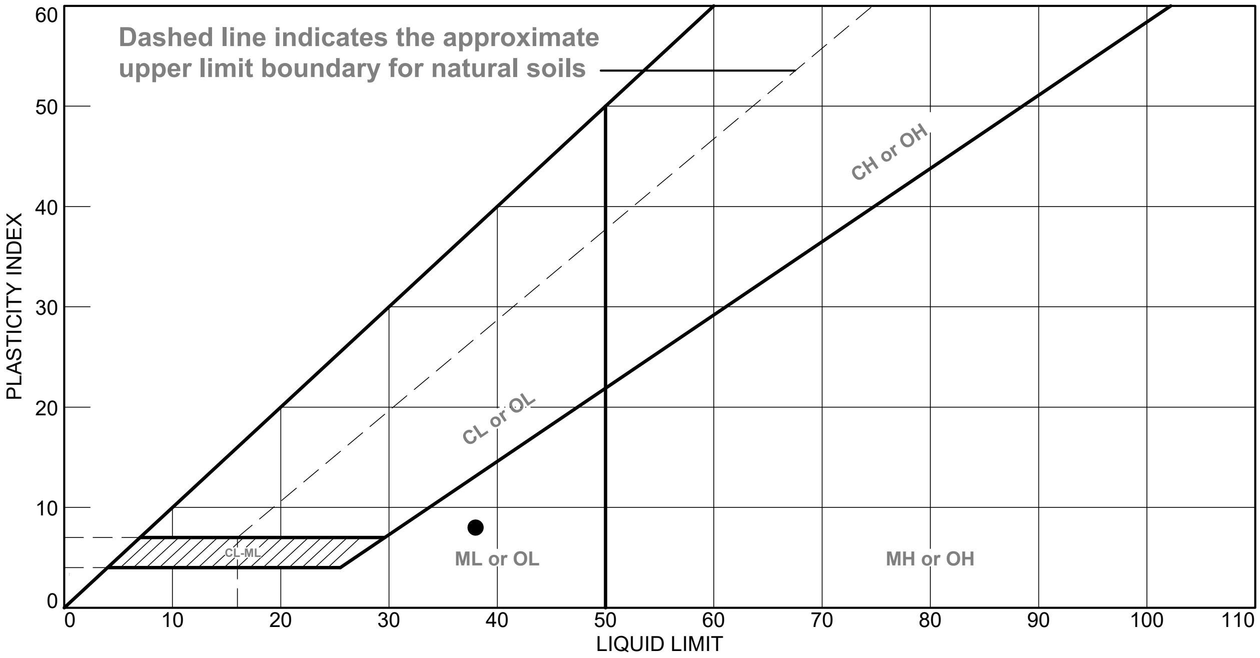
Remarks:

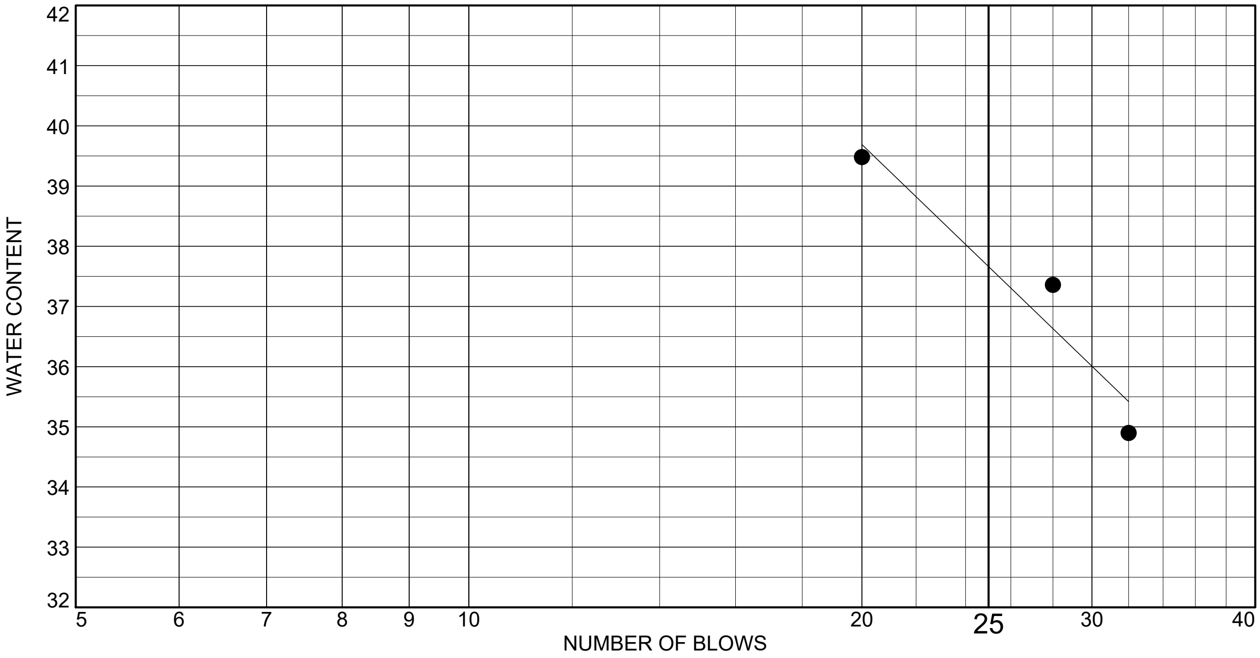
● Sampled at B-12

Figure PI B-12

Tested By: LB **Checked By:** IJ

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Reddish Brown, Sandy SILT	38	30	8	87.3	57.1	ML

Project No. JA20-4021- **Client:** Town of Sylva

Project: Allen Street Slope Failure 2.0

Source of Sample: Onsite **Depth:** 28.5' - 30'
Sample Number: 4

Kessel Engineering Group

Asheville, NC

Remarks:
● Sampled at B-13

Figure PI B-13b

Tested By: LB **Checked By:** IJ