ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 090579

FEDERAL AID PROJECT NO. NHPP-0044(36)

HWY. 23 STRS. & APPRS. (MADISON CO.) (S)

STATE HIGHWAY 23 **SECTION** 8

IN MADISON **COUNTY**

The information contained herein was obtained by the Department for design and estimating purposes only. It is being furnished with the express understanding that said information does not constitute a part of the Proposal or Contract and represents only the best knowledge of the Department as to the location, character and depth of the materials encountered. The information is only included and made available so that bidders may have access to subsurface information obtained by the Department and is not intended to be a substitute for personal investigation, interpretation and judgment of the bidder. The bidder should be cognizant of the possibility that conditions affecting the cost and/or quantities of work to be performed may differ from those indicated herein.

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ARKANSAS DEPARTMENT OF TRANSPORTATION

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April 18, 2019

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 090552
Hawkins Hollow Creek Str. & Apprs. (S)
Route 23 Section 8
Madison County

Based on soil information from projects in the surrounding area, an estimated R-Value of 8 is appropriate for pavement design.

Listed below is the additional information requested for use in developing the plans:

Asphalt Concrete Hot Mix

Type	Asphalt Cement %	Mineral Aggregate %
Surface Course	5.5	94.5
Binder Course	4.5	95.5
Base Course	4.1	95.9

Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment
cc: State Constr. Eng. – Master File Copy
District 9 Engineer
System Information and Research Div.
G. C. File

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April 18, 2019

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 090553
Slow Tom Creek Str. & Apprs. (S)
Route 23 Section 8
Madison County

Based on soil information from projects in the surrounding area, an estimated R-Value of 8 is appropriate for pavement design.

Listed below is the additional information requested for use in developing the plans:

Asphalt Concrete Hot Mix

Type	Asphalt Cement %	Mineral Aggregate %
Surface Course	5.5	94.5
Binder Course	4.5	95.5
Base Course	4.1	95.9

Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment
cc: State Constr. Eng. – Master File Copy
District 9 Engineer
System Information and Research Div.
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May 3, 2021
Job No. 20-138

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Attn: Mr. Mike Burns, P.E.
Executive Vice President, Transportation

GEOTECHNICAL INVESTIGATION
ARDOT JOB No. 090579
HWY. 23 STRS & APPRS (MADISON CO) (S)
MADISON COUNTY, ARKANSAS

INTRODUCTION

Submitted herein are the final results of the geotechnical investigation performed for ARDOT Job 090579 Hwy. 23 Strs. & Apprs. (Madison Co.) (S). This geotechnical investigation was authorized by the Crafton Tull & Associates, Inc. Subconsultant Task Order Agreement of November 25, 2019. The results of this study have been provided as data were developed.

We understand the project consists of replacing two (2) bridges (bridge numbers #M0547 and #02947) with reinforced concrete box (RCB) culverts and the Hwy. 23 bridge over Hankins Hollow Creek (bridge number #02946) with a replacement bridge. The project facets are designated as "Site 1" (#M0547), "Site 2" (#02946), and "Site 3" (#02947). The site locations are shown on Plate 1 of Attachment 1. Limited information on the RCB culverts has been provided at this time.

The Hwy. 23 Bridge over Hankins Hollow Creek (Site 2) will be a continuous composite prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 137 feet. We also understand that steel pile foundations are planned at the bridge ends (Bents 1 and 4) and footing foundations will be utilized for support of the interior bents (Bents 2 and 3). Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized for the bridge end embankments. A preliminary bridge layout for Site 2 is provided on Plate 5 of Attachment 1.

The purposes of this study were to explore subsurface conditions at the replacement structure locations and to develop recommendations to guide design and construction of foundations and approach roads. These purposes have been achieved by a multi-phased study that included the following.

- Visiting the site to observe landforms and surface conditions.
- Exploring subsurface conditions by drilling sample and core borings and excavating a test pit at planned bridge and roadway locations to evaluate subsurface conditions and to obtain samples of the subgrade and foundation soil and rock for laboratory testing.
- Performing laboratory tests to evaluate pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations for seismic site class, seismic performance zone/seismic design category, foundation design, slope stability, pavement subgrade support parameters, and construction considerations.

The relationship of these factors to design and construction of the replacement bridge and approach roads and the two (2) RCB culverts has been considered in developing the recommendations and conclusions discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface conditions at the Hwy. 23 sites were investigated by drilling 12 sample and core borings to depths of 2 to 40 feet. A bulk sample of the subgrade soils was obtained from a shallow test pit (Test Pit 1).

The vicinity of each of the three (3) sites is shown on Plate 1 of Attachment 1. The approximate boring locations of the Site 2 borings are shown on the Plan of Borings, Plates 2a through 2c of Attachment 1. The locations of the RCB culvert borings are noted on the logs. Keys to the terms and symbols used on the logs are provided as Plates 3 and 4 of Attachment 1. A preliminary bridge layout for the Site 2 Hwy. 23 bridge over Hankins Hollow Creek (Bridge #02946) is included as Plate 5 of Attachment 1. The subsurface exploration program is summarized in the table below.

Summary of Exploration Program

Project Facet	Boring No.	Approx Hwy. 23 Sta	Offset, ft	Approx Surf El, ft	Completion Depth, ft
Site 1 (#M0547)	A1	North of Bridge End		---	5.5
	A2	South of Bridge End		---	2
Site 2 (#02946)	S1	215+40	5 Lt	1571	30
	S2	215+85	5 Lt	1573	40

Project Facet	Boring No.	Approx Hwy. 23 Sta	Offset, ft	Approx Surf El, ft	Completion Depth, ft
	S3	216+25	5 Rt	1570	36
	S4	216+90	5 Lt	1571	20
	P1	219+60	10 Rt	1586	4
	P2	218+10	10 Rt	1585	6
	P3	214+70	10 Rt	1580	10
	P4	212+50	10 Rt	1576	6.5
Site 3 (#02947)	B1	North of Bridge End		---	8
	B2	South of Bridge End		---	3.5

The results of the borings performed for the Site 1 (#M0547) RCB culvert location are provided as Plates 1 and 2 of Attachment 2. Logs of the borings performed in the Site 2 (#02946) alignment are provided as Plates 1 through 8 of Attachment 3. Logs of the borings performed for the Site 3 (#02947) alignment are provided as Plates 1 and 2 of Attachment 4. Where available, the centerline station and offset of the boring locations and the inferred ground surface elevation are noted on the logs. The approximate boring surface elevation was inferred from the topographic information provided by the Engineer. It must be recognized that the elevations shown are approximate and actual elevations may vary.

To aid in visualizing subsurface conditions at the Site 2 Hwy. 23 bridge over Hankins Hollow Creek location, a generalized subsurface profile is provided as Plate 9 of Attachment 3. The stratigraphy illustrated by the profile has been inferred between discrete boring locations. In view of the natural variations in stratigraphy and conditions, variations from the stratigraphy illustrated by the profile should be anticipated.

The borings performed for this project were drilled with a truck-mounted SIMCO 2400 and a track-mounted CME-55 rotary-drill rig. The SIMCO 2400 was utilized to perform the borings at the RCB culvert locations. The CME-55 was used at the Hankins Hollow Creek location (Site 2). The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. Samples were typically obtained at 2-ft intervals to 10-ft depth and at 5-ft intervals thereafter. Samples were recovered using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb hammer with 30-in. drop in accordance with Standard Penetration Test (SPT) procedures. For the SPTs, a safety hammer was utilized with the SIMCO 2400 drill rig and an automatic hammer was utilized with the CME-55. The number of blows required to drive the standard split-barrel sampler the final 12 in. of an 18-in. total drive, or a portion thereof, is defined as the Standard Penetration Number (N). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column. Where rock hardness precluded recovery with the split-spoon, cuttings were recovered for use in visual classification.

Representative samples of the shale and sandstone bedrock were obtained using a 5-ft-long NQ_{WL}-size double-tube core barrel with a diamond bit. For each core run, the percent recovery was determined as the ratio of recovery to total length of core run. Rock Quality Designation (RQD) was also determined for the core run as the sum of intact, sound rock core greater than 4-in. length divided by the total length of the run and expressed in percent. Both of these values are presented in the right-hand columns of the log forms, opposite the corresponding core run. Photographs of the recovered rock cores from the Hankins Hollow Creek location (Site 2) are provided in Attachment 5.

All samples were removed from sampling tools in the field, examined and visually classified. Samples were then placed in appropriate containers to prevent moisture loss and/or change in condition during transfer to our laboratory for further examination and testing.

The borings were advanced using dry-auger procedures to the extent possible to facilitate evaluation of shallow groundwater conditions. Observations regarding groundwater levels are noted in the lower-right portion of each log and are discussed in subsequent sections of this report. All boreholes were backfilled after obtaining the final water level readings.

LABORATORY TESTING

To evaluate pertinent physical and engineering characteristics of the foundation and subgrade strata, laboratory tests consisting of natural water content determinations and classification tests and measurement of rock compressive strength were performed on selected representative soil and rock samples. Laboratory test results are shown on the logs. The laboratory testing program is discussed in the following report sections.

The laboratory testing program included 29 natural water content determinations performed to develop information on *in-situ* soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field visual classification and to evaluate soil plasticity, six (6) liquid and plastic limit (Atterberg limits) determinations and 11 sieve analyses were performed on selected representative samples. The Atterberg limits are plotted on the logs as pluses inter-connected with a dashed line using the water content scale. The percentage of soil passing through the No. 200 Sieve is noted in the "- No. 200 %" column on the appropriate log forms. Classification test results, along with soil classification by the Unified Soil Classification System and AASHTO designations, are summarized in Attachment 6. Grain-size distribution curves are also provided in Attachment 6.

Compressive strength of representative samples of the shale and sandstone was evaluated by performing seven (7) uniaxial compression tests on representative rock cores. The measured compressive strength is plotted on the log forms, in lbs per sq in., at the appropriate depth. The total unit weight is also shown with the compression test results.

One (1) laboratory moisture-density relationship (Proctor) test (AASHTO T 180) was performed on a representative bulk soil sample to evaluate compaction properties of on-site subgrade soils. The Proctor test and bulk sample classification test results are provided in Attachment 7. Pavement subgrade support properties of the potential subgrade soils were evaluated by performing one (1) California Bearing Ratio (CBR, AASHTO T 193) test on the collected bulk sample. The CBR test results are also provided in Attachment 7.

GENERAL SITE and SUBSURFACE CONDITIONS

Site Conditions

The three (3) sites comprising Job 090579 are on Hwy. 23 in south central Madison County, Arkansas. These structures are on alignments which are generally oriented north-south. Site 1 is located at L.M. 0.539, Site 2 is located at L.M. 7.041, and Site 3 is located at L.M. 15.305 on Hwy. 23. The project areas are primarily thickly wooded and undeveloped. Surface drainage of the existing roadways is good and drainage in areas adjacent to the highway varies from poor to fair. The area topography is undulating to rolling.

Site Geology

Geologically, the project sites are underlain by units of the Pennsylvanian Period Undivided Atoka formation. Characteristically, the Atoka in this area is comprised of moderately dipping, interbedded shale and sandstone units, which are typically fractured and jointed. This formation has a large areal extent and is the predominant surface rock in the Boston Mountains and the Arkansas River Valley. The maximum thickness of the Undivided Atoka is reported to be approximately 25,000 ft in the Boston Mountains.

Seismic Conditions

Based on the site geology, the average soil and rock conditions revealed by the borings, and our experience in the area, a Seismic Site Class B (rock profile) is considered fitting for the Hwy. 23 over Hawkins Hollow (Site 2, bridge #02946) with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[1]. Given the project location and AASHTO code-based values, the 1.0-sec period spectral acceleration coefficient (S_1) for Site Class B is 0.059 and the 1.0-sec period spectral acceleration coefficient (S_{D1}) value for Site Class B is 0.059. Utilizing these parameters, Table 3.10.6-1[2] indicates that a Seismic Performance Zone 1 is fitting for the Hwy 23 bridge site. In reference to the 2011 edition of the AASHTO Guide Specifications, the Peak Ground Acceleration (PGA) having a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years) is predicted to be 0.062 for a Seismic Site Class B for the bridge location.

The liquefaction potential for the relatively thin layer of overburden soils is considered low.

Subsurface Conditions - Hankins Hollow Creek

Based on the results of the borings drilled for the Site 2 Hwy. 23 bridge over Hankins Hollow Creek, the subsurface stratigraphy may be generalized into three (3) primary strata as follows.

Stratum I: The existing embankment fill found at the bridge ends and roadway consists of firm to stiff brown, reddish tan, red, and gray fine sandy / silty clay with a variable amount of crushed stone and shale or loose brown silty fine sand. These soils typically classify as A-4 and A-6 by the AASHTO classification system (AASHTO M 145). These classifications correlate with fair to poor subgrade support for pavement structures. The low- to medium-plasticity embankment fill exhibits variable fair to good compaction with moderate compressibility. The depth, content, and compaction of the embankment fill is likely to vary along the alignment.

Stratum II: The natural overburden soil units in the roadway alignment and at the replacement bridge location are predominantly stiff to very stiff brown, reddish tan, and red fine sandy clay / silty clay, dense brown and tan silty fine sand with numerous sandstone fragments, or medium dense brownish gray, reddish brown, and gray clayey fine to coarse gravel. The overburden soils extend to variable depths of 2 to 15 ft (approximately El 1558 to El 1566). The clayey overburden soils have low plasticity, moderate shear strength, and moderate to low compressibility. The granular soils have medium to high relative density and moderate to low compressibility.

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1 AASHTO LRFD Bridge Design Specifications, 7th Edition; AASHTO; 2014.

2 AASHTO LRFD Bridge Design Specification, AASHTO; 2012

Stratum III: The overburden soils are underlain by moderately hard to hard dark gray shale. The shale is flat bedded and contains very close to close sandstone partings and seams. Locally, moderately hard to hard dark gray fine-grained sandstone is present(see Boring S4). The shale and sandstone exhibit fair to excellent rock quality with RQD values ranging from 55 to 100. In general, rock quality is considered good. Measured compressive strength values range from 6060 to 12,400 lbs per sq inch.

Subsurface Conditions – Sites 1 and 3

The results of the borings drilled at Sites 1 and 3 indicate overburden soils comprised of firm to very stiff silty clay and fine sandy clay with a variable content of rock fragments. The overburden soils exhibit low plasticity. Shear strength varies from low to moderate.

Auger refusal was encountered at 2- to 5.5-ft at Site 1 and at 3.5- to 8-ft depth at Site 3. Given the sandstone fragments encountered in these borings and the observed sandstone in the creek channels, auger refusal is believed to have occurred on sandstone.

Groundwater Conditions

Groundwater was locally encountered at 3.2- to 4.4-ft depth at the various site locations drilled in December 2020 and January 2021. Seasonal seeps and springs could be locally present as infiltrated water migrates from areas of higher terrain through the upper fractured zones of the shale. Perched water could also occur locally at shallow depths within the fill-soil-rock interface. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and water levels in nearby surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design - Hankins Hollow Creek (Site 2)

Foundations for the Hwy. 23 over Hankins Hollow Creek replacement bridge must satisfy two (2) basic and independent design criteria: a) foundations must have an acceptable factor of safety against bearing failure under maximum design loads, and b) foundation movement due to consolidation or swelling of the underlying strata should not exceed tolerable limits for the structures. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

In light of the results of the borings performed for the replacement bridge and the anticipated moderate bridge foundation loads, we recommend that foundation loads at the bridge ends (Bents 1 and 4) be supported on steel piling and at the interior bents (Bents 2 and 3) on footings. Recommendations for foundations are discussed in the following paragraphs.

<u>Bridge Ends (Bent 1 and Bent 4): Steel Pile Foundations</u>. Driven steel HP12×53 or HP14x73 piles are recommended for support of the replacement Hwy. 23 Bridge over Hawkins Hollow foundation loads at the bridge ends. Other pile sizes or types may be evaluated if desired. Point-bearing steel piles driven to refusal should extend through the embankment fill and overburden soils into the moderately hard to hard shale or sandstone. Piles should be driven to practical refusal. We recommend that all the steel piles be fitted with rock points.

Piles should be designed based on applicable AASHTO Load and Resistance Factor Design (LRFD) design procedures[3]. An effective resistance factor (φ) of 0.50 is recommended for geotechnical determination of factored uplift capacities.

Bearing capacities of steel piles driven to refusal should be determined using the LRFD <u>structural</u> design procedure. We recommend that nominal (ultimate) resistance (P_n) of HP piles be determined based on the yield strength of steel H piles (f_y) and the net end area (A_{net}) of the section. An effective resistance factor (φ) of 0.50 is recommended for structural determination of factored bearing capacities. This effective resistance factor for H piles has been based on the assumption of severe driving conditions.

It has been our experience that allowable compression pile capacities of 97 tons for HP12×53 sections and 133 tons for HP14x73 sections are common for steel with a yield strength (f_y) of 50 kips per sq inch. These capacities are based on allowable stress design (ASD) with an allowable compression stress of $0.25f_y$. However, the appropriate factored bearing capacity should be confirmed by the Engineer. Post-construction settlement of piles driven to refusal will be negligible.

Post-construction settlement of piles driven to refusal in rock will be negligible. Given the plan to utilize the existing embankments, the age of the existing embankments with minor amounts of new fill expected, and the predominance of granular embankment fill and natural overburden soils, downdrag loads due to long-term embankment settlement are considered minor.

<u>Estimated</u> pile tip elevations are summarized in the table below. A minimum pile length of 10 ft and a minimum pile embedment of 5 ft below natural grade are recommended.

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[3] <u>Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures</u>, Publication No. FHWA HI-98-032, National Highway Institute, May 2001.

Estimated Tip Elevations of Steel Piles Driven to Refusal

Bent No.	Estimated Pile Tip Elevation, ft	Comments
1 (South Abutment)	1562	Bear in moderately hard to hard shale, estimated 10 ft prebore from cap bottom at El 1572
4 (North Abutment)	1562	Bear in moderately hard to hard shale / fine-grained sandstone

It should be noted that tip elevations shown in the table above are estimates only based on the results of the relevant borings and the inferred surface elevations at the particular locations. Pile capacity and final depth must be field verified by the Engineer or Department.

Piles should be installed in compliance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Preboring could be required to install piles at Bent 1. Preboring is not expected at Bent 4. We recommend that steel piles at the abutments be driven with a hammer system capable of delivering at least 22,000 lb-ft per blow. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows. We also recommend that blow counts be limited to a maximum of about 20 blows per ft for steel piles.

Interior Bents – Footings. The results of the borings indicate that competent moderately hard to hard shale is present at depths of 4 to 15 ft below existing grades (approximately El 1569 to El 1558). For the Hwy. 23 over Hawkins Hollow Bridge interior bents (Bents 2 and 3), foundation loads may be supported on footings bearing in the competent moderately hard to hard dark gray shale. It is recommended that footings be founded with at least 2 ft of embedment into the competent moderately hard to hard shale and at least 2 ft below the potential scour depth, whichever depth is greater. For Bent 2, a minimum footing bottom at ±El 1556. At Bent 3, a minimum footing bottom at ±El 1563 is anticipated. The suitability of the footing bottom elevations must be field verified.

Footings founded with a minimum embedment of 2 ft into the competent moderately hard to hard shale or sandstone may be sized based on a maximum nominal bearing pressure (q_{stat}) of 50 kips per sq foot. A resistance factor (φ_b) of 0.45 is recommended for footings fully bearing in the competent shale. Accordingly, a factored unit bearing resistance (q_R) of 22.5 kips per sq ft is considered appropriate. Post-construction settlement of footings bearing in the competent shale or sandstone as recommended is expected to be negligible.

Uplift loads on footings will be resisted by footing weights and structure dead loads. Depending on the magnitude of uplift loads, deeper embedment or increased footing dimension may be required. If needed, additional uplift resistance can be developed by rock anchors. Recommendations for rock anchors can be provided upon request.

Resistance to lateral forces will be developed by sliding resistance at the footing bottom and the passive resistance of the foundation strata. Resistance to sliding can be evaluated using a nominal friction factor (tan δ) value of 0.60 between the foundation concrete and the competent and sound dark gray shale bearing stratum. A resistance factor (φ_τ) of 0.85 should be applied to sliding resistance. The passive resistance of the upper 2 ft of weathered shale/shale or the scour depth, whichever is greater, should be neglected. Below the greater of 2-ft depth or the scour depth, a nominal unit passive resistance of 2000 lbs per sq ft of foundation area in hard contact with the competent shale can be utilized. A resistance factor (φ_{ep}) of 0.50 should be applied to passive resistance. Where footings in shale are formed, the lateral resistance must be re-evaluated based on the properties of the backfill. For footings that are overexcavated and formed, a limiting maximum nominal lateral passive resistance value of 400 lbs per sq ft should be utilized.

Footing excavations must extend through the overburden soils and any zones of weathered shale to bear fully in the competent moderately hard to hard dark gray and gray shale. A minimum embedment of 2 ft into the competent shale and at least 2 ft below the potential scour depth are recommended. Any overexcavation of footings must be backfilled with concrete. Weathered zones or open fractures in the shale which are exposed at the bearing stratum elevation should be excavated, cleaned out, and filled with concrete. Footing bottoms should be essentially horizontal. Use of dental concrete to level footing bottoms and to repair minor deficient areas is suitable.

Footings should have a minimum width of 6 ft and a minimum embedment of 2 ft into competent shale. All footing excavations should be observed by the Engineer or Department to verify suitable bearing. Any footing undercuts or overbreaks should be backfilled with concrete.

Embankment Slope Stability. The Hawkins Hollow replacement bridge will include new end and side slope configurations. The embankments on the north and south sides are planned with 2-horizontal to 1-vertical (2H:1V) configurations and 3H:1V configurations are planned for side slopes. The north and south embankments will have maximum heights of about 9 to 12 feet.

To evaluate suitability of the plan slope configurations, slope stability analyses were performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a Bishop analysis. For the embankment slopes, four (4) general loading conditions were evaluated, i.e., End of Construction, Long Term, Rapid Drawdown, and Seismic Conditions. For analysis of the seismic condition, a horizontal seismic acceleration coefficient (k_h) of one-half the peak acceleration (A_s) was used, a value of 0.031. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 1574 to El 1569 was assumed. The sections used for the analyses are shown in the graphical results provided in Attachment 8.

The results of the stability analyses indicate that stability of the plan embankment side and end slope configurations are acceptable with respect to all loading conditions evaluated. Consequently, it is our conclusion that the plan embankment slope configurations are suitable with respect to slope stability.

Reinforced Concrete Box Culverts - Site 1 and Site 3 (#M5047 and #02947)

Foundation Design for Box Culverts. In light of the planned replacement of the existing bridge with RCB culverts, shallow foundations, either mats or continuous footings, are considered suitable for support of structural loads of the box and roadway. The results of the borings indicate that rock, primarily sandstone but some shale as well, is present at and below the level of the box inverts. The rock is considered suitable for foundation bearing. Recommendations for foundations are discussed in the following report section.

Footings or Mats for RCB Culverts. The new box culverts may be supported on mat foundations or continuous footings supported in the moderately hard sandstone, stiff to very stiff fine sandy clay or silty clay, or in undercut backfill. Where warranted by channel conditions, undercut should consist of stone backfill (ARDOT Standard Specifications Section 207), Select Granular Backfill (AASHTO M 43 No. 57), or alternates approved by the Engineer or Department. Where clean crushed stone backfill is used over or against overburden soils, the stone should be fully incapsulated by a geotextile filter fabric complying with ARDOT Subsection 625.02, Type 2. The culvert foundation depths must be adequate to resist scour or must be protected from scour.

Foundation recommendations for the RCB culverts are summarized below.

- Bearing Stratum: moderately hard sandstone, stiff to very stiff fine sandy clay or silty clay or undercut backfill
- Maximum nominal bearing pressure (q_{ult}): 6000 lbs per sq ft
- Recommended resistance factor (φ_b): 0.45
- Factored bearing pressure (q_b): 2700 lbs per sq ft

[4] Slope/W 2007; GEO-SLOPE International; 2008.

- Maximum nominal sliding resistance ($\tan \delta$): 0.40
- Sliding resistance factor (φ_τ): 0.80

Uplift resistance of the bottom mat or footings will be developed by structure dead loads and the weight of foundation units. Resistance to lateral forces will be developed by the passive resistance of the foundation soil or rock and sliding resistance at the mat or footing bottom. The passive resistance of the soil and within the upper 1 ft of embedment or above the scour depth should be neglected. Below the 1 ft embedment or scour depth, whichever is greater, a nominal passive resistance value of 350 lbs per sq ft may be used for the undisturbed overburden soils. For intact sandstone, a nominal passive resistance value of 1000 lbs per sq ft may be utilized. A resistance factor (φ_{ep}) of 0.50 is recommended for passive pressure resistance.

A minimum width of 18 in. is recommended for continuous footings. All culvert bottom and foundation excavations should be observed by the Engineer or Department to verify suitable bearing. Post-construction total and differential settlement of foundations supported as recommended is expected to be less than 1 inch.

<u>Lateral Earth Pressures on Culvert Walls</u>. It is anticipated that culvert walls and any wingwalls will be backfilled with either unclassified borrow or selected material. Unclassified borrow is expected to be locally available soils which could be silty clay or sandy clay with rock fragments. Selected material should comply with ARDOT Standard Specifications Section 302 for SM-1 or Select Granular Backfill (AASHTO M 43 No. 57).

Recommendations for lateral earth pressures on box walls are summarized below.

- Total unit weight (γ) for unclassified backfill: 125 lbs per cu ft
- Angle of internal friction (ϕ) for unclassified backfill: 20°
- Equivalent fluid pressure for unclassified backfill:
  - At-rest condition for walls that are fixed against rotation, backfilled with unclassified borrow, and fully drained: 85 lbs per sq ft per ft depth.
  - At-rest condition for walls that are fixed against rotation, backfilled with unclassified borrow, and no provision for internal drainage: 105 lbs per sq ft per ft depth.
- Angle of internal friction (ϕ) for SM-1 backfill: 32°
- Total unit weight (γ) for SM-1: 125 lbs per cu ft

- Equivalent fluid pressure for SM-1 backfill:
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and fully drained: 60 lbs per sq ft per ft depth.
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and no provision for internal drainage: 92 lbs per sq ft per ft depth.
- Angle of internal friction (ϕ) for Select Granular Backfill: 38°
- Total unit weight (γ) for SM-1: 105 lbs per cu ft
- Equivalent fluid pressure for Select Granular Backfill:
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and fully drained: 40 lbs per sq ft per ft depth.
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and no provision for internal drainage: 79 lbs per sq ft per ft depth.

To utilize the lower earth pressure values of the "drained" condition, positive and continuous drainage from behind walls must be provided. This may include a clean, free draining crushed stone, gravel, or granular soil zone or a geosynthetic drainage board approved by the Engineer. Drainage zones should be fully isolated from all soil and shale by a suitable geotextile complying with ARDOT Standard Specifications Subsection 625.02, Type 2. Water should be discharged from backfill by a system of regularly-spaced, functioning weep holes or drain pipes.

Subgrade Support - Sites 1, 2, and 3

The results of the borings indicate that the subgrade soils are typically sandy clay with numerous shale and sandstone fragments. These soils include classifications of A-2-4, A-2-6, and A-6 as per the AASHTO classification system (AASHTO M 145). We believe that the on-site soils and locally-available borrow are likely to be similar soils with similar classification.

We recommend that the subgrade be specifically evaluated by the Engineer or Department during pavement construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives approved by the Engineer or Department. Based on the results of the borings and our site observations, undercuts on the order of 2 ft, more or less, below existing grades could be warranted for subgrade improvement. Alternatively, addition of lime, cement, or other suitable additives could be utilized to develop a stable, non-pumping subgrade.

We also recommend that any soils classifying as A-7-6 and soils with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation.

The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18. Where A-7-5 or A-7-6 soils are encountered at the subgrade elevation, we recommend that these soils be undercut as required to provide at least 18 in. of low-plasticity subgrade soils. Alternatively, stabilization additives may be utilized to develop a stable subgrade with a maximum PI of 18.

Based on the results of the borings and laboratory tests and correlation with the AASHTO classification, subgrade support of the on-site soils is expected to be fair to good. The following parameters are recommended for use in pavement design for a subgrade of the on-site soils and similar borrow soils.

- Resilient Modulus (M_R): 2750 lbs per sq inch
- R value: 5.1

Site Grading and Subgrade Preparation

Site grading and site preparation in the project alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. Where fill depths in excess of 3 ft are planned, stumps may be left after close cutting trees to grade, as per ARDOT criteria. Otherwise, tree stumps must be completely excavated and stumpholes properly backfilled.

The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in the areas of higher terrain. In general, the stripping depth is estimated to be about 6 to 9 inches in cleared areas but may be 18 to 24 in. or more in the localized wooded areas and areas with thick underbrush. The zone of organic surface soils should be completely stripped in the embankment footprint areas and at least 5 ft beyond the projected embankment toes. Particular care must be taken to muck our all saturated and organic-laden soils in the existing roadside swale.

Where existing pavements are to be demolished, consideration may be given to utilizing the processed asphalt concrete and aggregate base for embankment fill. In this case, the demolished materials should be thoroughly blended and processed to a reasonably well-graded mixture with a maximum particle size of 2 in. as per ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. If abandoned pavements are within 3 ft of the plan subgrade elevation, the existing pavement surface should be scarified to a minimum depth of 6 inches. The scarified material should be recompacted to a stable condition.

Following required pavement demolition, clearing and grubbing, stripping, any cut, and prior to fill placement or otherwise continuing with subgrade preparation, the subgrade should be evaluated by thorough proof-rolling. Proof-rolling should be performed with a loaded tandem-wheel dump truck or similar equipment. Unstable soils exhibiting a tendency to rut and/or pump should be undercut and replaced with suitable fill. Care should be taken that undercuts, stump holes, and other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted embankment fill or as directed by the Engineer. Based on the results of the borings, localized undercutting could be required to develop subgrade stability. Potential undercut depths are estimated to range from 2 to 4 ft, more or less.

In lieu of undercutting and replacing unsuitable subgrade soils, consideration may be given to using additives to improve soil workability and to stabilize weak areas. Hydrated lime, quick lime, Portland cement, fly ash, or suitable alternate materials may be used as verified by appropriate testing and approved by the Engineer. Additives can be effective where the depth of unstable soils is relatively shallow. Treatment will be less effective in areas where the zone of unstable soils is deep. The optimum application rate of stabilization additive must be determined by specific laboratory tests performed on the alignment subgrade soils. We recommend a minimum treatment depth of 8 inches.

In areas of deep fills, the potential exists for use of thick initial lifts ("bridging"), as per ARDOT criteria. Bridge lifts will be subject to some consolidation. Settlement of a primarily granular fill suitable for use in bridging would be expected to be relatively rapid in this case, long-term post-construction settlement would not be expected to be a significant concern. Where clayey soils are placed in thick lifts, long term settlement will be more significant. We recommend that the use of "bridging" techniques be limited to granular borrow soils, i.e., sand or gravel. Where fill amounts are limited to less than about 3 ft, bridging will be less effective and the potential for undercut or stabilization will increase. <u>Use of bridging techniques and fill lift thickness must be specifically approved by the Engineer or Department</u>.

Subgrade preparation and mass undercuts should extend at least 5 ft beyond the embankment toes to the extent possible. Subgrade preparation in roadway areas should extend at least 3 ft outside pavement shoulder edges to the extent possible. Existing drainage features should be completely mucked out and all loose and/or organic soils removed prior to fill placement.

Fill and backfill may consist of unclassified borrow free of organics and other deleterious materials as per ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 210.06. Granular soils must be protected from erosion with a minimum 18-in.-thick armor of clayey soil. Embankment slopes configured steeper than 2.5H:IV should be protected with riprap.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT criteria (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210). Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each lift and should be tested to verify compliance with the specified density and water content prior to placement of subsequent lifts.

CONSTRUCTION CONSIDERATIONS

Groundwater and Seepage Control

Positive surface drainage should be established at the start of the work, be maintained during construction and following completion of the work to prevent surface water ponding and subsequent saturation of subgrade soils. Cofferdam construction could be required for interior bent foundation construction. Density and water content of all earthwork should be maintained until the embankments and bridge work is completed.

Subgrade soils that become saturated by ponding water or runoff should be excavated to undisturbed soil or rock. The embankment and roadway subgrade should be evaluated by the Engineer during subgrade preparation.

Shallow perched groundwater may be encountered in the near-surface soils, particularly at lower elevations and during times of high seepage flow. The volume of groundwater produced can be highly variable depending on stream levels and the condition of the soils in the immediate vicinity of excavations. In addition, seasonal surface seeps or springs could develop as infiltrated surface water from areas of higher terrain migrate downgradient.

Seepage into excavations and cuts can typically be controlled by ditching or sump-and-pump methods. If seepage into excavations becomes a problem, backfill should consist Of Select Granular Backfill (AASHTO M 43 No. 57), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or approved alternates up to an elevation above the inflow of seepage. In areas of seepage infiltration, the granular fill should be encapsulated with a filter fabric complying with ARDOT Standard Specifications for Highway Construction, 2014

Edition, Subsection 625.02, Type 2 and vented to positive discharge. Use of coarse stone fill should be avoided in areas where piles will be driven. Where surface seeps or springs are encountered during site grading, we recommend the seepage be directed via French drains or blanket drains to positive discharge at daylight or to storm drainage lines.

Piling

Piles should be installed in compliance with Standard Specifications for Highway Construction, 2014 Edition, Section 805. Preboring for pile installation is expected at Bent 1, with an estimated prebore depth of 10 feet. Rock drilling methods could be required to achieve the recommended prebore depth.

Based on local experience, we recommend a hammer system capable of delivering at least 22,000 lb-ft per blow for the steel piles at the abutments. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed by the Engineer prior to hammer acceptance and start of pile driving.

As a minimum, safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method A. Driving records should be available for review by the Engineer during pile installation. Piles should be carefully examined prior to driving and piles with structural defects should be rejected. Any splices in steel piles should develop the full cross-sectional capacity of un-spliced piles. Pile installation should be monitored by qualified personnel to maintain specific and complete driving records and to observe pile installation procedures. Blow counts on steel piles should be limited to about 20 blows per inch. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

Footings

All footing excavations should be observed by the Engineer or the Department to verify suitable bearing and final cleanup. Footing excavations must be clean and dry at the time of concrete placement. Final cleanup of footing excavations with compressed air should be considered to facilitate removal of all loose cuttings and debris from the bearing surface. All footing bottoms should be essentially horizontal. The use of stepped footings is suitable. Where footing excavations will remain open for extended periods, consideration may be given to protecting the bearing stratum with a thin layer of seal concrete. Any overexcavation of footings, including overbreak and localized removal of weak zones, should be backfilled with concrete.

Rock Excavation

Rock excavation methods may be required for excavation of footings and for pile prebores. Some overbreak should be anticipated for footing excavations advanced into the moderately hard to hard shale with sandstone seams and layers. Preboring for piles is also likely to require rock drilling equipment and methods such as coring, a pneumatic hammer, or similar tools.

CLOSURE

The Engineer or Department or a designated representative thereof should monitor site preparation, grading work and foundation and pavement construction. Subsurface conditions significantly at variance with those encountered in the borings should be brought to the attention of the Geotechnical Engineer. The conclusions and recommendations of this report should then be reviewed in light of the new information.

The following illustrations are attached and complete this report.

Attachment 1	Site Vicinity Map
	Plans of Borings
	Keys to Terms and Symbols
Attachment 2	Site 1 (#M0547) Boring Logs
Attachment 3	Site 2 (#02946) Boring Logs
Attachment 4	Site 3 (#02947) Boring Logs
Attachment 5	Rock Core Photographs
Attachment 6	Laboratory Test Results
Attachment 7	Subgrade Support Test Results
Attachment 8	Site 2 Stability Analyses Results

* * * * *

We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this report, or if we may be of additional assistance, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

Ben Davis, P.E.
Project Engineer

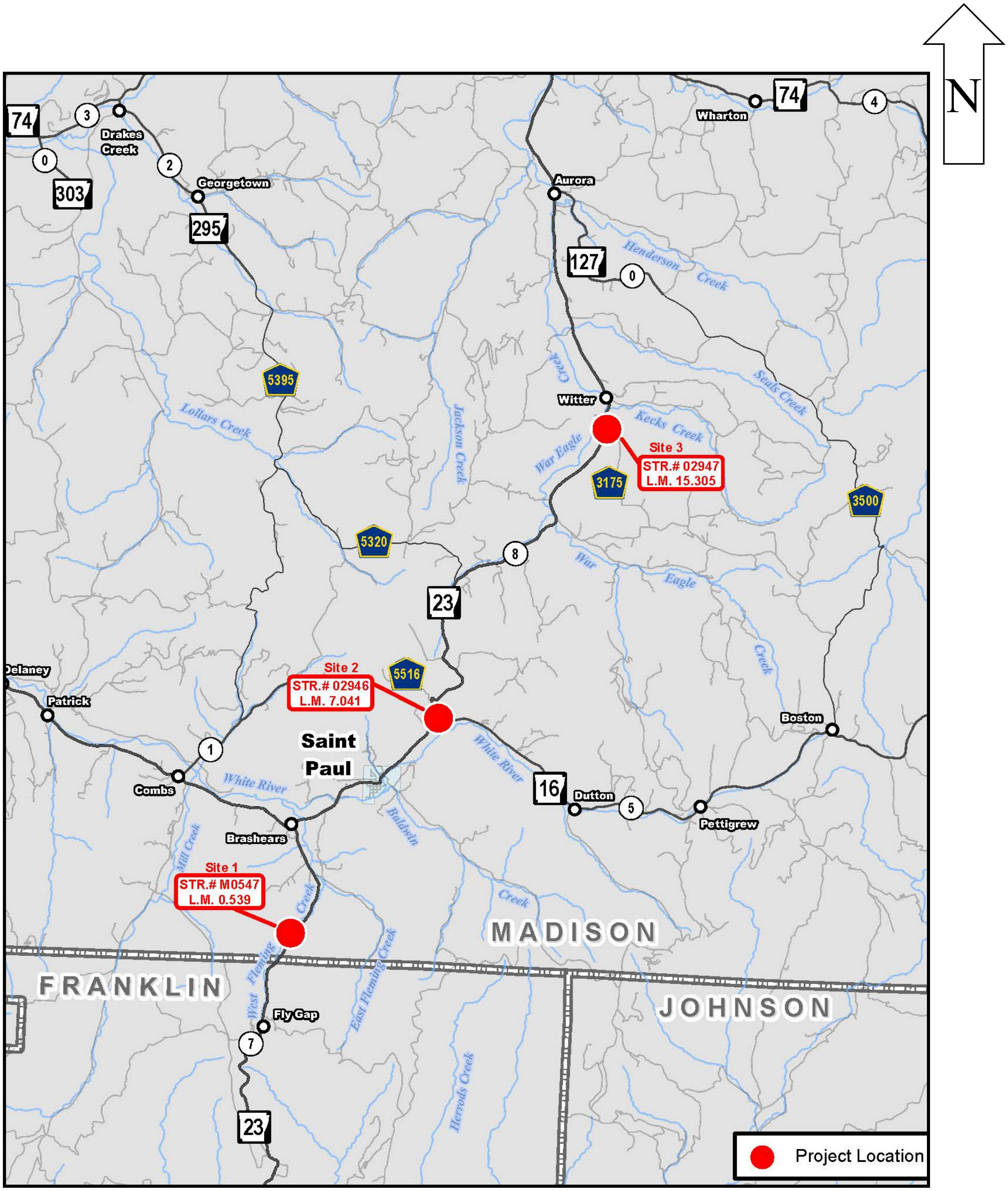
Mark E. Wyatt, P.E.
President

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 7791
MARK E. WYATT

BJD/MEW:jw

Copies Submitted:	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

ATTACHMENT 1

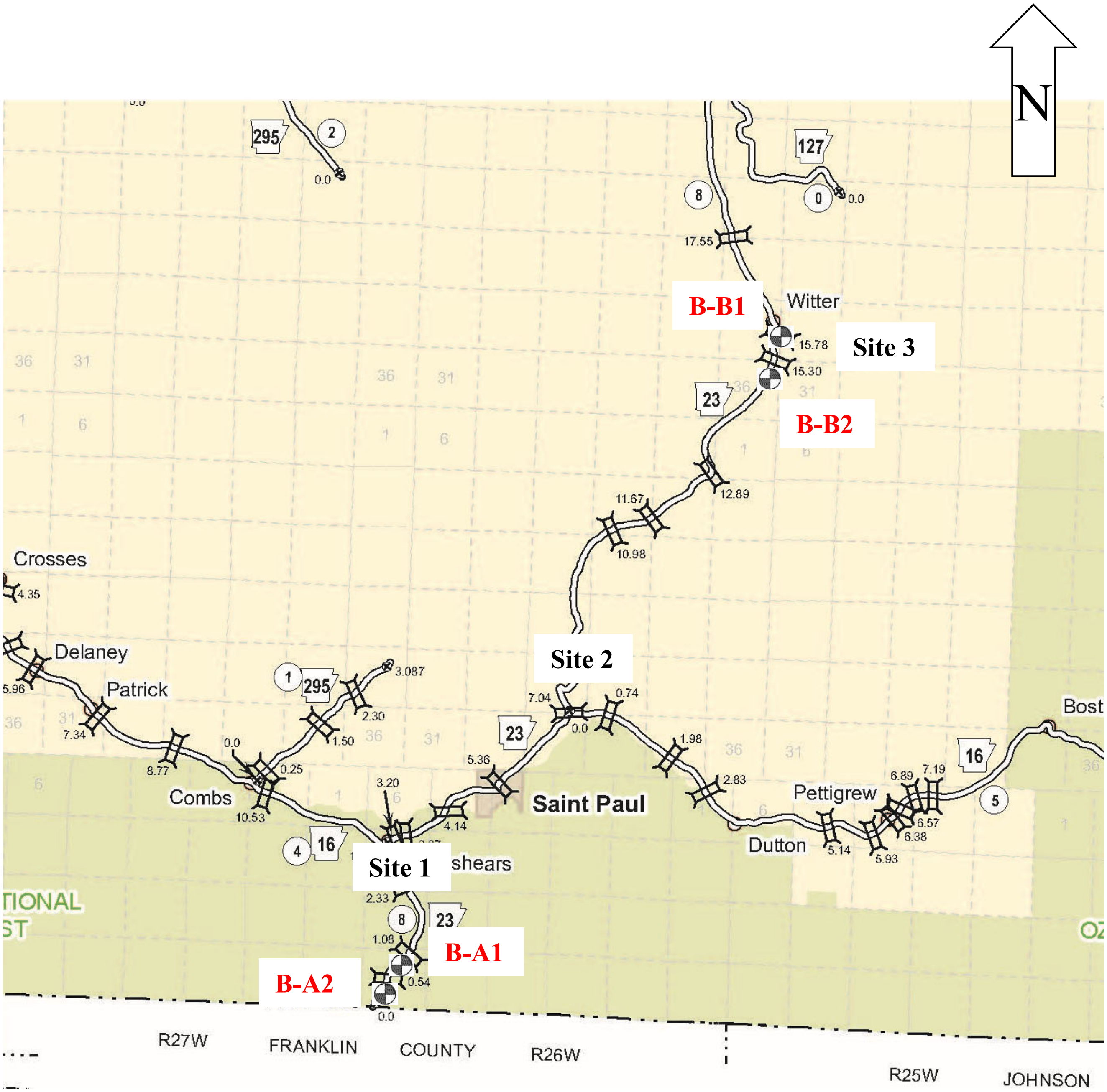


Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

SITE VICINITY MAP
090579 Hwy.23 Strs. & Apprs
Madison County, Arkansas

Job No. 20-138

Plate 1



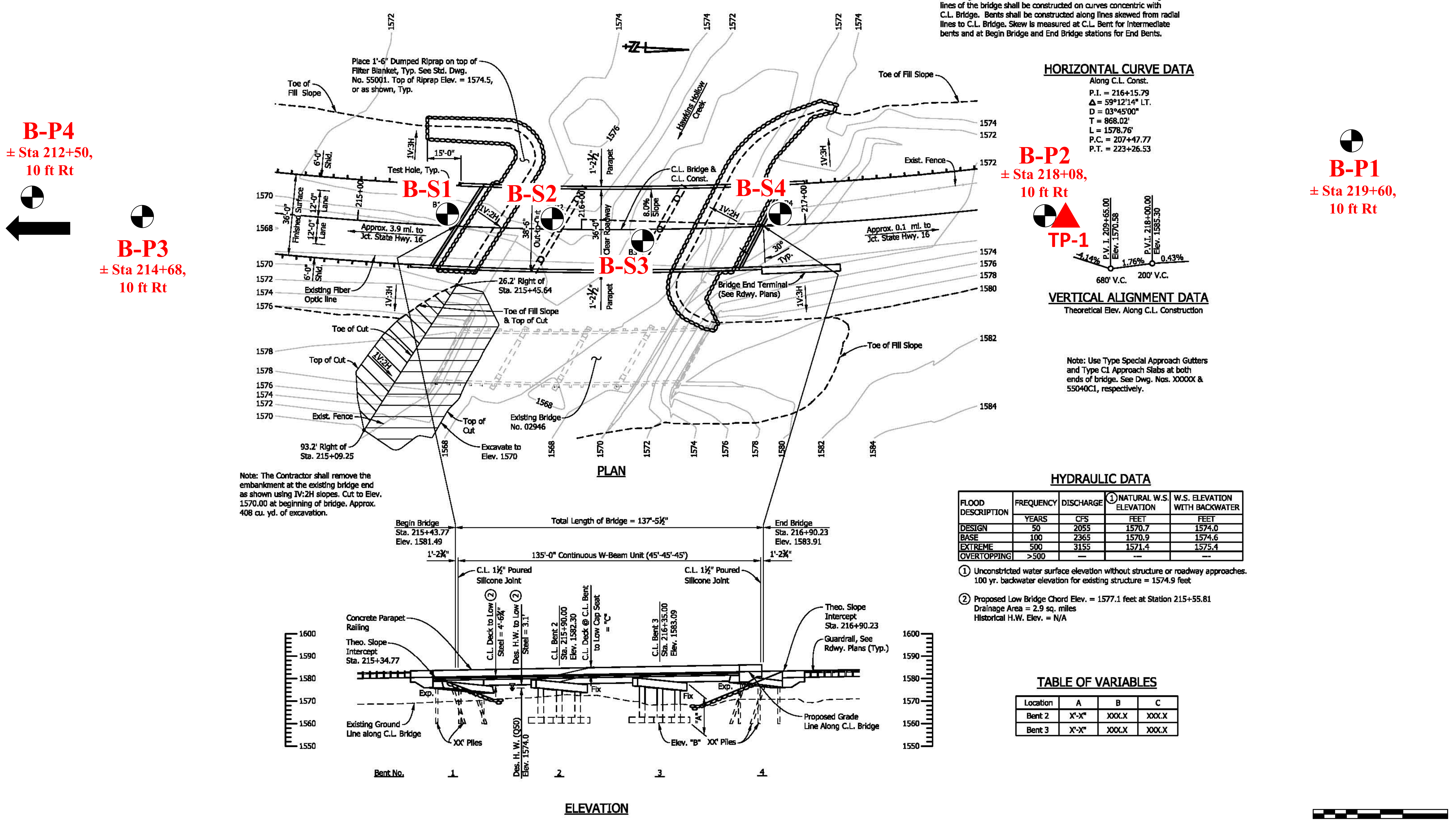
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

PLAN OF BORINGS
090579 RCB Approach Roads
Madison County, Arkansas

Scale: As Shown	
Job No. 20-138	
Plate 2A	

For R/W Data, See Roadway Plans.

Note: C.L. Bridge is on a 03°45'00" curve left. C.L. Beams and the longitudinal lines of the bridge shall be constructed on curves concentric with C.L. Bridge. Bents shall be constructed along lines skewed from radial lines to C.L. Bridge. Skew is measured at C.L. Bent for intermediate bents and at Begin Bridge and End Bridge stations for End Bents.



Note: Use Type Special Approach Gutters and Type C1 Approach Slabs at both ends of bridge. See Dwg. Nos. XXXXX & 55040C1, respectively.

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	(1) NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	2055	1570.7	1574.0
BASE	100	2365	1570.9	1574.6
EXTREME	500	3155	1571.4	1575.4
OVERTOPPING	>500	---	---	---

(1) Unconstricted water surface elevation without structure or roadway approaches. 100 yr. backwater elevation for existing structure = 1574.9 feet

(2) Proposed Low Bridge Chord Elev. = 1577.1 feet at Station 215+55.81
Drainage Area = 2.9 sq. miles
Historical H.W. Elev. = N/A

TABLE OF VARIABLES

Location	A	B	C
Bent 2	X'-X"	XXX.X	XXX.X
Bent 3	X'-X"	XXX.X	XXX.X

20 ft 0 20 ft 40 ft

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

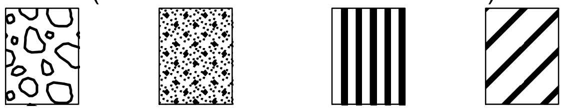
PLAN OF BORINGS
090579 – Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

Scale: As shown | Job No. 20-138 | PLATE 2B

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

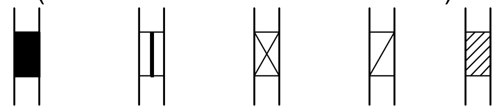
SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES
(SHOWN IN SYMBOLS COLUMN)



Gravel Sand Silt Clay
Predominant type shown heavy

SAMPLER TYPES
(SHOWN ON SAMPLES COLUMN)



Shelby Tube, Rock Core, Split Spoon, No Recovery, Cutting

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (major portion retained on No. 200 sieve): Includes (I) Clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as determined by laboratory tests.

DESCRIPTIVE TERM	N-VALUE	RELATIVE DENSITY
VERY LOOSE	0-4	0-15%
LOOSE	4-10	15-35%
MEDIUM DENSE	10-30	35-65%
DENSE	30-50	65-85%
VERY DENSE	50 and above	85-100%

FINE GRAINED SOILS (major portion passing No. 200 sieve): Includes (1) Inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests.

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH TON/SQ. FT.
VERY SOFT	Less than 0.25
SOFT	0.25-0.50
FIRM	0.50-1.00
STIFF	1.00-2.00
VERY STIFF	2.00-4.00
HARD	4.00 and higher

NOTE: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above, because of planes of weakness or cracks in the soil. The consistency ratings of such soils are based on penetrometer readings.

TERMS CHARACTERIZING SOIL STRUCTURE

SLICKENSIDED - having inclined planes of weakness that are slick and glossy in appearance.
FISSURED - containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
LAMINATED - composed of thin layers of varying color and texture.
INTERBEDDED - composed of alternate layers of different soil types.
CALCAREOUS - containing appreciable quantities of calcium carbonate.
WELL GRADED - having a wide range in grain sizes and substantial amounts of all intermediate particle sizes.
POORLY GRADED - predominantly of one grain size, or having a range of sizes with some intermediate sizes missing.

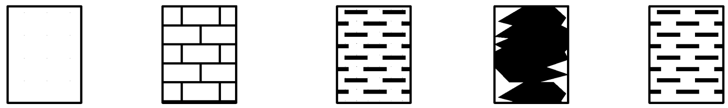
Terms used on this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No.3-357, Waterways Experiment Station, March 1953

KEY 3-14-12

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

BORING LOG TERMS – ROCK

ROCK TYPES
(SHOWN IN SYMBOLS COLUMN)



Sandstone · Limestone · Siltstone · Coal · Shale

Joint Characteristics –

Spacing	
Very Close	0.75 to 2.5 in.
Close	2.5 to 8 in.
Moderately Close	8 to 24 in.
Wide	2 to 6 ft
Very Wide	More than 6 ft

Bedding Characteristics –

Very Thin	0.75 to 2.5 in.
Thin	2.5 to 8 in.
Medium	8 to 24 in.
Thick	2 to 6 ft
Massive	More than 6 ft

Lithologic Characteristics –

Clayey
Shaly
Calcareous (limy)
Siliceous
Sandy (Arenaceous)
Silty
Plastic Seams

Parting –	Less than 1/16 inch
Seam –	1/16 to 1/2 inch
Layer –	1/2 to 12 inches
Stratum –	Greater than 12 inches

Hardness–

Soft (S) – Reserved for plastic material alone.

Friable (F) – Easily crumbled by hand, pulverized or reduced to powder and is too soft to be cut with a pocket knife.

Low Hardness (LH) – Can be gouged deeply or carved with a pocket knife.

Moderately Hard (MH) – Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and scratch is readily visible after the powder has been blown away.

Hard (H) – Can be scratched with difficulty; scratch produces little powder and is often faintly visible; traces of the knife steel may be visible.

Very hard (VH) – Cannot be scratched with a pocket knife. Knife steel marks left on surface.

Texture –

Fine – Barely seen with naked eye
Medium – Barely seen up to 1/8 in.
Coarse – 1/8in. to 1/4 in.

Structure –

Bedding
- Flat – 0° – 5°
- Gently Dipping – 5° – 35°
- Moderately Dipping – 55° – 85°
- Steeply Dipping – 55° – 85°

Fractures, scattered
- Open
- Cemented or Tight

Fractures, closely spaced
- Open
- Cemented or Tight

Brecciated (Sheared and Fragmented)
- Open
- Cemented or Tight

Joints
- Faulted
- Slickensides

Degree of Weathering –

Fresh – No visible signs of decomposition or discoloration. Rings under hammer impact.

Slighty Weathered – Slight discoloration inwards from open fractures, otherwise similar to fresh.

Moderately Weathered – Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat less than fresh rock, but cores cannot be broken by hand or scraped by knife. Texture preserved.

Highly Weathered – Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric

Completely Weathered – Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.

Residual Soil – Advanced state of decomposition resulting in plastic soils. Rock fabric and structure completely destroyed. Large volume change.

Solution and Void Conditions –

Solid, contains no voids
Vuggy (pitted)
Vesicular (igneous)
Porous
Cavities
Cavernous

Swelling Properties –

Nonswelling
Swelling

Slaking Properties –

Nonslaking
Slakes slowly on exposure
Slakes readily on exposure

Rock Quality Designation (RQD) –

RQD (Percent)	Diagnostic Description
Greater than 90	Excellent
75 – 90	Good
50 – 75	Fair
25 – 50	Poor
Less than 25	Very Poor

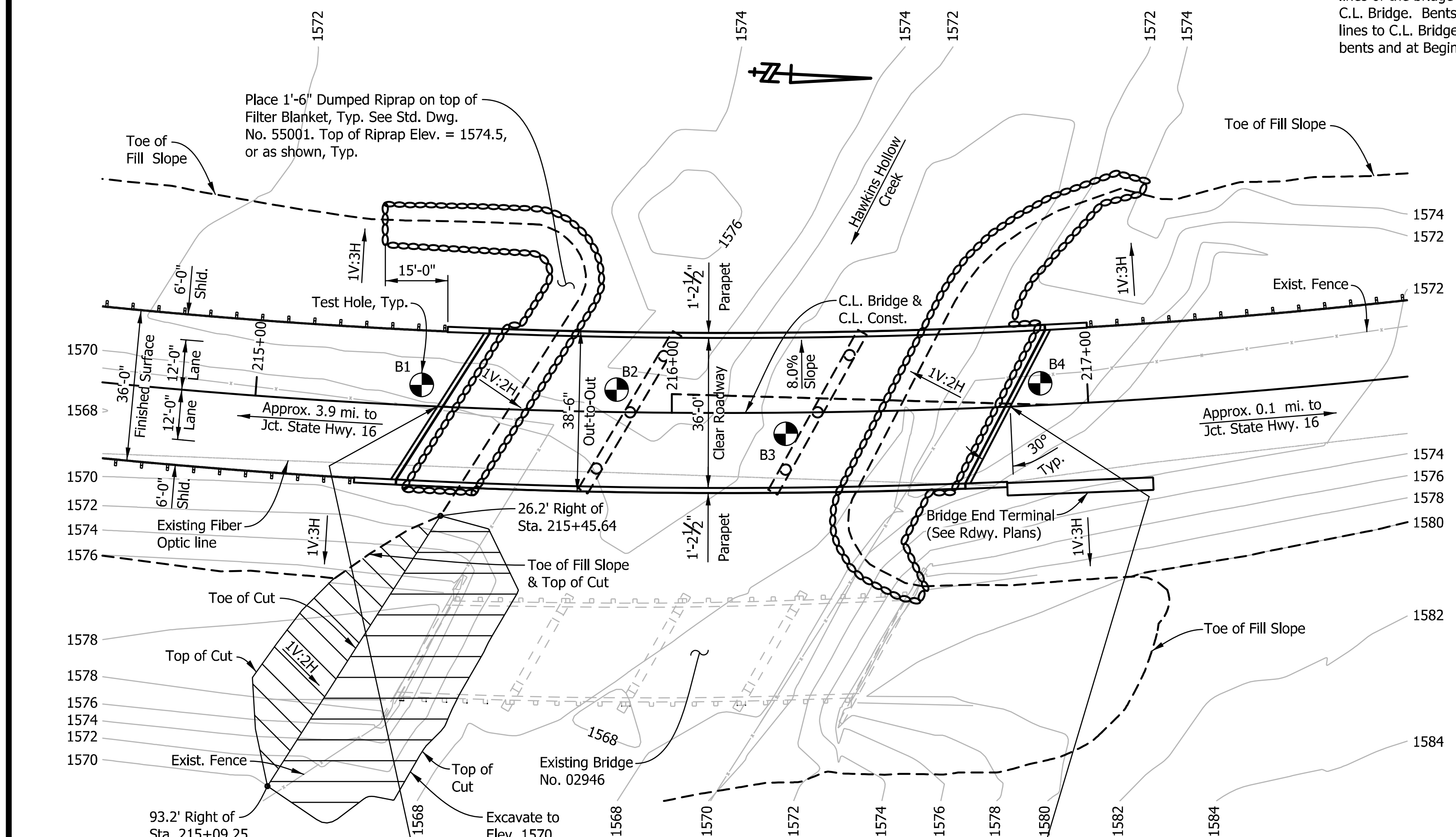
KEYROCK FHWA 3-2-12

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		090579	1	1

(1) XXXXX LAYOUT XXXXX

For R/W Data, See Roadway Plans.

Note: C.L. Bridge is on a 03°45'00" curve left. C.L. Beams and the longitudinal lines of the bridge shall be constructed on curves concentric with C.L. Bridge. Bents shall be constructed along lines skewed from radial lines to C.L. Bridge. Skew is measured at C.L. Bent for intermediate bents and at Begin Bridge and End Bridge stations for End Bents.



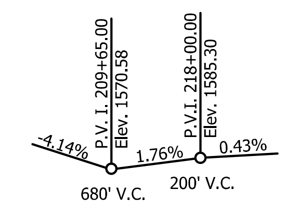
PLAN

Note: The Contractor shall remove the embankment at the existing bridge end as shown using 1V:2H slopes. Cut to Elev. 1570.00 at beginning of bridge. Approx. 408 cu. yd. of excavation.

HORIZONTAL CURVE DATA

Along C.L. Const.

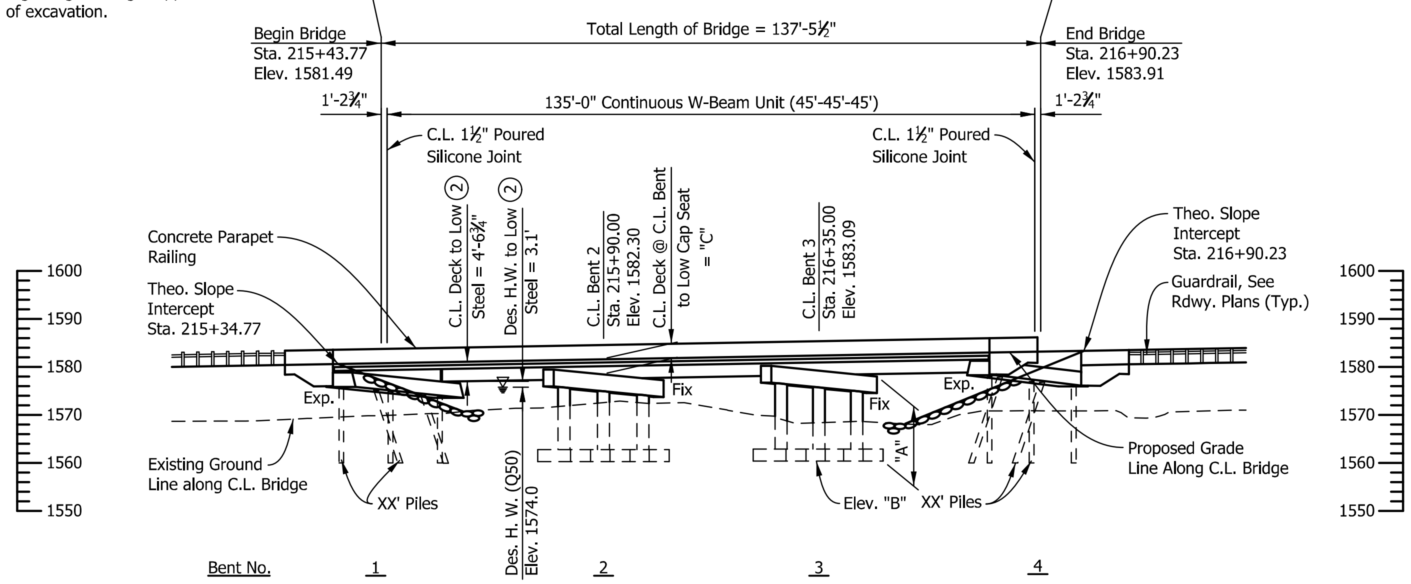
P.I. = 216+15.79
Δ = 59°12'14" LT.
D = 03°45'00"
T = 868.02'
L = 1578.76'
P.C. = 207+47.77
P.T. = 223+26.53



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along C.L. Construction

Note: Use Type Special Approach Gutters and Type C1 Approach Slabs at both ends of bridge. See Dwg. Nos. XXXXX & 55040C1, respectively.



ELEVATION

NOTE: Stations shown are along C.L. Construction. Elevations shown are actual top of deck elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on actual top of deck elevation at C.L. Bridge. See "CROSS SLOPE TRANSITION SKETCH" on Dwg. No. XXXXX for more information.

215+00 216+00 217+00

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	(1) NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
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DESIGN	50	2055	1570.7	1574.0
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Drainage Area = 2.9 sq. miles
Historical H.W. Elev. = N/A

TABLE OF VARIABLES

Location	A	B	C
Bent 2	X'-X"	XXX.X	XXX.X
Bent 3	X'-X"	XXX.X	XXX.X

GENERAL NOTES:

BENCH MARK: Vertical Control Data are shown on Survey Control Details.

CONSTRUCTION SPECIFICATIONS: Arkansas State Highway and Transportation Department Standard Specifications for Highway Construction (2014 edition) with applicable Supplemental Specifications and Special Provisions. Section and Subsection refer to the Standard Construction Specifications unless otherwise noted in the Plans.

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, Eighth Edition (2017).

LIVE LOADING: HL-93

SEISMIC ZONE: X S_{D1}: X.XXX SITE CLASS: X

SEISMIC OPERATIONAL CLASSIFICATION: ESSENTIAL

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

BORING LOGS: Boring logs may be obtained from the Construction Contract Procurement Section of the Program Management Division.

STEEL PILING: All piling shall be HP 12x53 (Grade 50) and shall be driven with an approved air, team, or diesel hammer to a minimum safe bearing capacity of XX tons per pile and into the material designated as XXXXX on the boring legend. Minimum penetration shall be 10' below natural ground for all piles in Bents 1 and 4. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths shown are for estimating quantities and for use in determining payment for cut-off and build-up in accordance with Section 805. The Contractor shall use approved steel H-Pile driving points on all piles.

PREBORING: Preboring is required for all piles in Bents 1 and 4. The depth of preboring shall be to a depth sufficient to provide the specified minimum penetration and to a minimum 3' depth into material designated as XXXXX on the boring legend, whichever is lower. The actual size and depth of preboring shall be determined in the field by the Engineer. The Contractor shall be responsible for keeping prebored holes free of debris prior to driving piles and backfilling which may require the use of temporary casings or other approved methods. After driving is completed, the prebored hole shall be backfilled with Class S Concrete to the top of the rock and the remaining length backfilled in accordance with Subsection 805.08(a). Any related cost for backfilling and temporary casing will not be paid for directly, but shall be considered subsidiary to the item "Preboring".

SPREAD FOOTINGS: Footings shall be set a minimum of 2' into material designated as XXXXX on the boring legend. The top of the footings at Bents 2 thru 3 shall be set a minimum 2' below the channel bottom as determined by the lowest channel elevation within the footprint of the footing. Foundations for footings shall be prepared in accordance with Subsection 801.04. Rock excavations shall be made to neat lines of the concrete footings. Concrete in footings shall be poured directly against excavated surfaces of rock. Excavations shall be backfilled and compacted to the level of the existing ground in accordance with Subsection 801.08.

PAINTING: All Grade 50W structural steel, except galvanized members, surfaces in contact with concrete, and the expansion device, within five feet of bridge deck expansion joints shall be painted as specified in Subsection 807.75. The color of paint shall be Brown equal or close to Federal Std. 595B, Color Chip No. 30070 and as approved by the Engineer. The finish system may be applied in the shop. Any damage to the paint system occurring during transport or installation shall be corrected according to the manufacturer's recommendations at no cost to the Department.

For Additional General Notes see Dwg. No. XXXXX.

PRELIMINARY SUBJECT TO REVISION

SHEET 1 OF 2

LAYOUT OF BRIDGE
HIGHWAY 23 OVER HAWKINS HOLLOW CREEK
HWY. 23 STRS. & APPRS. (MADISON CO.) (S)
MADISON COUNTY
ROUTE 23 SEC. 8

ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: BWC	DATE: 04-21-20	FILENAME: b090579_l1.dgn	
CHECKED BY: CAW	DATE: 04-22-20	SCALE: 1" = 20'	
DESIGNED BY: KRM	DATE: 04-21-20		

BRIDGE NO. XXXXX DRAWING NO. XXXXX

Plate 5

USER: KM5125
DESIGN FILE: G:\9105001_090579x23\TRANSP\dgn\bridge\b090579_l1.dgn
SCALE: 1:40
PLOTTED: 11/16/2020 11:21

ATTACHMENT 2

20-138

Grubbs, Hoskyn,
Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. A1

090579 - #02947 RCB Approach Road
Madison County, Arkansas

TYPE: Auger

LOCATION: Site 1 - North of Bridge End

SURF. EL:

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
0	Stiff reddish brown silty clay w/sandstone fragments			
1		13		
3		32		18
4	- very stiff below 4 ft			
5		38		
5.5	- auger refusal at 5.5 ft in sandstone			

COHESION, TON/SQ FT: 0.2 0.4 0.6 0.8 1.0 1.2 1.4

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70

COMPLETION DEPTH: 5.5 ft
DATE: 12-23-20

DEPTH TO WATER
IN BORING: Dry

DATE: 12/23/2020

LGBNEW 20-138.GPJ 1-29-21

20-138

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. A2

090579 - #02947 RCB Approach Road
Madison County, Arkansas

TYPE: Auger

LOCATION: Site 1 - South of Bridge End

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL:	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2 0.4 0.6 0.8 1.0 1.2 1.4) PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT (10 20 30 40 50 60 70)	- No. 200 %
0–2			Very stiff reddish brown fine sandy clay w/numerous sandstone fragments	50/11"			29
			- auger refusal at 2 ft in sandstone	25/0"			

COMPLETION DEPTH: 2.0 ft
DATE: 12-23-20

DEPTH TO WATER
IN BORING: Dry

DATE: 12/23/2020

LGBNEW 20-138.GPJ 1-29-21

PLATE 2

ATTACHMENT 3

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. S1

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger to 5 ft /Wash

LOCATION: Approx Sta 215+40, 5 ft Lt

SURF. EL: 1571±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT / PLASTIC & LIQUID LIMIT / COHESION, TON/SQ FT	- No. 200 %	% Recovery	% RQD
0-2	Stiff brown, red and gray silty clay w/crushed stone (fill)	14					
2-6	Very stiff brown fine sandy clay w/sandstone fragments	42					
	- with sandstone cobbles below 4 ft	50/5"					
6-8	Dense brown and tan silty fine sand w/numerous sandstone fragments (completely weathered sandstone)	43		Water content ≈ 10	13		
8-30	Moderately hard to hard dark gray and gray shale, flat bedded w/very close sandstone partings, seams and layers	25/0"					
				q_u= 1110 psi, TUW= 148 pcf		100	100
						100	100
				q_u= 6060 psi, TUW= 183 pcf			
						100	100
	- with less sandstone below 27 ft					100	100

COMPLETION DEPTH: 30.0 ft
DATE: 12-28-20

DEPTH TO WATER
IN BORING: Dry to 5 ft

DATE: 12/28/2020

RECRQDN200-2 20-138 S LOGS.GPJ 5-3-21

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. S2

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 215+85, 5 ft Lt

SURF. EL: 1573±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	- No. 200 %	% Recovery	% RQD
0–2	Firm reddish tan and brown silty clay w/crushed stone (fill)	8					
2–15	Stiff brown fine sandy clay w/sandstone fragments	14					
5		22		Water content ≈ 4; PL ≈ 17; LL ≈ 26			
	- very stiff below 6 ft	25					
10		25					
15		50/6"					
15–40	Hard dark gray shale, flat bedded w/very close interbedded fine-grained sandstone partings, seams and layers			q_u= 12,400 psi, TUW= 168 pcf		100	83
20–25						100	100
25–30	- with less sandstone below 24.5 ft					100	100
30–35						90	85
35–40						100	60

COMPLETION DEPTH: 40.0 ft
DATE: 12-23-20

DEPTH TO WATER IN BORING: Dry to 15 ft

DATE: 12/23/2020

RECRQDN200-2 20-138_S LOGS_.GPJ 5-3-21

20-138

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. S3

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger to 6 ft /Wash

LOCATION: Approx Sta 216+25, 5 ft Rt

SURF. EL: 1570±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT	- No. 200 %	% Recovery	% RQD
0–4	Very stiff brown, reddish tan and red fine sandy clay w/numerous sandstone fragments	27 29					
4–6	Moderately hard dark gray and yellowish tan highly weathered shale	50/5"					
6–36	Hard dark gray shale, flat bedded w/very close interbedded sandstone partings, seams and layers			q_u= 10,860 psi, TUW= 182 pcf		100	55
				q_u= 8830 psi, TUW= 161 pcf		93	93
	- with close sandstone seams and layers below 15.3 ft					93	93
						92	92
						100	100
						100	100

COMPLETION DEPTH: 36.0 ft
DATE: 12-29-20

DEPTH TO WATER IN BORING: 3.2 ft

DATE: 12/28/2020

RECRQDN200-2 20-138_S LOGS_GPJ 5-3-21

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. S4

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 216+90, 5 ft Lt

SURF. EL: 1571±

COHESION, TON/SQ FT: 0.2 0.4 0.6 0.8 1.0 1.2 1.4

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %	% Recovery	% RQD
0–4	Loose brown fine silty fine to coarse sand w/a little fine gravel and occasional cobbles (possible fill)	5		19		
		9				
4–8	Medium dense brownish gray, reddish brown and gray clayey fine to coarse gravel w/occasional shale fragments	11		29		
		24				
8–10	Moderately hard dark gray shale	50/3"				
10–20	Moderately hard to hard dark gray fine-grained sandstone w/interbedded shale seams and layers					
	q_u= 10,120 psi, TUW= 170 pcf				98	83
	q_u= 10,660 psi, TUW= 180 pcf				95	80
20–30	Moderately hard to hard dark gray shale, flat bedded				97	97
					97	97

COMPLETION DEPTH: 30.0 ft
DATE: 1-5-21

DEPTH TO WATER IN BORING: 4.4 ft

DATE: 1/4/2021

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. P1

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 219+60, 10 ft Rt

SURF. EL: 1586±

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT	- No. 200 %
0–4			Stiff reddish brown, gray and tan fine sandy clay, silty w/sandstone fragments				
1		X		20			
2			- very stiff below 2 ft				
3		X		36			23
4–10			NOTE: Auger refusal at 4 ft in shale w/interbedded sandstone.				

Cohesion scale: 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4
Water content scale: 10, 20, 30, 40, 50, 60, 70

COMPLETION DEPTH: 4.0 ft
DATE: 1-6-21

DEPTH TO WATER
IN BORING: 2 ft

DATE: 1/6/2021

20-138

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. P2

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 218+10, 10 ft Rt

SURF. EL: 1585±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT / LIMITS	- No. 200 %
0–2	Soft brown silty clay w/shale fragments (fill)	5			41
2–4	Very stiff maroon, reddish brown and gray fine sandy clay w/sandstone fragments	36			
4–6	Moderately hard to hard gray, tan and brown weathered shale w/silty clay laminations	50/6"			

NOTE: Auger refusal at 6 ft in shale w/interbedded sandstone.

COMPLETION DEPTH: 6.0 ft
DATE: 1-6-21

DEPTH TO WATER
IN BORING: 4 ft

DATE: 1/6/2021

LGBNEW 20-138 PAVEMENT LOGS.GPJ 5-3-21

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. P3

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger   LOCATION: Approx Sta 214+70, 10 ft Rt

SURF. EL: 1580±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT	PLASTIC LIMIT	LIQUID LIMIT	- No. 200 %
0–4	Stiff brown silty cay w/sandstone seams and fragments	16		~14			
		19		~13	~18	~28	26
4–6	Moderately hard pale red, gray and tan weathered fine-grained sandstone	50/2"					
6–10	Moderately hard to hard dark gray shale	25/0"					
		25/0"					

COHESION, TON/SQ FT: 0.2 0.4 0.6 0.8 1.0 1.2 1.4
PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70

Depth scale: 5, 10, 15

COMPLETION DEPTH: 10.0 ft
DATE: 1-5-21

DEPTH TO WATER
IN BORING: 4 ft

DATE: 1/8/2021

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. P4

090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 212+50, 10 ft Rt

SURF. EL: 1576±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT	- No. 200 %
0	Stiff brown and reddish brown silty clay w/shale and sandstone fragments				
1		11			31
3		19			
5		13			
6	Moderately hard dark gray shale w/interbedded fine-grained sandstone	50/2"			

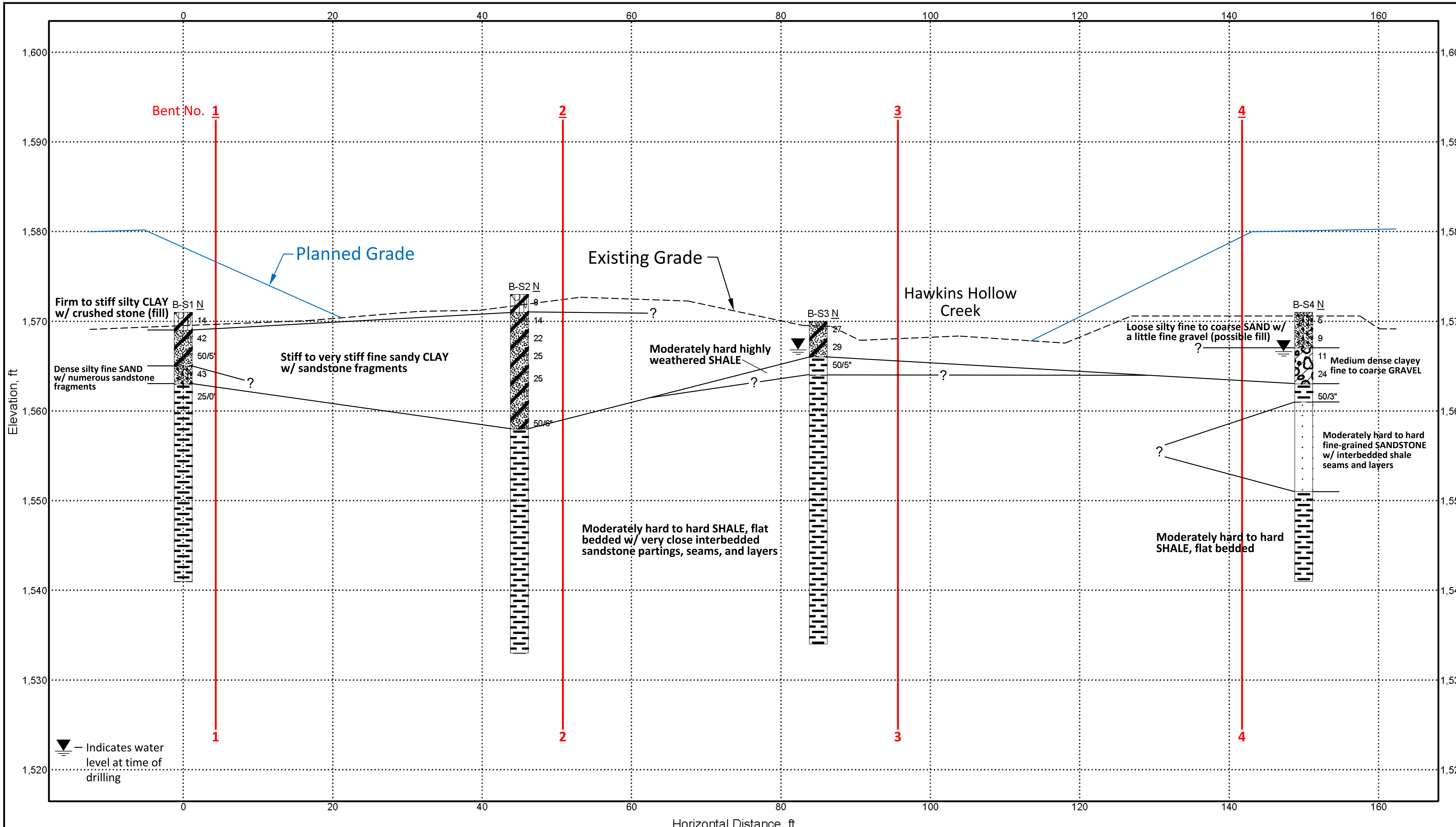
Plot scales: COHESION, TON/SQ FT 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4; PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10, 20, 30, 40, 50, 60, 70

NOTE: Auger refusal at 6.5 ft in shale w/interbedded sandstone.

COMPLETION DEPTH: 6.5 ft
DATE: 1-5-21

DEPTH TO WATER
IN BORING: Dry

DATE: 1/5/2021



Grubbs, Hoskyn, Barton & Wyatt, Inc.

NOTES:
1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface approximate.

SCALE:
1" = 12' Horizontal
1" = 10' Vertical

Generalized Subsurface Profile
090579 - Hwy 23 over Hawkins Hollow Creek
Madison County, Arkansas

Project Number: 20-138

Plate 9

ATTACHMENT 4

Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. B1

090579 - #M0547 RCB Approach Road
Madison County, Arkansas

TYPE: Auger

LOCATION: Site 3 - North of Bridge End

COHESION, TON/SQ FT: 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10, 20, 30, 40, 50, 60, 70

SURF. EL:

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
0	Stiff reddish brown and brown silty clay w/numerous sandstone fragments			
1		23		
2	- very stiff below 2 ft			
3		37		24
5		27		
7		31		
8	- auger refusal at 8 ft in sandstone			
10				
15				

COMPLETION DEPTH: 8.0 ft
DATE: 12-23-20

DEPTH TO WATER IN BORING: Dry

DATE: 12/23/2020

20-138

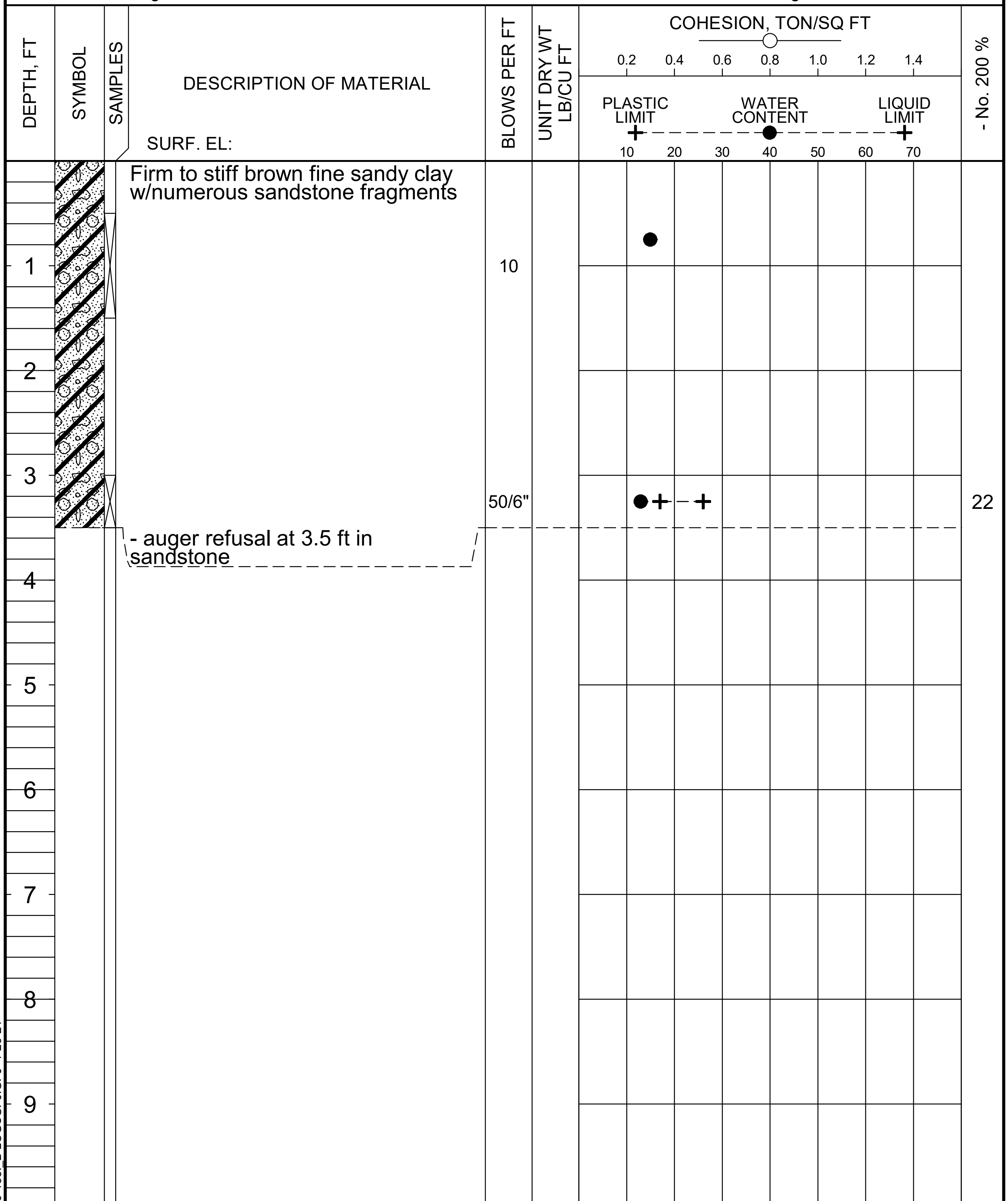
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. B2

090579 - #M0547 RCB Approach Road
Madison County, Arkansas

TYPE: Auger

LOCATION: Site 3 - South of Bridge End



LGBNEW 20-138_B LOGSGPJ.GPJ 1-29-21

COMPLETION DEPTH: 3.5 ft
DATE: 12-23-20

DEPTH TO WATER
IN BORING: Dry

DATE: 12/23/2020

PLATE 2

ATTACHMENT 5


20-138
B-52
Run 3: 25'-30'
Run 4: 30'-35'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
20'


20-138
B-52
Run5: 35'-40'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
24'
Bottom 29'


20-138
B-52
Run1: 15'-20'
Run2: 20'-25'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
Top

20-138
B-53
Run1: 6'-11'
Run2: 11'-16'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
Top
25
GHBW


11/28/18
B-2
5-8
20-25
20-138
B-53
Run 5: 26'-31'
Run 6: 31'-36'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS


20-138
B-53
Run 3: 16'-21'
Run 4: 21'-26'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
TOP
20'


S-7 14-19'
S-8 19
20-138
B-54
Run 3: 20'-25'
Run 4: 25'-30'
Grubbs, Hoskyn, Barton & Wyatt, Inc.
CONSULTING ENGINEERS
Top 14'
24'


20-138
B-54
Run1: 10'-15'
Run2: 15'-20'
Grubbs, Hoskyn, Barton & Wyatt INC.
CONSULTING ENGINEERS


20-138
B-51
Run 3: 20'-25'
Run 4: 25'-30'
Grubbs, Hoskyn, Barton & Wyatt INC.
CONSULTING ENGINEERS


20-138
B-51
Run 1: 10'-15'
Run 2: 15'-20'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS
BOART LONGYEAR
1-800-453-8740
NO. 79737 NQ

ATTACHMENT 6

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 090579 Hwy. 23 Strs & Apprs
LOCATION: Madison County
GHBW JOB NUMBER: 20-138

BORING No.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING								USCS CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
A1	2.5-3.5	18	39	24	15	100	100	81	65	60	55	36	18	SM-SC	A-2-6
A2	0.5-1.5	10	26	17	9	100	100	71	53	44	38	31	29	GM-GC	A-2-4
S1	6.5-7.5	10	---	---	---	100	79	68	49	37	29	22	13	GM	A-1-a
S2	4.5-5.5	7	26	17	9	---	---	---	---	---	---	---	---	CL	A-4
S4	0.5-1.5	17	---	---	---	100	100	100	90	82	71	50	19	SM	A-1-b
S4	4.5-5.5	19	28	18	10	100	100	100	87	76	64	50	29	SC	A-2-6
P1	2.5-3.5	13	22	20	2	---	---	---	---	67	---	---	23	SM-SC	A-2-4
P2	0.5-1.5	22	38	21	17	---	---	---	---	74	---	---	41	SM-SC	A-6
P3	2.5-3.5	13	28	18	10	---	---	---	---	75	---	---	26	SM-SC	A-2-4
P4	0.5-1.5	14	35	21	14	---	---	---	---	58	---	---	31	SM-SC	A-2-6
B1	2.5-3.5	10	27	18	9	100	100	100	76	71	66	53	24	SM-SC	A-2-4
B2	2.5-3.5	13	26	17	9	100	100	90	85	83	76	60	22	SM-SC	A-2-4

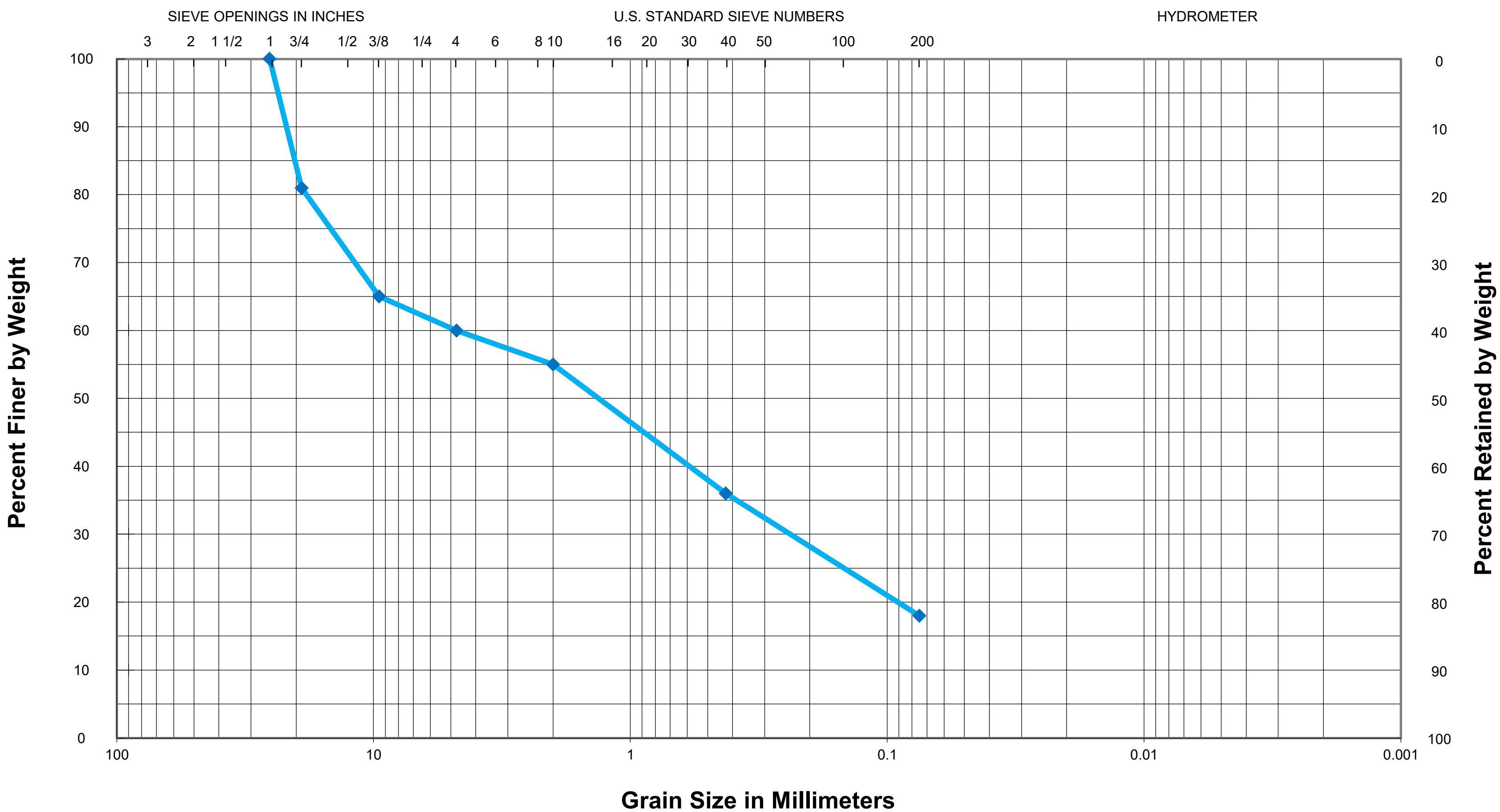
Grubbs, Hoskyn,
Barton & Wyatt, Inc.
CONSULTING ENGINEERS

PLATE

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

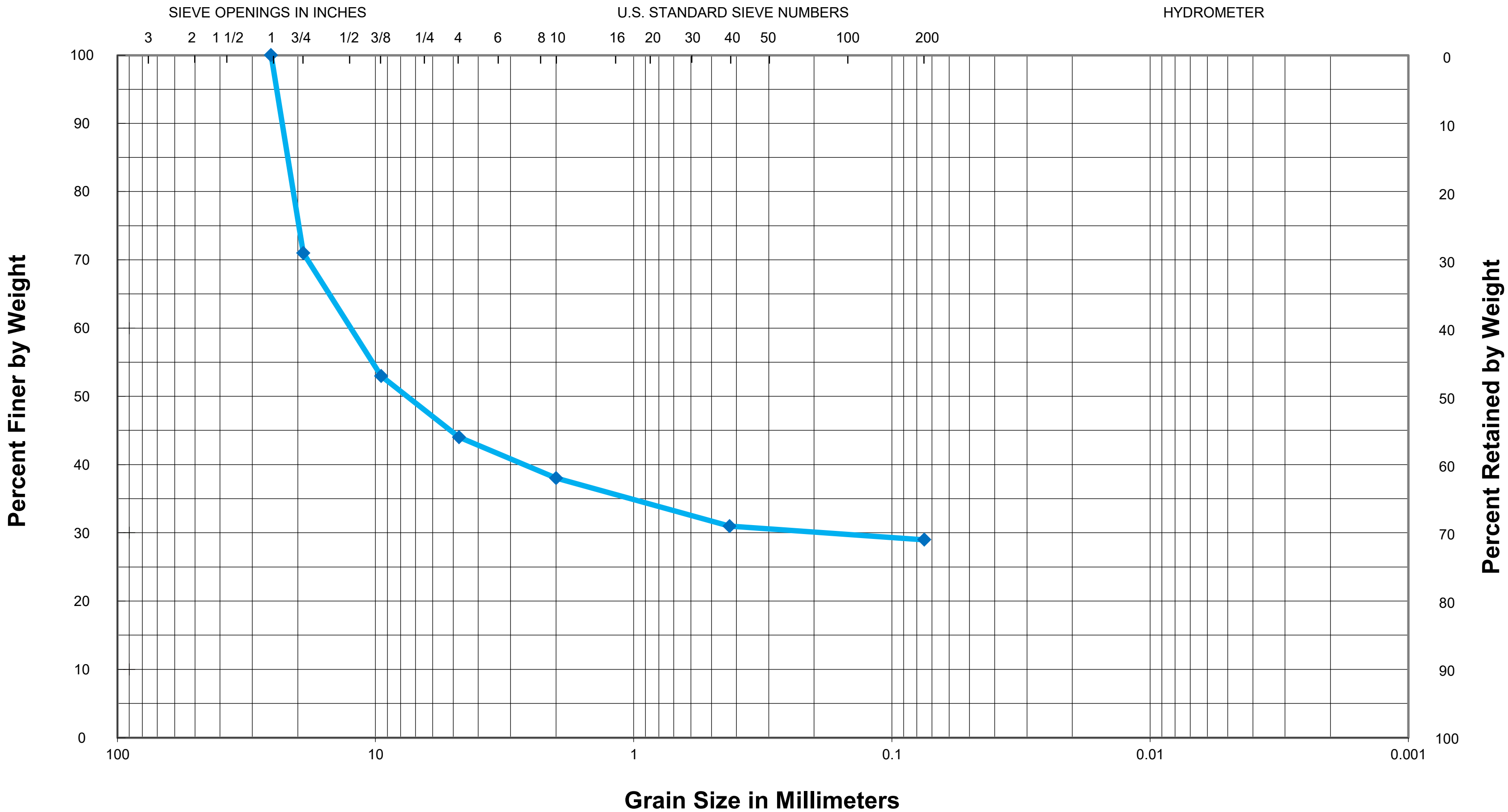
Sample: Boring A1, 2.5-3.5 ft; LL = 39, PL = 24, PI = 15
Description: Reddish brown clayey fine to coarse SAND with sandstone fragments

USCS Classification = SM-SC
AASHTO Classification = A-2-4

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

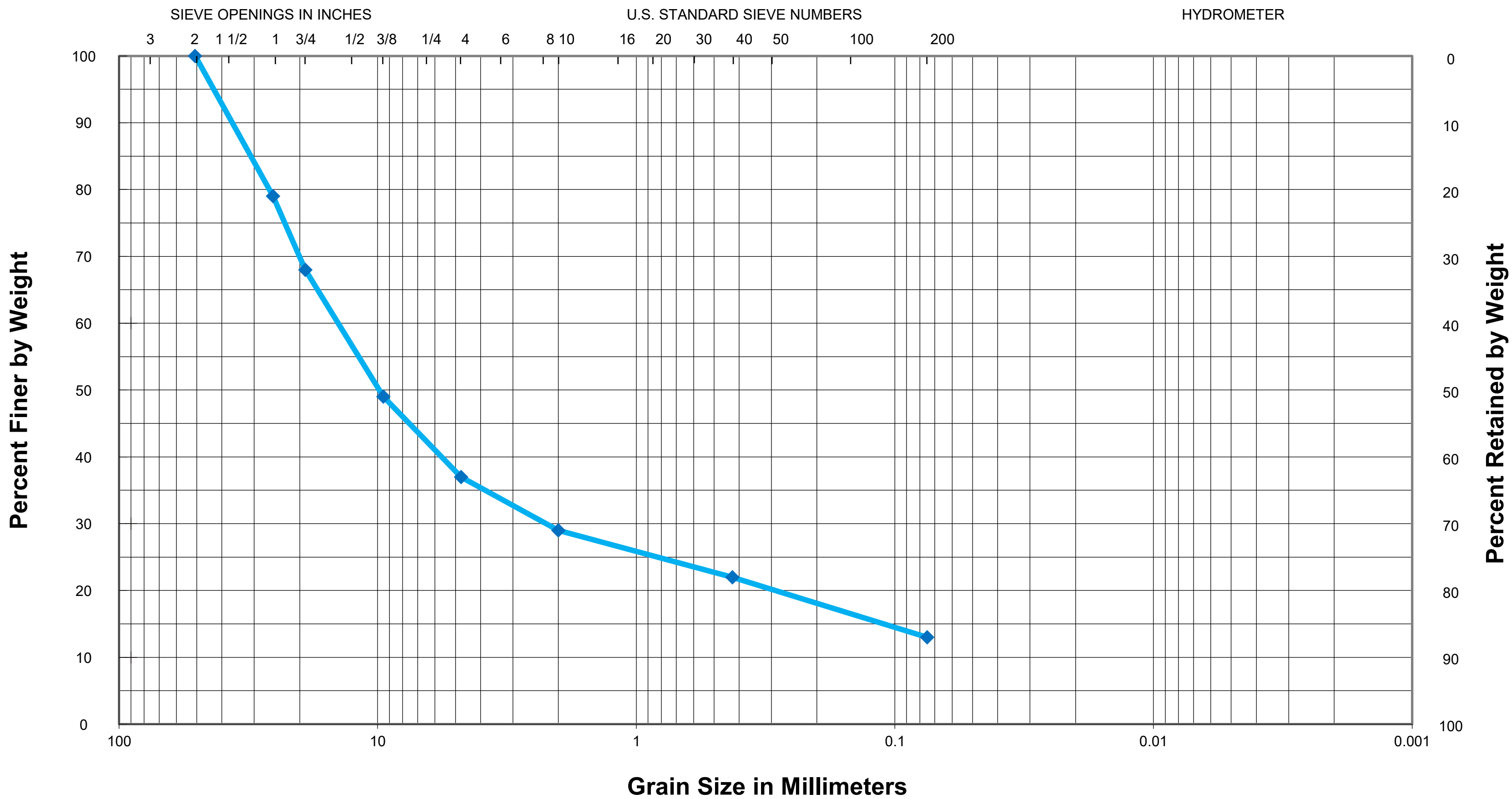
Sample: Boring A2, 0.5-1.5 ft; LL = 26, PL = 17, PI = 9
Description: Reddish brown medium to coarse sandy CLAY with numerous fine to coarse sandstone fragments

USCS Classification = GM-GC
AASHTO Classification = A-2-4

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

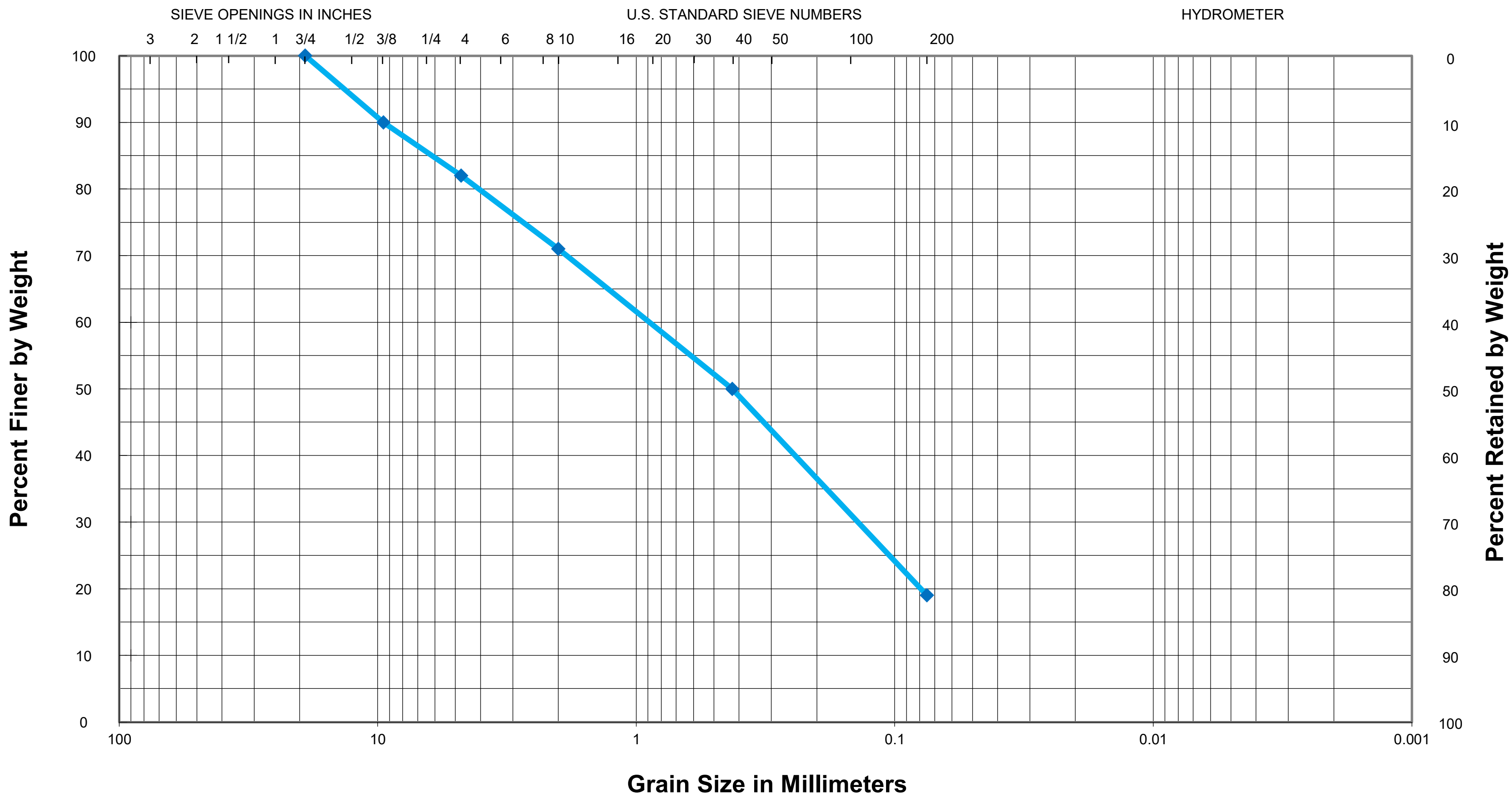
Sample: Boring S1, 6.5-7.5 ft
Description: Brown silty fine to coarse SAND with numerous sandstone fragments

USCS Classification = GM
AASHTO Classification = A-1-a

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

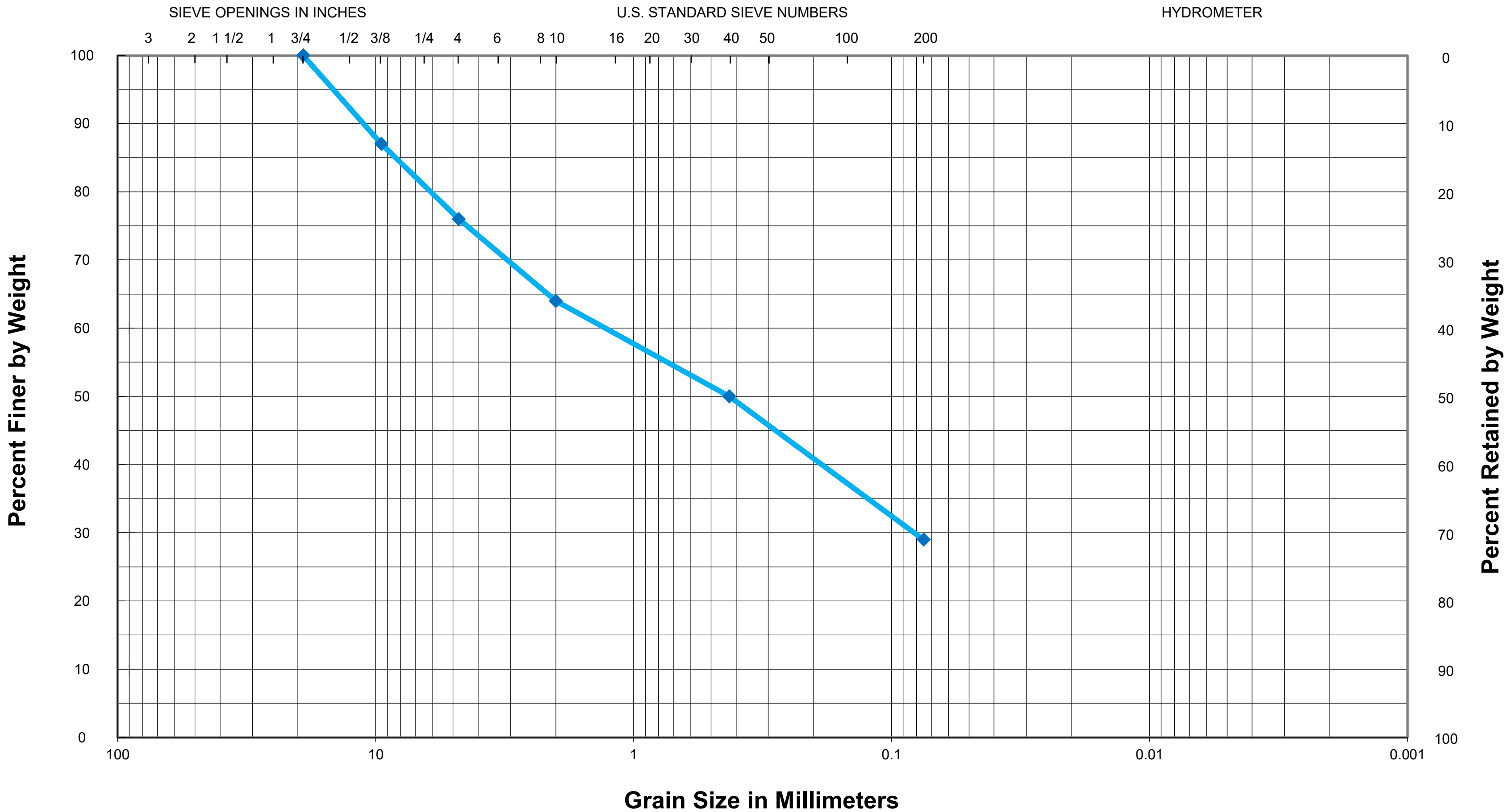
Sample: Boring S4, 0.5-1.5 ft
Description: Brown silty fine to coarse SAND with a little fine gravel

USCS Classification = SM
AASHTO Classification = A-1-b

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

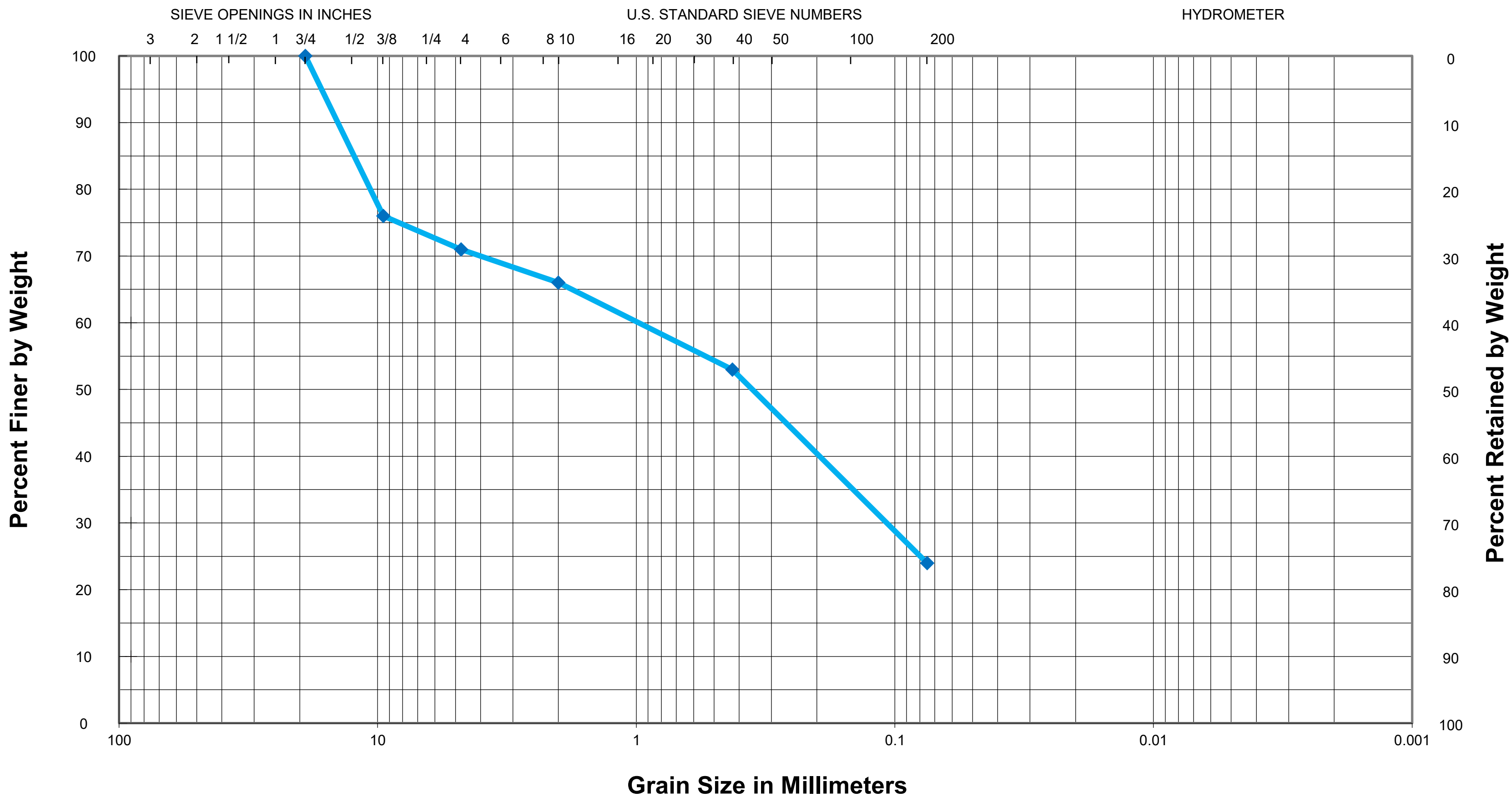
Sample: Boring S4, 4.5-5.5 ft; LL = 28, PL = 18, PI = 10
Description: Reddish brown and gray clayey fine to coarse SAND with some fine gravel

USCS Classification = SC
AASHTO Classification = A-2-6

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

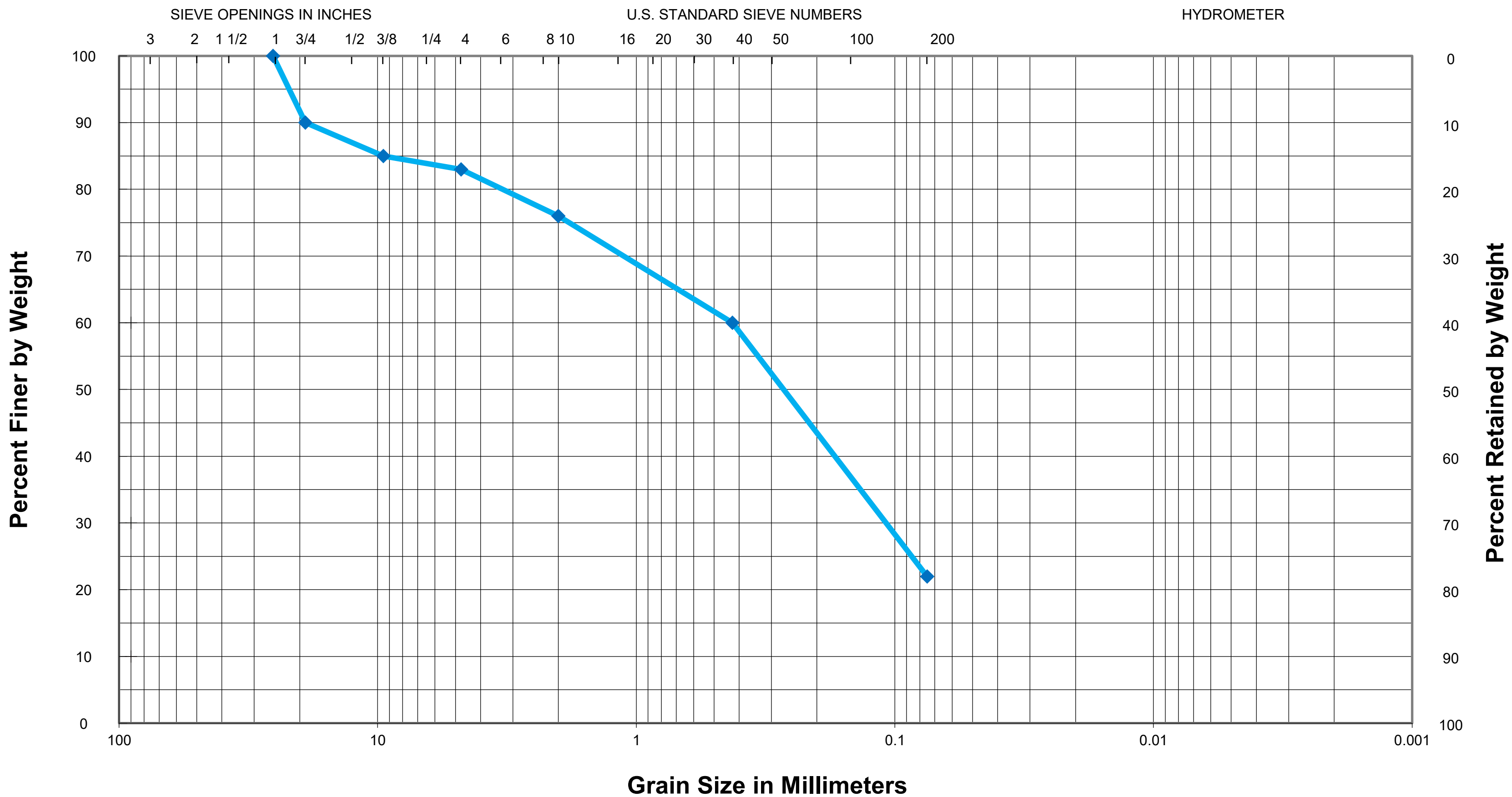
Sample: Boring B1, 2.5-3.5 ft; LL = 27, PL =18 , PI = 9
Description: Reddish brown and brown clayey fine to coarse SAND with fine sandstone fragments

USCS Classification = SM-SC
AASHTO Classification =A-2-4

20-138

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring B2, 2.5-3.5 ft; LL = 26, PL = 17, PI = 9
Description: Brown clayey fine to coarse SAND with a little fine to coarse sandstone fragments

USCS Classification = SM-SC
AASHTO Classification = A-2-4

ATTACHMENT 7

Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

P.O. Box 30970
Little Rock, Arkansas 72260-0970
#1 Trigon Place 72209
(501) 455-2536
FAX (501) 455-4137

REPORT OF MODIFIED PROCTOR TEST
(AASHTO T 180)

Project: 090579 Hwy 23 Strs and Apprs - Madison County, AR   Job No: 20-138
Material Description: Brown fine sandy CLAY w/ shale fragments and cobbles (fill)
Location Sampled/Source: TP-1
Sample Depth, ft: 1-2
Date Sampled: 12/10/2020
Date Tested: 12/16/2020
Tested By: JW
Report Date: 1/7/2021

LAB COMPACTION PROCEDURE: AASHTO T-180 Method: D	
Maximum Unit Dry Wt. (pcf):	121.6
Optimum Water Content (%):	11.9

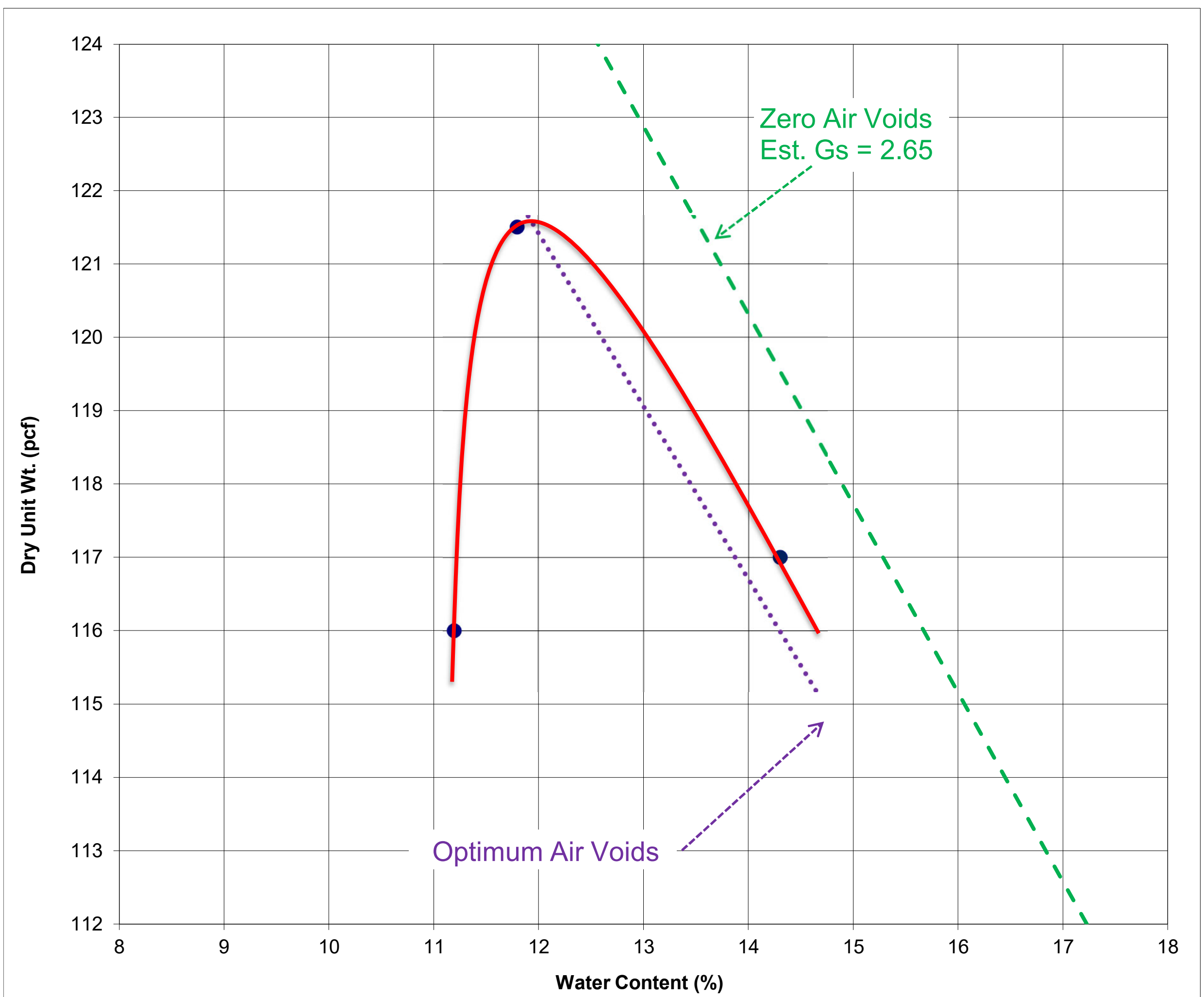
As Received Water Content: 12.8 %

ATTERBERG LIMITS AASHTO T-89 & T-90
Liquid Limit: 44
Plastic Limit: 23
Plasticity Index: 21

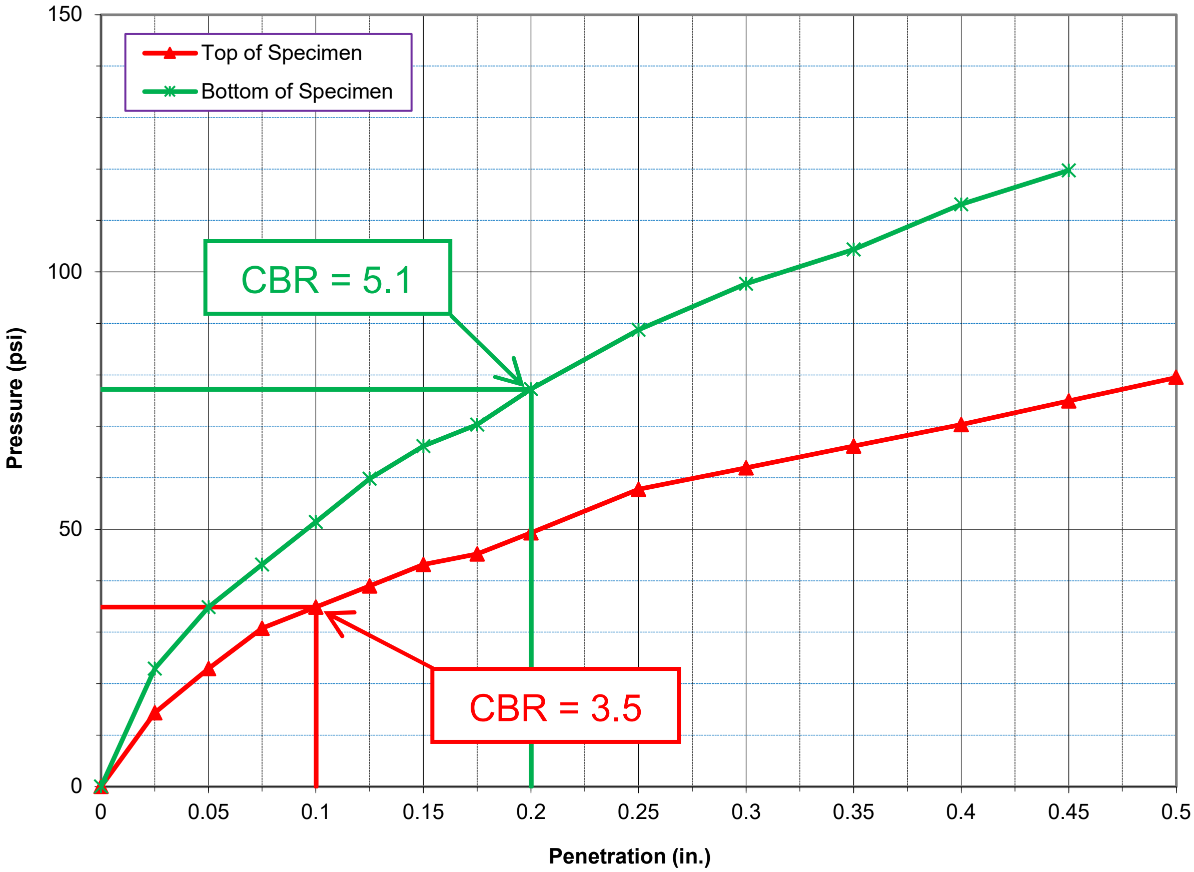
USCS Classification:
GC

AASHTO Classification:
A-2-7

GRADATION AASHTO T-88	
Sieve Number	Percent Passing
3 in.	100
2 in.	96
3/4 in.	85
3/8 in.	70
#4	63
#10	55
#40	45
#200	34



Laboratory CBR Test Report
(AASHTO T-193)



Boring No./Depth, ft	Classification		Natural Moisture Content, %	Assumed Specific Gravity	Liquid Limit, %	Plastic Limit, %	% Retained No.4	% Passing No.200
	USCS	AASHTO						
TP-1 / 1-2'	GC	A-2-7	12.8	2.65	44	23	37	34
PROCTOR TEST RESULTS (AASHTO T-180)				**MATERIAL DESCRIPTION**				
Optimum Moisture Content = 11.9% Maximum Dry Density = 121.6 pcf				Brown fine sandy CLAY w/ shale fragments and cobbles (fill)				

Remarks:

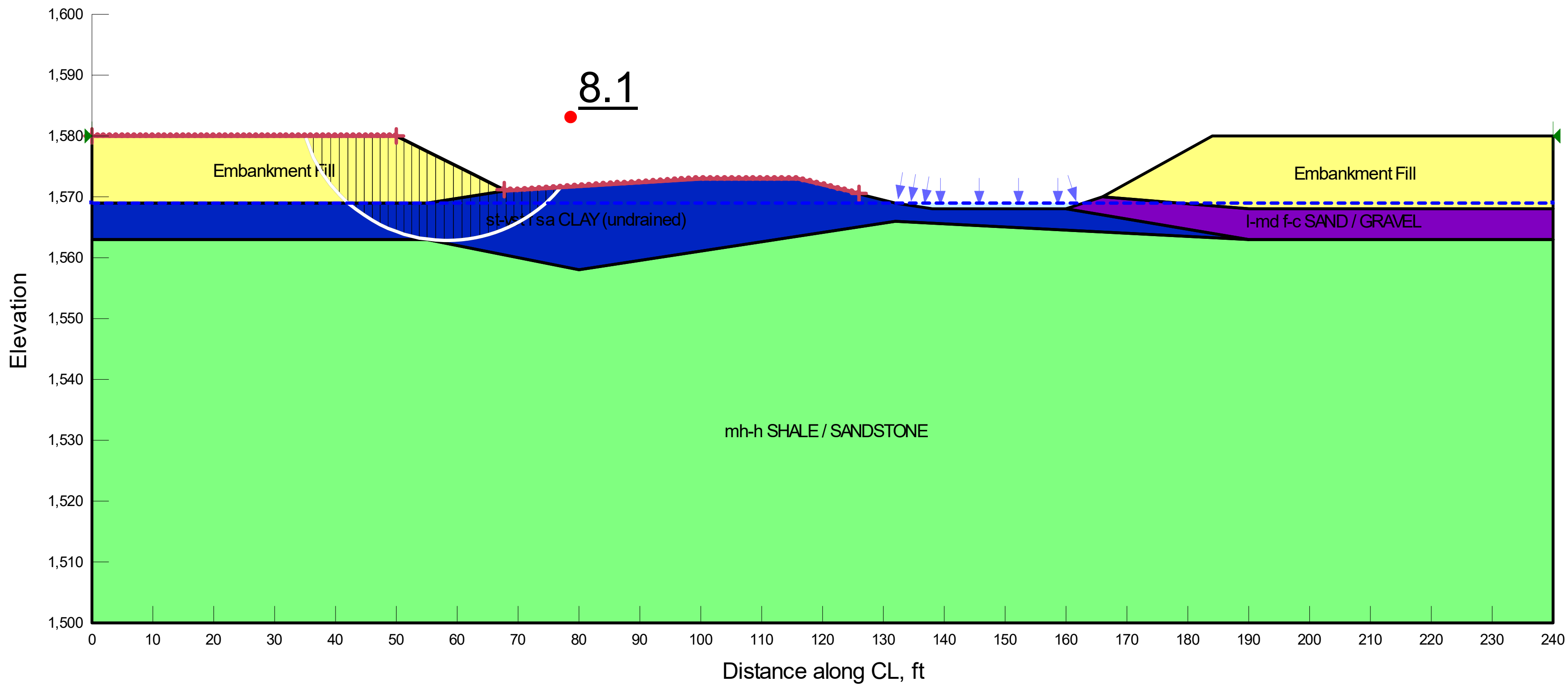
As Molded: γ_d = 115.0 pcf @ 14.0%; Percent swell: 2.6%

Grubbs, Hoskyn, Barton & Wyatt, INC. CONSULTING ENGINEERS	Project: 090579 Hwy 23 Strs and Apprs
	GHBW Project Number: 20-138
	Location: Marion County, Arkansas
	Sample Date: 12/10/20
	Test Date: 12/19/20

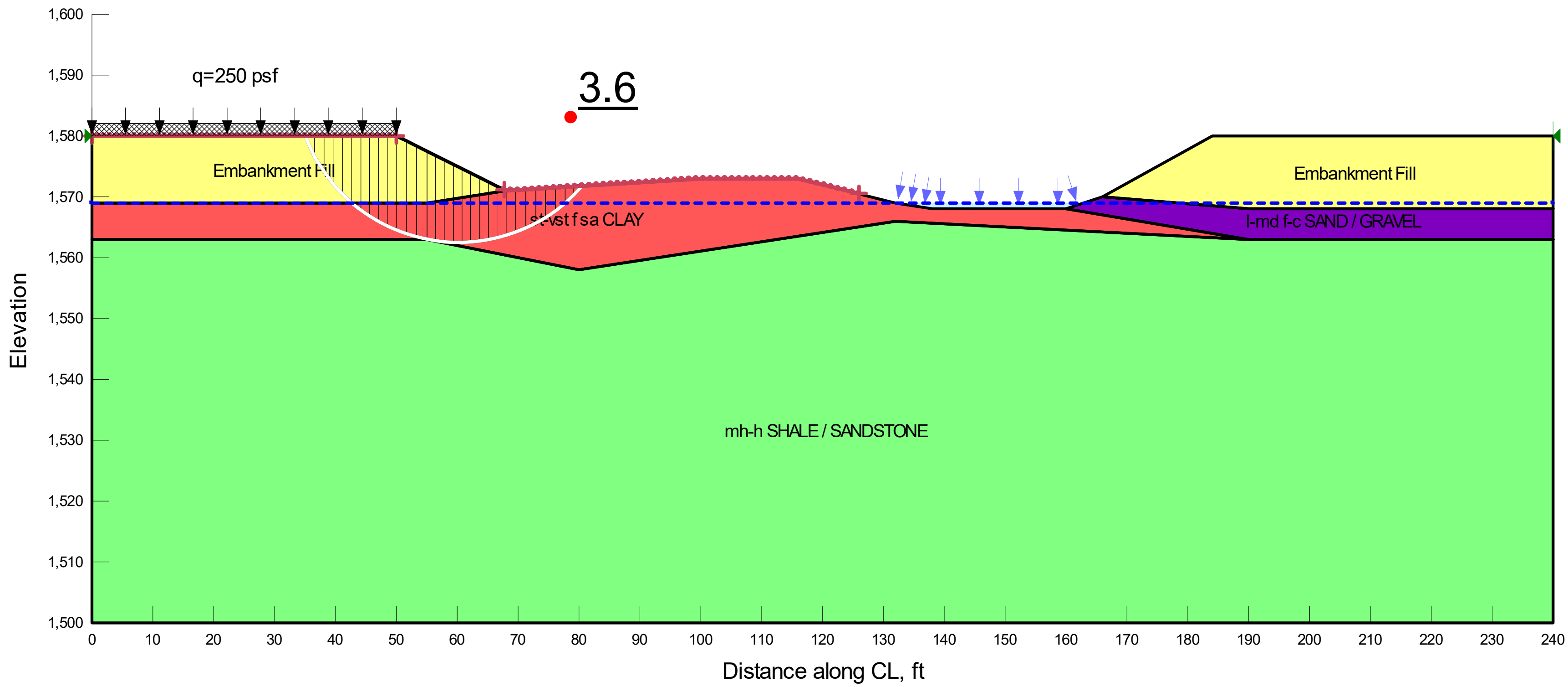
ATTACHMENT 8

Summary of Stability Analysis Results
ARDOT 090579 Bridge #02946
GHBW Job No. 20-138
Madison County, Arkansas

Embankment Slope	Design Loading Condition	Calculated Minimum Factor of Safety
South End Slope (Bent 1)	End of Construction	8.6
	Long Term	3.6
	Rapid Drawdown from El 1574 to El 1569	4.1
	Seismic ($k_h = A_S/2 = 0.031$)	4.0
South Side Slope (Bent 1)	End of Construction	9.3
	Long Term	4.2
	Rapid Drawdown from El 1574 to Existing Grade	4.5
	Seismic ($k_h = A_S/2 = 0.031$)	4.4
North End Slope (Bent 4)	End of Construction	4.4
	Long Term	3.3
	Rapid Drawdown from El 1574 to El 1569	3.6
	Seismic ($k_h = A_S/2 = 0.031$)	3.6
North Side Slope (Bent 4)	End of Construction	5.3
	Long Term	4.4
	Rapid Drawdown from El 1574 to Existing Grade	4.5
	Seismic ($k_h = A_S/2 = 0.031$)	4.7

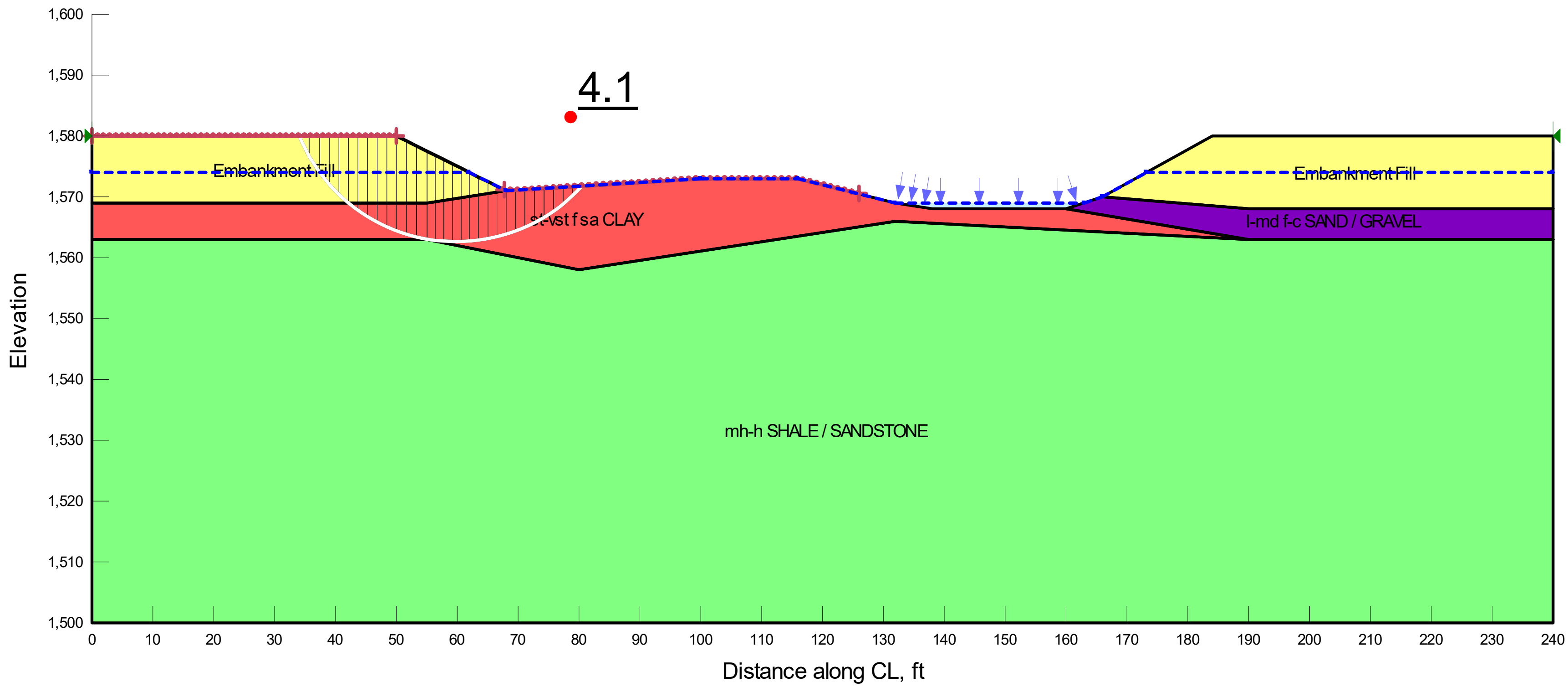


Results of Stability Analyses – End of Construction
South End Slope (Bent 1)
2H:1V Slope, H=9 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

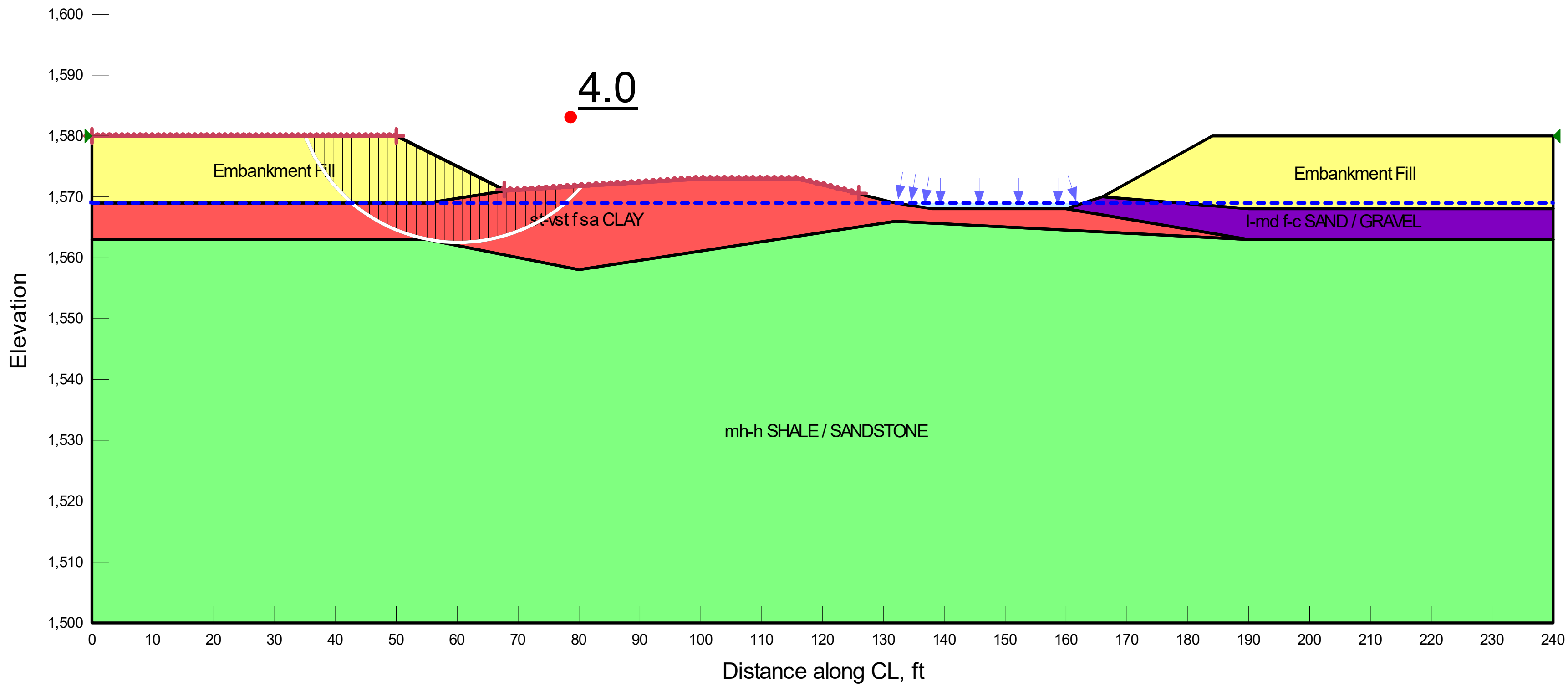


Results of Stability Analyses – Long Term Condition
South End Slope (Bent 1)
2H:1V Slope, H=9 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition, El 1574 to El 1569
South End Slope (Bent 1)
2H:1V Slope, H=9 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.031$)
South End Slope (Bent 1)
2H:1V Slope, H=9 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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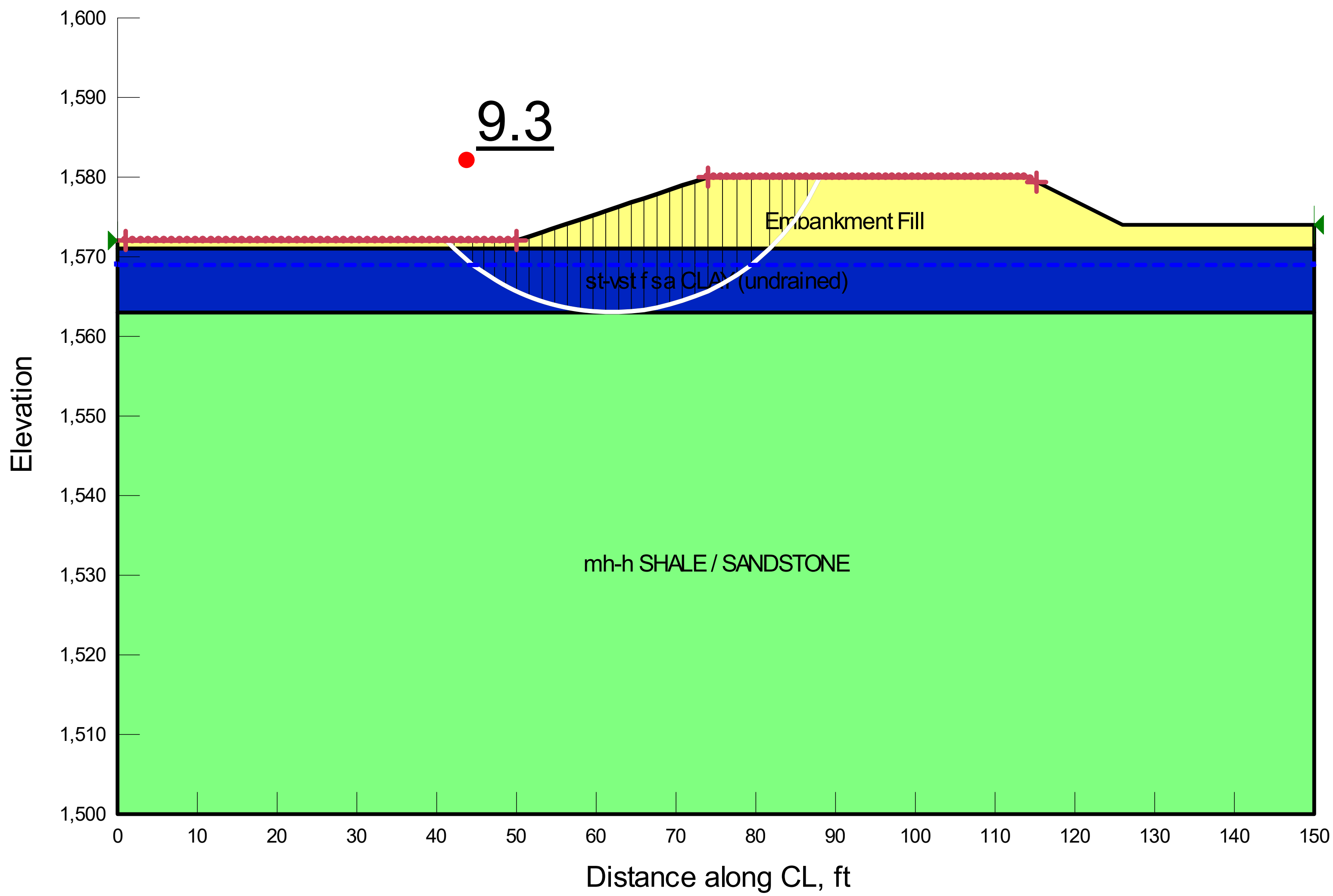

Results of Stability Analyses – End of Construction
South Side Slope (Bent 1)
3H:1V Slope, H=8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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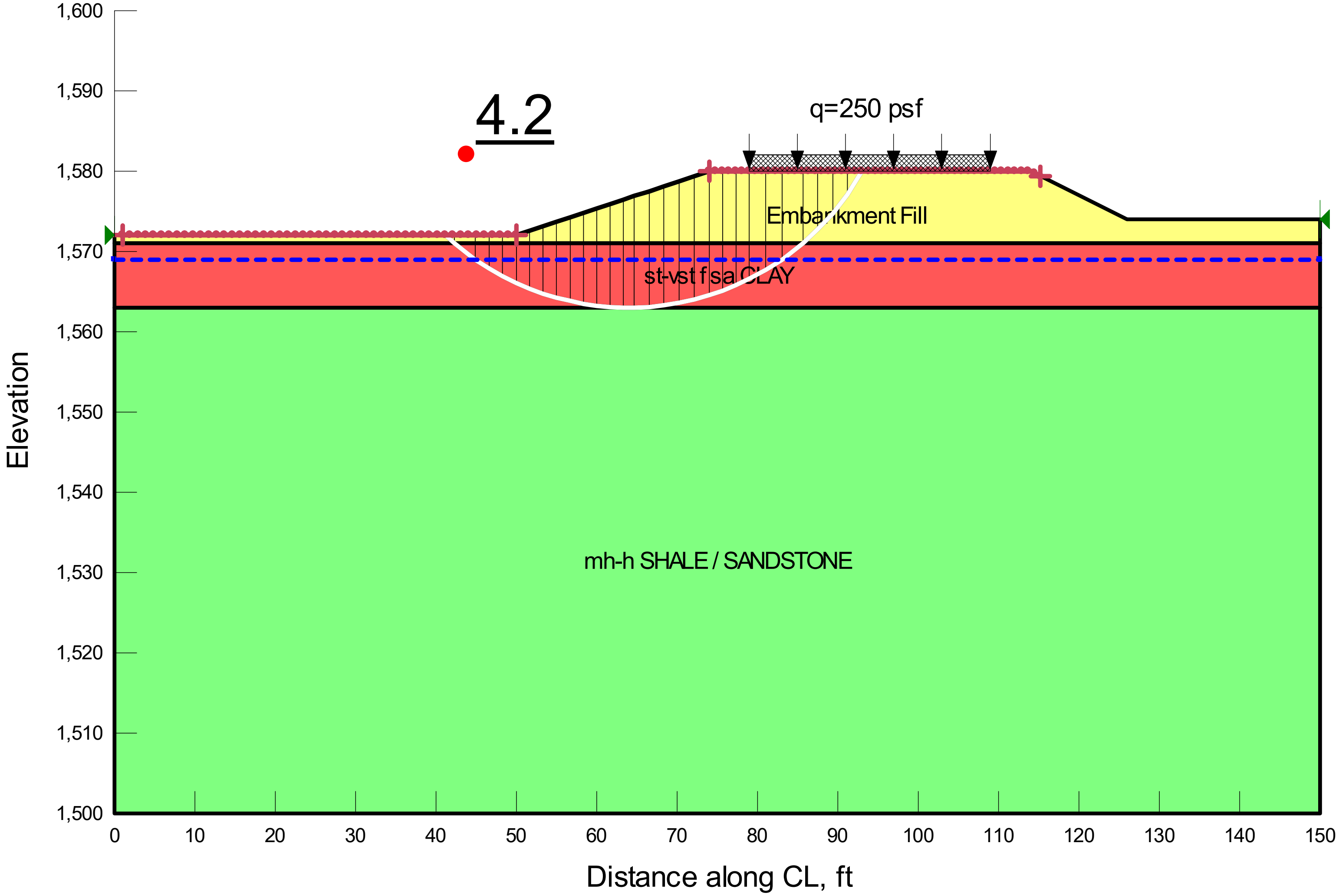
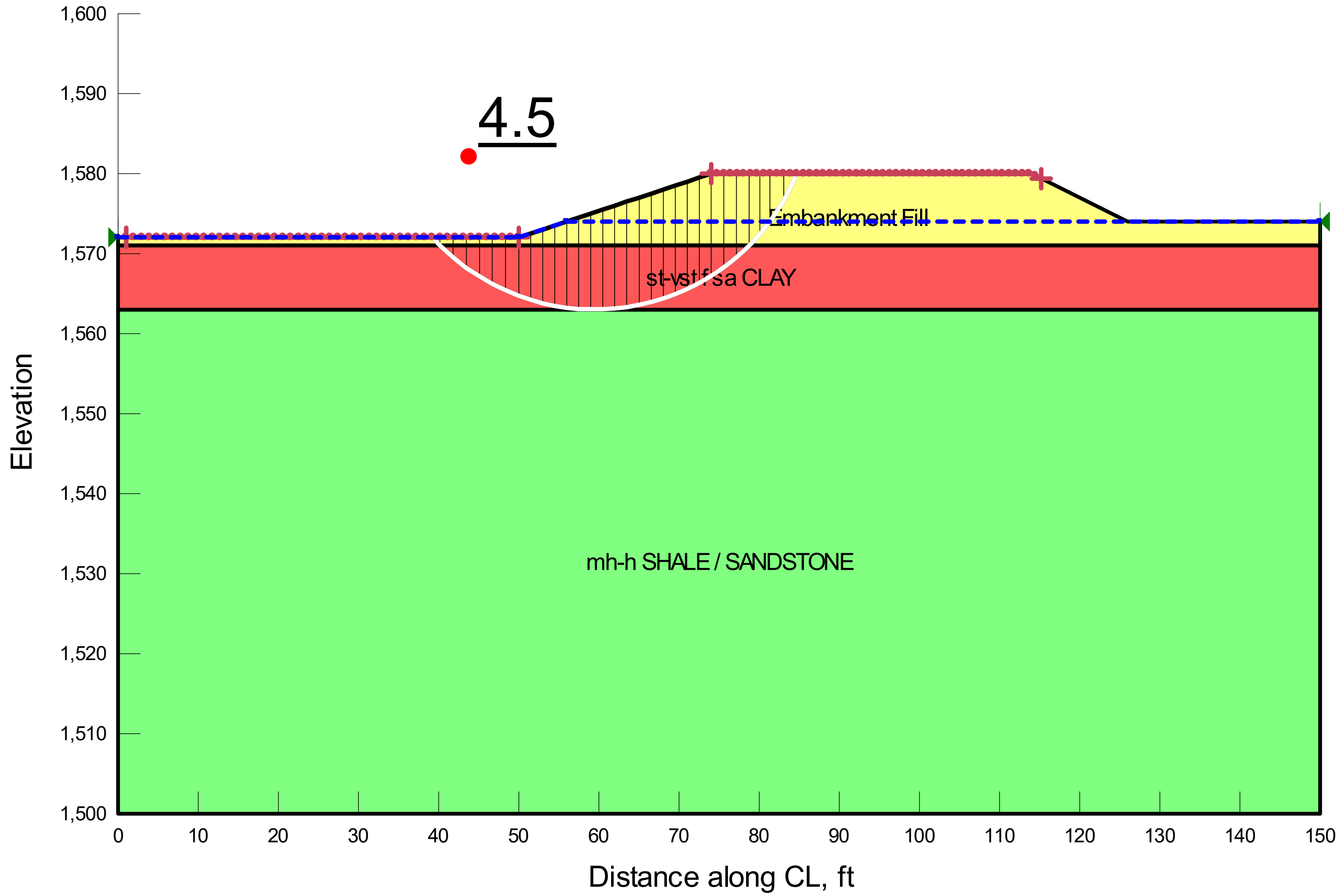


Results of Stability Analyses – Long Term Condition
South Side Slope (Bent 1)
3H:1V Slope, H=8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946



Results of Stability Analyses – Rapid Drawdown Condition, El 1574 to Existing Grade
South Side Slope (Bent 1)
3H:1V Slope, H=8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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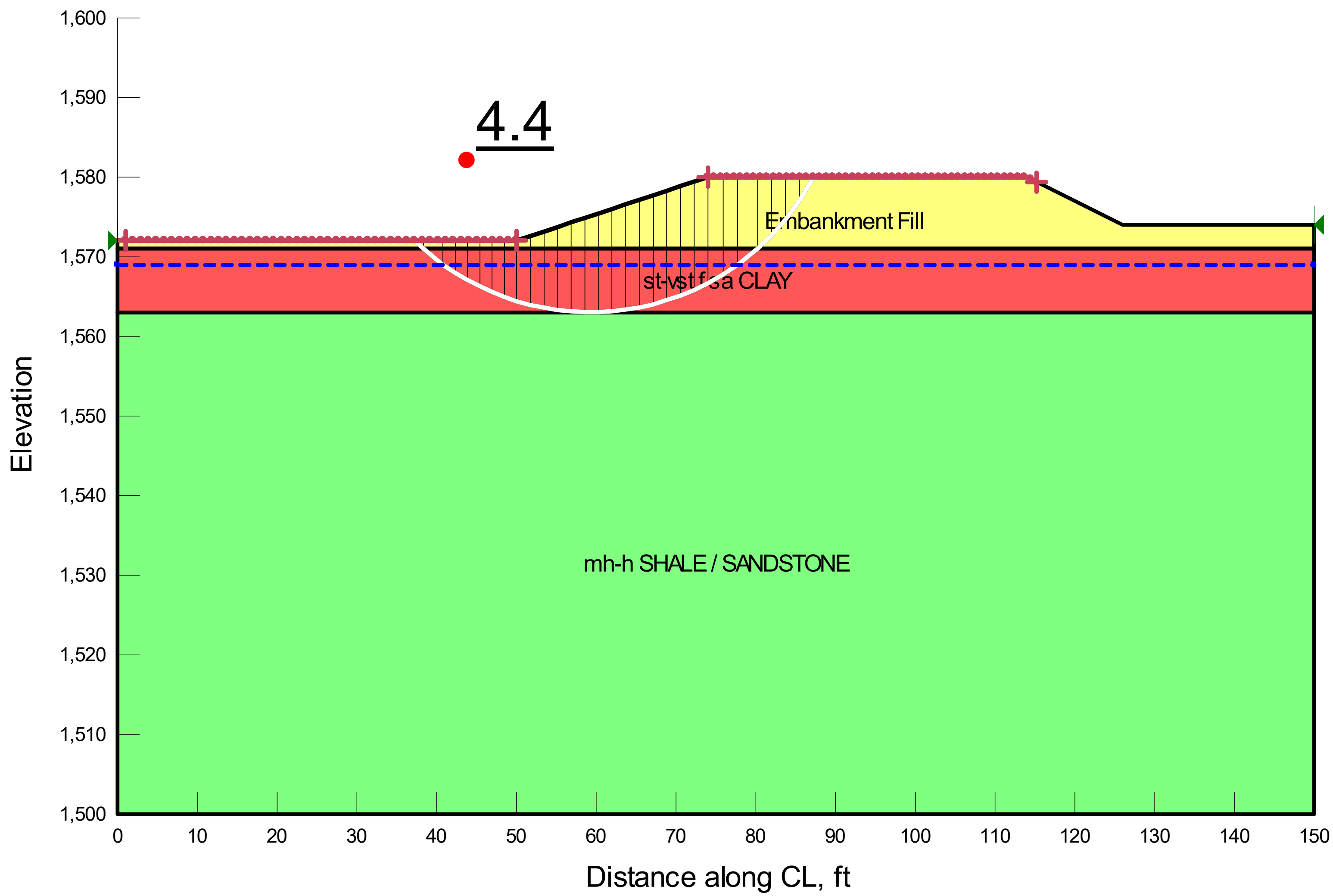

Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.031$)
South Side Slope (Bent 1)
3H:1V Slope, H=8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

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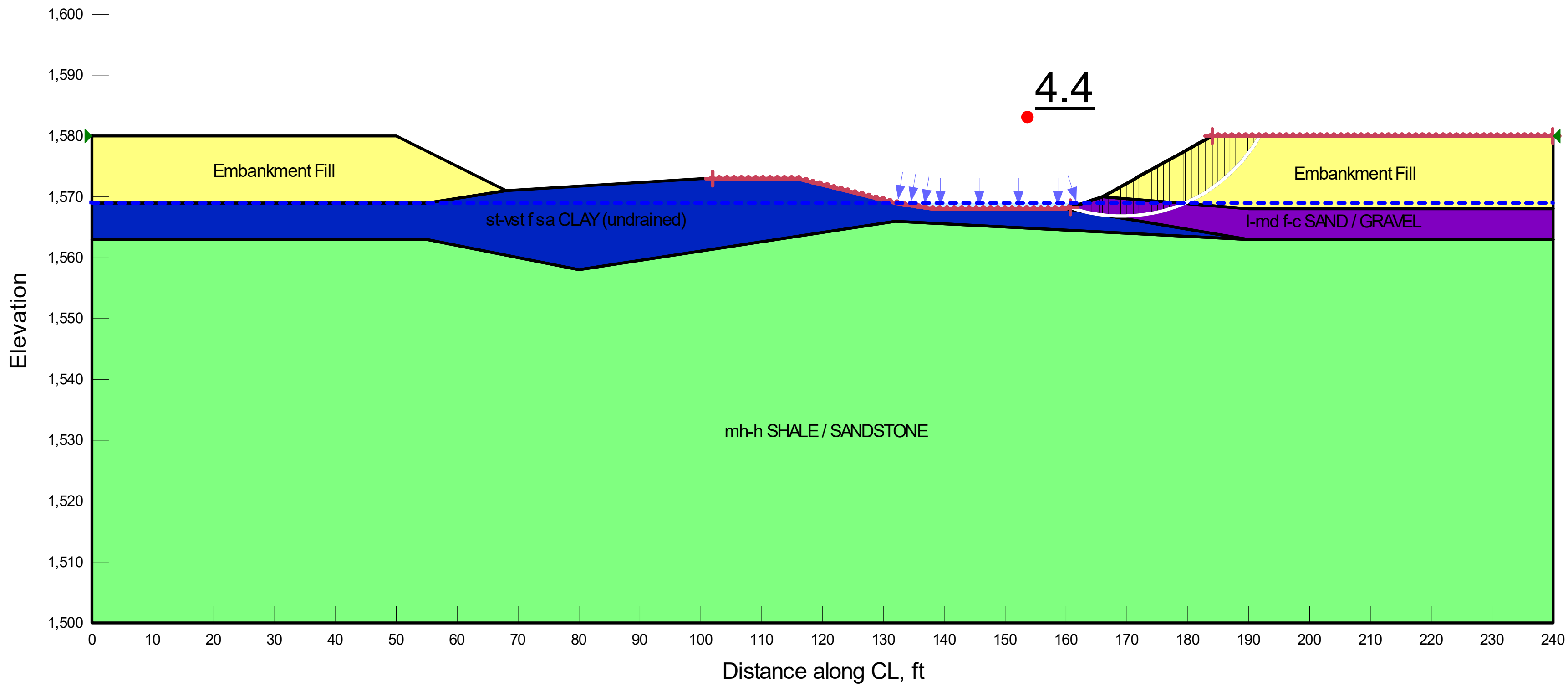

Results of Stability Analyses – End of Construction
North End Slope (Bent 4)
2H:1V Slope, H= 12 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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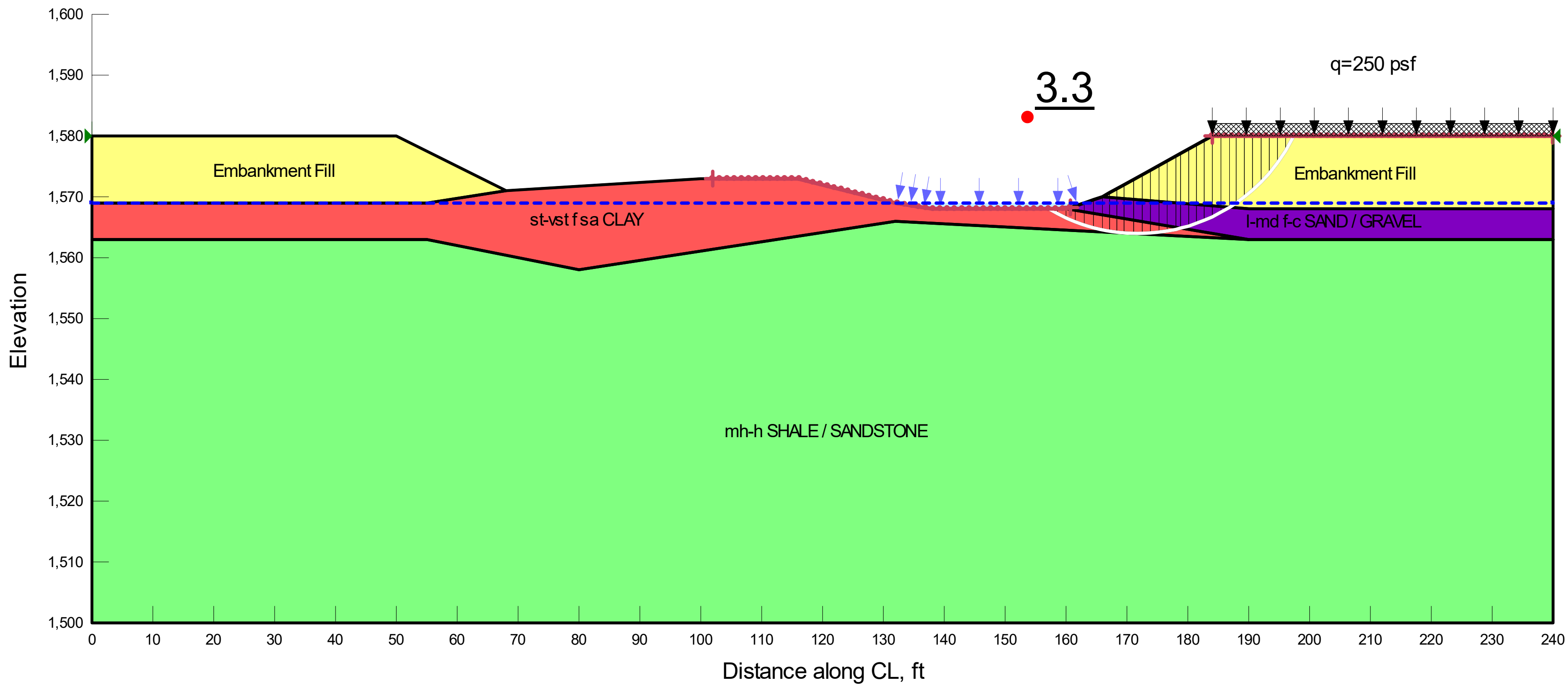
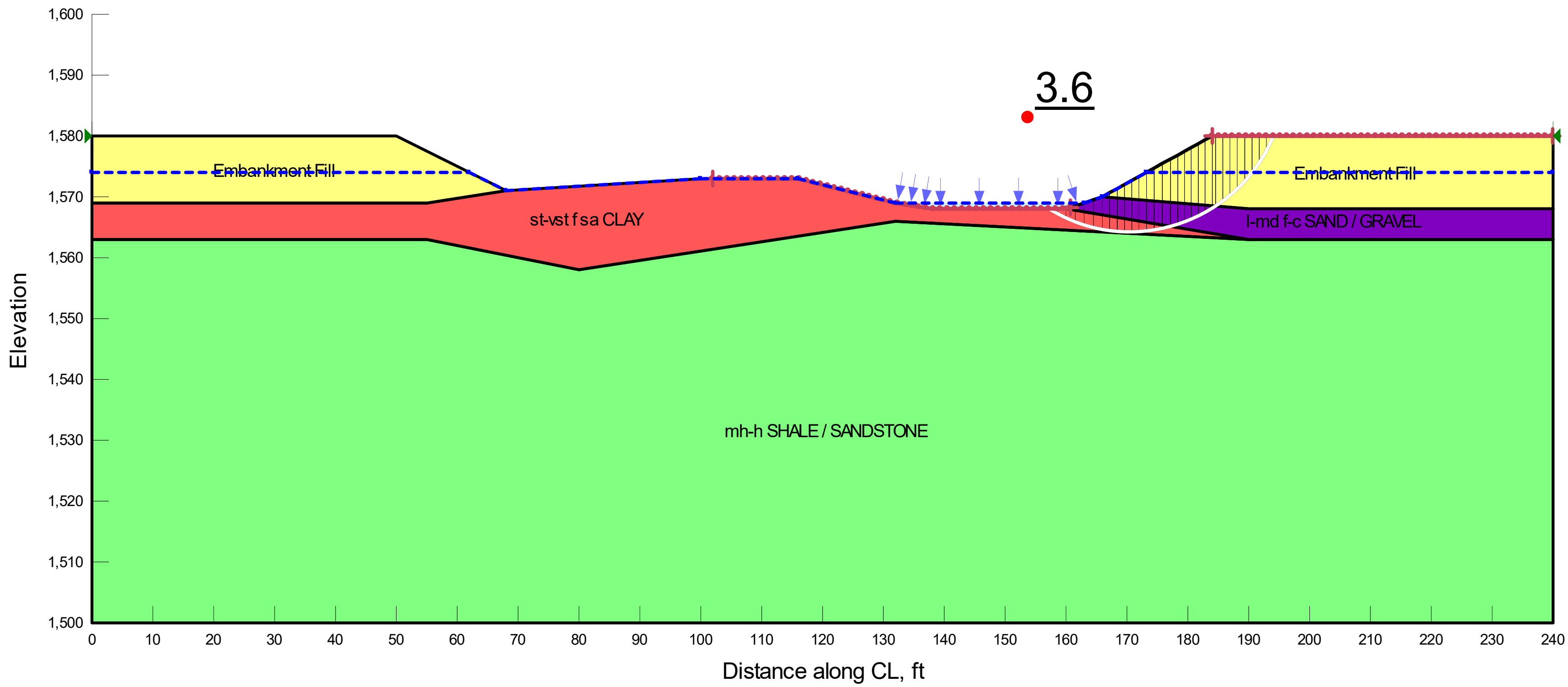


Results of Stability Analyses – Long Term Condition
North End Slope (Bent 4)
2H:1V Slope, H= 12 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946



Results of Stability Analyses – Rapid Drawdown Condition, El 1574 to El 1569
North End Slope (Bent 4)
2H:1V Slope, H= 12 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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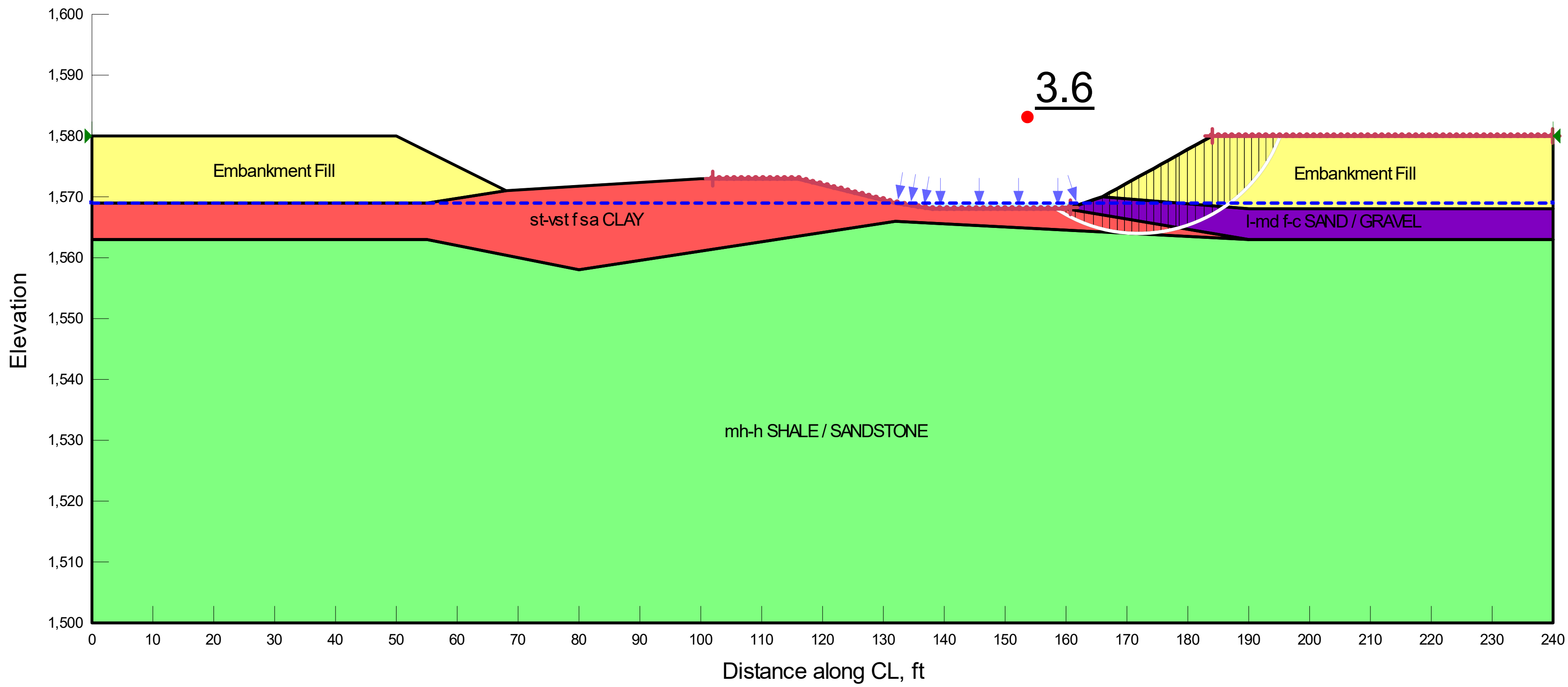
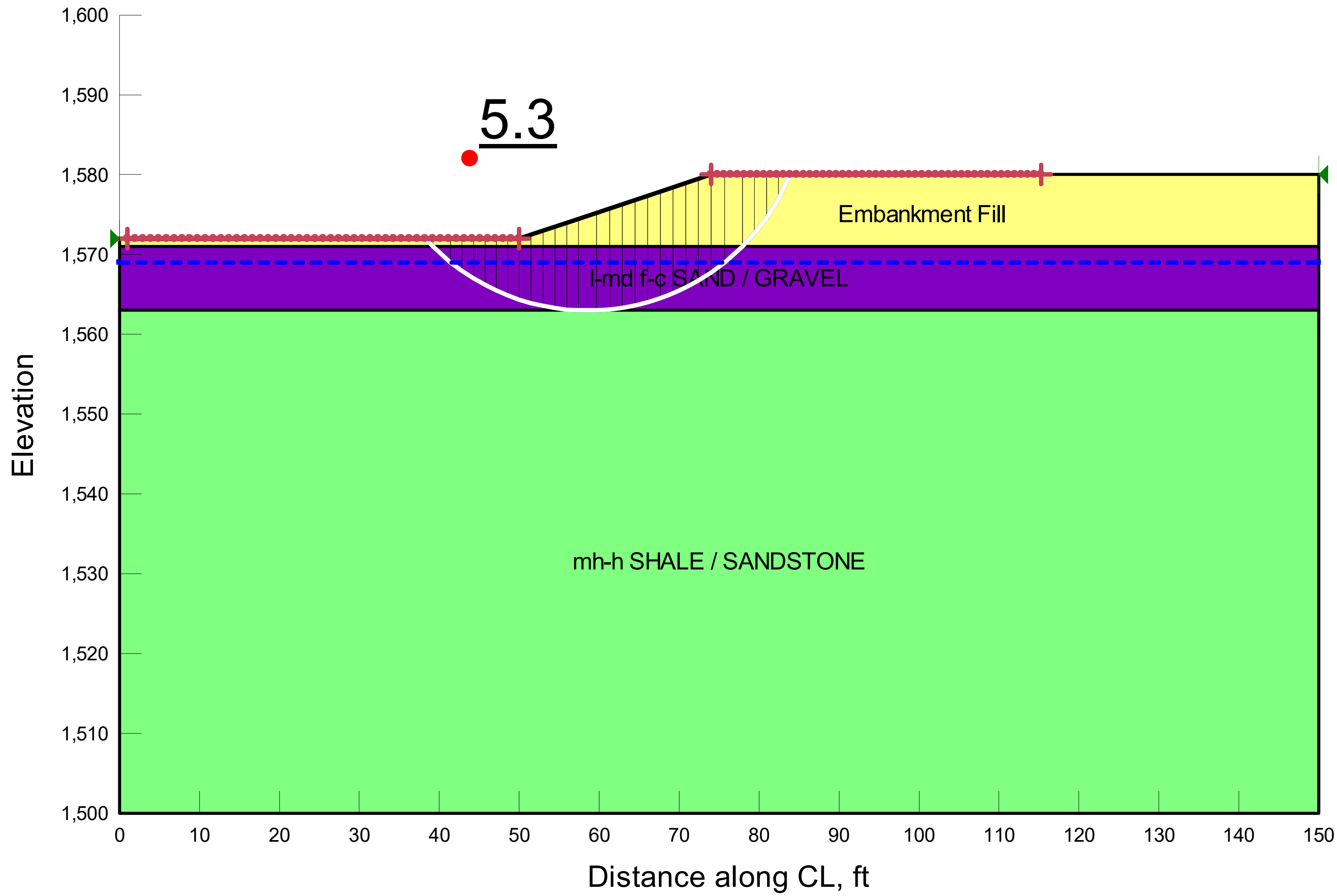
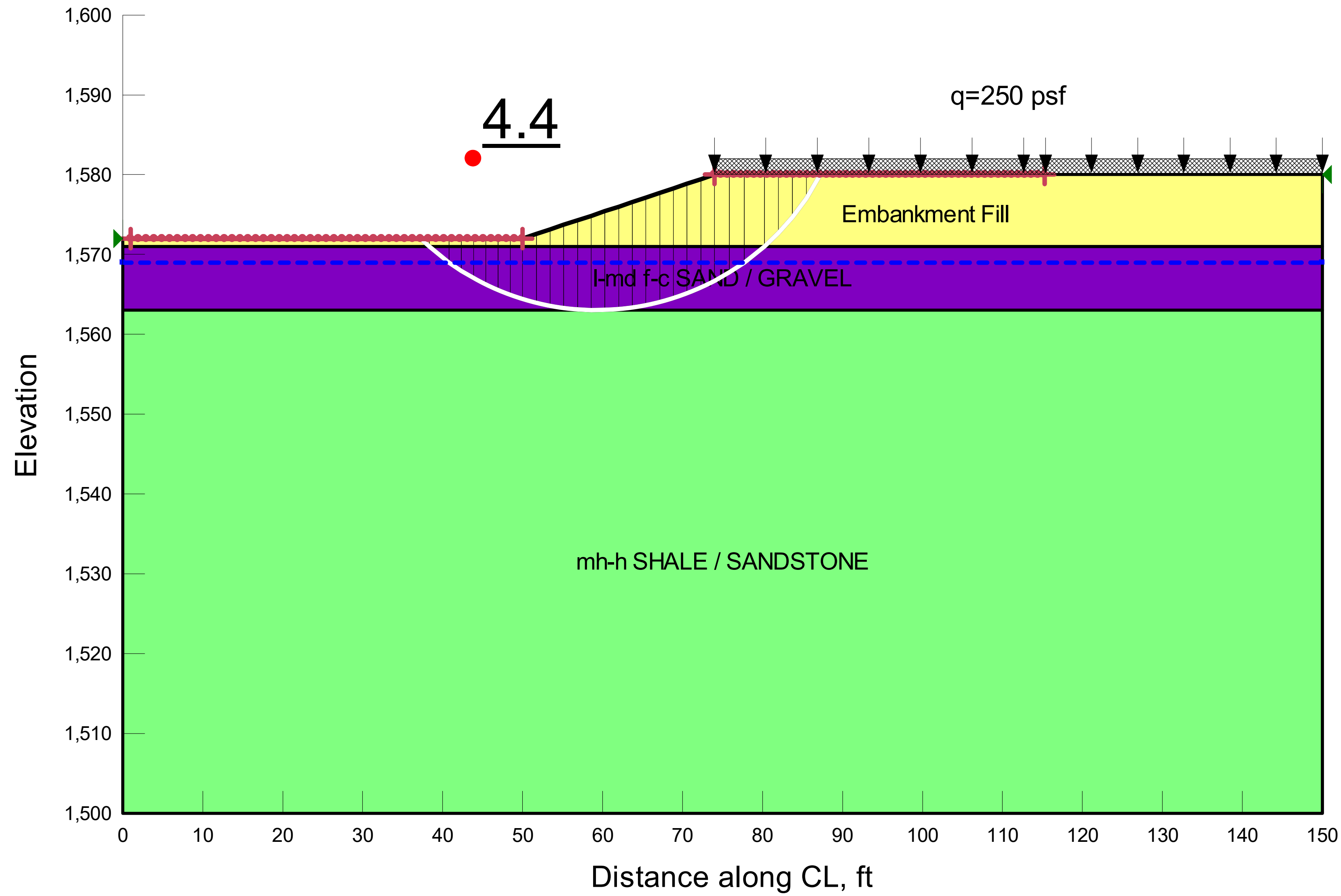


Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.031$)
North End Slope (Bent 4)
2H:1V Slope, H= 12 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946



Results of Stability Analyses – End of Construction
North Side Slope (Bent 4)
3H:1V Slope, H= 8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Long Term Condition
North Side Slope (Bent 4)
3H:1V Slope, H= 8 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946

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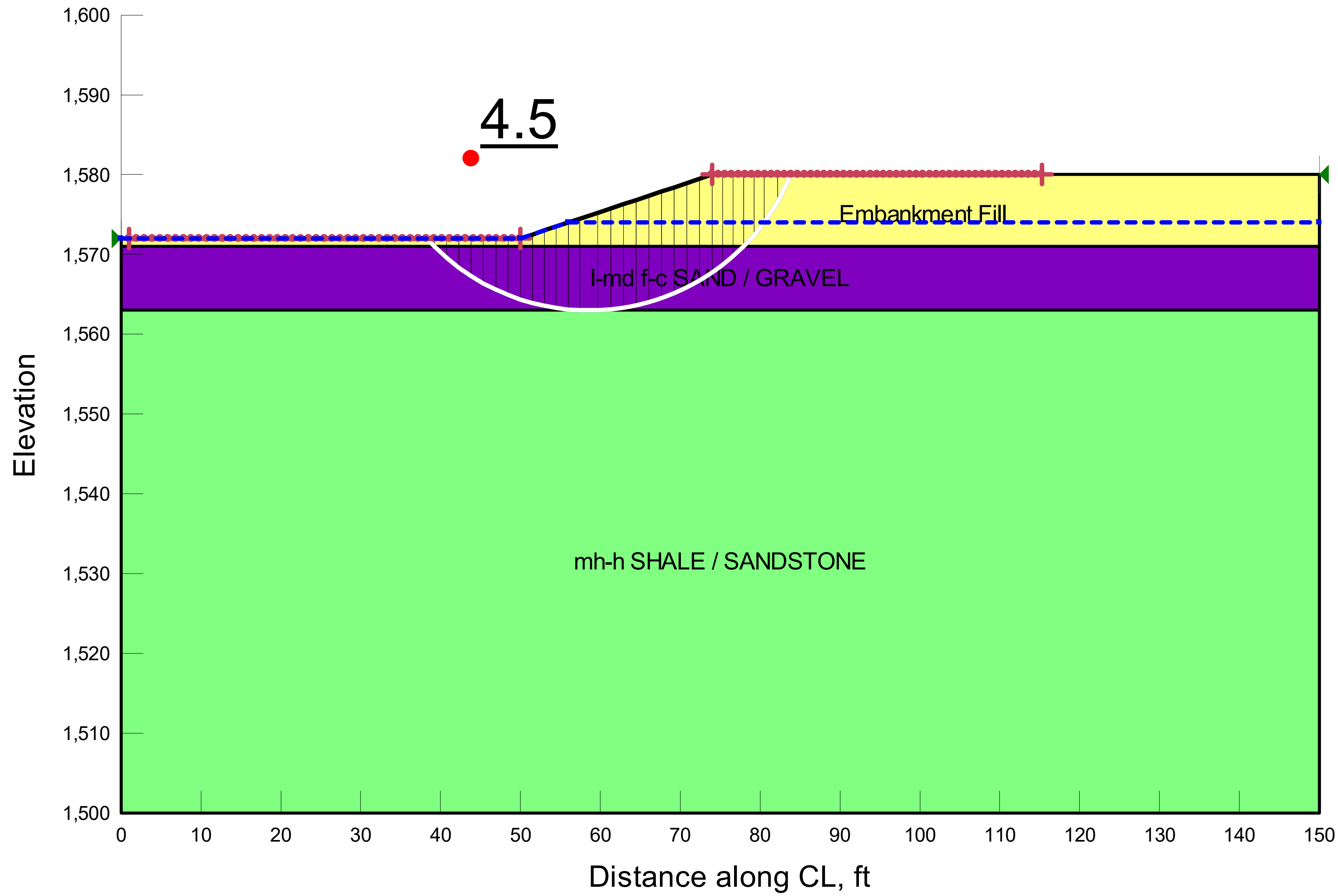

Results of Stability Analyses – Rapid Drawdown Condition, El 1574 to Existing Grade
North Side Slope (Bent 4)
3H:1V Slope, H= 8 ft±
20-138 – ArDOT Job No. 090579 – Bridge #02946

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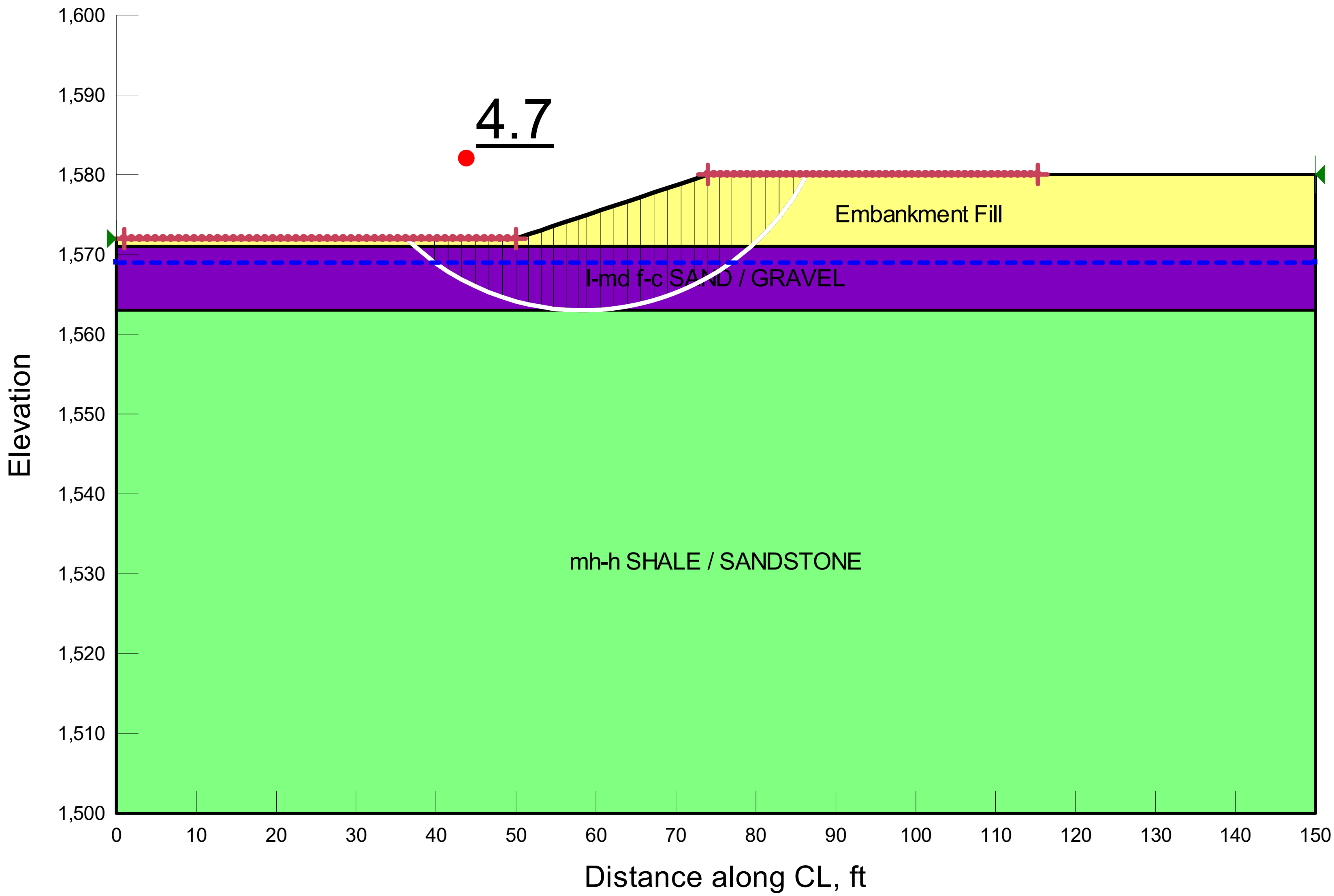

Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.031$)
North Side Slope (Bent 4)
3H:1V Slope, H= 8 ft±
20-138 – ARDOT Job No. 090579 – Bridge #02946

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