ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 040781

FEDERAL AID PROJECT NO. BFP-0065(61)

PRAIRIE CREEK STR. & APPRS. (HWY. 71) (S)

STATE HIGHWAY 71 **SECTION** 13

IN SEBASTIAN **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.

ARKANSAS DEPARTMENT OF TRANSPORTATION

ARDOT.gov | IDriveArkansas.com | Scott E. Bennett, P.E., Director

MATERIALS DIVISION

11301 West Baseline Road | P.O. Box 2261 | Little Rock, AR 72203-2261 | Phone: 501.569.2185 | Fax: 501.569.2368

October 15, 2019

TO: Mr. Trinity Smith, Engineer of Roadway Design

SUBJECT: Job No. 040781
Prairie Creek Str. & Apprs. (S)
Route 71 Section 13
Sebastian County

Based on soil information from projects in the surrounding area, an estimated R-Value of seven 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	6.1	93.9
Binder Course	4.2	95.8
Base Course	3.7	96.3

Michael C. Benson
Materials Engineer

MCB:pt:bjj
Attachment
cc: State Constr. Eng. – Master File Copy
District 4 Engineer
System Information and Research Div.
G. C. File

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

April 26, 2021
Job No. 20-064

Materials Division
Arkansas Department of Transportation
11301 West Baseline Road
Little Rock, Arkansas 72209

Attn: Mr. Paul Tinsley, P.E.

GEOTECHNICAL INVESTIGATION
ARDOT JOB No. 040781
HWY. 71 over PRAIRIE CREEK (#02240)
SEBASTIAN COUNTY, ARKANSAS

INTRODUCTION

Submitted herein are the final results of the geotechnical investigation performed for Bridge 02240, ARDOT Job 040781 Prairie Creek Strs. & Apprs. (Hwy. 71) (S) in Huntington, Sebastian County, Arkansas. This geotechnical investigation was authorized by ARDOT On-Call Geotech Task Order No. G002 on June 1, 2020. An interim report with recommendations for structure foundations was provided on September 26, 2020. These recommendations are confirmed herein.

We understand that the Hwy. 71 over Prairie Creek replacement bridge will be an integral prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 148 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 is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions at the replacement bridge location and to develop recommendations to guide design and construction of foundations. 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 at the planned bridge location to evaluate subsurface conditions and to obtain samples of the foundation soil and rock for laboratory testing.

- ♦ Performing laboratory tests to evaluate pertinent engineering properties of the foundation strata.
- ♦ Analyzing field and laboratory data to develop recommendations for seismic site class, seismic performance zone/seismic design category, foundation design, slope stability, and construction considerations.

The relationship of these factors to design and construction of the replacement bridge has been considered in developing the recommendations and considerations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface conditions at the Hwy. 71 over Prairie Creek replacement bridge location were investigated by drilling three (3) sample and core borings to depths of 29 to 41 feet. The project location is shown on Plate 1. The approximate locations of the borings are shown on Plate 2. The subsurface exploration program is summarized in the table below.

Summary of Subsurface Exploration Program

Boring No.	Approx. Hwy. 8 Sta	Approx Offset, ft	Surface El, ft	Boring Completion Depth, ft
1	114+30	15 Lt	580.6	29
2	115+12	20 Lt	566.3	41
3	116+22	10 Lt	570.7	29

Boring logs, presenting descriptions of the soil and rock strata encountered in the borings and the results of the field and laboratory tests, are included as Plates 3 through 5. The centerline station and offset of the boring locations, as inferred from the preliminary layout provided by the Department (ARDOT), are also shown on the logs. A level survey was performed to determine the surface elevation at the boring locations. The results of the survey are shown on the boring logs as the surface elevation reported to the nearest tenth (0.1) of a foot. Keys to the terms and symbols used on the logs are presented as Plates 6 and 7. To aid in visualizing subsurface conditions at the replacement bridge location, a generalized subsurface profile is presented in Appendix B. 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. Photographs of rock cores recovered from the core borings are provided in Appendix C.

The borings were drilled with a buggy-mounted CME-550 rotary-drilling rig using a combination of hollow-stem auger and rotary-wash drilling methods. Samples of the overburden soils were obtained at approximately 2-ft intervals using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb automatic hammer dropped 30 inches, in accordance with Standard Penetration Test (SPT) procedures. The number of blows required to drive the standard split-barrel sampler each 6 in. of an 18-in. total drive, or a portion thereof, is shown on the boring logs in the "Blows Per Ft" column. N_{60} values, i.e., SPT N values corrected to 60 percent of the theoretical free-fall hammer energy, are shown as blows per ft (BPF) in the column to the right of that column. The hammer energy correction factor is shown on each log, in this case a value of 1.48.

Samples of the shale bedrock were obtained using an NQ2-size double-tube core barrel. 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 each 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 these values are presented in the right hand column of the log forms, opposite the corresponding core run.

The borings were advanced using dry-auger procedures to the extent possible to facilitate evaluation of shallow groundwater conditions. Observations regarding groundwater 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

Laboratory tests were performed to evaluate pertinent physical and engineering characteristics of the foundation and embankment soil and rock. The laboratory testing included determination of natural water contents and classification tests. The results of water content determinations 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.

Atterberg limits are plotted on the logs as pluses inter-connected with a dashed line using the water content scale. The percent of soil passing the No. 200 Sieve is noted in the "-No. 200%" column on the log forms. Classification test results, as well as soil classification by the Unified Soil Classification System and AASHTO Classification System, are summarized in Appendix D.

The compressive strength of the shale bedrock was evaluated by performing ten (10) compression tests on representative rock cores. The measured rock compressive strength (q_u)

values, in lbs per sq in., along with total unit weight, in lbs per cu ft, are tabulated on the log forms at the appropriate depth. Rock compression test results are also summarized in Appendix D.

GENERAL SITE and SUBSURFACE CONDITIONS

Site Conditions

The replacement bridge location is in the south-central portion of Sebastian County where Hwy. 71 crosses Prairie Creek. This site is about two (2) miles north of the town of Huntington. At the bridge location, the creek channel is narrow and relatively shallow. Gravel bars are apparent upstream and downstream of the bridge location. The terrain around the bridge is predominantly flat. Thick woodlands border the creek to the west and east.

The existing bridge is a two-lane structure that is oriented north-south. The bridge piers are concrete columns apparently supported on footings. The bridge deck is in fair condition. The alignment of the new bridge (upstream and east of the existing bridge) is presently wooded. The existing roadway is on embankment. The terrain is undulating flat and drainage is considered fair.

Site Geology

Geologically, the project locale is in the mapped exposure of the Pennsylvanian Period Upper Atoka Formation. Characteristically, the Atoka in this area is comprised of flat to moderately dipping interbedded shale and sandstone units. The rock units 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 total thickness of the Atoka is reported to range from about 6500 ft to more than 25,000 ft (in the Boston Mountains). The Atoka formation is conformable with the Johns Valley Shale in the Ouachita 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 replacement Bridge 02240 site with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[1]. The liquefaction potential is considered low for the predominantly cohesive and coarse granular overburden soils and underlying rock units encountered within the exploration depths of the borings.

Given the project location and AASHTO code-based values, the 1.0-sec period spectral acceleration coefficient for Site Class B (S_1) is 0.053 and the 1.0-sec period spectral acceleration

---

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

coefficient (S_{D1}) value for Site Class B is 0.053. Utilizing these parameters, Table 3.10.6-1[2] indicates that a Seismic Performance Zone 1 is fitting for the Bridge 02240 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 0.056 for a Seismic Site Class B for the bridge location.

Subsurface Conditions

Based on the results of the borings drilled for the new bridge, the subsurface stratigraphy in the bridge alignment may be generalized into four (4) primary strata as follows.

Stratum I: The existing embankment fill found at the bridge ends consists of soft to firm brown, reddish brown, and gray silty clay with crushed stone and shale fragments. These soils typically classify as A-2-6 and A-4 by the AASHTO classification system (AASHTO M 145), which correlates with fair to poor subgrade support for pavement structures. The low- to medium-plasticity embankment fill exhibits variable poor to good compaction with moderate compressibility. The depth, content, and compaction of the on-site fill may vary with location.

Stratum II: The natural overburden soil units at the replacement bridge location are predominantly stiff to very stiff brown, reddish tan, and gray silty clay or very loose dark gray fine to coarse shale gravel. The overburden soils extend to variable depths of 3 to 11 ft (approximately El 560 to El 570). The clayey overburden soils have low plasticity, moderate to high shear strength, and moderate to low compressibility. The gravel has low relative density and high compressibility.

Stratum III: The overburden soils are underlain by moderately hard tan and gray weathered shale. The weathered shale is flat bedded. Though rock quality is poor, the weathered shale is relatively strong and generally competent. SPT N-values in the moderately weathered shale typically exceed 50 blows per foot.

Stratum IV: The weathered shale grades to fresh moderately hard dark gray, slightly arenaceous shale with very close sandstone partings at depths of 3 to 14 ft below existing grades. The basal dark gray shale is competent and strong. Laboratory tests indicate compressive strengths ranging from 1020 to 2160 lbs per sq in. in the basal shale.

Groundwater Conditions

Groundwater was locally encountered at 0.5-ft depth at the replacement bridge location (see Boring 2) in June 2020. It is opined that this represents localized perched water in the granular soils in the channel bottom. Groundwater was not encountered at deeper elevations prior to the introduction of drilling fluids at 9- to 14-ft depth. Seasonal seeps and springs could be locally

---

2 AASHTO LRFD Bridge Design Specification, AASHTO; 2012

present as infiltrated water migrates from areas of higher terrain through the upper fractured zones of the embankment fill and weathered shale/shale. Perched water could also develop 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 Prairie Creek and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design for Bridges

Foundations for the 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 this study, the anticipated moderate bridge foundation loads, and our understanding of the project, we recommend that foundation loads be supported on steel piles at the bridge ends (Bents 1 and 4) and on footings at the interior bents (Bents 2 and 3). Recommendations for foundations are discussed in the following report sections.

Bridge Ends (Bent 1 and Bent 4): Steel Pile Foundations

We recommend that the foundation loads at the bridge ends be supported on steel piles. Steel HP12x53 or HP14x73 piles, or heavier sections, are recommended. Other pile sizes or types may be evaluated if desired. Piles should extend through all embankment fill and overburden soils to bear in the moderately hard dark gray, brown, and tan weathered shale or moderately hard dark gray shale. Piles should be driven to practical refusal. All steel piles should be fitted with rock points.

Bearing capacities of piles driven to refusal must be determined using the AASHTO Load and Resistance Factor Design (LRFD) structural design procedure. We recommend that nominal resistance (P_n) of steel piles be determined based on the yield strength of steel H piles (f_y) and the net end area (A_{net}) of the section. Given that the piles will be driven to refusal in hard rock with the potential for driving damage, we recommend a maximum allowable stress (σ_{all}) of 0.25 f_y. An effective resistance factor (φ_b) of 0.50 is recommended for end bearing piles. This effective resistance factor for steel piles has been based on the assumption of difficult driving. For the extreme limit state, resistance factors of 1.0 for compression and 0.8 for uplift are recommended.

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.

A minimum pile embedment of 10 ft below natural grade is recommended. In light of the results of the borings, preboring is not expected to be required for pile installation. If needed, prebores would be expected to extend into moderately hard shale and rock drilling methods could be required. We recommend that any prebore diameter be sufficiently large to prevent pile damage during construction and to allow appropriate flexure for the integral bridge. A minimum prebore diameter of 20 to 24 in. is expected to be required.

Estimated pile tip elevations are summarized in the following table.

Estimated Tip Elevations of Steel Piles Driven to Refusal

Bent No.	**Estimated Pile Tip El, ft**	**Comments**
1 (South Abutment)	566	Estimated 14 ft below plan pile cap bottom (El 580)
4 (North Abutment)	562	Estimated 15 ft below plan pile cap bottom (El 577)

It should be noted that the pile tip elevations shown in the table above are estimates only based on the results of the borings and the surface elevations at particular boring locations. As-built pile tip elevations may vary. Pile safe bearing capacity and final depth must be field verified. Battered piles can be utilized to resist lateral loads.

Piles should be installed in compliance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. We have recommended that all piles be fitted with rock points.

Interior Bents - Footings

The results of the borings indicate that competent moderately hard shale is present at depths of 1.5 to 2 ft below the channel bottom (i.e., rock at approximately El 563) at the interior bent locations. For the Hwy. 71 over Prairie Creek interior bents (Bents 2 and 3), foundation loads may be supported on footings bearing in the competent moderately hard dark gray shale. It is recommended that footings be founded with at least 2 ft of embedment into the competent moderately hard shale and at least 2 ft below the potential scour depth, whichever depth is greater. The preliminary footing bottom at El 560, as shown on the preliminary bridge layout, is considered suitable with respect to bearing.

Footings founded with a minimum embedment of 2 ft into the competent moderately hard shale may be sized based on a maximum nominal bearing pressure (q_{stat}) of 32 kips per sq foot. A compression resistance factor (φ_b) of 0.45 is recommended for footings fully bearing in the competent shale. Accordingly, a factored compression unit bearing resistance (q_R) of 14.4 kips per sq ft is considered appropriate. Post-construction settlement of footings bearing in the competent shale as recommended is expected to be negligible.

Uplift loads from the bridge structure 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 dark 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 replacement bridge will include new embankments and modifying the existing end slope configuration. The proposed end slopes 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 14 to 19 feet.

To evaluate suitability of the plan embankment slope configurations, slope stability analyses have been 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[3] 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 0.03. For evaluating the rapid drawdown condition, a water surface elevation drop from El 574 to El 567 has been assumed. The sections used for the analyses are shown in the graphical results provided in Appendix E.

For the purposes of the stability analyses, unclassified embankment as per ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 210.06, was assumed for embankment fill. Accordingly, an undrained shear strength value of 1500 lbs per sq ft has been assumed for the embankment fill. Depending on the specific borrow utilized for embankments, verification of embankment fill properties and/or stability could be warranted.

The results of the stability analyses performed for this study indicate that stability of the plan embankment side and end slope configurations is acceptable with respect to all loading conditions evaluated. It is our conclusion that the plan embankment slope configurations are suitable with respect to slope stability.

Site Grading and Subgrade Preparation

Site grading and subgrade 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

---

3 Slope/W 2020; GEO-SLOPE International; 2020.

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 toe. 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 soils in roadway areas, 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 should 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 (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 are 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 or Department during subgrade preparation.

Shallow perched groundwater may be encountered in the near-surface soils, particularly at lower elevations and during times of high creek 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) or stone backfill (Standard Specifications for Highway Construction, 2014 Edition, Section 207) placed 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. Based on local experience, we recommend a hammer system capable of delivering at least 22,000 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 or Department prior to hammer acceptance and start of pile driving.

As a minimum, safe bearing capacity of production piles should be determined by ARDOT 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 should 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. Some overbreak of excavations advanced into the moderately hard to hard shale and localized sandstone units should be anticipated. Any preboring for piles is 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 submittal.

Plate 1	Site Vicinity
Plate 2	Plan of Borings
Plates 3 through 5	Preliminary Boring Logs
Plates 6 and 7	Keys to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Rock Core Photographs
Appendix D	Classification Test Results
Appendix E	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.

Mark E. Wyatt, P.E.
President

BJD/MEW:jw

Copies Submitted:	Materials Division - Arkansas Department of Transportation		
	Attn:	Mr. Paul Tinsley, P.E.	(1-email)
	Attn:	Mr. Paul Tierney, P.E.	(1-email)
	Attn:	Mr. Yongsheng Zhao, PhD, P.E.	(1-email)
	Jacobs Engineering Group Inc.		
	Attn:	Mr. Mark A. Asher, P.E.	(1-email)
	Attn:	Mr. Justin Carney, P.E., S.E.	(1-email)

ARKANSAS DEPARTMENT OF TRANSPORTATION
CONSTRUCTION PLANS FOR STATE HIGHWAY

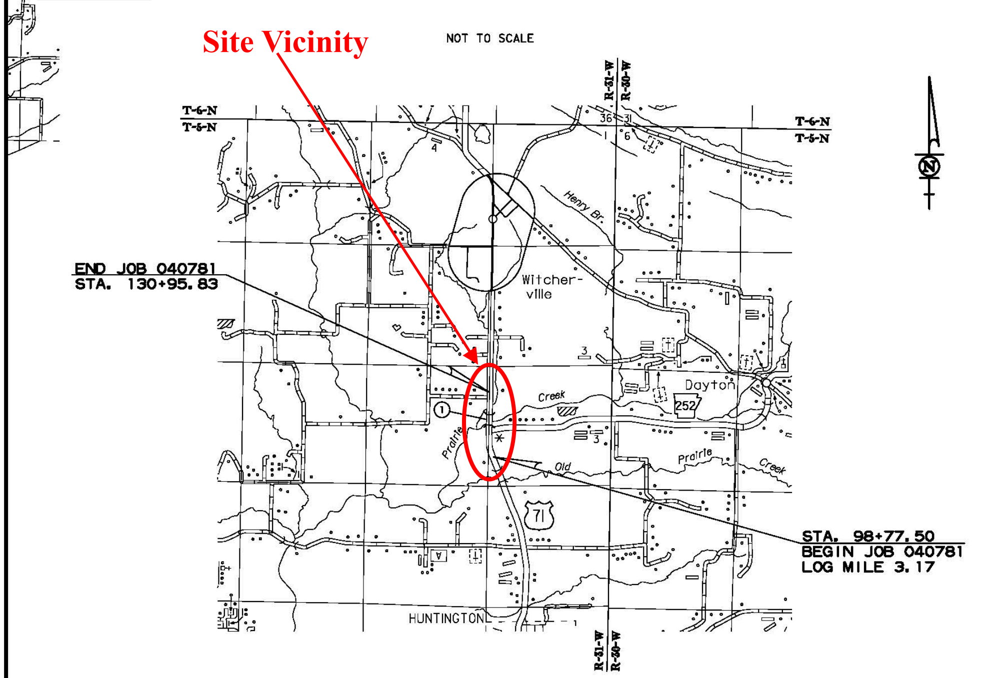
PRAIRIE CREEK STR. & APPRS. (HWY. 71) (S)

SEBASTIAN COUNTY

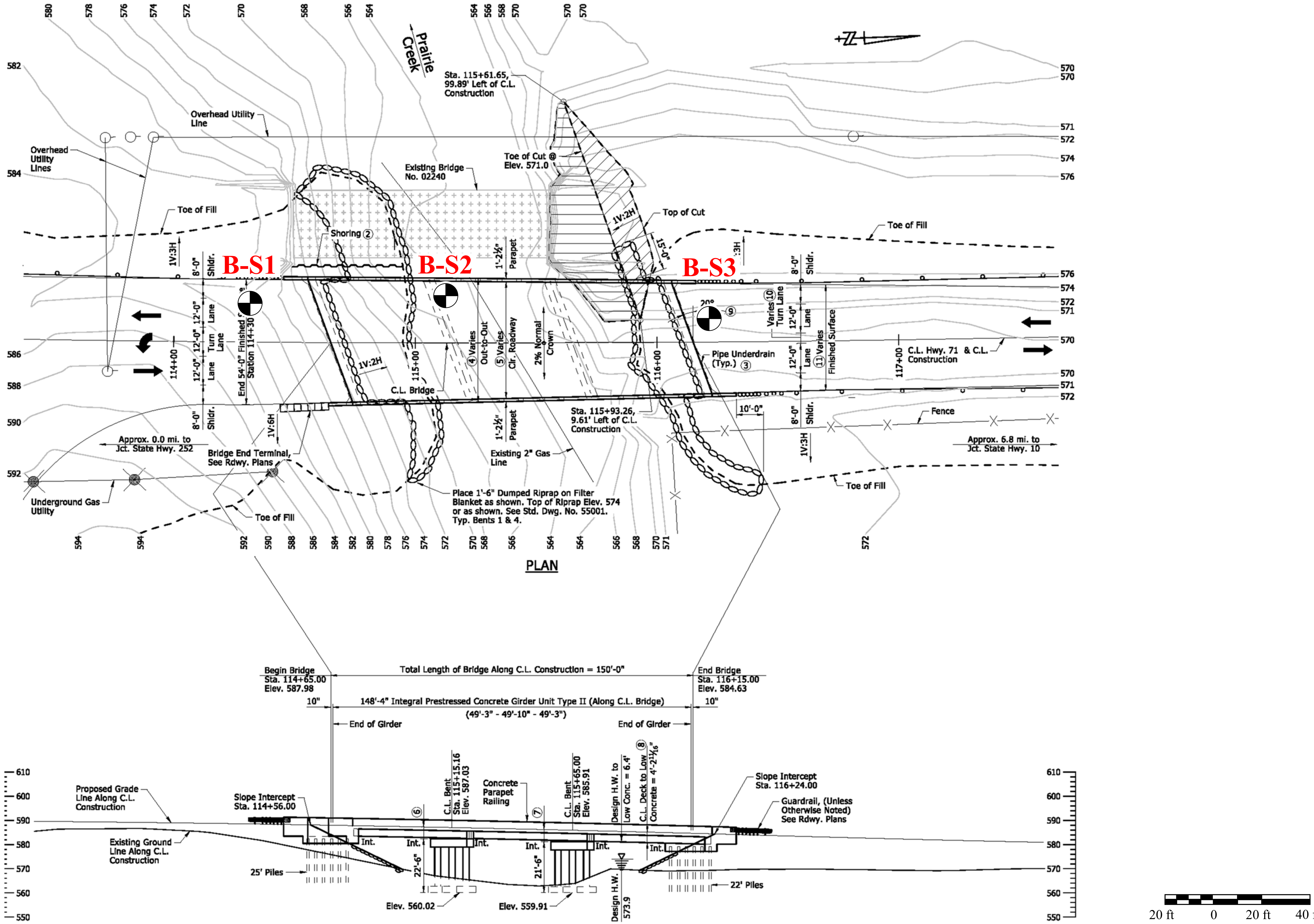
ROUTE 71 SECTION 13

FEDERAL AID PROJ. HHPP-0065(61)

JOB 040781



Grubbs, Hoskyn, Barton & Wyatt, INC. CONSULTING ENGINEERS	**SITE VICINITY MAP** **040781 Hwy. 71 over Prairie Creek** **Sebastian Co., Arkansas**	**Job No. 20-064**	**Plate 1**



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

PLAN OF BORINGS
040781 Hwy. 71 over Prairie Creek
Sebastian County, Arkansas

Scale: As shown	Job No. 20-064	PLATE 2

20-064

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

LOG OF BORING NO. S1

040781 Hwy 71 over Prairie Creek
Sebastian Co., Arkansas

EQUIPMENT: CME 550 w/ CME Auto Hammer
TYPE: HSA to 14 ft /Wash

Hammer Correction Factor: 1.48
LOCATION: Approx Sta 114+30, 15 Lt

SURF. EL: 580.6

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	N_{60}, BPF	Water Content / Plastic Limit / Liquid Limit / Cohesion, TON/SQ FT	- No. 200 %	% Recovery	% RQD
0–6	Stiff brown, reddish brown and gray silty clay w/crushed stone (fill)	6 / 9-6	22				
		3 / 6-6	18		29		
5		4 / 5-8	19				
6–8	Stiff reddish brown and gray silty clay, sandy w/ferrous stains	3 / 5-8	27		72		
8–11	Very stiff reddish tan and gray silty clay w/highly weathered shale seams and layers (completely weathered shale)	14 / 21-18	58				
11–14	Moderately hard dark gray and tan weathered shale w/ferrous stains in bedding plans	11 / 50/3"	100				
14–29	Moderately hard dark gray shale, slightly arenaceous and flat bedded w/very close sandstone partings - with near-vertical fractures at 17.5 to 20 ft			q_u= 1020 psi, TUW= 153 pcf		96	42
				q_u= 1380 psi		100	39
				q_u= 1810 psi, TUW= 158 pcf		100	100

COMPLETION DEPTH: 29.0 ft
DATE: 6-10-20

DEPTH TO WATER
IN BORING: Dry to 14 ft

DATE: 6/10/2020

RECRQDN200-2 20-064.GPJ 7-27-20

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

LOG OF BORING NO. S2

040781 Hwy 71 over Prairie Creek
Sebastian Co., Arkansas

EQUIPMENT: CME 550 w/ CME Auto Hammer
TYPE: Wash

Hammer Correction Factor: 1.48
LOCATION: Approx Sta 115+12, 20 Lt

SURF. EL: 566.3

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	N_{60}, BPF	COHESION, TON/SQ FT / WATER CONTENT	- No. 200 %	% Recovery	% RQD
0–1	Very loose dark gray fine to coarse shale fragments (fill)						
1–2	Very loose dark gray fine to coarse shale gravel	1-2	4	● (water content ≈ 6)	1		
2–8	Moderately hard dark gray slightly weathered shale w/ferrous stains in bedding planes			q_u= 1550 psi, TUW= 155 pcf		90	78
8–43	Moderately hard dark gray shale, slightly arenaceous and flat bedded w/very close sandstone partings			q_u= 1370 psi, TUW= 160 pcf		100	100
				q_u= 1440 psi, TUW= 154 pcf		100	100
				q_u= 1300 psi, TUW= 158 pcf		100	100
				q_u= 1790 psi, TUW= 158 pcf			
				q_u= 1260 psi		100	100
						100	100
						100	100
						100	100

COMPLETION DEPTH: 43.0 ft
DATE: 6-19-20

DEPTH TO WATER
IN BORING: 0.5 ft

DATE: 6/19/2020

RECRQDN200-2 20-064.GPJ 8-5-20

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

LOG OF BORING NO. S3

040781 Hwy 71 over Prairie Creek
Sebastian Co., Arkansas

EQUIPMENT: CME 550 w/ CME Auto Hammer
TYPE: HSA to 9 ft /Wash

Hammer Correction Factor: 1.48
LOCATION: Approx Sta 116+22, 10 Lt

SURF. EL: 570.7

Depth, ft	Description of Material	Blows per ft	N_{60}, BPF	Notes / Test Results	- No. 200 %	% Recovery	% RQD
0–4	Firm brown and reddish brown silty clay w/shale fragments (fill)	2 / 3-4	10		47		
	- soft below 2 ft	4 / 2-3	7				
4–6.5	Stiff brown and gray silty clay w/shale fragments (completely weathered shale)	3 / 6-11	25				
6.5–9	Moderately hard dark gray and brown weathered shale w/ferrous stains	12 / 50/6"	100				
		50/3"	100				
9–29	Moderately dark shale, slightly arenaceous and flat bedded w/very close sandstone partings and close healed high-angle fractures					96	78
						98	58
				q_u= 2160 psi, TUW= 155 pcf		98	98
						100	100

COMPLETION DEPTH: 29.0 ft
DATE: 6-9-20

DEPTH TO WATER
IN BORING: Dry to 9 ft

DATE: 6/9/2020

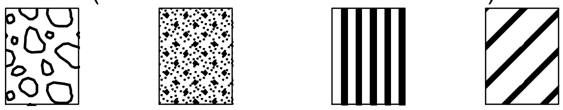
RECRQDN200-2 20-064.GPJ 7-27-20

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

(SHOWN IN SYMBOLS COLUMN)

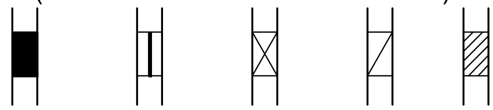


Gravel Sand Silt Clay

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.

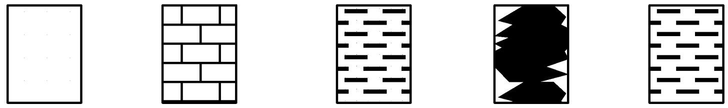
Terms used on this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No.3-357, Waterways Experiment Station, March 1953

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 1/1 6inch
Seam –	1 /1 6to 1 /2inch
Layer –	1 /2to 1 2inches
Stratum –	Greater than 1 2inches

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 –

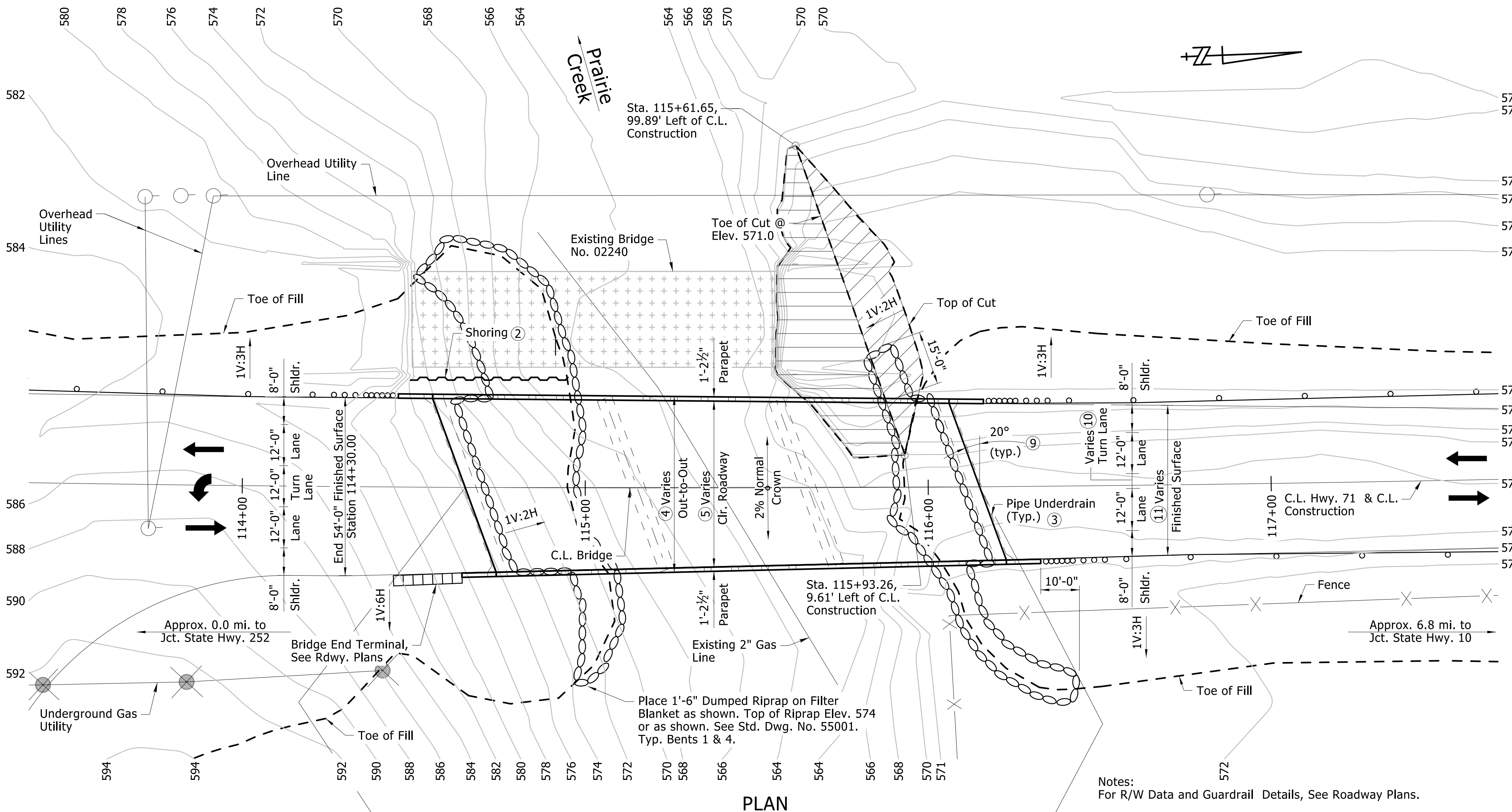
Nonslaking
Slakes slowly on exposure
Slakes readily on exposure

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

APPENDIX A



PLAN

HORIZONTAL CURVE DATA

Along C.L. Construction

P.I.= 114+91.33
Δ = 7°40' 17" Lt.
D = 0°30' 00"
T = 768.29'
L = 1534.28'
e = N.C.
R = 11459.16'

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	(1) NATURAL WATER SURFACE ELEVATION	WATER SURFACE ELEV. WITH BACKWATER
	YEARS	CFS	FEET	FEET
Design	50	5,658	573.4	573.9
Base	100	6,753	573.8	574.2
Extreme	500	9,664	574.9	576.6
Overtopping	>500	>500	>500	>500

(1) Unconstricted water surface elevation without structure or roadway approaches.

Q100 backwater elevation for existing structure = 575.2 feet.

Drainage Area = 12.7 square miles.

Historical H.W. Elevation = N/A

(2) Shoring may be required during construction. See Job Special Provision "Shoring".

(3) Install 4" Dia. Pipe Underdrain with Outlet Protectors at both bridge ends in accordance with Section 611 and Std. Dwg. PU-1. For additional details, see Dwg. No. 61958. Pipe Underdrain will not be paid for directly but shall be considered subsidiary to "UNCLASSIFIED EXCAVATION".

(4) Varies - 53'-4¹⁄₁₆" at Begin Bridge to 48'-4" at End Bridge

(5) Varies - 50'-10" at Begin Bridge to 45'-10" at End Bridge

(6) C.L. Deck at C.L. Bent to Low Side Top of Cap = 4'-4⅛"

(7) C.L. Deck at C.L. Bent to Low Side Top of Cap = 4'-3⅞"

(8) Proposed Low Bridge Chord Elevation = 580.28 feet and occurs at Sta. 116+19.30

(9) Skew measured from a line normal to C.L. Bridge, see "STAKING DIAGRAM" on Dwg. No. 61943.

(10) Turn lane varies from 12'-0" at Sta. 114+30.00 to 0'-0" at Sta. 117+90.00.

(11) Finished Surface width varies from 54'-0" at Sta. 114+30.00 to 40'-0" at Sta. 117+90.00.

Notes:
For R/W Data and Guardrail Details, See Roadway Plans.

Use Type 1 Special Approach Slab at begin bridge and Type 2 Special Approach Slab at end bridge. See Dwg. Nos. 61964 & 61965, respectively.

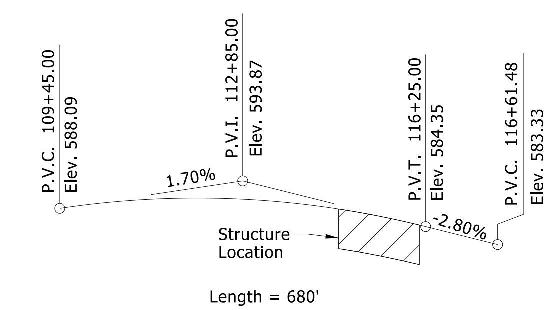
Use Type 1 & 2 Special Approach Gutters at begin bridge and Type 3 & 4 Special Approach Gutters at end bridge. See Dwg. Nos. 61962 & 61963, respectively. Eliminate or modify Type 2 Special Approach Gutter curb section at SE corner of bridge to fit bridge end terminal. No additional payment will be made for this modification.

The Contractor shall excavate the existing embankment as shown at North end of Bridge to Elev. 571 .0 Approx. 445 cubic yards of excavation.

For "GENERAL NOTES" and "ELEVATION OF SOIL BORINGS", See Dwg. No. 61942.

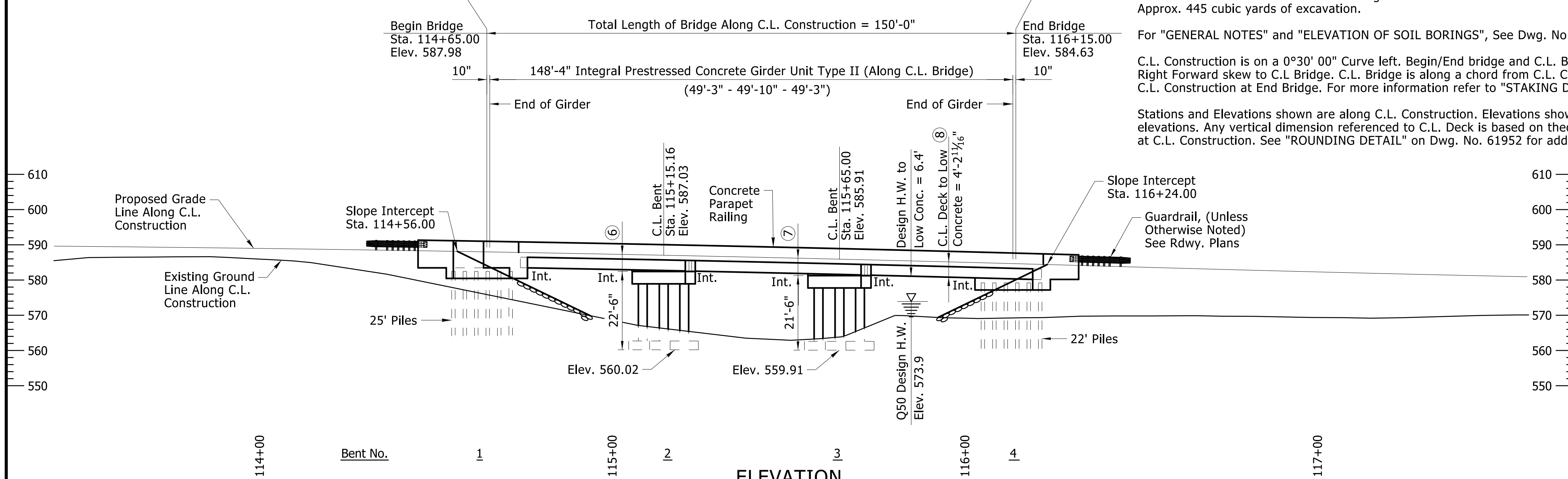
C.L. Construction is on a 0°30' 00" Curve left. Begin/End bridge and C.L. Bent Lines are on a 20°00' 00" Right Forward skew to C.L Bridge. C.L. Bridge is along a chord from C.L. Construction at Begin Bridge to C.L. Construction at End Bridge. For more information refer to "STAKING DIAGRAM" on Dwg. No. 61943.

Stations and Elevations shown are along C.L. Construction. Elevations shown are theoretical working point elevations. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Construction. See "ROUNDING DETAIL" on Dwg. No. 61952 for additional information.



VERTICAL CURVE DATA

Stations and Elevations shown are along C.L. Construction
No Scale



ELEVATION

SOIL BORINGS NOT AVAILABLE AT TIME OF PRINTING

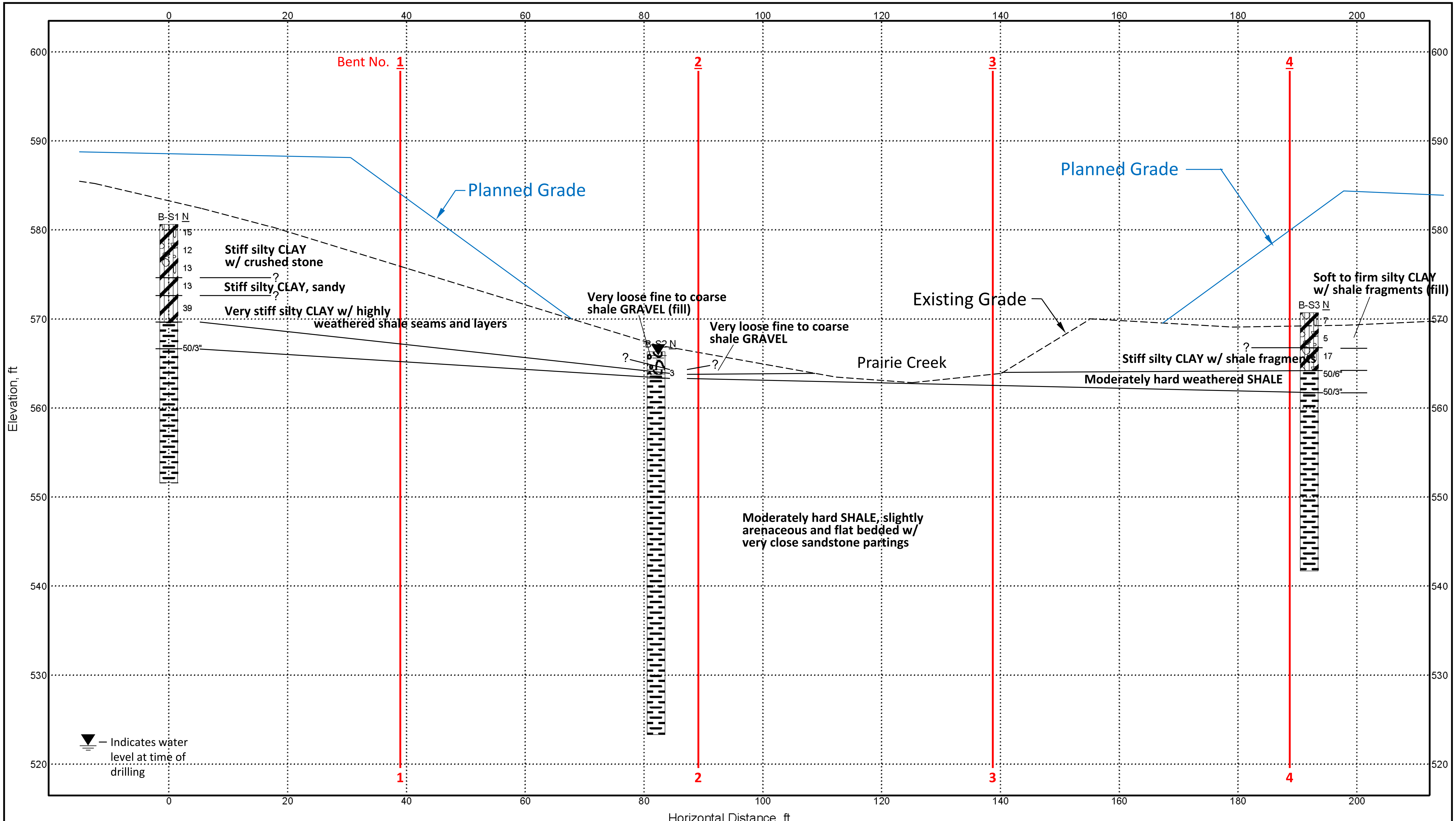
STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No.18018
JUSTIN P. CARNEY

BRIDGE ENGINEER
PRINT DATE: 7/7/2020

SHEET 1 OF 3
LAYOUT OF BRIDGE
HIGHWAY 71 OVER PRAIRIE CREEK
PRAIRIE CREEK STR. & APPRS. (HWY. 71) (S)
SEBASTIAN COUNTY
ROUTE 71 SECTION 13
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: JPC DATE: 04/2020 FILENAME: b040781_l1.dgn
CHECKED BY: MAA DATE: 04/2020
DESIGNED BY: JPC DATE: 04/2020 SCALE: 1" = 20'
BRIDGE NO. 07501 DRAWING NO. 61941

APPENDIX B



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" = 15' Horizontal
1" = 10' Vertical

Generalized Subsurface Profile
040781 Hwy 71 over Prairie Creek
Sebastian County, Arkansas

Project Number: 20-064

APPENDIX C


040781
B-51
Run 1: 14'-19'
Run 2: 19'-24'
Grubbs, Hoskyn, Barton & Wyatt, INC
CONSULTING ENGINEERS


040781
B-51
Run3: 24'-29'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS


040781
B-52
Run 1: 1'-6'
Run 2: 6'-11'
Grubbs, Hoskyn,
Barton & Wyatt, INC
CONSULTING ENGINEERS


040781
B-52
Run 3: 11'-16'
Run 4: 16'-21'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS


040781
B-52
Run 5: 21'-26'
Run 6: 26'-31'
Grubbs, Hoskyn, Barton & Wyatt, INC
CONSULTING ENGINEERS


040781
B-53
Run1: 9'-14'
Run2: 14'-19'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS


040781
B-53
Run 3: 19'-24'
Run 4: 24'-29'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

APPENDIX D

SUMMARY of CLASSIFICATION TEST RESULTS

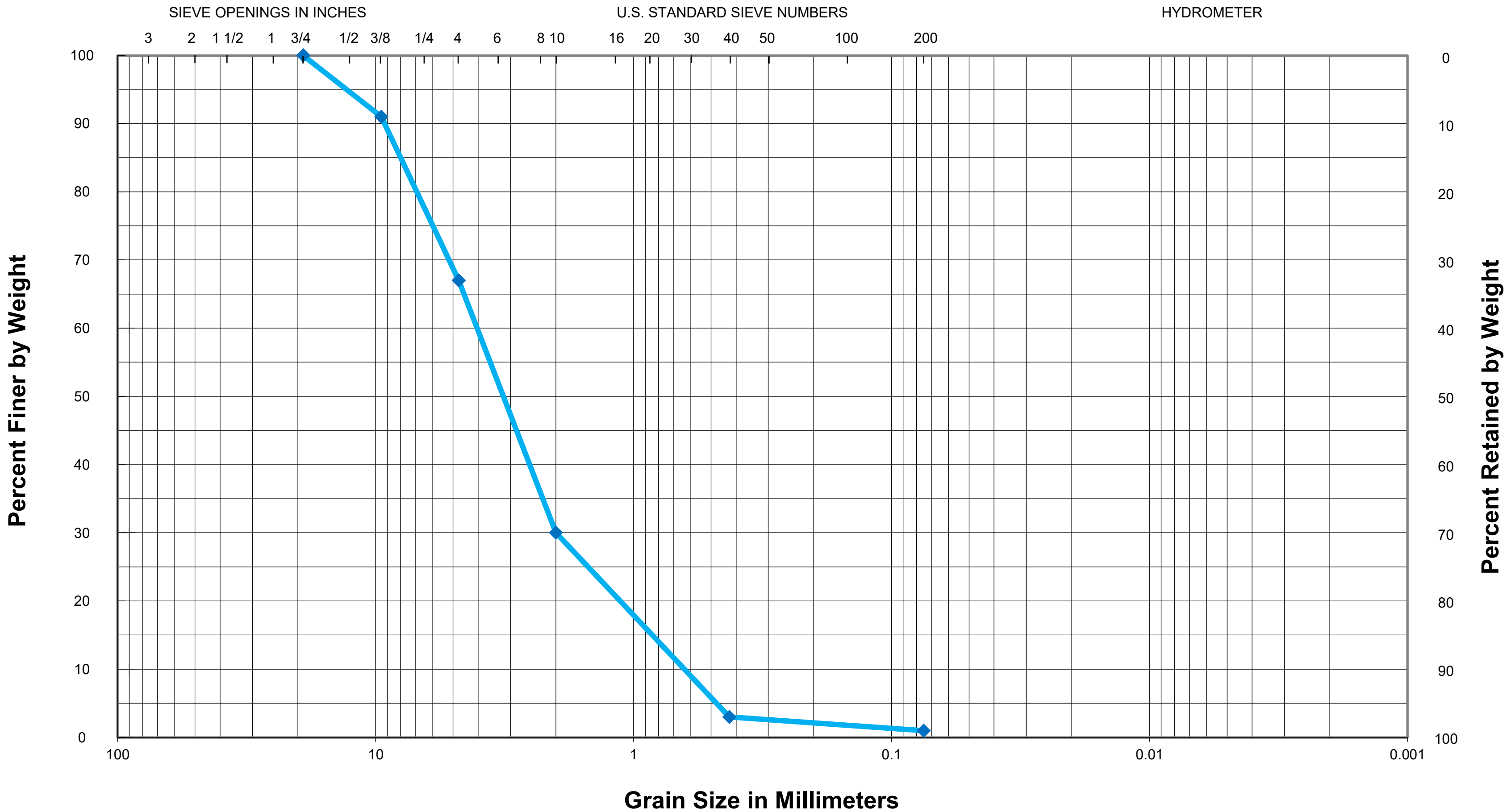
PROJECT: 040781 Hwy 71 over Prairie Creek
LOCATION: Sebastian Co., Arkansas
GHBW JOB NUMBER: 20-064

BORING No.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING						USCS CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	3/4 in.	3/8 in.	#4	#10	#40	#200		
S1	2.5-3.5	11	34	22	12	---	---	70	---	---	29	SC	A-2-6
S1	6.5-7.5	20	40	23	17	---	---	96	---	---	72	CL	A-6
S1	9-10	14	29	22	7	---	---	---	---	---	---	SHALE	
S2	0-1	6	---	---	---	100	91	67	30	3	1	SW	A-1-a
S3	0.5-3.5	15	32	22	10	---	---	84	---	---	47	SC	A-4
S3	4.5-5.5	21	35	21	14	---	---	---	---	---	---	SHALE	

20-064

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring S2, 0-1 ft;
Description: Dark gray fine to coarse crushed shale fragments

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

SUMMARY of COMPRESSION TEST RESULTS

PROJECT: 040781 Hwy 71 over Prarie Creek
LOCATION: Sebastian Co., Arkansas
GHBW JOB NUMBER: 20-064

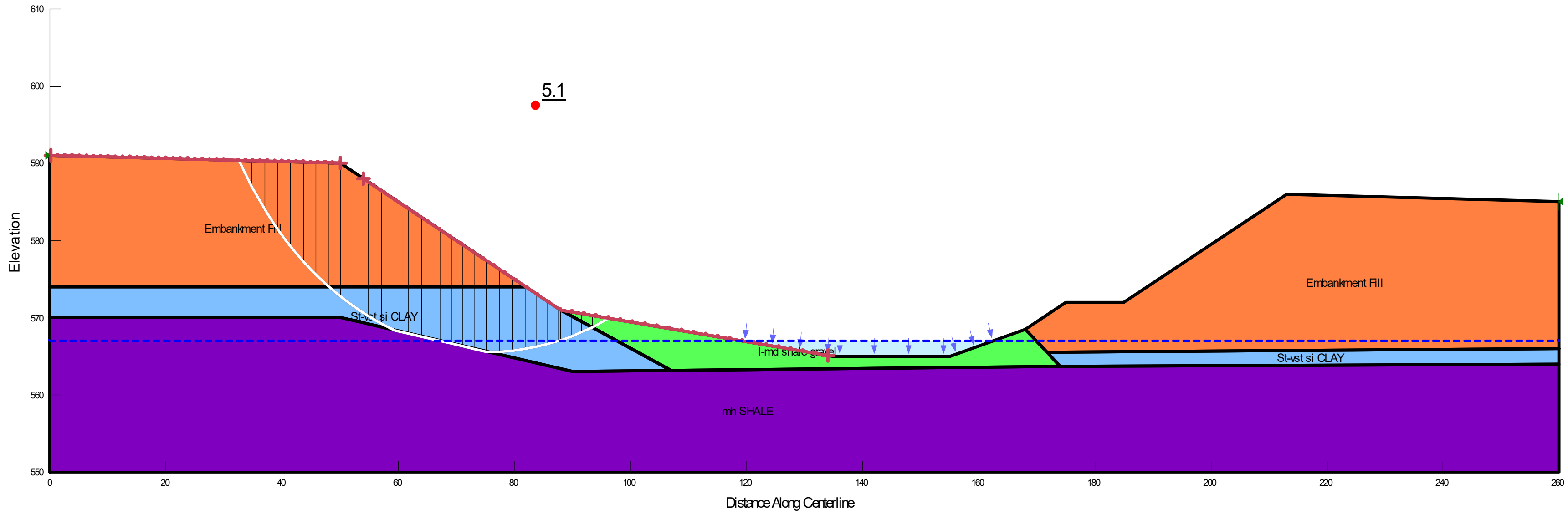
Boring No.	Depth, ft	Total Unit Weight, pcf	Compressive Strength, psi	Description
S1	15.5-16	153	1020	Dark gray SHALE
S1	22.5-23	not measured	1380	Dark gray SHALE
S1	25.5-26	158	1810	Dark gray SHALE
S2	1.5-2	155	1550	Dark gray SHALE
S2	5-5.5	160	1370	Dark gray SHALE
S2	11-11.5	154	1440	Dark gray SHALE
S2	19.5-20	158	1300	Dark gray SHALE
S2	21-21.5	158	1790	Dark gray SHALE
S2	23-23.5	not measured	1260	Dark gray SHALE
S3	19-19.5	155	2160	Dark gray SHALE

Notes:
1. All tests performed on NQ2 rock cores.
2. Tested as per ASTM D7012

APPENDIX E

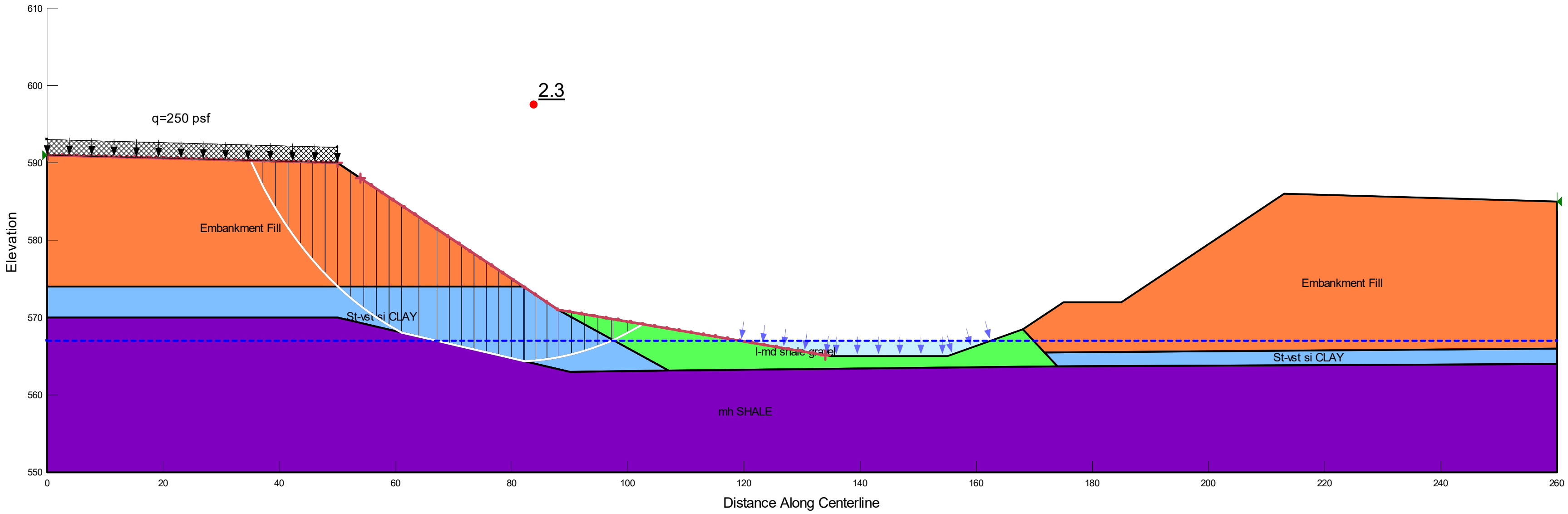
Summary of Stability Analysis Results
ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
GHBW Job No. 20-064
Sebastian Co., Arkansas

Bridge End	Design Loading Condition	Calculated Minimum Factor of Safety
Bent 1 End Slope (2H:1V)	End of Construction	5.1
	Long Term	2.3
	Rapid Drawdown from El 574 to El 567	2.3
	Seismic ($k_h = A_S/2 = 0.03$)	2.4
Bent 1 Side Slope (3H:1V)	End of Construction	4.6
	Long Term	3.9
	Rapid Drawdown from El 574 to Existing Grade	4.2
	Seismic ($k_h = A_S/2 = 0.03$)	3.8
Bent 4 End Slope (2H:1V)	End of Construction	4.9
	Long Term	2.7
	Rapid Drawdown from El 574 to El 567	2.8
	Seismic ($k_h = A_S/2 = 0.03$)	2.8
Bent 4 Side Slope (3H:1V)	End of Construction	3.1
	Long Term	2.8
	Rapid Drawdown from El 574 to Existing Grade	3.1
	Seismic ($k_h = A_S/2 = 0.03$)	2.8

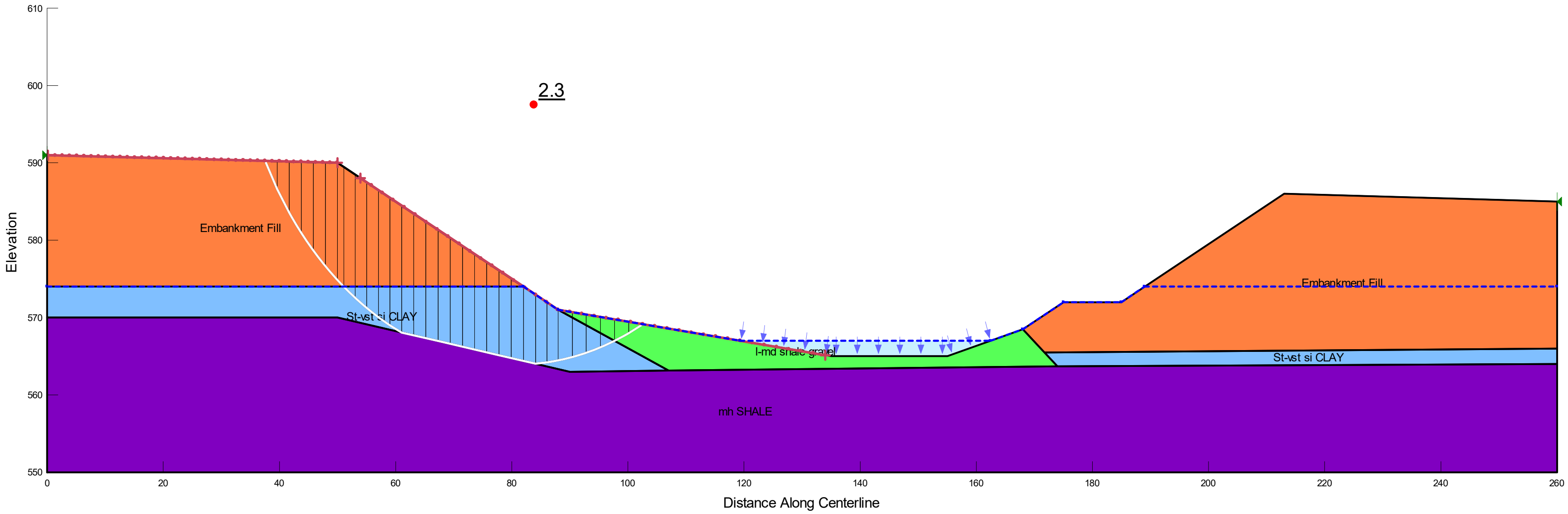


Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=19 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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

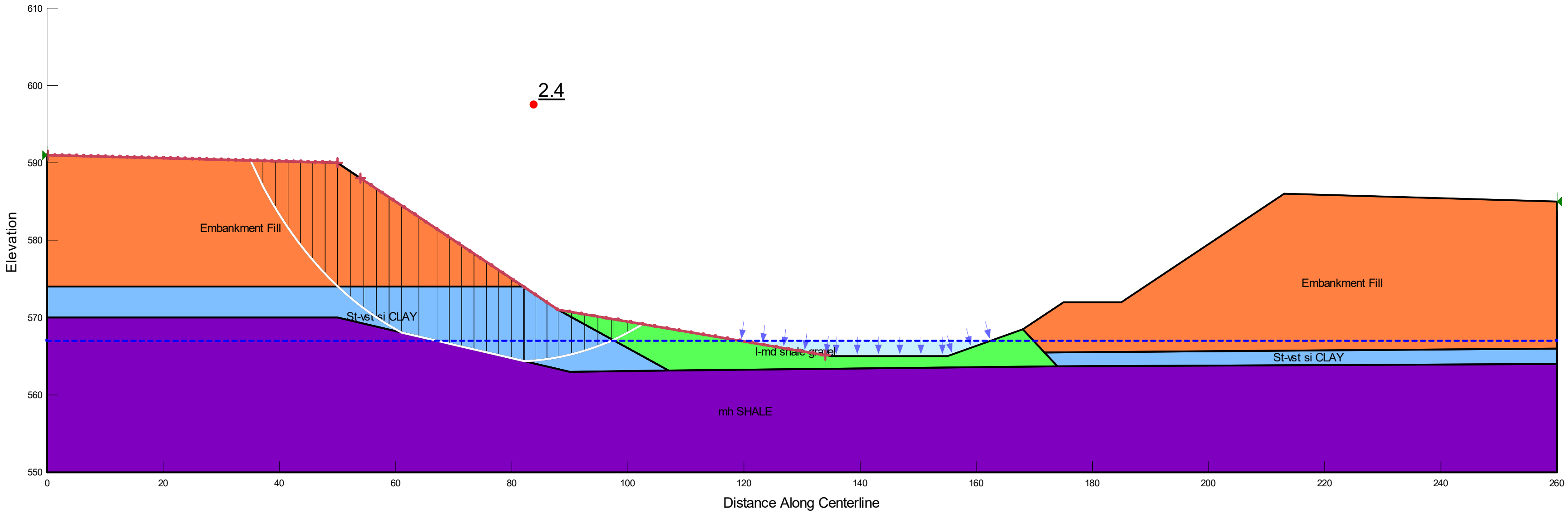


Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=19 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

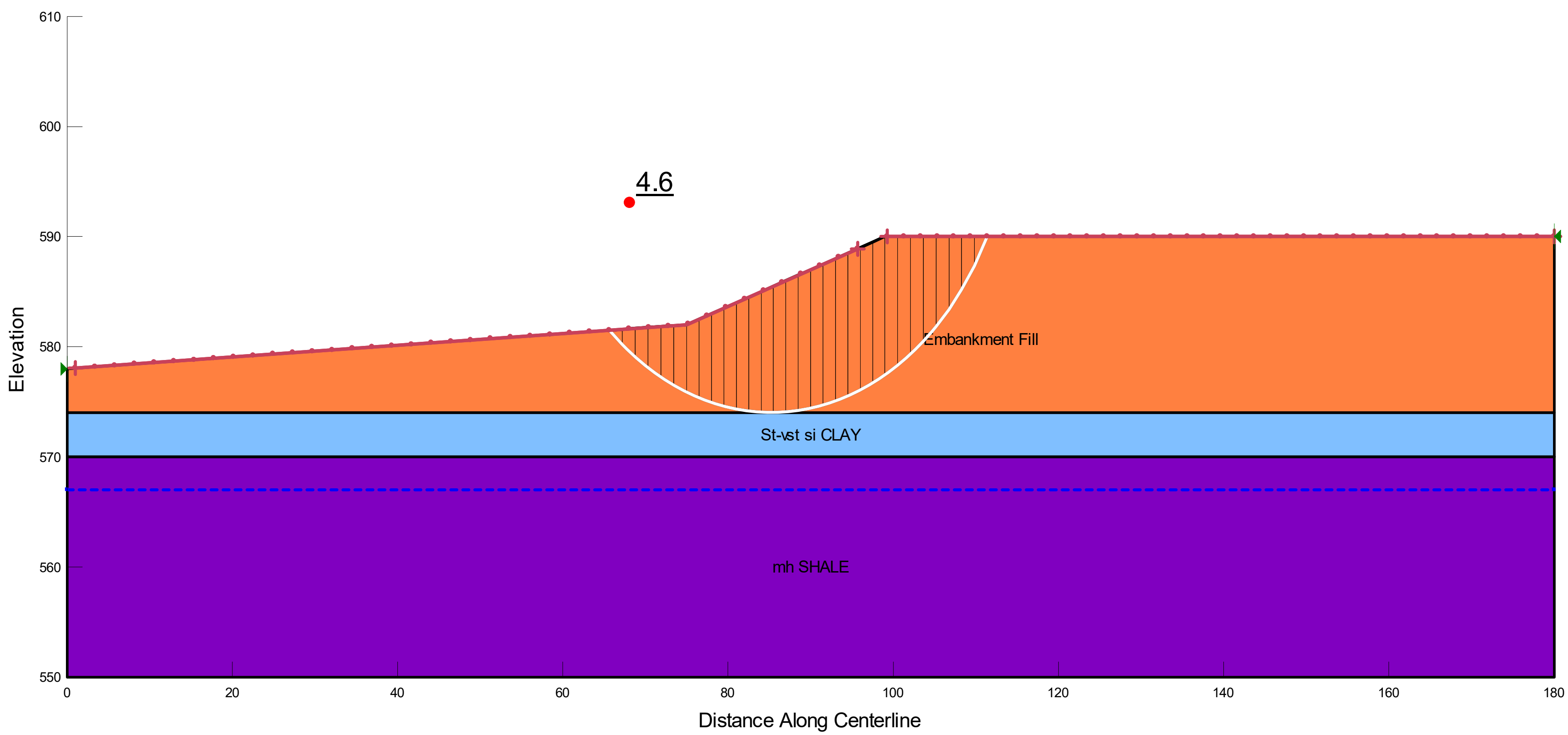


Results of Stability Analyses – Rapid Drawdown Condition, El 574 to El 567
Bent 1 End Slope
2H:1V Slope, H=19 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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

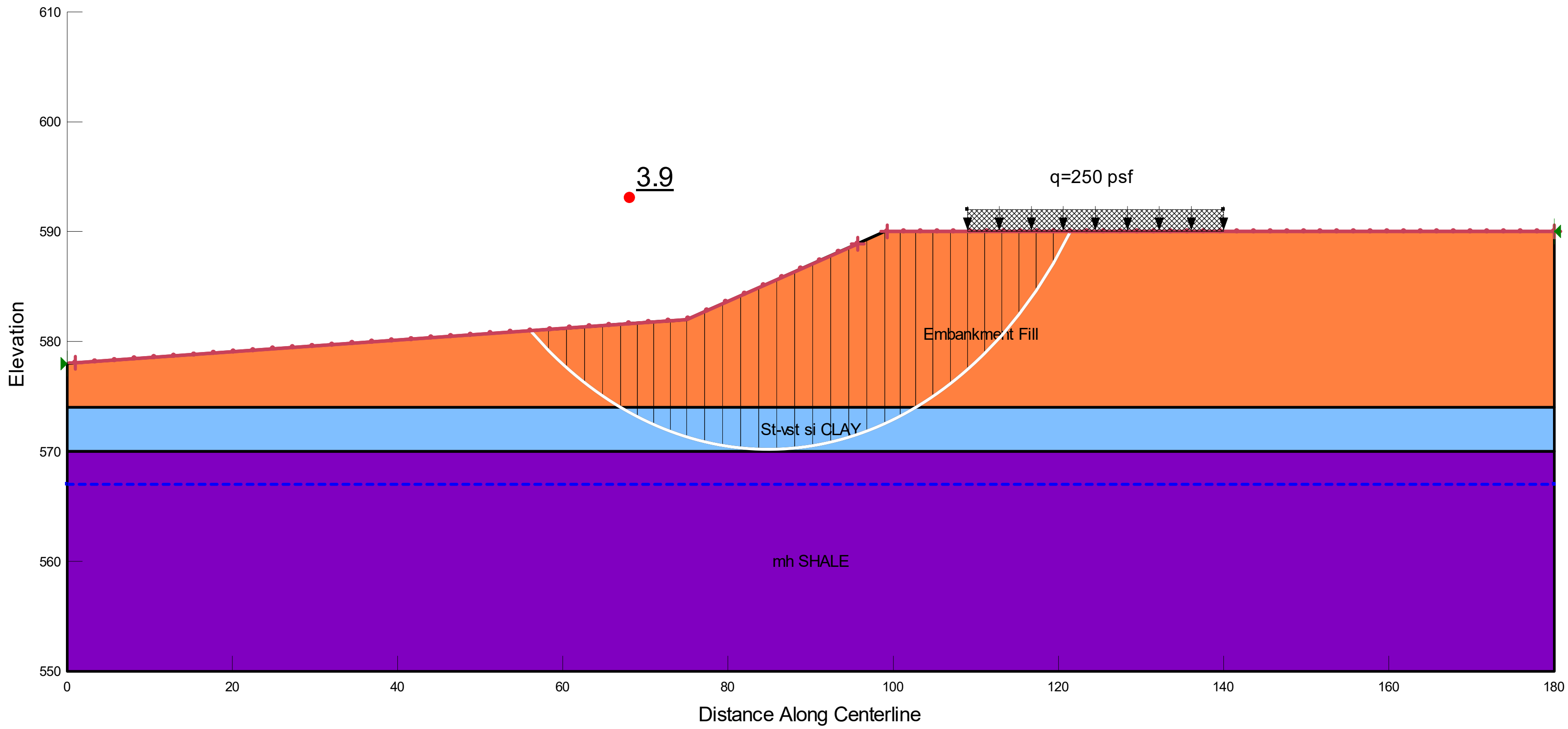


Results of Stability Analyses – Seismic Condition ($k_h = A_S/2 = 0.03$)
Bent 1 End Slope
2H:1V Slope, H=19 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas



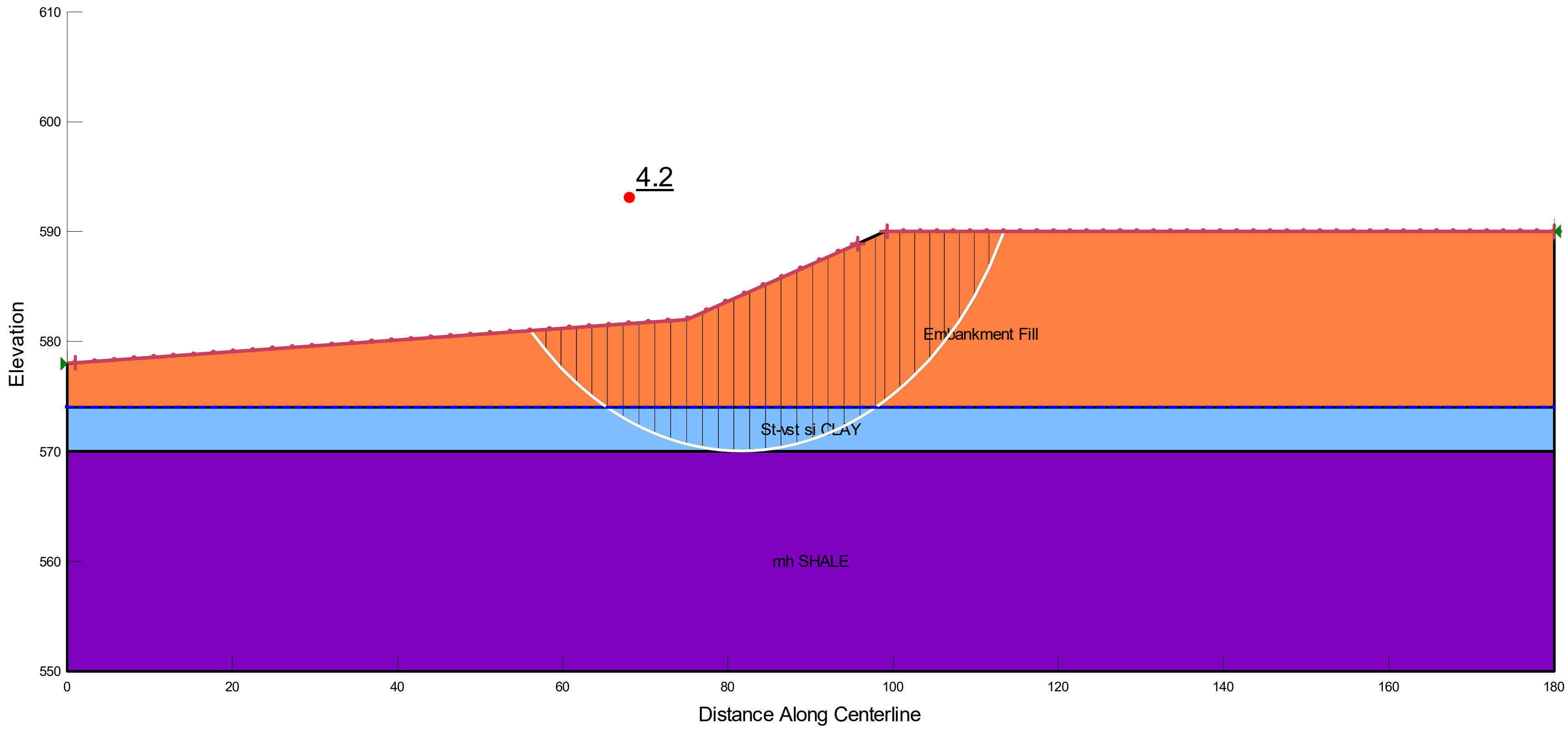
Results of Stability Analyses – End of Construction
Bent 1 Side Slope
3H:1V Slope, H=8 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



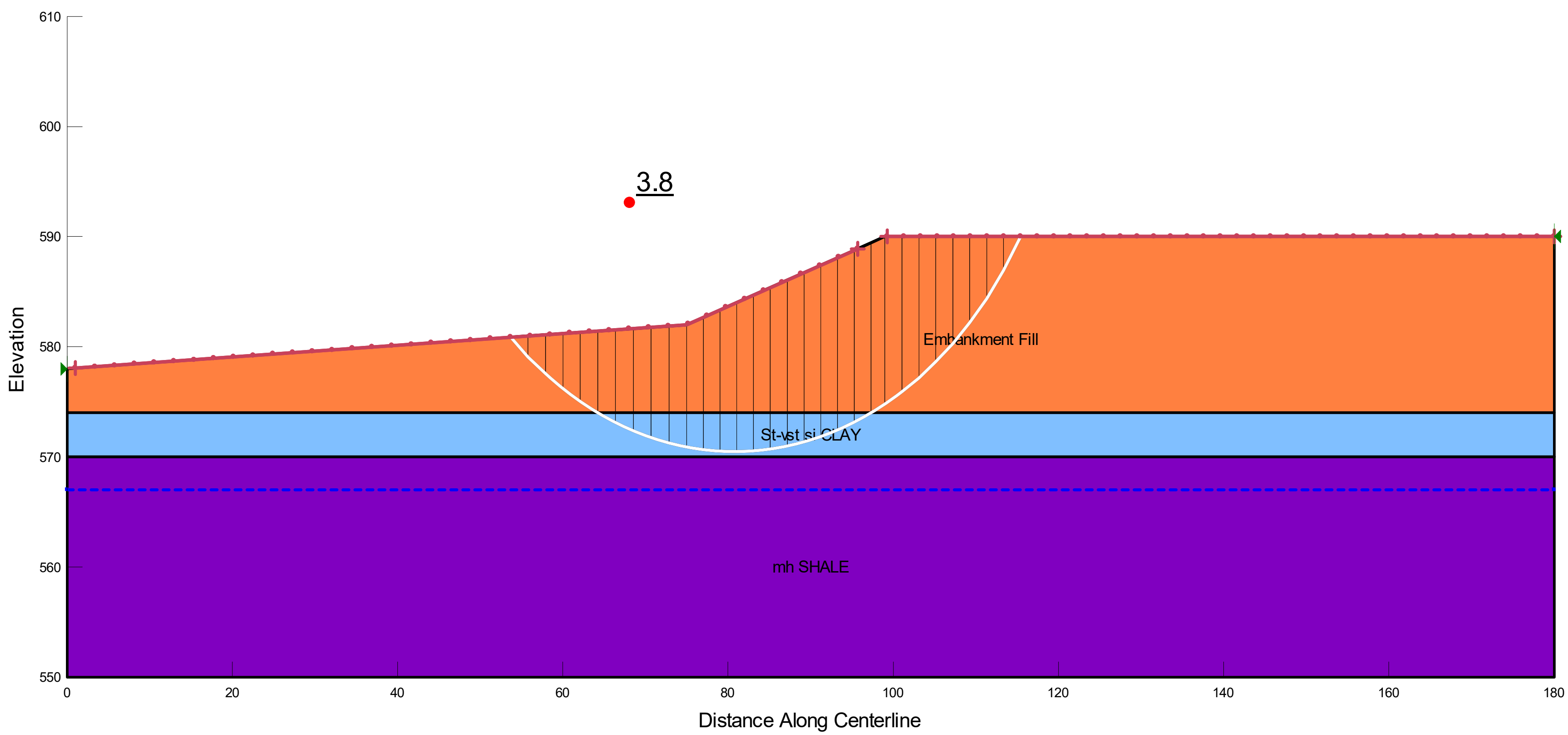
Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
3H:1V Slope, H=8 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



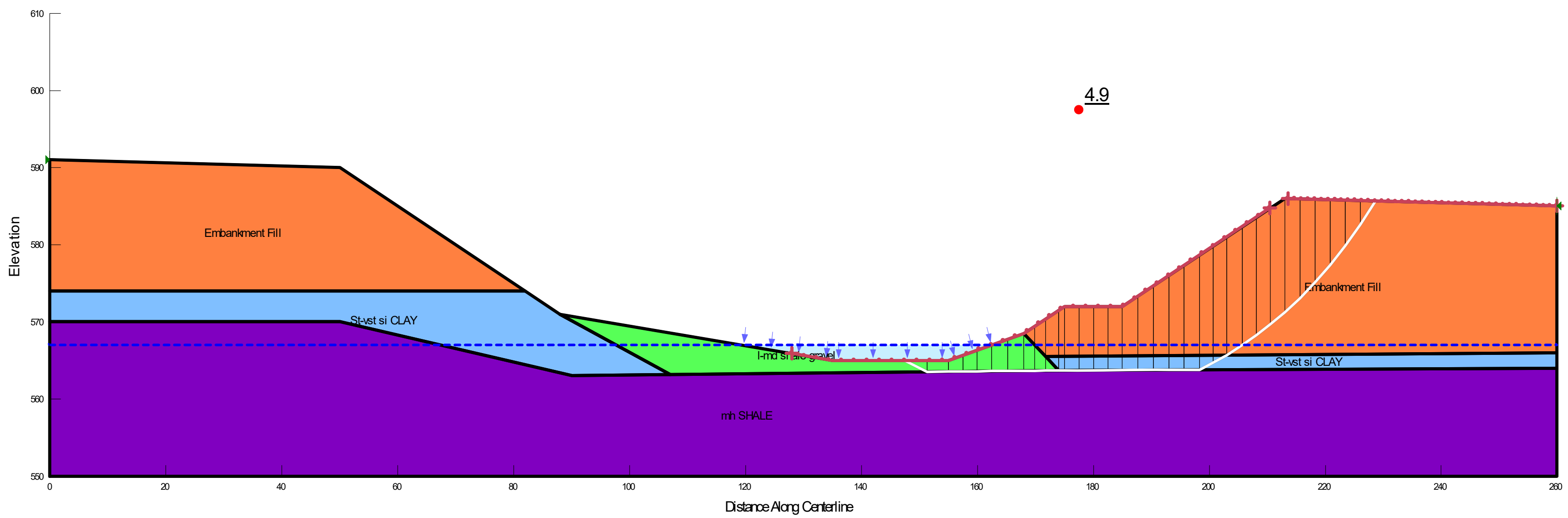
Results of Stability Analyses – Rapid Drawdown Condition, El 574 to Existing Grade
Bent 1 Side Slope
3H:1V Slope, H=8 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



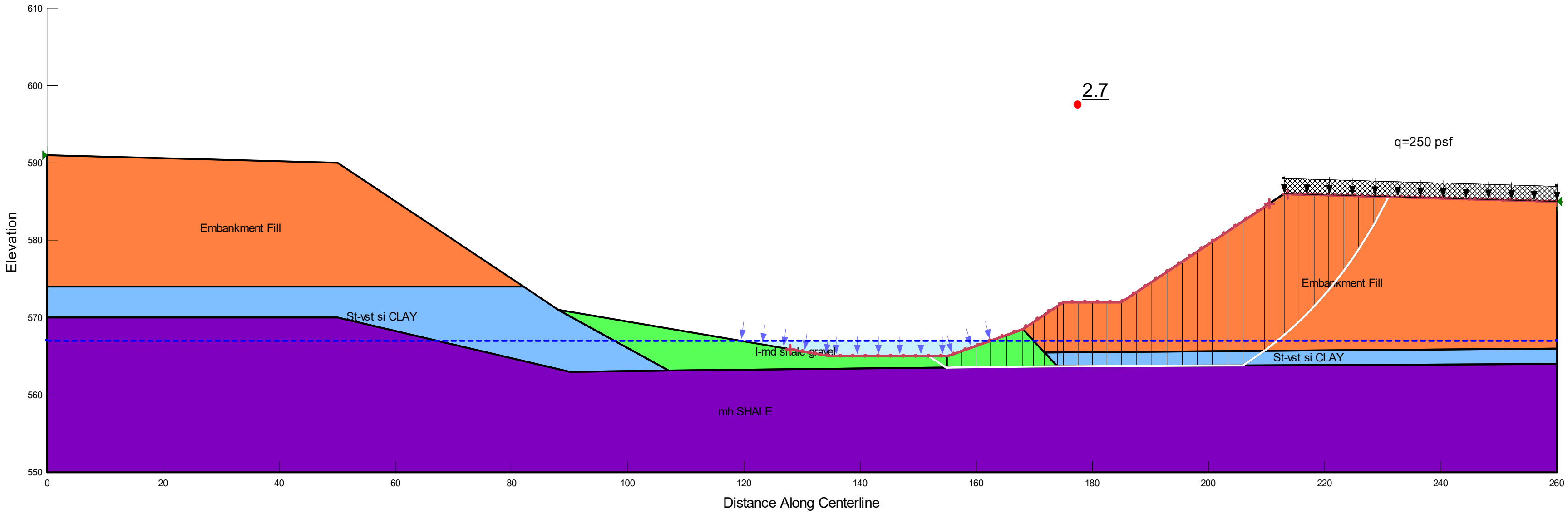
Results of Stability Analyses – Seismic Condition (kh = A_S /2= 0.03)
Bent 1 Side Slope
3H:1V Slope, H=8 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



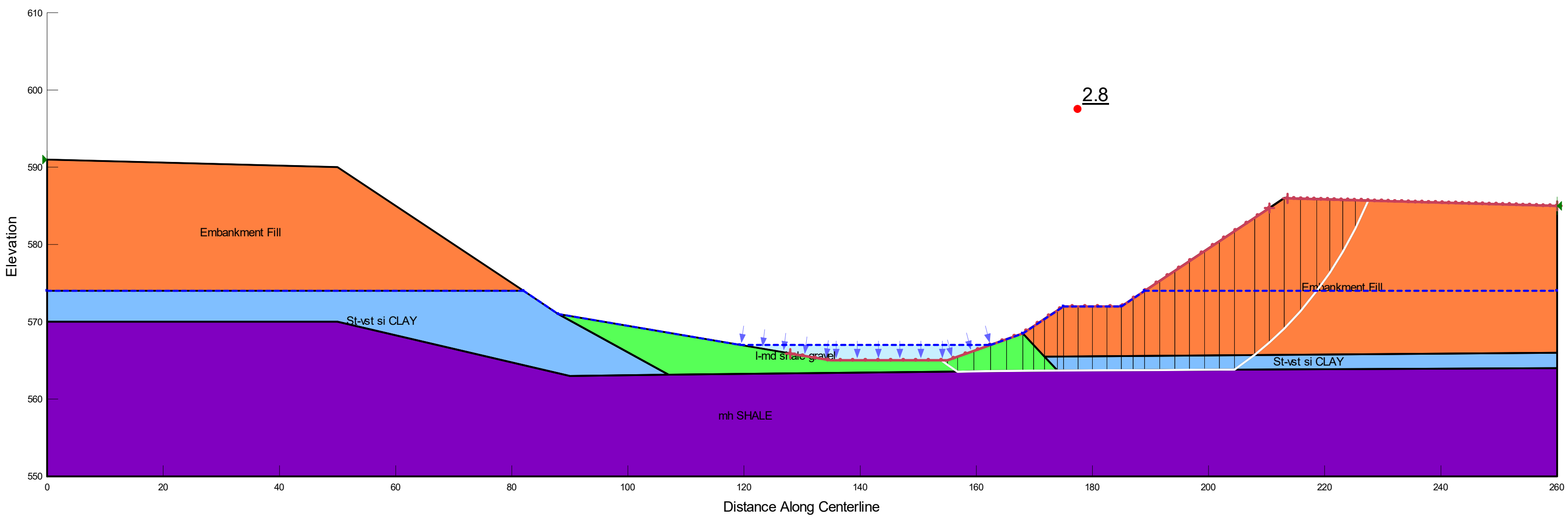
Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=14 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



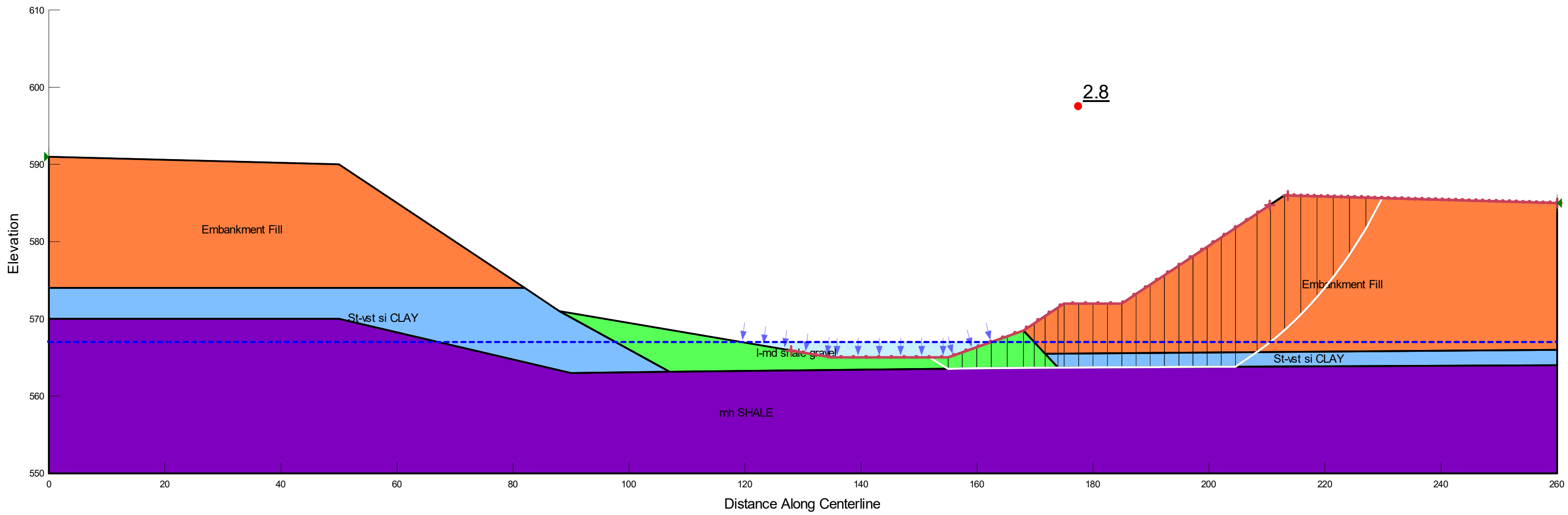
Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=14 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



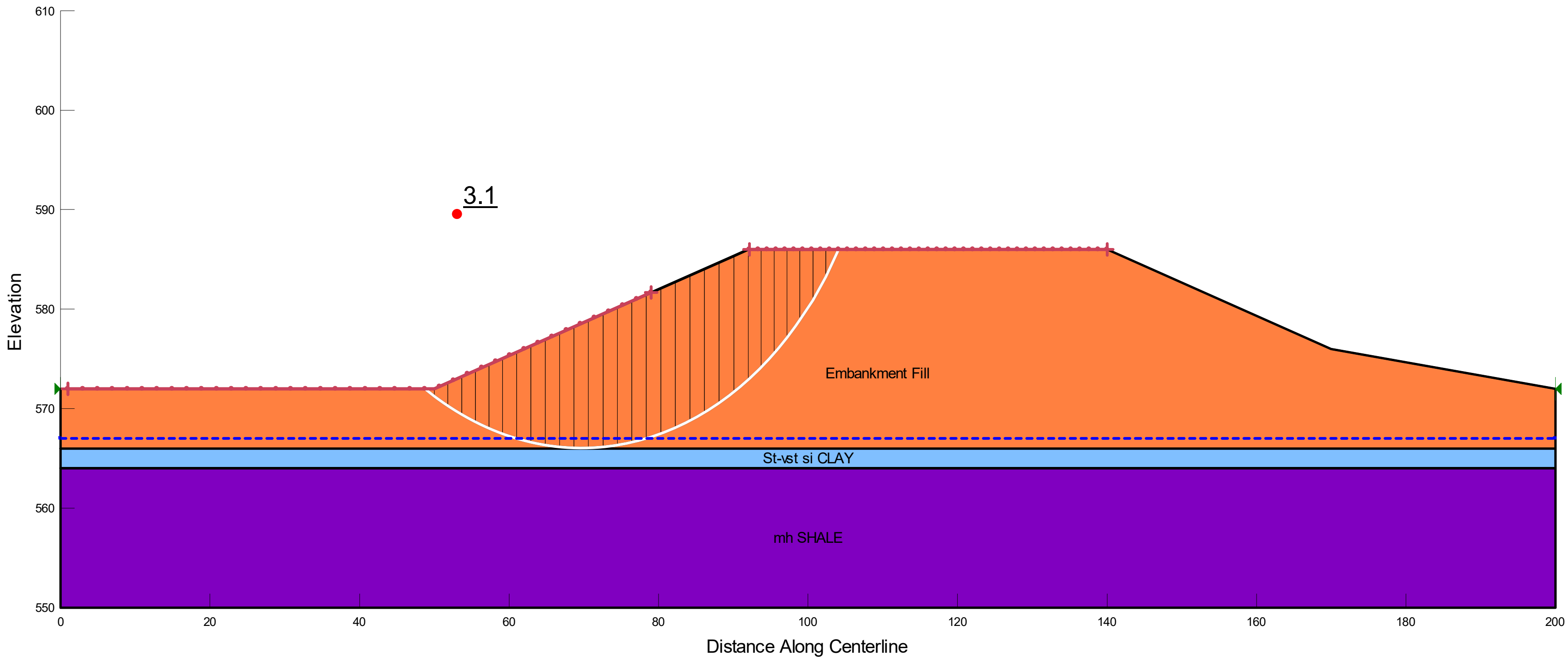
Results of Stability Analyses – Rapid Drawdown Condition, E1 574 to El 567
Bent 4 End Slope
2H:1V Slope, H=14 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



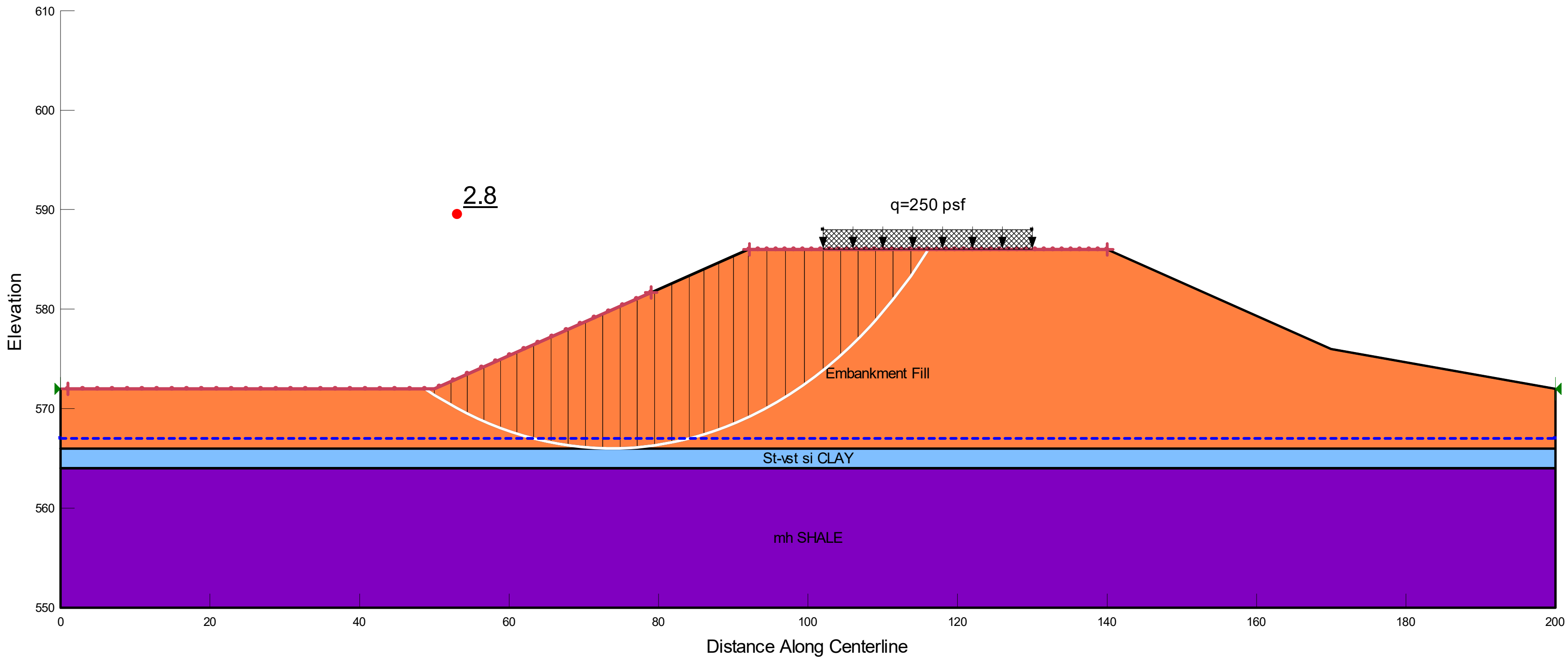
Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.03$)
Bent 4 End Slope
2H:1V Slope, H=14 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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

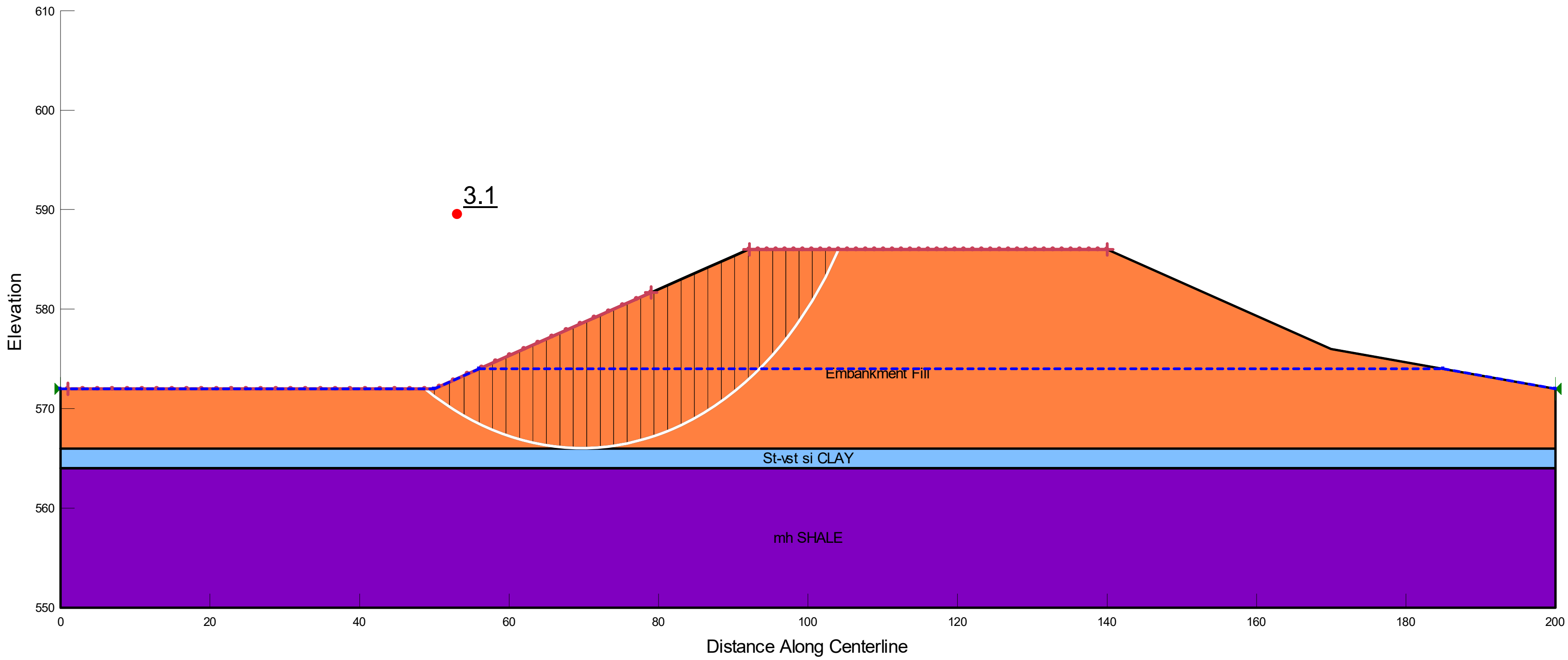


Results of Stability Analyses – End of Construction
Bent 4 Side Slope
3H:1V Slope, H=14 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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

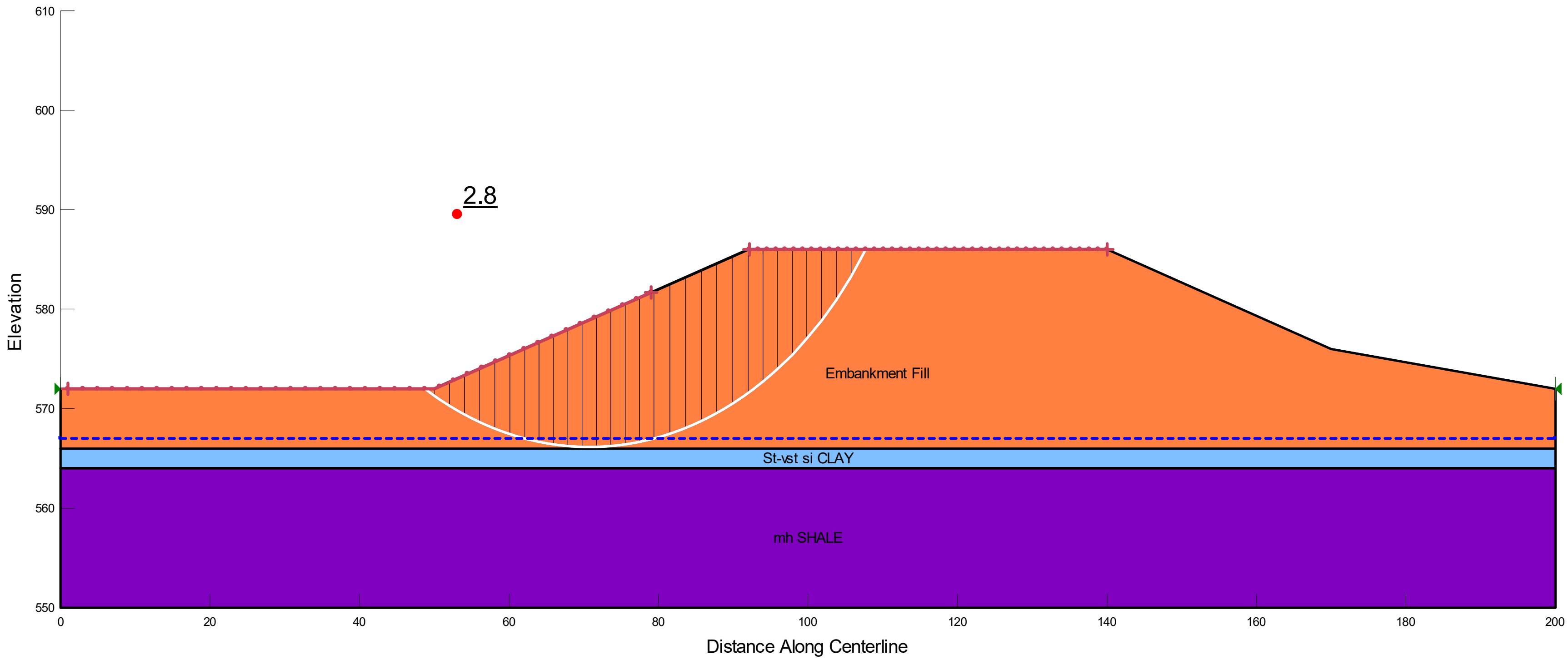


Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
3H:1V Slope, H=14 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas



Results of Stability Analyses – Rapid Drawdown Condition, El 574 to Existing Grade
Bent 4 Side Slope
3H:1V Slope, H=14 ft±
20-064 - ArDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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



Results of Stability Analyses – Seismic Condition (kh = A_S /2= 0.03)
Bent 4 Side Slope
3H:1V Slope, H=14 ft±
20-064 - ARDOT Job No. 040781 Hwy. 71 over Prairie Creek (#02240)
Sebastian Co., Arkansas

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