ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 050413

FEDERAL AID PROJECT NO. NHPP-0012(40)

CADRON CREEK STR. & APPRS. (S)

STATE HIGHWAY 25 **SECTION** 2

IN CLEBURNE **COUNTY**

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GEOTECHNICAL INVESTIGATION
HWY. 25 OVER CADRON CREEK
ARDOT 050413 CADRON CREEK STRS. & APPRS. (S)
CLEBURNE COUNTY, ARKANSAS

INTRODUCTION

Submitted herein are the final results of the geotechnical investigation performed for the Hwy. 25 over Cadron Creek Replacement Bridge, ARDOT Job 050413 Cadron Creek Strs. & Apprs. (S). This geotechnical investigation was authorized on behalf of Garver, LLC by the subconsultant agreement of April 16, 2019. Results of this study have been provided to Garver, LLC (Engineer) as data were developed. Recommendations for structure foundations were provided on April 11, 2020.

We understand the replacement bridge will be a continuous integral prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 166 feet. We also understand that a foundation system consisting of steel piles is planned at the bridge ends (Bents 1 and 4) and drilled shaft foundations are planned for the interior bents (Bents 2 and 3). Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends and for side slopes. A preliminary bridge layout is provided in Appendix A.

The purposes of this 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 borings at planned bridge location to evaluate subsurface conditions and to obtain samples of the foundation soil 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 have been considered in developing the recommendations and considerations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface conditions at the Hwy. 25 over Cadron Creek Replacement Bridge location have been investigated by drilling three (3) sample and core borings to depths of 40 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 Exploration Program

Boring No.	Approx. Sta	Approx. Offset, ft	Approx. Surf El, ft	Completion Depth, ft
S1	795+90	10 Lt	650	40
S2	797+55	10 Lt	646	40
S3	796+80	5 Lt	642	40

The 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 and approximate ground surface elevation, as inferred from the topographic information provided by the Engineer (Garver, LLC), are also shown on the logs. It must be recognized that the elevations shown are approximate and actual elevations may vary. 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 profiles should be anticipated.

The borings were drilled with track-mounted CME-850X and CME-55 rotary-drilling rigs. A combination of dry-auger and rotary-wash drilling techniques was utilized to advance the borings. Soil samples were typically obtained using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb automatic hammer dropped 30 in. in accordance with Standard Penetration Test (SPT) procedures. 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 obtaining samples via the SPT, cuttings were obtained for use in visual classification.

Selected rock cores were recovered using a 5-ft-long, NQ_{wl}-size, double-walled 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 each core run as the sum of sound rock core greater than 4-inch 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. Photographs of the rock cores are provided in Appendix C.

All samples were examined and visually classified by a geotechnical engineer, geologist, or geotechnical technician. Representative samples were placed in appropriate containers to prevent moisture loss and/or change in condition during transfer to our laboratory for further examination and testing. Rock cores were removed from the core barrel and placed in waxed cardboard core boxes. All field logs, soil samples, and rock cores were reviewed by a GHBW geotechnical engineer.

The borings were drilled using dry-auger procedures to the extent possible in order to facilitate groundwater observations. Groundwater levels were measured during and at the completion of drilling operations. Observations regarding groundwater are noted in the lower-right portion of each log and are discussed in subsequent sections of this report.

LABORATORY TESTING

To evaluate pertinent physical and engineering characteristics of the foundation and subgrade soil and rock, laboratory tests consisting of natural water content determinations and classification tests were performed on selected representative soil and rock samples. A total of 14 natural water content determinations were performed to develop a water content profile 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 classification and to evaluate soil plasticity, five (5) liquid and plastic (Atterberg) limit determinations and seven (7) sieve analyses were performed on selected representative samples. The Atterberg limits are plotted on the logs as small 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 "Minus No. 200" column on the log forms. Classification test results, as well as soil classification by the Unified Soil Classification System and AASHTO Soil Classification System, are summarized in Appendix D.

The compressive strength of the shale and sandstone bedrock was evaluated by performing seven (7) uniaxial compression tests on representative rock cores. The measured compressive strength is tabulated 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.

GENERAL SITE and SUBSURFACE CONDITIONS

Site Conditions

The project site is located in the south-central portion of Cleburne County where Hwy. 25 crosses Cadron Creek. The replacement bridge location is about two (2) miles east of the town of Pearson. At the project site, the creek channel is narrow and relatively shallow but the flood plain is relatively broad. The terrain surrounding the bridge is predominantly flat. Thick woodlands border the bridge alignment and line the channel sides.

The existing bridge has two (2) traffic lanes and the bridge deck is in fair condition. The structure is apparently supported on concrete footings. The alignment of the new bridge (downstream and west of the existing bridge) is wooded. The existing roadway is on embankment. The area terrain is undulating to flat and surface drainage is considered poor to fair.

Site Geology

Geologically, the project site is 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 up to approximately 25,000 feet. The formation is conformable with the Johns Valley Shale in the Ouachita Mountains.

Seismic Conditions

Based on the Hwy. 25 over Cadron Creek Replacement Bridge 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 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 for Site Class B (S_1) is 0.108 and the 1.0-sec period spectral acceleration coefficient (S_{D1}) value for Site Class B is 0.108. Utilizing these parameters, Table 3.10.6-1[2] indicates that a Seismic Performance Zone 1 is fitting for the Hwy. 25 bridge site. In reference to 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 0.179 with respect to Seismic Site Class B and the Cadron Creek bridge location.

Analyses were performed to evaluate the liquefaction potential of the subsurface soils and to aid in developing suitable foundation systems. The analyses were performed utilizing the methodology proposed by Idriss and Boulanger[3] in 2008 and the Microsoft Excel© spreadsheet provided by the Department. The results of the liquefaction analyses indicate that the potential for liquefaction is low with the calculated factor of safety against liquefaction exceeding 1.0. The results of the liquefaction analyses are provided in Appendix E.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Cadron 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 at the bridge locations, the anticipated moderate bridge foundation loads, and our understanding of the project, we recommend that

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1 AASHTO LRFD Bridge Design Specifications, 7th Edition; AASHTO; 2014.
2 AASHTO LRFD Bridge Design Specification, AASHTO; 2012.
3 "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

foundation loads be supported on steel piling at the bridge ends (Bents 1 and 4) and on drilled shafts at the interior bents (Bent 2 and 3). Recommendations for foundations are discussed in the following report sections.

Bridge Ends – Steel Piles

Driven HP12x53 or HP14x73 steel piles are recommended for support of the bridge end foundation loads. Alternative pile sizes or types may be considered if desired. Point-bearing steel piles should be driven to refusal, extending through embankment fill, the natural overburden soils, and to refusal in the moderately hard weathered fine-grained sandstone, shale, or graywacke sandstone. Piles should be driven to practical refusal. We recommend that all the steel piles be fitted with rock points.

Steel piles driven to refusal should be designed for the structural capacity of the pile, as per applicable AASHTO Load and Resistance Factor Design (LRFD) procedures[4]. An effective resistance factor (φ_c) of 0.50 is recommended for structural determination of factored bearing capacities. This effective resistance factor for H piles is based on the assumption of damage due to driving conditions. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 and 0.8 are recommended for evaluating compression and uplift capacities, respectively. Post-construction settlement of piles driven to refusal should be less than 0.5 inch.

For determination of bearing capacities of steel piles driven to refusal 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. It has been our experience that allowable pile capacities of 97 tons for HP12x53 piles and 134 tons for HP14x73 piles are typical for f_y = 50 ksi steel pile sections. These capacities are based on allowable stress design (ASD). However, the appropriate factored bearing capacity should be confirmed by the Engineer. Post-construction settlement of piles driven to refusal will be negligible.

Estimated pile tip elevations are summarized below in the table below.

Estimated Tip Elevations of Steel Piles Driven to Refusal

Bent No.	Estimated Pile Tip Elevation, ft	Comments
1 (South Bridge End)	636	Refusal in moderately hard weathered fine-grained sandstone
4 (North Bridge End)	632	Refusal in moderately hard weathered shale or graywacke sandstone

4 Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures, Publication No. FHWA HI-98-032, National Highway Institute, May 2001.

It should be noted that the 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 refusal and final depth must be field verified.

Piles should be installed in compliance with Standard Specifications for Highway Construction, 2014 Edition, Section 805. However, we recommend a hammer delivering a minimum energy of 22,000 ft-lbs per blow. 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 driving. We have recommended that all piles be fitted with rock points.

A minimum pile length of 10 ft is recommended. Preboring is not expected to be required for pile installation. However, some preboring could be required to attain the recommended minimum pile length. As a minimum, safe bearing capacity of piles should be determined by Standard Specifications Section 805.09, Method A. 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. Driving records should be available for review by the Engineer during pile installation.

Interior Bents - Drilled Shaft Foundations

We recommend that the foundation loads of the interior bents be supported on drilled shafts. Drilled shafts should be founded with a minimum embedment of two (2) shaft diameters or 6 ft, whichever is greater, into the moderately hard gray with tan weathered shale and/or moderately hard gray fine-grained graywacke sandstone. Drilled shafts founded as recommended may be sized using a maximum nominal end-bearing pressure (R_n) of 150 kips per sq foot. A resistance factor (φ_{stat}) of 0.50 is recommended for drilled shaft end bearing. Total and differential settlement of properly installed drilled shafts founded in the competent moderately hard gray with tan weathered shale and/or moderately hard gray fine-grained graywacke sandstone as recommended is expected to be negligible. We also recommend that drilled shafts be sized for axial compression loads based on end bearing alone.

Resistance to uplift loads will be developed by circumferential shaft friction. Drilled shafts will penetrate the overburden soils to bear in the competent moderately hard gray with tan weathered shale and/or moderately hard gray fine-grained graywacke sandstone. Uplift resistance for the top 5 ft of shaft length, all penetration through the overburden soils, and the length of permanent casing should be neglected. For shaft penetration through the competent moderately hard weathered shale and moderately hard fine-grained graywacke sandstone, a maximum nominal skin resistance value of 12 kips per sq ft is recommended. For evaluation of uplift capacity, a resistance factor (φ_{up}) of 0.40 is recommended for shaft skin friction.

A minimum shaft diameter of 36 in. and a minimum shaft rock socket length of two (2) shaft diameters or 6 ft, whichever is greater, are recommended for drilled shafts. Based on the results of the borings, the anticipated minimum shaft depths for the interior bents of the replacement bridge structure are summarized below. The estimated shaft length is based on a shaft diameter of 48 in. and depth below existing grade.

Estimated Minimum Shaft Lengths – 48-in.-diameter Shafts

Bent No.	Estimated Minimum Shaft Length, ft	Estimated Minimum Shaft Bottom El, ft
2	15	623
3	18	624

The minimum shaft lengths and minimum shaft bottom elevations shown above are estimates only based on the results of the borings, the inferred surface elevation at the particular bent location, and existing grades. Suitable bearing stratum and final shaft lengths must be field verified. Plan shaft lengths and shaft tip elevations must be based on the magnitude of foundation loads, specific subsurface conditions, and actual shaft diameters. Depending on specific subsurface conditions and rock quality, localized deepening or shortening of shaft depths will be warranted.

All drilled shaft excavations should be observed by the Engineer to verify suitable bearing and adequate penetration. Heavy-duty drilling equipment will be required to advance the shaft excavations. The moderately hard fine-grained graywacke sandstone bearing stratum will be resistant to drilling and rock drilling methods are expected to be required to achieve the required shaft penetration.

To verify competence of the moderately hard weathered shale/fine-grained graywacke sandstone bearing stratum, we recommend that all shaft excavations be probed. Probe holes should consist of continuous rock core borings advanced from the plan shaft bottom elevation into the bearing stratum a depth of at least two (2) shaft diameters below the plan bottom elevation. Rock cores from probe holes should be reviewed by the Engineer to verify foundation stratum competence and suitability of the plan shaft bottom elevation.

Temporary MSE Wall Bearing Capacity

It is understood that temporary mechanically stabilized earth (MSE) retaining walls are planned to extend from the north and south bridge ends parallel to the existing roadway/bridge embankment to facilitate maintenance of traffic. Maximum wall heights on the order of 14 ft are anticipated. Preliminary wall layout plans indicate wall subgrades at El 650 on the south end (see Boring S1) and El 638 to El 650 on the north bridge end (see Boring S2). The subgrade elevation rises and wall height decreases with distance from the bridge ends. A minimum wall embedment of 2 ft below lowest adjacent grade is recommended.

The subsurface information available for evaluation of wall bearing is currently limited to the borings drilled at the bridge ends, i.e., Borings S1 and S2. These data indicate that rock, moderately hard weathered sandstone or moderately hard weathered shale, are present at El 637 on the south (see Boring S1) and at El 638 on the north (see Boring S2). The overburden soils at the south bridge end are very soft to firm fine sandy clay to about El 644 and very loose to loose silty fine sand to about El 637. The overburden soils at the south bridge end have low bearing capacity and high compressibility. These soils are not suitable for wall bearing. At the north end, the overburden soils to about El 644 are loose fine sand with medium dense to dense fine sand and silty fine sand below that and extending to about El 637. On the south bridge end, the soils below about El 644 have moderate bearing capacity and low compressibility.

Based on the currently available information, preliminary considerations for temporary MSE wall bearing capacity are summarized below.

- South Wall (Boring S1)
  - Undercut to the top of rock, estimated about 13 ft (El 637±).
    - Backfill with crushed stone aggregate base (ARDOT Standard Specifications Section 303, Class 7) or Select Granular Backfill (AASHTO M 43 No. 57).
    - For undercuts backfilled with Class 7 base or No. 57 stone, a nominal bearing capacity of 15,000 lbs per sq ft is recommended.
    - A resistance factor (φ_b) of 0.65 is recommended for evaluation of bearing.
  - Alternatively, bearing capacity may be improved by use of rammed aggregate piers.
    - For nominal 24-in.-diameter rammed aggregate piers and a 20 percent area ratio, a nominal composite bearing capacity of 8400 lbs per sq ft may be assumed. A rammed aggregate pier length of about 10 to 12 ft is anticipated. A resistance factor (φ_b) of 0.65 is recommended for evaluation of bearing.

   - Detailed design for use of rammed aggregate piers must be provided by the aggregate pier provider.
- North Wall (Boring S2)
  - Undercut to medium dense to dense fine sand and/or silty fine sand, estimated at about 2-ft depth (El 644±).
    - Backfill with crushed stone aggregate base (ARDOT Standard Specifications Section 303, Class 7) or Select Granular Backfill (AASHTO M 43 No. 57).
    - For undercuts backfilled with Class 7 base or No. 57 stone, a nominal bearing capacity of 6500 lbs per sq ft is recommended.
    - A resistance factor (φ_b) of 0.65 is recommended for evaluation of bearing.
  - Alternatively, bearing capacity may be improved by use of rammed aggregate piers.
    - For nominal 24-in.-diameter rammed aggregate piers and a 20 percent area ratio, a nominal composite bearing capacity of 11,000 lbs per sq ft may be assumed. A rammed aggregate pier length of about 8 to 10 ft is anticipated. A resistance factor (φ_b) of 0.65 is recommended for evaluation of bearing.
    - Detailed design for use of rammed aggregate piers must be provided by the aggregate pier provider.

Resistance to wall sliding can be evaluated using an <u>ultimate</u> friction factor (tan δ) value of 0.55 between the temporary MSE walls and undercut backfill or composite rammed aggregate piers and the on-site soils. A resistance factor (φ_τ) of 1.0 is recommended for evaluation of sliding resistance. Long-term post-construction settlement of the wall foundation soil is expected to be less than 2 inches, depending on the method of bearing stratum improvement.

Currently it is planned to obtain additional information on subsurface conditions in the temporary MSE wall alignments. More detailed recommendations for temporary MSE wall bearing will be provided when those data are available.

<u>Embankment Slope Stability</u>

The replacement bridge will include new end slope configurations on the north and south ends of the bridge. The plan embankment configurations for the north and south bridge ends are planned with 2-horizontal to 1-vertical (2H:1V) configurations.

To evaluate suitability of the plan 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 2007[5] and a

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[5] <u>Slope/W 2007</u>; GEO-SLOPE International; 2008.

Morgenstern-Price 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 0.085. For evaluating the rapid drawdown condition, a water surface elevation drop from El 646 to El 640 has been assumed. The sections used for the analyses are shown in the graphical results provided in Appendix F.

For the purposes of the stability analyses, unclassified embankment as per 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 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 are 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 Earthwork Considerations

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 toe.

Where existing pavements will 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 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 1 to 3 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. Given the predominance of granular soils, the use of cement would be expected to be advantageous on this site. 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, and 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 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. Slope configurations steeper than 2.5H:1V should be protected from erosion with riprap.

Subgrade preparation should comply with 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 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 precipitation or stream 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 (Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (Standard Specifications for Highway Construction, 2014 Edition, Subsections 403.01 and 403.02, Class 3 mineral aggregate) 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 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 to achieve the minimum pile length is anticipated.

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.

Drilled Shafts

Groundwater could be encountered in drilled shaft excavations. Limited seepage into drilled shaft excavations can probably be controlled by close coordination of drilling, cleanup and concrete placement. Use of permanent casing is anticipated for drilled shafts. We recommend that temporary casing also be on site in the event it is needed to control seepage and/or caving into shaft excavations. Drilled shaft excavations should essentially be dry at the time of concrete placement. Where more than about 3 in. of water is present in shaft excavations, the excavation should be dewatered prior to concrete placement. Where shaft excavations cannot be dewatered, underwater concrete placement should be performed with a concrete pump fitted with a rigid end extension. A muck bucket or similar tools should be utilized to clean the shaft excavation bottom prior to underwater concrete placement.

Some hard drilling could be experienced when advancing drilled shafts into the more resistant units of the moderately hard gray with tan weathered shale and moderately hard gray fine-grained graywacke sandstone. Heavy-duty drilling equipment and rock drilling tools will be required to advance shaft excavations to the recommended minimum penetration in these more resistant units. Coring or other rock excavation methods is likely to be required to achieve the recommended penetration into the shale and sandstone bearing strata. All drilled shaft excavations should be observed by the Engineer to verify suitable bearing and adequate penetration.

CLOSURE

The Engineer or Department or a designated representative thereof should monitor site preparation, grading work and all wall 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 included and complete this final report.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 5	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	Liquefaction Analysis Results
Appendix F	Stability Analyses Results

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

Sincerely,

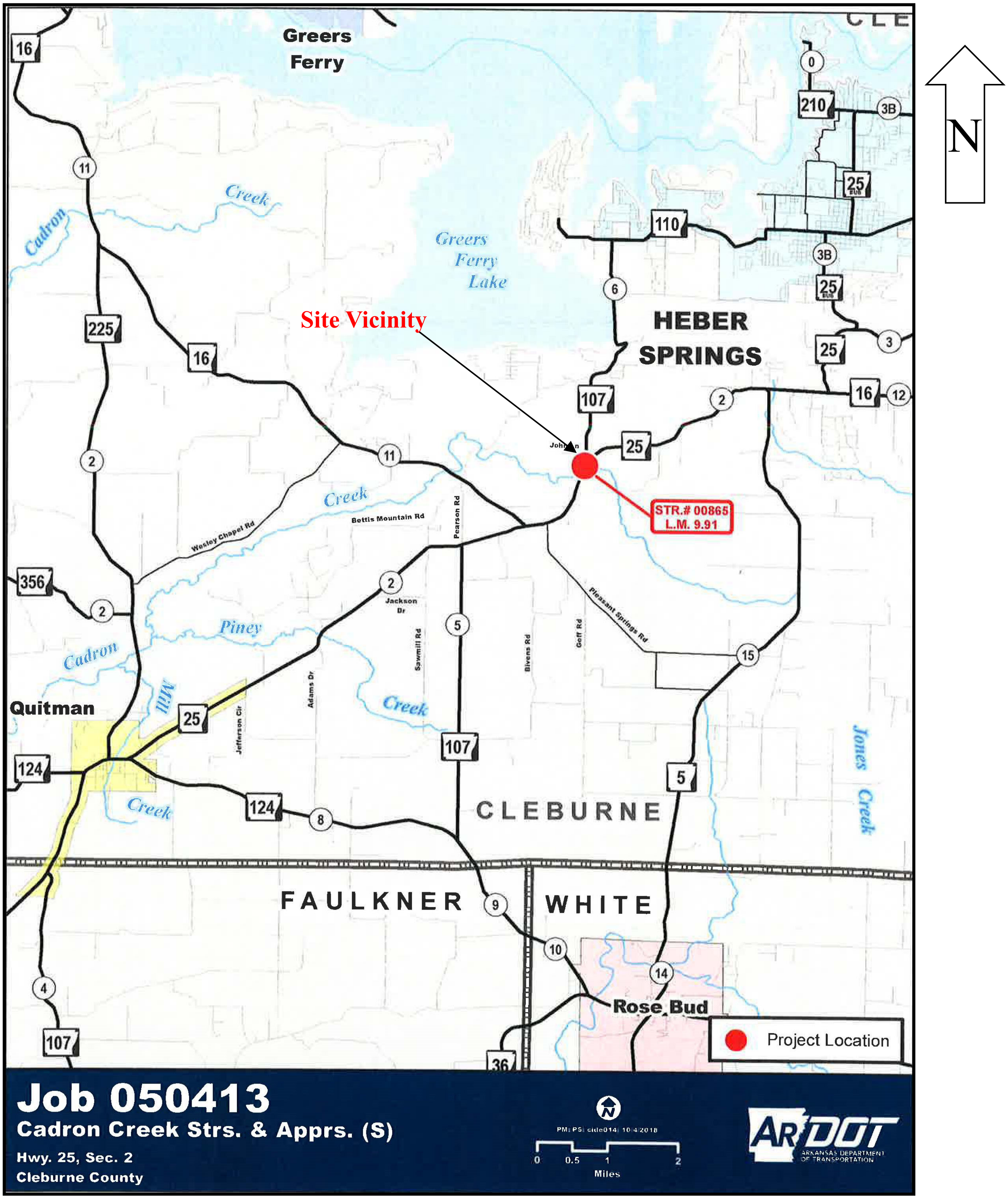
GRUBBS, HOSKYN,
BARTON &WYATT, INC.

Ben Davis, P.E.
Project Engineer

Mark E. Wyatt, P.E.
President

BJD/MEW:jw

Copies Submitted:	Garver, LLC		
	Attn:	Mr. John H. Ruddell, P.E., S.E.	(1-email)
	Attn:	Mr. Daniel Goad, P.E.	(1-email)

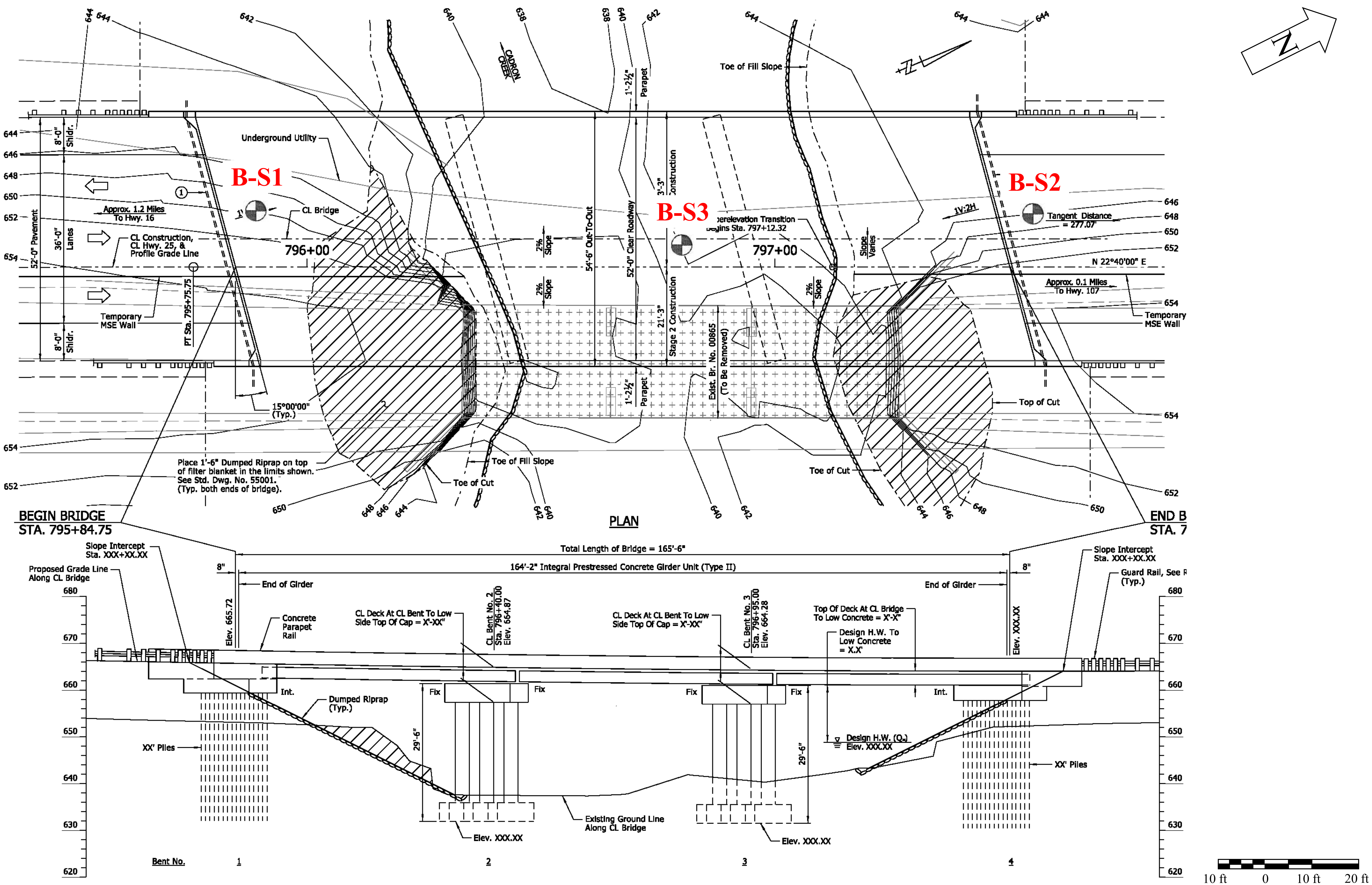


Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

SITE VICINITY MAP
050413 Hwy. 25 over Cadron Creek
Cleburne Co., Arkansas

Job No. 20-035

Plate 1



PLAN OF BORINGS
050413 Hwy. 25 over Cadron Creek
Cleburne Co., Arkansas

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

Scale: As shown

Job No. 20-035

PLATE 2

20-035

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

LOG OF BORING NO. S1

050413 Hwy. 25 over Cadron Creek
Cleburne County, Arkansas

TYPE: Auger to 13 ft /Wash

LOCATION: Approx Sta 795+90, 10 ft Lt

SURF. EL: 650±

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	Firm brown and reddish brown fine sandy clay w/sandstone fragments (fill) - very soft to soft with strong hydrocarbon odor below 2 ft	8 4			38		
4–6	Soft brown fine sandy clay w/a few sandstone fragments	5			37		
6–13	Loose brown silty fine sand w/trace sandstone fragments - very loose, slightly clayey below 8 ft	7 2 25/0"					
13–20	Moderately hard gray weathered fine-grained sandstone w/very close healed horizontal fractures.					43	0
20–21	Moderately hard gray fine-grained graywacke sandstone					98	98
21–28	Moderately hard dark gray shale, carbonaceous, flat bedded - with very close interbedded graywacke sandstone seams and layers below 26.3 ft			q_u= 1510 psi Tuw= 162 pcf		95	95
28–36	Moderately hard gray fine-grained graywacke sandstone					93	93
36–38	Moderately hard dark gray shale, carbonaceous, flat bedded w/occasional arenaceous seams					92	92
38–40	Moderately hard to hard gray fine-grained graywacke sandstone - fine-grained sandstone layer at 38.7 to 39.1 ft						

COMPLETION DEPTH: 40.0 ft
DATE: 3-25-20

DEPTH TO WATER IN BORING: Dry to 13 ft

DATE: 3/25/2020

20-035

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

LOG OF BORING NO. S2

050413 Hwy. 25 over Cadron Creek
Cleburne County, Arkansas

TYPE: Auger to 8.5 ft /Wash

LOCATION: Approx Sta. 797+55, 10 ft Lt

SURF. EL: 646±

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	Loose brown fine sand, slightly silty w/numerous sandstone fragments and crushed stone (fill) - dense below 2 ft	8 49					
4–8.5	Medium dense tan, brown and reddish tan silty fine sand w/sandstone fragments - dense below 6 ft	25 37		-NON-PLASTIC-	17		
8.5–14	Moderately hard gray w/tan weathered shale, flat bedded	50/4" 25/0"					
14–23	Moderately hard gray fine-grained graywacke sandstone w/interbedded shale seams and layers			q_u= 3780 psi, Tuw= 165 pcf		90 70	78 70
23–26.5	Moderately hard dark gray shale, flat bedded w/very close sandstone partings, seams and inclusions						
26.5–40	Moderately hard to hard gray fine-grained sandstone w/pyrite crystals and inclusions and very close argillaceous laminations, seams and inclusions - shale seam at 27.8 to 27.9 ft			q_u= 5160 psi, Tuw= 159 pcf q_u= 9060 psi, Tuw= 164 pcf		88 98 100	88 98 100

NOTE: Water at 1.7 ft after 5 minutes.

COMPLETION DEPTH: 40.0 ft
DATE: 3-23-20

DEPTH TO WATER IN BORING: 8.5 ft

DATE: 3/23/2020

RECRQDN200-2 20-035.GPJ 4-1-20

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

LOG OF BORING NO. S3

050413 Hwy. 25 over Cadron Creek
Cleburne County, Arkansas

TYPE: Auger to 6 ft /Wash

LOCATION: Approx Sta 796+80, 5 ft Lt

SURF. EL: 642±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / WATER CONTENT	- No. 200 %	% Recovery	% RQD
0–2	Loose brown fine sand, slightly silty w/shale and sandstone fragments and occasional organics	9		-NON-PLASTIC-	8		
2–8	Very loose to loose brown, gray and tan fine sand, slightly clayey - medium dense with shale fragments below 4 ft	4 16 50/6"		-NON-PLASTIC-	9		
8–10	Moderately hard to hard gray highly weathered shale						
10–35	Moderately hard gray fine-grained graywacke sandstone w/very close shale seams and layers and numerous pyrite crystals - reddish tan weathered fine-grained sandstone layer at 10 to 10.5 ft and 15 to 15.5 ft					55	53
				q_u= 3990 psi, Tuw= 163 pcf		67	62
	- with very close argillaceous seams and inclusions below 20 ft			q_u= 4450 psi, Tuw= 162 pcf		90	90
				q_u= 3270 psi, Tuw= 161 pcf		100	100
						97	97
35–38	Moderately hard to hard dark gray shale, flat bedded, carbonaceous - graywacke layer at 37.4 to 38 ft					92	92
38–40	Hard gray fine-grained sandstone w/very close argillaceous partings and seams						

COMPLETION DEPTH: 40.0 ft
DATE: 3-17-20

DEPTH TO WATER IN BORING: 3.3 ft

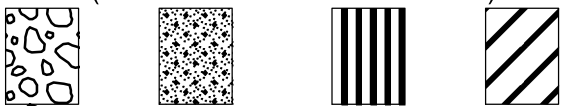
DATE: 3/17/2020

RECRQDN200-2 20-035.GPJ 4-1-20

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

SYMBOLS AND TERMS USED ON BORING LOGS

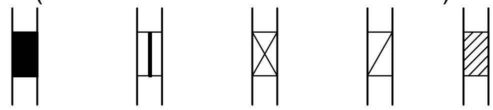
SOIL TYPES
(SHOWN IN SYMBOLS COLUMN)



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

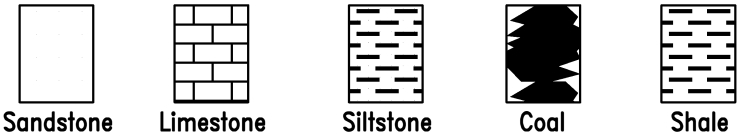
SLICKENSIDED - having inclined planes of weakness that are slick and glossy in appearance.
FISSURED - containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
LAMINATED - composed of thin layers of varying color and texture.
INTERBEDDED - composed of alternate 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

BORING LOG TERMS – ROCK

ROCK TYPES
(SHOWN IN SYMBOLS COLUMN)



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 – 1/8 in. 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) –

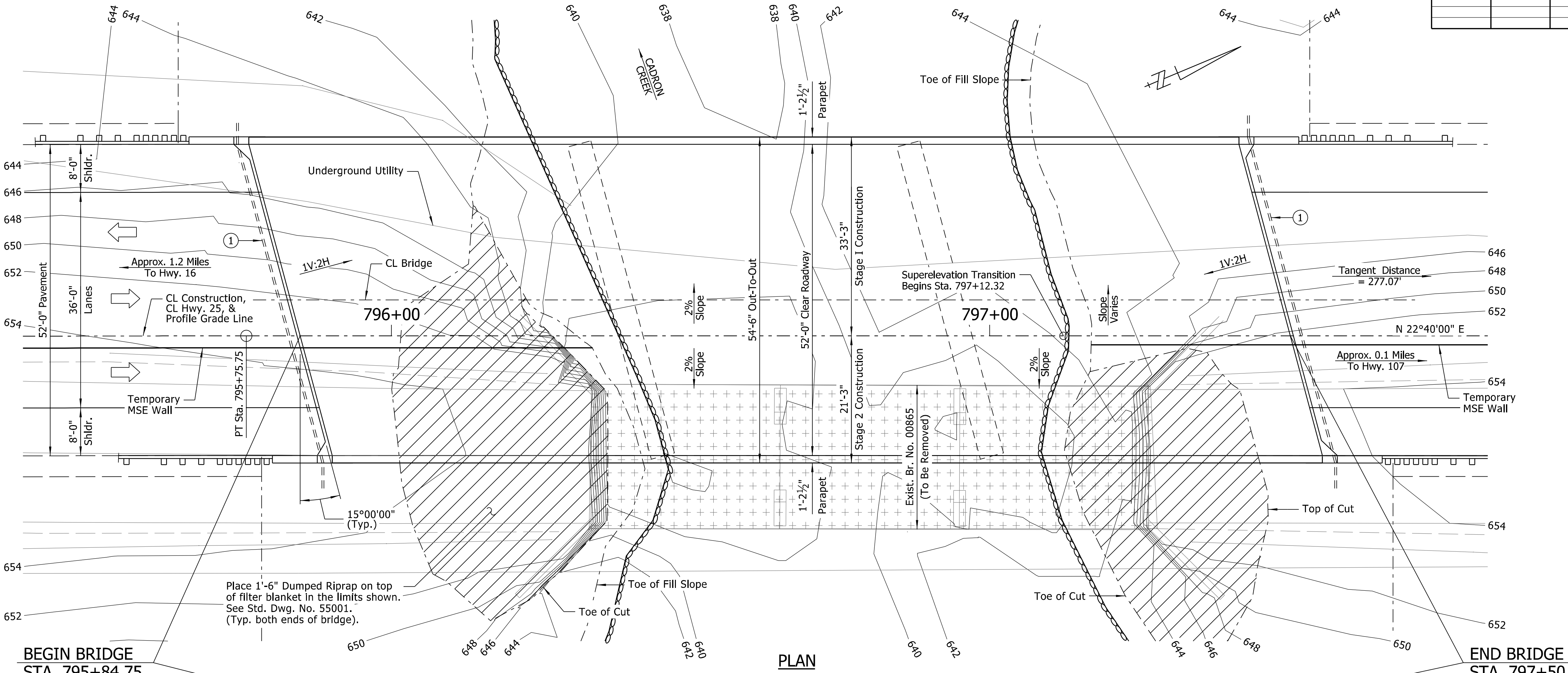
RQD (Percent)	Diagnostic Description
Greater than 90	Excellent
75 – 90	Good
50 – 75	Fair
25 – 50	Poor
Less than 25	Very Poor

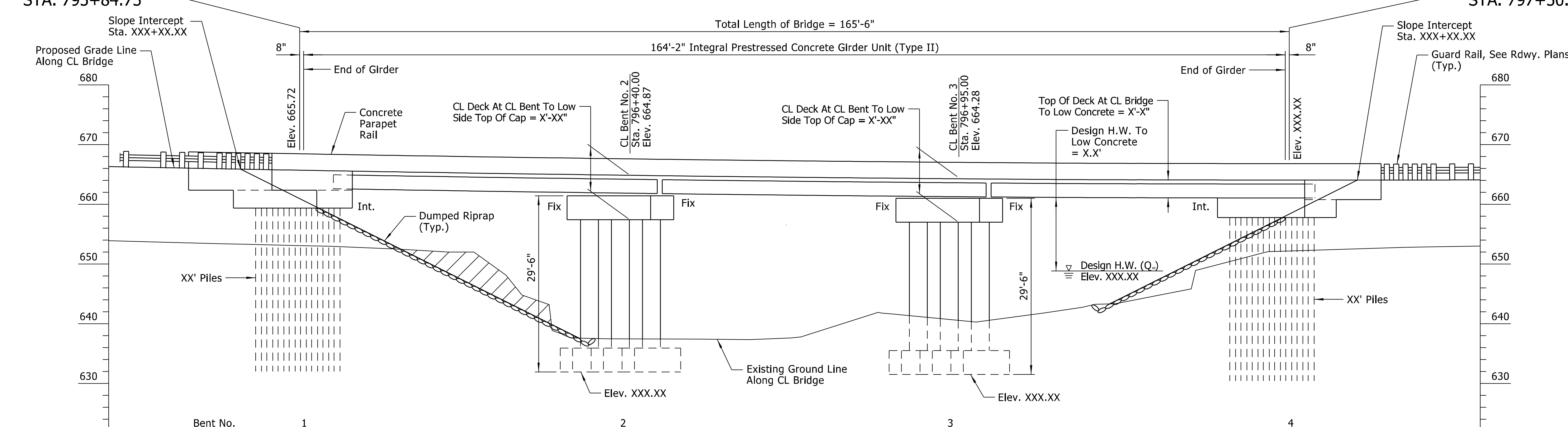
KEYROCK FHWA 3-2-12

APPENDIX A

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		050413	$SNXXXX$	ST

(1) XXXXX LAYOUT $DNXXXX$





NOTE:
Stations shown are along CL Construction. Elevations shown are along the Profile Grade Line at CL Bridge.

FOR R/W DATA AND GUARD RAIL DETAILS, SEE ROADWAY PLANS

796+00

797+00

SOIL BORINGS NOT AVAILABLE AT TIME OF PRINTING

NOTES:
Use Type Special Approach Slab at each end of bridge, see Dwg. No. XXXXX.

Use Type Special Approach Gutters at each end of bridge, see Dwg. No. XXXXX.

For "LOCATION SKETCH" & SOIL BORING ELEVATION", see Dwg. No. XXXXX.

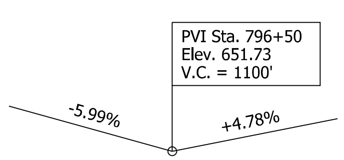
For "GENERAL NOTES", "BORING LEGEND", & "N-VALUES", see Dwg. No. XXXXX.

HORIZONTAL CURVE DATA

Highway 25
PI = 793+51.86
Δ = 02°14'21" Rt.
D = 00°30'00"
T = 223.95'
L = 447.84'
e = N/A
R = 11459.16'

The Contractor shall excavate the existing embankments as shown in "LOCATION SKETCH" on Dwg. No. XXXXX.

(1) Install 4" 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. XXXXX. Pipe Underdrains will not be paid for directly but shall be considered subsidiary to "UNCLASSIFIED EXCAVATION". (Typ. at each end bent)



VERTICAL CURVE DATA

"Name of Roadway"
(Profile Grade Along CL Bridge)

SHEET 1 OF X
LAYOUT OF BRIDGE
CADRON CREEK
STR. & APPRS. (S)
CLEBURNE COUNTY

ROUTE 25 SEC. 2

ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

PRELIMINARY NOT FOR CONSTRUCTION

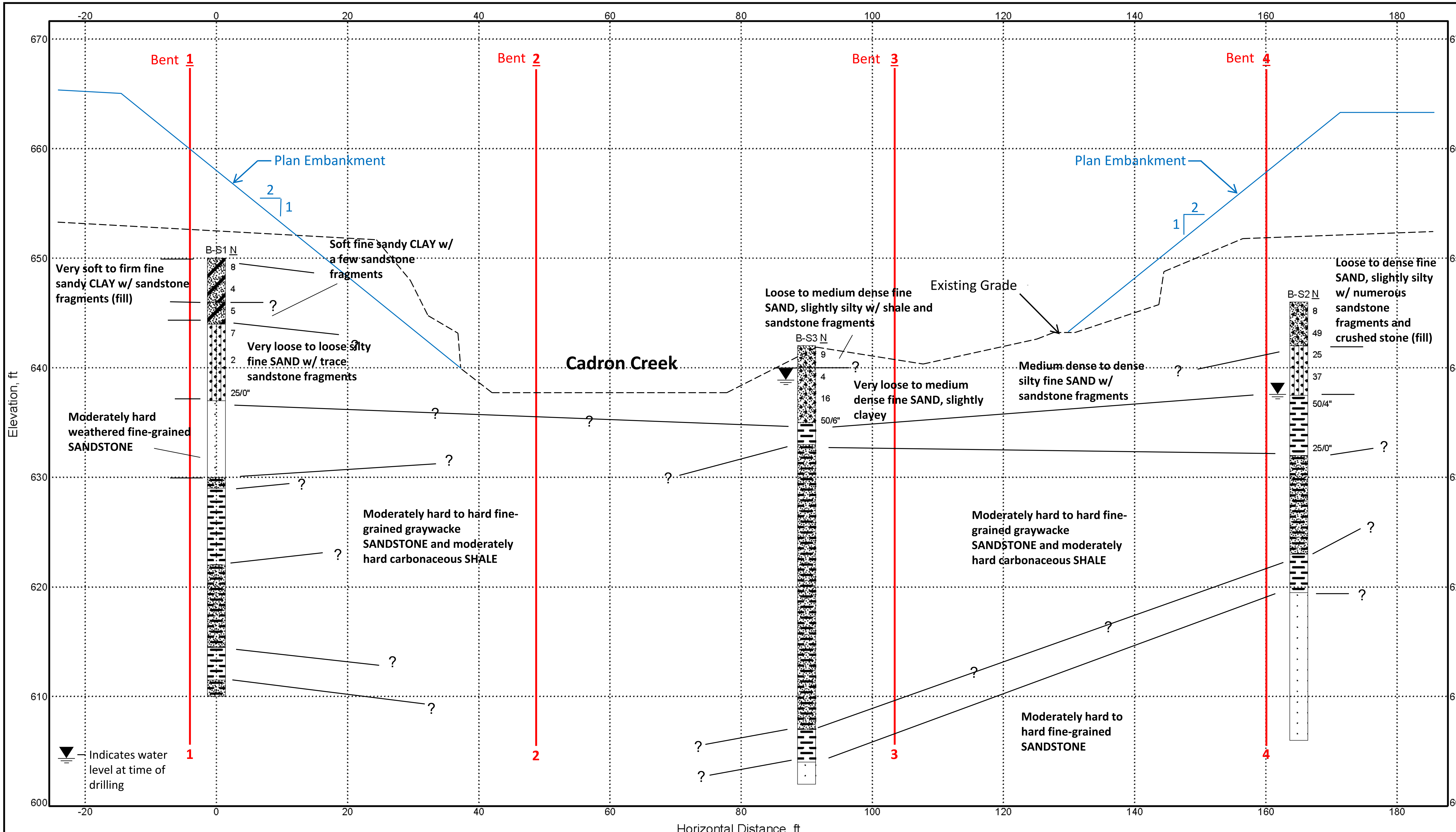
BRIDGE ENGINEER

DRAWN BY: DRG	DATE: FEB. 2020	FILENAME: b050413_L1.dgn
CHECKED BY: XXX	DATE: XXX	SCALE: 1" = 10'-0"
DESIGNED BY: DRG	DATE: FEB. 2020	

BRIDGE NO. $BNXX$ DRAWING NO. $DNXXXX$

drgood 2/5/2020 1:00:30 PM
WORKSPACE: ARDOT Bridge (2019)
L:\20\T\T07608 - 050413 Cadron Creek Str-Apprs\Drawings\b050413-SI0l.dgn
REVISED DATE:

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: As Shown

Generalized Subsurface Profile
050413 Hwy. 25 over Cadron Creek
Cleburne County, Arkansas

Project Number: 20-035

APPENDIX C


20-035
Boring S1
Run 1: 15-20'
Run 2: 20-25'
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

20-035
Boring S1
Run 3: 25-30'
Run 4: 30-35'
Grubbs, Hoskyn, Barton & Wyatt INC.
CONSULTING ENGINEERS


20-035
Boring S1
Run 5 = 35-40'
Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS


20-035
Boring S2
Run 1: 15'-20'
Run 2: 20'-25'
Grubbs, Hoskyn, Barton & Wyatt, INC
CONSULTING ENGINEERS


20-035
Boring S2
Run 3: 25'-30'
Run 4: 30'-35'
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers


20-035
Boring S2
Run 5: 35'-40'
Grubbs, Hoskyn, Barton & Wyatt Inc
Consulting Engineers


20-035
Boring 53
Run1: 10'-15'
Grubbs, Hoskyn, Barton & Wyatt INC.
CONSULTING ENGINEERS


20-035
Boring S3
Run 2: 15'-20'
Run 3: 20'-25'
Grubbs, Hoskyn, Barton & Wyatt, Inc.
CONSULTING ENGINEERS


20-035
Boring S3
Run 4: 25'-30'
Run 5: 30'-35'
Grubbs, Hoskyn, Barton & Wyatt INC
CONSULTING ENGINEERS


20-035
Boring S3
Run 6: 35-40'
Grubbs, Hoskyn, Barton & Wyatt, Inc.
CONSULTING ENGINEERS

APPENDIX D

SUMMARY of CLASSIFICATION TEST RESULTS

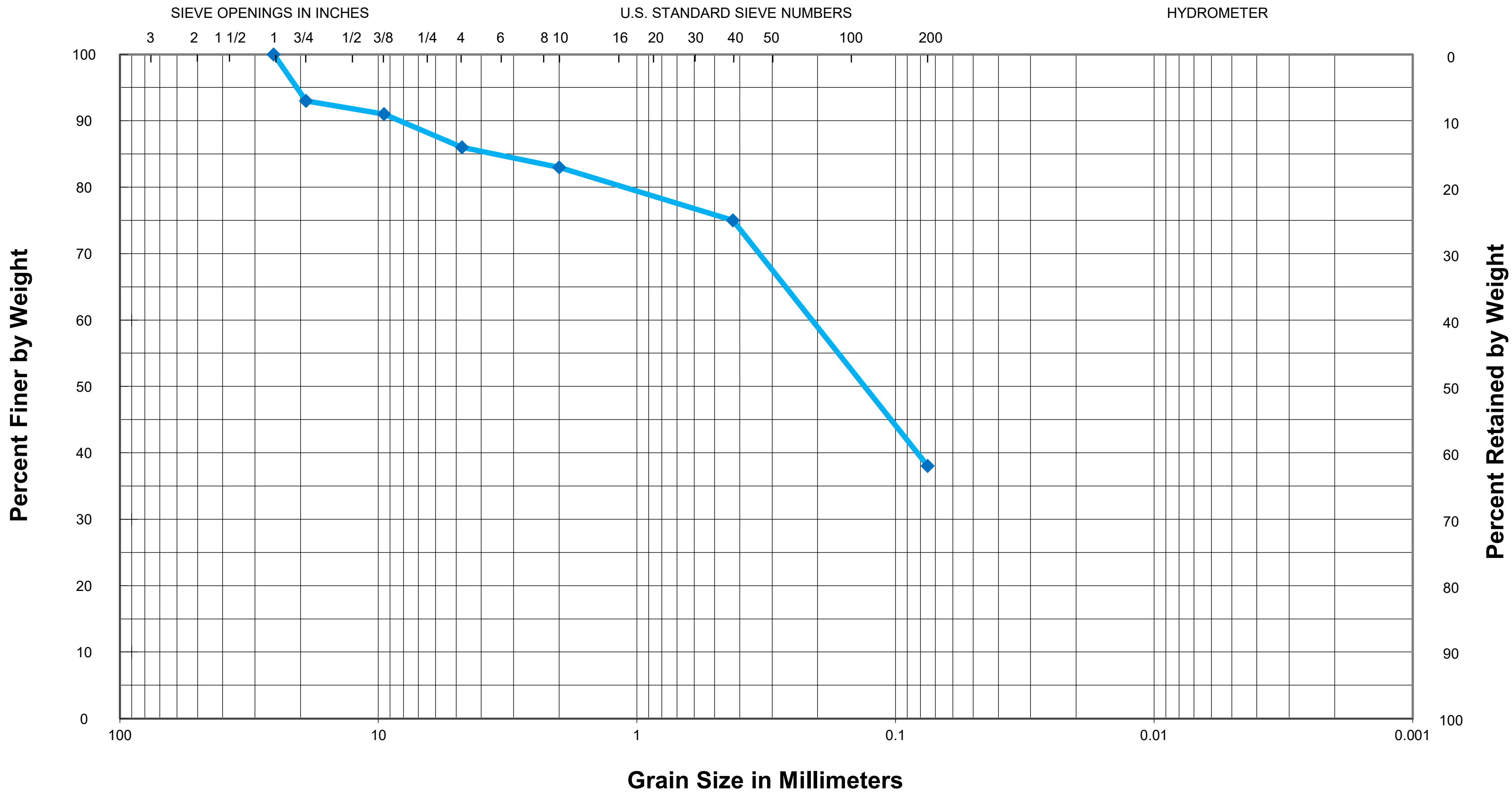
PROJECT: 050413 Hwy. 25 over Cadron Creek
LOCATION: Cleburne County, Arkansas
GHBW JOB NUMBER: 20-035

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		
S1	0.5-1.5	12	20	15	5	100	100	93	91	86	83	75	38	SM-SC	A-4
S1	4.5-5.5	14	19	16	3	---	---	---	---	89	---	---	37	SM	A-4
S1	6.5-7.5	11	---	---	---	100	100	100	100	100	99	98	19	SM	A-2-4
S2	2.5-3.5	8	---	---	---	100	75	65	43	36	29	24	10	GM-GP	A-1-a
S2	4.5-5.5	15	NON-PLASTIC			---	---	---	---	82	---	---	17	SM	A-2-4
S3	0.5-1.5	12	NON-PLASTIC			100	100	82	67	59	53	49	8	SM-SP	A-1-b
S3	4.5-5.5	20	NON-PLASTIC			100	100	100	94	83	76	70	9	SM-SP	A-3

20-035

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

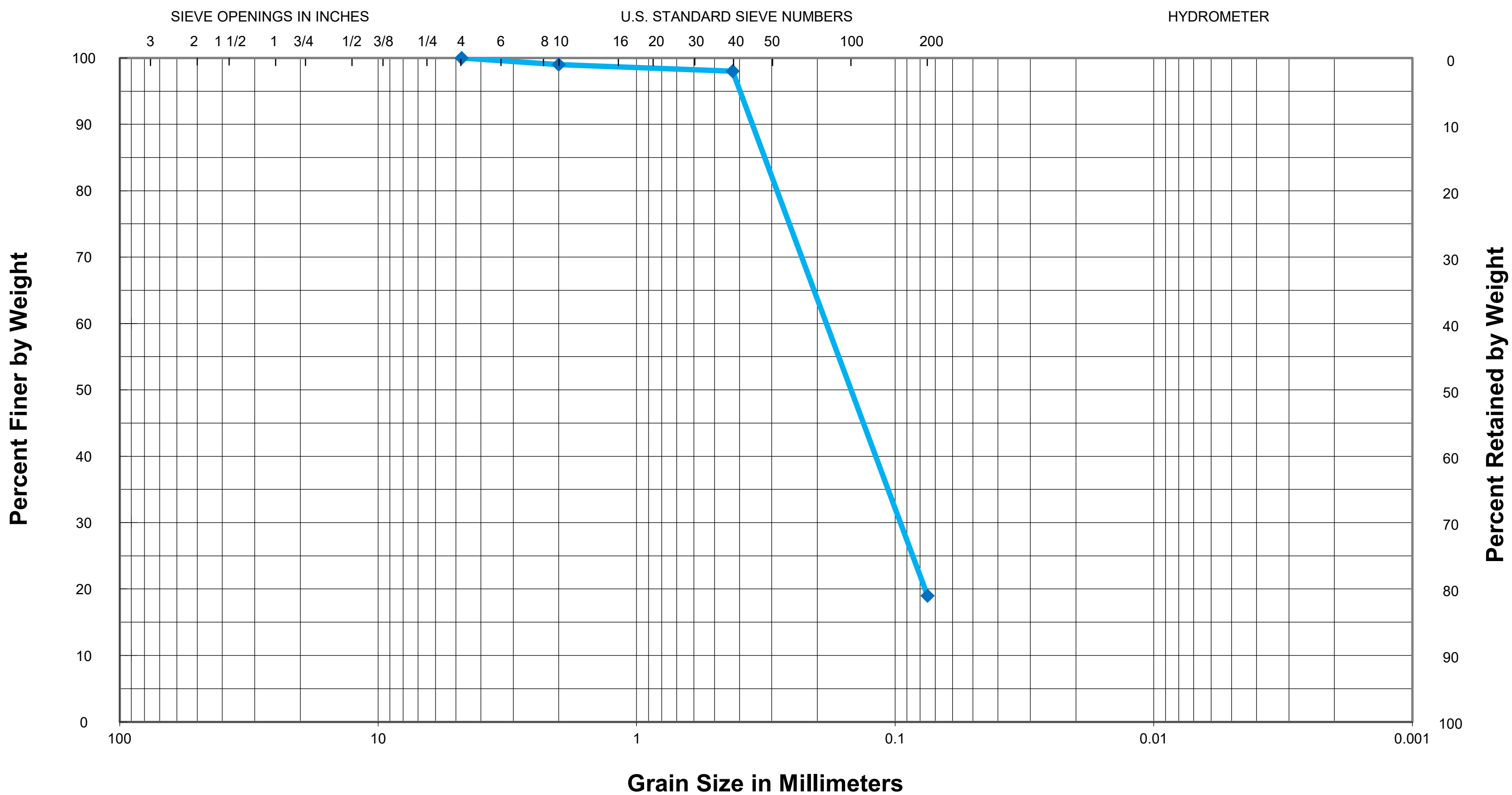
Sample: Boring S1, 0.5-1.5 ft; LL=20, PL=15, PI=5
Description: Brown and reddish brown fine sandy CLAY w/ sandstone fragments (fill)

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

20-035

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

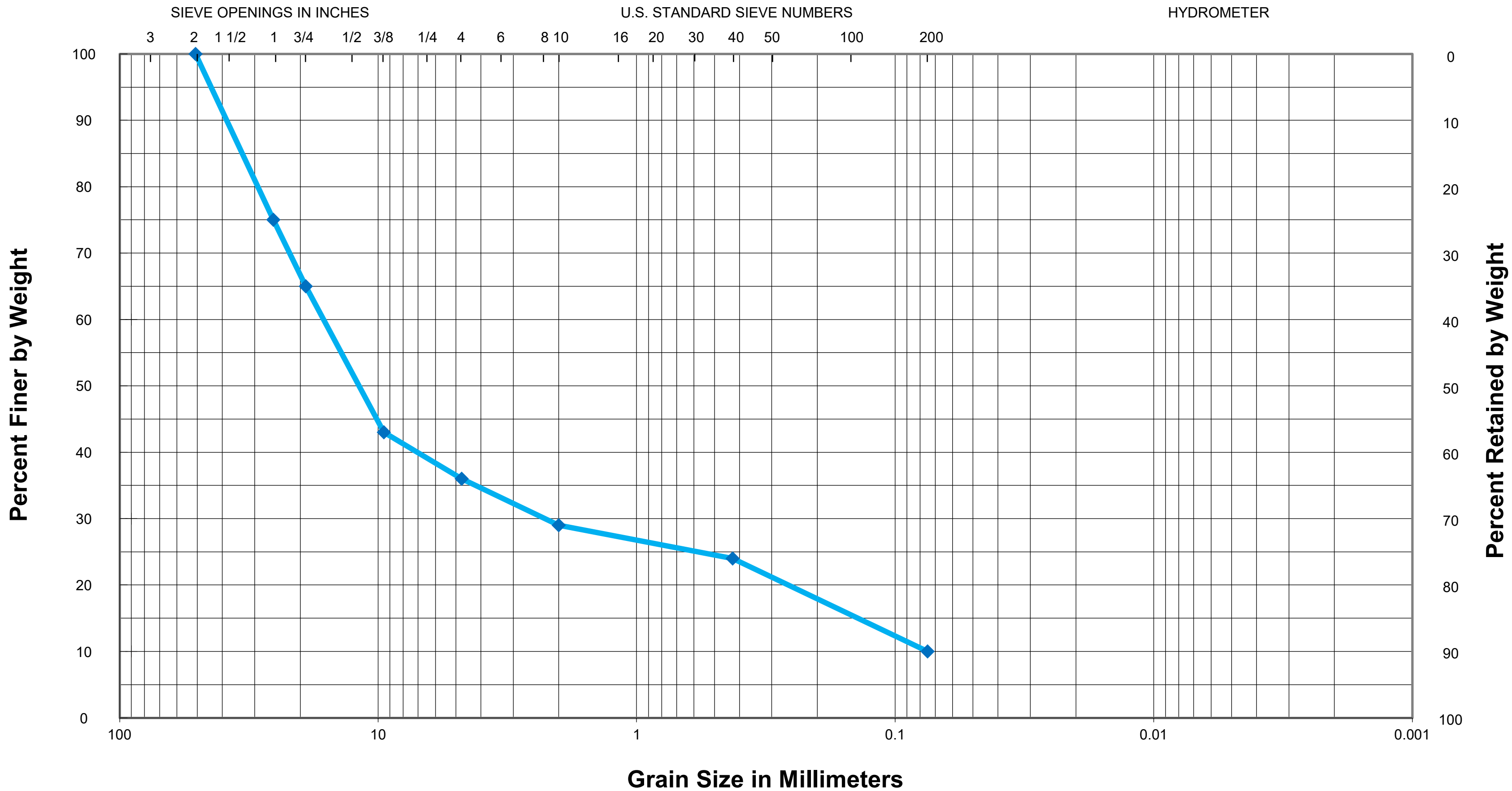
Sample: Boring S1, 6.5-7.5 ft;
Description: Brown silty fine SAND w/ trace sandstone fragments

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

20-035

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

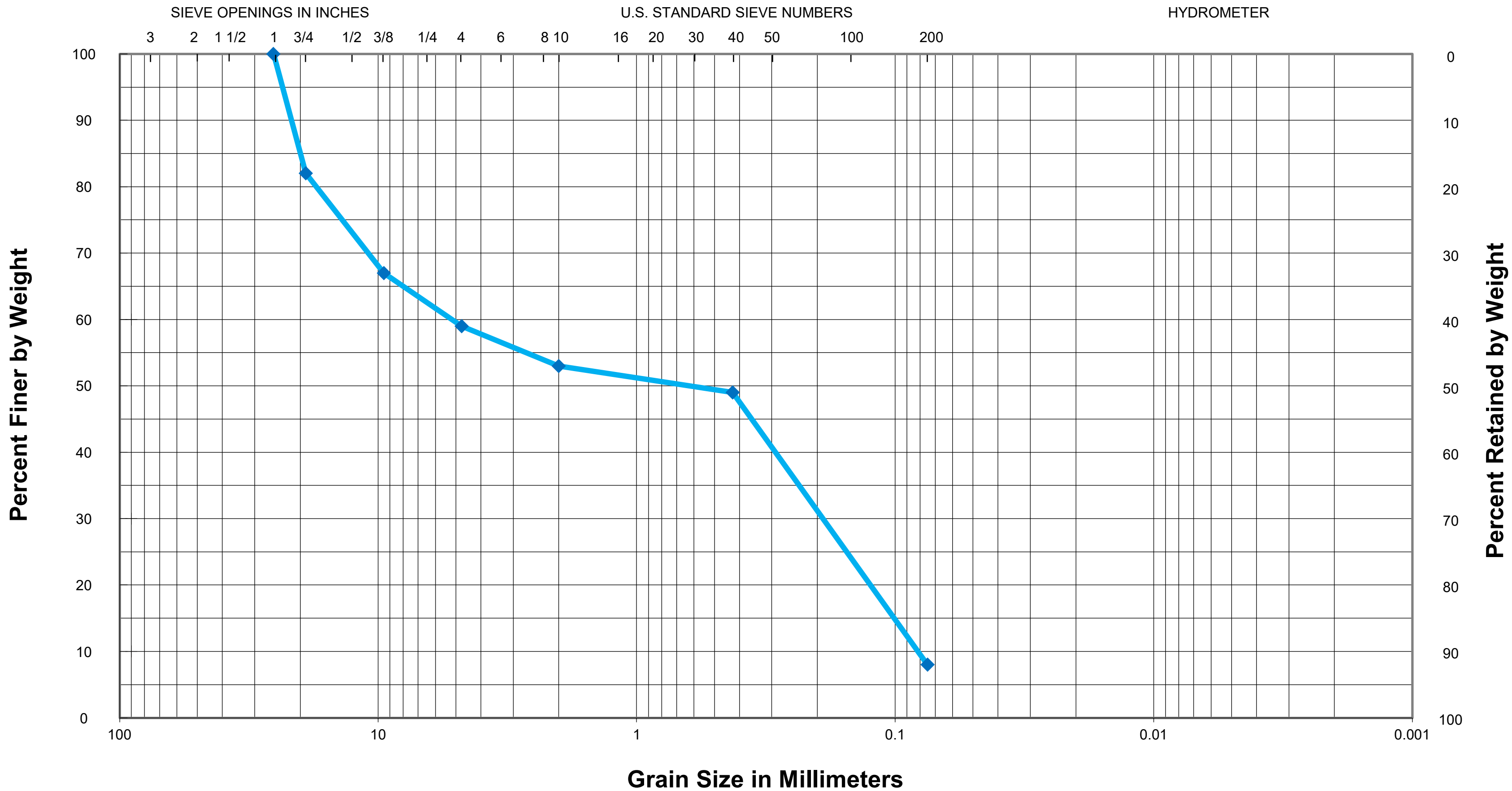
Sample: Boring S2, 2.5-3.5 ft;
Description: Brown fine SAND, slightly silty w/ numerous sandstone fragments and crushed stone (fill)

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

20-035

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

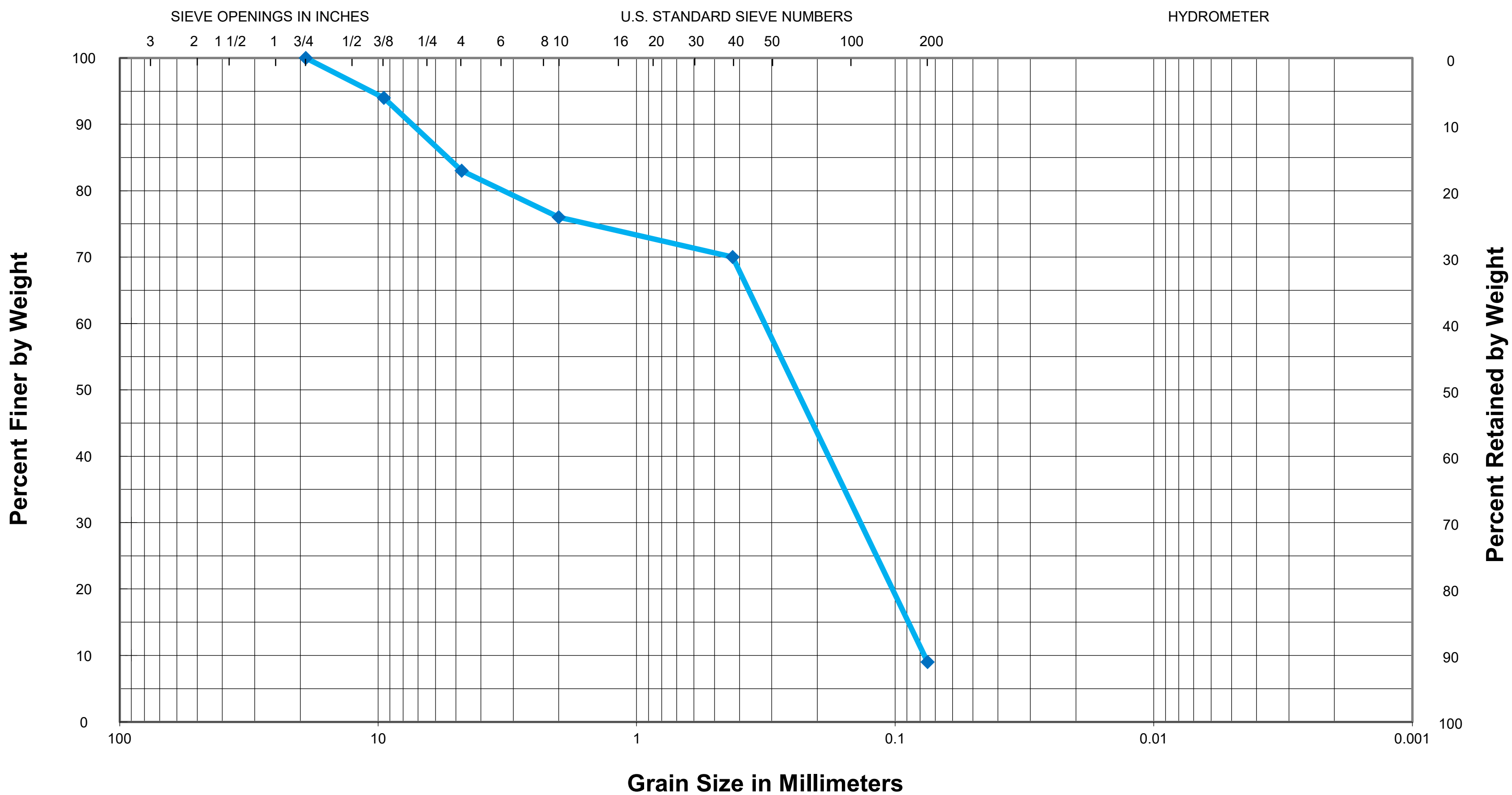
Sample: Boring S3, 2.5-3.5 ft; NON-PLASTIC
Description: Brown fine SAND, slightly silty w/ shale and sandstone fragments and occasional organics

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

20-035

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

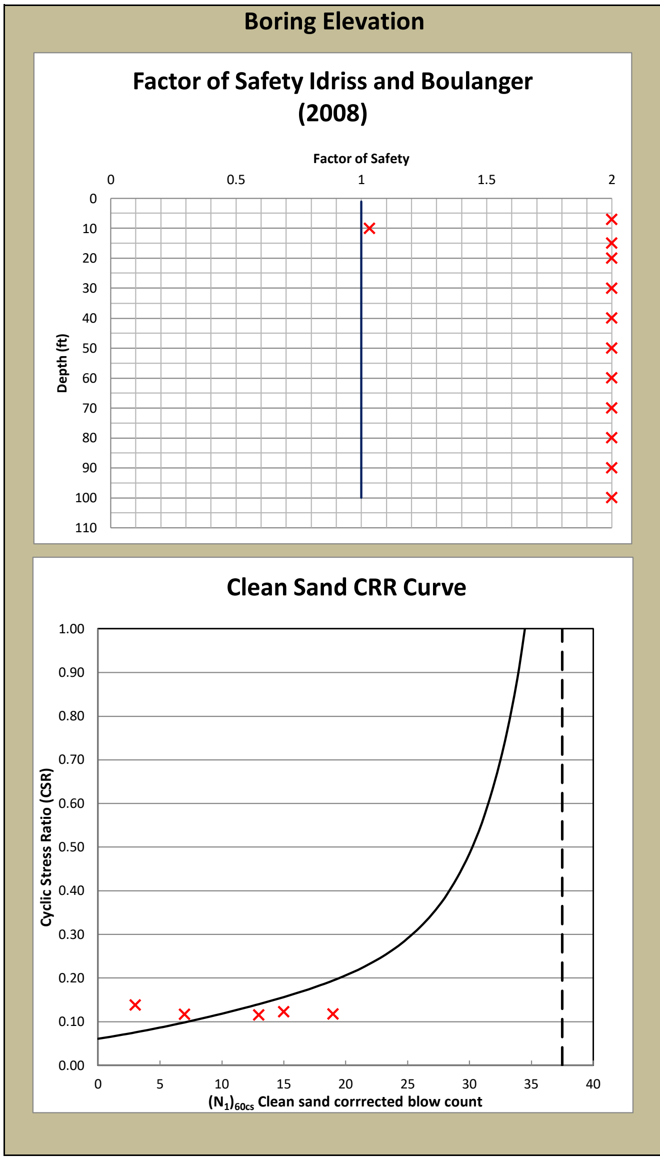
Sample: Boring S3, 4.5-5.5 ft; NON-PLASTIC
Description: Brown, gray, and tan fine SAND, slightly clayey

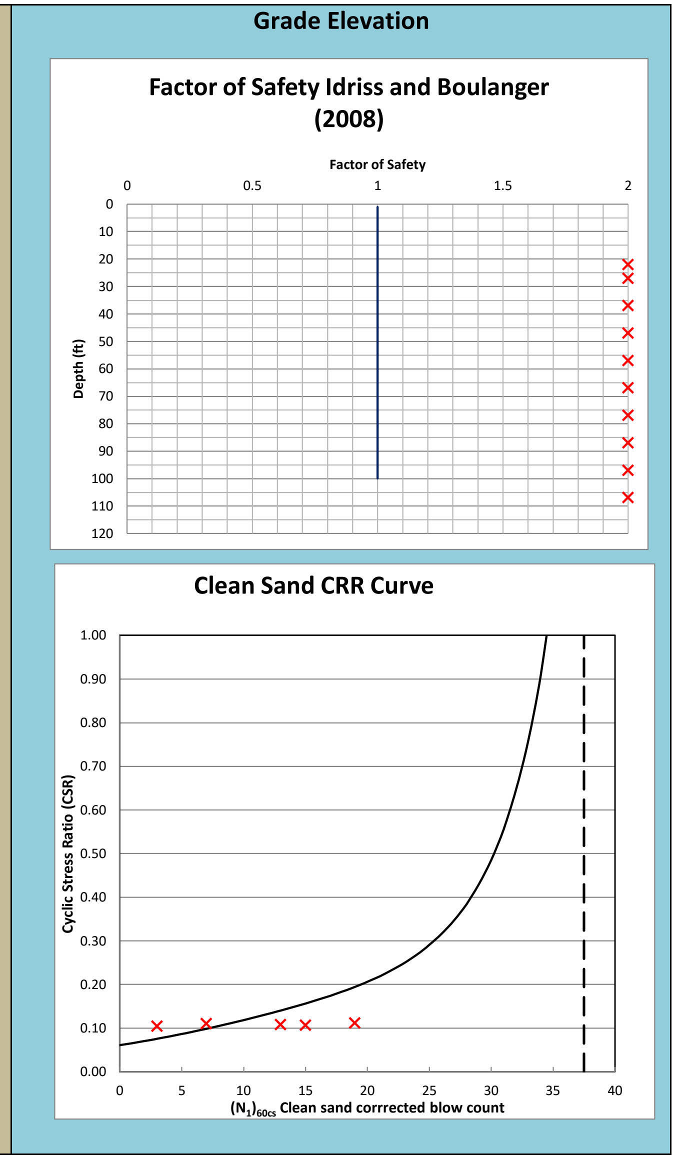
USCS Classification = SM-SP
AASHTO Classification = A-3

APPENDIX E

RESULTS of LIQUEFACTION ANALYSES

HWY. 25 OVER CADRON CREEK
ARDOT 050413 CADRON CREEK STRS. & APPRS. (S)
CLEBURNE COUNTY, ARKANSAS

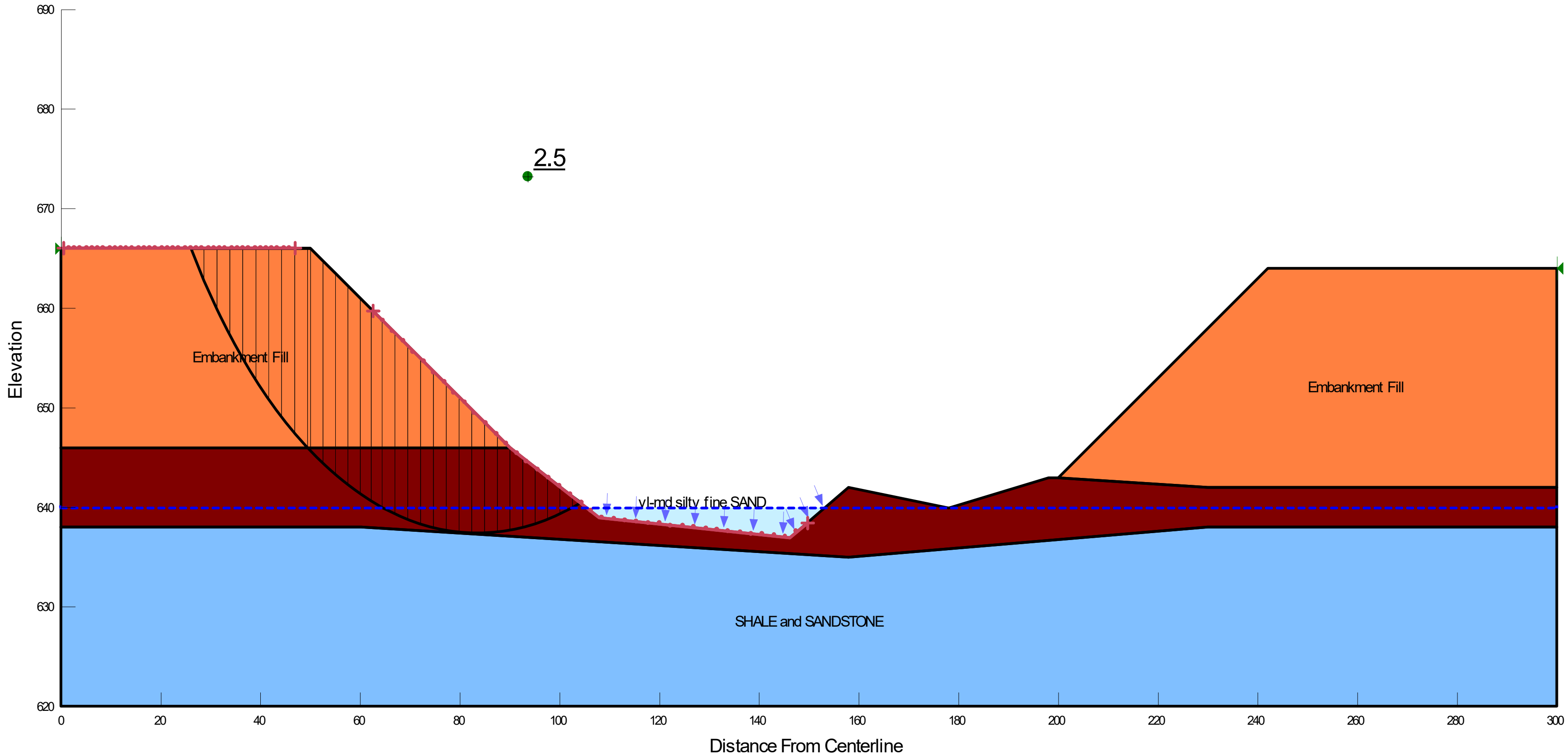




APPENDIX F

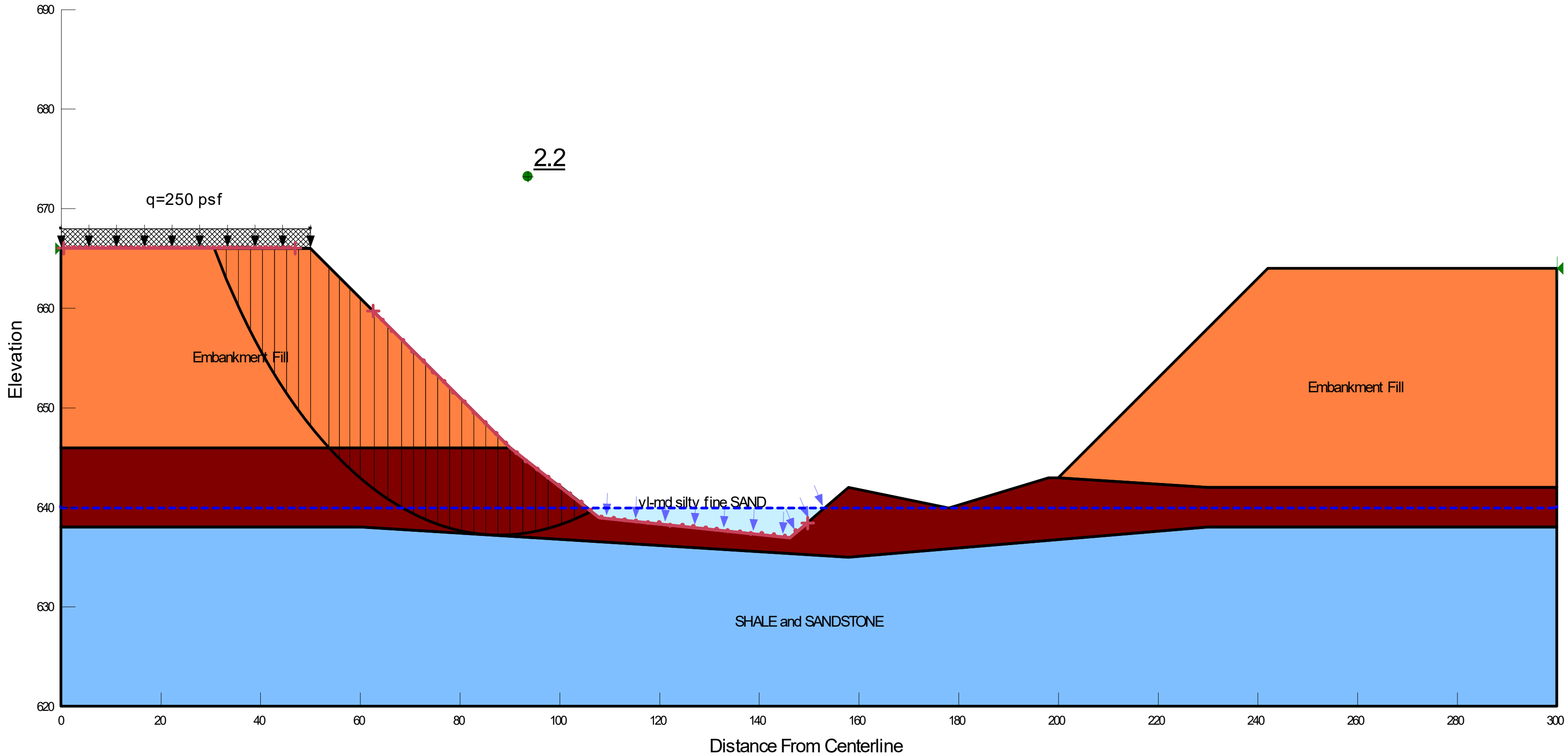
Summary of Stability Analysis Results
ARDOT Job No. 050413 Hwy. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

Bridge End	Design Loading Condition	Calculated Minimum Factor of Safety
Bent 1 End Slope (2H:1V)	End of Construction	2.5
	Long Term	2.2
	Rapid Drawdown from El 646 to El 640	2.4
	Seismic (k_h =A_S/2 = 0.085)	1.9
Bent 4 End Slope (2H:1V)	End of Construction	2.7
	Long Term	2.4
	Rapid Drawdown from El 646 to El 640	2.6
	Seismic (k_h =A_S/2 = 0.085)	2.2



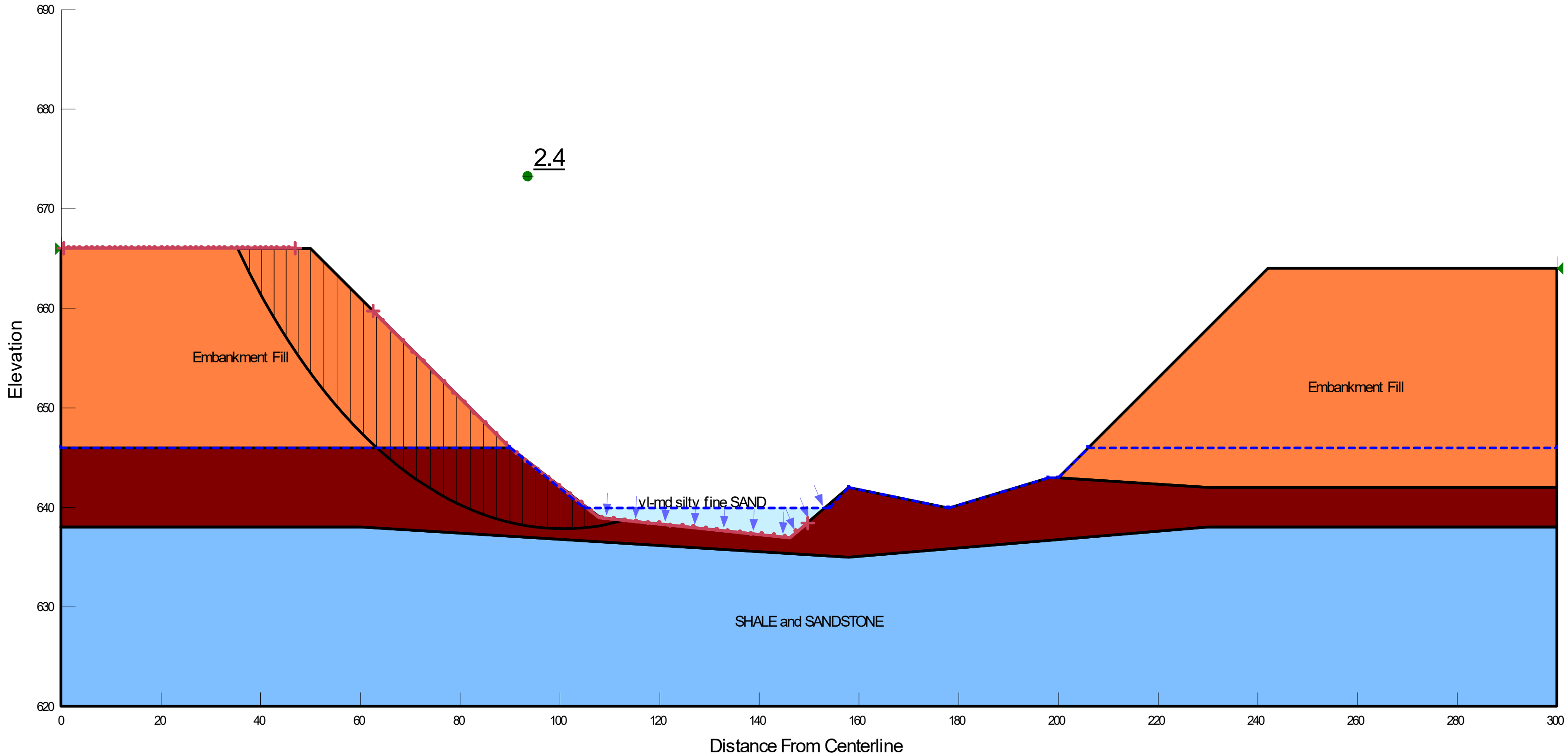
Results of Stability Analyses – End of Construction
Bent 1 End Slope - 2H:1V Slope, H=18 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

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

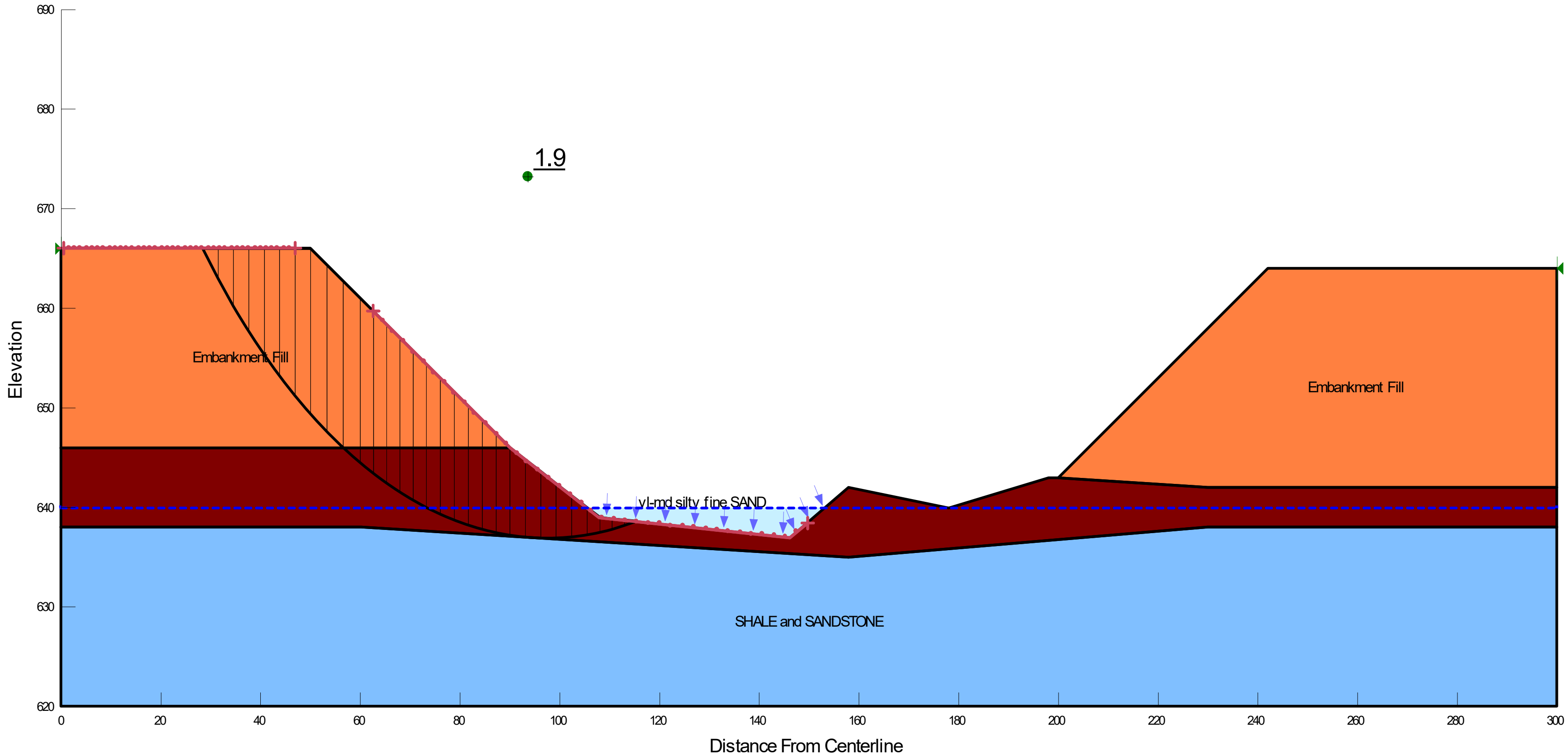


Results of Stability Analyses – Long Term Condition
Bent 1 End Slope - 2H:1V Slope, H=19 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

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

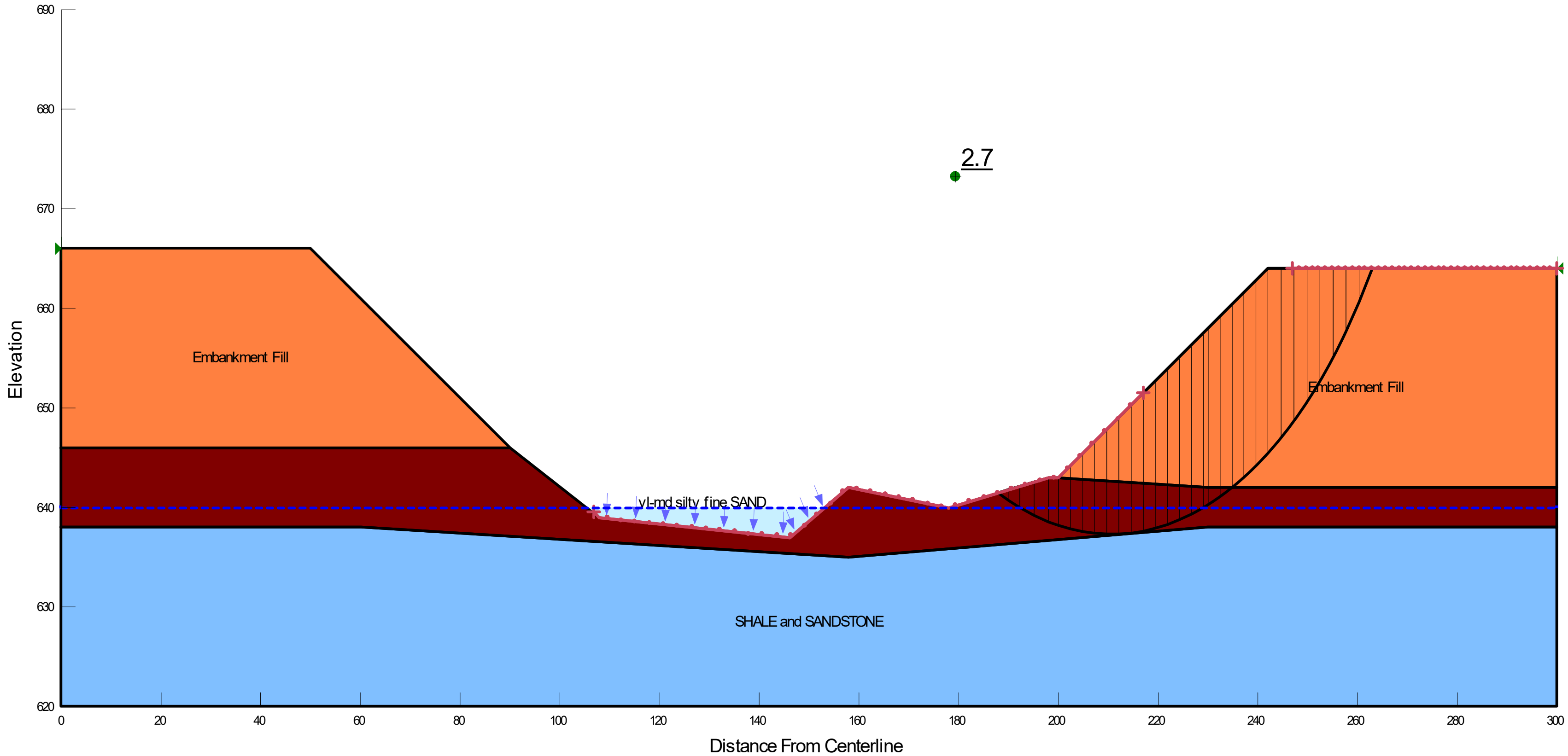


Results of Stability Analyses – Rapid Drawdown Condition, El 646 to El 640
Bent 1 End Slope - 2H:1V Slope, H=19 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

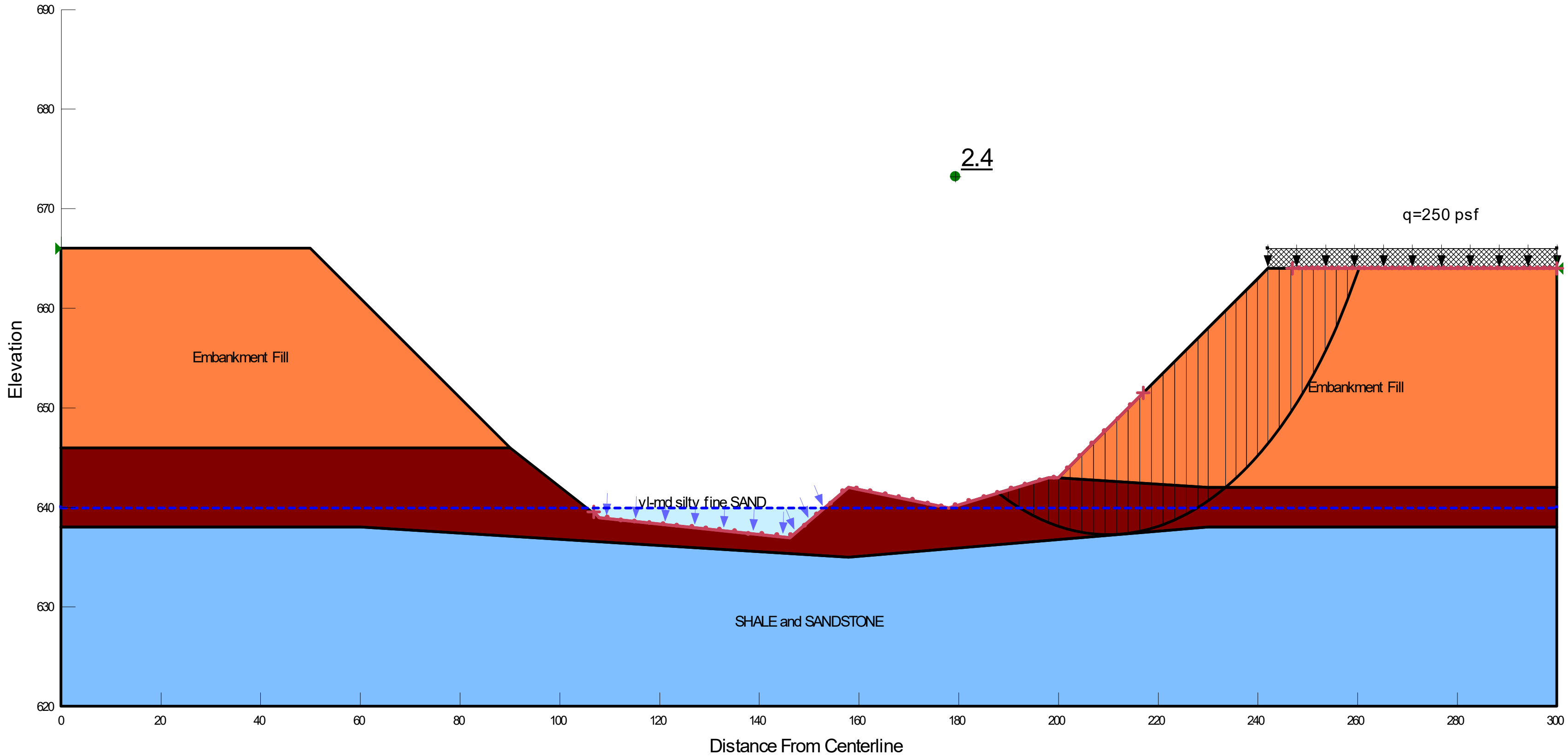


Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.085
Bent 1 End Slope - 2H:1V Slope, H=19 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

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

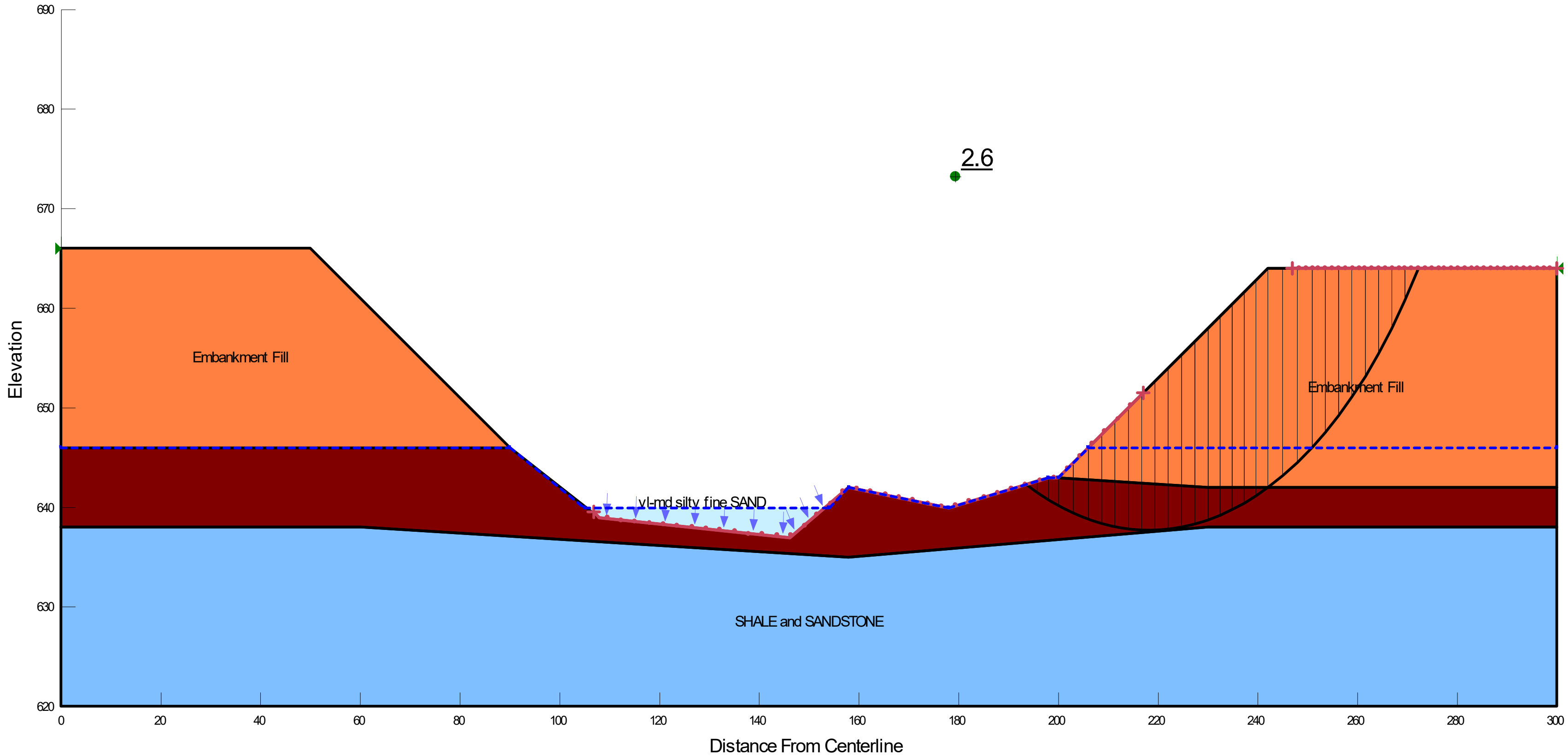


Results of Stability Analyses – End of Construction
Bent 4 End Slope - 2H:1V Slope, H=21 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

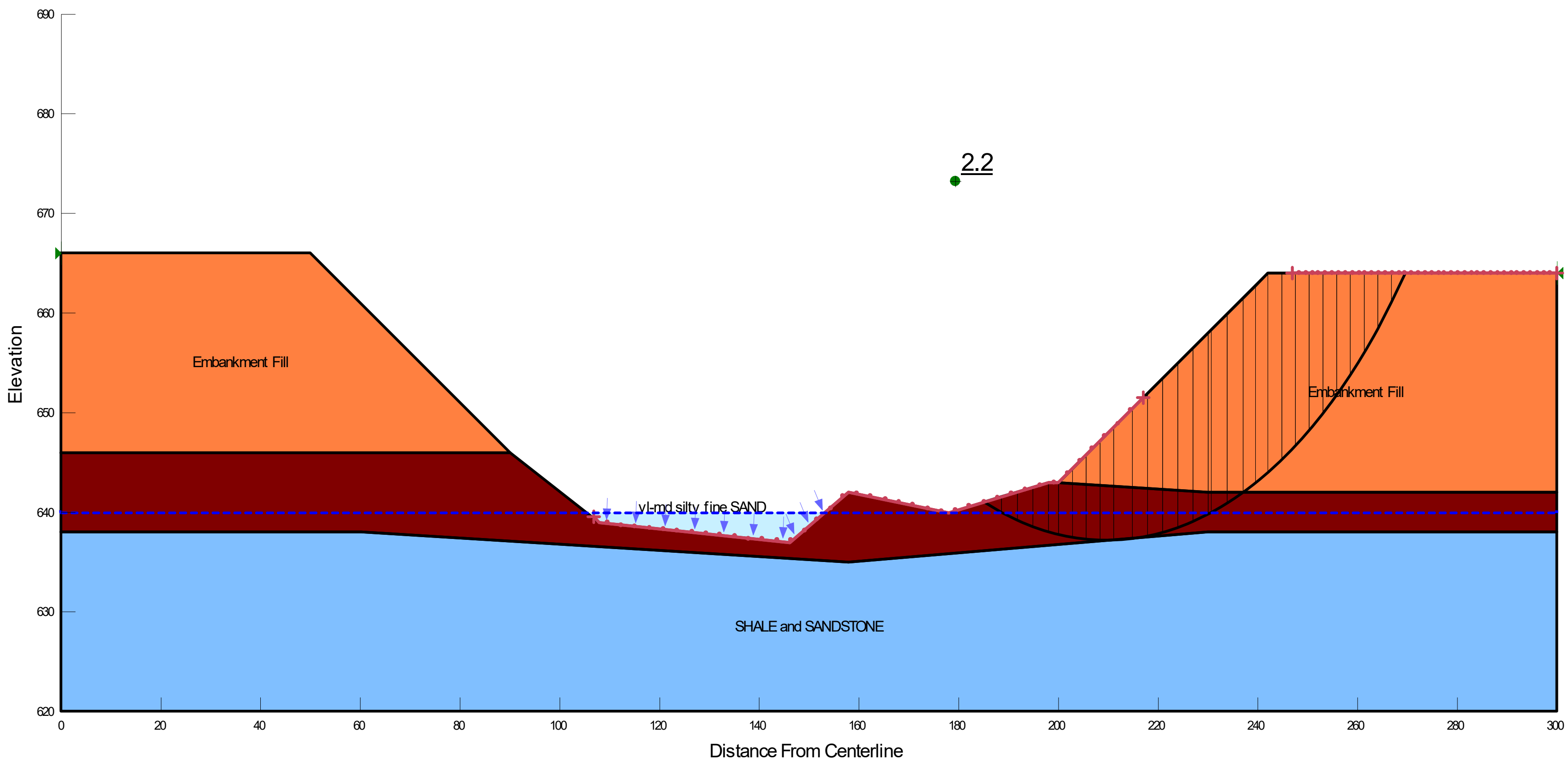


Results of Stability Analyses – Long Term Condition
Bent 4 End Slope - 2H:1V Slope, H=21 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas

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



Results of Stability Analyses – Rapid Drawdown Condition, E1 646 to El 640
Bent 4 End Slope - 2H:1V Slope, H=21 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas



Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.085)
Bent 4 End Slope - 2H:1V Slope, H=21 ft ±
ARDOT Job No. 050413 Hw. 25 over Cadron Creek
GHBW Job No. 20-035
Cleburne County, Arkansas