RETAINING WALL AND
ACID PRODUCING ROCK EVALUATION REPORT
STATE ROUTE 29 WIDENING FROM STATE ROUTE 61
TO 0.6 MILE SOUTH OF WHETSTONE ROAD
MORGAN AND ROANE COUNTIES, TENNESSEE
TDOT PE No. 65001-1256-14
GES File No. 6520300
PIN 101411.01
ARCADIS Project No. CT052497.0009.00SME
S&ME Project No. 1811-12-177A

Prepared For:

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June 19, 2013

June 19, 2013

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Subject: Retaining Wall and Acid Producing Rock Evaluation Report
State Route 29 Widening From State Route 61
To 0.6 Mile South of Whetstone Road
Morgan and Roane Counties, Tennessee
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Dear Mr. Butler:

S&ME, Inc. has completed the subsurface exploration for the above referenced project site in Morgan and Roane Counties, Tennessee. We conducted this exploration in general accordance with our Proposal No. 1113020 dated January 22, 2013.

This report presents our understanding of the project, documents our findings, and presents our recommendations for the planned retaining walls on State Route 29 between Stations 154+66 and 230+75, and along the east side of the portion of Coal Hill Road where a realignment is planned. Each of the retaining walls will be constructed right of the centerline. After you have reviewed our report, we recommend either a meeting or a telephone conference to discuss our recommendations.

S&ME, Inc. appreciates the opportunity to be of service to Arcadis, U.S., Inc. and the Tennessee Department of Transportation, and we look forward to helping you through project completion. Please contact us if you have any questions.

Respectfully submitted,

S&ME, Inc.

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Enclosures

FORTY YEARS OF ENGINEERING INTEGRITY

RETAINING WALL AND ACID PRODUCING ROCK EVALUATION REPORT
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1.0 EXECUTIVE SUMMARY

This report presents the results of two subsurface explorations performed to evaluate the presence of acid producing rock and to provide subsurface information for the planned retaining walls on State Route 29 in Morgan and Roane Counties, Tennessee. Laboratory acid base accounting tests were performed by others on samples of the soil and rock encountered at depths of about every 5 feet in the test borings. The laboratory test results indicate the presence of acid producing rock (APR) and soil in the planned cut areas. To decrease the amount of excavation that will be required to construct the planned roadway widening project (and therefore decrease the amount of APR that will require disposal), five retaining walls will be constructed. Each of the walls will be a cut wall that will support a predominantly rock cut, although soil was encountered the full boring depth at a few of the boring locations.

Based on the test boring results, the provided project information, and TDOT Special Provision 624, the acceptable wall type for this project is an anchor wall. Additional wall design and construction recommendations are presented in this report. Attached to this report are the Retaining Wall Sheets, Test Boring Records, rock core photographs, and the geotechnical and acid base accounting laboratory test results.

This summary cannot be used by itself for planning, design, or construction. The necessary information is included within the body of this report.

2.0 INTRODUCTION

The project will consist of the widening of State Route 29 from two to four lanes starting at State Route 61 and extending to 0.6 mile south of Whetstone Road. The widening project will extend a total distance of about three miles. To widen the road, cuts on the east side (right of the centerline) of existing State Route 29 will be required. The results of acid base accounting laboratory testing performed on soil and rock core samples collected in conjunction with our Report of Acid Producing Rock for this project dated January 4, 2013 indicate acid producing rock (APR) is present in planned cut areas. To decrease the amount of excavation that will be required to construct the planned roadway widening project (and therefore decrease the amount of APR that will require disposal), retaining walls will be constructed in five areas. The retaining

walls will be constructed between State Route 29 Stations 154+66 and 230+75 and along the planned realignment of the west end of Coal Hill Road, right of the centerline. The approximate beginning and ending Station Numbers for the walls are as follows:

- State Route 29 Station 154+66 to 167+10
- Coal Hill Road Station 40+60 to 48+17
- State Route 29 Station 180+00 to 183+25
- State Route 29 Station 184+10 to 191+43
- State Route 29 Station 222+37 to 230+75

The provided drawings indicate the State Route 29 Station 154+66 to 167+10 wall will consist of two tiers about 15 feet apart. The lower tier wall will extend from about Station 154+66 to about Station 167+10 and will have a maximum above grade height of about 50 feet. The maximum top of wall elevation will be about 821.8 feet at about Station 167+10. Current site grades in the planned lower wall tier construction area range from about 770 feet near Station 156+00 to about 858 feet at Station 166+00. The upper tier wall will extend from about Station 158+17 to 167+10 and will have a maximum above grade height of about 50 feet. The maximum top of wall elevation for the upper tier will be about 873 feet near Station 167+10. Current site grades in the upper wall tier construction area range from about 825 feet near Station 160+50 to about 865 feet at Station 166+00.

The Coal Hill Road Station 40+60 to 48+17 wall will have three tiers that will be constructed about 15 feet apart. The lower tier wall will extend from about Coal Hill Road Station 40+60 to 48+17 and will have a maximum above grade height of about 50 feet. The maximum top of wall elevation will be about 838 feet at Station 44+50. Current site grades in the lower tier wall construction area range from about 775 feet at Station 48+17 to about 902 feet at Station 41+63. The middle tier wall will extend from about Station 40+75 to 46+32 and will have a maximum above grade height of about 50 feet. The maximum top of wall elevation will be about 886 feet at Station 43+50. Current site grades in the middle tier wall construction area range from about 833 feet at Station 46+32 to about 906 feet at Station 42+00 The upper tier wall will extend from about Station 40+84 to 44+00 and will have a maximum above grade height of about 43 feet. The maximum top of wall elevation will be about 922 feet at Station 42+00. Current site grades in the upper tier wall construction area range from about 875 feet at Station 40+84 to about 915 feet at Station 42+00.

The State Route 29 Station 180+00 to 183+25 wall will be a single wall with a maximum above grade height of about 19 ½ feet. The maximum top of wall elevation will be about 796 feet at Station 181+00. Current site grades in the planned wall construction area range from about 775 feet at Station 183+25 to about 792 feet at Station 181+50.

The State Route 29 Station 184+10 to 191+43 wall will be a single wall with a maximum above grade height of about 37 feet. The maximum top of wall elevation will be about 813 feet at Station 187+50. Current site grades in the planned wall area range from about 776 feet at Station 191+43 to about 811 feet at Station 187+50.

The State Route 29 Station 222+37 to 230+75 wall will be a single wall with a maximum above grade height of about 31 feet. The maximum top of wall elevation will be about 804 feet at Station 228+50. Current site grades in the wall area range from about 766 feet at Station 222+37 to about 798 feet at Station 228+50.

The current project included drilling test borings with rock coring in the planned retaining wall areas and performing acid base accounting laboratory testing on soil and rock core samples recovered from the test borings. The results of the exploration and testing performed for the current project and for the January 4, 2013 Report of Acid Producing Rock for this site are presented in this report.

3.0 GEOLOGY AND SITE CONDITIONS

3.1 RETAINING WALL SITE GEOLOGY

The southernmost portion of the project site is located in the Valley and Ridge Physiographic Province. Elongated ridges that trend in a northeast-southwest direction characterize this province. The ridges are typically formed on highly resistant sandstones and shales, while the valleys and rolling hills are formed on less resistant limestone, dolomite, and shales.

Based on our review of the Geologic Map of Tennessee, East Central Sheet, dated 1966, geologic formations found in the Valley and Ridge Physiographic Province extend from the beginning of the project north to about Station 165+00 of State Route 29. This portion of the site is underlain by undifferentiated formations of the Chickamauga Group. In the northwest part of the Valley and Ridge, the bedrock of this group is predominantly limestone about 2,000 feet thick, and produces a thin residuum typically less than 15 feet thick.

The remainder of the project site is located in the Cumberland Plateau Physiographic Province. The Cumberland Plateau is a highland area extending across Tennessee east of the Highland Rim and west of the Valley and Ridge Physiographic Province. The 30 to 60 mile wide area is dissected by streams extending from prominent mountains into valleys.

The East Central Sheet of the Geologic Map of Tennessee indicates two geologic formations are present in the planned retaining wall construction areas north of about Station 165+00. From south to north, these formations include the Crab Orchard Mountains and Gizzard Group, which extends from about Station 165+00 to about Station 170+45, and the Rockcastle Conglomerate, which underlies the retaining wall construction areas north of about Station 170+45.

The Crab Orchard Mountains Group includes the Rockcastle Conglomerate, Vandever Formation, Newton Sandstone, and Whitwell Shale and Sewanee Conglomerate formations. The Gizzard Group includes the Signal Point Shale, Warren Point Sandstone, and Raccoon Mountain formations. These formations consist of sandstone, conglomerate, siltstone, shale and thin coal beds. The thicknesses of these formations range from about 1,200 to 1,400 feet.

The Rockcastle Conglomerate, which is included in the Crab Orchard Mountains and is also mapped individually in the project area, is composed of conglomeratic sandstone and sandstone that is gray to brown and fine- to coarse-grained. Thin coal-bearing shale is locally present near the middle of the formation. The formation ranges from about 150 to 220 feet in thickness.

3.2 EXISTING SITE CONDITIONS IN PLANNED RETAINING WALL AREAS

In general, the planned retaining wall construction areas are rugged, steeply sloping and wooded. Rock outcrops are visible in several areas. The ground elevation slopes down from the east toward existing State Route 29, and varies in the retaining wall construction areas from about 770 feet near State Route 29 Station 156+00 to about 915 feet at Coal Hill Road Station 42+00.

4.0 SUBSURFACE EXPLORATION AND CONDITIONS

4.1 SUBSURFACE EXPLORATION PROCEDURES

The procedures used by S&ME for field and laboratory sampling and testing are in general accordance with AASHTO and ASTM procedures and established engineering practice in the State of Tennessee. The approximate boring locations are shown on the Retaining Wall Conceptual Drawings presented in Appendix I of this report. The field testing procedures, Test Boring Records, and photographs of the rock core recovered from the test borings are presented in Appendix II, and the laboratory test procedures and test results are presented in Appendix III.

Subsurface conditions in the vicinity of the proposed retaining walls were explored with 23 soil test borings. The locations where the borings were drilled were restricted to areas that could be accessed by all terrain and track mounted drilling equipment. A bulldozer was also necessary to clear access to and perform limited grading at the boring locations.

An ARCADIS survey crew staked the field locations of borings drilled in association with our January 4, 2013 report for this site. The borings were drilled right of the planned roadway centerline at the following approximate State Route 29 Station Numbers: 160+00, 163+26, 166+00, 168+00, 172+00, 186+00, 189+00, 225+50, and 227+50. The elevations of these borings were provided to us by Arcadis. The remaining 14 boring locations were drilled in association with the current project and were located in the field by one our engineers using a hand held GPS device referenced to the Tennessee state plane coordinate system. The locations of these 14 borings were superimposed onto the provided topographic drawing which was used to

estimate the ground surface elevation at the boring locations. The boring locations are depicted on the attached Retaining Wall Conceptual Drawings in Appendix I, and the elevations are shown on the Test Boring Records in Appendix II.

The drillers took soil samples from the borings using a split-barrel sampler according to AASHTO T 206 on a maximum 2 ½ foot interval in the upper 10 feet and on 5 foot intervals thereafter. An engineer logged the results of the Standard Penetration Tests (SPTs). After the borings were completed, or upon encountering auger refusal and prior to coring rock in 21 of the borings, we checked the boreholes for the presence of groundwater. The borings were backfilled with auger cuttings and a borehole plugging device before leaving the site.

Our field engineer logged the soil and rock core samples and photographed the rock core, selected soil and rock core samples from each boring on an about five foot interval for acid base accounting laboratory testing, packaged the samples in sealed containers, and labeled them for identification. We visually classified the soil samples according to the Unified Soil Classification System (ASTM D 2488) and measured the rock core recovery and the Rock Quality Designation (RQD). The rock core recovery is a measure of the length of core drilled to that recovered expressed as a percentage. The RQD is a measure of rock quality based on the percent of the core run containing pieces at least 4 inches long. The resulting soil and rock descriptions are shown on the Test Boring Records in Appendix II. A general description of the subsurface conditions encountered at the boring locations is provided below in Section 4.2.

The samples selected for acid base accounting laboratory testing were transported to Galbraith Laboratories, Inc.'s Knoxville office. The remaining samples were transported to the TDOT Region 1 office in Knoxville for storage.

4.2 SUBSURFACE CONDITIONS

General: The results of our field testing program and the moisture content and plasticity index laboratory test results are shown on the Test Boring Records in Appendix II. These records present our interpretation of the subsurface conditions at specific boring locations at the time of our exploration. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

State Route 29 Station 154+66 to 167+10 Wall

Six test borings were drilled in the vicinity of this wall. The borings were drilled at the following approximate Station Numbers:

- Station 154+90, 106 feet right
- Station 156+33, 95 feet right
- Station 160+00, 150 feet right
- Station 161+23, 170 feet right
- Station 163+26, 165 feet right
- Station 166+00, 157 feet right

In addition to the test borings, three auger borings were drilled to depths ranging from about 6 to 16 feet in an effort to obtain undisturbed Shelby tube samples suitable for laboratory testing.

The Station 154+90 boring initially encountered about ½ foot of topsoil. Ground cover was not encountered in the remaining borings drilled for this wall.

At Stations 161+23 and 166+00, the borings initially encountered fill to depths of about 6 and 6 ½ feet, respectively. The fill encountered at Station 161+23 was composed of clayey sand with sandstone fragments. Standard Penetration Test (SPT) N values in the fill ranged from 32 to 55 blows per foot, indicating a dense to very dense granular soil consistency.

At Station 166+00, auger refusal was encountered at a depth of about 1 ½ feet in fill consisting of sandy clay. Rock coring was begun, and fill consisting of sandstone boulders and sandy clay was encountered to a depth of about 6 ½ feet. Our field engineer visually evaluated the sandstone boulders as moderately hard and the sandy clay as soft in consistency.

Below the topsoil at Station 154+90, below the fill at Station 161+23, and from the ground surface at the remaining boring locations except for Station 166+00, residual soils were encountered to auger refusal. The residual soils typically consisted of silty or sandy clay, sandy silt and clayey sand with rock and sandstone fragments. SPT N values in the residuum ranged from 4 to greater than 50 blows per foot, indicating a soft to hard soil consistency for the predominantly clay and silt soils, and a loose to very dense soil consistency for the predominantly sand soils.

In the borings drilled at Stations 156+33 and 161+23, auger refusal was encountered at depths of about 8 ½ feet and 19 ½ feet, respectively. Rock coring was begun, and residual materials were encountered. In the Station 156+33 boring, the residual materials consisted of sandy clay with floating sandstone boulders to a depth of about 22 feet. In the Station 161+23 boring, the residual materials consisted of sandstone and sandy clay to a depth of about 41 feet, and silty clay with shale fragments from about 41 to 70 feet. In both of the borings, the sandy and silty clay was observed washing out in the coring water during rock coring.

Auger refusal was encountered in the test borings at depths ranging from about 1 ½ feet to 25 ½ feet below the existing ground surface. Rock coring was performed in each of the borings to depths ranging from about 30 ½ to 155 feet. As previously discussed, the rock core samples were logged by one of our field engineers, who measured the recovery and RQD and also photographed the rock core.

Below the previously described fill and residuum encountered below auger refusal in the Station 156+33, 161+23 and 166+00 borings, and below auger refusal at the remaining boring locations, bedrock consisting of siltstone, sandstone, and shale was encountered to boring termination. Soil and coal seams were observed in some of the recovered core. The rock core recovery ranged from 0 to 100 percent. Of the 102 core runs, 82 (about 53 percent) of the core runs had recoveries of 90 percent or greater. The RQD for the rock core ranged from 0 to 100

percent, with about 53 percent of the RQD values ranging from 75 to 100 percent, indicating a predominantly good to excellent rock quality.

Coal Hill Road Station 40+60 to 48+17 Wall

The Coal Hill Road wall area was explored with four test borings drilled at the following approximate Station Numbers:

- SR 29 Station 168+00, 170 feet right (Coal Hill Road Station 41+67, 26 feet right)
- Coal Hill Road Station 43+68, 12 feet right
- SR 29 Station 172+00, 100 feet right (Coal Hill Road Station 45+74. 68 feet left)
- Coal Hill Road Station 46+70, 25 feet right

Ground cover consisting of less than ½ foot of topsoil was encountered at the Station 46+70 boring. Below the ground surface, the Station 43+68 boring encountered about 1 ½ feet of fill consisting of soft consistency sandy clay with rock fragments. Below the topsoil at Station 46+70, below the fill at Station 43+68, and below the ground surface at the other two boring locations, residual materials were encountered to auger refusal. The residual materials consisted of sandy clay, silt clay, silty sand, and weathered shale or sandstone. SPT N values in the residuum ranged from 4 to greater than 50 blows per foot, indicating a soft to hard consistency for the predominantly clay, silt and shale materials, and a very firm to very dense consistency for the predominantly sand and sandstone materials.

Auger refusal was encountered in the test borings at depths ranging from about 7 ½ feet to 25 ½ feet below the existing ground surface. Rock coring was performed in each of the borings to depths ranging from about 60 to 140 feet. The bedrock consisted of weathered sandstone, sandstone, weathered shale, shale, sandy siltstone, and siltstone. Clay, shale and coal seams were observed in some of the recovered rock core. The rock core recovery ranged from 53 to 100 percent. Of the 52 core runs, 43 (about 83 percent) of the core runs had recoveries of 90 percent or greater. The RQD for the rock core ranged from 0 to 100 percent, with about 58 percent of the RQD values ranging from 75 to 100 percent, indicating a predominantly good to excellent rock quality.

State Route 29 Station 180+00 to 183+25 Wall

Three test borings were drilled in this wall area at the following approximate locations:

- SR 29 Station 180+19, 96 feet right
- SR 29 Station 181+72, 87 feet right
- SR 29 Station 183+05, 89 feet right

In addition to the test borings, one auger boring was drilled near the Station 180+19 test boring to a depth of about 4 feet in an effort to obtain undisturbed Shelby tube samples suitable for laboratory testing. Also, an attempt was made to collect an undisturbed sample in the boring drilled near Station 181+72.

Below the ground surface, residual materials were encountered to auger refusal. The residual materials were composed of silty clay, sandy clay, rock fragments and weathered shale. SPT N values in the residuum ranged from 1 to greater than 50 blows per foot, indicating very soft to hard material consistencies.

Auger refusal was encountered in the test borings at depths ranging from about 5 ½ to 28 feet below the existing ground surface. Rock coring was performed in each of the borings to depths ranging from about 25 to 40 feet. The recovered rock core consisted of sandstone and siltstone. Clay and shale seams were observed in some of the recovered core. Recovery ranged from 40 to 100 percent, with nine (about 69 percent) of the recovery values for the 13 core runs drilled in these borings ranging from 90 to 100 percent. RQD values ranged from 0 to 89 percent, with only five (about 38 percent) of the values falling in the good to excellent rock quality ranges.

State Route 29 Station 184+10 to 191+43 Wall

Five test borings were drilled in this wall area at the following approximate locations:

- SR 29 Station 184+61, 56 feet right
- SR 29 Station 186+00, 60 feet right
- SR 29 Station 187+63, 49 feet right
- SR 29 Station 189+00, 80 feet right
- SR 29 Station 190+66, 48 feet right

About one foot of topsoil was encountered at the Station 186+00 and 189+00 borings. Below the topsoil in these two borings, and from the ground surface in the other three borings, residual materials were encountered to auger refusal. The residual materials consisted of silty and sandy clay, sandy silt, clayey sand, rock fragments, and weathered sandstone. SPT N values in the residual materials ranged from 3 to greater than 50 blows per foot, indicating soft to hard consistencies for the clays and silts, and a very dense consistency for the clayey sand and weathered sandstone.

Auger refusal was encountered in each of the borings at depths ranging from about 3 to 7 ½ feet. Rock coring was performed in each of the borings to depths of about 30 to 50 feet below the existing ground surface. The recovered rock core consisted of sandstone, shale and weathered shale. Clay seams were present in some of the recovered core samples. Rock core recovery ranged from 50 to 100 percent for the 37 core runs drilled in these borings. Twenty one (about 57 percent) of the recovery values were greater than 90 percent. RQDs ranged from 0 to 94 percent, with seven (about 19 percent) falling in the good to excellent rock quality ranges.

State Route 29 Station 222+37 to 230+75 Wall

Five test borings were drilled in the vicinity of this wall at the following approximate locations.

- SR 29 Station 221+89, 110 feet right

- SR 29 Station 225+50, 120 feet right
- SR 29 Station 226+60, 67 feet right
- SR 29 Station 227+50, 180 feet right
- SR 29 Station 229+04, 68 feet right

In addition to the test borings, three auger borings were drilled to depths of about 6 and 8 feet in an effort to obtain undisturbed Shelby tube samples suitable for laboratory testing.

Below the ground surface, residual materials were encountered to boring termination or auger refusal. The residual materials consisted of silty and sandy clay, clayey and sandy silt, weathered shale, silty sand, and rock and sandstone fragments. A sandstone boulder was encountered from the ground surface to a depth of about 1 ½ feet in the Station 227+50 boring. SPT N values ranged from 4 to greater than 50 blows per foot, indicating a soft to hard material consistency for the clays, silts, and weathered shale, and a firm to very firm consistency for the sand.

The borings drilled near Stations 221+89 and 226+60 were terminated in weathered shale at a depth of about 35 ½ feet. The remaining three borings encountered auger refusal at depths ranging from about 3 ½ to 39 feet. Rock was cored in these borings to depths ranging from about 41 to 86 feet. In the Station 229+04 boring, sandy clay mixed with sandstone boulders was encountered from auger refusal at a depth of about 15 feet to a depth of about 27 feet. Below this material, and below auger refusal in the Station 225+50 and 227+50 borings, bedrock composed of siltstone, weathered sandstone and weathered shale was encountered. Fine sand, clay seams and pyrite was observed in some of the recovered core. Rock core recovery for the 28 core runs drilled in these borings ranged from 6 to 100 percent, and eight (about 29 percent) of these values were greater than 90 percent. RQD values ranged from 0 to 100 percent. Five (about 18 percent) of the RQD values were in the good to excellent rock quality range.

4.3 GROUNDWATER

Groundwater was encountered in the following three test borings at the time of drilling and prior to coring rock:

- Coal Hill Road Station 43+68, 12 feet right: 24 feet
- State Route 29 Station 225+50, 120 feet right: 35 feet
- State Route 29 Station 226+60, 67 feet right: 26 feet

Delayed groundwater level measurements are shown on some of the test borings. However, the introduction of water into the borings during the rock coring process and rainfall during the time our field services were performed likely influenced the delayed measurements. It should be noted that groundwater levels can fluctuate with seasonal, climatic, and environmental changes. Further, groundwater may be encountered at additional locations where we drilled test borings at some future time.

5.0 ACID PRODUCING ROCK EVALUATION

5.1 ACID BASE ACCOUNTING LABORATORY TEST RESULTS

Soil and rock core samples were selected from each boring on an about five foot interval and were delivered to Galbraith Laboratories, Inc.'s Knoxville office for acid base accounting laboratory testing. The test results were used to characterize the material's potential to produce acidic runoff based on TDOT's accepted criteria, Guideline for Acid Producing Rock Investigation, Testing and Monitoring, and Mitigation, prepared by Golder Associates, Inc. and dated October 2007. The test results indicated 236 of the 306 tested samples were non Acid Producing Rock (APR), 9 were potential APR and 61 were APR. A tabulation of the laboratory test results is presented in Appendix III. The Retaining Wall Conceptual Drawings in Appendix I include boring profiles that show the APR and potential APR as do the Test Boring Records included in Appendix II. A summary of the test results in each wall area is presented in the following paragraphs.

State Route 29 Station 154+66 to 167+10 Wall

Of the 121 soil and rock core samples tested from the borings drilled in this wall area, the test results indicated three samples met the criteria for potential APR and 11 samples met the criteria for APR. Four of the samples were soil or a mix of soil and rock consisting of sandy clay with rock fragments, clayey silt, and silty clay with shale fragments. The remaining 10 samples were rock including weathered shale, shale, siltstone with occasional shale seams, shale with coal seams and some interbedded sandstone, and sandstone with thin shale and coal seams. One coal sample was tested.

Two shale rock samples tested from the Station 156+33 boring had greater than 0.1 percent pyritic sulfur, but the net neutralization potentials were about 18 and 23.5 percent, which are greater than the limit of 12 set for potential APR and less than zero set for APR in the Golder document.

Coal Hill Road Station 40+60 to 48+17 Wall

Of the 73 soil and rock core samples tested from the four borings drilled in this wall area, 16 met the criteria for APR, with 15 of them being from the Station 43+68 boring where all but six of the 21 tested samples met the criteria for APR. One soil sample collected from the Station 46+70 boring met the criteria for APR out of 12 samples tested from this boring. The APR samples included silty clay with weathered shale, shale and sandy siltstone with shale interbeds.

None of the samples tested from the Station 168+00 (CHR Station 41+67) and Station 172+00 (CHR Station 45+74) borings met the criteria for potential APR or APR. However, coal seams and shale interbeds were present in some of the untested rock core recovered from these borings.

State Route 29 Station 180+00 to 183+25 Wall

Twenty soil and rock core samples were tested from the borings drilled in this wall area. Six of the samples met the criteria for APR. In the boring drilled at Station 183+05, three out of five tested samples were classified as APR. Of the six samples that met the criteria for APR in this wall area, three were soil and three were rock core. The soil included silty and sandy clay with rock fragments and weathered shale. The rock included sandstone and siltstone with shale seams.

State Route 29 Station 184+10 to 191+43 Wall

Of the 42 samples tested from the borings drilled in this wall area, 19 met the criteria for APR, including 4 soil and 15 rock core samples. In the Station 189+00 boring, seven of the 10 tested samples were APR, and in the Station 190+66 boring, four of the six tested samples were APR. The soil consisted of sandy clay and sandy silt with rock fragments. The rock consisted of sandstone with thin shale seams, interbedded shale and sandstone, and sandstone.

State Route 29 Station 222+37 to 230+75 Wall

Fifty samples were tested from the borings drilled in this wall area. Six of the samples met the criteria for potential APR and nine for APR. In the boring drilled at Station 221+89, three of the seven tested samples met the criteria for APR and one sample met the criteria for potential APR. Of the 15 samples that met the criteria for potential APR and APR, nine were soil, one was a mix of soil and rock, and five were rock core. The soil consisted of sandy clay with rock fragments and sandstone boulders, weathered shale, and clayey and sandy silt with claystone, sandstone and weathered shale fragments. The rock was composed of siltstone with thin pyrite veins and pyrite inclusions.

5.2 APR CONCLUSIONS AND RECOMMENDATIONS

Based on the laboratory test results, it appears that the presence of shale in the bedrock as well as the obvious APR producers coal and pyrite will likely result in potential APR and APR materials that will require special handling during construction. Since the pyrite may not be visible in the bedrock during blasting and removal from the site (it was not observed in most of the samples with positive APR test results), and the coal seams may be relatively thin, we recommend field testing be performed at the time of construction to identify additional APR materials at this site. We recommend test samples be collected from blast boreholes and soil excavated from cut areas, and the samples be collected and tested as described in Section 3.3.1.2 of the Golder document. Based on our experience with this project, S&ME should be retained to provide sample collection during the excavation process.

On cut slopes where APR and potential APR materials will be exposed after design grades are met, The Best Management Practices (BMPs) described in Section 5.2 of the Golder document should be incorporated into the design and construction documents. These include steepening slopes as much as possible without compromising geotechnical stability or overall public safety to reduce the surface area of exposed APR materials, the placement of an interval of non-acidic

material in direct contact with the APR followed by an application of soil/plant material, and intermediate benches that are designed to be free-draining.

5.3 APR QUANTITY ESTIMATIONS

The acid base accounting test results were used to estimate the quantities of potential APR and APR in the planned retaining wall and cut areas. The test results were interpolated between the widely spaced borings, and the quantities were estimated using the average end area method. Employing these methods resulted in the following material quantity estimates:

- Estimated APR requiring encapsulation: 218,000 cubic yards
- Estimated APR requiring blending or encapsulation: 14,000 cubic yards
- Estimated Potential APR: 9,000 cubic yards

6.0 RECOMMENDATIONS

Unless specifically stated otherwise in the contract plans, the bidding for, the design of and the construction of retaining walls shown in the plans shall be governed by the Tennessee Department of Transportation Special Provision 624 regarding retaining walls. This special provision shall be considered as one of those documents which the bidder/contractor has examined and made himself familiar with as described in Section 102.04 – Examination of the Site, the Work, the Plans and the Specifications in the TDOT Standard Specifications for Road and Bridge Construction.

Excavation for the walls and/or their footings shall not be accomplished until the contractor has submitted wall designs and calculations and has been issued an approved set of wall plans and has labor and material resources available to begin and continue wall construction immediately after excavation.

These walls shall be designed in accordance with LRFD design procedures and requirements as described in AASHTO LRFD Bridge Design Specifications, 2012 and Interims.

For proprietary wall systems that have been approved as shown in Special Provision 624, the wall designer shall be responsible for providing wall designs incorporating materials and component as was originally submitted and approved by TDOT. If a material and/or component of the wall system have been modified from the originally approved system, a wall design and set of plans and calculations for this wall system cannot be submitted for review and approval until the wall system designer who originally submitted the wall system for approval by TDOT submits a request for re-approval utilizing the modified elements of the wall. This submittal does not guarantee approval of the modified system. If this re-approval process does not meet the contractor’s schedule or if the modified system is not approved, the contractor/wall designer shall provide a wall design for one of the approved systems at no change in contract price for the retaining wall and no change in project schedule requirements will be allowed.

The wall designer shall provide retaining wall plans, details and calculations as required by Special Provision 624 and as required herein.

- The wall designer shall utilize the geotechnical parameters and resistance factors as provided for each project retaining wall on the wall concept sheet and related retaining wall sheets to prepare and submit design calculations. Load factors and other pertinent design requirements provided in AASHTO LRFD Bridge Design Specifications, 2012 and Interims shall be used.

- Calculations for both internal (bearing capacity, sliding, settlement) and external (global) stability shall be provided for each critical wall section which demonstrates the required capacity to demand ratio of 1.0 is met utilizing the design parameters provided. The wall designer/contractor plans must include any foundation improvements as required herein on the wall designer/contractor's wall elevation views and any cross-sectional detail drawings.

- Load combinations as given in AASHTO LRFD Bridge Design Specifications, 2012 and Interims, shall be evaluated for each wall.

NOTE REGARDING CONSTRUCTION SLOPES

The contractor shall be responsible for making the excavation in accordance with OSHA and other applicable state and local regulations regarding construction slopes and trenches, in addition to the following applicable regulatory requirements: as a minimum requirement, all temporary construction slopes required for site access shall not exceed three (3) vertical feet. The contractor building the wall shall ensure that these temporary back slopes are not and do not become unstable. If the slope is unstable, becomes unstable, is cut steeper than a 1:1 slope or is unacceptable for another reason, then temporary shoring shall be used. Any unusual soil conditions other than those estimated should be reported to the project engineer.

ACCEPTABLE WALL TYPES

The retaining walls shall be the wall type listed below. The specific wall system supplier/installer shall be one of those listed as pre-approved in Special Provision 624.

- Anchor Wall

TABLE 1 – DESIGN REQUIREMENTS AND PARAMETERS

DESCRIPTION	VALUE – ANCHOR WALLS	NOTE *
Design Life	75 years	
Seismic Acceleration Coefficient (A_s)	0.15	
Pullout Resistance of Anchors		
Cohesionless (granular) soils	0.65	1
Cohesive soils	0.70	1
Rock	0.50	1
Where proof tests are conducted	1.0	2
Tensile Resistance of Anchor Tendon		
Mild steel (e.g., ASTM A615 bars)	0.90	3
High strength steel (e.g., ASTM A722 bars)	0.80	3
Flexural Capacity of Vertical Elements	0.90	
Resistance Factors of a Single Driven Pile Static Analyses Methods		
Side resistance and end bearing: clay and mixed soils		
α-Method (Tomlinson, 1987; Skempton, 1951)	0.35	
β-Method (Esrig & Kirby, 1997; Skempton, 1951)	0.25	
λ-Method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40	
Side resistance and end bearing: sand		
Nordlund/Thurman Method (Hannigan et al, 2005)	0.45	
SPT-Method (Meyerhof)	0.30	
CPT-Method (Schmertmann)	0.50	
End bearing in rock (Canadian Geotech Society, 1985)	0.45	
Lateral geotechnical resistance of a single pile		
All soils and rock	1.0	
Resistance Factors of a Single Drilled Pile/Shaft		
Side resistance in clay		
α-Method (O'Neill and Reese, 1999)	0.45	
Tip resistance in clay		
Total stress (O'Neill and Reese, 1999)	0.40	
Side resistance in sand		
β-Method (O'Neill and Reese, 1999)	0.55	
Tip resistance in sand		
O'Neill and Reese, 1999	0.50	
Side resistance in intermediate geomaterials (IGMs)		
O'Neill and Reese, 1999	0.60	
Tip resistance in intermediate geomaterials (IGMs)		
O'Neill and Reese, 1999	0.55	
Side resistance in rock		
Horvath and Kenney (1979), O'Neill and Reese, 1999	0.55	
Carter and Kulhawy (1988)	0.50	
Tip resistance in rock		
Canadian Geotech Society (1985), O'Neill and Reese, 1999	0.50	
Lateral geotechnical resistance of a single pile/shaft		
All materials	1.0	
*Refer to Table 1.1 for notes.		

TABLE 1.1	
NOTES FOR TABLE 1	
NO.	**NOTE**
1	Apply to presumptive ultimate unit bond stresses for preliminary design only.
2	Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.
3	Apply to maximum proof test for anchors.
	Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating an extreme event limit state.

OTHER DESIGN REQUIREMENTS

Soldier pile/tie back walls shall have permanent structural concrete wall facing capable of resisting all design loads exerted on the wall components by the retained material. The mechanism connecting the soldier piles to the structural concrete facing shall have adequate capacity to transmit all loads from the facing, including self-weight, to the supporting soldier piles. Concrete lagging can be designed/used without application of permanent cast in place facing.

Timber lagging shall only be used to temporarily support material to allow for top down construction. Timber lagging is neither an acceptable permanent wall facing nor an acceptable permanent structural wall element.

The wall shall have a concrete drainage ditch at the top designed to carry surface runoff to either or both ends of walls. See Sheets 2Q and 2R of the Roadway Construction Plans for details on this concrete drainage ditch.

The wall designer/contractor's wall design plans shall include details for drainage behind the wall facing. Weep holes shall be constructed near the toe of walls 4-2, 5-2 and 5-3 to allow water to drain. See Roadway Construction Plans Sheet 2R for more information on weep hole outlet drainage. Weep holes shall not be allowed for walls 4-1, 5-1, 6, 7 and 8.

CONSTRUCTION NOTES

Assume drilling through in-place rock or colluvium required for installation of soldier piles.

Due to the site geology and the potential corrosive environment at the anchor area, it is recommended that the tendon bond length be encapsulated to provide additional corrosion protection (double corrosion protection). The encapsulation shall be fabricated from one of the following:

I. High density corrugated polyethylene tubing conforming to the requirements of AASHTO M 252 and having a minimum wall thickness of 0.06 inch except pregrouted tendons, which may have a minimum wall thickness of 0.04 inch.

II. Corrugated polyvinyl chloride tubes manufactured from rigid PVC compounds conforming to ASTM D 1784, Class 13646-B.

TABLE 2 – DESIGN PARAMETERS FOR ANCHOR WALL

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	FRICTION ANGLE	COHESION	UNIT WEIGHT
WALL 4-1					
154+66.02 to 159+00	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Soil	25°	200 psf	125 pcf
159+00 to 162+25	Top of wall to top of ground	ASTM D 448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Weathered shale, interbedded shale and sandstone	16°	4,300 psf	150 pcf
162+25 to 164+50	Top of wall to base of wall	Sandstone	47°	65,000 psf	160 pcf
164+50 to 167+09.99	Top of wall to elevation 780	Sandstone	47°	65,000 psf	160 pcf
	Elevation 780 to base of wall	Interbedded shale, sandstone and coal	27°	14,400 psf	155 pcf
WALL 4-2					
158+17 to 160+75	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Sandstone and weathered shale	16°	4,300 psf	150 pcf
160+75 to 162+25	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Soil	25°	200 psf	125 pcf
162+25 to 167+09.99	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Sandstone	47°	65,000 psf	160 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	FRICTION ANGLE	COHESION	UNIT WEIGHT
WALL 5-1					
40+59.65 to 42+50	Top of wall to base of wall	Sandstone	47°	65,000 psf	160 pcf
42+50 to 48+17	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Weathered interbedded shale and siltstone, sandstone with interbedded shale	16°	4,300 psf	150 pcf
WALL 5-2					
40+75.47 to 42+50	Top of wall to base of wall	Sandstone	47°	65,000 psf	160 pcf
42+50 to 44+75	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 850	Soil	25°	200 psf	125 pcf
	Elevation 850 to base of wall	Weathered interbedded shale and siltstone	16°	4,300 psf	150 pcf
44+75 to 46+31.94	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Soil	25°	200 psf	125 pcf
WALL 5-3					
40+84.23 to 44+00	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 890	Soil	25°	200 psf	125 pcf
	Elevation 890 to base of wall	Sandstone	47°	65,000 psf	160 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	FRICTION ANGLE	COHESION	UNIT WEIGHT
WALL 6					
180+00 to 181+00	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 782	Soil	25°	200 psf	125 pcf
	Elevation 782 to base of wall	Weathered sandstone	27°	14,400 psf	155 pcf
181+00 to 183+25	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Soil	25°	200 psf	125 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	FRICTION ANGLE	COHESION	UNIT WEIGHT
WALL 7					
184+10 to 185+25	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 789	Soil	25°	200 psf	125 pcf
	Elevation 789 to base of wall	Sandstone with interbedded shale	27°	14,400 psf	155 pcf
185+25 to 188+25	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 801	Soil	25°	200 psf	125 pcf
	Elevation 801 to base of wall	Sandstone with interbedded shale	27°	14,400 psf	155 pcf
188+25 to 189+75	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 799	Soil	25°	200 psf	125 pcf
	Elevation 799 to base of wall	Sandstone with interbedded shale	27°	14,400 psf	155 pcf
189+75 to 191+43.06	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to elevation 781	Soil	25°	200 psf	125 pcf
	Elevation 781 to base of wall	Sandstone with interbedded shale	27°	14,400 psf	155 pcf
WALL 8					
222+37 to 230+75	Top of wall to top of ground	ASTM D448 No. 57 stone	40°	0 psf	100 pcf
	Top of ground to base of wall	Soil	25°	200 psf	125 pcf

TABLE 3 – RESISTANCE PARAMETERS

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	NOMINAL ANCHOR BOND STRESS, τ_n	UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT
WALL 4-1					
154+66.02 to 159+00	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Soil	0.7 ksf	400 psf	125 pcf
159+00 to 162+25	Top of wall to top of ground	ASTM D 448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Weathered shale, interbedded shale and sandstone	10 ksf	8,600 psf	150 pcf
162+25 to 164+50	Top of wall to base of wall	Sandstone	20 ksf	130,000 psf	160 pcf
164+50 to 167+09.99	Top of wall to elevation 780	Sandstone	20 ksf	130,000 psf	160 pcf
	Elevation 780 to base of wall	Interbedded shale, sandstone and coal	15 ksf	28,800 psf	155 pcf
WALL 4-2					
158+17 to 160+75	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Sandstone and weathered shale	10 ksf	8,600 psf	150 pcf
160+75 to 162+25	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Soil	0.7 ksf	400 psf	125 pcf
162+25 to 167+09.99	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Sandstone	20 ksf	130,000 psf	160 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	NOMINAL ANCHOR BOND STRESS, τ_n	UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT
WALL 5-1					
40+59.65 to 42+50	Top of wall to base of wall	Sandstone	20 ksf	130,000 psf	160 pcf
42+50 to 48+17	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Weathered interbedded shale and siltstone, sandstone with interbedded shale	10 ksf	8,600 psf	150 pcf
WALL 5-2					
40+75.47 to 42+50	Top of wall to base of wall	Sandstone	20 ksf	130,000 psf	160 pcf
42+50 to 44+75	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 850	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 850 to base of wall	Weathered interbedded shale and siltstone	10 ksf	8,600 psf	150 pcf
44+75 to 46+31.94	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Soil	0.7 ksf	400 psf	125 pcf
WALL 5-3					
40+84.23 to 44+00	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 890	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 890 to base of wall	Sandstone	20 ksf	130,000 psf	160 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	NOMINAL ANCHOR BOND STRESS, τ_n	UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT
WALL 6					
180+00 to 181+00	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 782	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 782 to base of wall	Weathered sandstone	15 ksf	28,800 psf	155 pcf
181+00 to 183+25	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Soil	0.7 ksf	400 psf	125 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	NOMINAL ANCHOR BOND STRESS, τ_n	UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT
WALL 7					
184+10 to 185+25	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 789	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 789 to base of wall	Sandstone with interbedded shale	15 ksf	28,800 psf	155 pcf
185+25 to 188+25	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 801	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 801 to base of wall	Sandstone with interbedded shale	15 ksf	28,800 psf	155 pcf
188+25 to 189+75	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 799	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 799 to base of wall	Sandstone with interbedded shale	15 ksf	28,800 psf	155 pcf
189+75 to 191+43.06	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to elevation 781	Soil	0.7 ksf	400 psf	125 pcf
	Elevation 781 to base of wall	Sandstone with interbedded shale	15 ksf	28,800 psf	155 pcf

STATION LIMITS	ELEVATION INTERVAL	MATERIAL	NOMINAL ANCHOR BOND STRESS, τ_n	UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT
WALL 8					
222+37 to 230+75	Top of wall to top of ground	ASTM D448 No. 57 stone	Not applicable	0 psf	100 pcf
	Top of ground to base of wall	Soil	0.7 ksf	400 psf	125 pcf

8.0 LIMITATIONS OF REPORT

This report has been prepared for the exclusive use of Arcadis, U.S., Inc. and the Tennessee Department of Transportation and their designers for specific application to the project referenced in this report. Our conclusions and recommendations have been prepared using generally accepted standards of geotechnical engineering practice in the State of Tennessee. No other warranty is expressed or implied. S&ME, Inc. is not responsible for the conclusions, opinions, or recommendations of others based on this data.

Our conclusions and recommendations are based on the design information furnished to us, the data obtained during the geotechnical exploration, the laboratory test results, and our past experience. They do not reflect variations in the subsurface conditions that are likely to exist between our borings and in unexplored areas of the site due to the inherent variability of the subsurface conditions in this geologic region and past land use. If such variations are found during construction, re-evaluating our conclusions and recommendations will be necessary.

If changes are made in the locations or elevations of the planned retaining walls, the recommendations contained in this report will not be considered valid unless our firm has reviewed the changes and modified or verified our recommendations in writing. You should give us the opportunity to review the final design plans and the applicable portions of the project specifications when the designers complete the design. This review will allow us to check whether these documents are consistent with the intent of our recommendations.

For more information on the use and limitations of this report, please read the ASFE document included in Appendix V.