ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 101124

FEDERAL AID PROJECT NO. BFP-1656(5)

HWY. 135 STRS. & APPRS. (S)

STATE HIGHWAY 135 **SECTION** 1

IN POINSETT **COUNTY**

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CONSULTING ENGINEERS
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Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 13, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER DEAD TIMBER LAKE (SITE 1)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
POINSETT COUNTY, ARKANSAS

INTRODUCTION

Presented herein are the final results of the geotechnical investigation performed for the Hwy. 135 over Dead Timber Lake replacement bridge in Poinsett County, Arkansas. This bridge is Site 1 of the ARDOT 110124 Hwy. 135 Strs & Apprs (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on July 2, 2023. This revised report supersedes the previous submittal of September 10, 2023.

We understand the replacement bridge will be an integral prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 180 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed east of the existing bridge. Site grading will include about 12 ft of fill. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, and construction considerations.

The relationship of these factors to design and construction of the Site 1 replacement bridge has been considered in developing the recommendations and considerations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface conditions in the Site 1 replacement bridge alignment were explored by drilling four (4) sample borings to 110 ft each. The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Site 1 Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
A1	519+55	15 ft Lt	35.48416	-90.32248	219.1	110
A2	520+00	5 ft Lt	35.48435	-90.32248	213.4	110
A3	520+75	35 ft Lt	35.48451	-90.32249	212±	110
A4	521+50	20 ft Lt	35.48471	-90.32254	218.0	110

The boring logs, presenting descriptions of the soil strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 14. The centerline station and offset of the boring locations and ground surface elevation, as surveyed, is also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 15.

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 a truck-mounted CME-55 HTX rotary-drilling rig and a track-mounted Diedrich D-50 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the energy conversion factor is shown on each boring log.

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

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

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 46 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 10 liquid and plastic (Atterberg) limit determinations and 30 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

The laboratory testing program also included one (1) consolidation test performed in general accordance with ASTM D 2435. In this test, an undisturbed soil sample was placed in a cell, inundated with water, and incrementally loaded. The deflection was measured with time until vertical movement had essentially stopped. At that point, another load increment was applied. After the completion of all loading cycles, the load was removed incrementally and rebound was measured. The consolidation test results are presented graphically in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The Site 1 location is on Hwy. 135 where the Dead Timber Lake drainage channel crosses the highway approximately 480 ft north of Howard Road in Poinsett County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. Dead Timber Lake is located just east of the bridge site. The drainage channel at the bridge is broad with shallow to steep banks. The area around the bridge is low-lying and swampy, with standing water, thick underbrush, and numerous trees. The project locale is primarily agricultural land consisting of open flat fields. Grain storage bins are located southeast of the proposed bridge. The existing two-lane roadway is on embankment, and the existing pavements are in poor condition. Surface drainage along the roadway is poor and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent Alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends through exposures of Quaternary Terrace Deposits. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

Seismic Conditions

In light of the results of the borings and the surface geology, a Seismic Site Class D (stiff soil profile) is considered applicable to the bridge location at Site 1 with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[2]. Given the location and AASHTO code-based values, recommended seismic parameters are summarized below.

- Seismic Site Class D
- 1.0-sec period spectral acceleration coefficient (S_1) = 0.442
- Site amplification factor at 1.0 second (F_v) = 1.558
- 1.0-sec period spectral acceleration coefficient (S_{D1}) = 0.689
- Acceleration for a short (0.2 sec) period (S_S) = 1.689
- Site amplification factor for short period (F_a) = 1.0
- Peak ground acceleration (PGA) = 0.954
- Site amplification factor at PGA (F_{PGA}) = 1.0
- A_s = 0.954

Utilizing these parameters, AASHTO LRFD Seismic Bridge Design Specifications indicate that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Site 3 location of the Hwy. 135 bridge over Dead Timber Lake.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 0.954 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Appendix D as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

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

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

analyses results are shown on the generalized subsurface profile also provided in Appendix D. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix D.

Subsurface Conditions

Based on the results of the borings, the surface soils to 4- to 6-ft depth are locally comprised of soft to firm brown clay and fine sandy clay fill (see Borings A1 and A4). The fill contains fine gravel, crushed stone, and asphalt fragments. The fill has poor compaction and exhibits low shear strength and high compressibility. The fill typically classifies as A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with very poor subgrade support for pavement structures.

Below the fill or at the surface is natural soft to stiff gray, brown, tan, and reddish tan clay extending to 23 to 38 ft below existing grades. The clay has a blocky structure at depth and contains ferrous stains and nodules, calcareous nodules, decayed organics, and occasional silty sand and clayey silt seams and layers. The clay exhibits low shear strength, moderate to high plasticity, and high to low compressibility. The shear strength increases, and compressibility decreases below 13- to 23-ft. The clay typically classifies as A-6, A-7-5, and A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to very poor subgrade support for pavement structures.

The clayey soil units are underlain below 23 to 38 ft by medium dense to dense brown, gray, dark gray, grayish brown, grayish tan, and brownish gray silty fine sand and fine to medium sand units. Some coarse sand and fine gravel are present at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was encountered in the borings at 23.7- to 29.2-ft depth in June 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the ditch and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 1 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 16-in.-diameter steel shell piles are planned for bridge ends and 24-in.-diameter steel shell piles are planned for the interior bents. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix E. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for "static" conditions prior to a seismic event, with no liquefaction, and full soil shear strength is mobilized for the foundation soils. For the case where liquefaction occurs, the "end of earthquake" condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects. The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix F.

End Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 4) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 12 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a 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 0.477. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 215 to El 213 was assumed.

Stability analyses results are summarized and presented graphically in Appendix G. As shown in the results, the analyses of the seismic stability of the plan 2H:1V Bent 1 end slope

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[4] Slope/W 2020; GEO-SLOPE International; 2020.

indicates a calculated minimum factor of safety significantly less than 1.05. However, a simplified Newmark block analysis indicates that a maximum permanent displacement of 2.1 inches is expected for the south embankment. We understand that a Newmark displacement of less than 6 inches is typically acceptable for bridges designated as "Other."

The results of slope stability analyses utilizing residual strengths in soil zones susceptible to liquefaction triggering indicate a calculated minimum factor of safety against sliding in excess of 1.0. Consequently, the potential for flow slide instability is considered low. Given the results of the stability analyses and Newmark block analysis, the stabilities of the slope configurations are considered acceptable.

Subgrade Support

It is understood that pavement sections for the approach roads will be developed by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-3 and A-4. These classifications correlate with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, undercuts or improvement depths on the order of 2 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. for cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, undercutting is expected to be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 13 to 23 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix H. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

Maximum embankment heights on the order of 12 ft are anticipated. Given the predominance of cohesive soils in the embankment foundations, some consolidation settlement will occur. Based on the results of the borings and the anticipated maximum embankment height, total settlement of the natural foundation soils below the embankments is estimated to be on the order of 2 to 3 inches. Settlement of cohesive fill in the embankments is expected to be on the order of 1 to 2 in. with 40 to 60 percent of the settlement occurring during construction. We recommend that embankment fill be placed as early in the construction sequence as possible to limit post-construction settlement after foundation construction.

General 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow within about 100 ft of the bridge ends. An example special provision for cohesive embankment fill is provided in Appendix I.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until embankments and bridge work are completed.

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

Groundwater was encountered between 23- to 29-ft in June 2023. Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered. Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[5]. In the driveability analyses, the steel shell piles were assumed to be driven from the plan cap bottom elevation or existing grade. Graphical and tabulated results of these analyses are provided in Appendix J.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 91 ft-kips per blow for driving the steel shell piles at the end bents and at interior Bent 2. For intermediate Bent 3, we recommend a hammer system capable of delivering at least 125 ft-kips per blow for driving the steel shell piles. 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 installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. Driving records should be

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[5] GRLWEAP 2014; Pile Dynamics, Inc.

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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment 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 Map
Plate 2	Plan of Borings
Plates 3 through 14	Boring Logs
Plate 15	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Liquefaction Analysis Results
Appendix E	Nominal Pile Capacity Curves
Appendix F	Lateral Load Parameters
Appendix G	Results of Stability Analyses
Appendix H	Example SP – Woven Geotextile
Appendix I	Example SP – Cohesive Embankment Fill Special Provision
Appendix J	Driveability Analysis Results

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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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

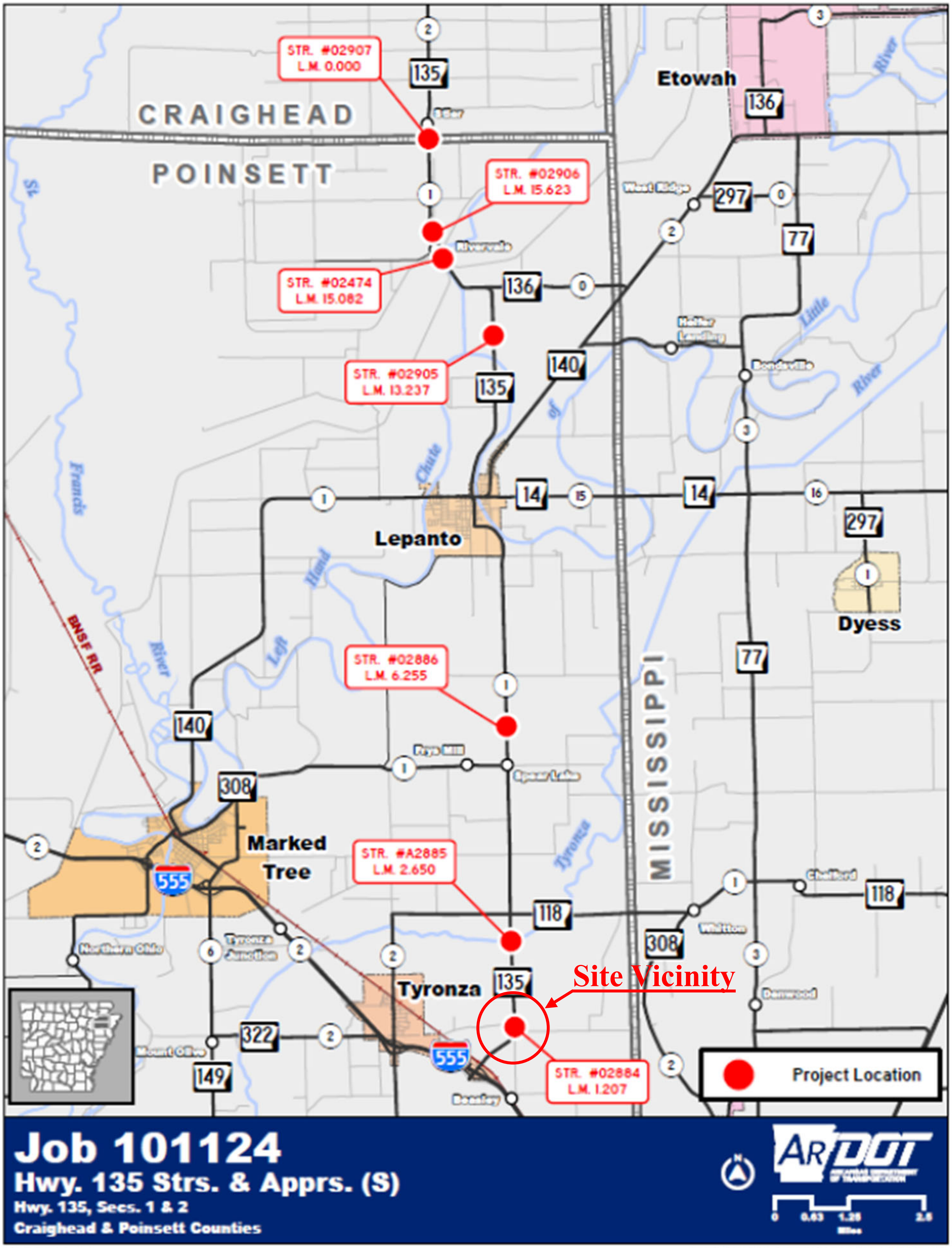
M E Wyatt

Mark E. Wyatt, P.E.
President

No. 7791
MARK E. WYATT

VMS/MEW:jw

Copies submitted:	Arkansas Department of Transportation		
	Attn:	Ms. Jessica Jackson, P.E.	(1-email)
	Attn:	Mr. Paul Tierney	(1-email)
	Attn:	Mr. Yongsheng Zhao, Ph.D., P.E.	(1-email)
	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

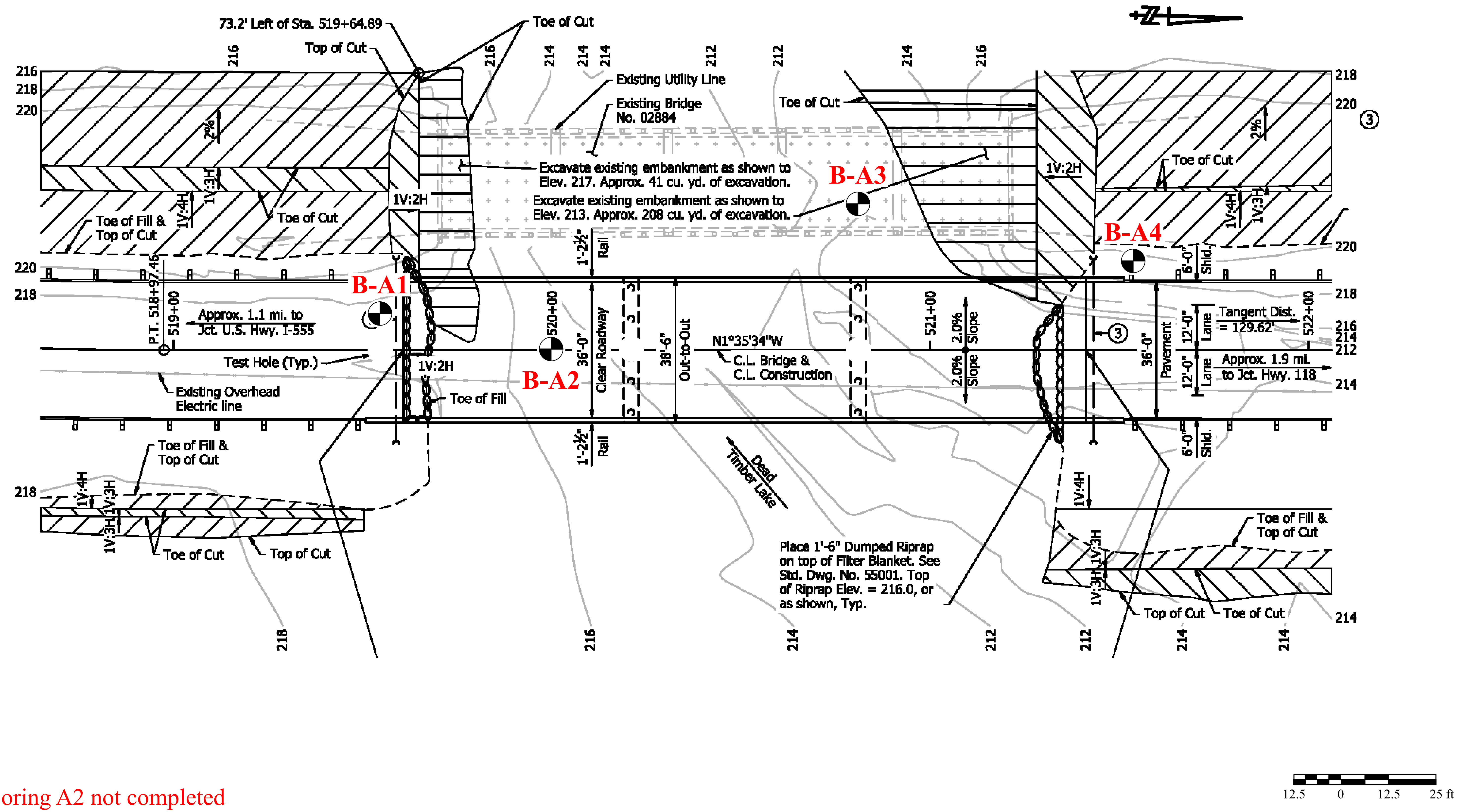


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Dead Timber Lake (Site 1/Bridge A)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Note: Boring A2 not completed

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORINGS
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Scale: As shown

Job No. 23-031

PLATE 2

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

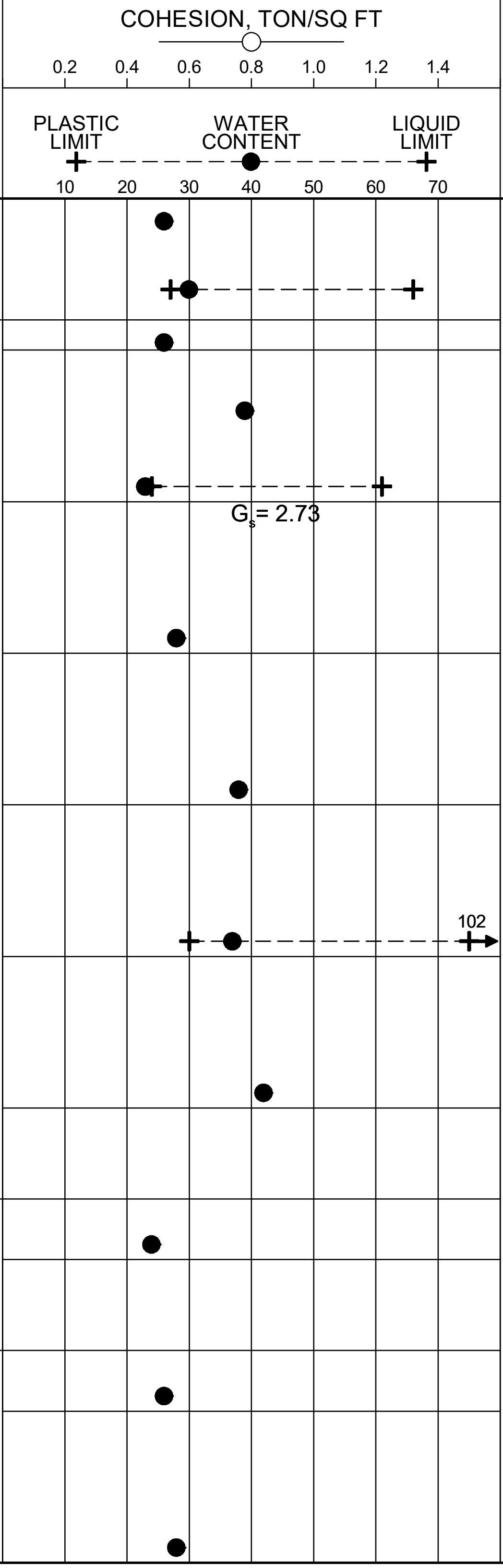
LOG OF BORING NO. A1

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 519+55, 15 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 219.1	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown clay, slightly sandy (CH) w/silty clay seams, trace fine gravel and occasional organics (fill)	9		
		11		96
5	Soft gray, tan and reddish tan clay (CH) w/ferrous stains and occasional decayed organics	7		
		9		
	- firm at 8 to 18 ft			
10		13		99
15		10		
20		13		
	- stiff, slightly blocky below 23 ft			
25		19		91
30		24		
35	Dense brownish gray silty fine sand (SM)	49		17
40	Dense grayish brown fine to medium sand, slightly silty (SM-SP)	70		6
		79		6



COMPLETION DEPTH: 110.0 ft
DATE: 6-2-23

DEPTH TO WATER
IN BORING: 29.2 ft

DATE: 6/2/2023

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

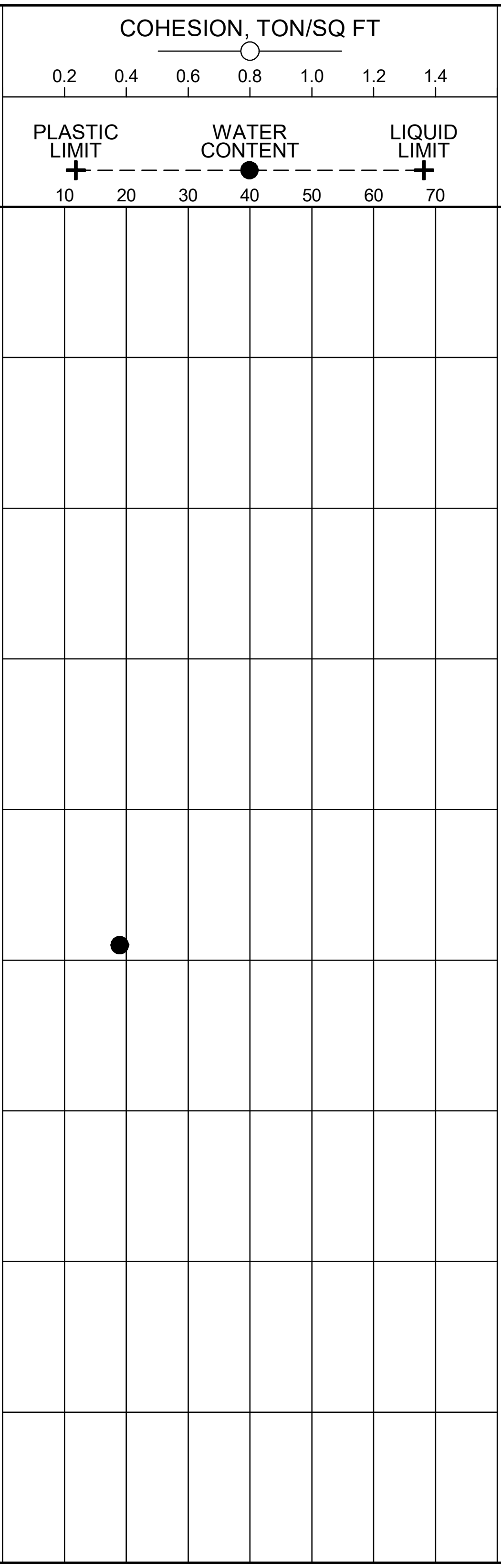
LOG OF BORING NO. A1

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 519+55, 15 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒	- with organic inclusions below 48 ft	63		
55		☒		64		
60		☒		49		
65		☒	- with more medium sand below 63 ft	57		
70		☒		53		5
75		☒		61		
80		☒		54		
85		☒		51		
		☒		53		



COMPLETION DEPTH: 110.0 ft
DATE: 6-2-23

DEPTH TO WATER
IN BORING: 29.2 ft

DATE: 6/2/2023

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

LOG OF BORING NO. A1

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 519+55, 15 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N60, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT +	- No. 200 %
95		☒		56			
100		☒		67			
105–110		☒	Dense to very dense tan fine to medium sand, slightly silty (SM-SP) w/trace coarse sand and fine to coarse gravel	107		●	10

NOTE: Drilled with Diedrich D-50 ECF= 1.43.

COMPLETION DEPTH: 110.0 ft
DATE: 6-2-23

DEPTH TO WATER IN BORING: 29.2 ft
DATE: 6/2/2023

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

LOG OF BORING NO. A2

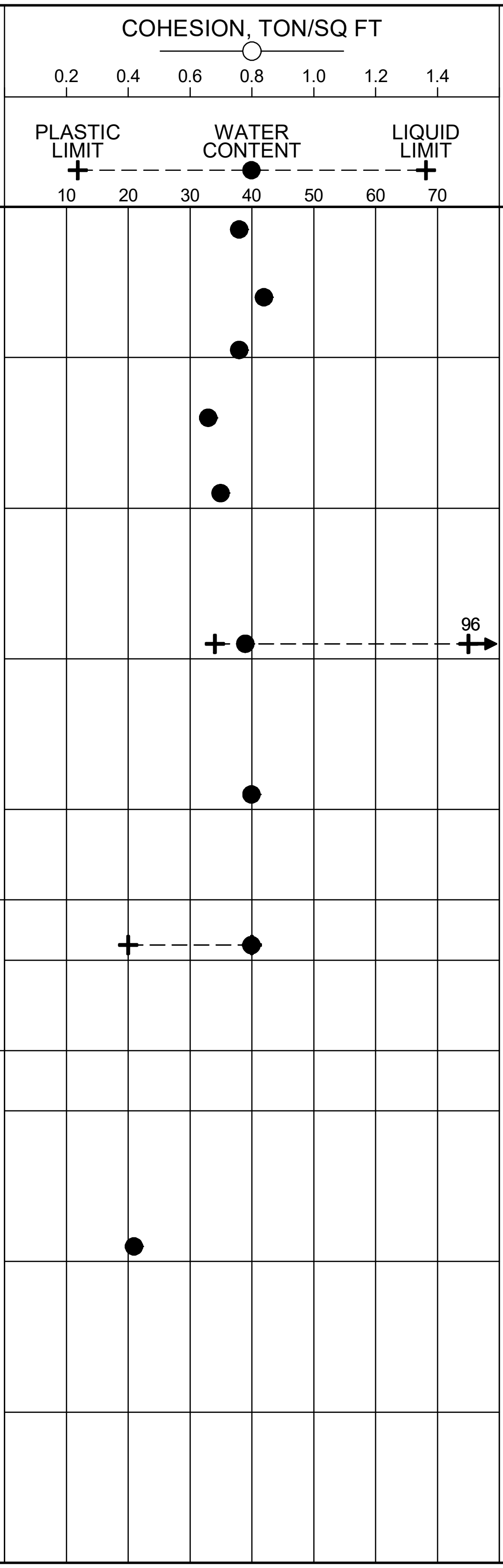
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 520+00, 5 ft Lt

SURF. EL: 213.4

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown and gray clay (CH) w/occasional decayed organics	9		
		7		
5		7		
		9		
10	- firm at 8 to 18 ft - with ferrous nodules and stains and occasional calcareous nodules below 8 ft	11		
	- slightly blocky below 13 ft			
15		11		96
	- firm to stiff below 18 ft			
20		14		
	Stiff gray silty clay, slightly sandy, wet (CL)			
25		19		87
	Medium dense gray fine sand, slightly silty (SM-SP)			
30		24		
	- dense, grayish tan below 33 ft			
35		54		5
40		60		
		54		



COMPLETION DEPTH: 110.0 ft
DATE: 6-26-23

DEPTH TO WATER
IN BORING: 23.7 ft

DATE: 6/26/2023

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

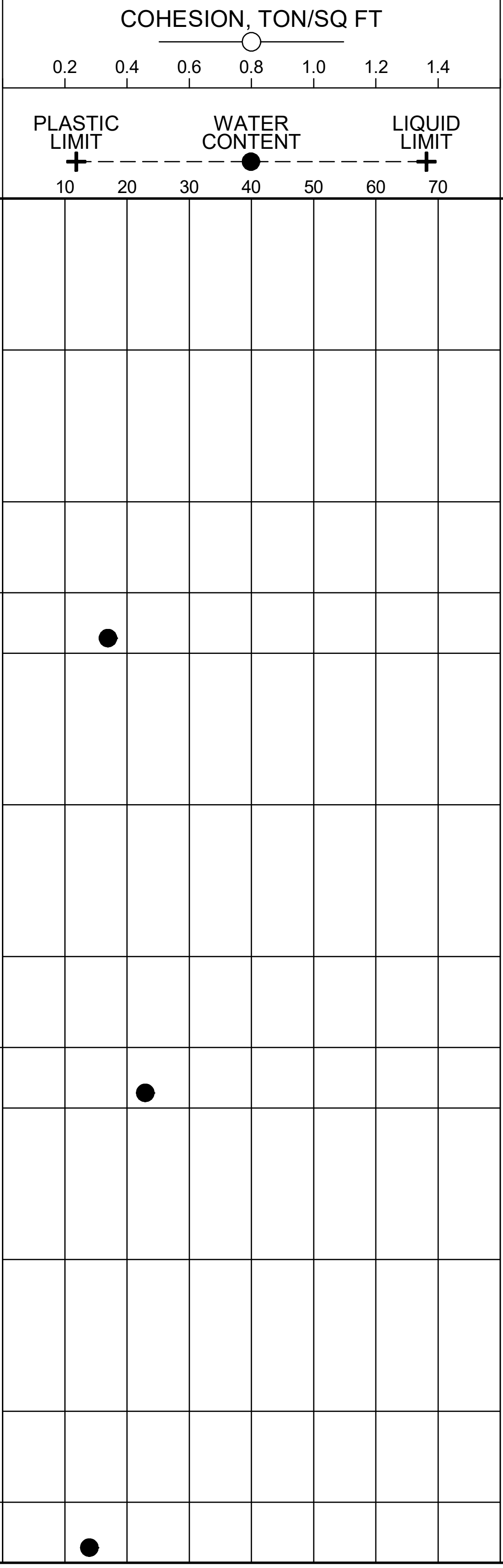
LOG OF BORING NO. A2

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 520+00, 5 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	- with occasional organic inclusions below 48 ft	49		
55		50		
60	Dense grayish tan fine to medium sand, slightly silty (SM-SW)	46		5
65		60		
70	- with organic inclusions below 68 ft	53		
75	Dense grayish tan fine sand, slightly silty (SM-SP) w/organic inclusions	47		8
80		49		
85		54		
	Dense grayish tan fine to coarse sand, slightly silty (SM-SW) w/trace	50		5



COMPLETION DEPTH: 110.0 ft
DATE: 6-26-23

DEPTH TO WATER
IN BORING: 23.7 ft

DATE: 6/26/2023

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

LOG OF BORING NO. A2

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 520+00, 5 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT	WATER CONTENT	- No. 200 %
	▨		fine gravel					
95	▨	☒		50				
	▨		Dense grayish tan fine sand, slightly silty (SM-SP)					
100	▨	☒		57			20	7
105	▨							
110	▨	☒		67				
			NOTE: Drilled with Diedrich D-50 ECF= 1.43					
115								
120								
125								
130								

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

COMPLETION DEPTH: 110.0 ft
DATE: 6-26-23

DEPTH TO WATER
IN BORING: 23.7 ft

DATE: 6/26/2023

23-031

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

LOG OF BORING NO. A3

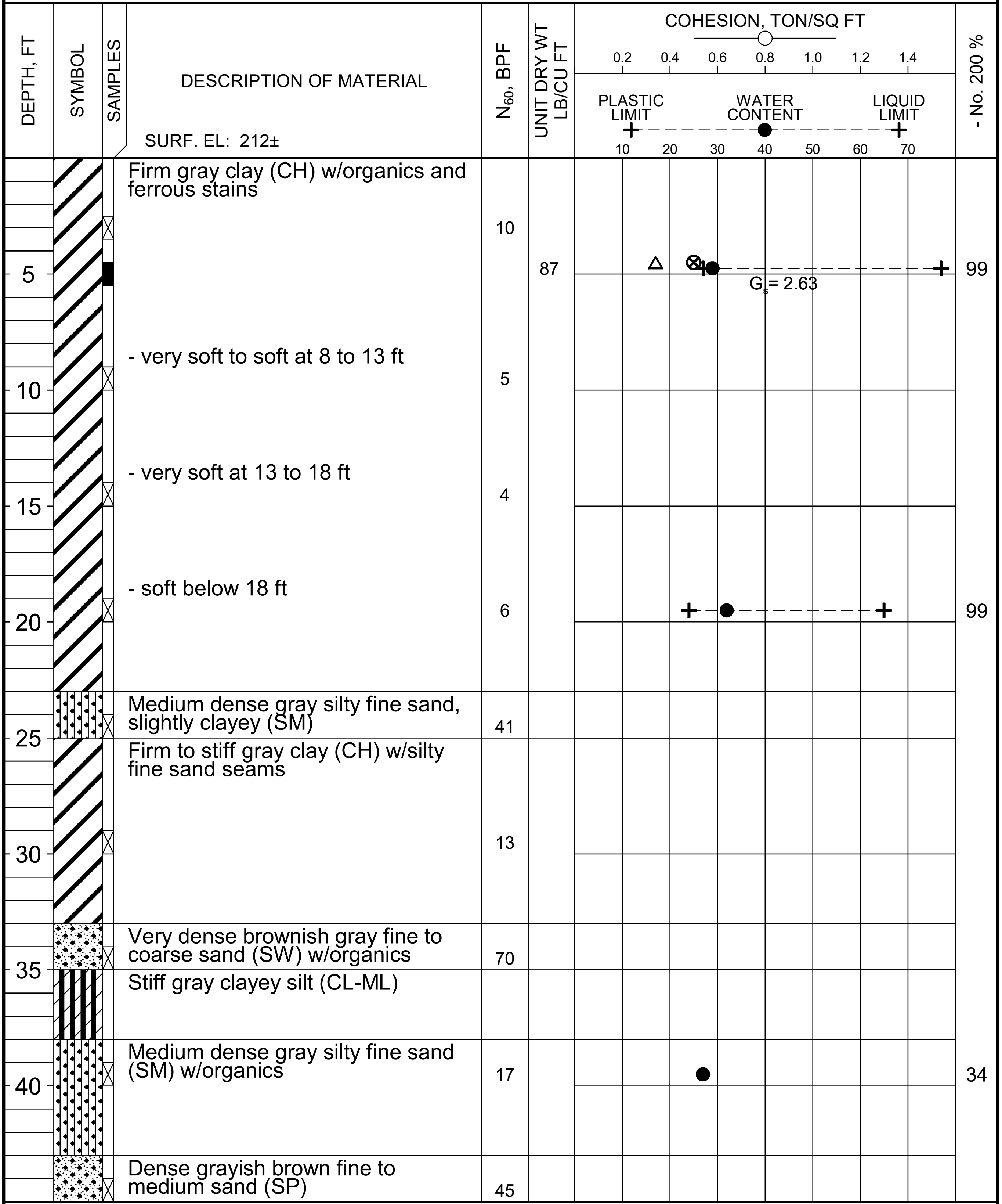
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Wash

LOCATION: Appros Sta 520+75, 35 ft Lt

SURF. EL: 212±

DESCRIPTION OF MATERIAL	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Firm gray clay (CH) w/organics and ferrous stains	10		
		87	99
- very soft to soft at 8 to 13 ft	5		
- very soft at 13 to 18 ft	4		
- soft below 18 ft	6		99
Medium dense gray silty fine sand, slightly clayey (SM)	41		
Firm to stiff gray clay (CH) w/silty fine sand seams	13		
Very dense brownish gray fine to coarse sand (SW) w/organics	70		
Stiff gray clayey silt (CL-ML)			
Medium dense gray silty fine sand (SM) w/organics	17		34
Dense grayish brown fine to medium sand (SP)	45		



COMPLETION DEPTH: 110.0 ft
DATE: 6-14-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/14/2023

LGBNEW 23-031 BRIDGE A.GPJ 7-28-23

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

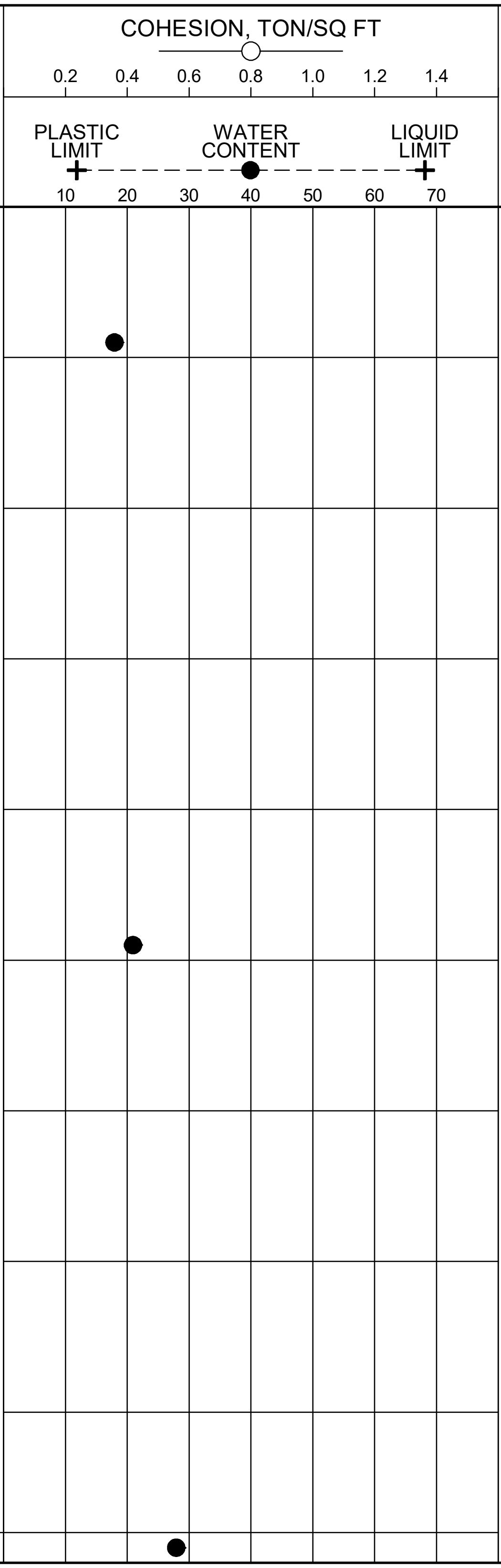
LOG OF BORING NO. A3

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Wash

LOCATION: Appros Sta 520+75, 35 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒		47		4
55		☒		55		
60		☒	- medium dense with trace fine gravel from 58 to 63 ft	33		
65		☒	- dense, brown and dark gray with trace fine gravel from 63 to 68 ft	47		
70		☒	- slightly silty (SM-SP) below 68 ft - medium dense from 68 to 73 ft	36		6
75		☒	- dense below 73 ft	97		
80		☒		61		
85		☒	- with trace coarse sand below 84 ft	63		
		☒	Dense brown and dark gray fine to	54		9



COMPLETION DEPTH: 110.0 ft
DATE: 6-14-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/14/2023

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

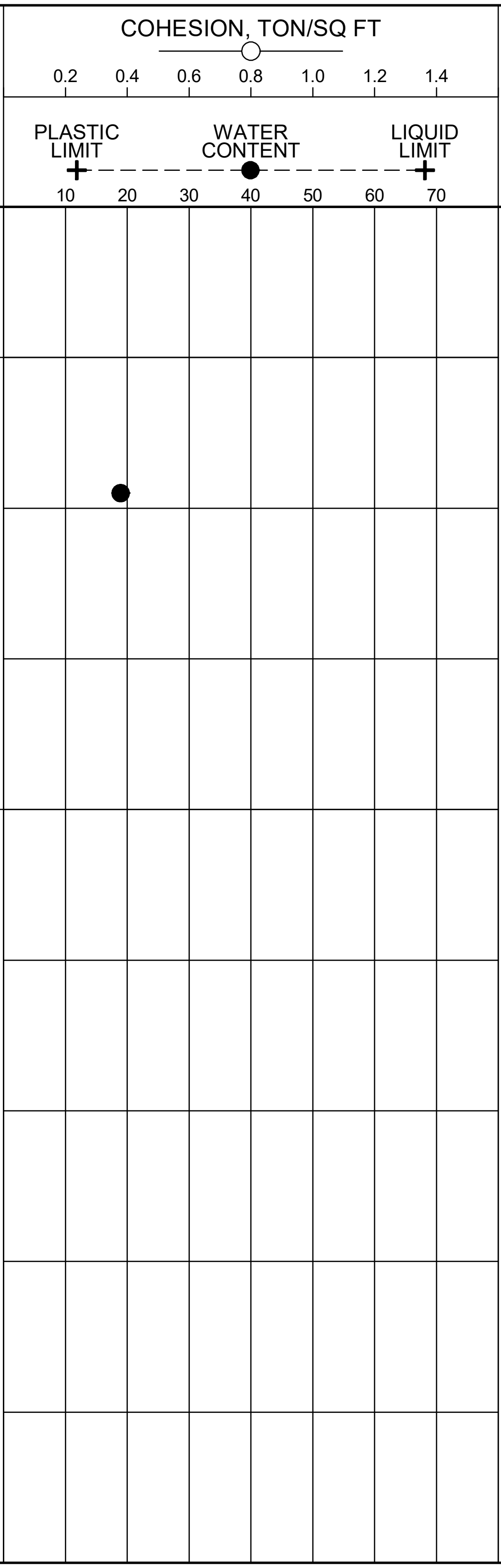
L O G O F B O R I N G N O. A3

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Wash

LOCATION: Appros Sta 520+75, 35 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
			coarse sand, slightly silty (SM-SW) w/decayed organics and a little fine gravel			
95		☒	- brown and gray below 94 ft	96		
			Dense gray fine sand, slightly silty (SM-SP)			
100		☒		58		7
105						
110		☒	- brownish gray below 108 ft	86		
			NOTE 1: Drilled from bridge deck with 20 ft HDX casing. NOTE 2: Deck to water: 7.6 ft NOTE 3: Deck to mudline: 9.8 ft NOTE 4: Drilled with CME-55 HTX ECF= 1.28			
115						
120						
125						
130						



COMPLETION DEPTH: 110.0 ft
DATE: 6-14-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/14/2023

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

L O G O F B O R I N G N O. A4

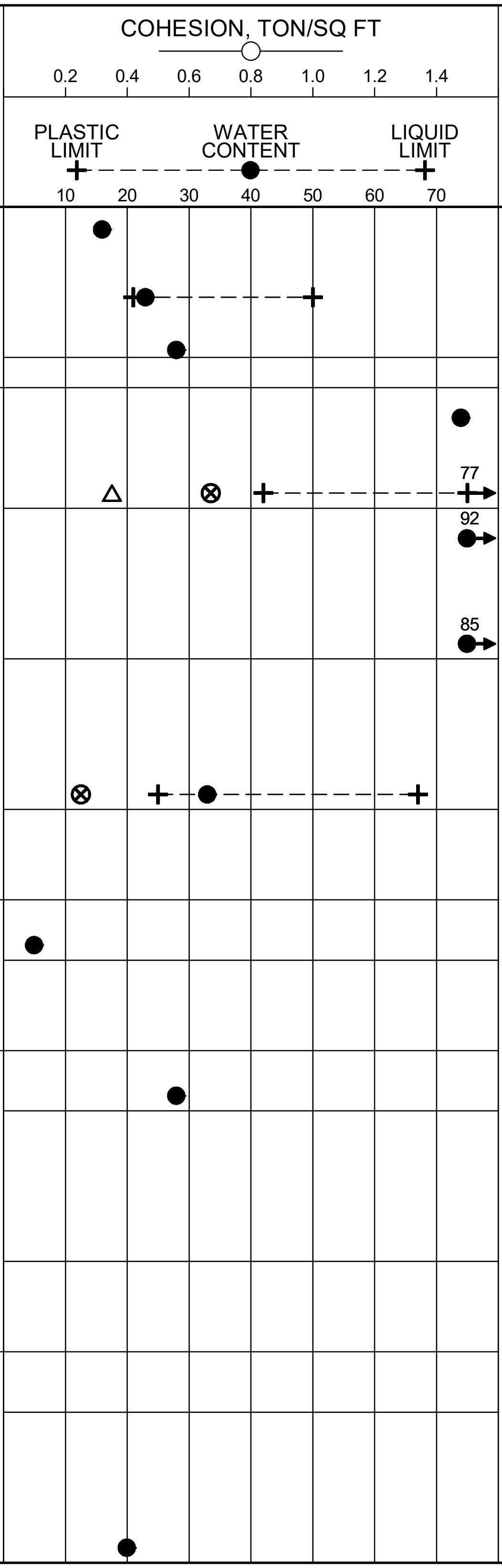
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 521+50, 20 ft Lt

SURF. EL: 218.0

DEPTH, FT	DESCRIPTION OF MATERIAL	N₆₀, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm brown clay, sandy (CH) w/occasional crushed stone and asphalt fragments (fill)	11		
		10		75
5	- soft, with less sand below 4 ft	9		
	Soft gray and reddish tan clay (CH) w/ferrous stains and decayed organics	9		
10	- stiff at 8 to 18 ft		47	85
15		33		
20	- soft below 18 ft		87	99
25	Dense grayish tan fine to medium sand, slightly silty (SP-SM)	67		6
30	Dense dark gray silty fine to medium sand (SM) w/occasional clayey sand pockets	47		14
35		44		
40	Dense grayish tan fine to medium sand (SP) w/organic inclusions	46		
		59		4



COMPLETION DEPTH: 110.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 28.6 ft

DATE: 6/5/2023

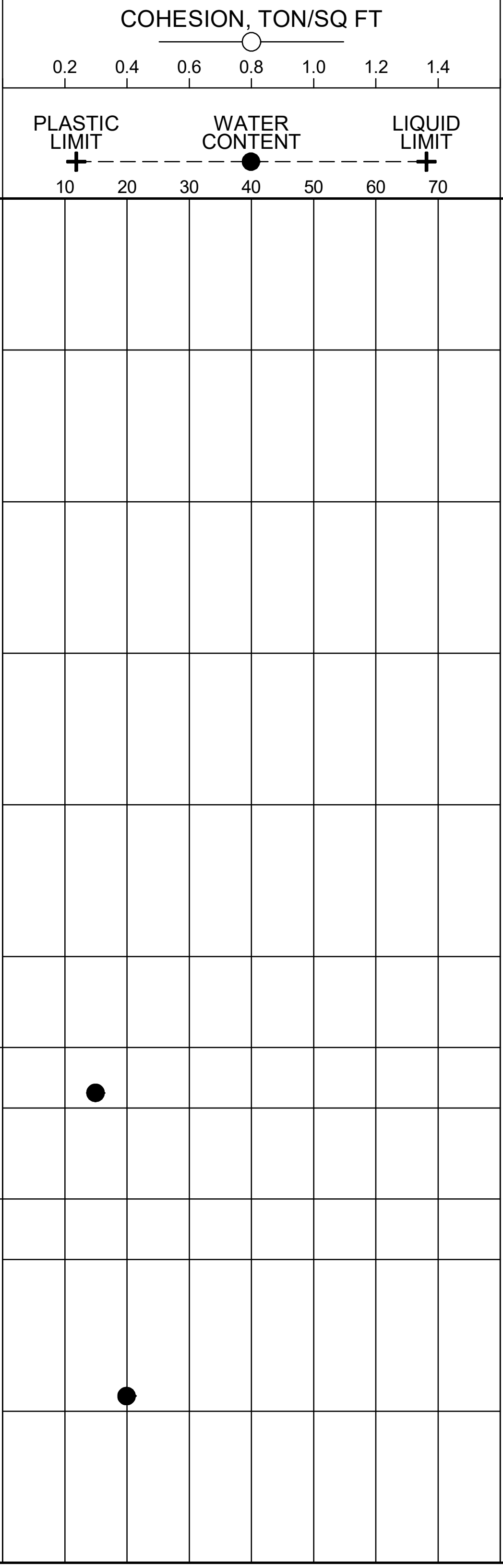
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. A4

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 521+50, 20 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		49		
55	- medium dense from 53 to 58 ft	37		
60	- dense below 58 ft	51		
65		63		
70	- with clayey sand pockets below 68 ft	57		
75	Dense grayish tan fine to medium sand, slightly silty (SW-SM) w/trace coarse sand	54		7
80	Dense tan fine to medium sand, slightly silty (SP-SM)	67		
85	- dense to very dense below 83 ft	107		6
	- with organic inclusions below 88 ft	107		

COMPLETION DEPTH: 110.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 28.6 ft

DATE: 6/5/2023

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

LOG OF BORING NO. A4

101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 521+50, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
	▨	☒		86			
95	▨						
	▨		- tan and gray below 98 ft				
	▨	☒		86			
100	▨						
105	▨						
	▨	☒		107			
110			NOTE: Drilled with Diedrich D-50 ECF= 1.43.				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 28.6 ft

DATE: 6/5/2023

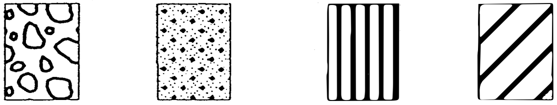
LGBNEW 23-031 BRIDGE A.GPJ 7-28-23

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

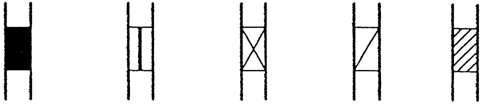
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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.

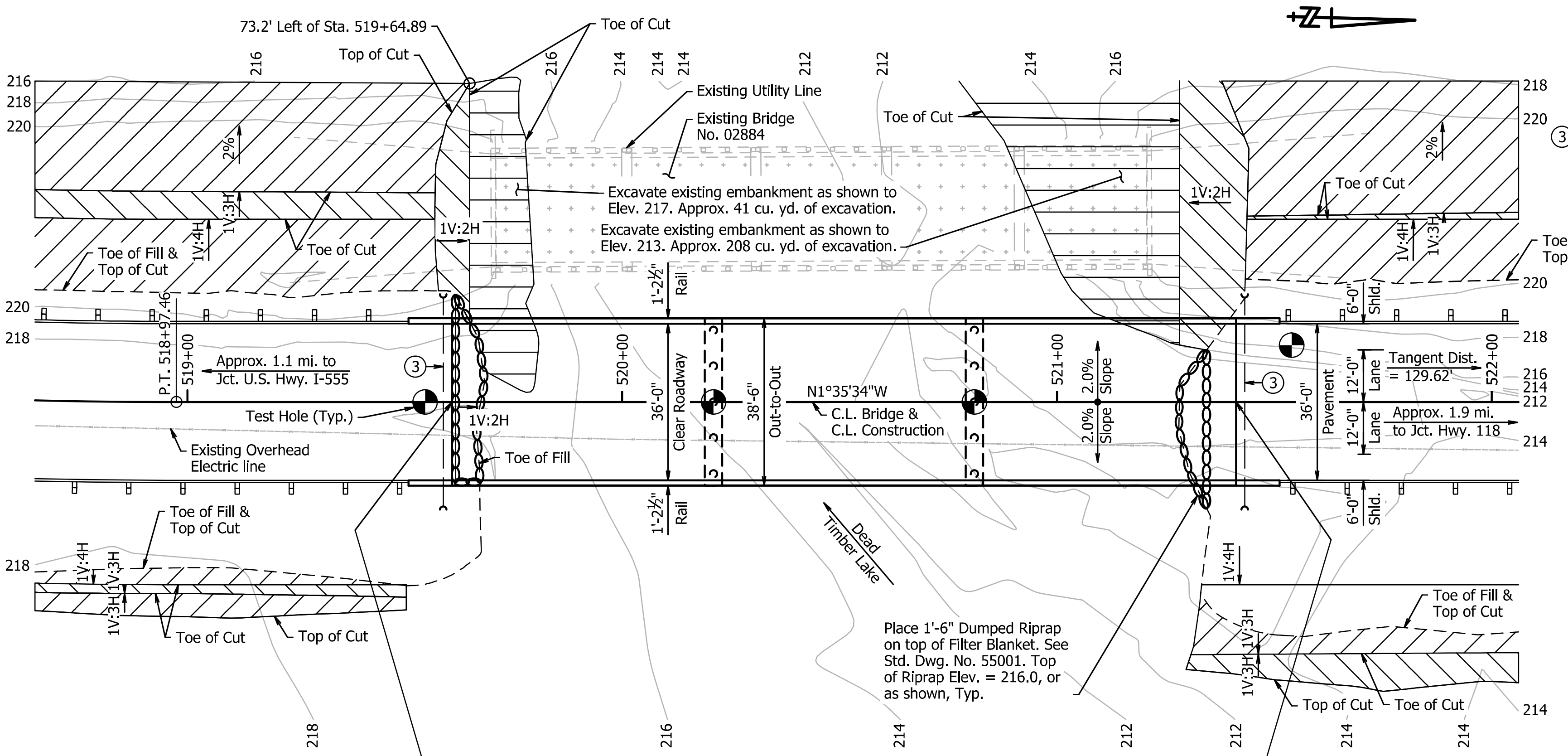
Terms used on this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No.3-357, Waterways Experiment Station, March 1953

KEY 9-26-02

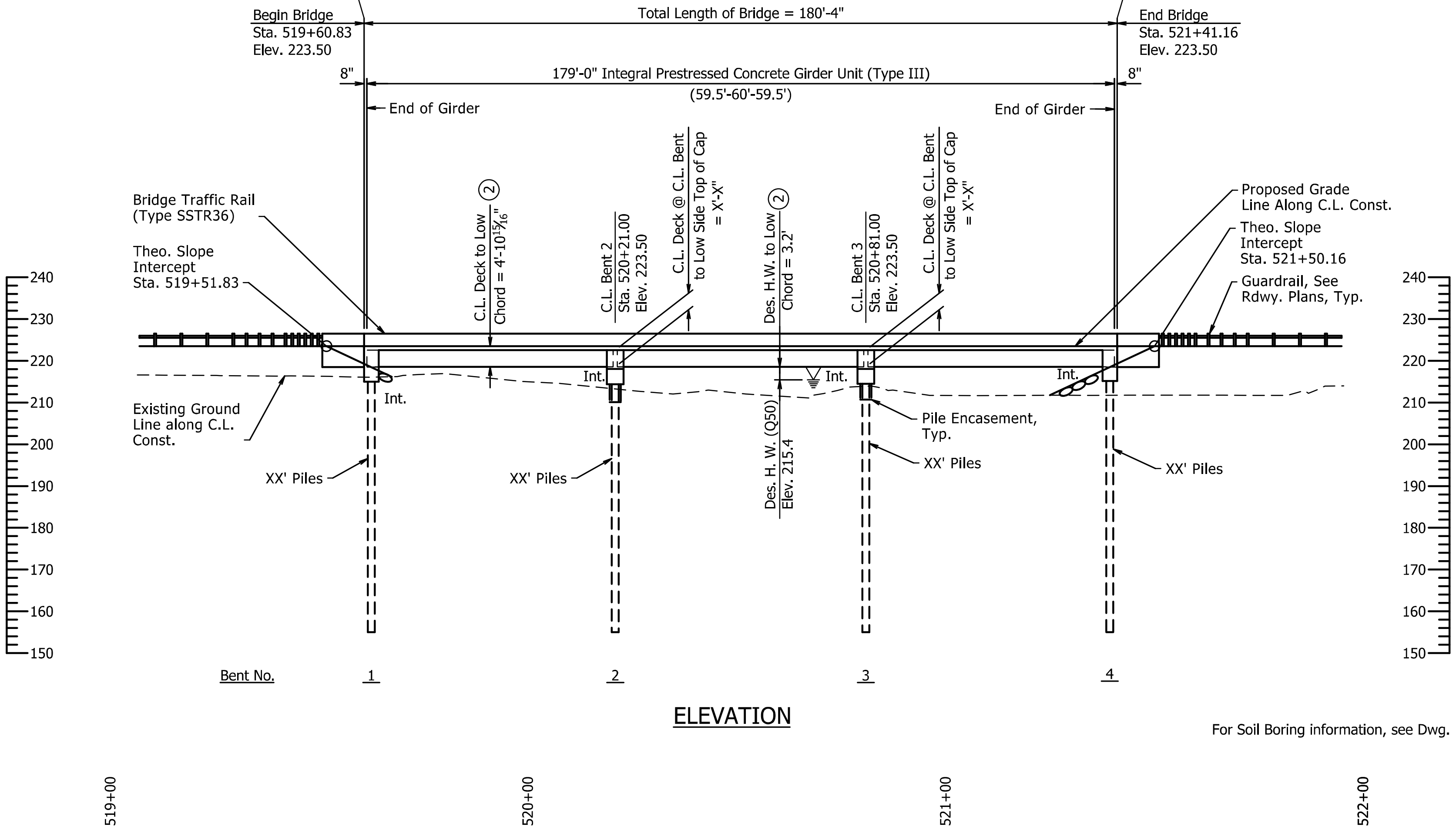
APPENDIX A

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	28	31
		XXXXX		LAYOUT		XXXXX

For R/W Data, See Roadway Plans.

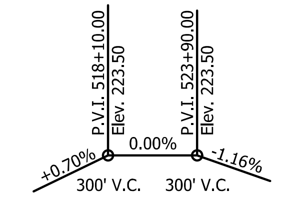


PLAN



ELEVATION

For Soil Boring information, see Dwg. No. XXXXX.



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along C.L. Construction

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

Notes:

Use Type Special Approach Gutters and Type C1 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. XXXXX & Std. Dwg. No. 55040C1, respectively.

③ 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".

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

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

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 6,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

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

STEEL SHELL PILING: Piling in Bents 1 and 4 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. Piling in Bents 2 & 3 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. All piling shall be driven with an approved air, steam, or diesel hammer to a minimum tip elevation of _____ or lower at Bents 1 and 4 and to a minimum tip elevation of ______ or lower at Bents 2 and 3. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (__" Dia.)".

PREBORING: Preboring is required for all piling at Bents 1 and 4. Prebored holes shall have a diameter 6" greater than the diameter of the pile for a depth of 10' below the bottom of the cap. The void space around the pile after completion of driving shall be backfilled with sand or pea gravel. The Contractor shall be responsible for keeping prebored holes free of debris prior to backfilling which may require the use of temporary casings or other approved methods. Any related cost for backfilling and temporary casing will not be paid for directly, but shall be considered subsidiary to the item "Preboring".

DRIVING SYSTEM: The driving system approval and the ultimate bearing capacity determination for piling shall be based on the requirements of Subsection 805.09(b), "Method B - Wave Equation Analysis (WEAP)". It is estimated that the minimum rated hammer energy required to obtain the ultimate bearing capacity for all piles will be _____foot pounds per blow.

For Additional General Notes, see Dwg. No. XXXXX.

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	① NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	710	215.0	215.4
BASE	100	780	215.1	215.5
EXTREME	500	920	215.2	215.7
OVERTOPPING	>500	--	--	--

① Unconstricted water surface elevation without structure or roadway approaches.

② Proposed Low Bridge Chord Elev. = 218.59 feet at Station 519+60.83

100 yr. backwater elevation for existing structure = 215.7 feet
Drainage Area = 5.2 sq. miles
Historical H.W. Elev. = N/A

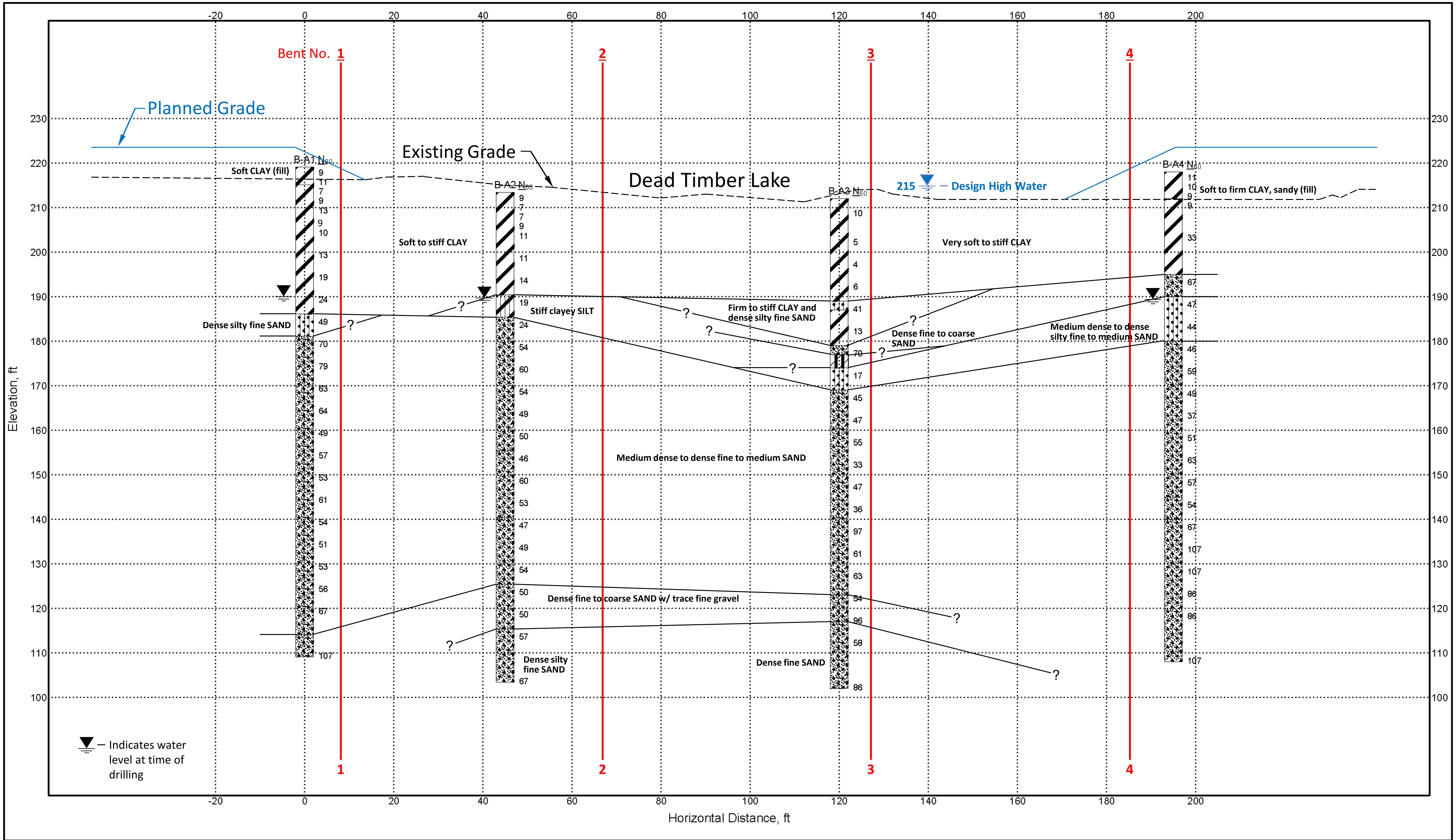
PRELIMINARY
SUBJECT TO
REVISION

SHEET 1 OF 2
LAYOUT OF BRIDGE
HWY. 135 OVER DEAD TIMBER LAKE
HWY. 135 STRS. & APPRS. (S)
CRAIGHEAD & POINSETT COUNTIES
ROUTE 135 SECTIONS 1 & 2
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: BWC DATE: 02-21-2023 FILENAME: b101124x1_l1.dgn
CHECKED BY: CAW DATE: 02-24-2023 SCALE: 1" = 20'
DESIGNED BY: KRM DATE: 02-14-2023
BRIDGE NO. XXXXX DRAWING NO. XXXXX

USER: CTAUSER
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x1_l1.dgn SCALE: 40.0000 ' / In.
PLOTTED: 3/7/2023 10:37:00 AM

APPENDIX B



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

SCALE:
1" = 20' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Dead Timber Lake (Site 1)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

BORING NO.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING								USCS CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	2 in.	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
A1	2.5-3.5	30	66	27	39	---	---	---	---	100	---	---	96	CH	A-7-6
A1	9-10	23	61	24	37	100	100	100	100	100	100	100	99	CH	A-7-6
A1	24-25	37	102	30	72	---	---	---	---	100	---	---	91	CH	A-7-6
A1	34-35	24	---	---	---	---	---	---	---	100	---	---	17	SM	A-2-4
A1	39-40	26	---	---	---	100	100	100	100	100	100	67	6	SM-SP	A-3
A1	44-45	28	---	---	---	100	100	100	100	100	100	78	6	SM-SP	A-3
A1	69-70	19	---	---	---	100	100	100	100	100	99	41	5	SM-SP	A-1-b
A1	109-110	15	---	---	---	100	100	89	89	84	81	69	10	SM-SP	A-3
A2	14-15	39	96	34	62	---	---	---	---	98	---	---	96	CH	A-7-5
A2	24-25	40	40	20	20	---	---	---	---	100	---	---	87	CL	A-6
A2	34-35	21	---	---	---	100	100	100	100	100	100	79	5	SM-SP	A-3
A2	59-60	17	---	---	---	100	100	100	99	98	95	31	5	SM-SW	A-1-b
A2	74-75	23	---	---	---	100	100	100	100	99	97	80	8	SM-SP	A-3
A2	89-90	14	---	---	---	100	100	100	93	85	72	27	5	SM-SW	A-1-b
A2	99-100	20	---	---	---	100	100	100	100	100	100	96	7	SM-SP	A-3
A3	4.5-5	29	77	27	50	100	100	100	100	100	100	100	99	CH	A-7-6
A3	19-20	32	65	24	41	---	---	---	---	100	---	---	99	CH	A-7-6
A3	39-40	27	---	---	---	---	---	---	---	100	---	---	34	SM	A-2-4
A3	49-50	18	---	---	---	100	100	100	100	100	99	56	4	SP	A-3
A3	69-70	21	---	---	---	100	100	100	100	100	98	59	6	SM-SP	A-3
A3	89-90	28	---	---	---	100	100	100	91	83	72	23	9	SM-SW	A-1-b
A3	99-100	19	---	---	---	100	100	100	99	98	97	84	7	SM-SP	A-3

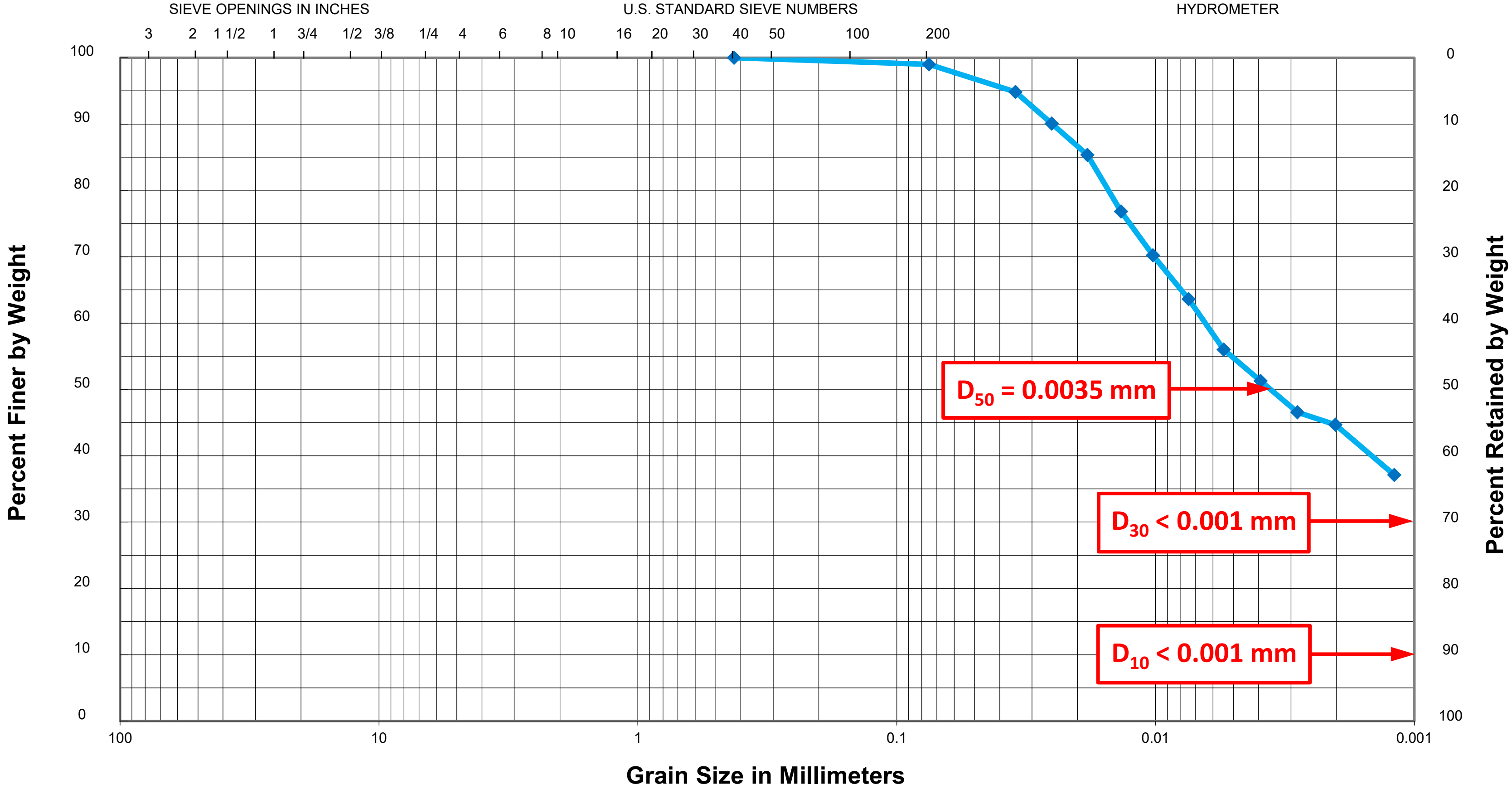
SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Dead Timber Lake (Site 1)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
A4	2.5-3.5	23	50	21	29	---	---	---	---	94	---	---	75	CH	A-7-6
A4	9-9.5	92	77	42	35	---	---	---	---	100	---	---	85	MH	A-7-5
A4	19-19.5	33	67	25	42	---	---	---	---	100	---	---	99	CH	A-7-6
A4	24-25	5	---	---	---	100	100	100	100	100	100	67	6	SM-SP	A-3
A4	29-30	28	---	---	---	100	100	100	94	94	93	69	14	SM	A-2-4
A4	44-45	20	---	---	---	100	100	100	100	100	99	59	4	SP	A-3
A4	74-75	15	---	---	---	100	100	100	100	99	93	26	7	SM-SW	A-1-b
A4	84-85	20	---	---	---	100	100	100	100	100	100	67	6	SM-SP	A-3

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.0035 mm

D_{30} < 0.001 mm

D_{10} < 0.001 mm

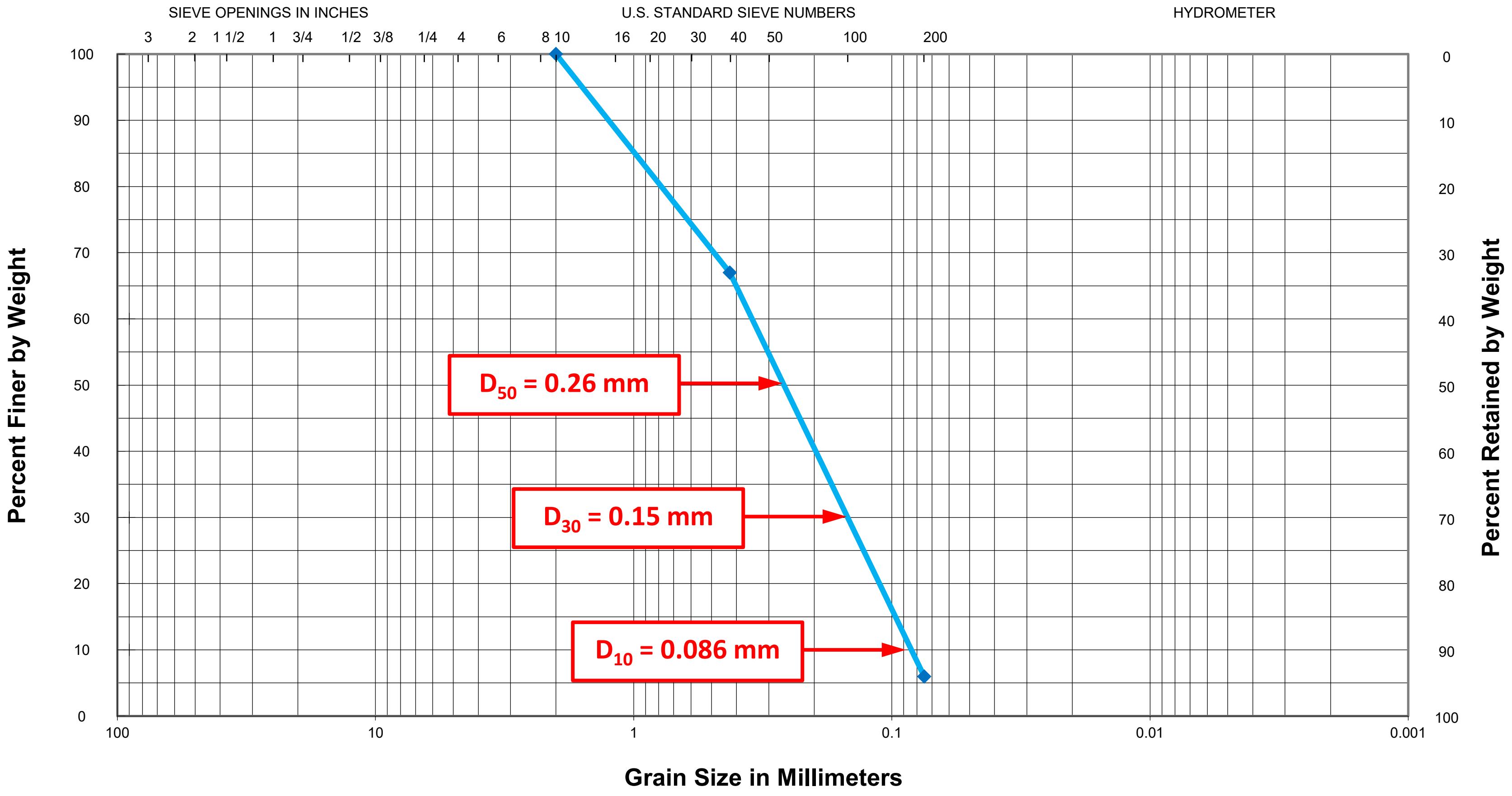
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring A1, 9-10 ft; LL=61, PL=24, PI=37
Description: Gray, tan, and reddish tan CLAY w/ ferrous stains and occasional decayed organics

USCS Classification = CH
AASHTO Classification = A-7-6

23-031

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

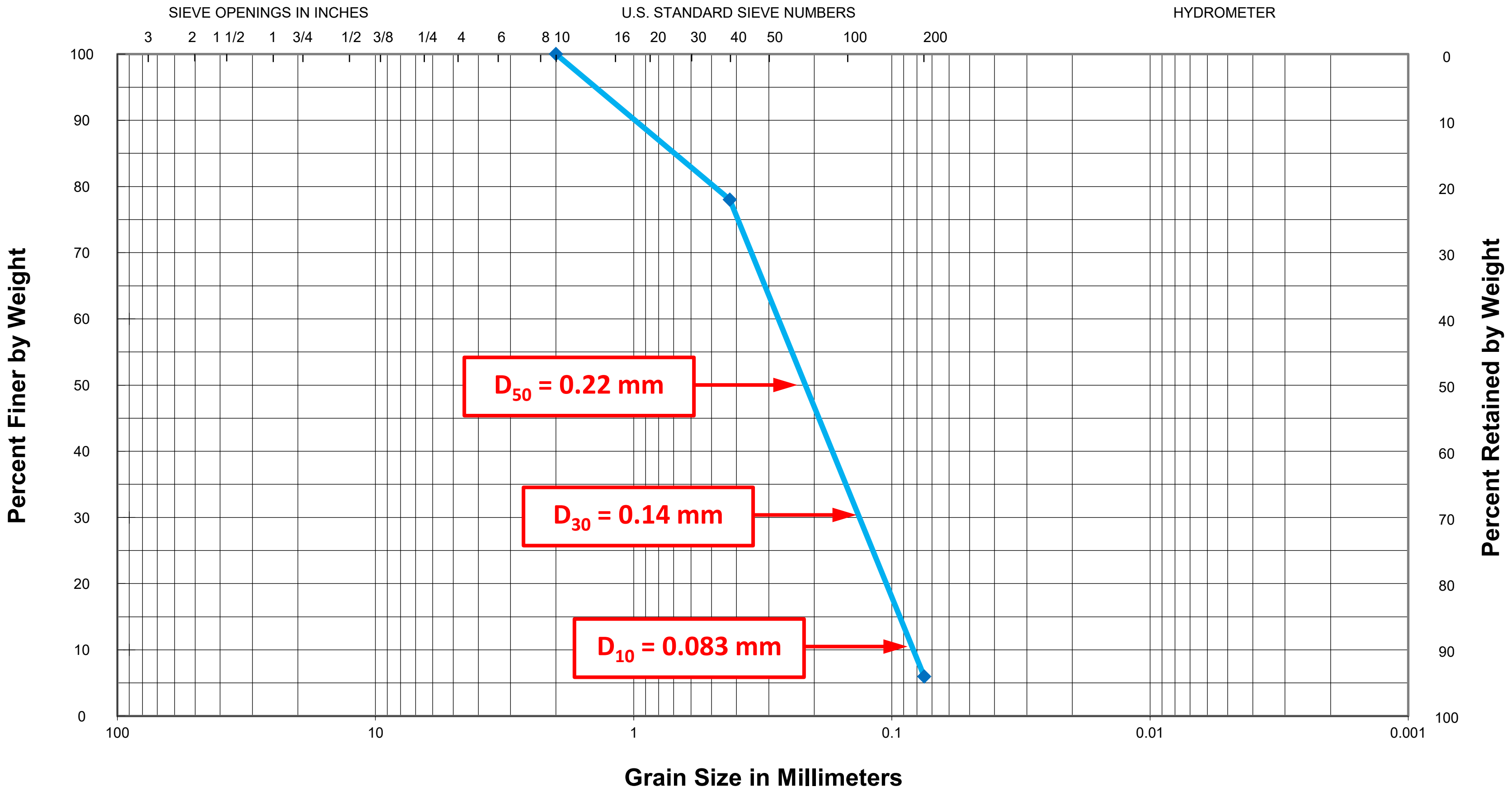
Sample: Boring A1, 39-40 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

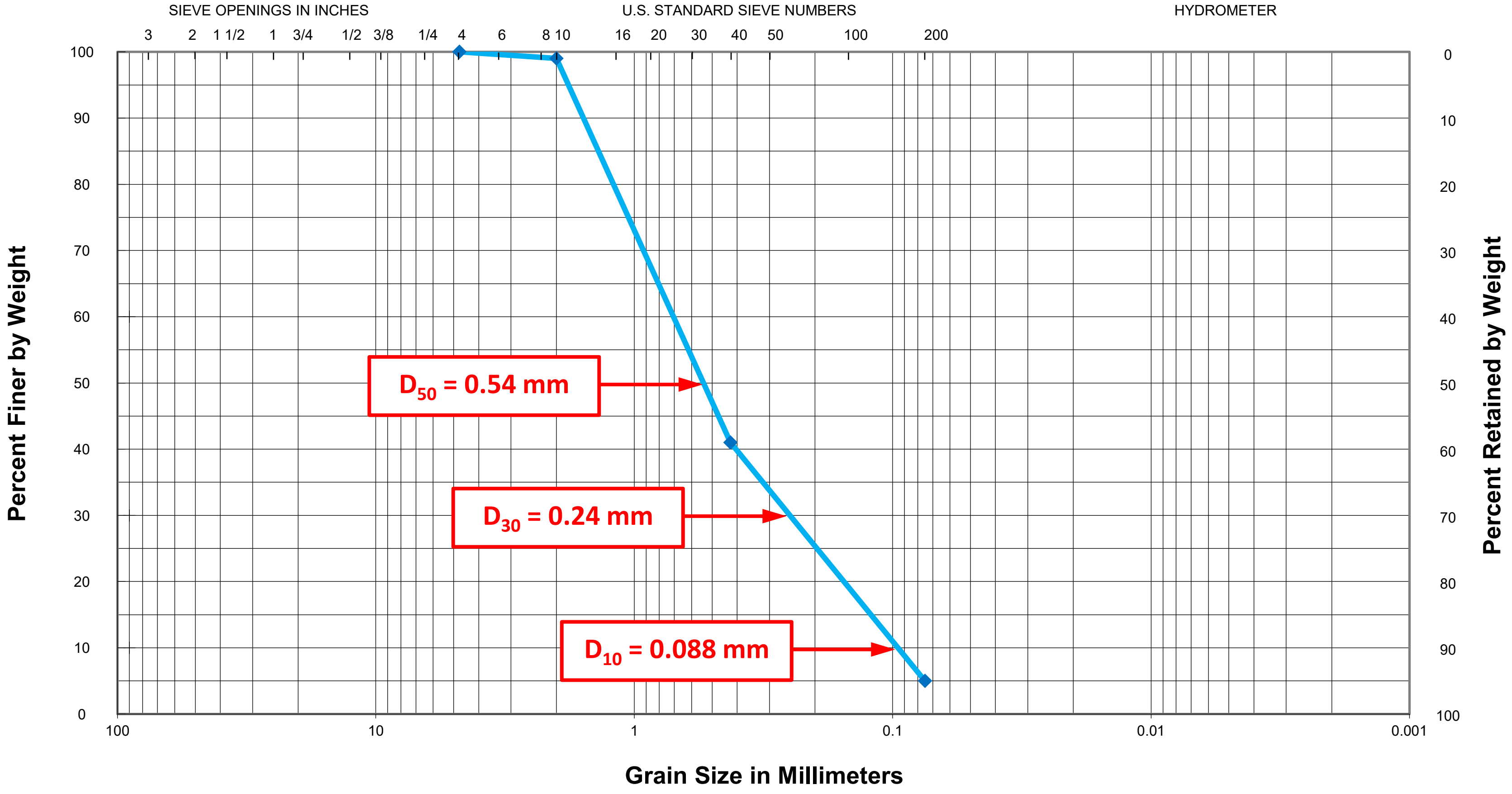
Sample: Boring A1, 44-45 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

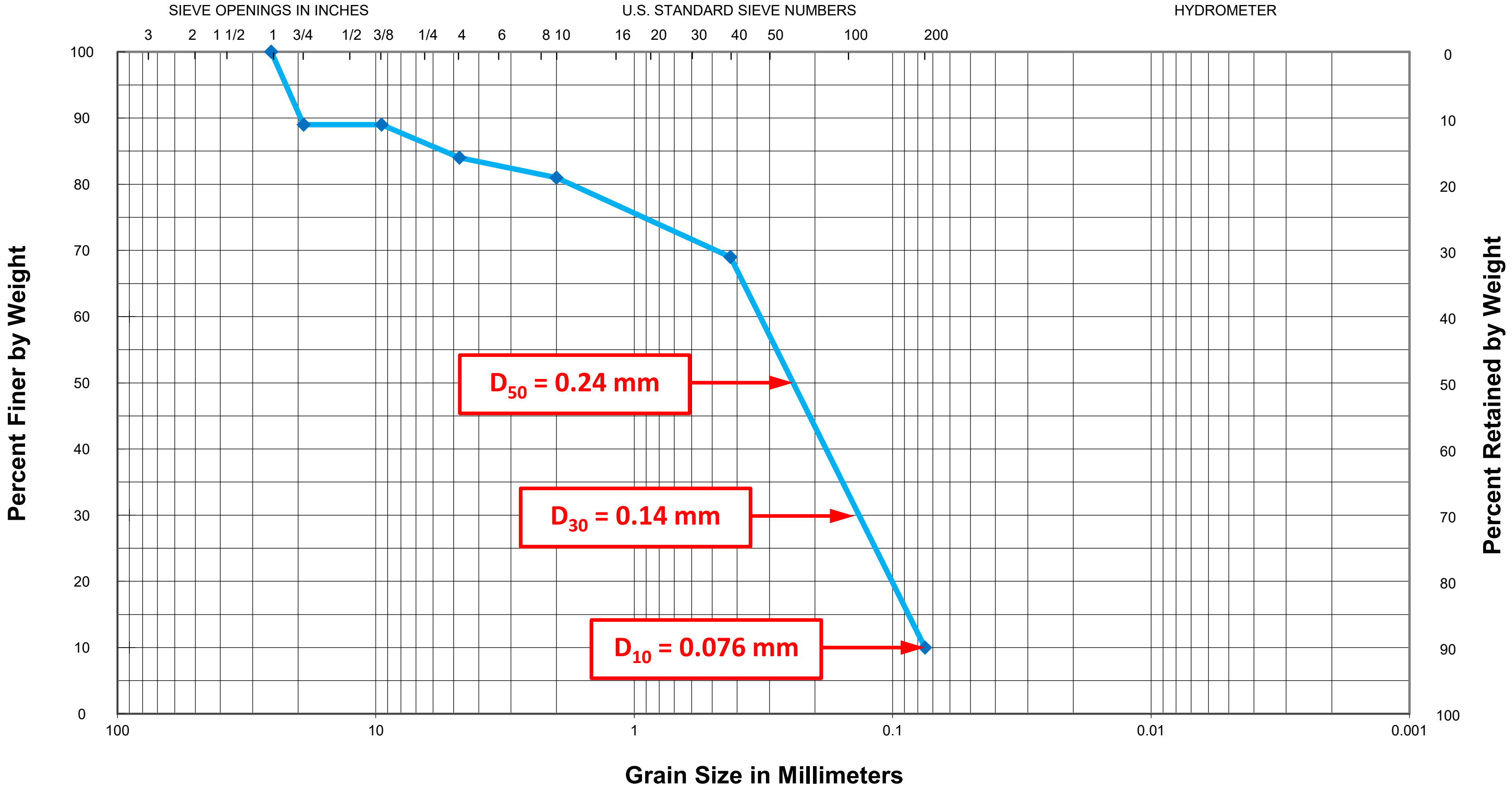
Sample: Boring A1, 69-70 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

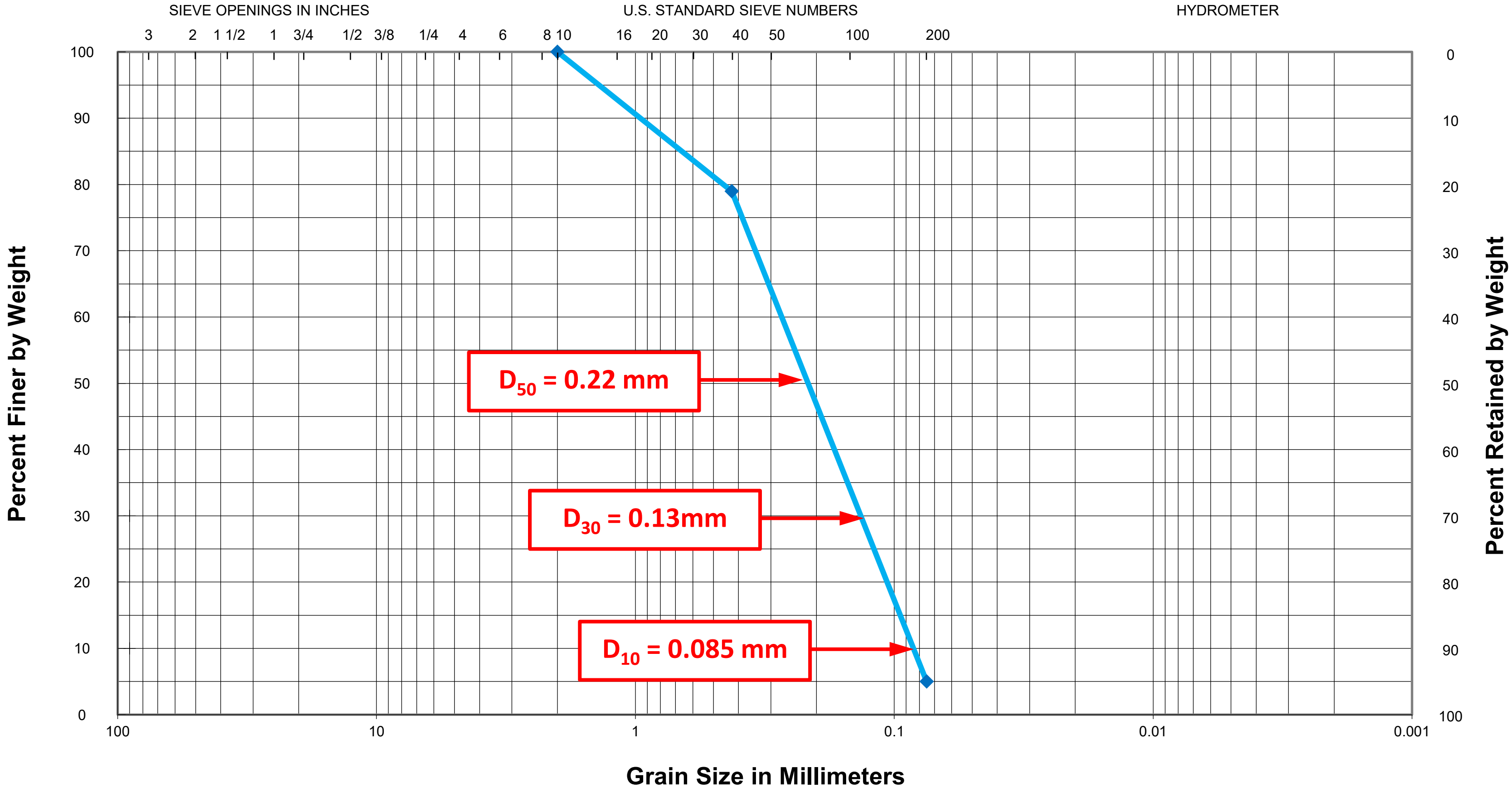
Sample: Boring A1, 109-110 ft
Description: Tan fine to medium SAND, slightly silty w/ trace coarse sand and fine to coarse gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

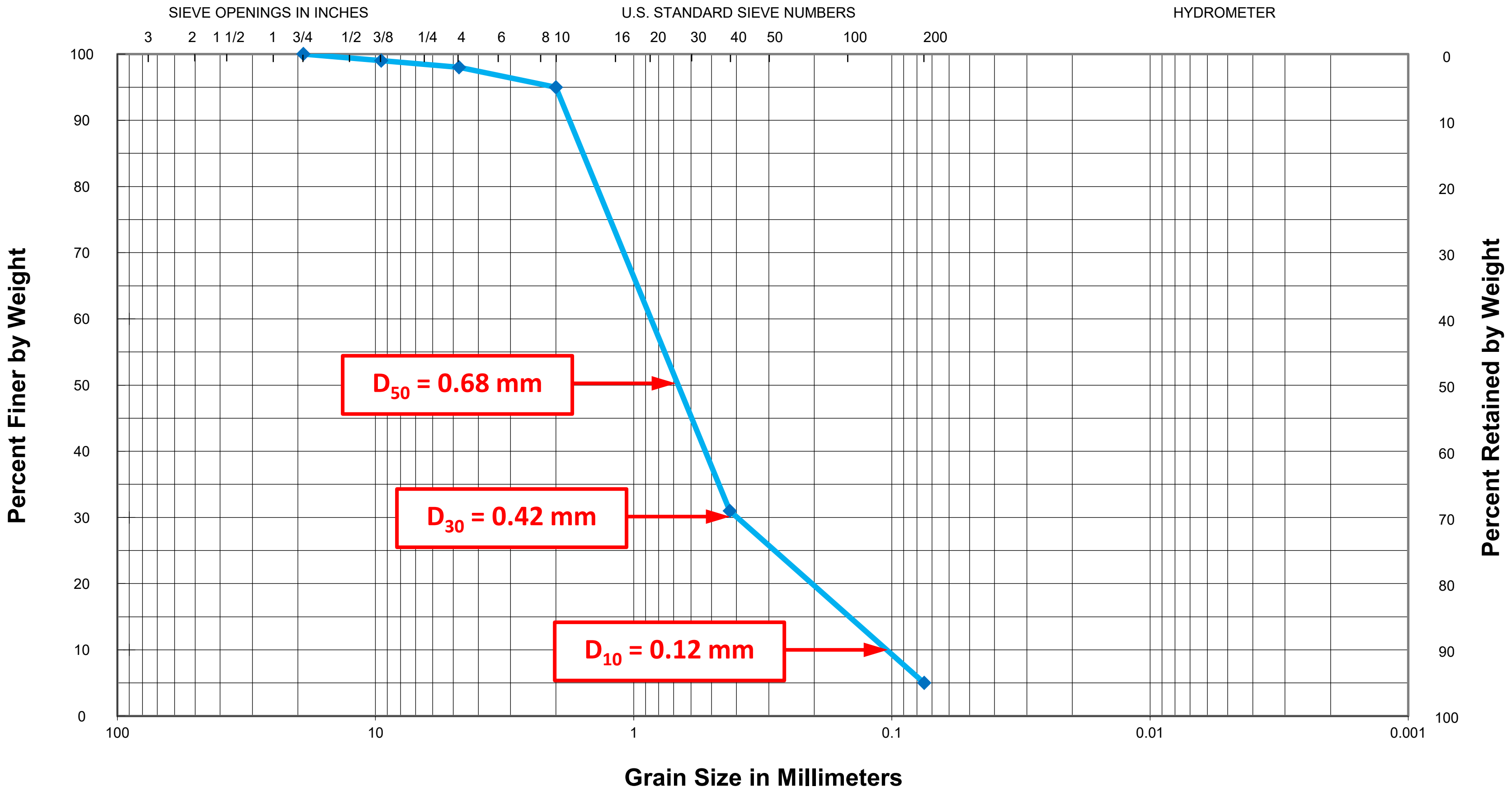
Sample: Boring A2, 34-35 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

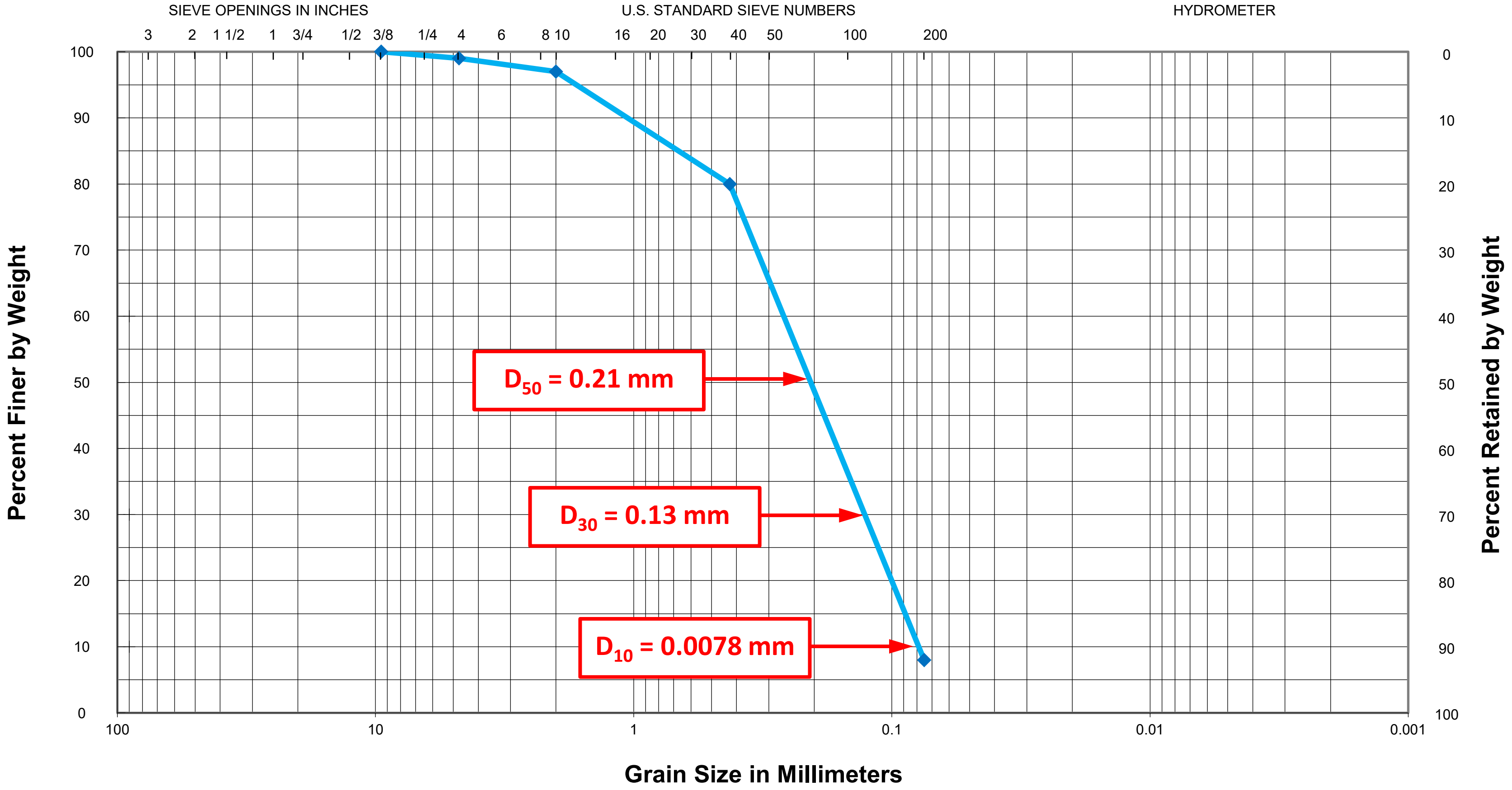
Sample: Boring A2, 59-60 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

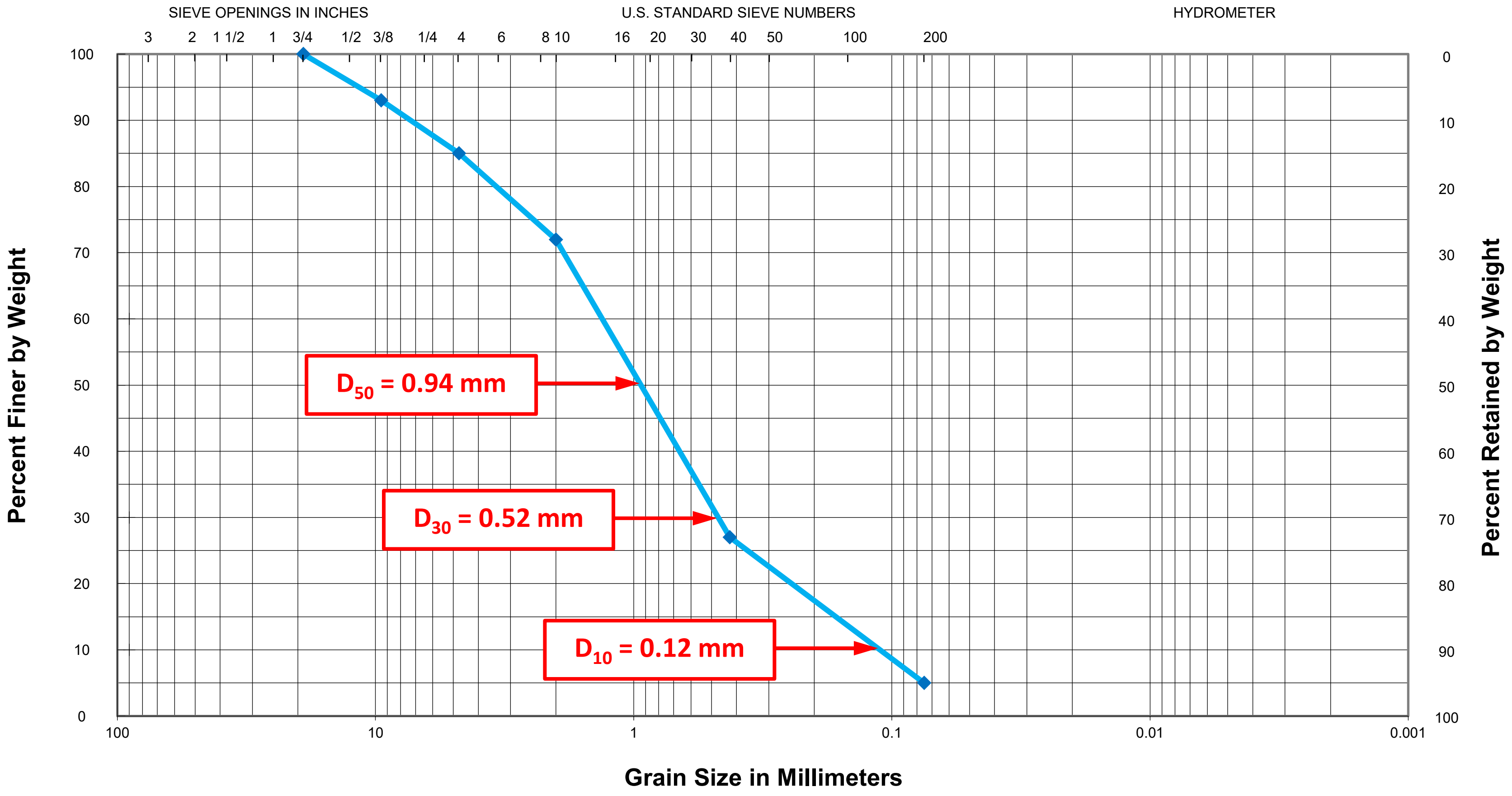
Sample: Boring A2, 74-75 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.94 mm

D_{30} = 0.52 mm

D_{10} = 0.12 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

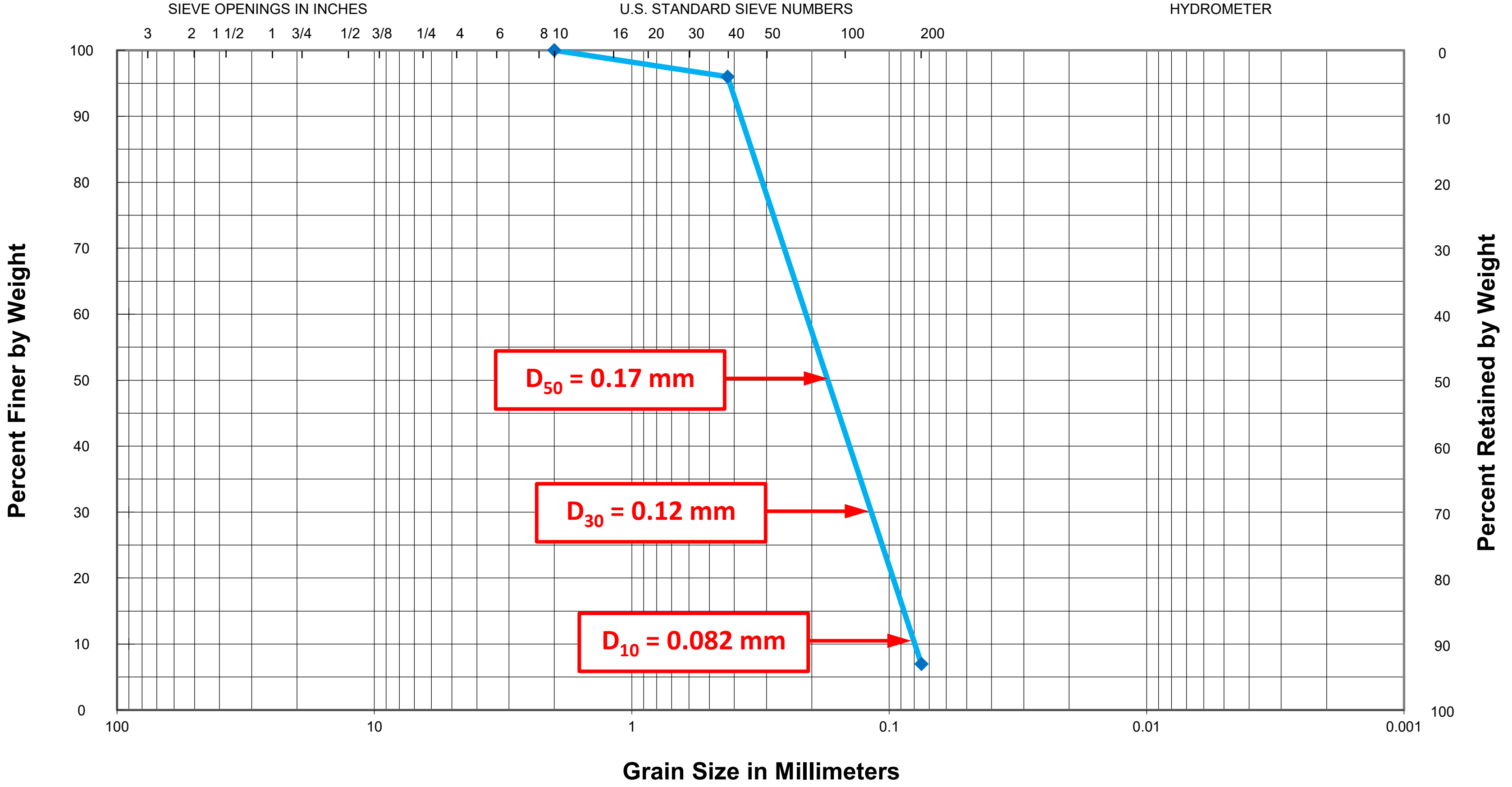
Sample: Boring A2, 89-90 ft
Description: Grayish tan fine to coarse SAND, slightly silty w/ trace fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.17 mm

D_{30} = 0.12 mm

D_{10} = 0.082 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

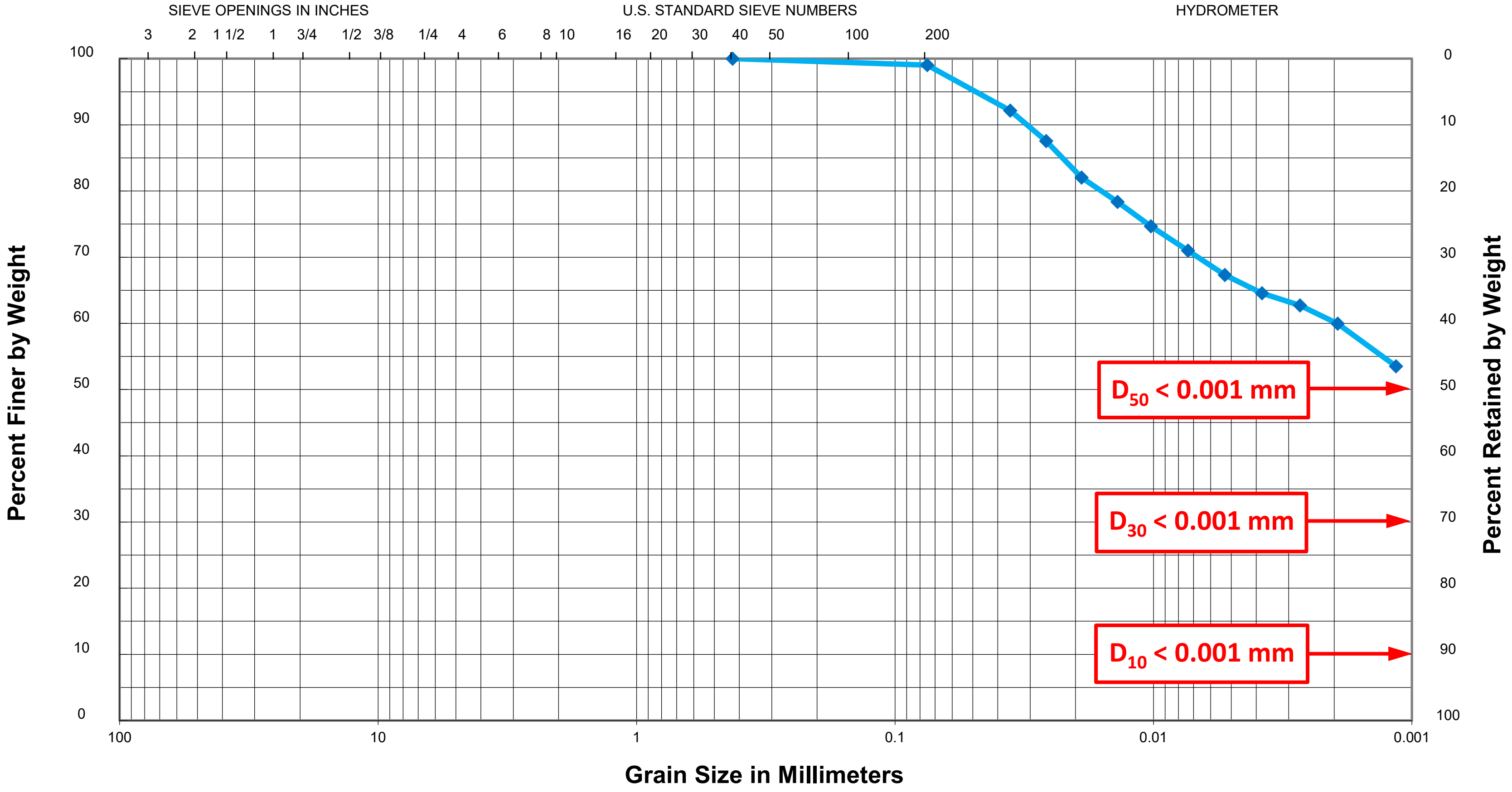
Sample: Boring A2, 99-100 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

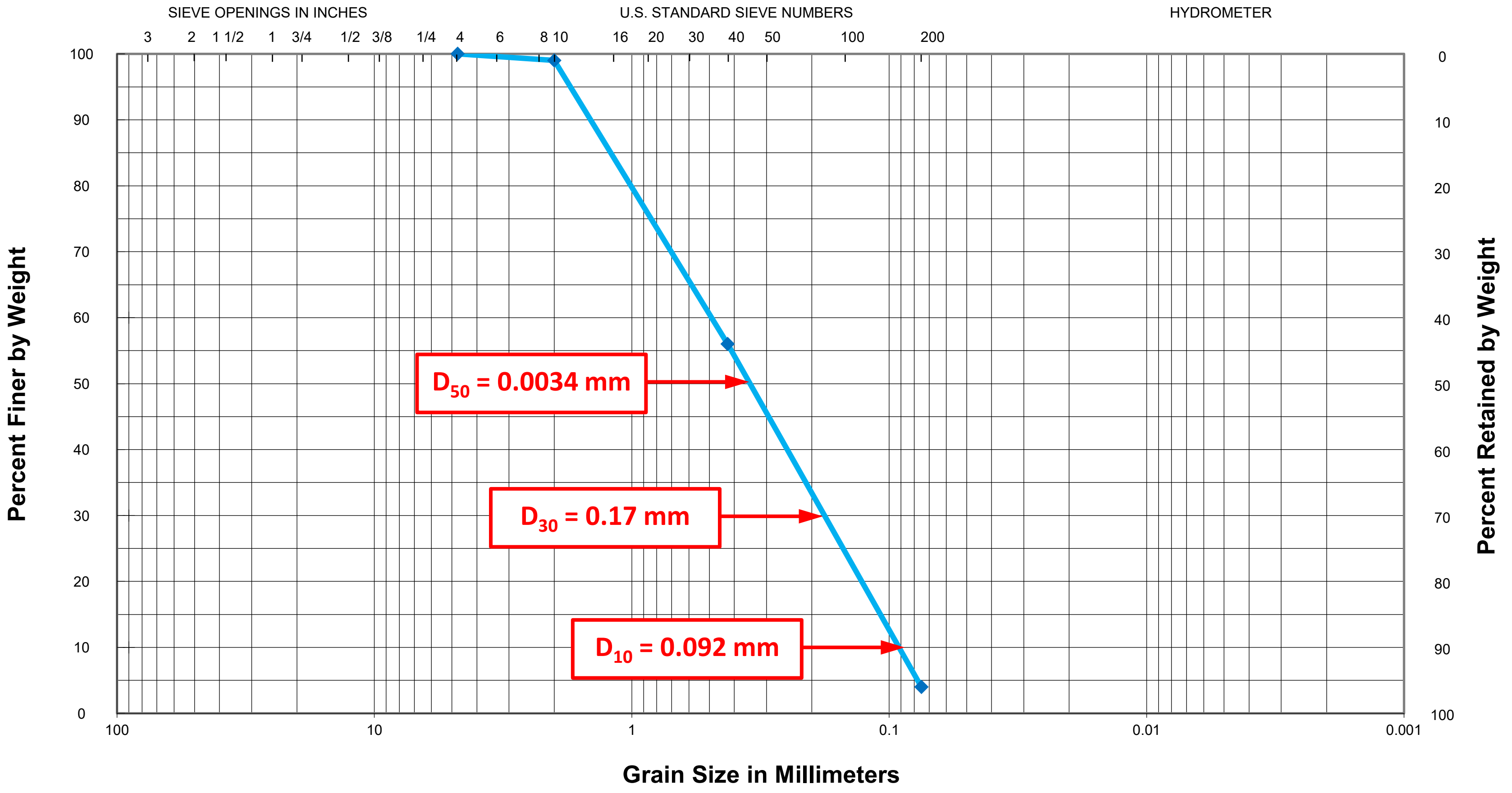
Sample: Boring A3, 4.5-5 ft; LL=77, PL=27, PI=50
Description: Gray CLAY w/ organics and ferrous stains

USCS Classification = CH
AASHTO Classification = A-7-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

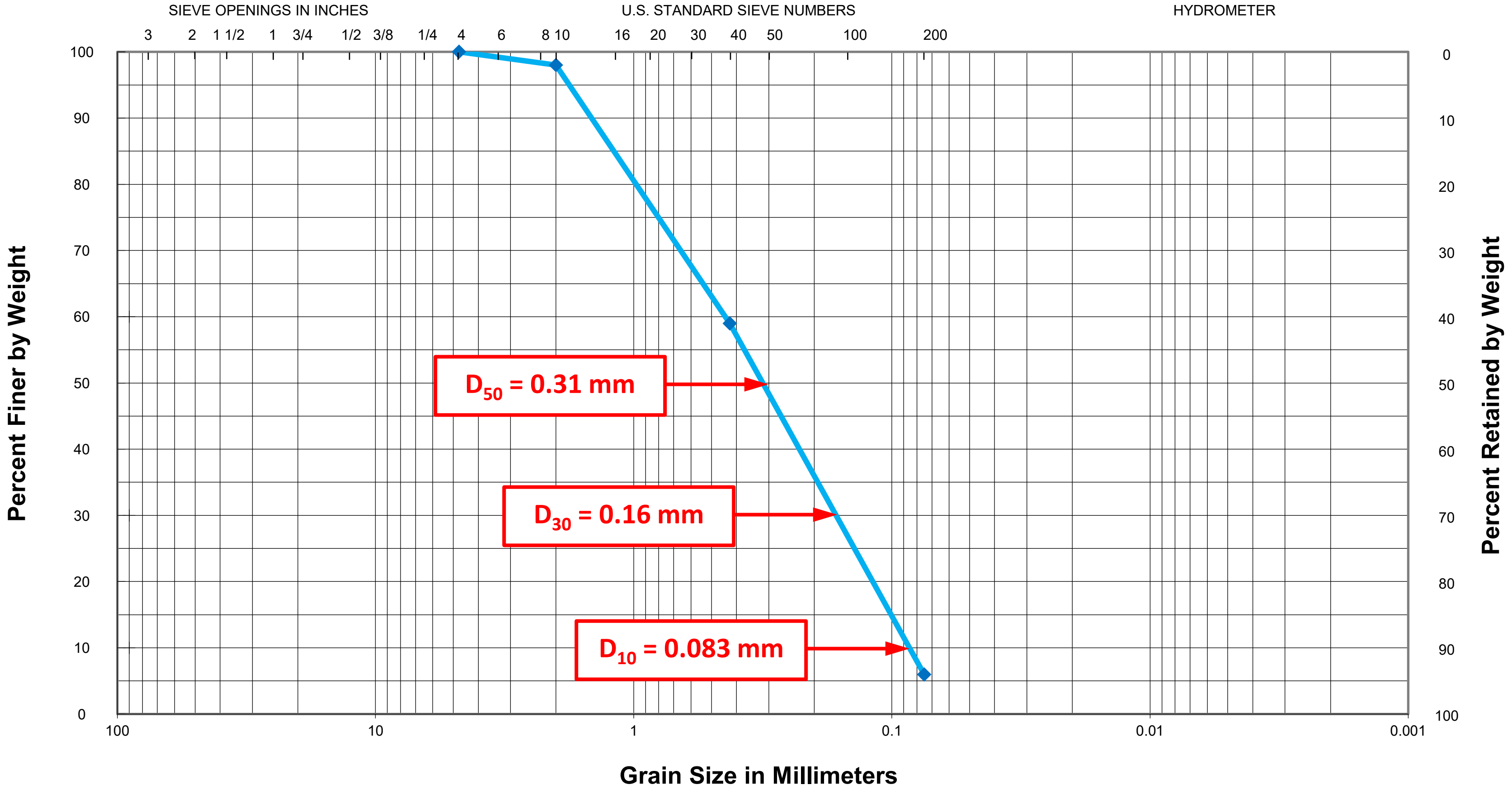


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring A3, 49-50 ft
Description: Grayish brown fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

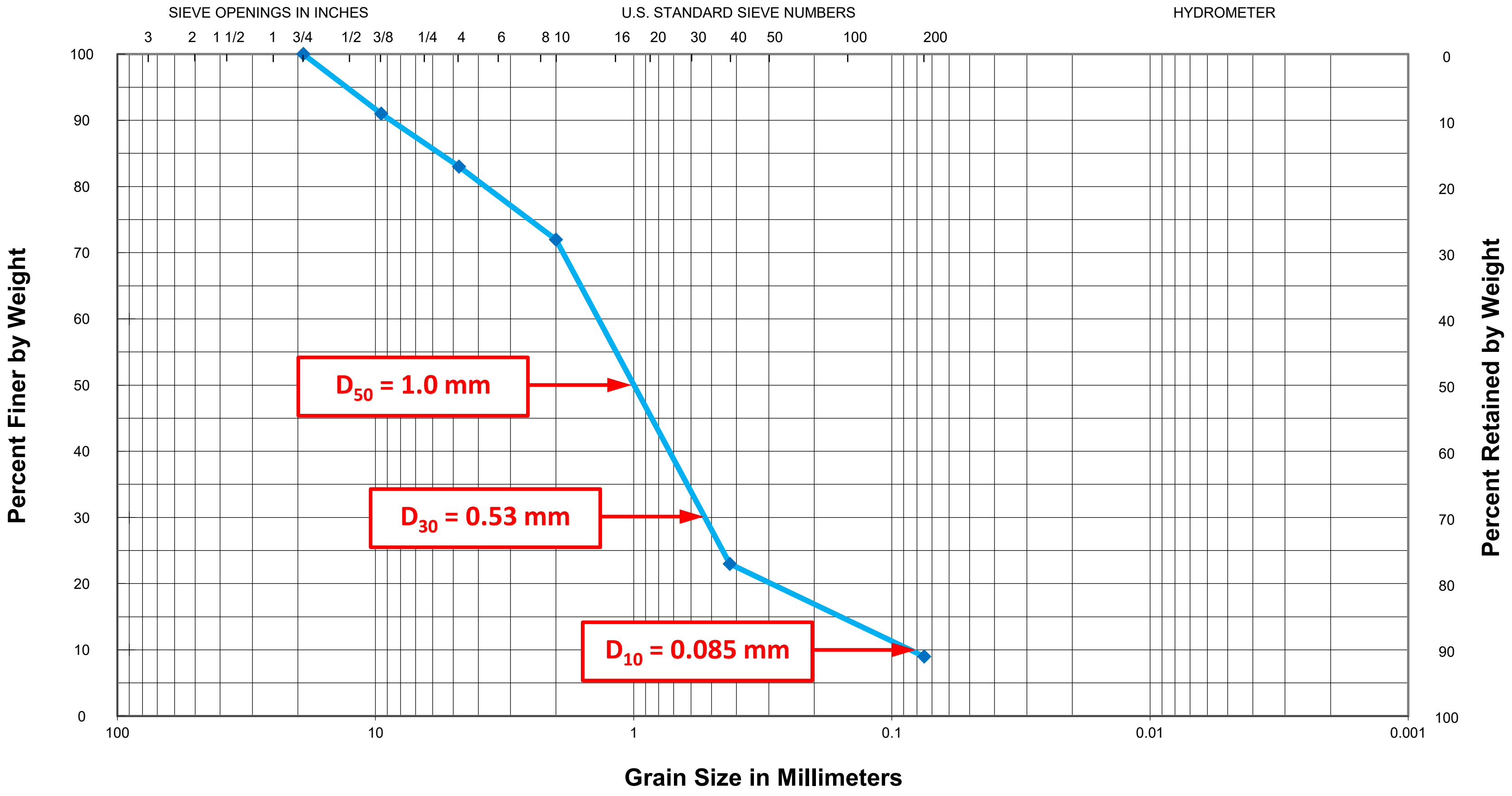
Sample: Boring A3, 69-70 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 1.0 mm

D_{30} = 0.53 mm

D_{10} = 0.085 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

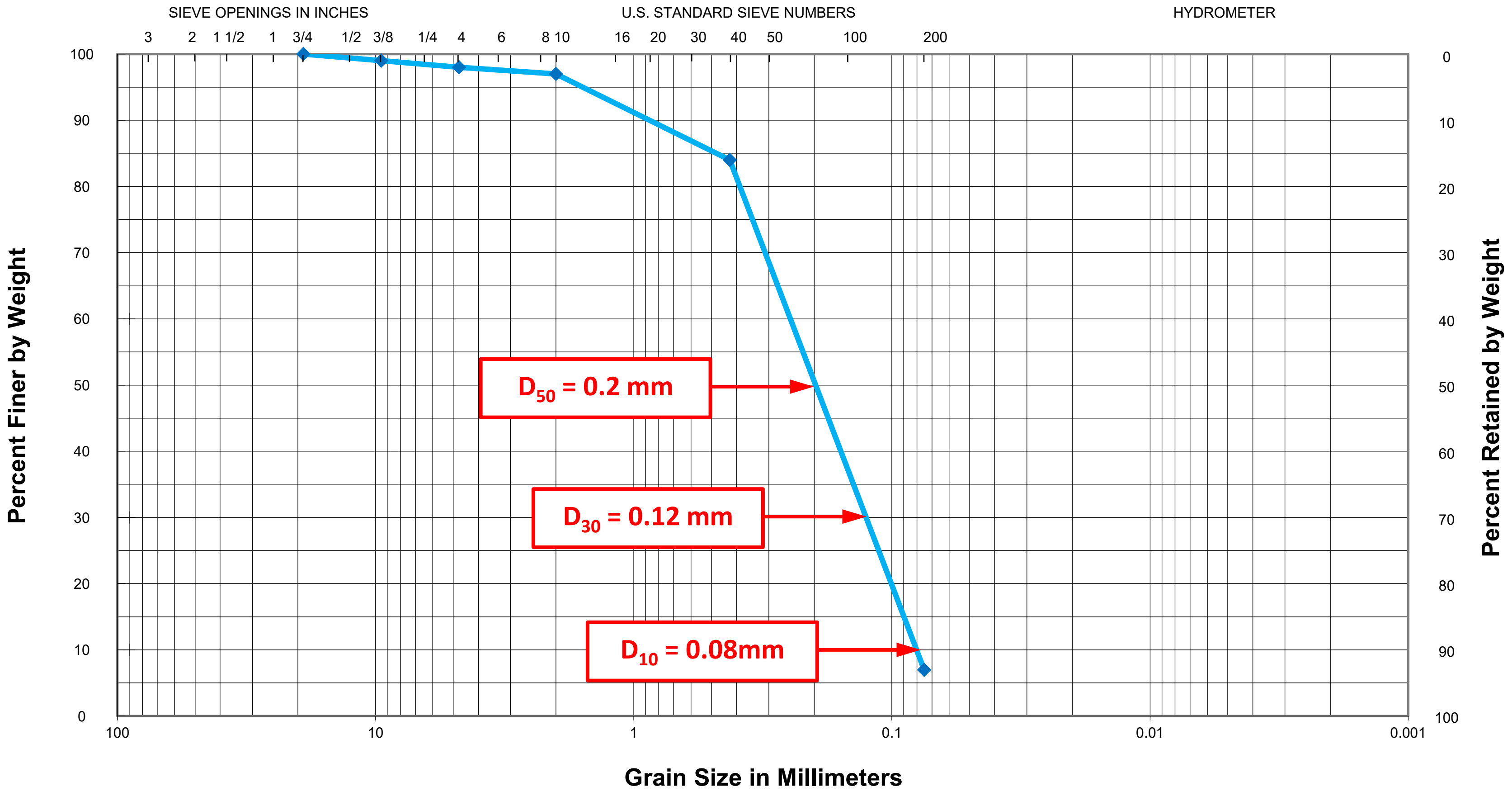
Sample: Boring A3, 89-90 ft
Description: Brown and dark gray fine to coarse SAND, slightly silty w/ a little fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.2 mm

D_{30} = 0.12 mm

D_{10} = 0.08mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

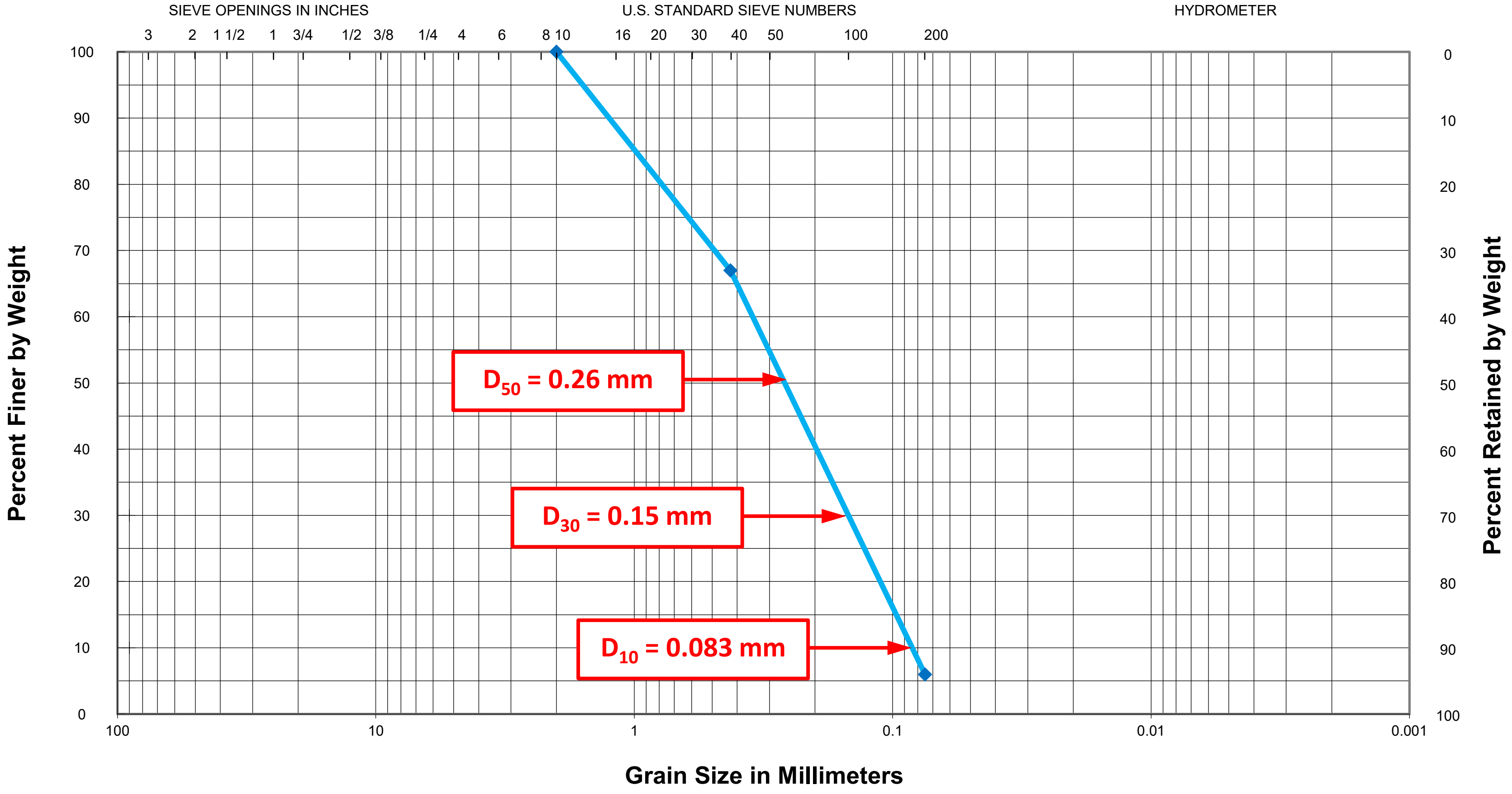
Sample: Boring A3, 99-100 ft
Description: Gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.26 mm

D_{30} = 0.15 mm

D_{10} = 0.083 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

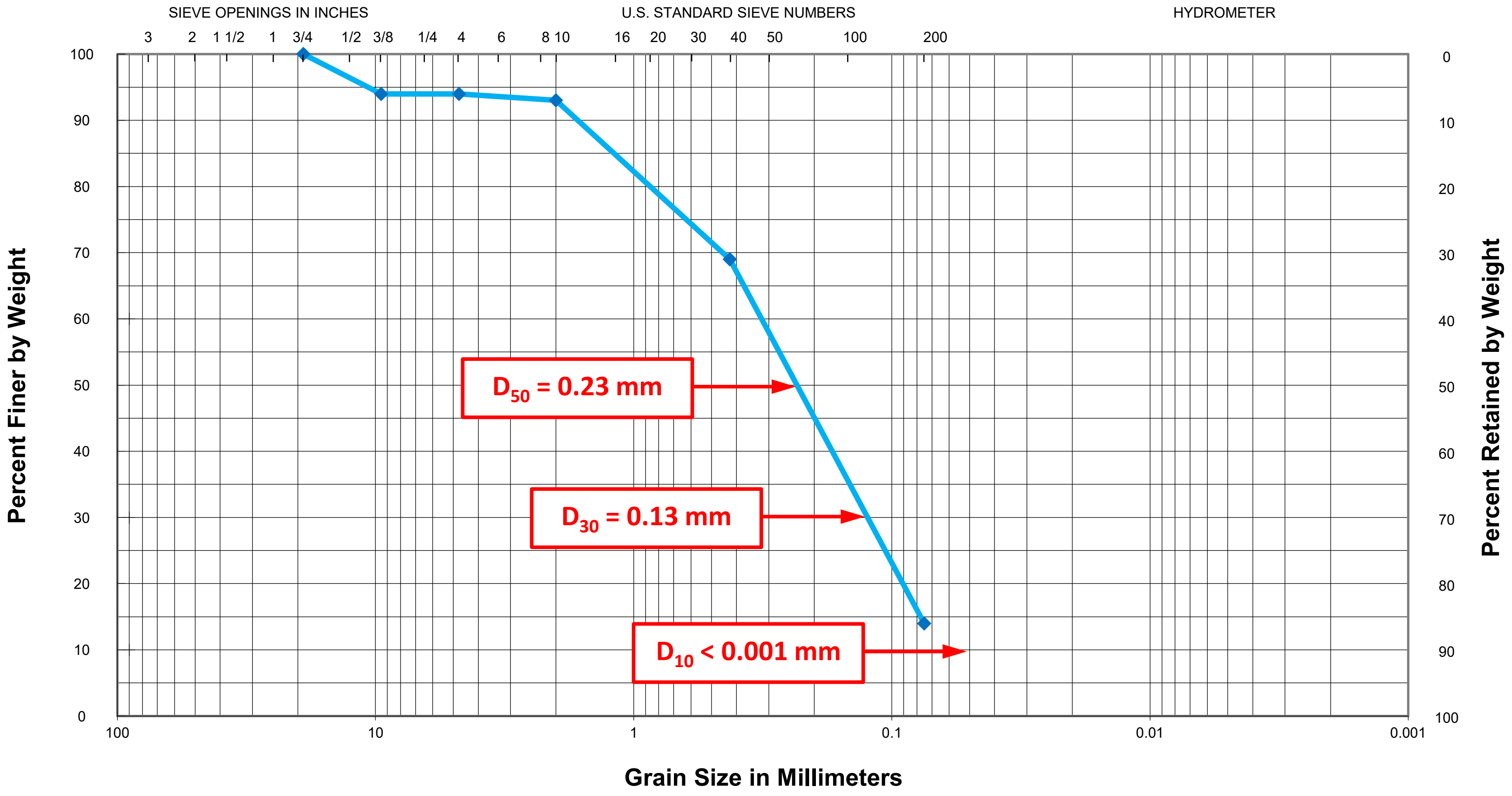
Sample: Boring A4, 24-25 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

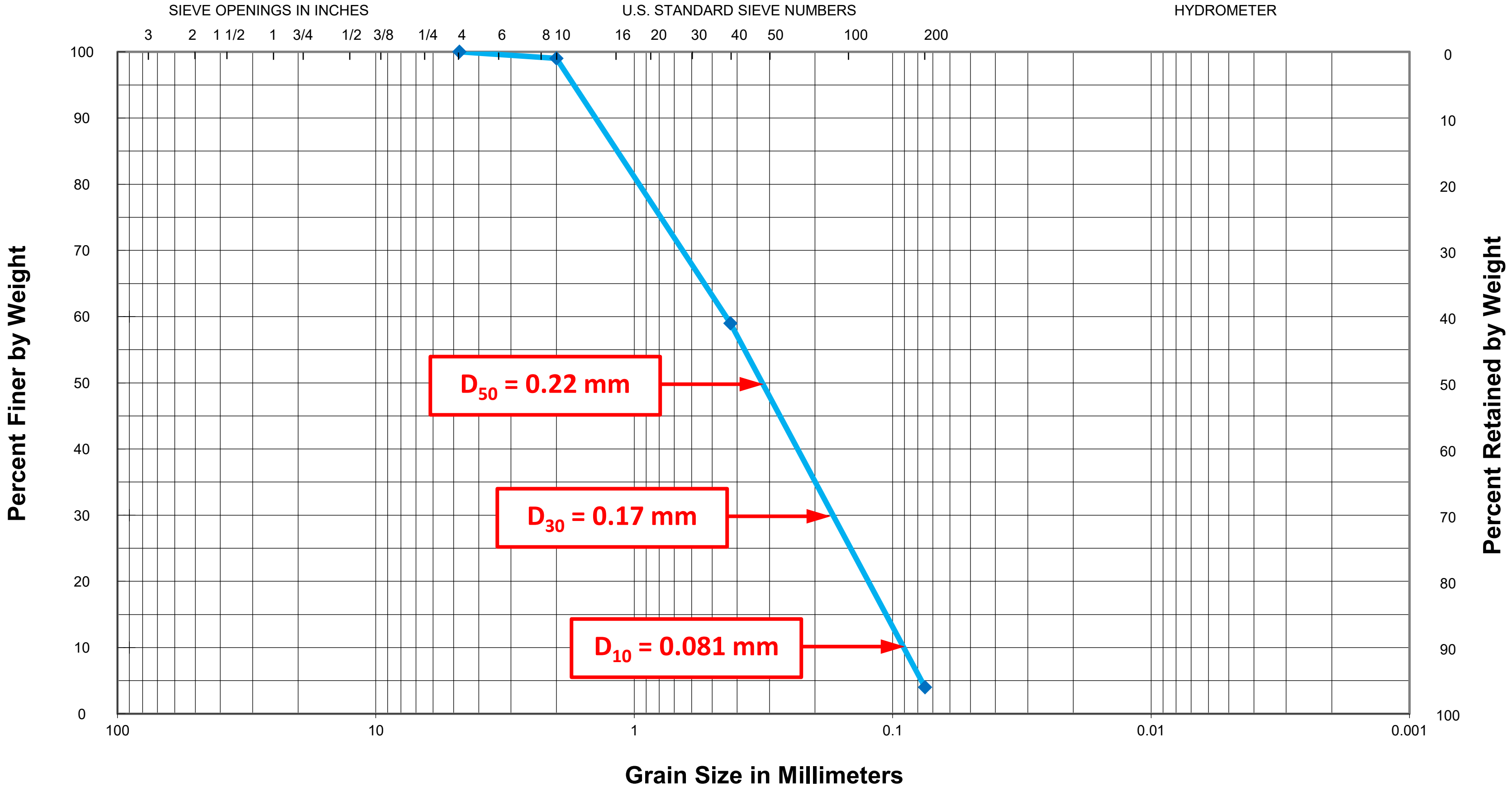
Sample: Boring A4, 29-30 ft
Description: Dark gray silty fine to medium SAND w/ occasional clayey sand pockets

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.22 mm

D_{30} = 0.17 mm

D_{10} = 0.081 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

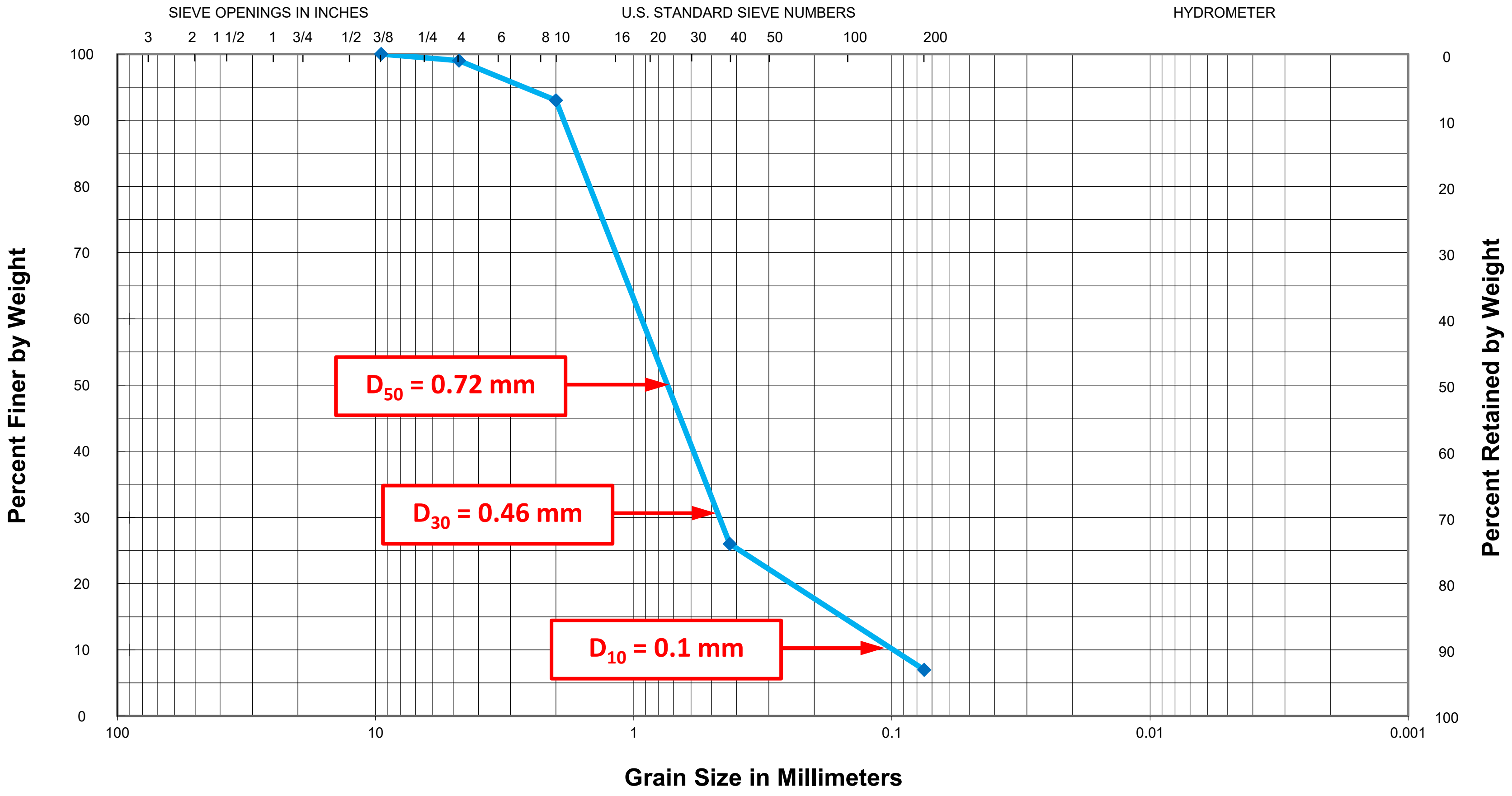
Sample: Boring A4, 44-45 ft
Description: Grayish tan fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

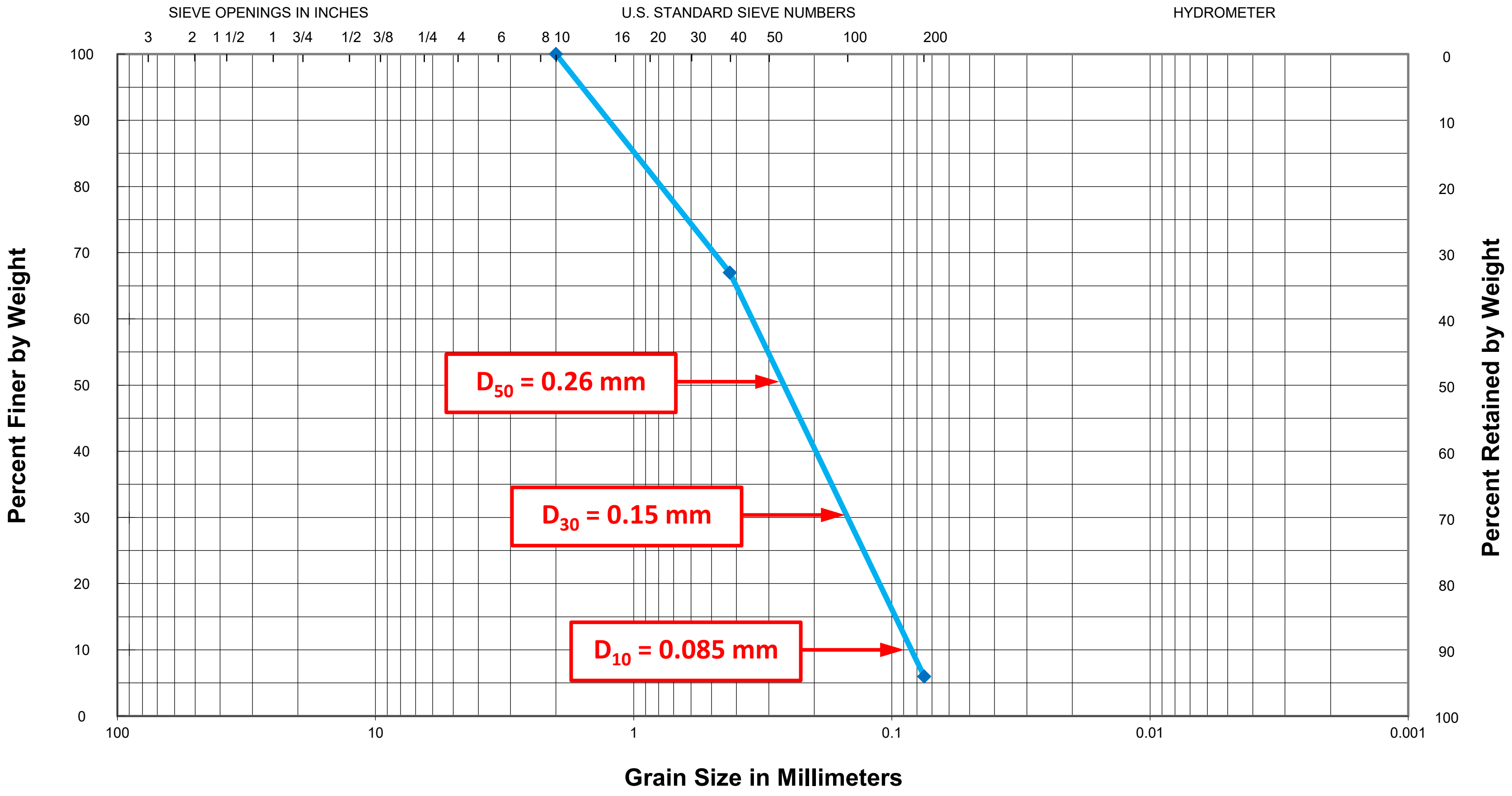
Sample: Boring A4, 74-75 ft
Description: Grayish tan fine to medium SAND, slightly silty w/ trace coarse sand

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

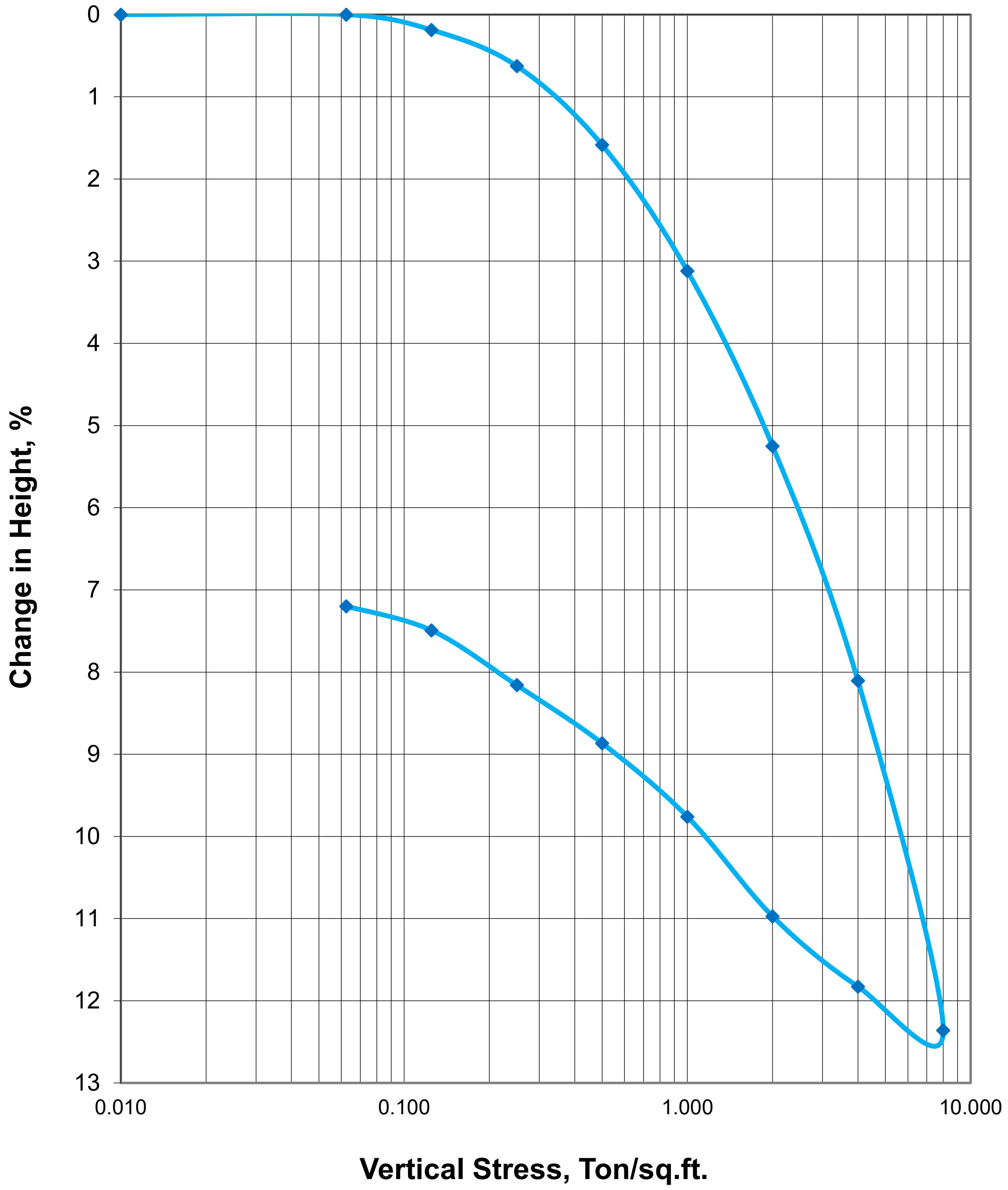
Sample: Boring A4, 84-85 ft
Description: Tan fine to medium SAND, slightly silty

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

CONSOLIDATION TEST RESULTS
(ASTM D 2435)

Project: 101124 Hwy 135 over Dead Timber Lake
GHBW Job Number: 23-031
Boring: A1
Sample Depth: 19-19.5 ft
Description: Gray and reddish tan CLAY
USCS Classification: CH
AASHTO Classification: A-7-6

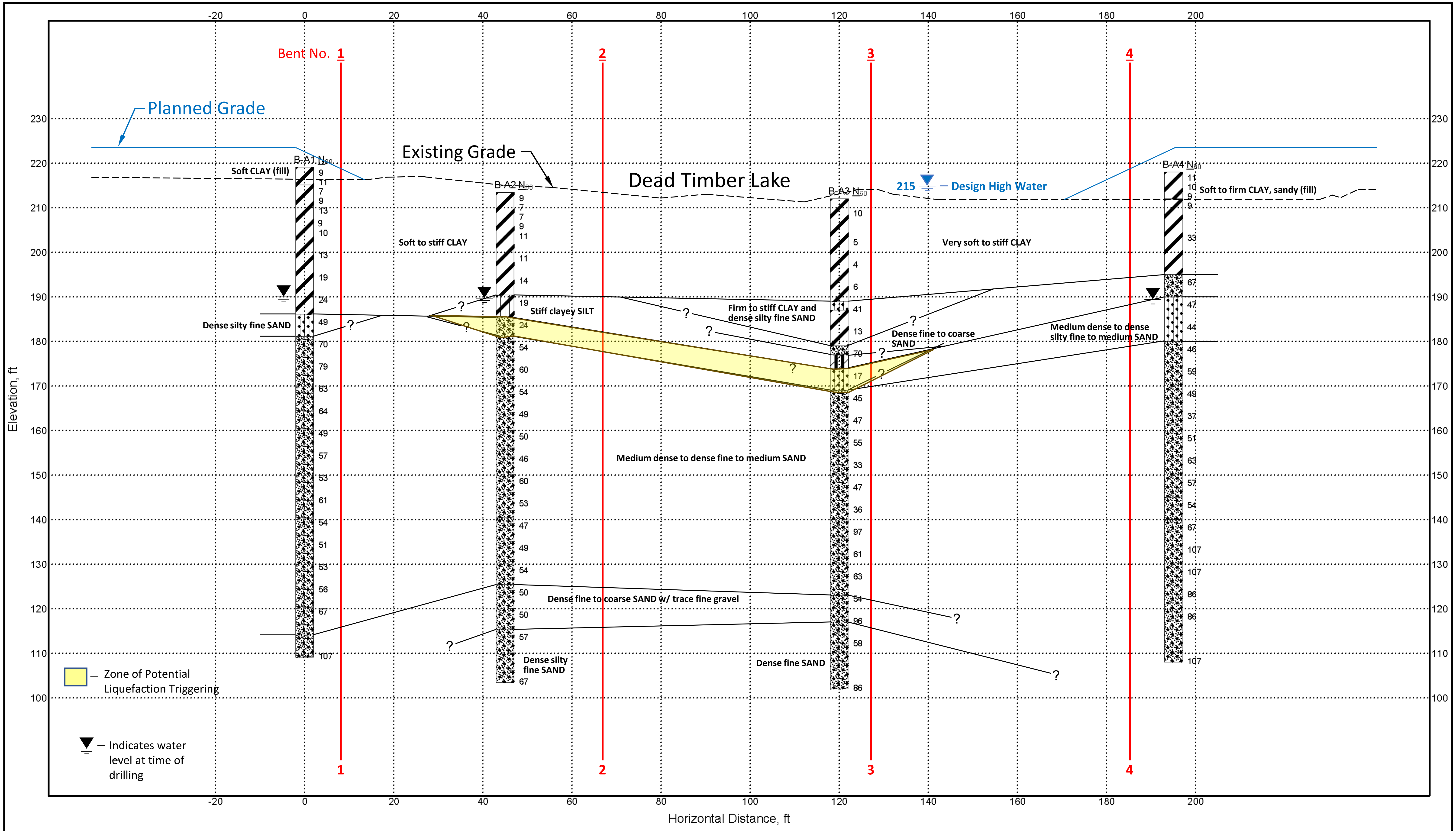
Unit Dry Weight: 86.8 lbs/cu ft
Initial Water Content: 33.1%
Final Water Content: 30.9%
Liquid Limit: 67
Plastic Limit: 25
Plasticity Index: 42
Percent Passing #200: 99%



Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS

PLATE

APPENDIX D



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

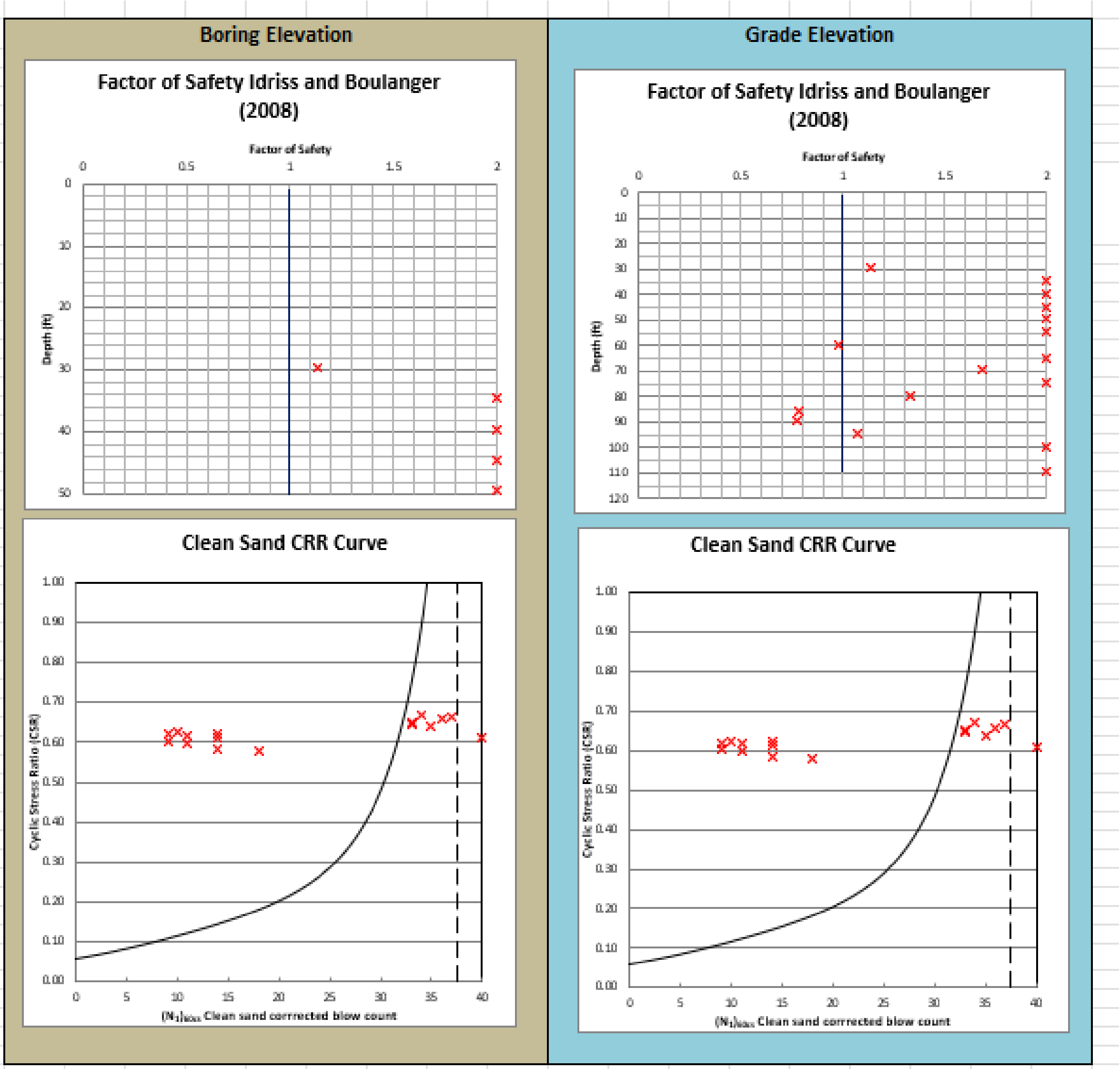
SCALE:
1" = 20' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Project Number: 23-031

Liquefaction Analysis Results

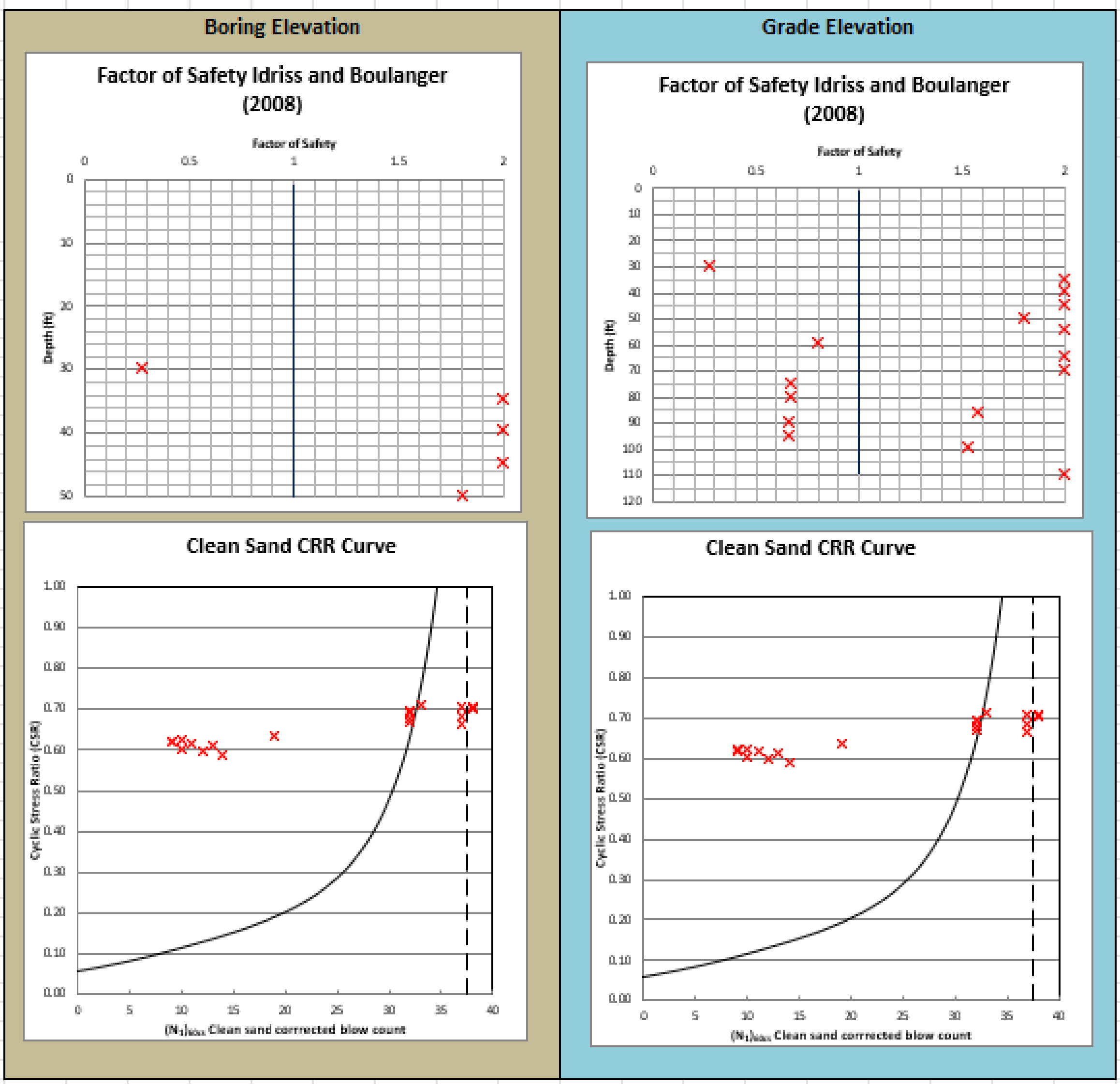
ARDOT 101124 Hwy 135 over Dead Timber Lake
Bent 1 / Boring A1
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

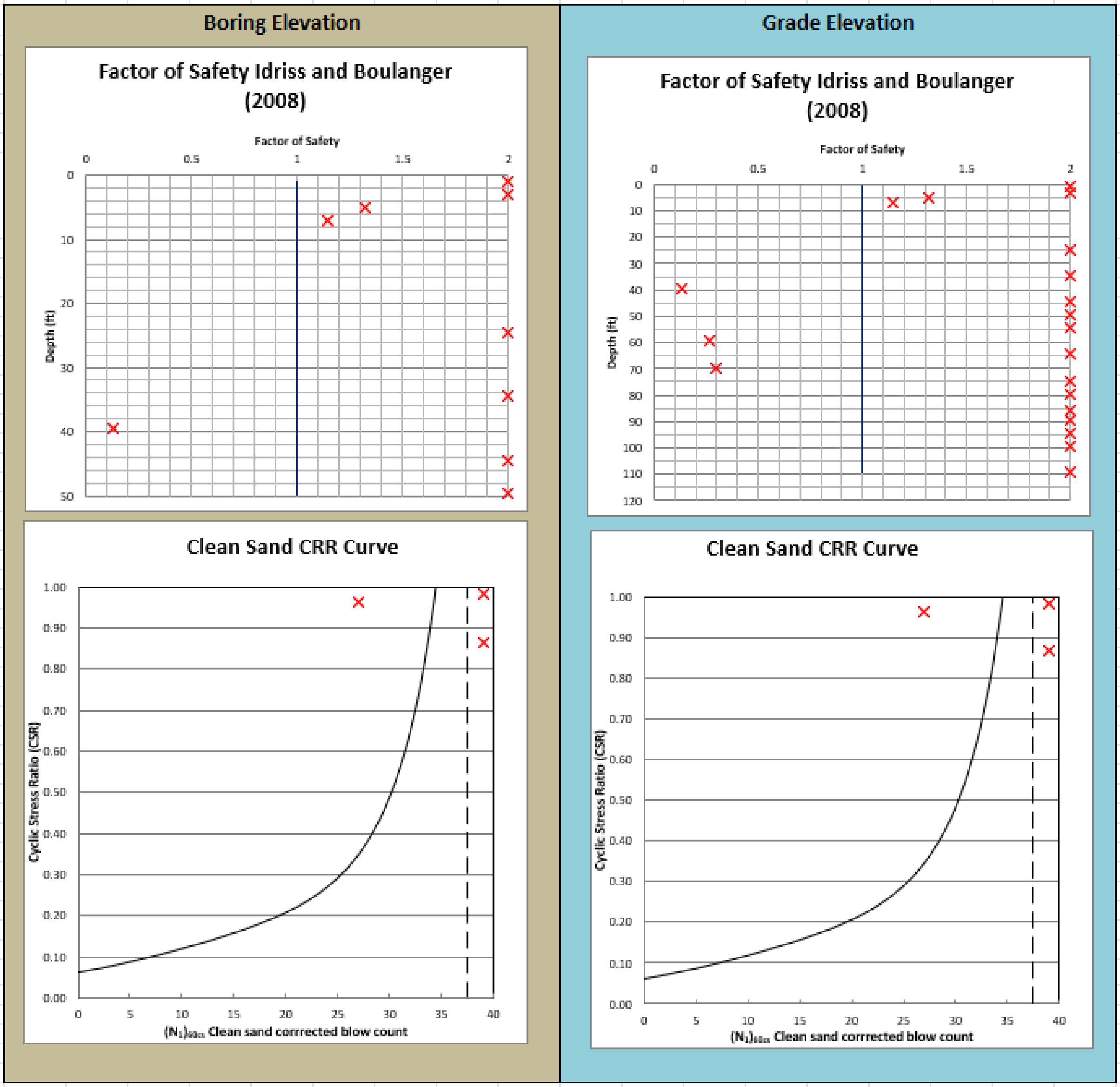
ARDOT 101124 Hwy 135 over Dead Timber Lake
Bent 2 / Boring A2
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

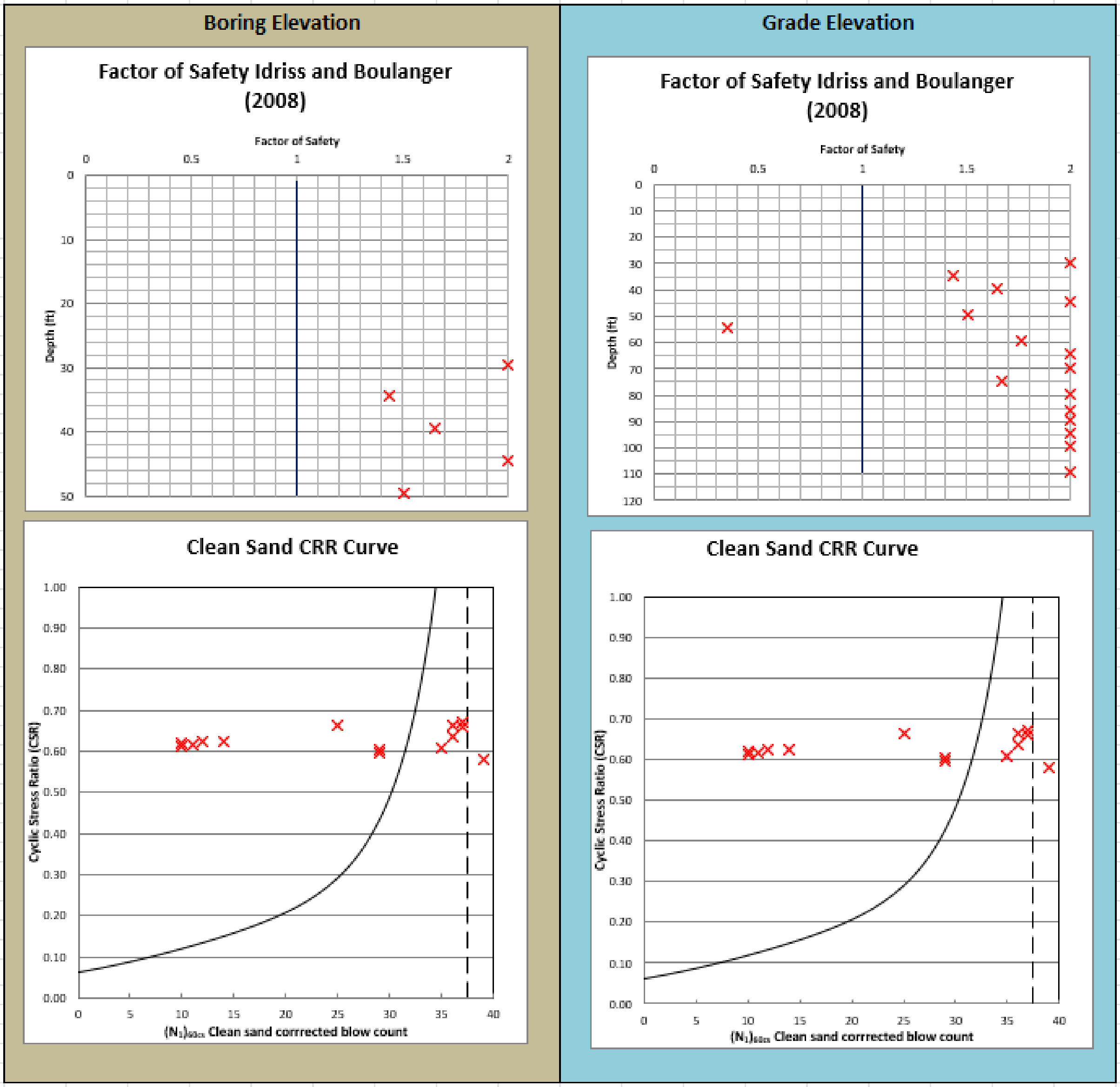
ARDOT 101124 Hwy 135 over Dead Timber Lake
Bent 3 / Boring A3
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Dead Timber Lake
Bent 4 / Boring A4
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
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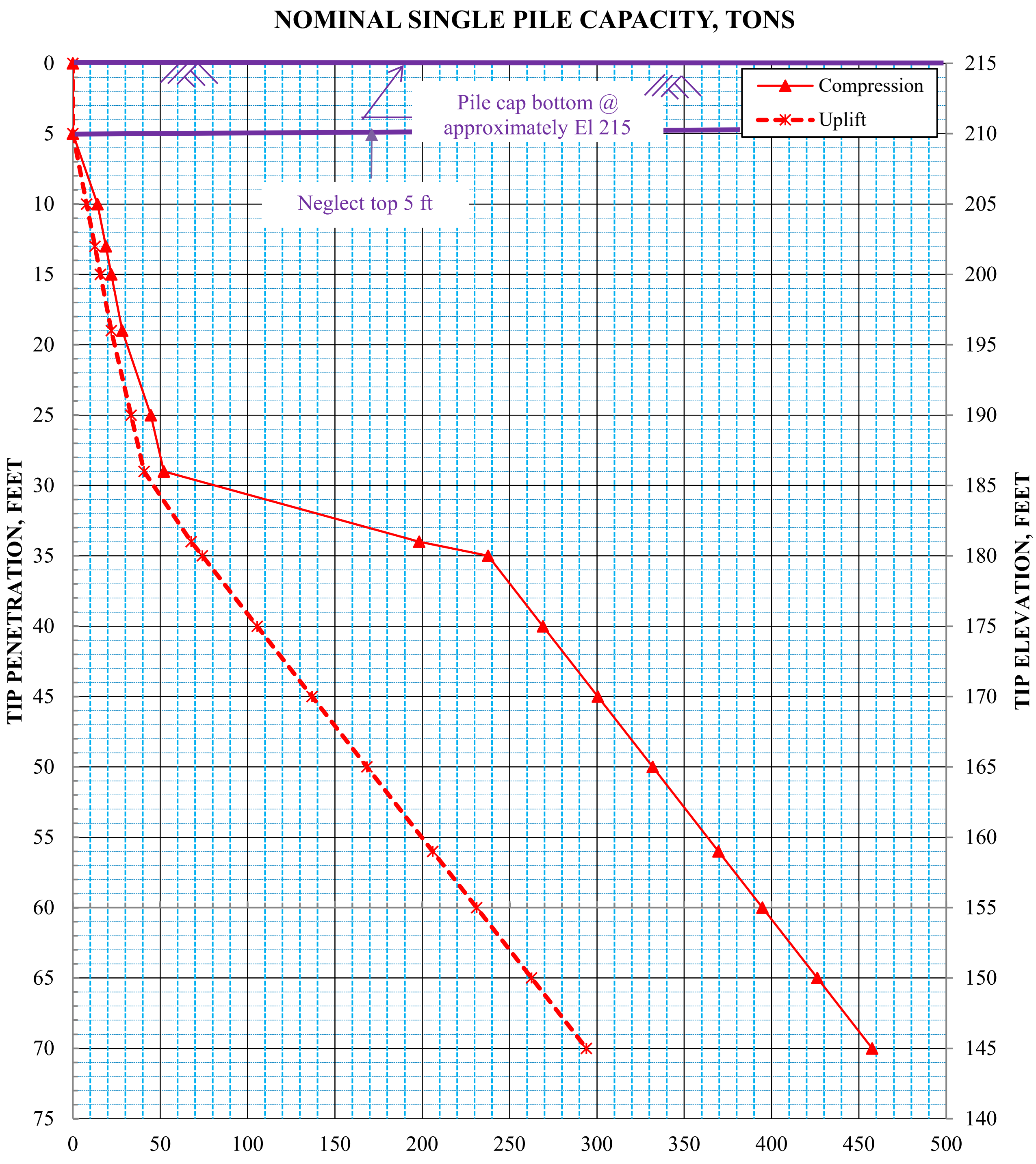
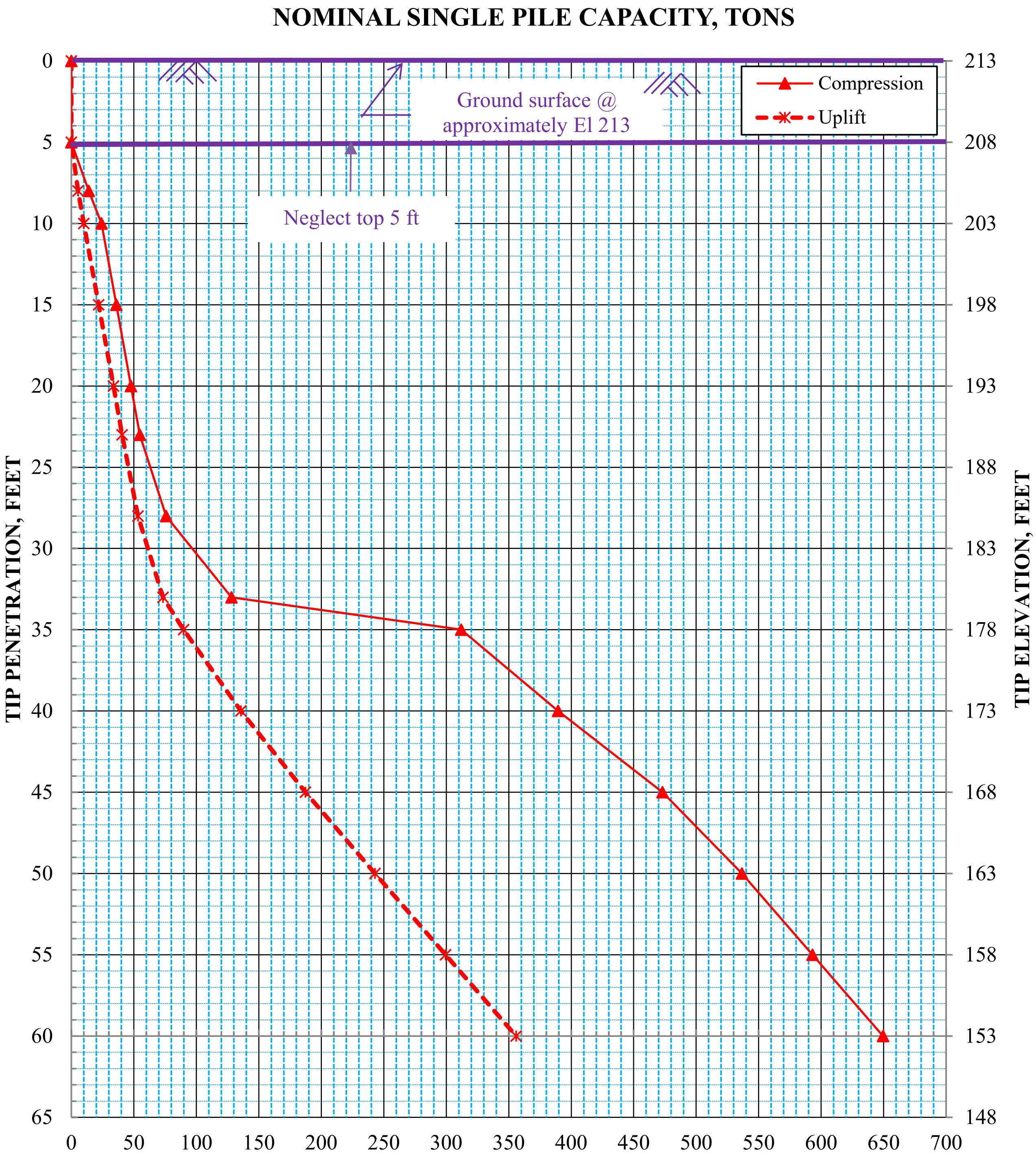
APPENDIX E



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 2 (Intermediate Bent)

24-in.-Diameter Steel Shell

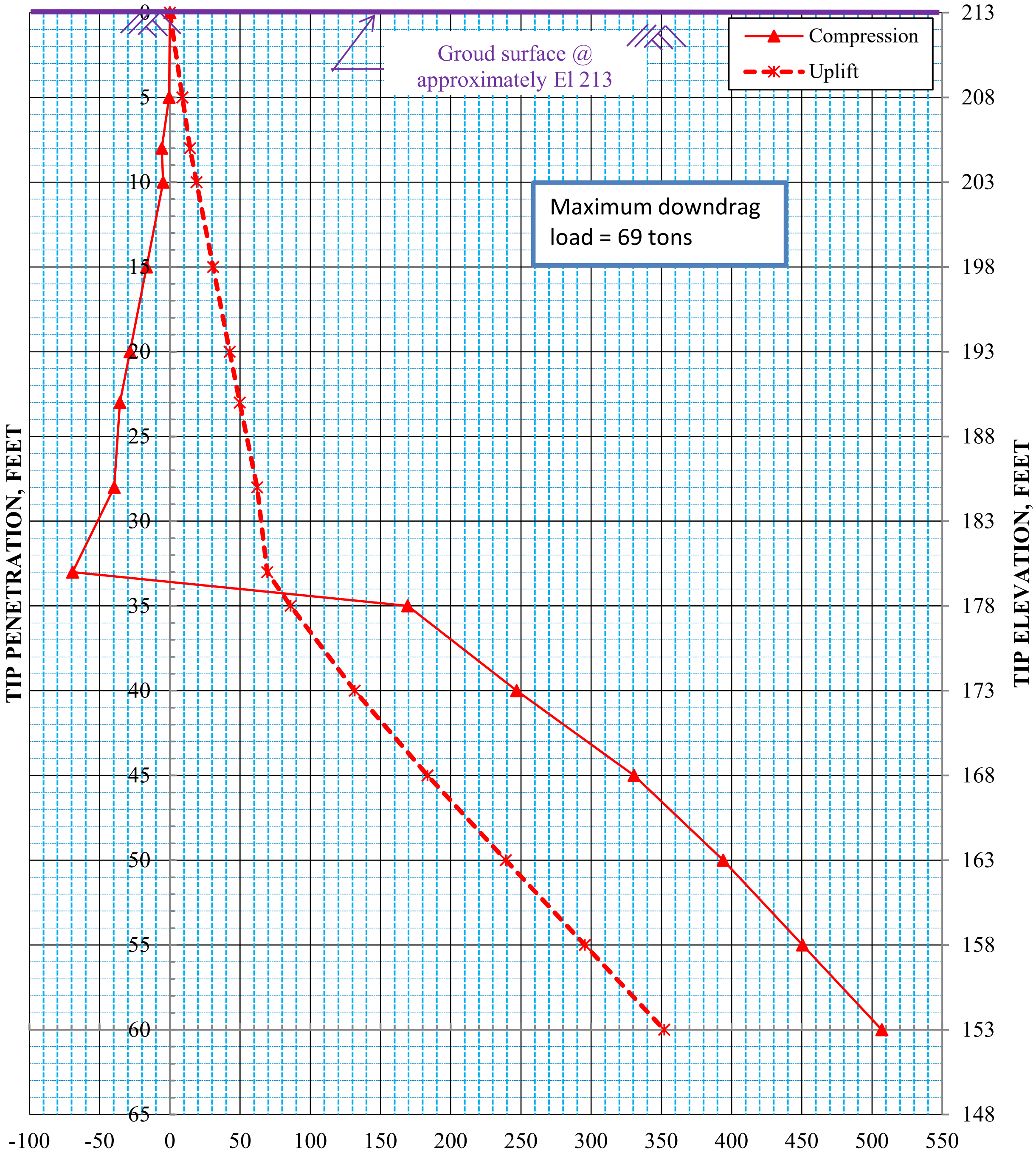
ARDOT 101124 Hwy. 135 over Dead Timber Lake

Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation

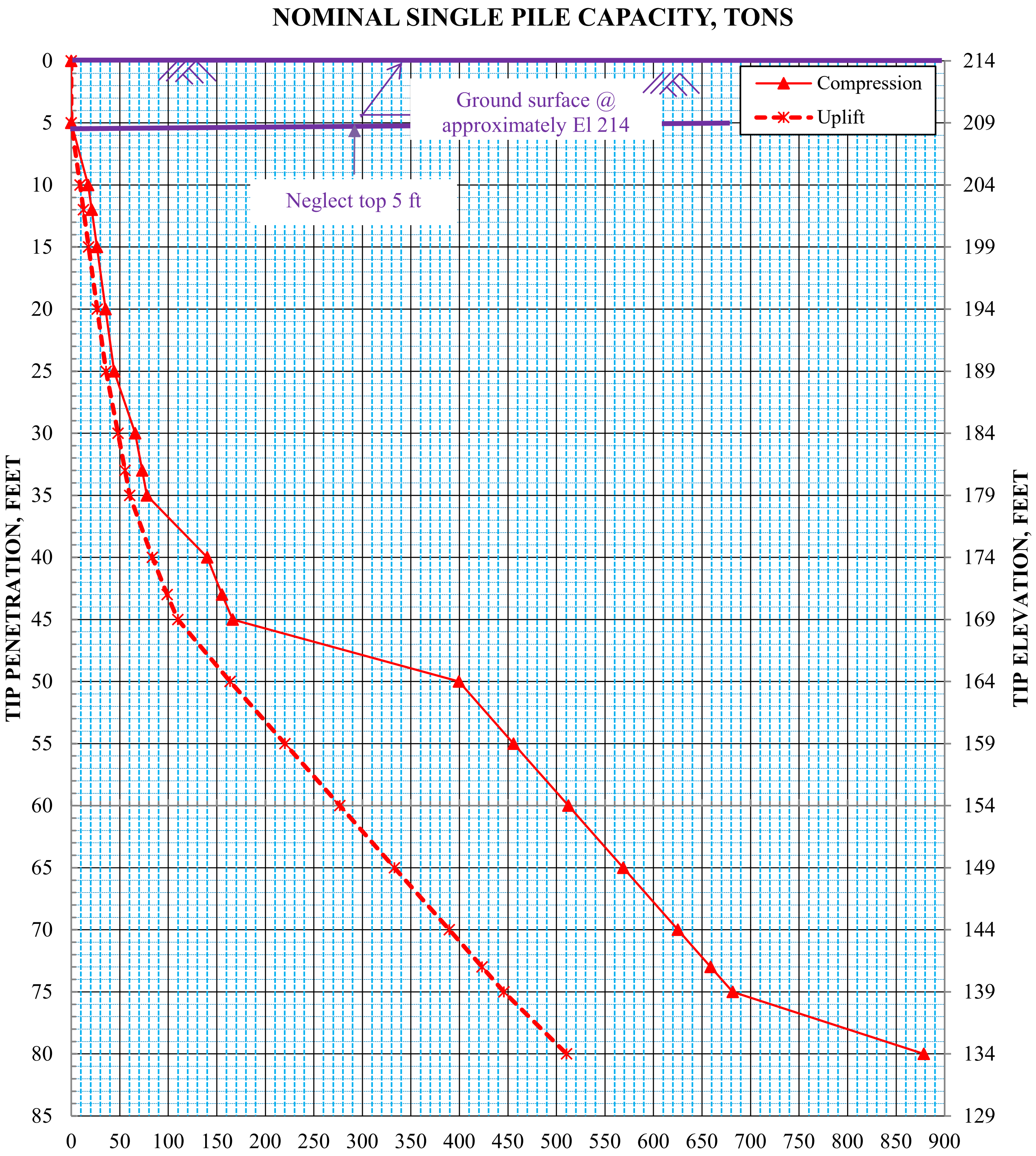
2. No downdrag

NOMINAL SINGLE PILE CAPACITY, TONS



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 2 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

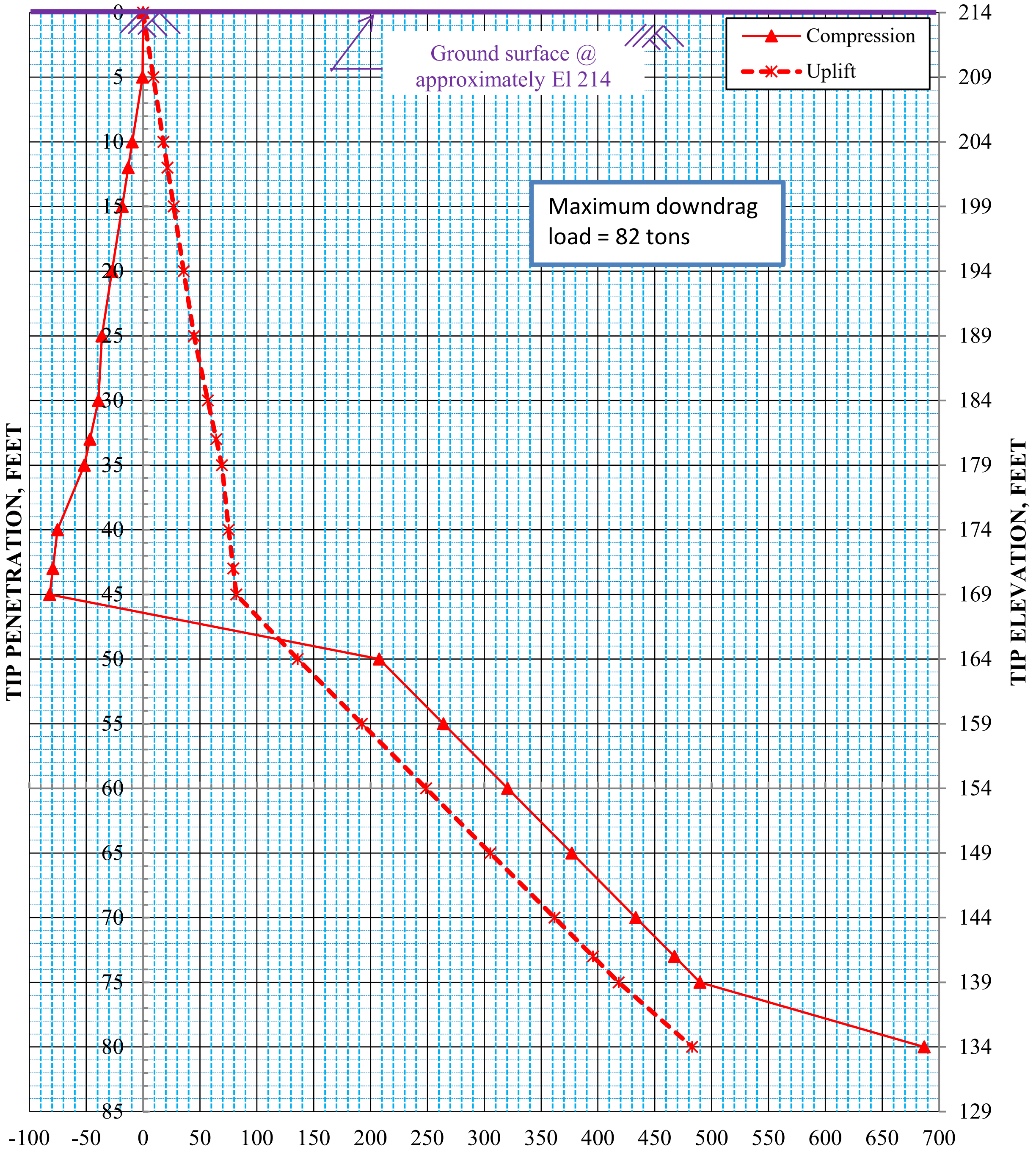
Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 180



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 3 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

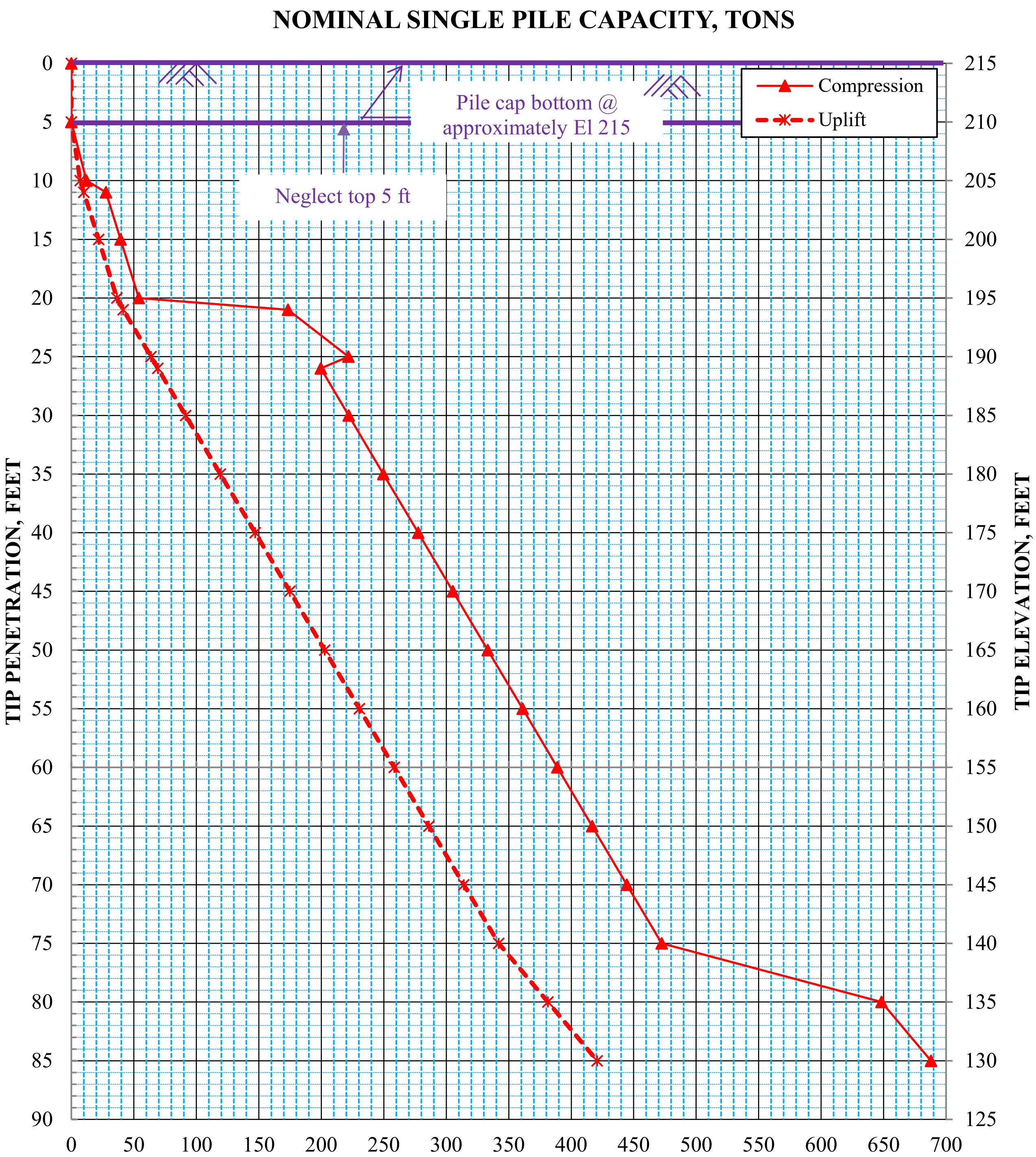
Notes: 1. Driven from channel bottom elevation
2. No downdrag

NOMINAL SINGLE PILE CAPACITY, TONS



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 3 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 169



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 4 (North Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Dead Timber Lake
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag

Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers

APPENDIX F

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Dead Timber Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft to firm CLAY**	**Stiff CLAY**	**Dense silty fine SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-19	19-29	29-34	34 and deeper
Approximate El, ft	215-196	196-186	186-181	below 181
Recommend soil type	Soft clay	Stiff clay with free water	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	63	65	68
Cohesion (c), lbs per sq ft	800	1800	0	0
Angle of internal friction (φ), °	0	0	37	38
Subgrade modulus (k), lbs per cu in.	100	500	115	125
Strain at 50% (EE50)	0.01	0.007	NA	NA

Note: Pile cap bottom at ±El 215

ARDOT 101124 Hwy. 135 over Dead Timber Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft CLAY	Firm to stiff CLAY	Stiff CLAY	Medium dense fine SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-8	8-23	23-28	28-33	33 and deeper
Approximate El, ft	213-205	205-190	190-185	185-180	below 180
Recommend soil type	Soft clay	Stiff clay with free water	Stiff clay with free water	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	58	63	56	65
Cohesion (c), lbs per sq ft	600	1000	1600	0	0
Angle of internal friction (φ), °	0	0	0	32	37
Subgrade modulus (k), lbs per cu in.	100	300	500	50	115
Strain at 50% (EE50)	0.01	0.009	0.007	NA	NA

Note: Ground surface at ±El 213

Seismic Loading with Liquefaction
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft CLAY	Firm to stiff CLAY	Stiff CLAY	Medium dense fine SAND (liquefiable)	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-8	8-23	23-28	28-33	33 and deeper
Approximate El, ft	213-205	205-190	190-185	185-180	below 180
Recommend soil type	Soft clay	Stiff clay with free water	Stiff clay with free water	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	58	63	56	65
Cohesion (c), lbs per sq ft	600	1000	1600	0	0
Angle of internal friction (φ), °	0	0	0	8	37
Subgrade modulus (k), lbs per cu in.	100	300	500	20	115
Strain at 50% (EE50)	0.01	0.009	0.007	NA	NA

Note: Ground surface at ±El 213

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Dead Timber Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft to firm CLAY**	**Stiff CLAY**	**Medium dense silty fine SAND**	**Dense fine to medium SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-25	25-35	35-45	45-75	75 and deeper
Approximate El, ft	214-189	189-179	179-169	169-139	below 139
Recommend soil type	Soft clay	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	63	56	65	68
Cohesion (c), lbs per sq ft	600	1250	0	0	0
Angle of internal friction (φ), °	0	0	32	37	38
Subgrade modulus (k), lbs per cu in.	100	500	50	115	125
Strain at 50% (EE50)	0.01	0.007	NA	NA	NA

Note: Ground surface at ±El 214

Seismic Loading with Liquefaction
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft to firm CLAY**	**Stiff CLAY**	**Medium dense silty fine SAND (liquefiable)**	**Dense fine to medium SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-25	25-35	35-45	45-75	75 and deeper
Approximate El, ft	214-189	189-179	179-169	169-139	below 139
Recommend soil type	Soft clay	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	63	56	65	68
Cohesion (c), lbs per sq ft	600	1250	0	0	0
Angle of internal friction (φ), °	0	0	8	37	38
Subgrade modulus (k), lbs per cu in.	100	500	20	115	125
Strain at 50% (EE50)	0.01	0.007	NA	NA	NA

Note: Ground surface at ±El 214

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Dead Timber Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

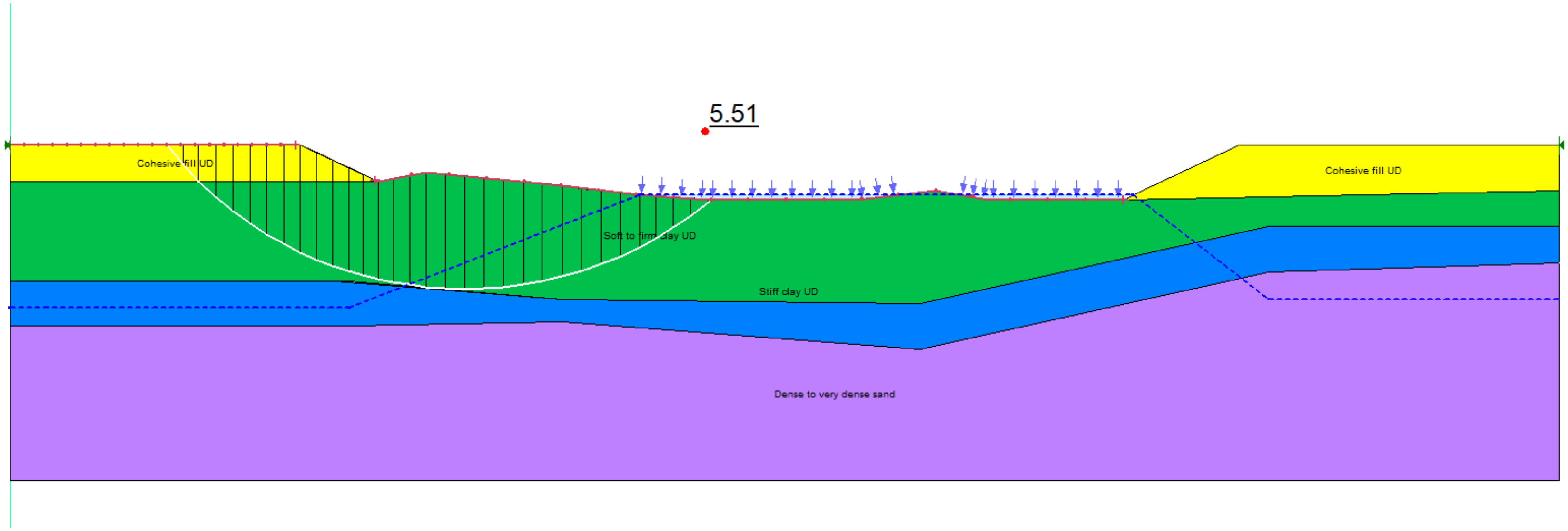
Generalized Stratigraphy	**Soft to firm fine sandy CLAY**	**Very stiff CLAY**	**Dense fine SAND**	**Dense fine to medium SAND**	**Dense to very dense fine SAND**
Depth below pile cap bottom, ft	0-10	10-20	20-25	25-75	75 and deeper
Approximate El, ft	215-205	205-195	195-190	190-140	below 140
Recommend soil type	Soft clay	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	120	68	65	68
Cohesion (c), lbs per sq ft	800	2800	0	0	0
Angle of internal friction (φ), °	0	0	38	37	40
Subgrade modulus (k), lbs per cu in.	100	1000	125	115	130
Strain at 50% (EE50)	0.01	0.005	NA	NA	NA

Note: Pile cap bottom at ±El 215

APPENDIX G

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Dead Timber Lake
GHBW Job No. 23-031
Poinsett County, Arkansas

	Design Loading Condition	Calculated Minimum Factor of Safety
South End Slope (Bent 1) (2H:1V)	End of Construction	5.51
	Long Term	1.88
	Rapid Drawdown from El 215 to El 213	2.21
	Seismic ($k_h = A_S/2 = 0.477$)	0.94
	Lateral Spread	5.51
South Side Slope (Bent 1) (3H:1V)	End of Construction	7.31
	Long Term	2.10
	Rapid Drawdown from El 215 to Existing Grade	2.19
	Seismic ($k_h = A_S/2 = 0.477$)	1.35
North End Slope (Bent 4) (2H:1V)	End of Construction	6.56
	Long Term	1.61
	Rapid Drawdown from El 215 to El 213	1.45
	Seismic ($k_h = A_S/2 = 0.477$)	1.80
North Side Slope (Bent 4) (3H:1V)	End of Construction	6.73
	Long Term	1.95
	Rapid Drawdown from El 215 to Existing Grade	1.72
	Seismic ($k_h = A_S/2 = 0.477$)	1.95



Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
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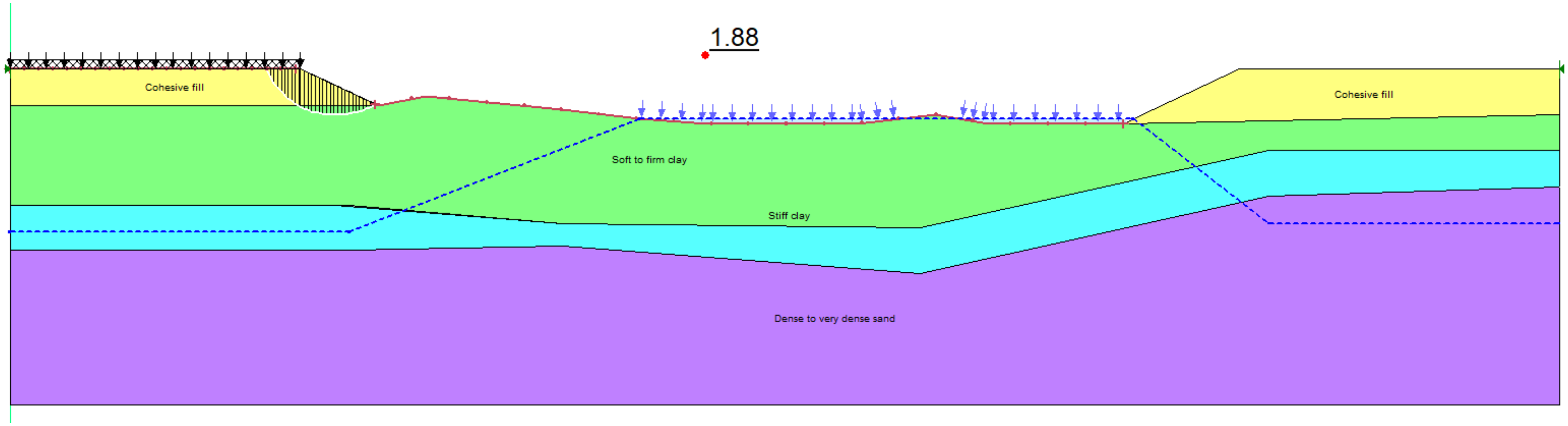
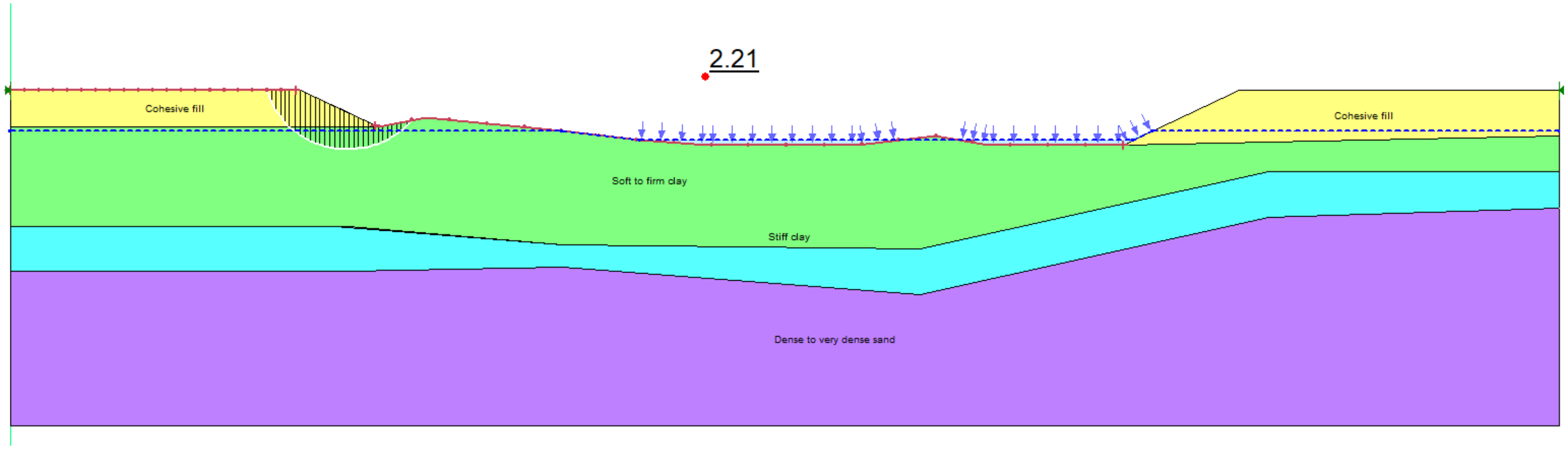


Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=8 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – Rapid Drawdown Condition from El 215 to El 213
Bent 1 End Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
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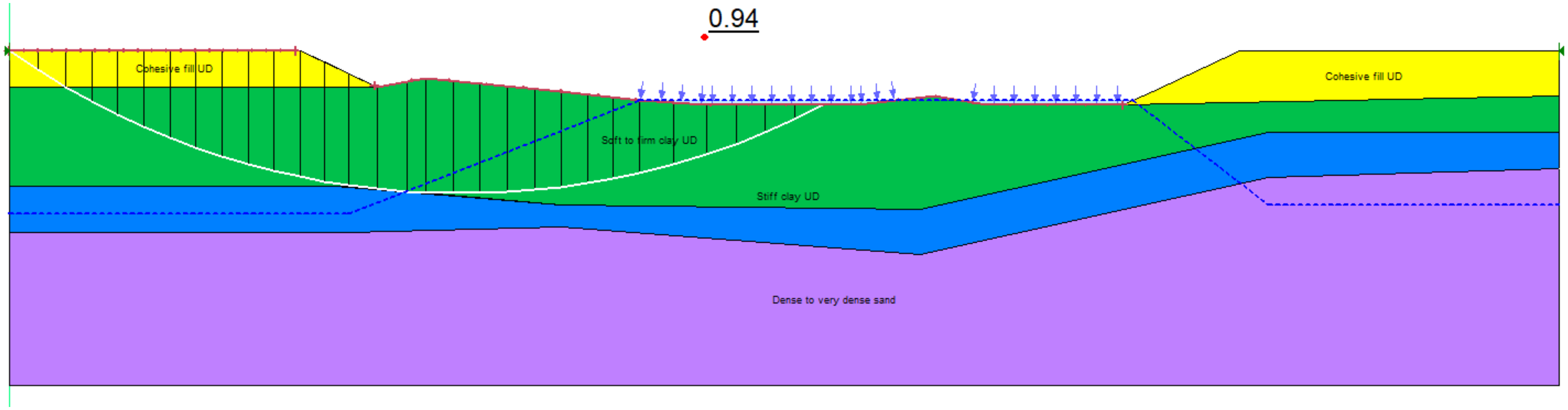

Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.477$)
Bent 1 End Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

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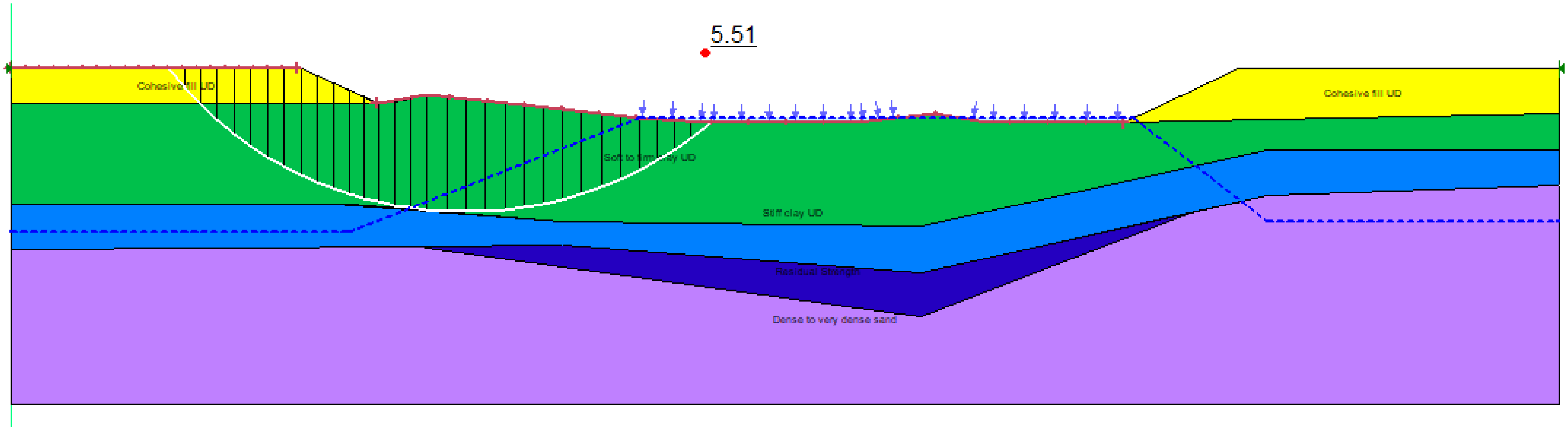
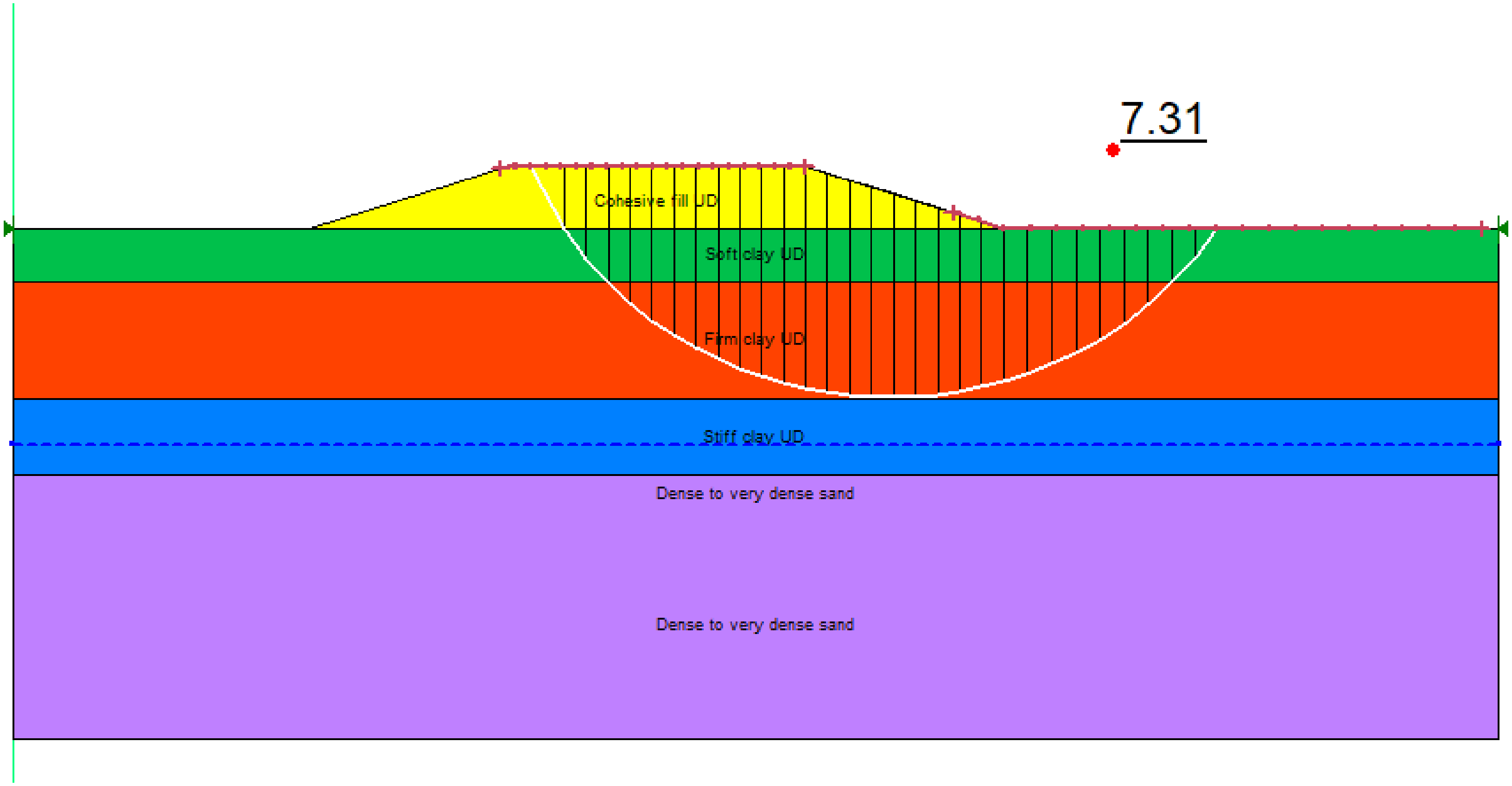


Results of Stability Analyses – Lateral Spread
Bent 1 End Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – End of Construction
Bent 1 Side Slope
3H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

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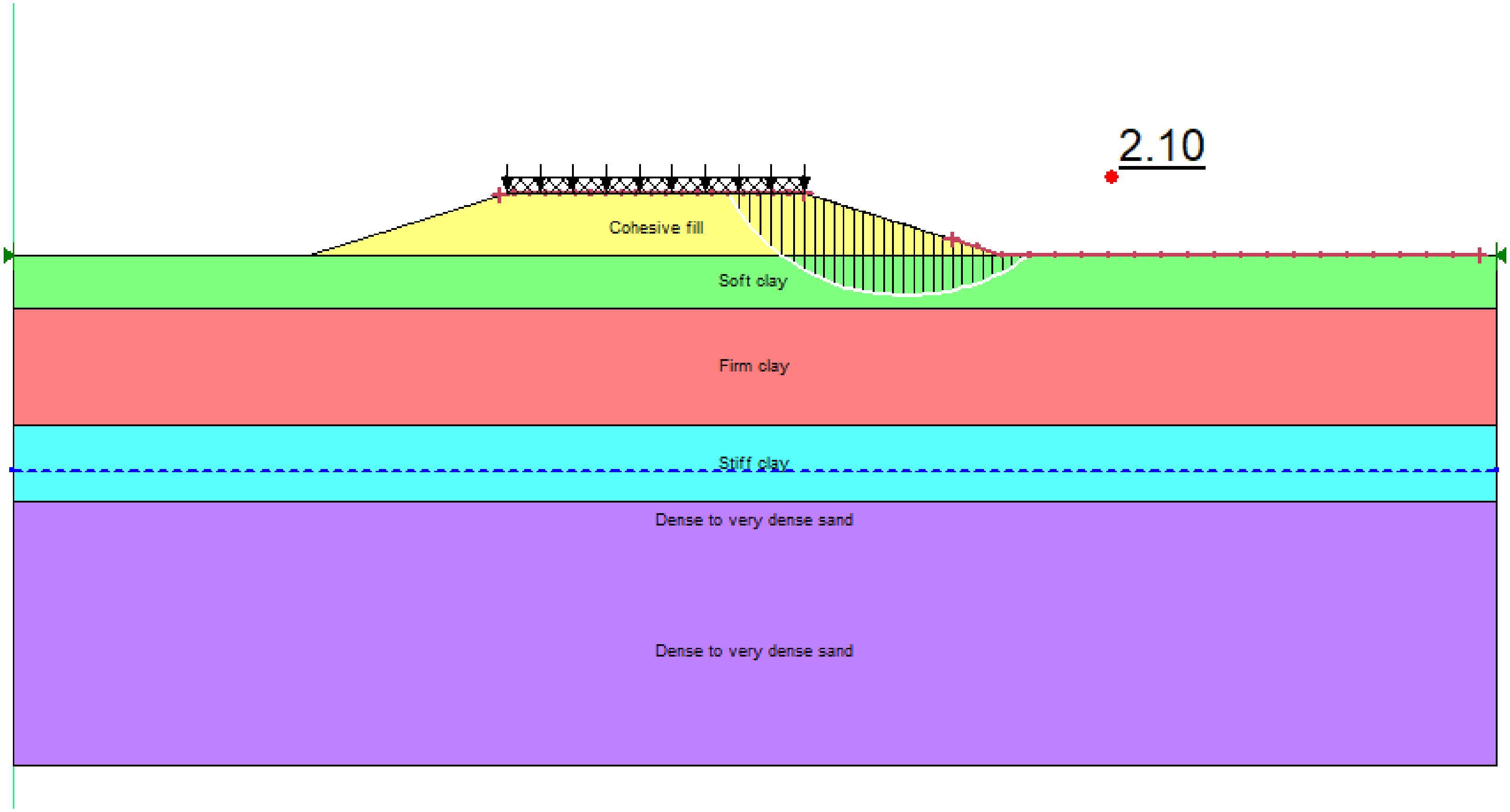
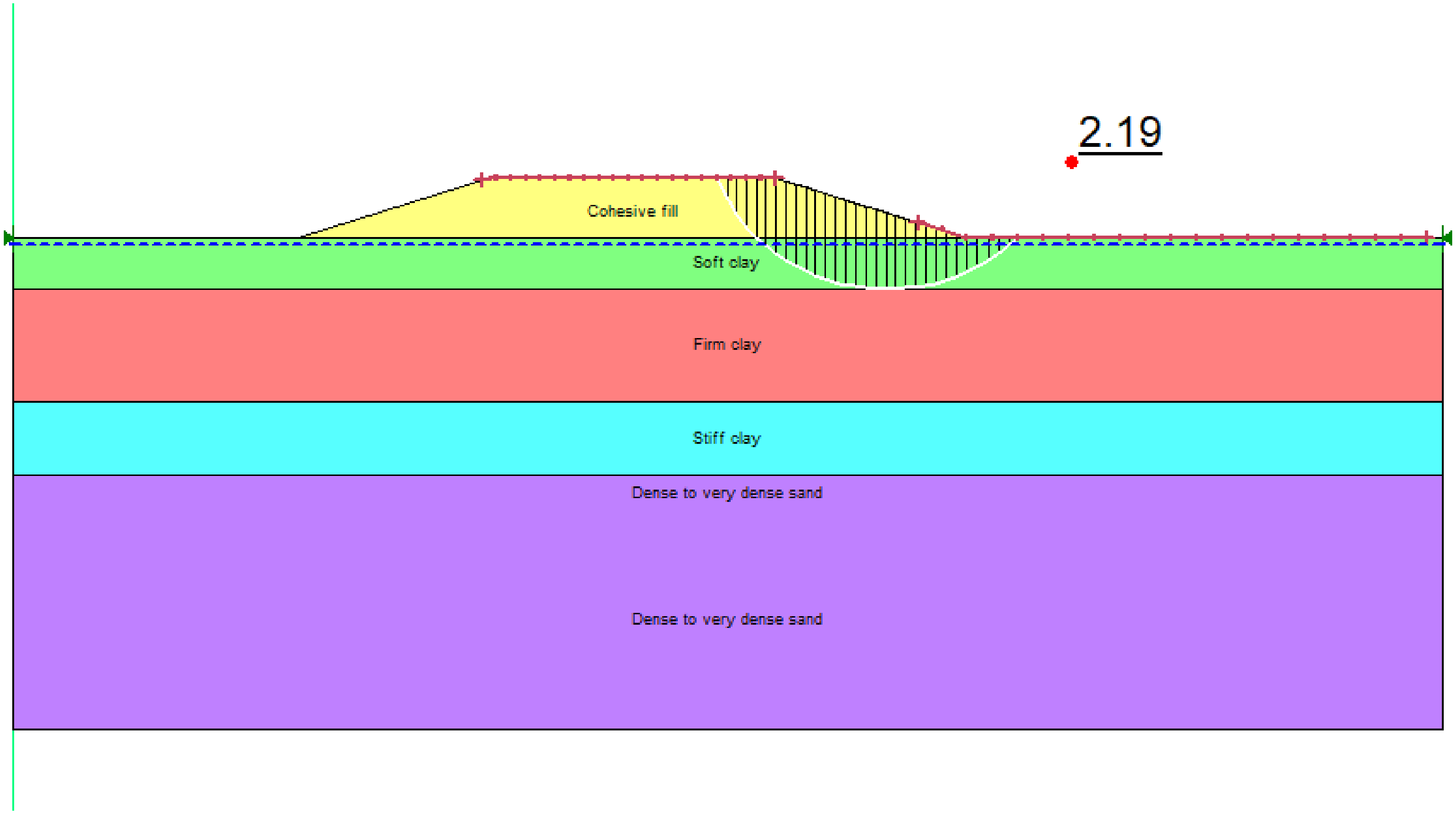


Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
3H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – Rapid Drawdown El 215 to Existing Grade
Bent 1 Side Slope
3H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

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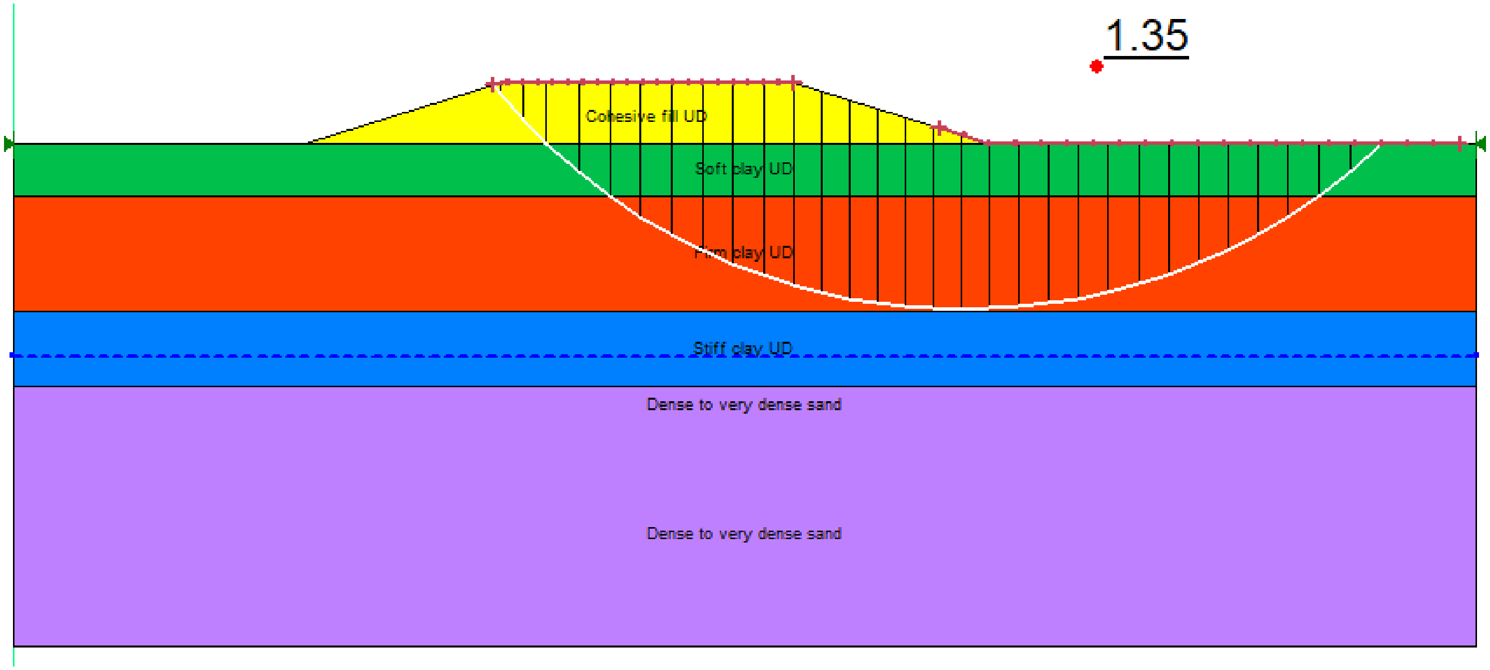

Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.477$)
Bent 1 Side Slope
3H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

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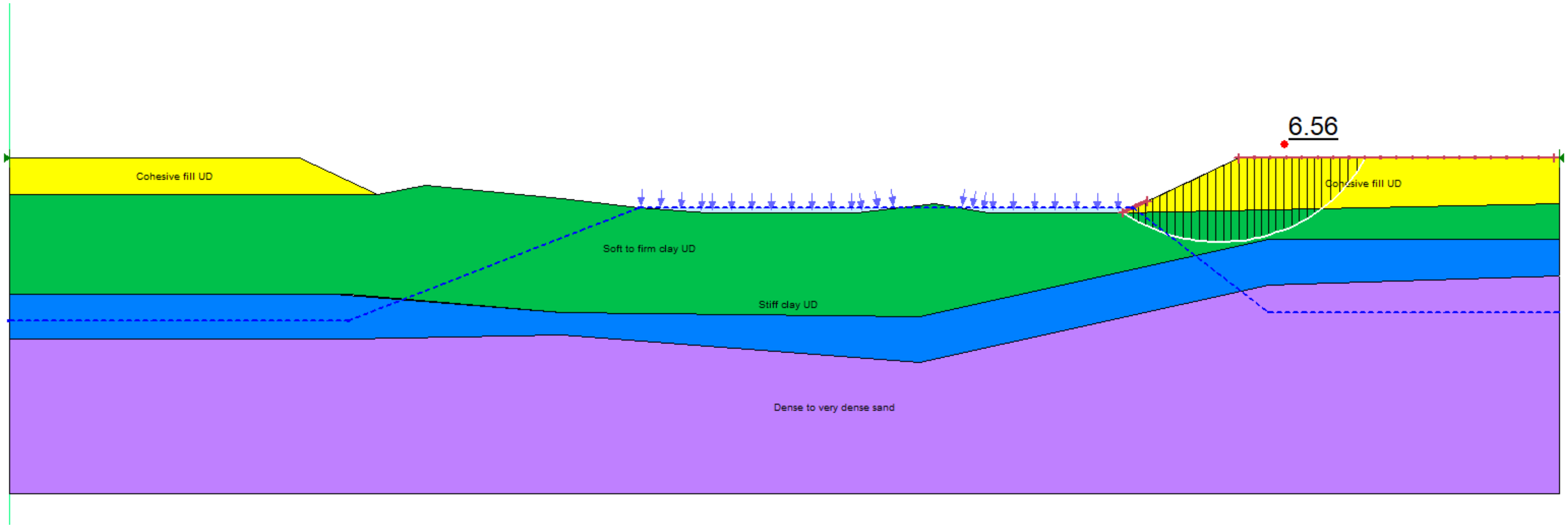
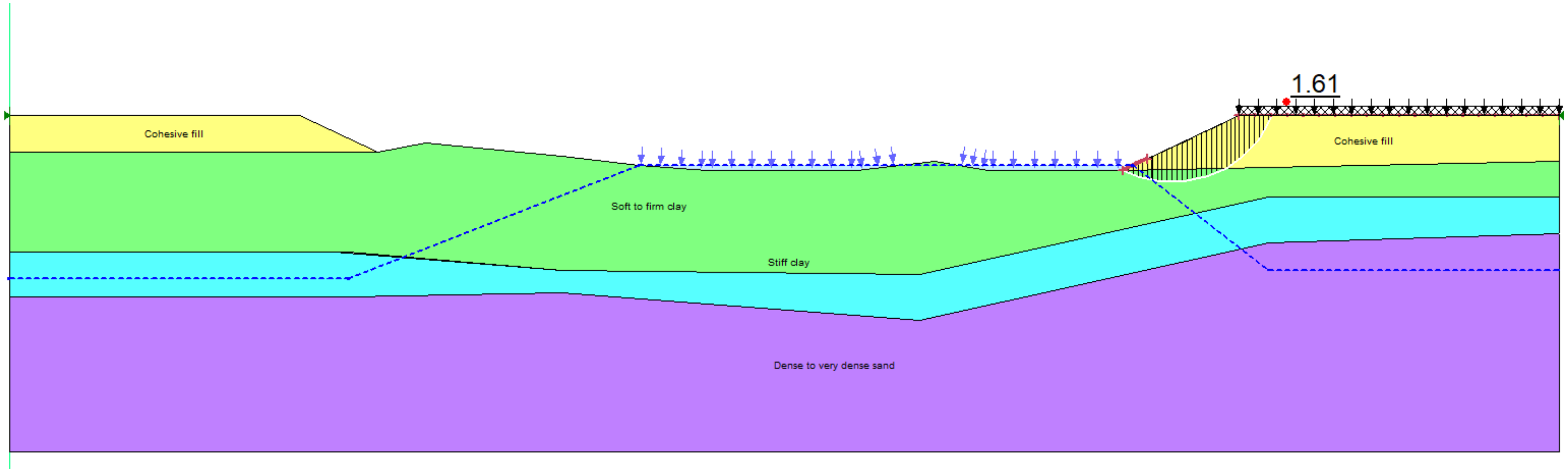
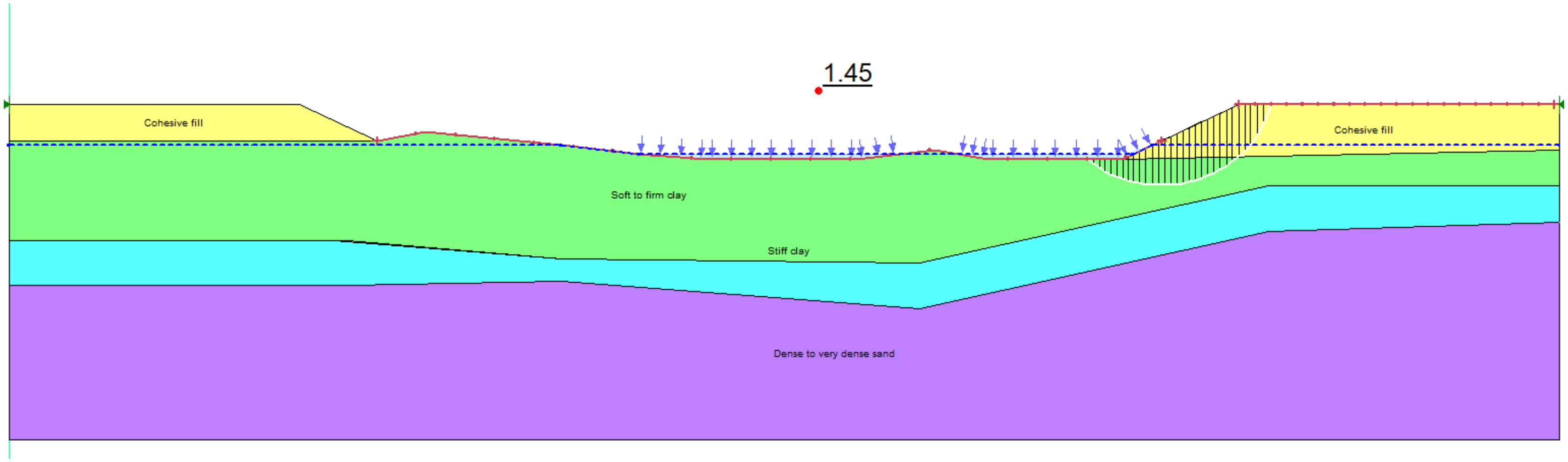


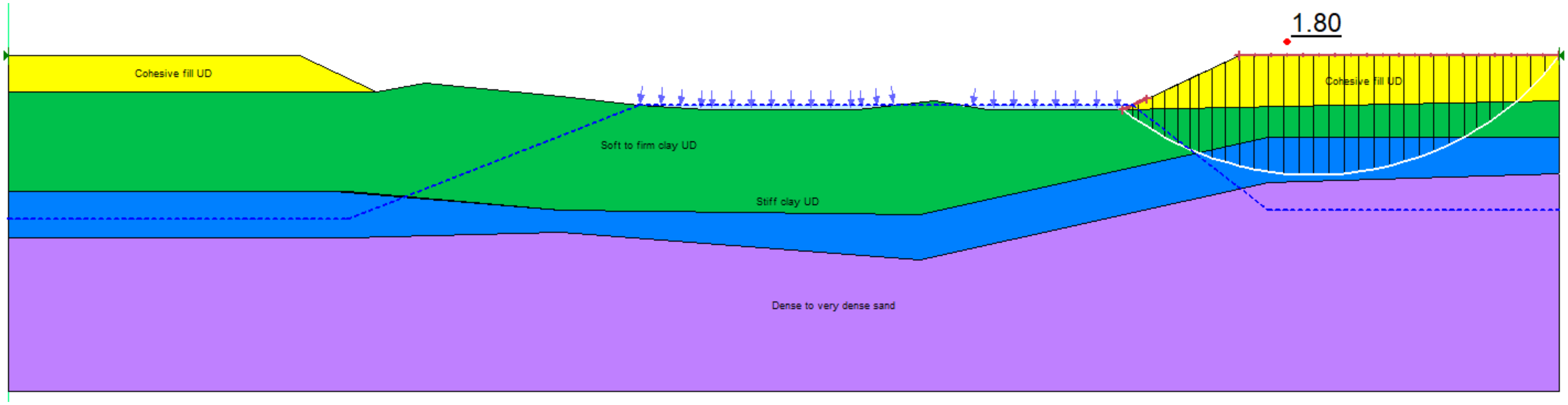
Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



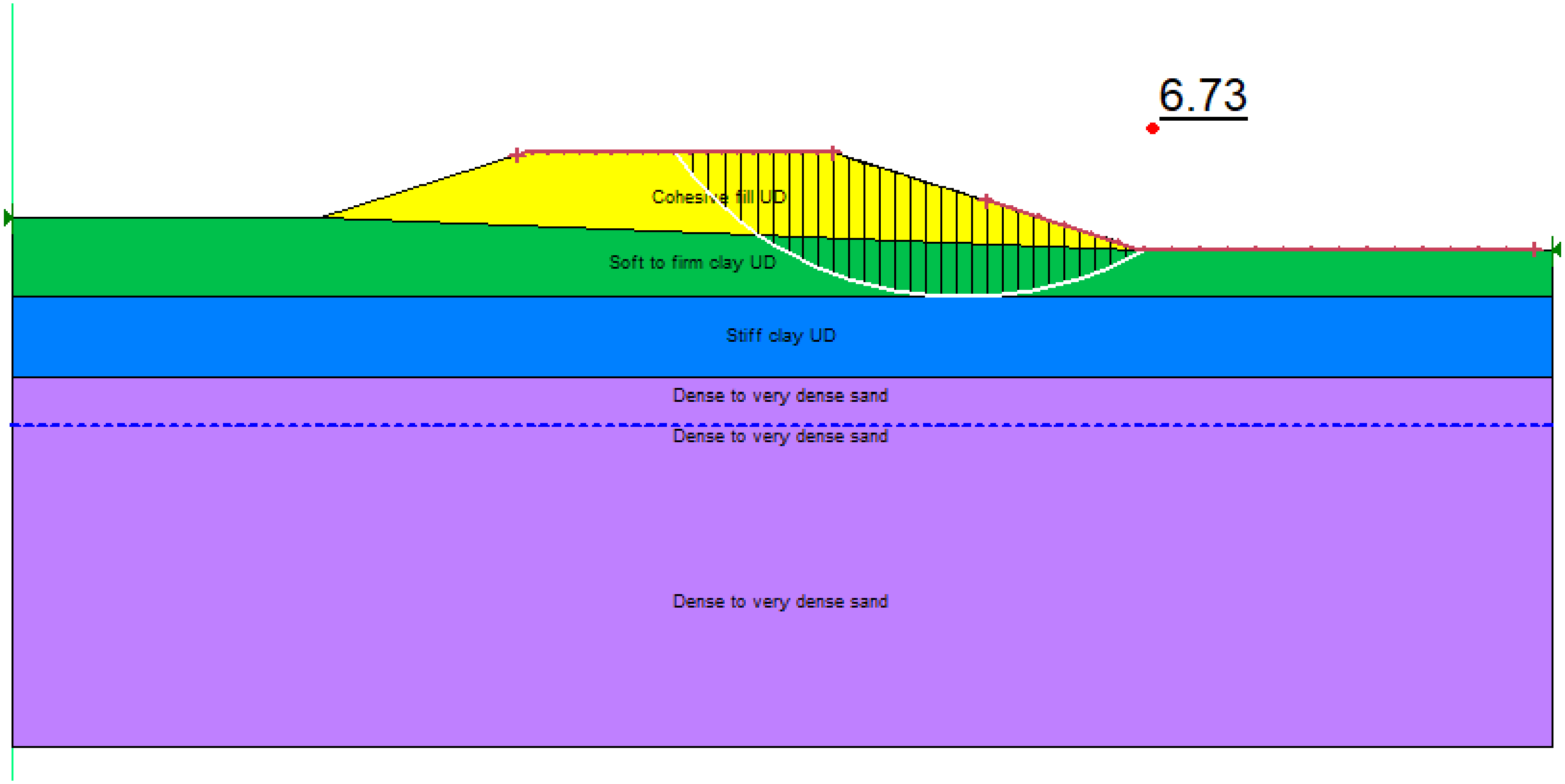
Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



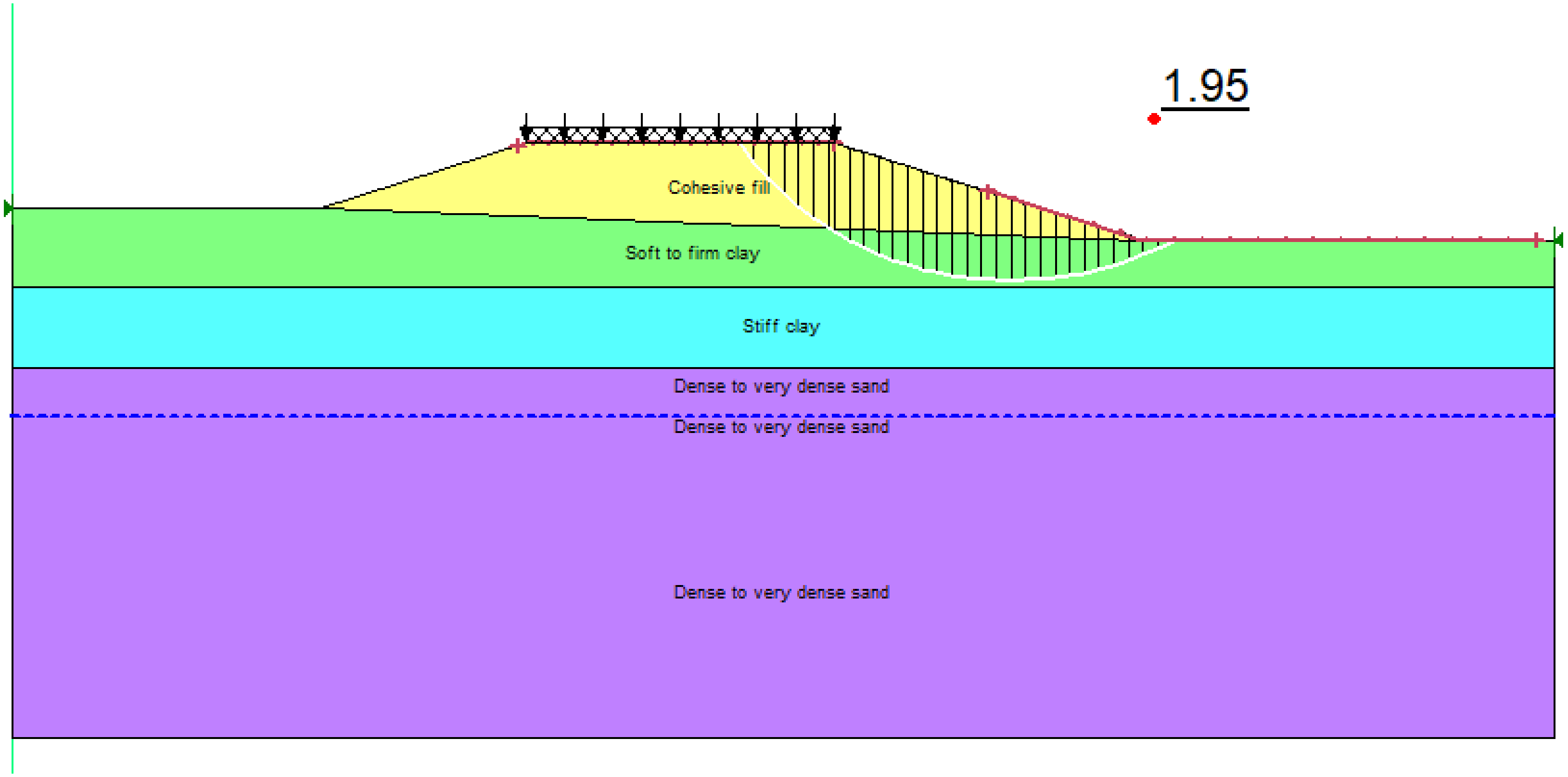
Results of Stability Analyses – Rapid Drawdown Condition, El 215 to El 213
Bent 4 End Slope
2H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.477$)
Bent 4 End Slope
2H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – End of Construction
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

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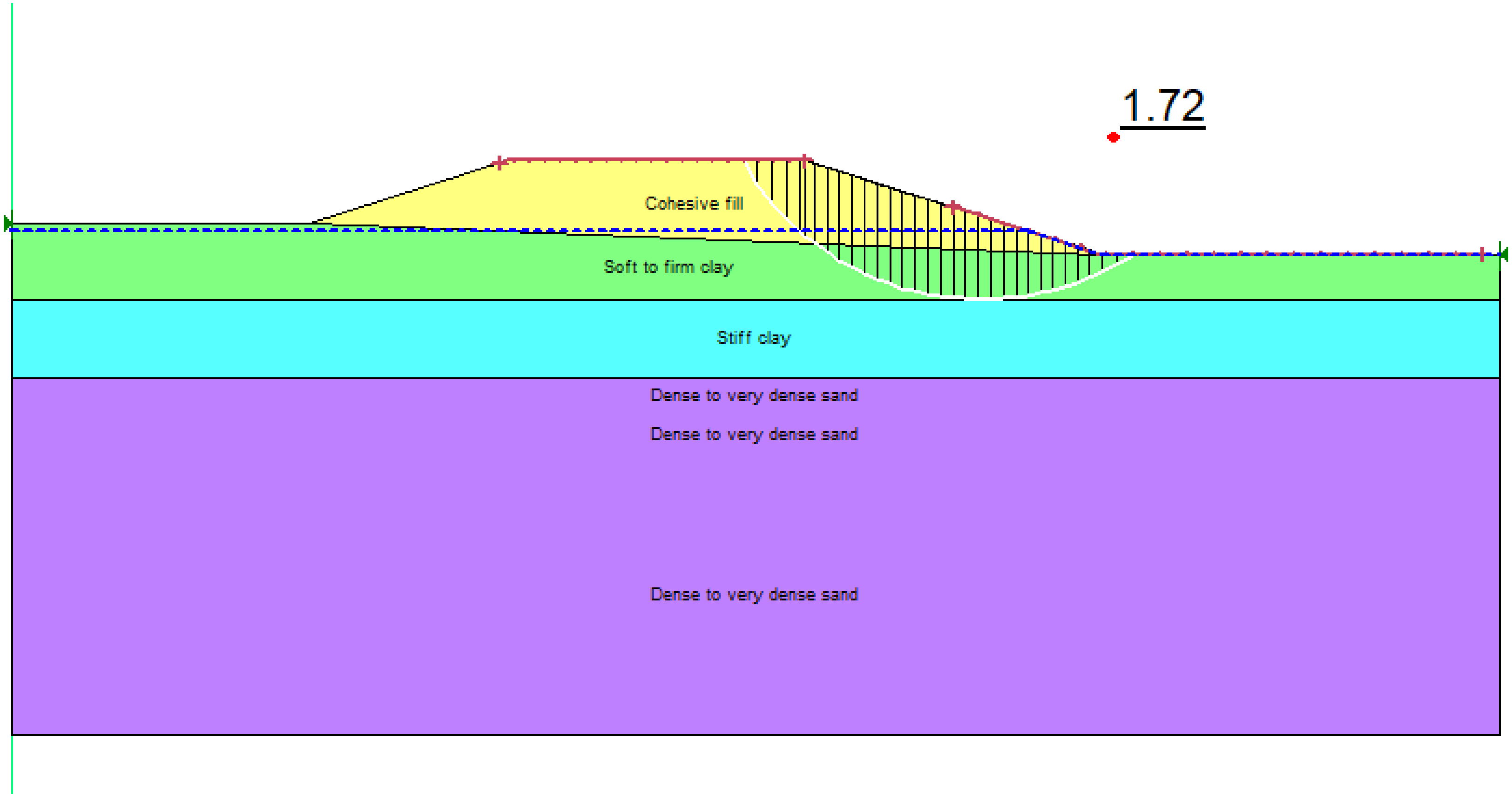
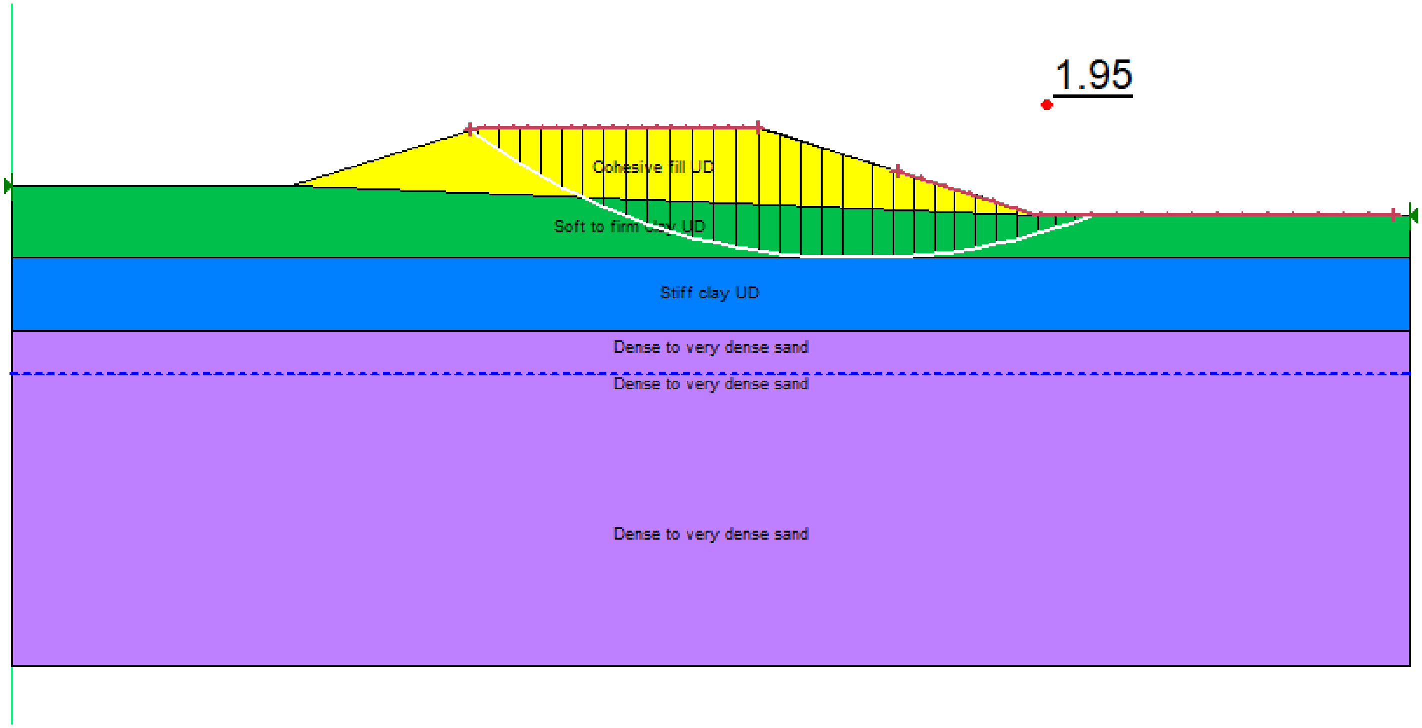
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Rapid Drawdown Condition, El 215 to Existing Grade
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.477$)
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Dead Timber Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

APPENDIX H

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer's installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX I

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, “The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project.” This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX J

WEAP ANALYSES - STEEL SHELL PILES

Project: 101124 - Hwy 135

Poinsett County, Arkansas

GHBW Project No: 23-031

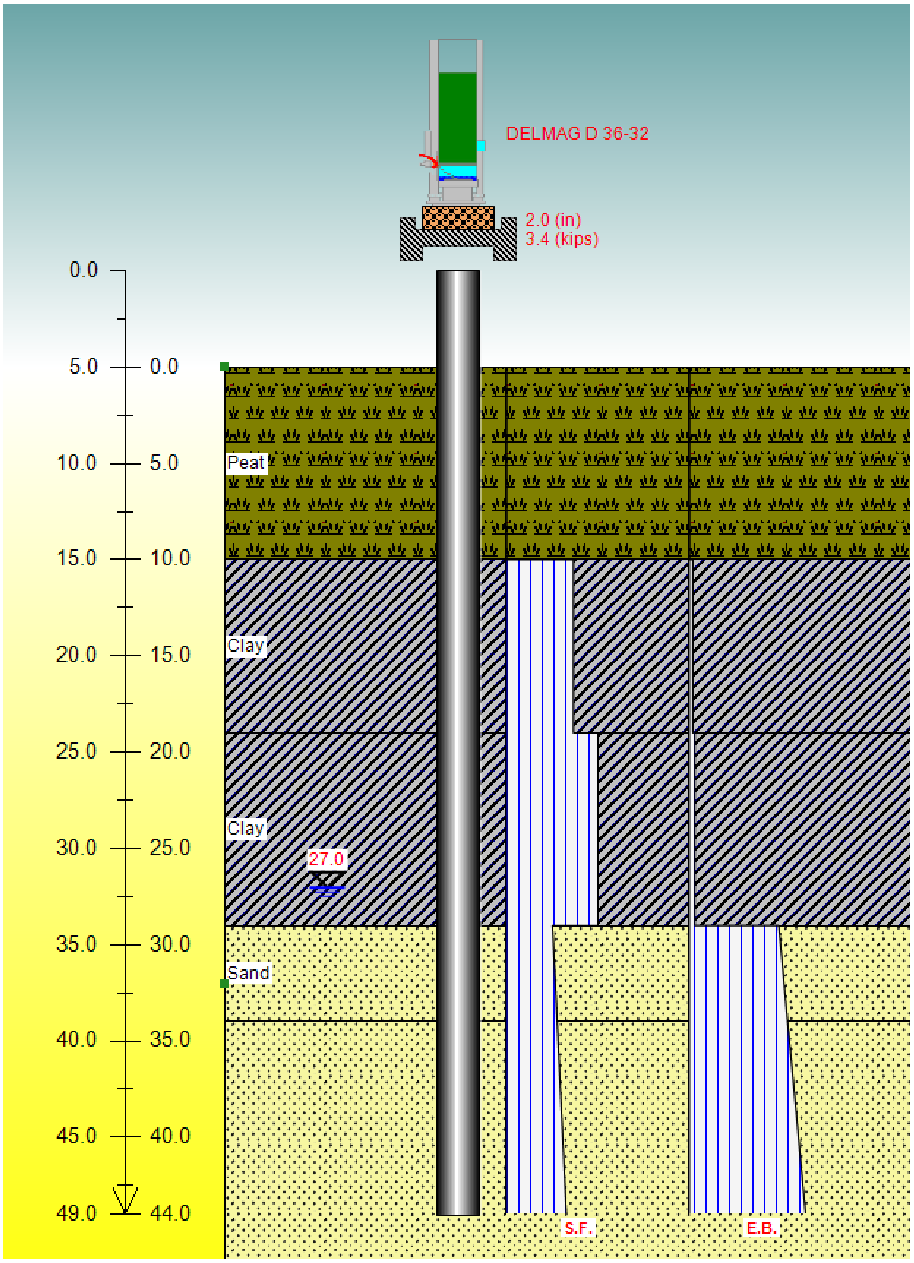
Bridge	Bent	Pile Diameter (in)	Wall Thickness (in)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El.	Min Tip El.	Pile Length (ft)	Min Hammer Energy (ft-kip)	Max Comp Stress, ksi
1 - Dead Timber Lake	1	16	0.50	270	215	171	44	91	39.6
	2	24	0.50	198	213	172	41	91	32.8
	3	24	0.50	289	214	156	58	125	37.5
	4	16	0.50	280	215	152	63	91	35.8

ArDOT 101124 Hwy 135 over Dead Timber Lake

Bent 1

16-in-diameter Steel Shell Pile

Delmag D36-32



Grubbs, Hoskyn, Barton & Wyatt, LLC
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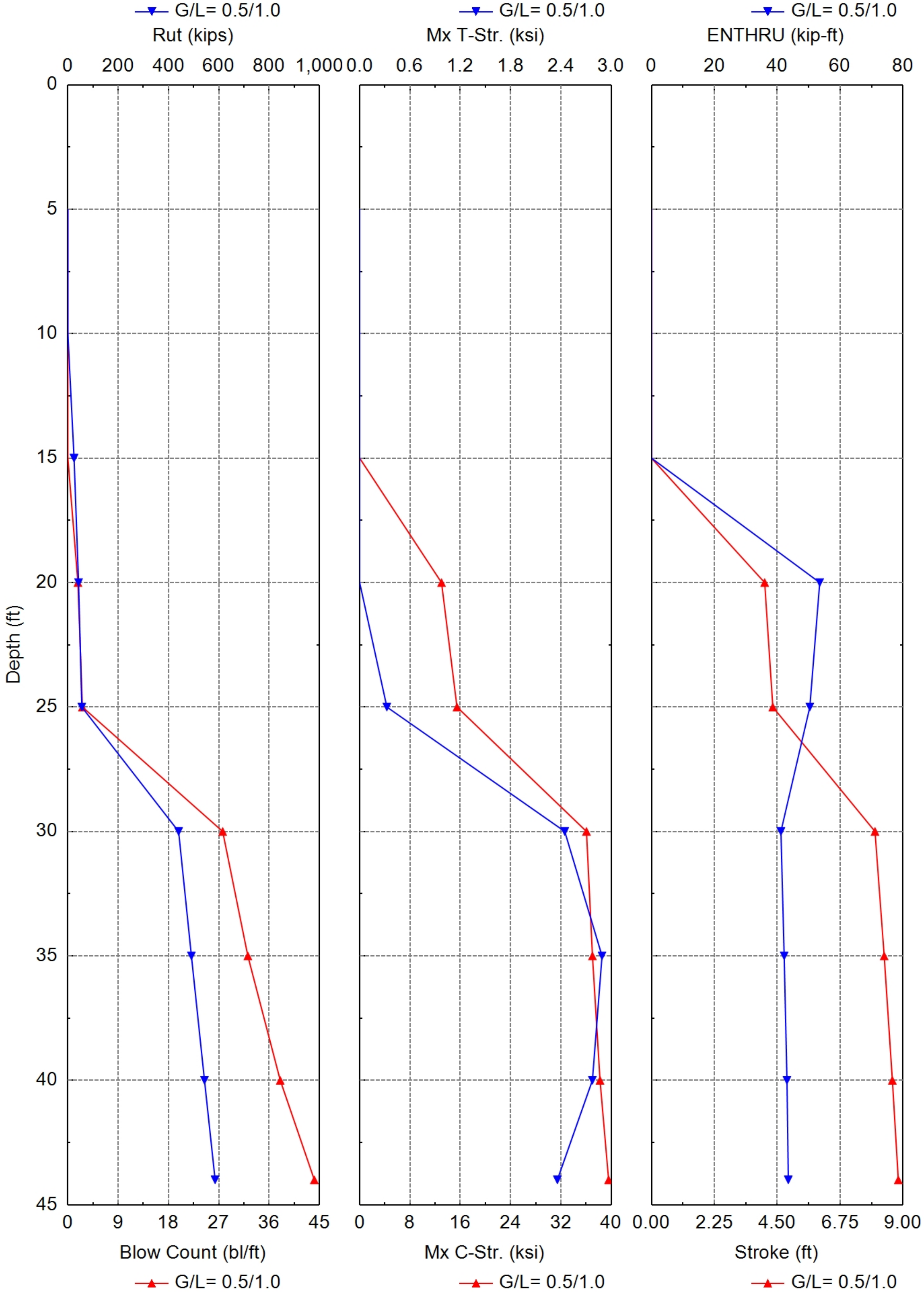
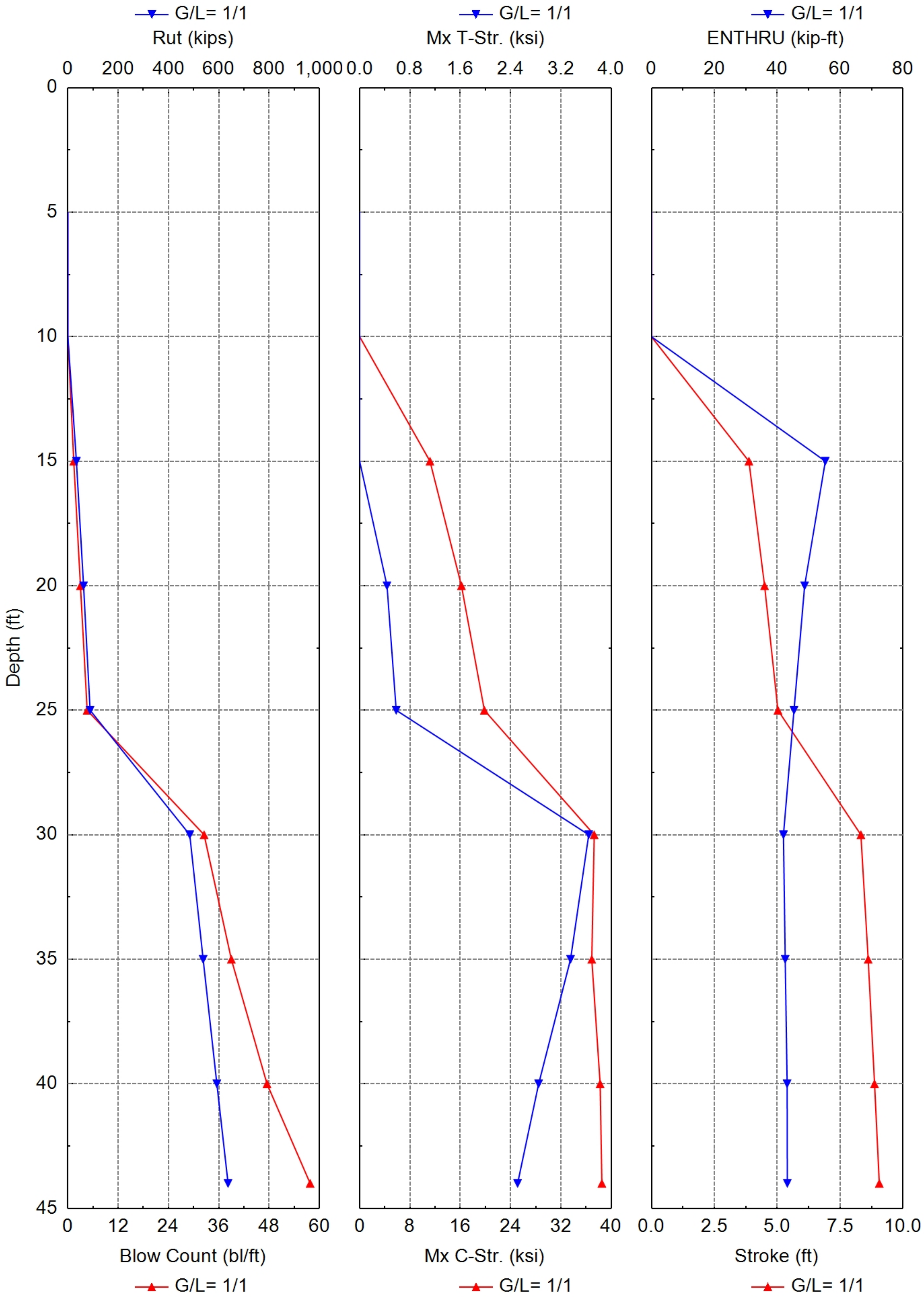

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
9
18
27
36
45
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
15.0	24.6	9.5	15.1	0.0	0.000	0.000	0.00	0.0	D 36-32
20.0	42.5	19.9	22.6	1.8	13.017	0.000	4.05	53.5	D 36-32
25.0	55.5	32.9	22.6	2.6	15.477	0.324	4.34	50.4	D 36-32
30.0	440.3	46.0	394.3	27.7	36.071	2.446	8.00	41.2	D 36-32
35.0	491.2	57.9	433.3	32.2	37.020	2.891	8.33	42.2	D 36-32
40.0	543.1	70.8	472.4	38.0	38.223	2.776	8.62	43.1	D 36-32
44.0	585.5	81.9	503.6	44.1	39.593	2.358	8.83	43.5	D 36-32

Total driving time: 14 minutes; Total Number of Blows: 583 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
200
400
600
800
1,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 1/1
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
12
24
36
48
60
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
15.0	34.0	18.9	15.1	1.4	11.181	0.000	3.87	55.3	D 36-32
20.0	62.4	39.8	22.6	3.0	16.186	0.435	4.50	48.7	D 36-32
25.0	88.4	65.8	22.6	4.6	19.833	0.581	5.03	45.3	D 36-32
30.0	484.6	90.3	394.3	32.5	37.298	3.643	8.33	42.0	D 36-32
35.0	537.8	104.5	433.3	39.0	36.913	3.353	8.62	42.5	D 36-32
40.0	592.3	120.0	472.4	47.5	38.243	2.847	8.87	43.2	D 36-32
44.0	636.9	133.4	503.6	57.8	38.520	2.512	9.06	43.2	D 36-32

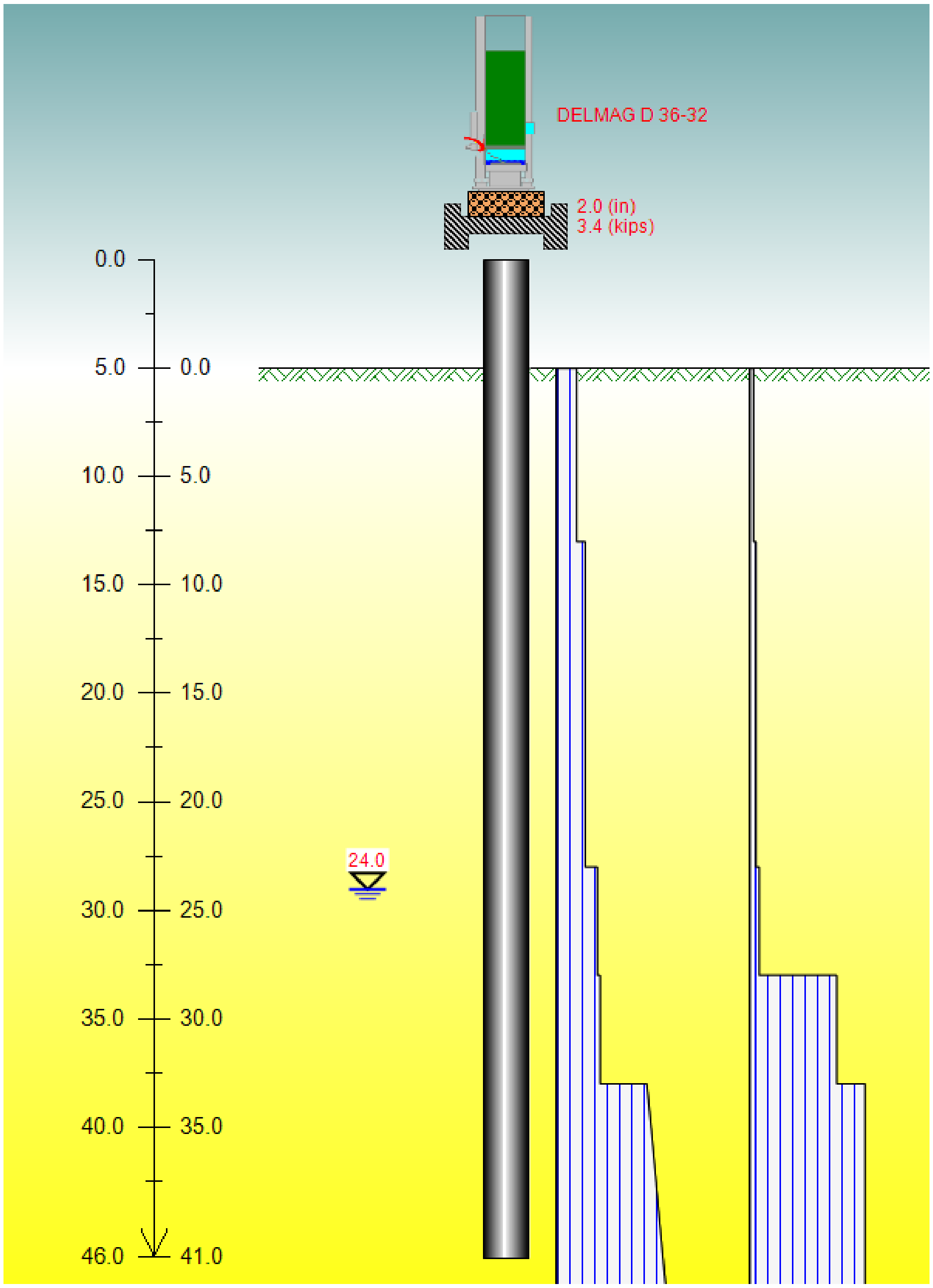
Total driving time: 18 minutes; Total Number of Blows: 734 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Dead Timber Lake

Bent 2

24-in-diameter Steel Shell Pile

Delmag D36-32



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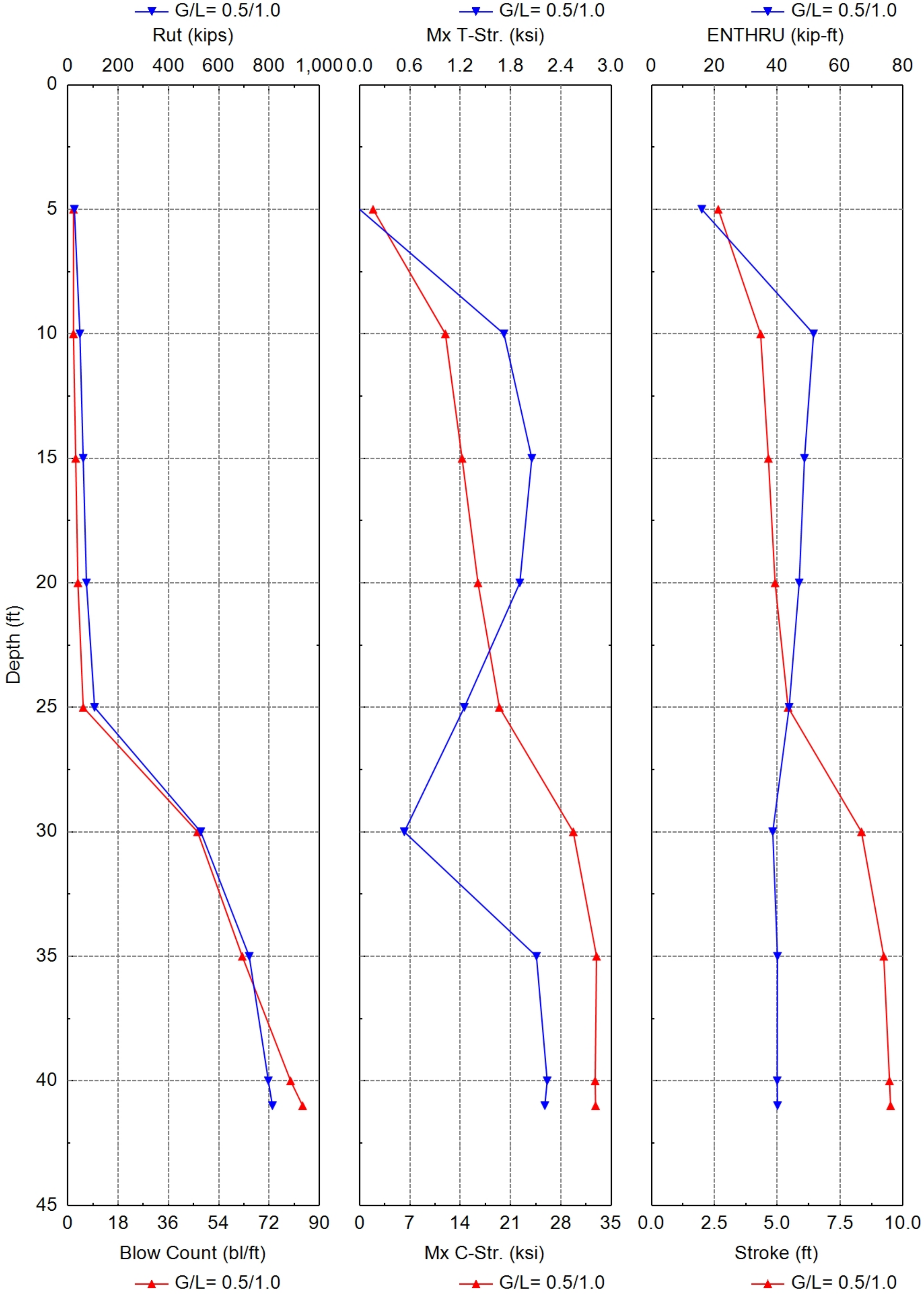
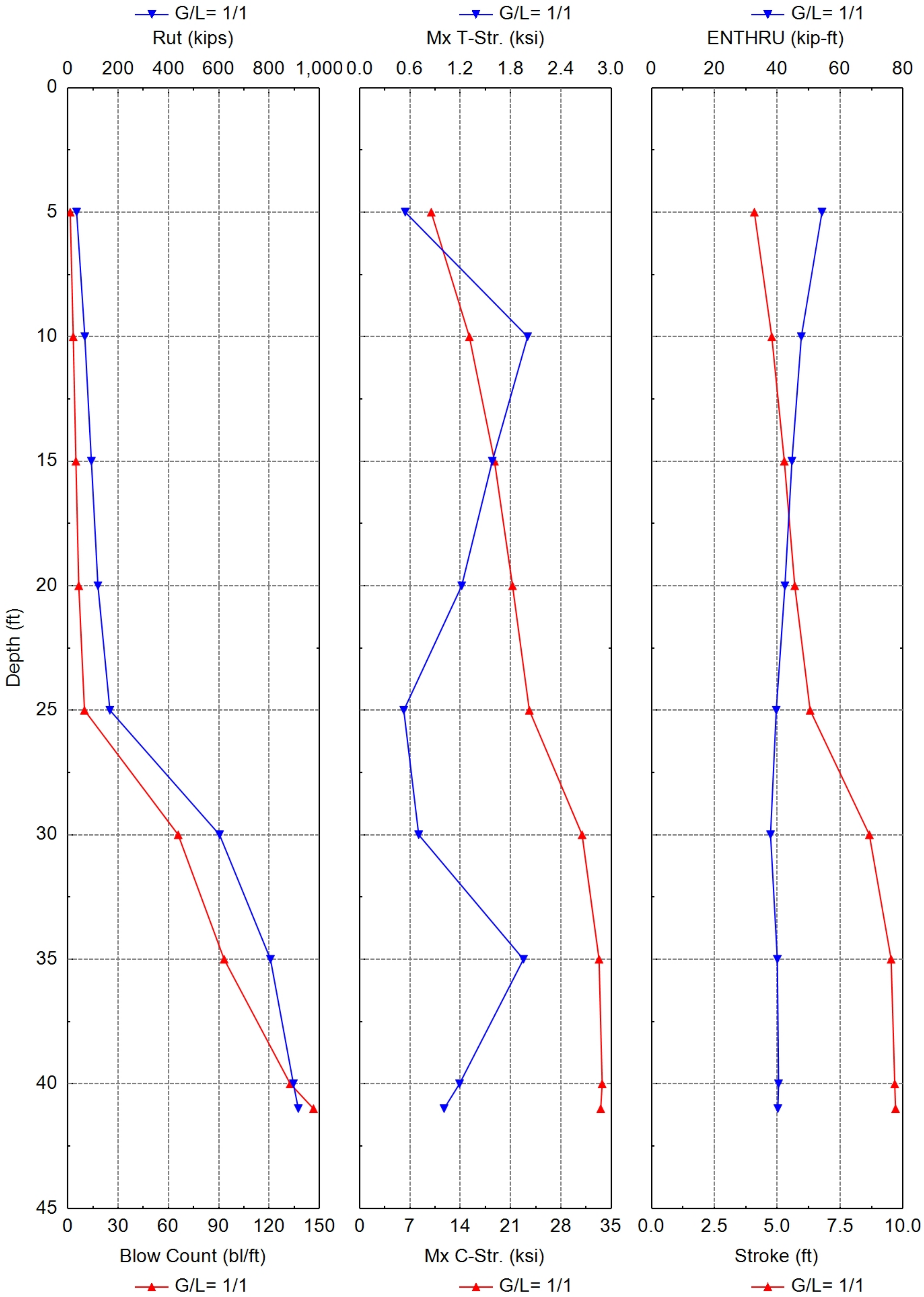

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
18
36
54
72
90
Blow Count (bl/ft)
G/L= 0.5/1.0
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	25.9	9.0	17.0	2.0	1.866	0.000	2.66	16.0	D 36-32
10.0	47.8	19.5	28.3	2.0	11.926	1.721	4.34	51.6	D 36-32
15.0	60.9	32.6	28.3	2.8	14.254	2.053	4.65	48.7	D 36-32
20.0	73.9	45.7	28.3	3.6	16.463	1.910	4.91	47.0	D 36-32
25.0	106.2	60.9	45.2	5.5	19.441	1.248	5.43	43.8	D 36-32
30.0	528.4	85.4	443.0	46.4	29.724	0.532	8.35	38.6	D 36-32
35.0	721.1	133.6	587.5	62.4	32.963	2.109	9.24	40.1	D 36-32
40.0	797.2	209.7	587.5	79.7	32.775	2.237	9.46	40.0	D 36-32
41.0	813.3	225.8	587.5	84.0	32.827	2.208	9.51	40.1	D 36-32

Total driving time: 22 minutes; Total Number of Blows: 899 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
200
400
600
800
1,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 1/1
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
30
60
90
120
150
Blow Count (bl/ft)
G/L= 1/1
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 1/1
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	34.9	17.9	17.0	1.4	9.958	0.544	4.09	54.2	D 36-32
10.0	67.4	39.1	28.3	3.2	15.263	2.003	4.78	47.7	D 36-32
15.0	93.5	65.2	28.3	4.8	18.762	1.582	5.28	44.7	D 36-32
20.0	119.6	91.3	28.3	6.5	21.273	1.217	5.70	42.4	D 36-32
25.0	167.1	121.9	45.2	9.9	23.610	0.526	6.31	39.7	D 36-32
30.0	603.2	160.2	443.0	65.8	30.954	0.704	8.66	37.9	D 36-32
35.0	805.5	218.0	587.5	93.2	33.309	1.954	9.53	40.0	D 36-32
40.0	896.8	309.3	587.5	132.6	33.745	1.191	9.67	40.4	D 36-32
41.0	916.1	328.7	587.5	146.5	33.549	1.006	9.70	40.2	D 36-32

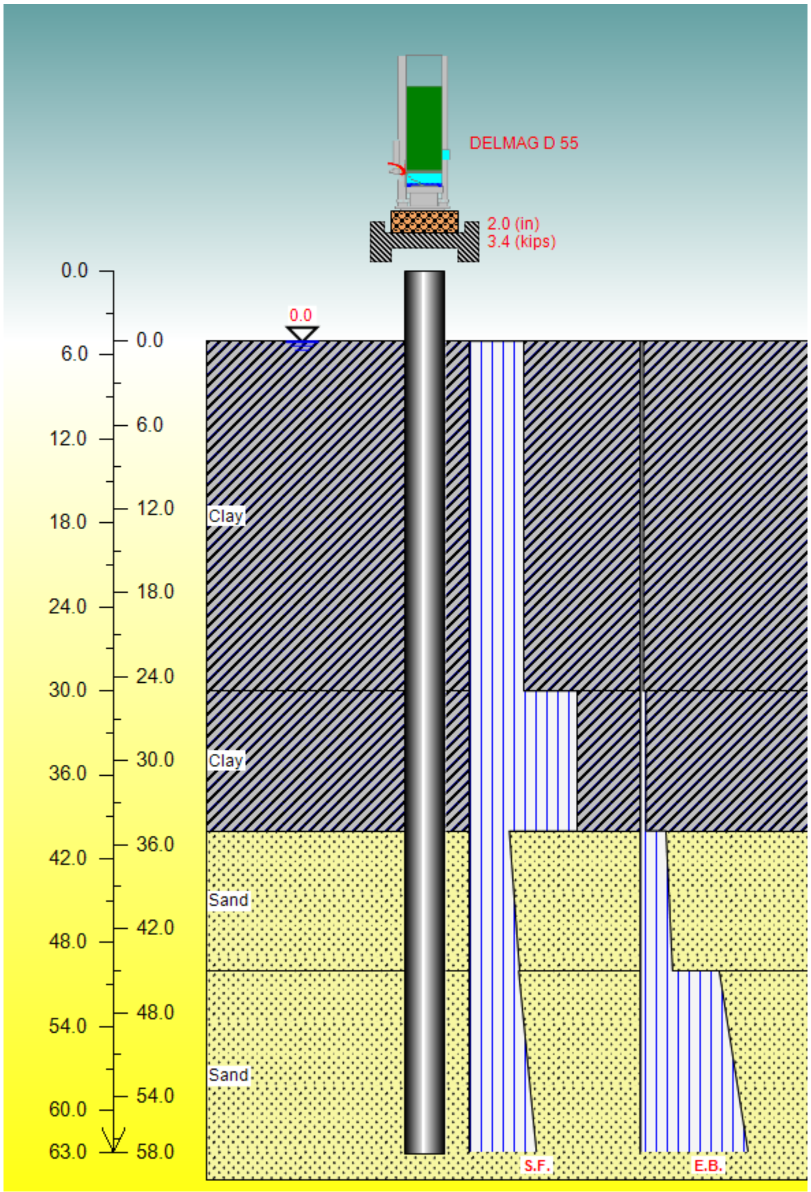
Total driving time: 35 minutes; Total Number of Blows: 1391 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Dead Timber Lake

Bent 3

24-in-diameter Steel Shell Pile

Delmag D55



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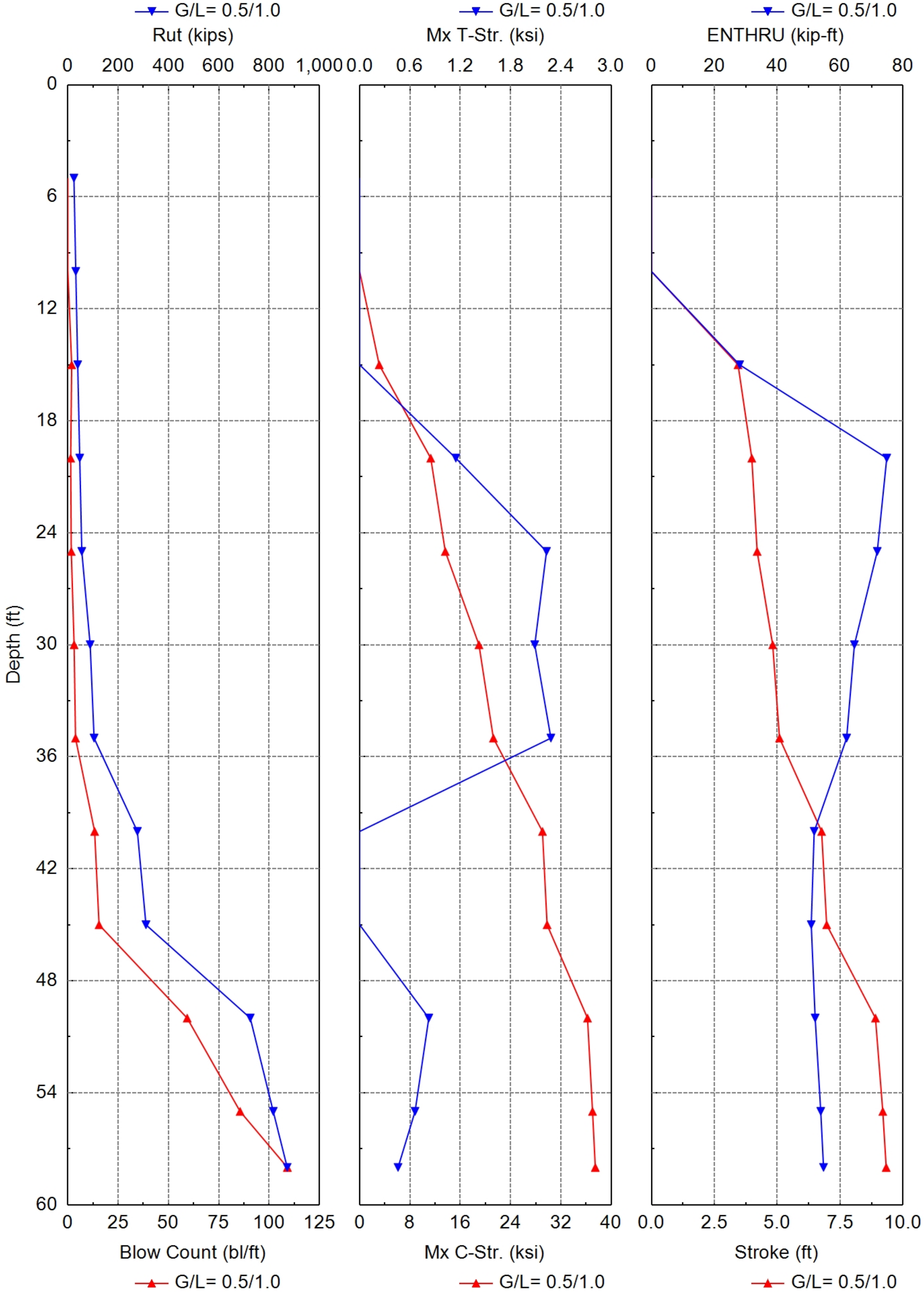
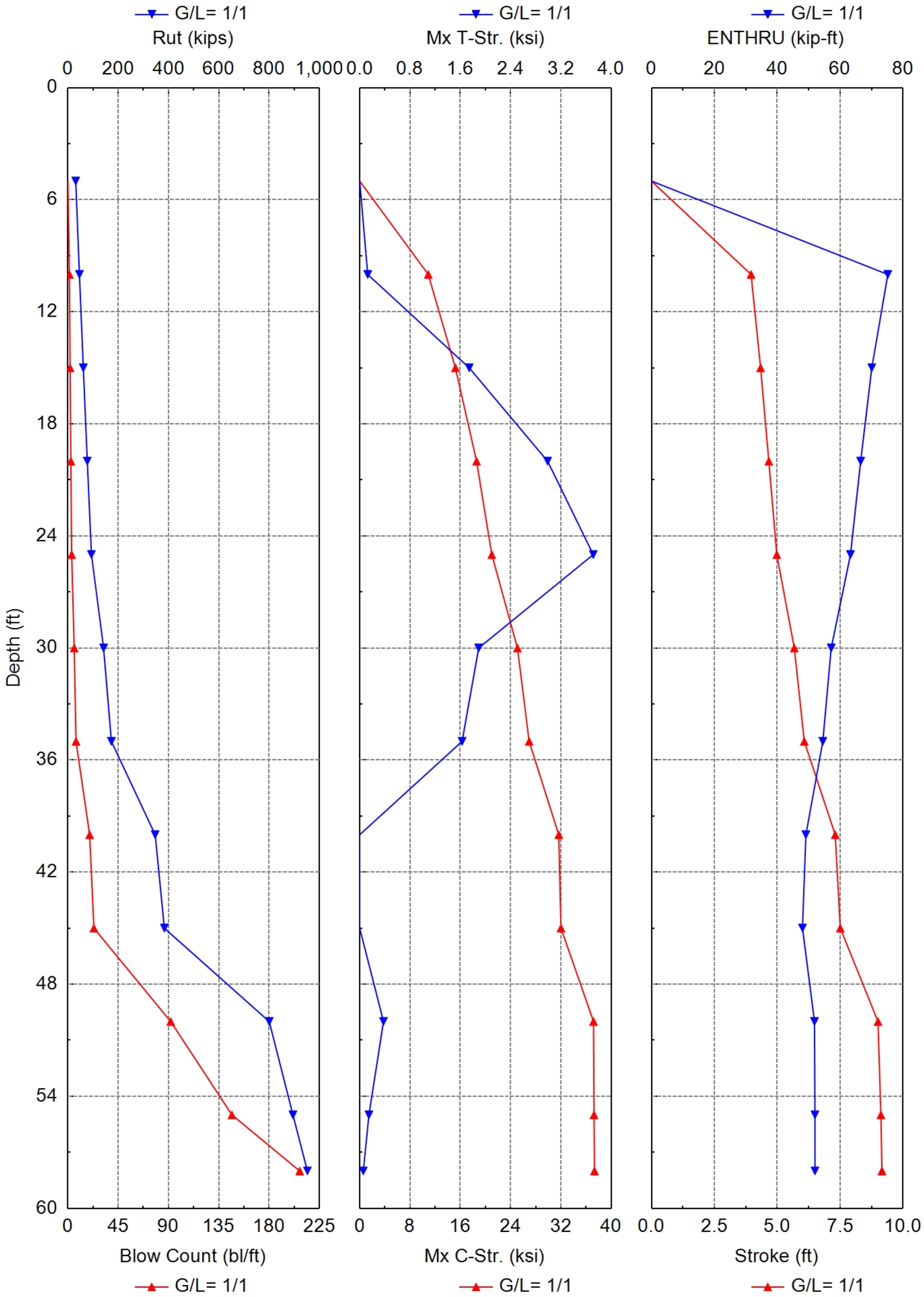

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	24.1	7.1	17.0	0.0	0.000	0.000	0.00	0.0	D 55
10.0	31.5	14.5	17.0	0.0	0.000	0.000	0.00	0.0	D 55
15.0	39.2	22.2	17.0	1.9	3.106	0.000	3.44	28.1	D 55
20.0	47.1	30.2	17.0	1.4	11.302	1.147	3.99	74.8	D 55
25.0	55.4	38.4	17.0	1.7	13.617	2.229	4.20	71.8	D 55
30.0	88.7	53.4	35.3	3.1	18.972	2.089	4.82	64.5	D 55
35.0	104.2	68.9	35.3	3.8	21.264	2.281	5.09	62.1	D 55
40.0	276.4	78.8	197.6	13.3	29.072	0.000	6.77	51.8	D 55
45.0	310.5	89.9	220.6	15.5	29.808	0.000	6.97	50.8	D 55
50.0	725.2	102.3	622.9	59.3	36.235	0.824	8.91	52.1	D 55
55.0	816.6	116.3	700.3	85.6	37.034	0.661	9.20	53.8	D 55
58.0	872.2	125.4	746.8	109.1	37.474	0.458	9.33	54.7	D 55

Total driving time: 25 minutes; Total Number of Blows: 1006 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
200
400
600
800
1,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 1/1
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
45
90
135
180
225
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	31.2	14.3	17.0	0.0	0.000	0.000	0.00	0.0	D 55
10.0	46.0	29.1	17.0	1.4	10.909	0.128	3.96	75.2	D 55
15.0	61.4	44.4	17.0	2.0	15.232	1.742	4.34	70.1	D 55
20.0	77.3	60.3	17.0	2.7	18.593	2.989	4.67	66.5	D 55
25.0	93.7	76.8	17.0	3.4	21.017	3.715	4.98	63.3	D 55
30.0	142.0	106.7	35.3	5.7	25.097	1.893	5.68	57.2	D 55
35.0	173.1	137.7	35.3	7.3	26.951	1.629	6.08	54.5	D 55
40.0	347.3	149.6	197.6	19.5	31.663	0.000	7.31	49.1	D 55
45.0	383.6	163.0	220.6	23.3	32.016	0.000	7.51	48.0	D 55
50.0	800.7	177.8	622.9	92.0	37.192	0.379	9.01	51.8	D 55
55.0	894.9	194.6	700.3	146.8	37.277	0.147	9.12	52.0	D 55
58.0	952.4	205.6	746.8	207.4	37.342	0.058	9.16	52.0	D 55

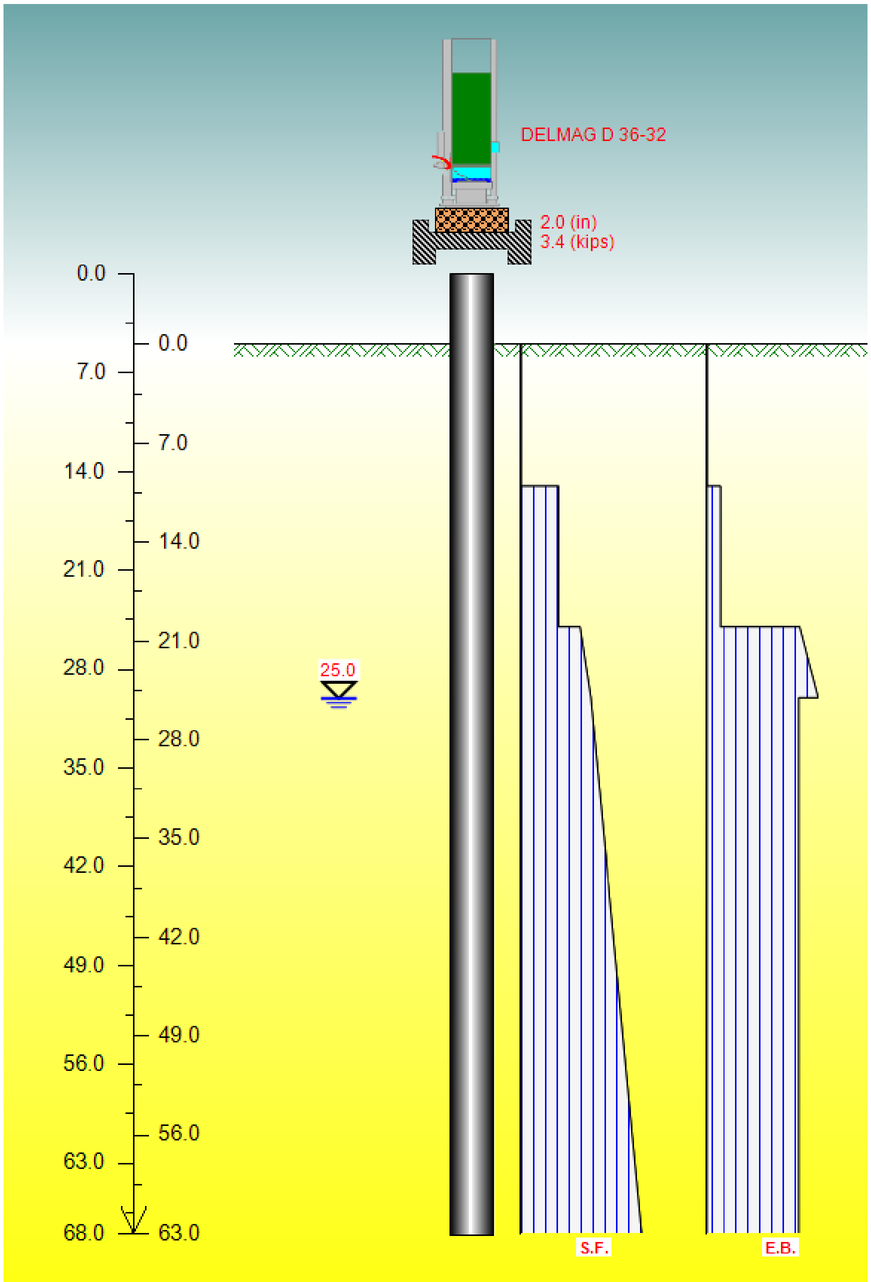
Total driving time: 42 minutes; Total Number of Blows: 1684 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Dead Timber Lake

Bent 4

16-in-diameter Steel Shell Pile

Delmag D36-32



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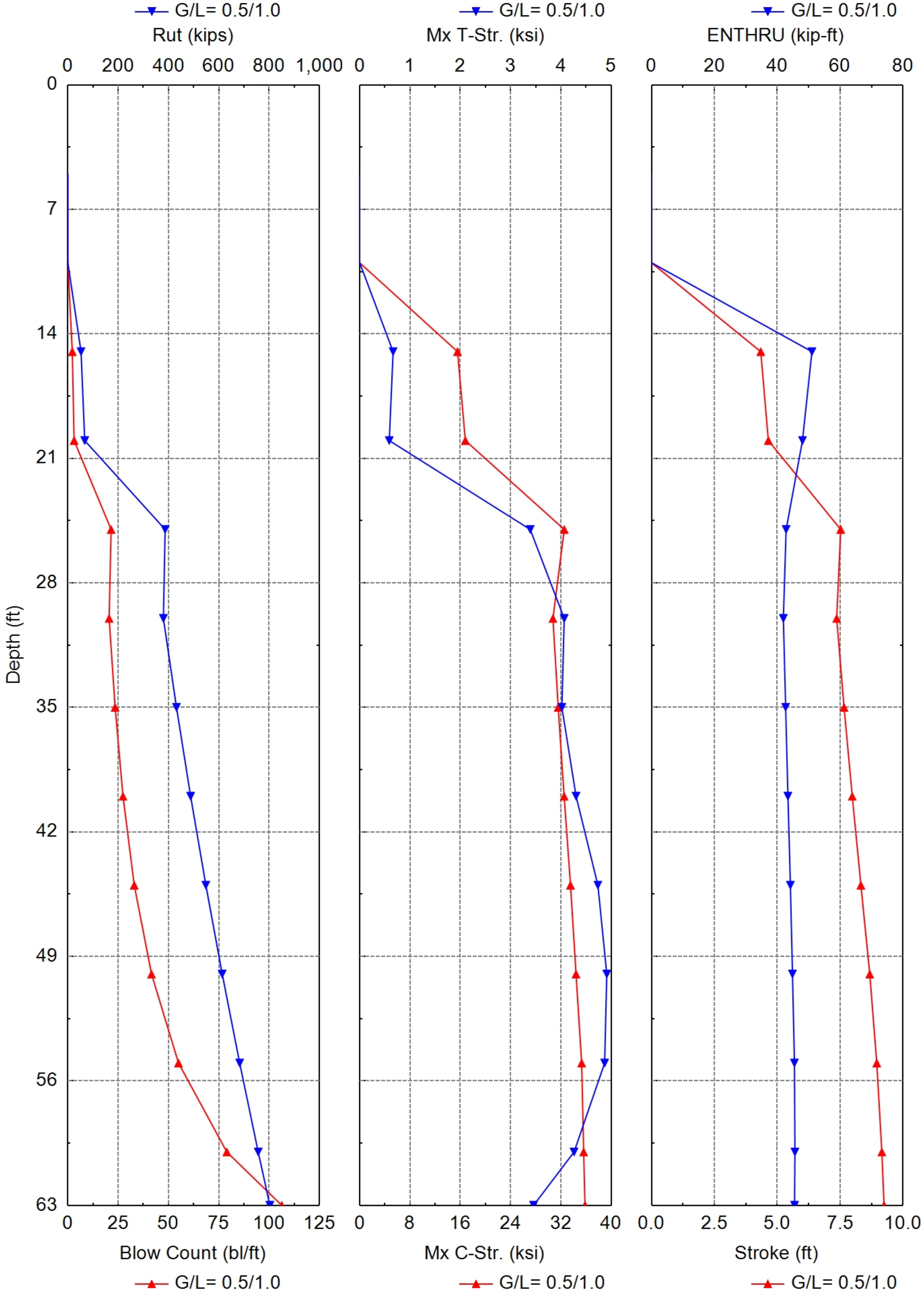
G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0
1
2
3
4
5
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
7
14
21
28
35
42
49
56
63
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.42	0.0	D 36-32
15.0	52.4	14.7	37.7	2.2	15.565	0.665	4.35	51.0	D 36-32
20.0	67.0	29.3	37.7	3.1	16.805	0.588	4.65	48.0	D 36-32
25.0	386.8	71.2	315.6	21.5	32.540	3.391	7.52	42.8	D 36-32
30.0	379.9	118.8	261.1	20.5	30.740	4.066	7.36	42.0	D 36-32
35.0	431.8	170.7	261.1	23.5	31.578	4.020	7.66	42.7	D 36-32
40.0	488.1	227.0	261.1	27.4	32.517	4.302	7.98	43.4	D 36-32
45.0	548.7	287.6	261.1	33.0	33.534	4.736	8.33	44.2	D 36-32
50.0	613.7	352.6	261.1	41.6	34.412	4.913	8.68	44.8	D 36-32
55.0	683.1	422.0	261.1	55.0	35.306	4.870	8.96	45.5	D 36-32
60.0	756.9	495.8	261.1	79.0	35.621	4.262	9.16	45.6	D 36-32
63.0	803.2	542.1	261.1	106.3	35.834	3.457	9.25	45.5	D 36-32

Total driving time: 40 minutes; Total Number of Blows: 1616 (starting at penetration 5.0 ft)

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 15, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER TYRONZA RIVER (SITE 2)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
POINSETT COUNTY, ARKANSAS

INTRODUCTION

Submitted herewith are the final results of the geotechnical investigation performed for the Hwy. 135 over Tyronza River replacement bridge in Poinsett County, Arkansas. This bridge is Site 2 of the ARDOT 110124 Hwy. 135 Str. & Apprs. (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by the Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on July 2 and August 19, 2023.

We understand the replacement bridge will be a prestressed concrete girder unit with five (5) bents, four (4) spans, and a total length of approximately 282.5 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed east of the existing bridge. Site grading will include about 20 ft of fill. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, 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 in the Tyronza River (Site 2) replacement bridge alignment were explored by drilling five (5) sample borings to 80- to 120-feet. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2. The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
B1	614+75	35 ft Lt	35.50502	-90.32281	221.8	110
B2	615+40	30 ft Lt	35.50514	-90.32299	216.6	120
B3	616+70	10 ft Rt	35.50547	-90.32288	203.3	80
B4	617+15	20 ft Lt	35.50565	-90.32293	201.3	100
B5	617+90	10 ft Rt	35.50585	-90.32286	214.9	110

The boring logs, presenting descriptions of the soil strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 16. The centerline station and offset of the boring locations and approximate ground surface elevation, as surveyed, are also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 17.

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.

The borings were drilled with a truck-mounted CME-55 HTX rotary-drilling rig and a track-mounted Diedrich D-50 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the appropriate energy conversion factor is shown on each boring log.

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

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

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 55 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 11 liquid and plastic (Atterberg) limit determinations and 30 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The location of 101124 Site 2 is on Hwy. 135 where the Tyronza River crosses the highway alignment, approximately 2430 ft south of Hwy. 118 in Poinsett County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. The Tyronza River channel at the bridge location is broad with well-defined banks. The banks are steep with tall grass and variable sparse to thick underbrush. The area around the bridge is low-lying and swampy, with standing water, thick underbrush, and numerous trees. The project locale is primarily agricultural land consisting of open and flat fields. The existing roadway is on embankment, and the existing pavements are in poor condition. Surface drainage along the roadway is poor and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent Alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends over exposures of Quaternary-aged Alluvium. The Alluvium is comprised of recent stream-deposited alluvial sediments which include gravel, sand, silt, clay and mixtures of all components. The thickness of the Alluvial deposits is variable. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

Seismic Conditions

A Site-Specific Ground Motion Response Analysis was performed for the 110124 project. The site-specific ground motion response analyses were performed by Geotechnology in accordance with Section 3.4.3.2 of the 2022 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition. Three (3) sites were analyzed for shear wave velocities: Sites 2, 5, and 7. The site-specific results from Site 2 were utilized in the analyses performed for this study.

Shear wave velocity profiles were developed for the Site-Specific Ground Motion Response Analysis. Summary results from the analysis are provided in Appendix D. An average shear wave velocity in the top 100 ft of subsurface soil was calculated to be 701 ft per second for Site 2. In light of the shear wave velocity profile and the results of the borings, a Seismic Site Class D (stiff soil profile) is considered fitting for the Site 2 bridge location.

Based on the results of the site-specific seismic hazard analysis, design earthquake spectral response acceleration of 0.769g for PGA, 1.565g for S_{DS}, 1.197g for S_{D1} and 7.7 for Design Earthquake Moment Magnitude (M_w) were determined. These calculated design seismic accelerations utilizing the site-specific procedure are 67 percent or greater of the corresponding counterparts as determined using the code-based procedure. A plot of design response spectra, showing the design earthquake spectral response accelerations versus period for both code-based and site-specific values, is also included in Appendix D. The design response spectra developed based on the results of the site-specific procedure are considered suitable for use in structural design.

Utilizing these parameters, Table 3.10.6-1[2] indicates that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Hwy. 135 bridge over Tyronza River site.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 0.769, as per the site-specific seismic analyses, and an earthquake Moment Magnitude (M_W) of 7.7 were utilized in the liquefaction analyses.

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[2] AASHTO LRFD Bridge Design Specification, AASHTO; 2012

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

The results of the liquefaction analyses are provided in Appendix E as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the analyses results are shown on the generalized subsurface profile also provided in Appendix E. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix E.

Subsurface Conditions

Based on the results of the borings, the surface and near-surface soils to 6- to 38-ft depth are comprised of very soft to stiff brown, dark brown, reddish tan, and gray clay (CH), silty clay (CL), and fine sandy clay (CH). This stratum contains occasional organic inclusions, ferrous stains, and clayey silt and silty fine sand seams. The clayey soils exhibit very low to low strength, moderate to high plasticity, and moderate to high compressibility. These soils typically classify as A-6, A-7-5, and A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor subgrade support for pavement structures.

The clayey soil units are underlain below 6- to 38-ft depth by loose to dense gray, brown, and brownish gray fine to medium sand (SP). This stratum contains clay seams and pockets as well as coarse sand and fine gravel at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was encountered in the borings at 13- to 31-ft depth in May through July 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in Tyronza River and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 2 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 18-in.-diameter steel shell piles are planned for bridge ends and 28-in.-diameter steel shell piles are planned for the interior bents. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix F. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for "static" conditions prior to a seismic event, with no liquefaction, and full soil shear strength is mobilized for the foundation soils. For the case where liquefaction occurs, the "end of earthquake" condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects. The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix G.

Bridge End Embankment Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 5) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 25 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a 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 0.3845. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value from the site-specific seismic hazard analysis. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 209 to El 200 was assumed.

Stability analyses results are summarized and presented graphically in Appendix H. These results indicate acceptable stability for all cases evaluated. A suitable factor of safety against lateral flow sliding was calculated for each bridge end embankments.

Subgrade Support

It is understood that “standard” pavement sections will be utilized by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-7-6. These classifications correlate with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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[4] Slope/W 2020; GEO-SLOPE International; 2020.

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

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, undercuts or improvement depths on the order of 3 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. in cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, undercutting is expected to be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 8 to 38 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix I. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

Maximum embankment heights on the order of 25 ft are anticipated. Given the predominance of cohesive soils in the embankment foundations, some consolidation settlement will occur. Based on the results of the borings and the anticipated maximum embankment height, total settlement of the natural foundation soils below the embankments is estimated to be on the order of 3 to 4 inches. Settlement of cohesive fill in the embankments is expected to be on the order of 2 to 3 in. with 40 to 60 percent of the settlement occurring during construction. We recommend that embankment fill be placed as early in the construction sequence as possible to limit post-construction settlement after foundation construction.

General 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow within about 100 ft of the bridge ends. An example special provision for cohesive embankment fill is provided in Appendix J.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until embankments and bridge work are completed.

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

Groundwater was encountered between 13- to 31-ft depth in May, June, and July 2023. Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered. Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[5]. In the driveability analyses, the steel shell piles were assumed to be driven from the plan cap bottom elevation or existing grade. The results of these analyses are provided in Appendix K.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 107 ft-kips per blow for driving the steel shell piles at the end bents. For the intermediate bents, we recommend a hammer system capable of delivering at least 186 ft-kips per blow for driving the steel shell piles. 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 installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. 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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment construction. Subsurface

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[5] GRLWEAP 2014; Pile Dynamics, Inc.

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 Map
Plate 2	Plan of Borings
Plates 3 through 16	Boring Logs
Plate 17	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Selected Results - Site-Specific Ground Motion Response Analysis
Appendix E	Liquefaction Analysis Results
Appendix F	Nominal Pile Capacity Curves
Appendix G	Lateral Load Parameters
Appendix H	Results of Stability Analyses
Appendix I	Example SP – Woven Geotextile
Appendix J	Example SP – Cohesive Embankment Fill Special Provision
Appendix K	Driveability Analysis Results

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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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

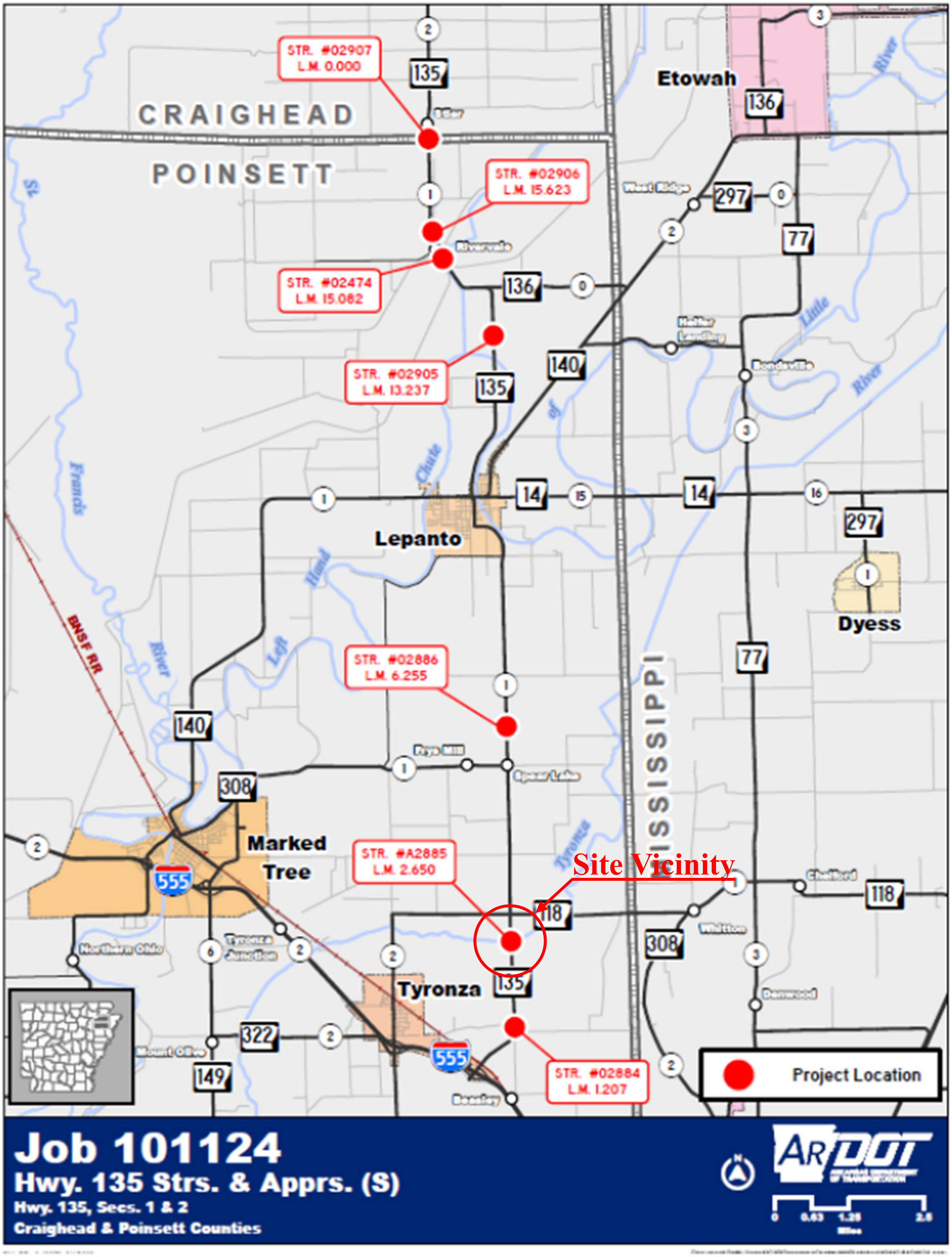
ME Wyatt

No. 7791
MARK E. WYATT

Mark E. Wyatt, P.E.
President

VMS/MEW:jw

Copies submitted:	Arkansas Department of Transportation		
	Attn:	Ms. Jessica Jackson, P.E.	(1-email)
	Attn:	Mr. Paul Tierney	(1-email)
	Attn:	Mr. Yongsheng Zhao, Ph.D., P.E.	(1-email)
	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

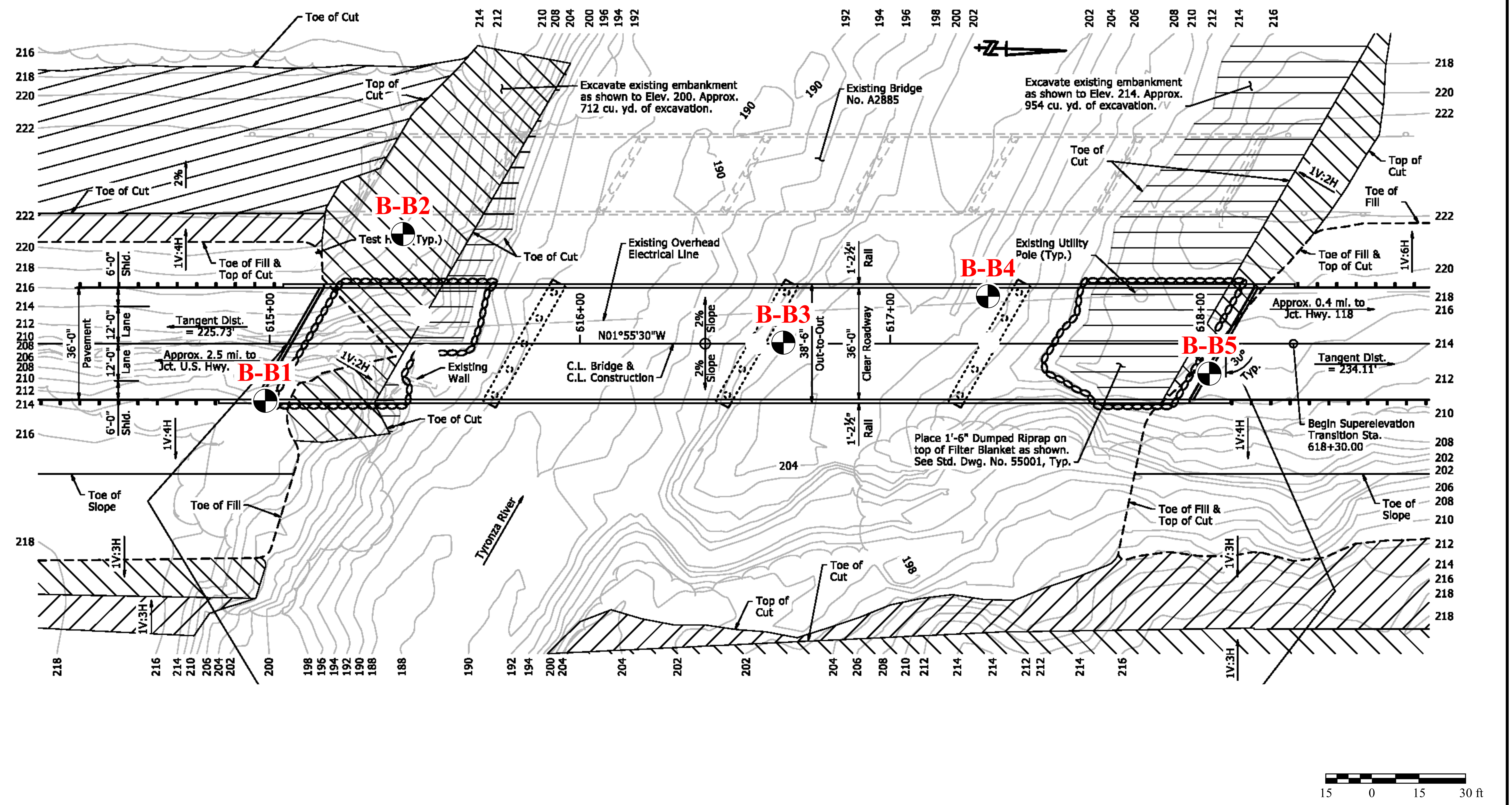


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Tyronza River
(Site 2/Bridge B)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORINGS
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Scale: As shown

Job No. 23-031

PLATE 2

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

LOG OF BORING NO. B1

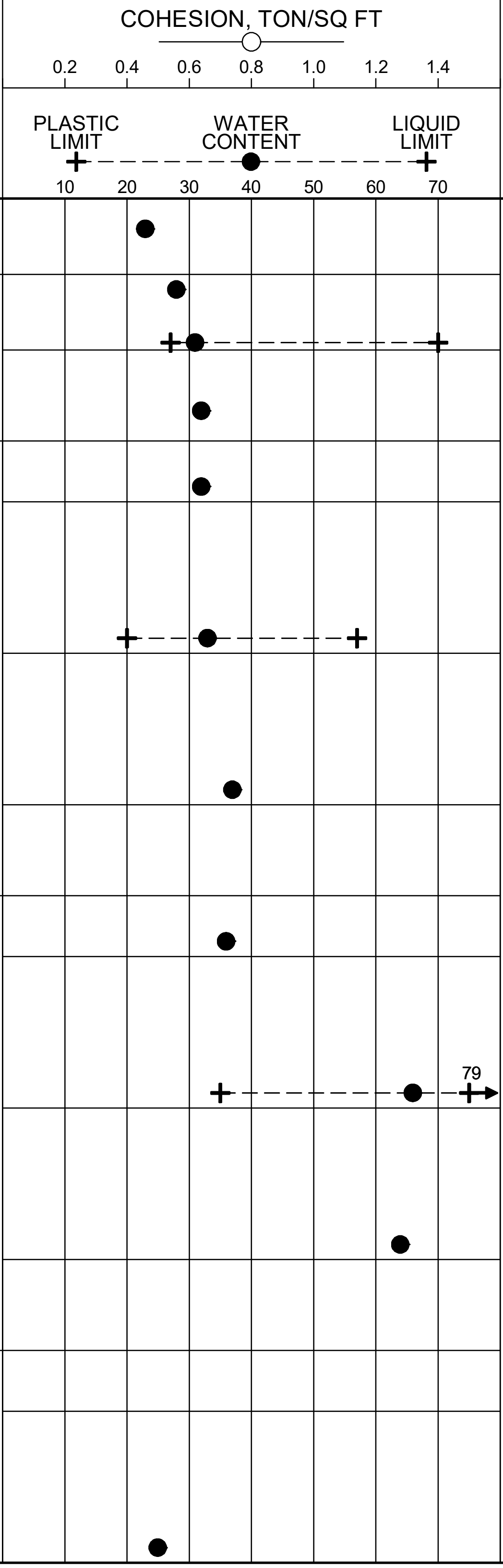
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 40 ft /Wash

LOCATION: Approx Sta 614+75, 35 ft Lt

SURF. EL: 221.8

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown silty clay, slightly sandy (CL)	7		
	Firm dark brown clay (CH)	11		
5		11		98
	- brown and gray below 6.5 ft	13		
10	Soft brown and gray clay (CH), sandy	7		
15		9		78
20	- very soft to soft below 18 ft	6		
25	Soft brown and gray fine sandy clay (CH)	9		
30	- with occasional organic inclusions below 28 ft	7		65
35	- firm with silty clay seams and layers below 33 ft	10		
40	Medium dense brownish gray fine to medium sand (SP)	21		
		29		3



COMPLETION DEPTH: 110.0 ft
DATE: 6-8-23

DEPTH TO WATER
IN BORING: 31.4 ft

DATE: 6/8/2023

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

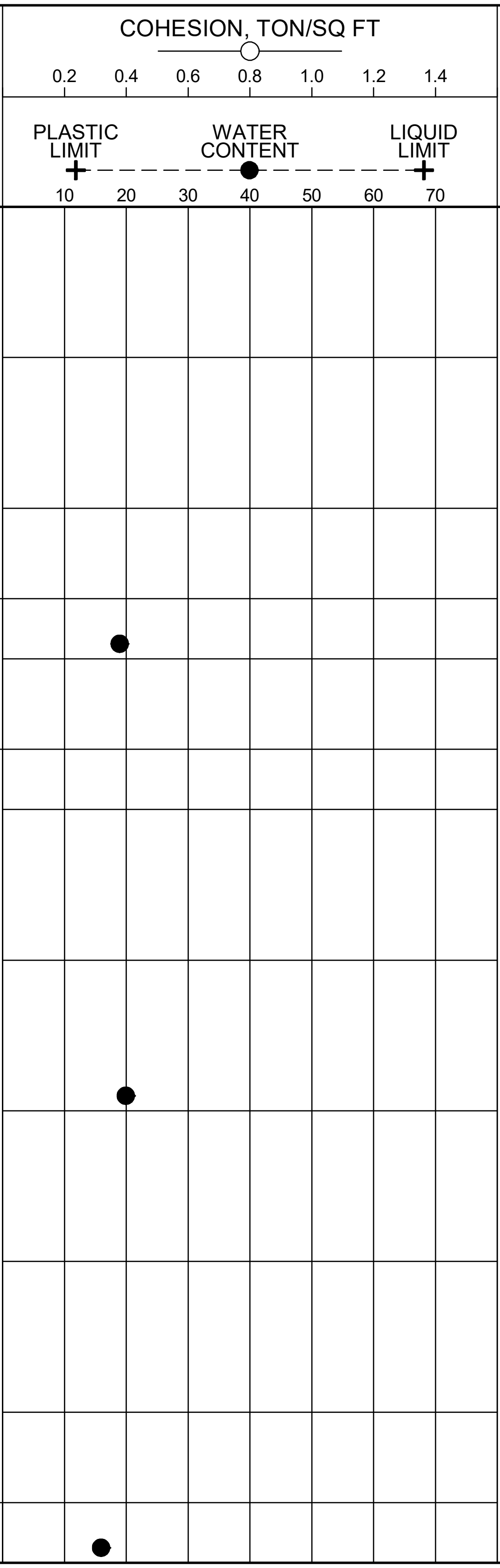
LOG OF BORING NO. B1

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 40 ft /Wash

LOCATION: Approx Sta 614+75, 35 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		34		
55	- with gray clay pockets below 54 ft	30		
60	Dense gray and brown fine sand, slightly silty (SP)	47		5
65	Medium dense brownish gray fine to medium sand, slightly silty (SM-SP) w/trace fine gavel	50		
70	- dense below 68 ft	44		
75		46		5
80		53		
85	- gray and brown below 84 ft	60		
90	Dense brown and dark gray fine to coarse sand, slightly silty (SM-SW)	49		5



COMPLETION DEPTH: 110.0 ft
DATE: 6-8-23

DEPTH TO WATER
IN BORING: 31.4 ft

DATE: 6/8/2023

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

LOG OF BORING NO. B1

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 40 ft /Wash

LOCATION: Approx Sta 614+75, 35 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
			w/trace fine gravel				
95		☒	- brown and gray below 94 ft	54			
100		☒		57			
105							
110		☒		60			
			NOTE: Drilled with Diedrich D-50 ECF=1.43				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-8-23

DEPTH TO WATER
IN BORING: 31.4 ft

DATE: 6/8/2023

23-031

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

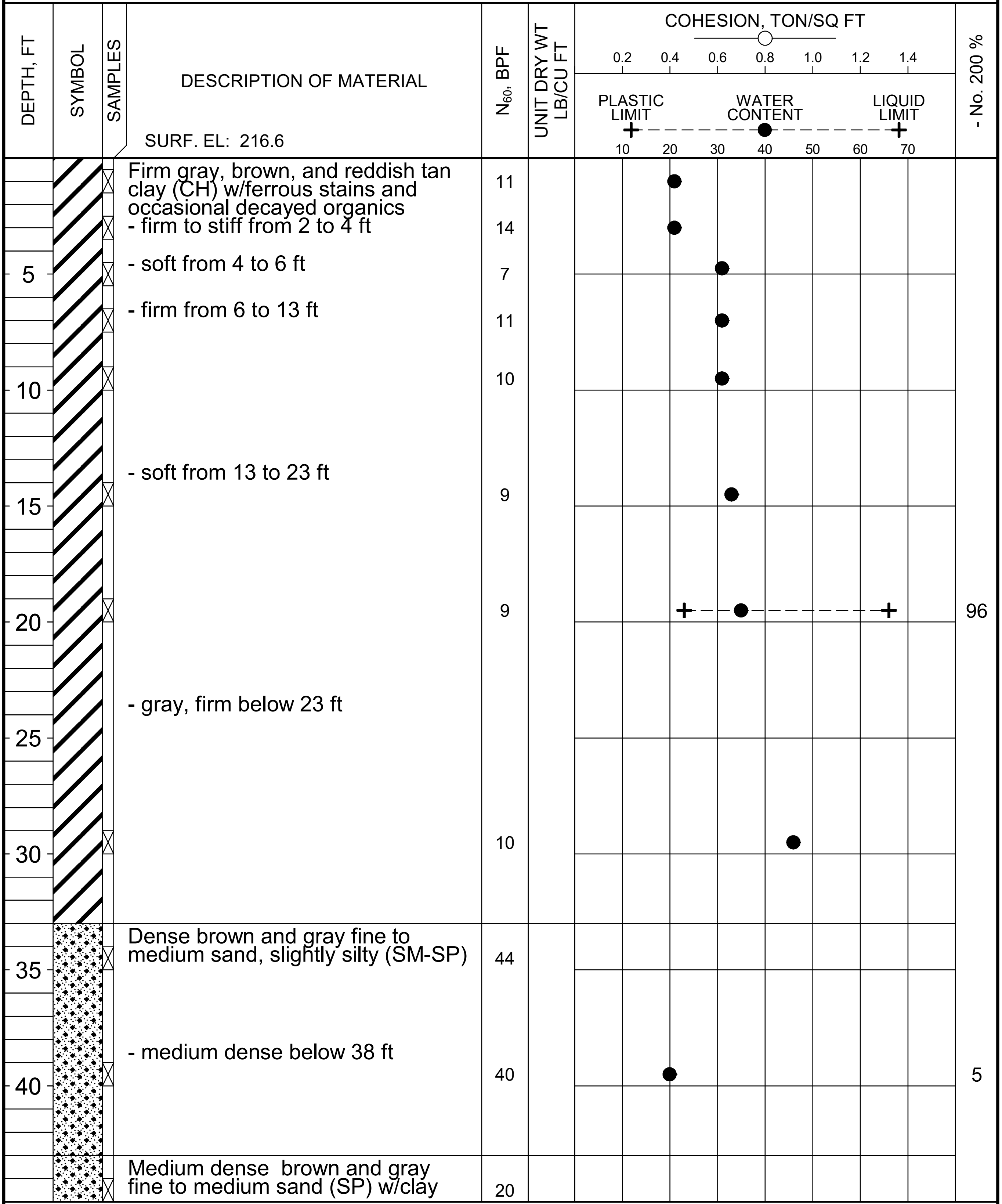
LOG OF BORING NO. B2

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 20 ft /Wash

LOCATION: Approx Sta 615+40, 30 ft Lt

SURF. EL: 216.6



DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Firm gray, brown, and reddish tan clay (CH) w/ferrous stains and occasional decayed organics	11		
- firm to stiff from 2 to 4 ft	14		
- soft from 4 to 6 ft	7		
- firm from 6 to 13 ft	11		
	10		
- soft from 13 to 23 ft	9		
	9		96
- gray, firm below 23 ft			
	10		
Dense brown and gray fine to medium sand, slightly silty (SM-SP)	44		
- medium dense below 38 ft	40		5
Medium dense brown and gray fine to medium sand (SP) w/clay	20		

COMPLETION DEPTH: 120.0 ft
DATE: 6-6-23

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 5/28/2023

23-031

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

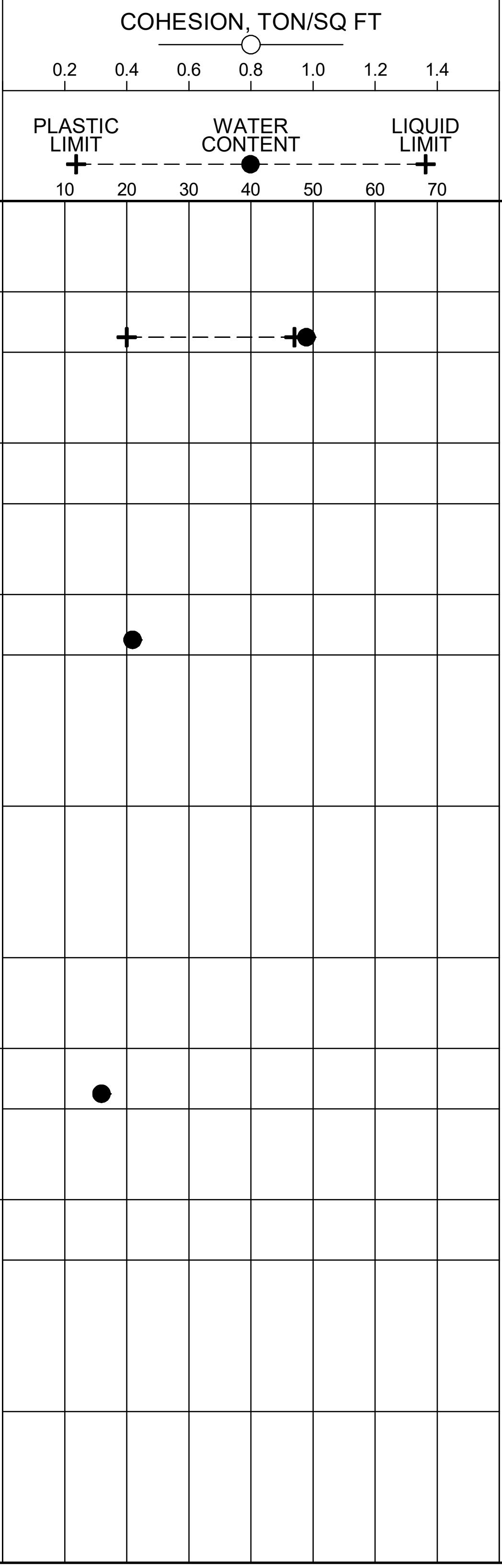
LOG OF BORING NO. B2

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 20 ft /Wash

LOCATION: Approx Sta 615+40, 30 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	pockets and seams			
50	Firm gray silty clay (CL) w/trace fine to coarse gravel	10		88
55	Dense brownish gray fine to medium sand, slightly silty (SM-SP)	56		
60	Dense grayish brown fine sand, slightly silty (SM-SP) w/occasional organic inclusions	49		5
65		51		
70		43		
75	Medium dense grayish brown fine to medium sand (SW) w/trace coarse sand and fine gravel	36		4
80	Dense grayish brown fine to medium sand, slightly silty (SM-SP)	43		
85	- tan from 83 to 88 ft	67		
	- with dark gray nodules from 88 to 93 ft	86		



COMPLETION DEPTH: 120.0 ft
DATE: 6-6-23

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 5/28/2023

LGBNEW 23-031 BRIDGE B.GPJ 8-1-23

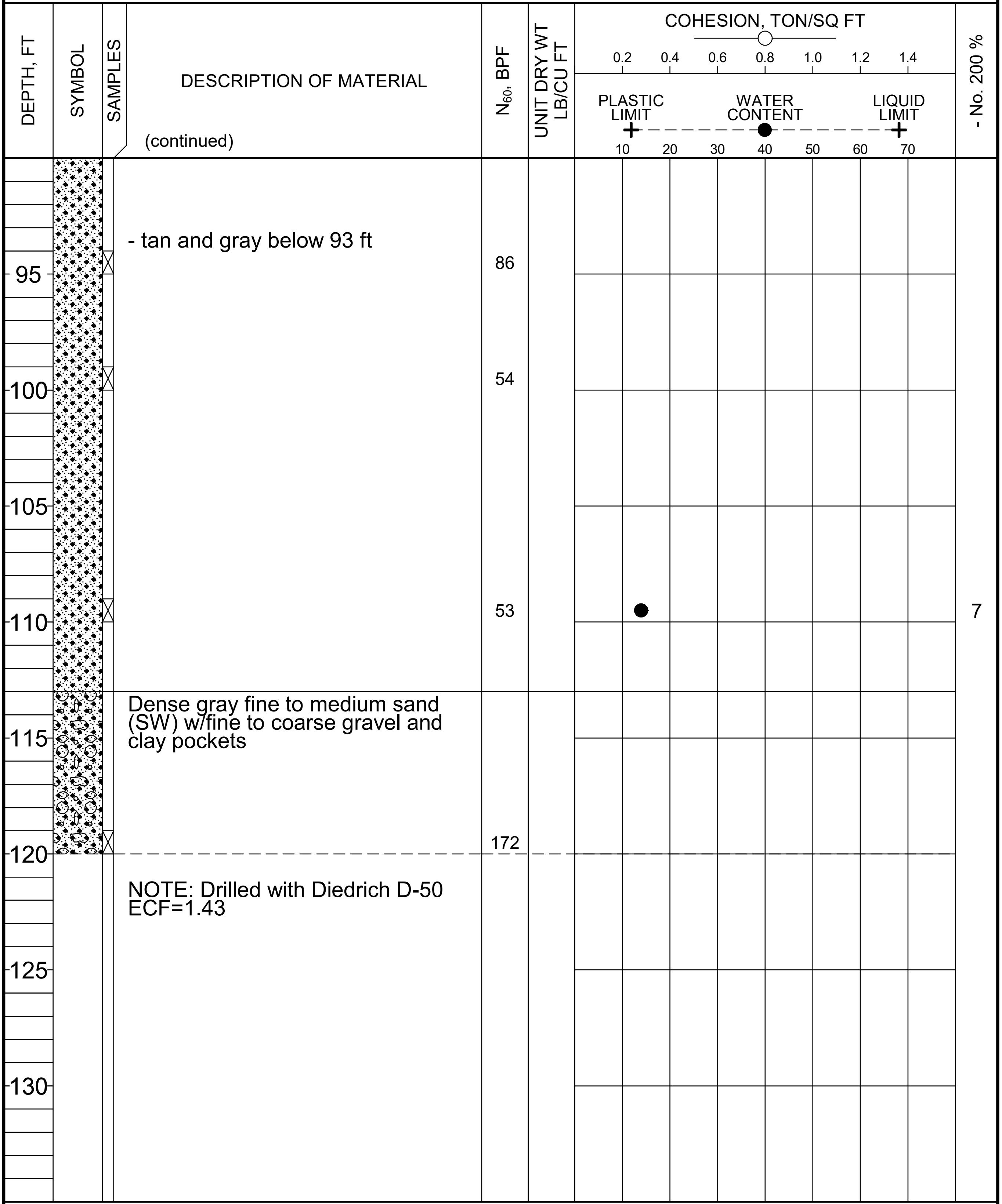
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. B2

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 20 ft /Wash

LOCATION: Approx Sta 615+40, 30 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	- tan and gray below 93 ft	86		
		54		
		53		7
	Dense gray fine to medium sand (SW) w/fine to coarse gravel and clay pockets			
120		172		
	NOTE: Drilled with Diedrich D-50 ECF=1.43			

COMPLETION DEPTH: 120.0 ft
DATE: 6-6-23

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 5/28/2023

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

LOG OF BORING NO. B3

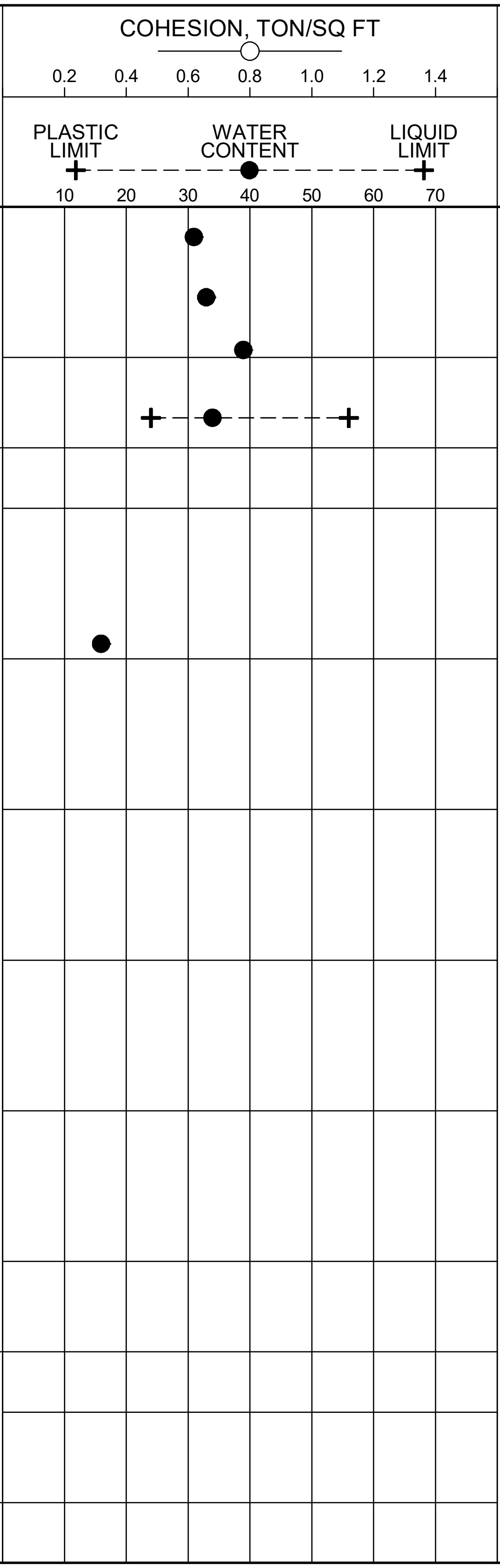
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 616+70, 10 ft Rt

SURF. EL: 203.3

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft gray and brown clay (CH)	6		
	- brown, slightly sandy below 2 ft	6		
5	- soft below 4 ft	7		
		7		83
10	Medium dense tan and brown fine to medium sand, slightly silty (SP-SM)	38		
	- grayish brown below 13 ft			
15		17		5
	- loose at 18 to 23 ft			
20		10		
	- medium dense to dense below 23 ft			
25		37		
30		62		
35		31		
40	Dense brownish gray fine to medium sand, slightly silty (SP-SM) w/trace coarse sand	48		
	Dense brownish gray fine to medium sand, slightly silty	44		



COMPLETION DEPTH: 80.0 ft
DATE: 7-26-23

DEPTH TO WATER
IN BORING: 13 ft

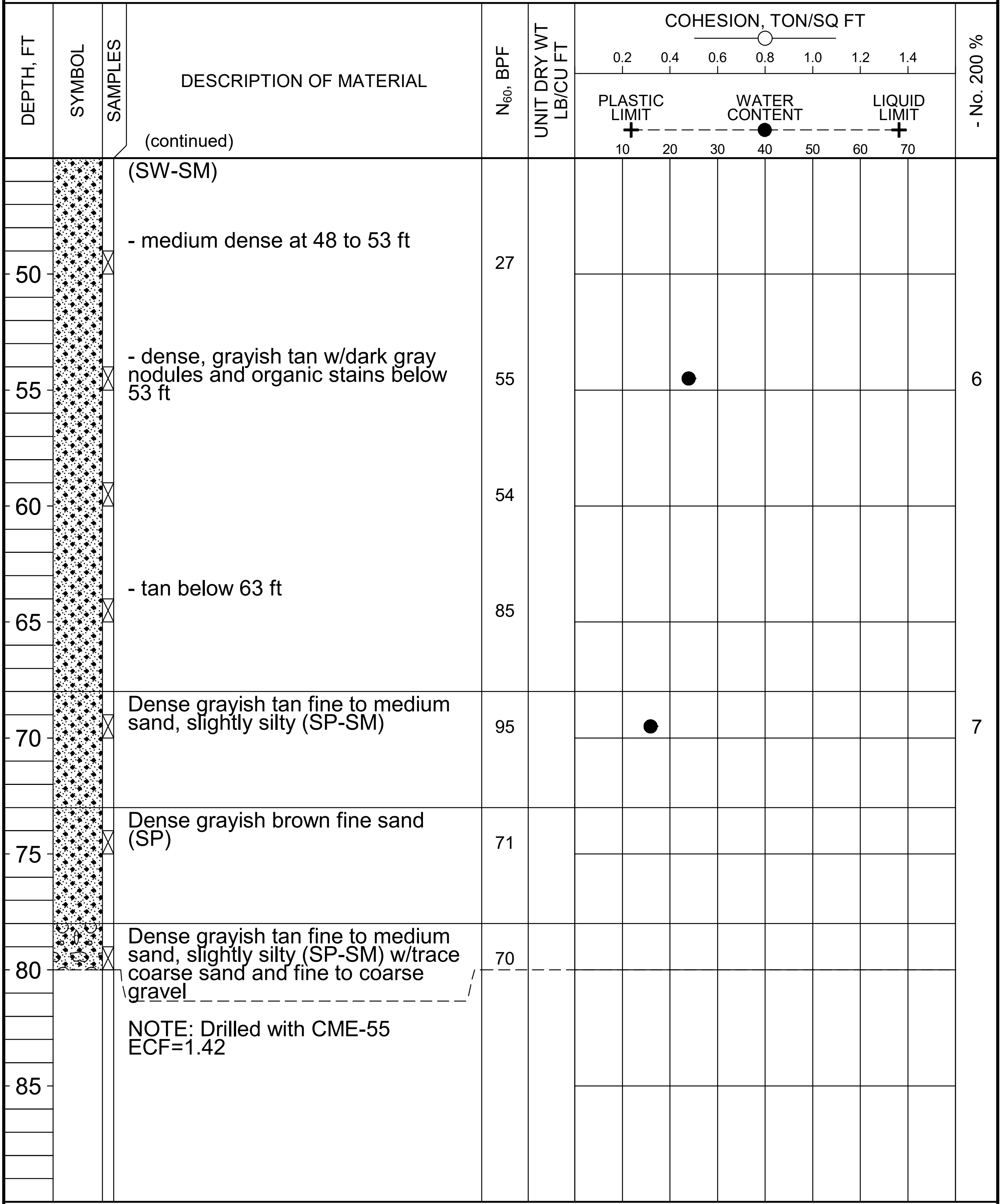
DATE: 7/26/2023

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

LOG OF BORING NO. B3

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: Auger to 15 ft /Wash
LOCATION: Approx Sta 616+70, 10 ft Rt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	(SW-SM)			
	- medium dense at 48 to 53 ft			
50		27		
55	- dense, grayish tan w/dark gray nodules and organic stains below 53 ft	55		6
60		54		
65	- tan below 63 ft	85		
70	Dense grayish tan fine to medium sand, slightly silty (SP-SM)	95		7
75	Dense grayish brown fine sand (SP)	71		
80	Dense grayish tan fine to medium sand, slightly silty (SP-SM) w/trace coarse sand and fine to coarse gravel	70		
85				

NOTE: Drilled with CME-55
ECF=1.42

COMPLETION DEPTH: 80.0 ft
DATE: 7-26-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 7/26/2023

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

LOG OF BORING NO. B4

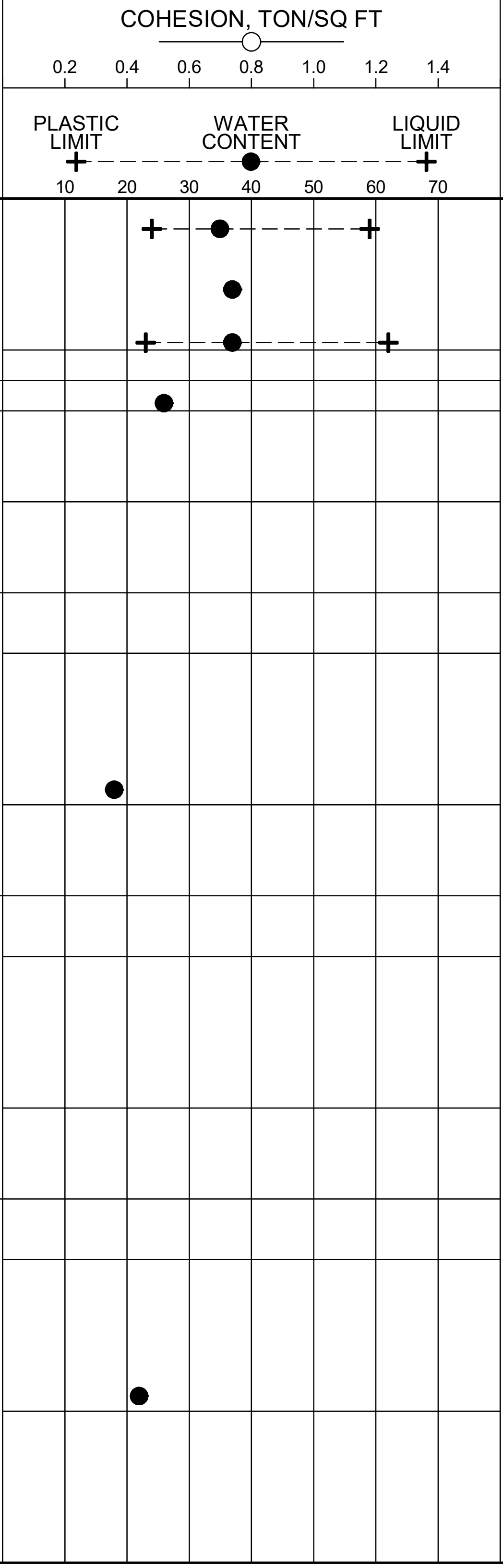
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 15 ft /Wash

LOCATION: Approx Sta 617+15, 20 ft Lt

SURF. EL: 201.3

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft gray and brown clay (CH)	7		97
		7		
5	- very soft to soft, slightly sandy below 4 ft	6		90
	Firm gray clayey silt, slightly sandy (CL-ML) w/ferrous stains	13		
10	Loose grayish tan fine sand, slightly silty (SP-SM) - medium dense below 8 ft	37		
15	Medium dense grayish tan fine to medium sand (SP)	40		
20	- medium dense to dense below 18 ft	43		4
25	Dense brownish gray fine sand, slightly silty (SP-SM) w/occasional dark gray nodules and organic stains	50		
30	- medium dense below 28 ft	34		
35	Medium dense grayish tan fine to medium sand (SP) w/occasional dark gray nodules and organic stains	38		
40	- dense below 38 ft	58		4
		51		



COMPLETION DEPTH: 100.0 ft
DATE: 7-25-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 7/25/2023

LGBNEW 23-031 BRIDGE B.GPJ 8-1-23

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

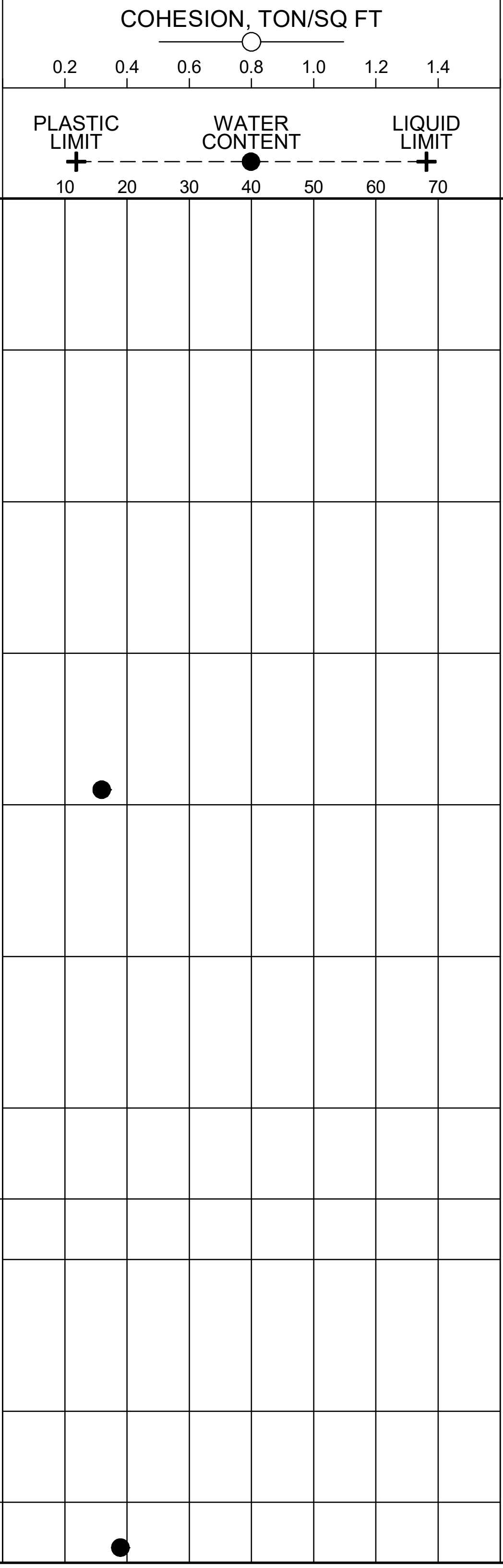
LOG OF BORING NO. B4

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 15 ft /Wash

LOCATION: Approx Sta 617+15, 20 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		68		
55		62		
60		61		
65	- with trace coarse sand and fine gravel (SW) below 63 ft	53		4
70		48		
75		47		
80	Dense grayish tan fine sand, slightly silty (SP-SM) w/occasional dark gray nodules and organic stains	85		
85		84		
	Dense grayish tan fine to medium sand, slightly silty (SP-SM) w/trace	95		6



COMPLETION DEPTH: 100.0 ft
DATE: 7-25-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 7/25/2023

LGBNEW 23-031 BRIDGE B.GPJ 8-1-23

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

LOG OF BORING NO. B4

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 15 ft /Wash

LOCATION: Approx Sta 617+15, 20 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT	- No. 200 %
95	coarse sand and occasional dark gray nodules and organic stains - with more coarse sand below 93 ft	78			
100	Dense grayish tan fine to coarse sand, slightly silty (SP-SM) w/trace fine gravel	78			
	NOTE: Drilled with CME-55 ECF=1.42				
105					
110					
115					
120					
125					
130					

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

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

COMPLETION DEPTH: 100.0 ft
DATE: 7-25-23

DEPTH TO WATER IN BORING: 13 ft
DATE: 7/25/2023

23-031

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

LOG OF BORING NO. B5

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

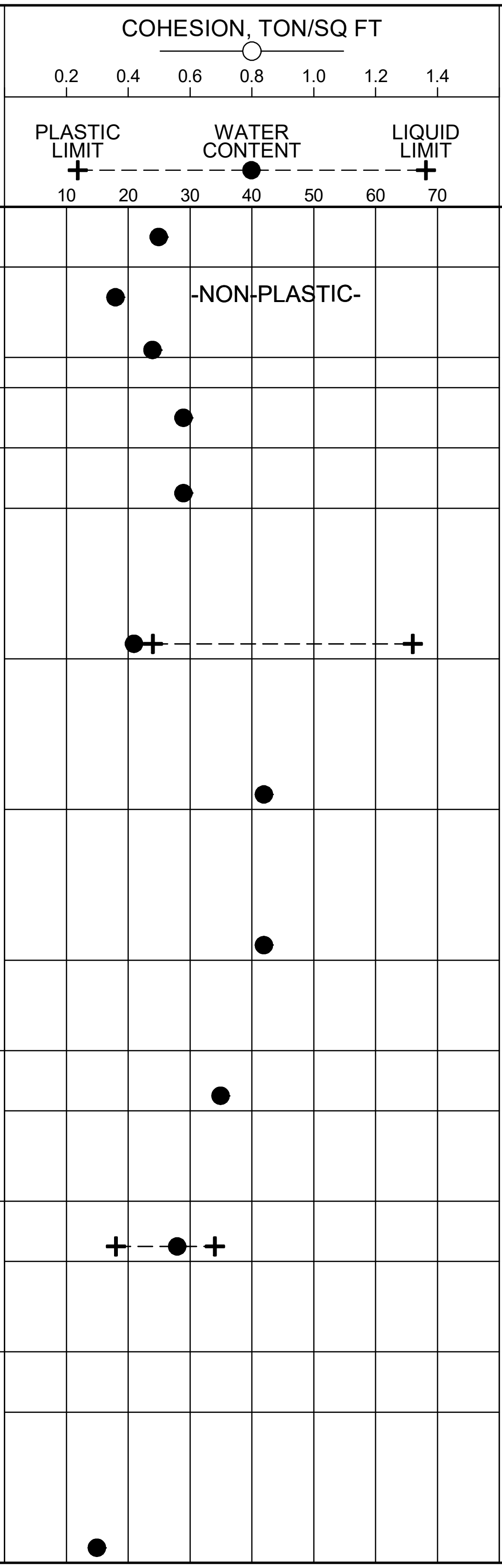
TYPE: HSA to 30 ft /Wash

LOCATION: Approx Sta 617+90, 10 ft Rt

SURF. EL: 214.9

DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Soft brown silty clay, slightly sandy (CL)	7		
Firm brown fine sandy clay (CL) w/silty fine sand seams	10		48
	10		
Soft brown clay (CH) w/silty fine sand seams	7		
Very soft to soft brown and gray clay, slightly sandy (CH)	6		
- stiff below 13 ft	16		83
	17		
	17		
Stiff gray clay (CH), slightly sandy w/occasional organic inclusions	19		
Stiff gray fine sandy clay (CL)	17		55
Medium dense brownish gray fine to medium sand (SP)	37		
- dense below 43 ft	43		4

DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40



COMPLETION DEPTH: 110.0 ft
DATE: 6-13-23

DEPTH TO WATER
IN BORING: 29.2 ft

DATE: 6/12/2023

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

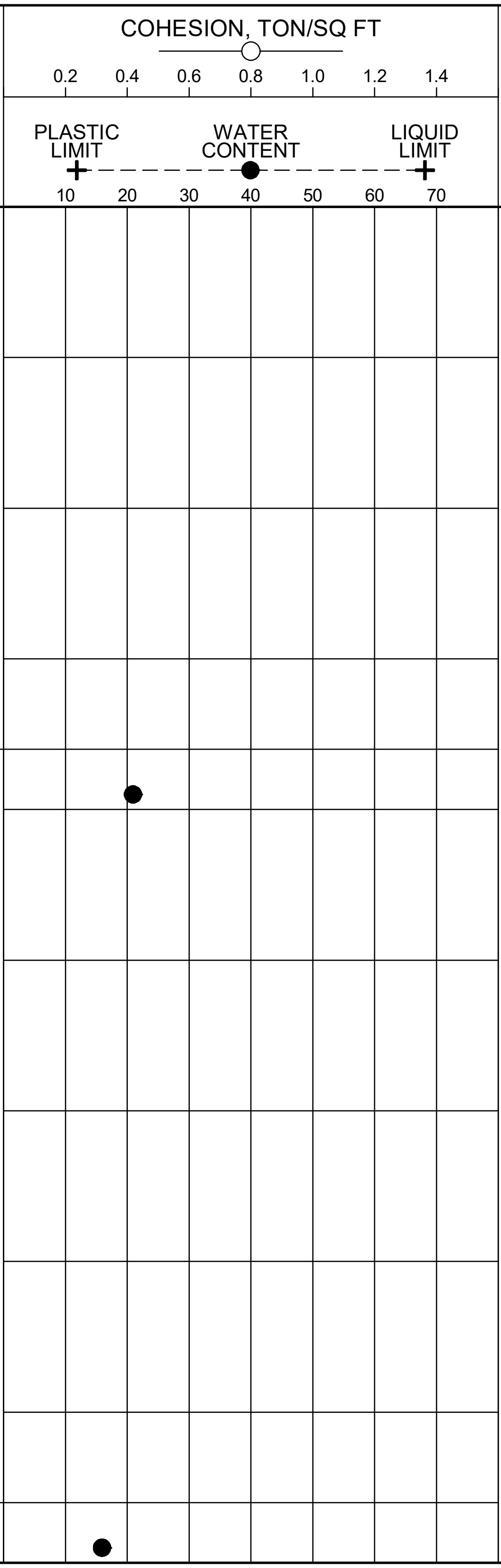
LOG OF BORING NO. B5

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 30 ft /Wash

LOCATION: Approx Sta 617+90, 10 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	- tan and gray with fine gravel at 49 to 54 ft	46		
55		53		
60	- dark gray and gray below 59 ft	56		
65	Dense brownish gray fine sand, slightly silty (SM-SP)	64		6
70		60		
75		70		
80		69		
85		66		
	Dense brownish gray fine to medium sand, slightly silty	60		5



COMPLETION DEPTH: 110.0 ft
DATE: 6-13-23

DEPTH TO WATER
IN BORING: 29.2 ft

DATE: 6/12/2023

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

LOG OF BORING NO. B5

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

TYPE: HSA to 30 ft /Wash

LOCATION: Approx Sta 617+90, 10 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
			(SM-SW) w/trace coarse sand				
95		☒		51			
100		☒		69			
105							
110		☒		84			
			NOTE: Drilled with Diedrich D-50 ECF=1.43				
115							
120							
125							
130							

LGBNEW 23-031 BRIDGE B.GPJ 8-1-23

COMPLETION DEPTH: 110.0 ft
DATE: 6-13-23

DEPTH TO WATER
IN BORING: 29.2 ft

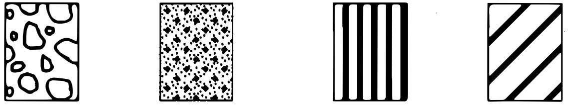
DATE: 6/12/2023

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

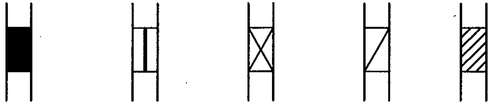
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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

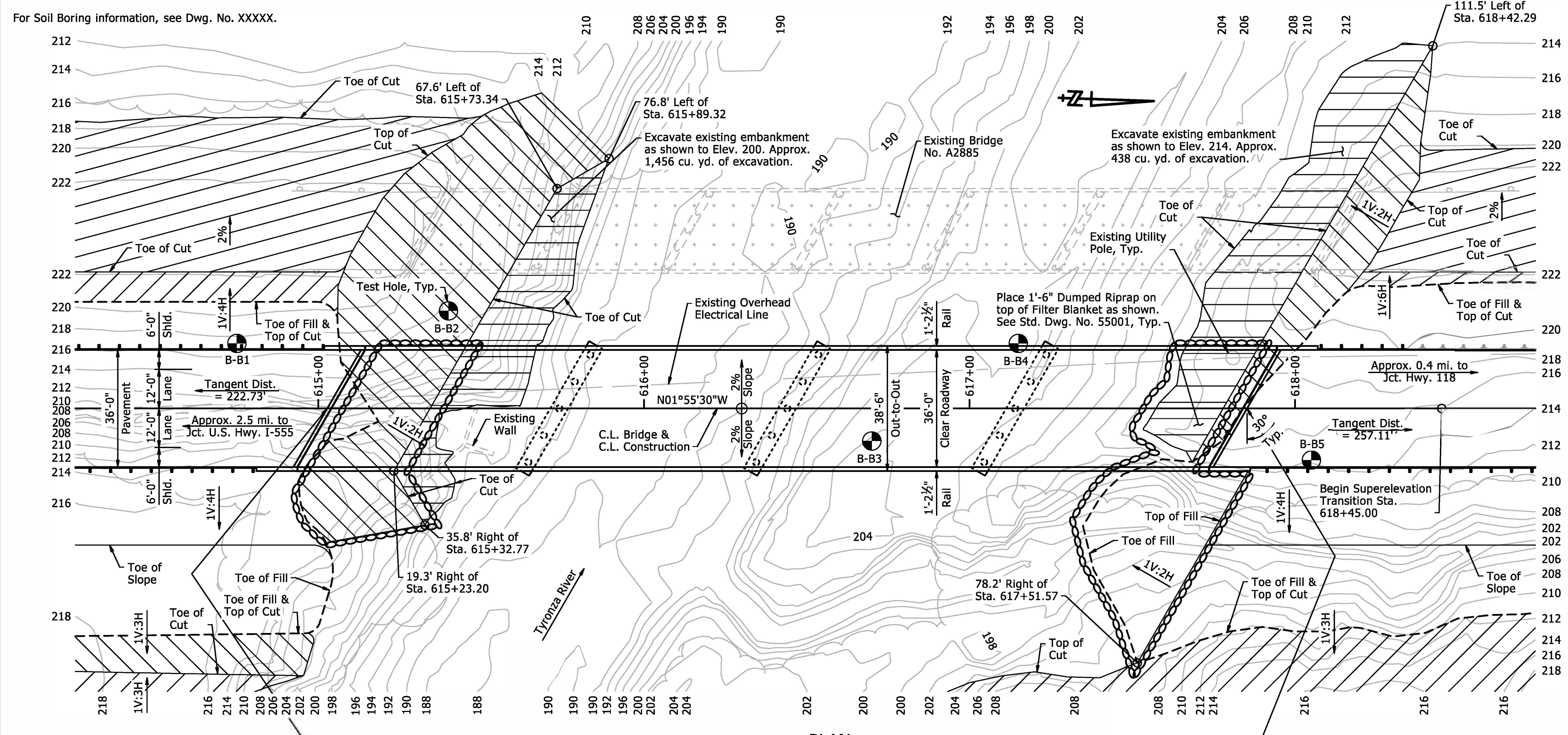
KEY 7-28-11

APPENDIX A

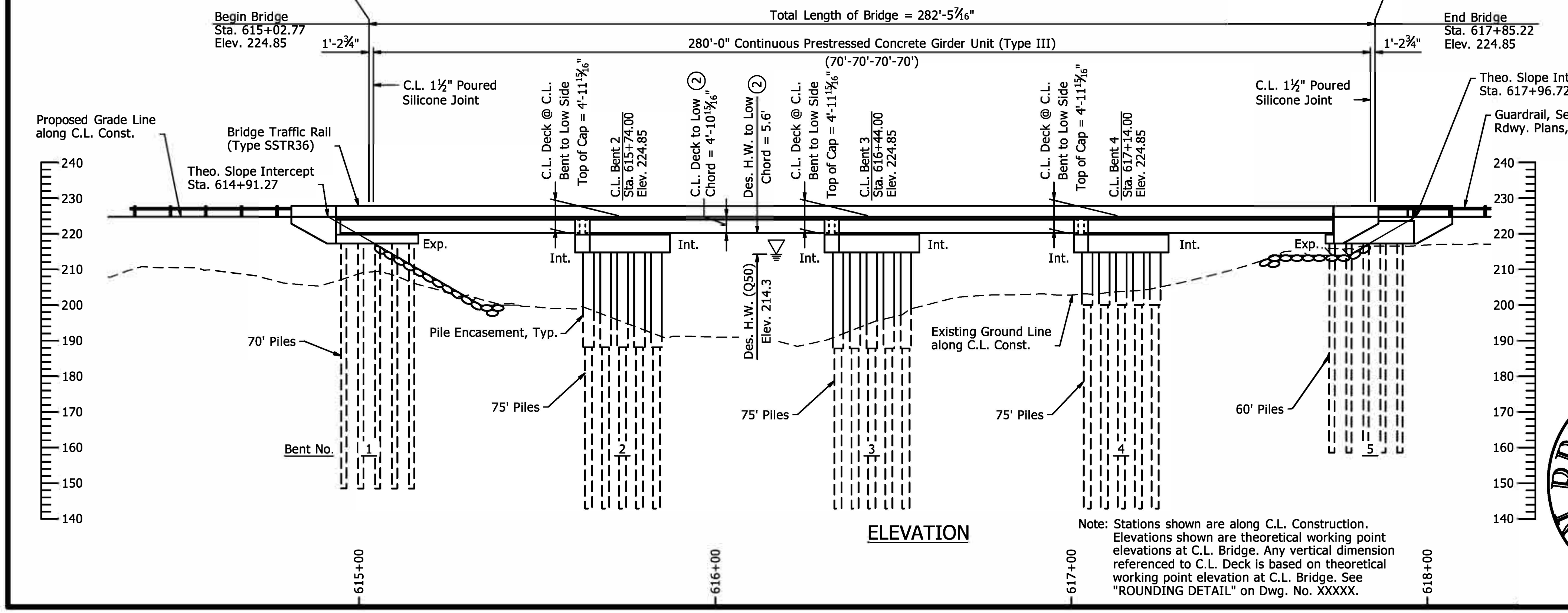
DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	170	356
		07649		LAYOUT		66616

For R/W Data, See Roadway Plans.

For Soil Boring information, see Dwg. No. XXXXX.



PLAN



ELEVATION

Note: Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "ROUNDING DETAIL" on Dwg. No. XXXXX.

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

SEISMIC ZONE: 4 S_{D1} = 1.197 SITE CLASS: D

SEISMIC OPERATIONAL CLASS: OTHER

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 6,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

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

STEEL SHELL PILING: Piling in Bents 1 and 5 shall be 18" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of 385 and 352 tons per pile, respectively. Piling in Bents 2, 3, and 4 shall be 28" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of 607, 807, and 1045 tons per pile, respectively. All piling shall be driven with an approved air, steam, or diesel hammer to a minimum tip elevation of 148 and 158 or lower at Bents 1 and 5, respectively, and to a minimum tip elevation of 143 or lower at Bents 2 thru 4. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (__" Dia.)".

For additional General Notes, see Dwg. No. 66617.

HYDRAULIC DATA

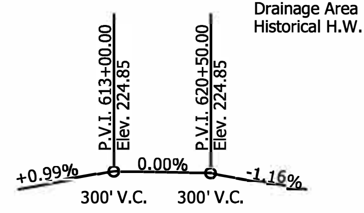
FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	① NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	9,260	214.3	214.3
BASE	100	10,050	215.3	215.3
EXTREME	500	11,800	216.9	217.0
OVERTOPPING	>500	---	---	---

① Unconstricted water surface elevation without structure or roadway approaches.

② Proposed Low Bridge Chord Elev. = 219.94 feet

100 yr. backwater elevation for existing structure = 215.3 feet
Drainage Area = 290.0 sq. miles
Historical H.W. Elev. = N/A

Note: Use Type 2 Special Approach Gutters and Type C2 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. XXXXX, XXXXX, & 55040C2, respectively.



VERTICAL ALIGNMENT DATA

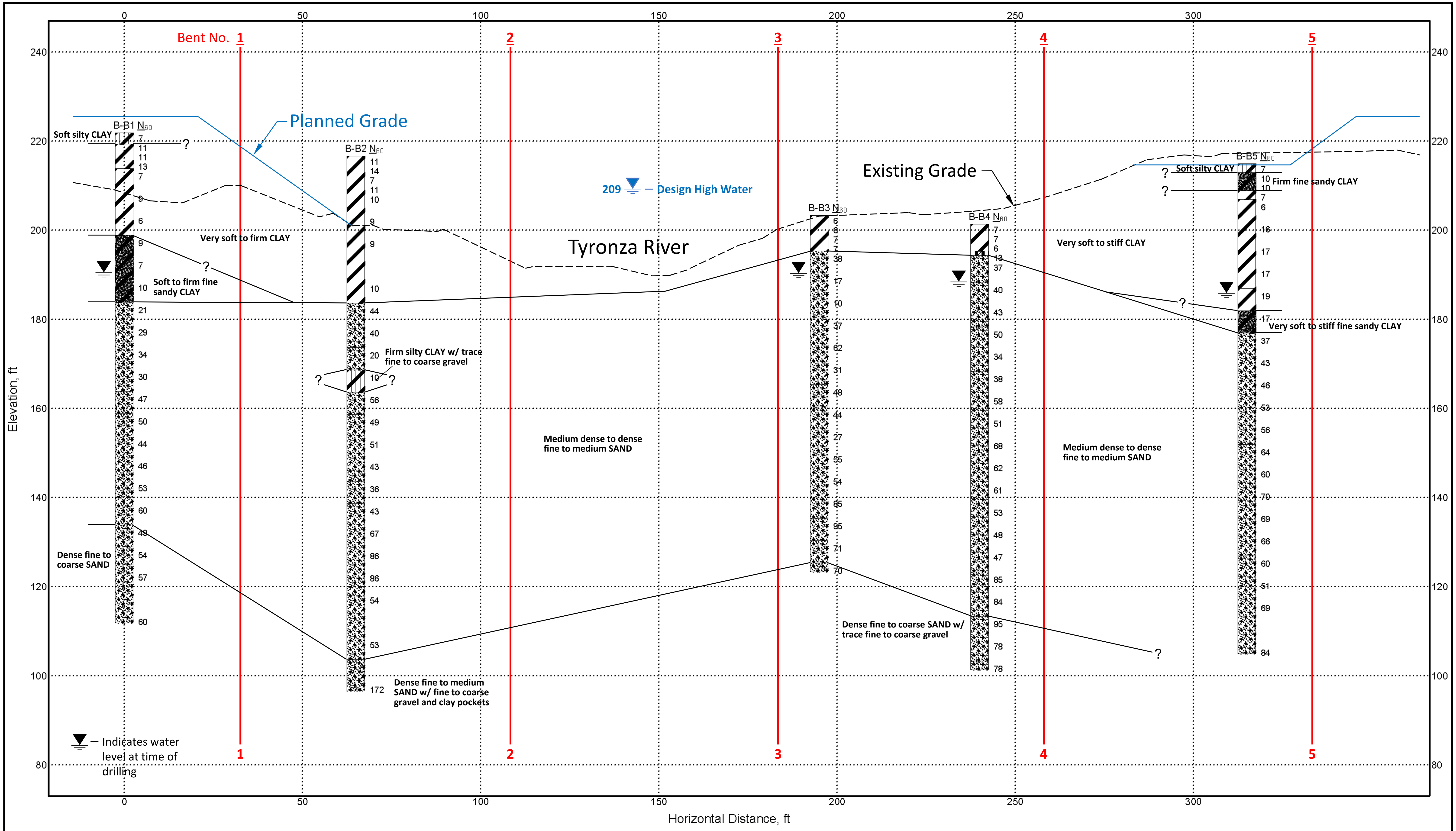
Theoretical Elev. Along
C.L. Construction

PRELIMINARY SUBJECT TO REVISION

SHEET 1 OF 2
LAYOUT OF BRIDGE
HWY. 135 OVER TYRONZA RIVER
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: LDG DATE: 02-14-2023 FILENAME: b101124x2_l1.dgn
CHECKED BY: CAW DATE: 02-28-2023 SCALE: 1" = 20'
DESIGNED BY: LDG DATE: 02-10-2023
BRIDGE NO. 07649 DRAWING NO. 66616

APPENDIX B



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

SCALE:
1" = 25' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Tyronza River (Site 2)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
B1	4.5-5.5	31	70	27	43	---	---	---	---	100	---	---	98	CH	A-7-6
B1	14-15	33	57	20	37	---	---	---	---	100	---	---	78	CH	A-7-6
B1	29-30	66	79	35	44	---	---	---	---	100	---	---	65	CH	A-7-5
B1	44-45	25	---	---	---	100	100	100	100	100	100	27	3	SW	A-1-b
B1	59-60	20	---	---	---	100	100	100	100	100	100	83	5	SM-SP	A-3
B1	74-75	20	---	---	---	100	100	100	100	100	99	46	5	SM-SP	A-1-b
B1	89-90	16	---	---	---	---	---	---	---	88	---	---	5	SM-SP	A-1-b
B2	19-20	35	66	23	43	---	---	---	---	100	---	---	96	CH	A-7-6
B2	39-40	20	---	---	---	100	100	100	100	100	100	41	5	SM-SP	A-1-b
B2	49-50	49	47	20	27	---	---	---	---	94	---	---	88	CL	A-7-6
B2	59-60	21	---	---	---	100	100	100	100	99	98	82	5	SM-SP	A-3
B2	74-75	16	---	---	---	100	100	100	100	96	88	18	4	SW	A-1-b
B2	109-110	14	---	---	---	100	100	100	100	99	98	29	7	SM-SW	A-1-b
B3	6.5-7.5	34	56	24	32	---	---	---	---	100	---	---	83	CH	A-7-6
B3	14-15	16	---	---	---	100	100	100	100	100	100	66	5	SM-SP	A-3
B3	54-55	24	---	---	---	100	100	100	100	99	98	39	6	SM-SW	A-1-b
B3	69-70	16	---	---	---	100	100	100	100	100	97	42	7	SM-SP	A-1-b
B3	74-75	21	---	---	---	100	100	100	100	100	100	83	4	SP	A-3
B4	0.5-1.5	35	59	24	35	---	---	---	---	100	---	---	97	CH	A-7-6
B4	4.5-5.5	37	62	23	39	---	---	---	---	100	---	---	90	CH	A-7-6
B4	19-20	18	---	---	---	100	100	100	99	97	94	57	4	SP	A-3
B4	39-40	22	---	---	---	100	100	100	100	99	98	55	4	SP	A-3
B4	64-65	16	---	---	---	100	100	100	100	94	85	25	4	SW	A-1-b
B4	89-90	19	---	---	---	100	100	100	100	99	95	65	6	SM-SP	A-3

SUMMARY of CLASSIFICATION TEST RESULTS

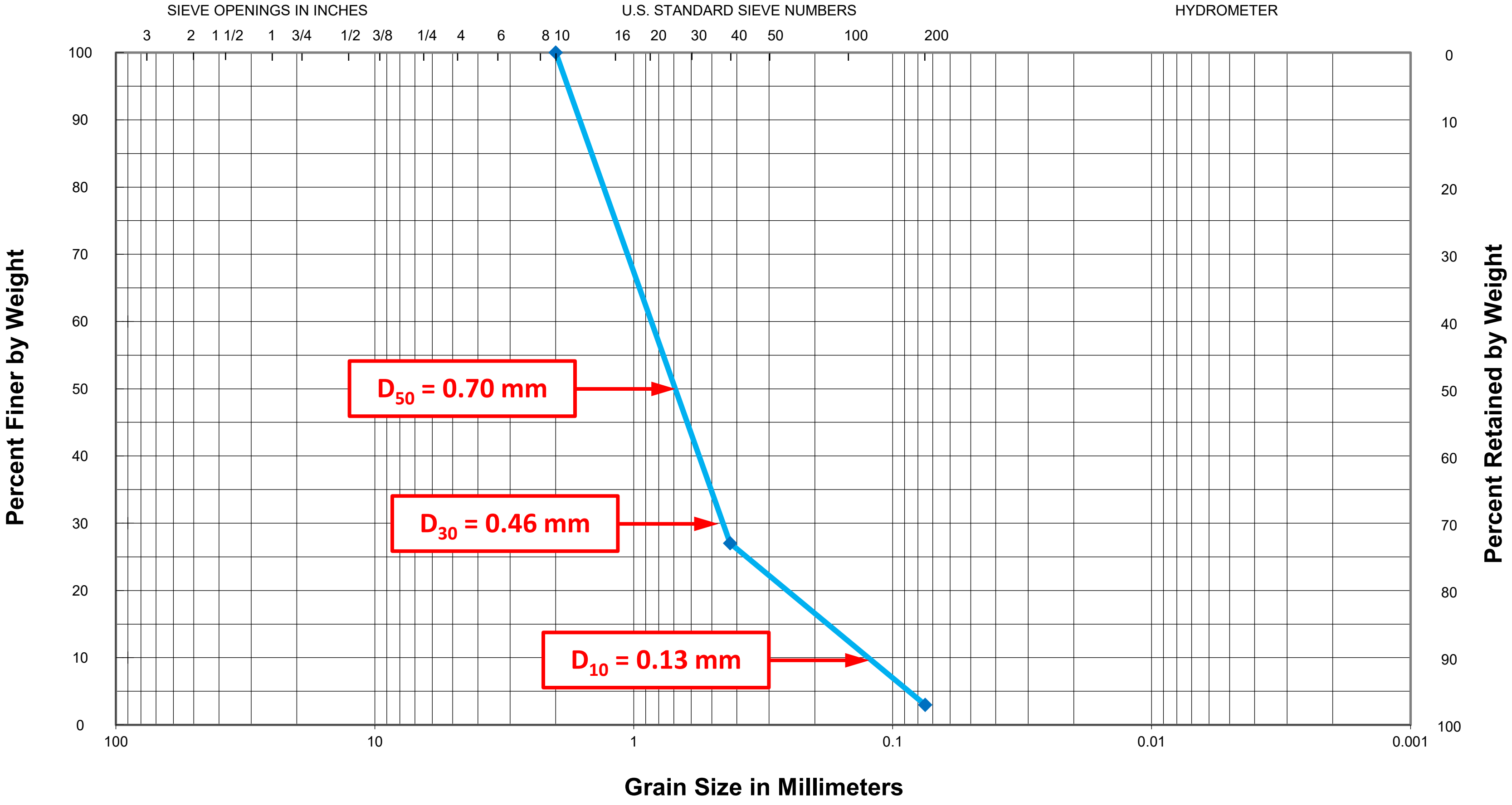
PROJECT: 101124 Hwy. 135 over Tyronza River (Site 2)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
B5	2.5-3.5	18	NON-PLASTIC			---	---	---	---	100	---	---	41	SM	A-4
B5	14-15	21	66	24	42	---	---	---	---	99	---	---	83	CH	A-7-6
B5	34-35	28	34	18	16	---	---	---	---	100	---	---	55	CL	A-6
B5	44-45	15	---	---	---	100	100	100	100	100	97	47	4	SP	A-1-b
B5	64-65	21	---	---	---	100	100	100	100	100	100	90	6	SM-SP	A-3
B5	89-90	16	---	---	---	100	100	100	100	97	91	36	5	SM-SW	A-1-b

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.70 mm

D_{30} = 0.46 mm

D_{10} = 0.13 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

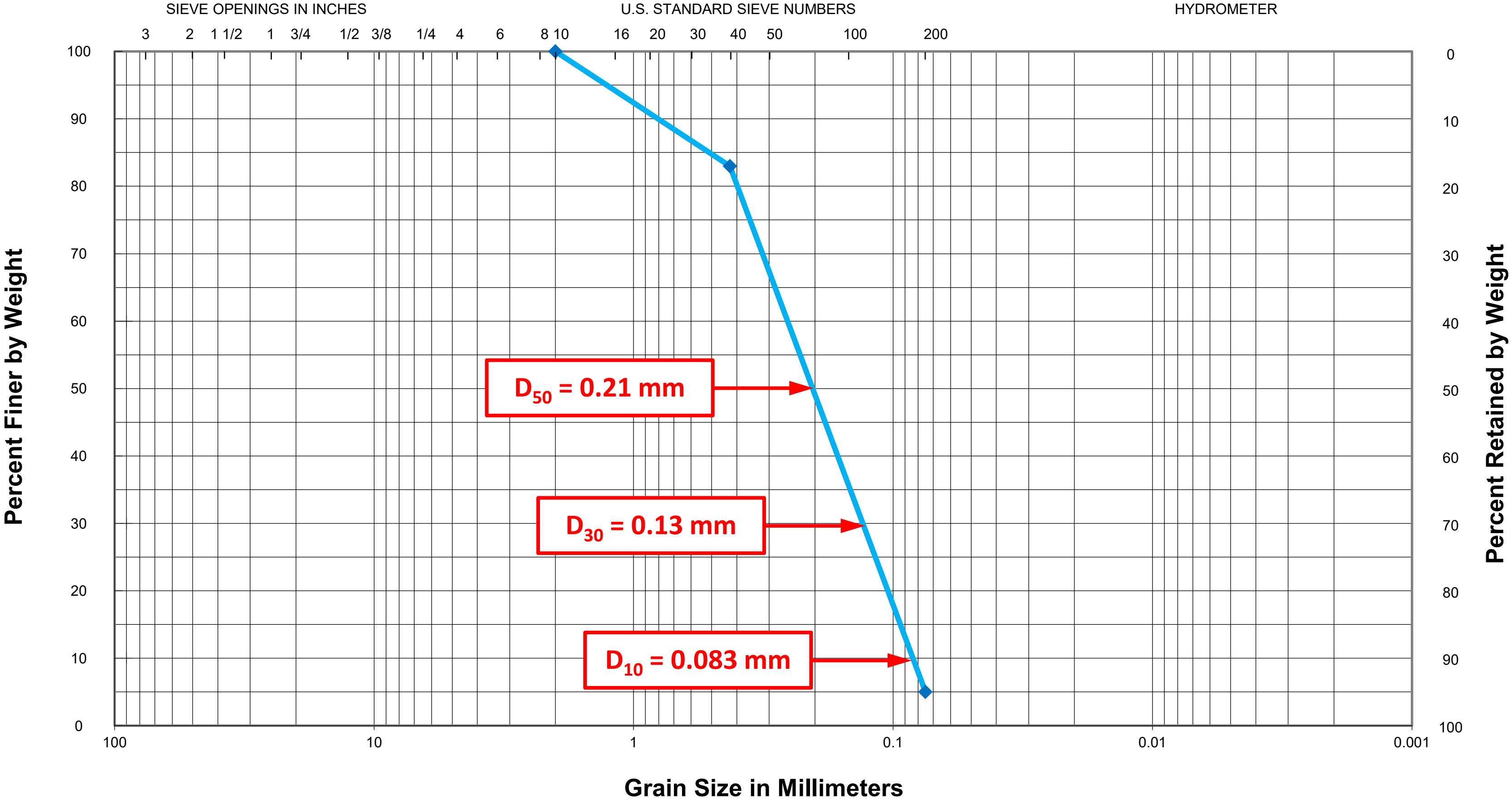
Sample: Boring B1, 44-45 ft
Description: Brownish gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

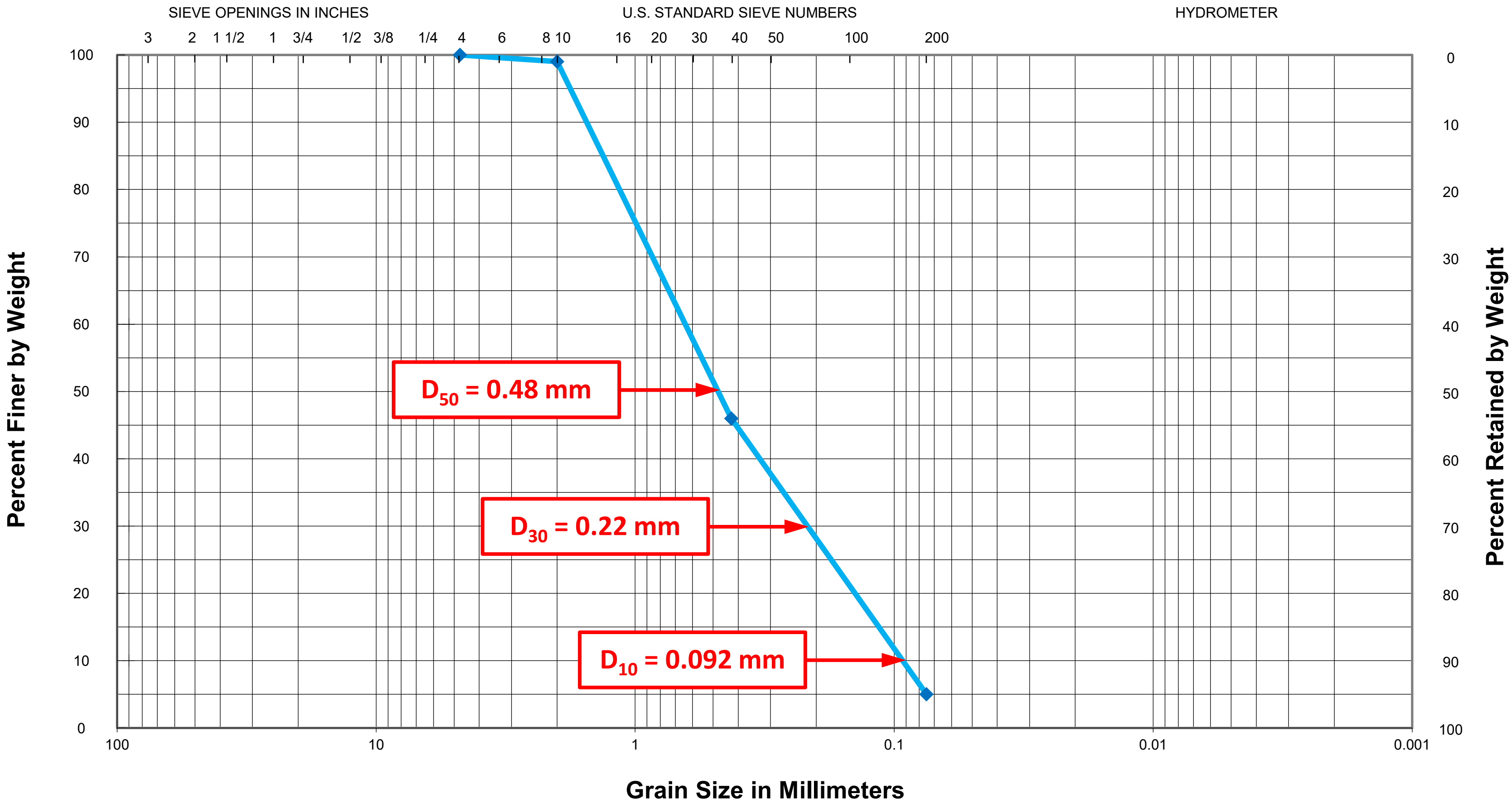
Sample: Boring B1, 59-60 ft
Description: Gray and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

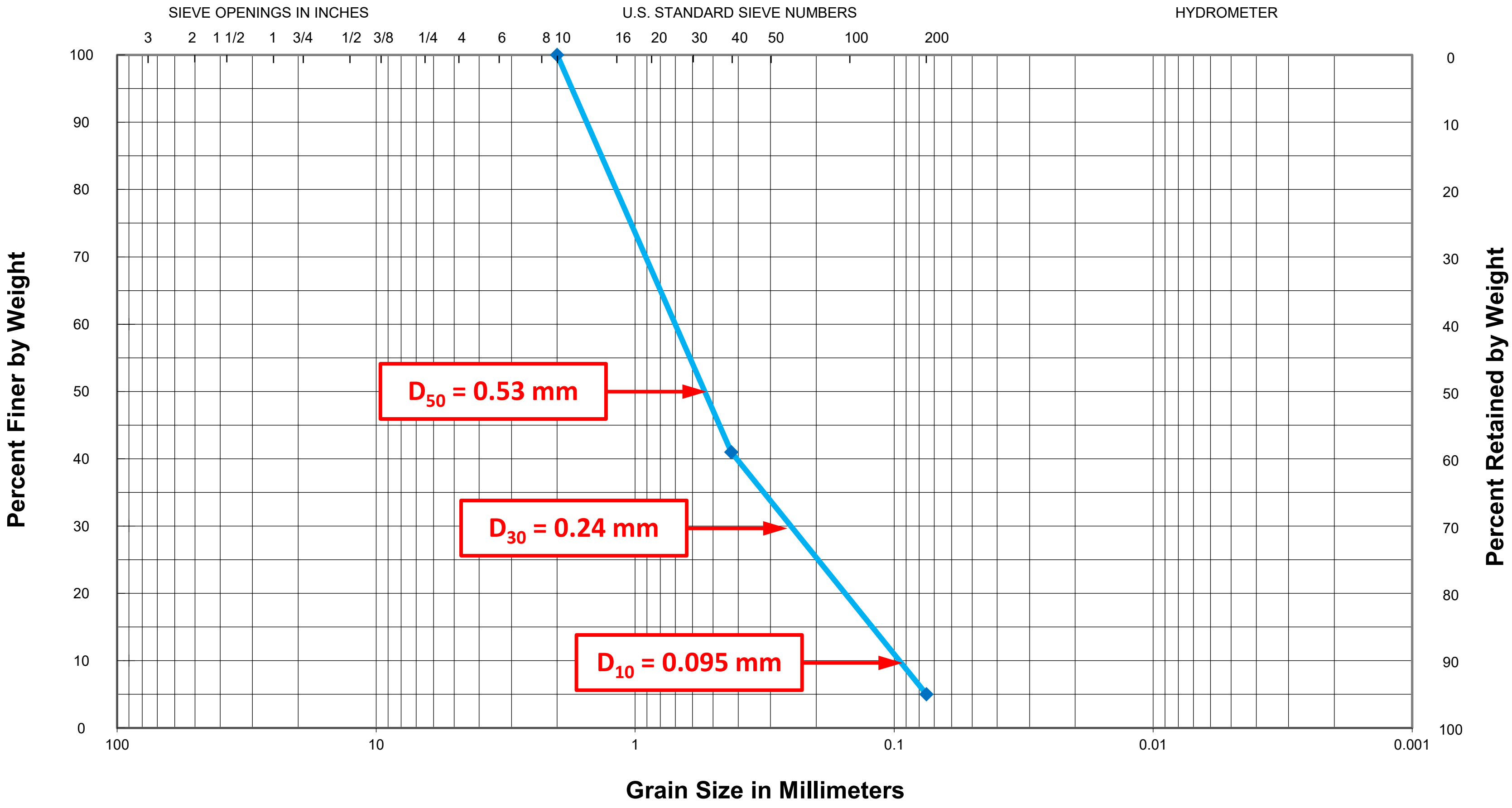
Sample: Boring B1, 74-75 ft
Description: Brownish gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

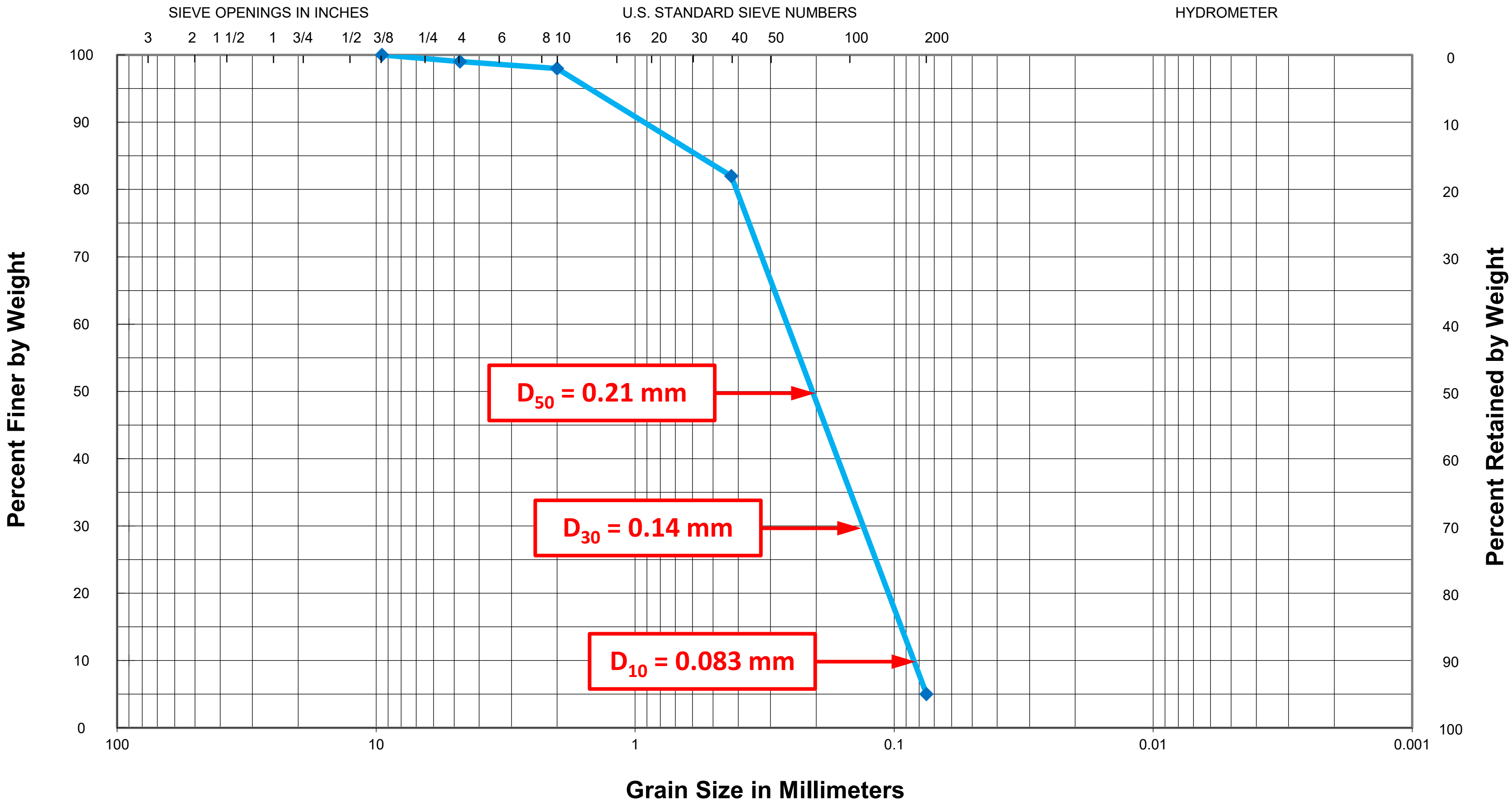
Sample: Boring B2, 39-40 ft
Description: Brown and gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

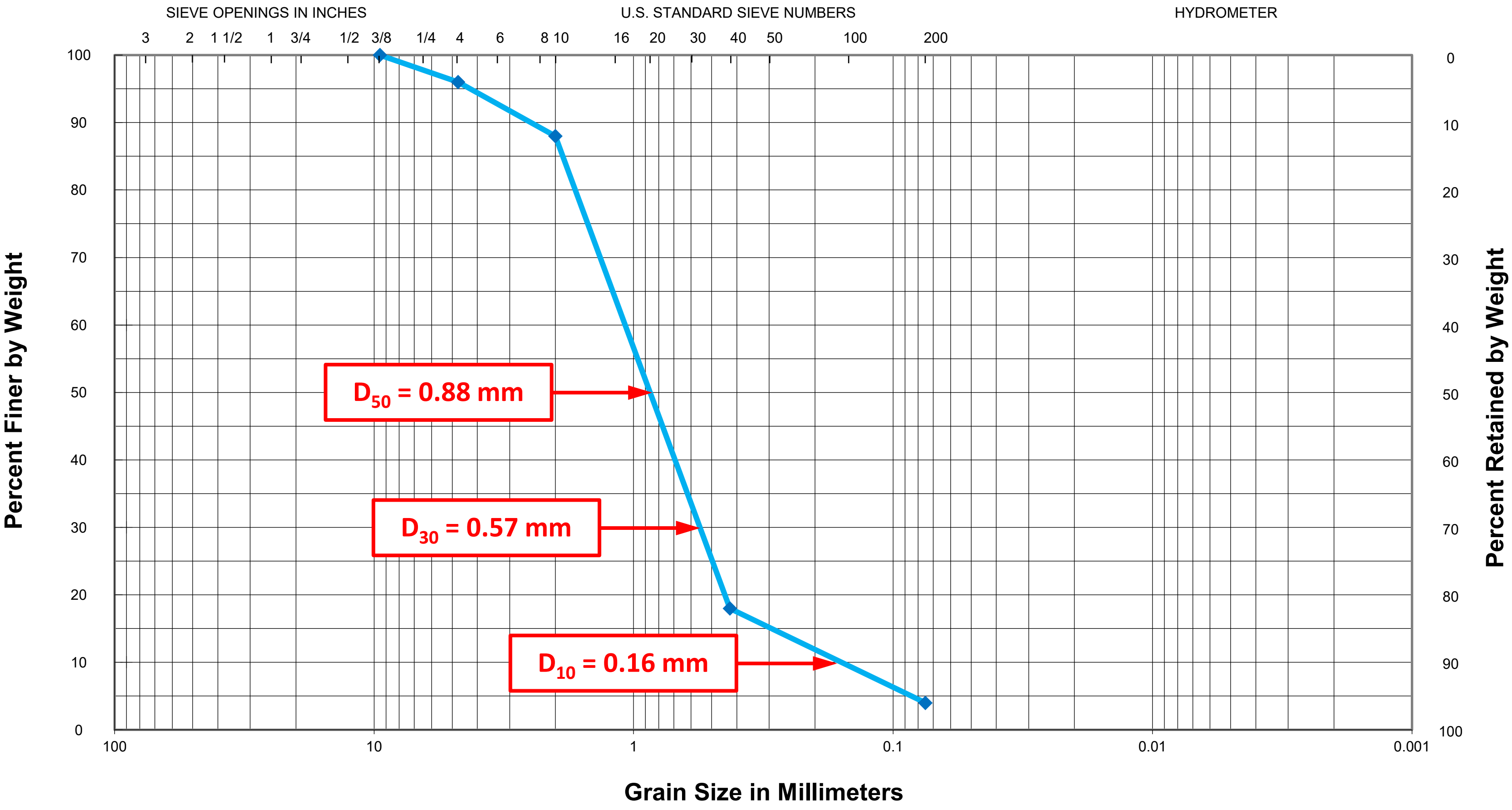
Sample: Boring B2, 59-60 ft
Description: Grayish brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

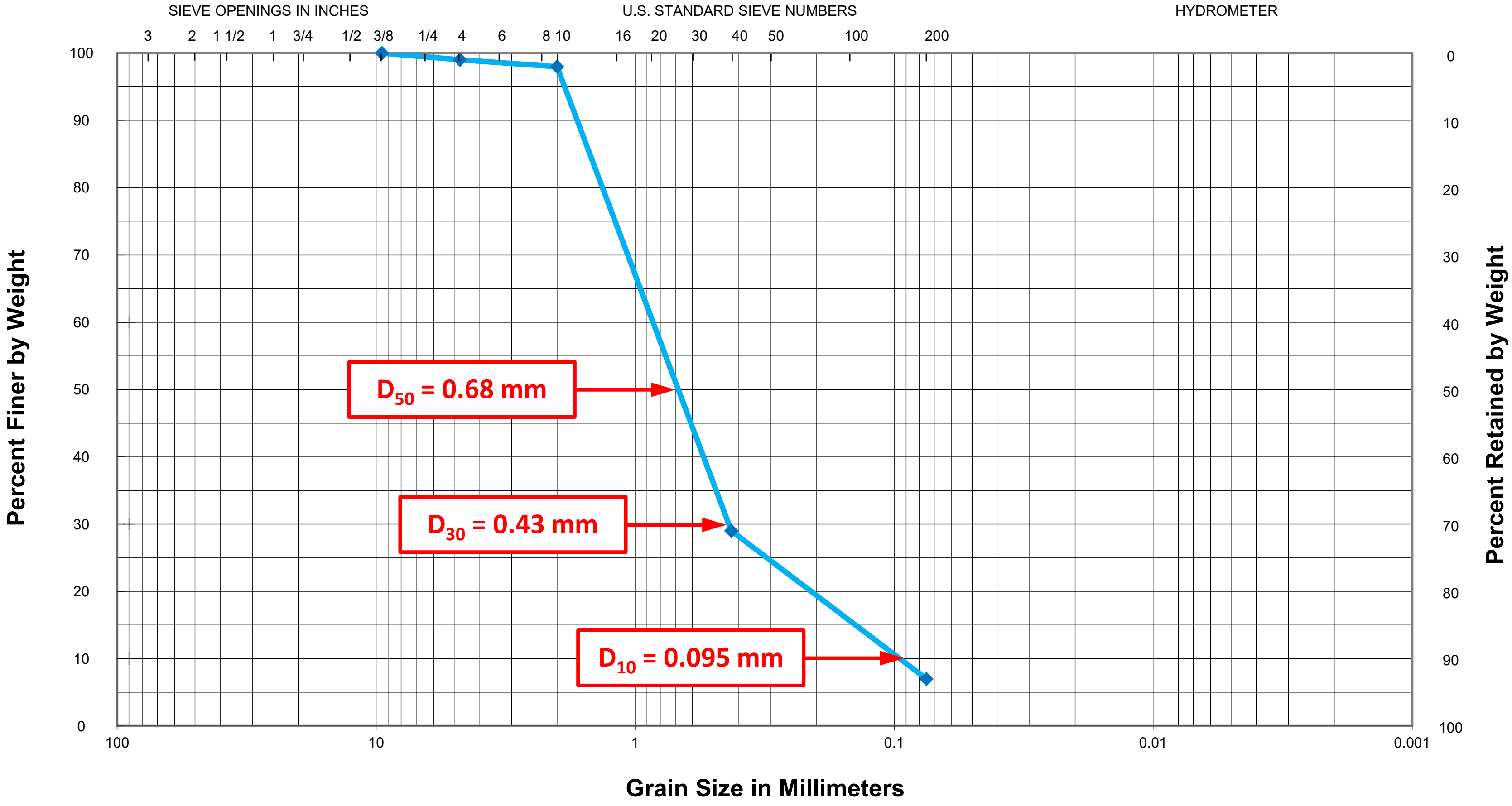
Sample: Boring B2, 74-75 ft
Description: Grayish brown fine to medium SAND w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

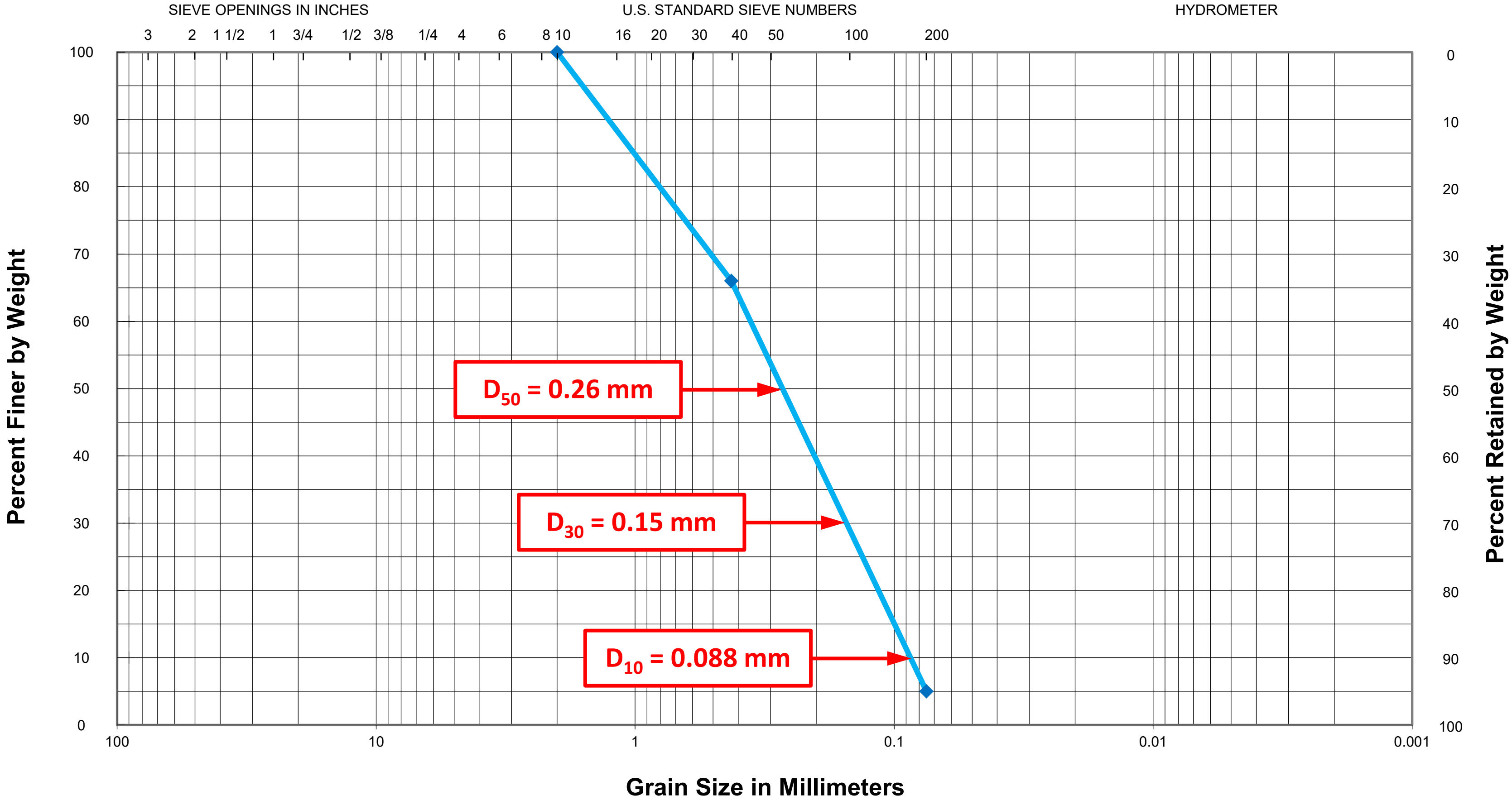
Sample: Boring B2, 109-110 ft
Description: Tan and gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

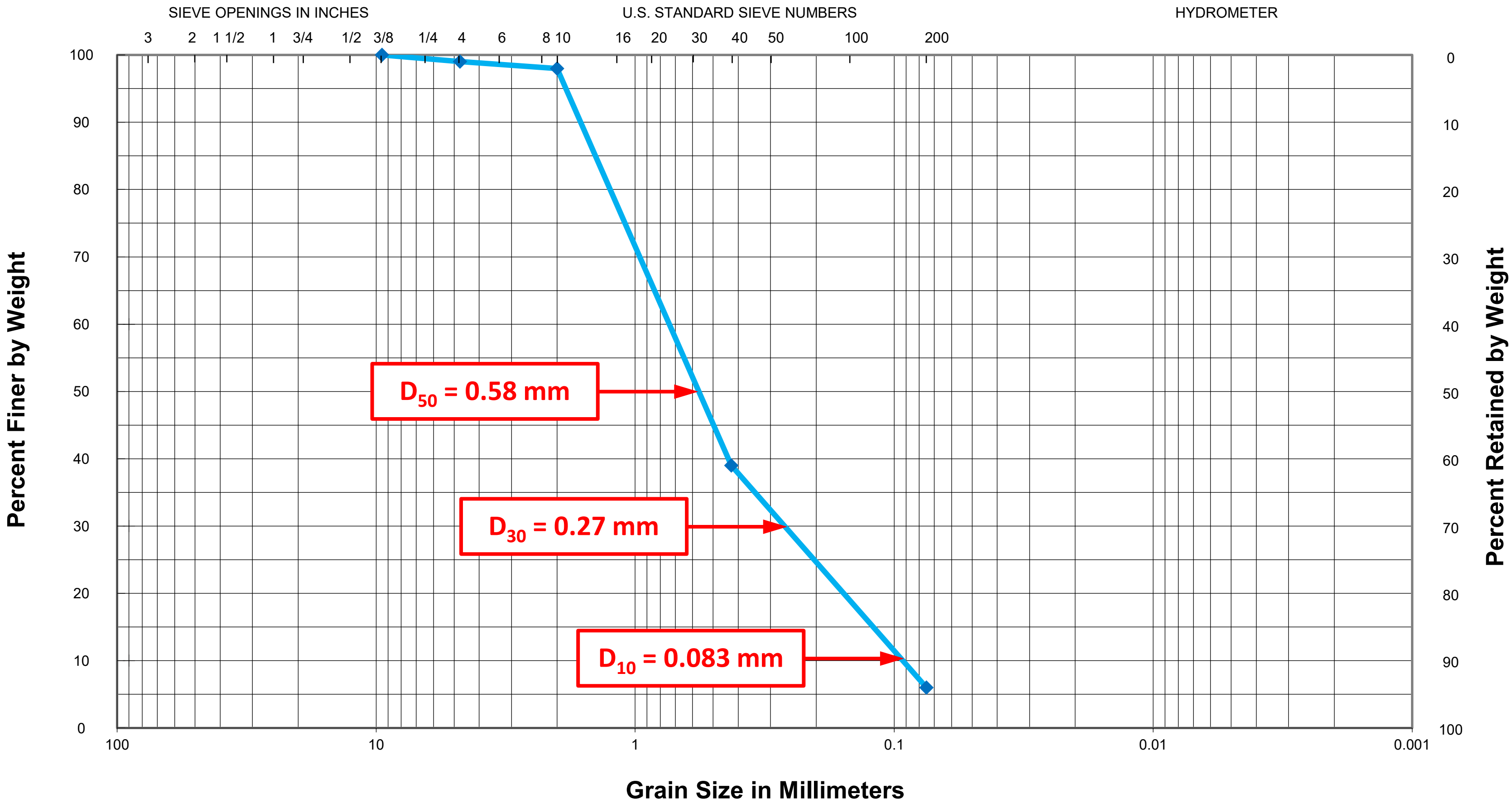
Sample: Boring B3, 14-15 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

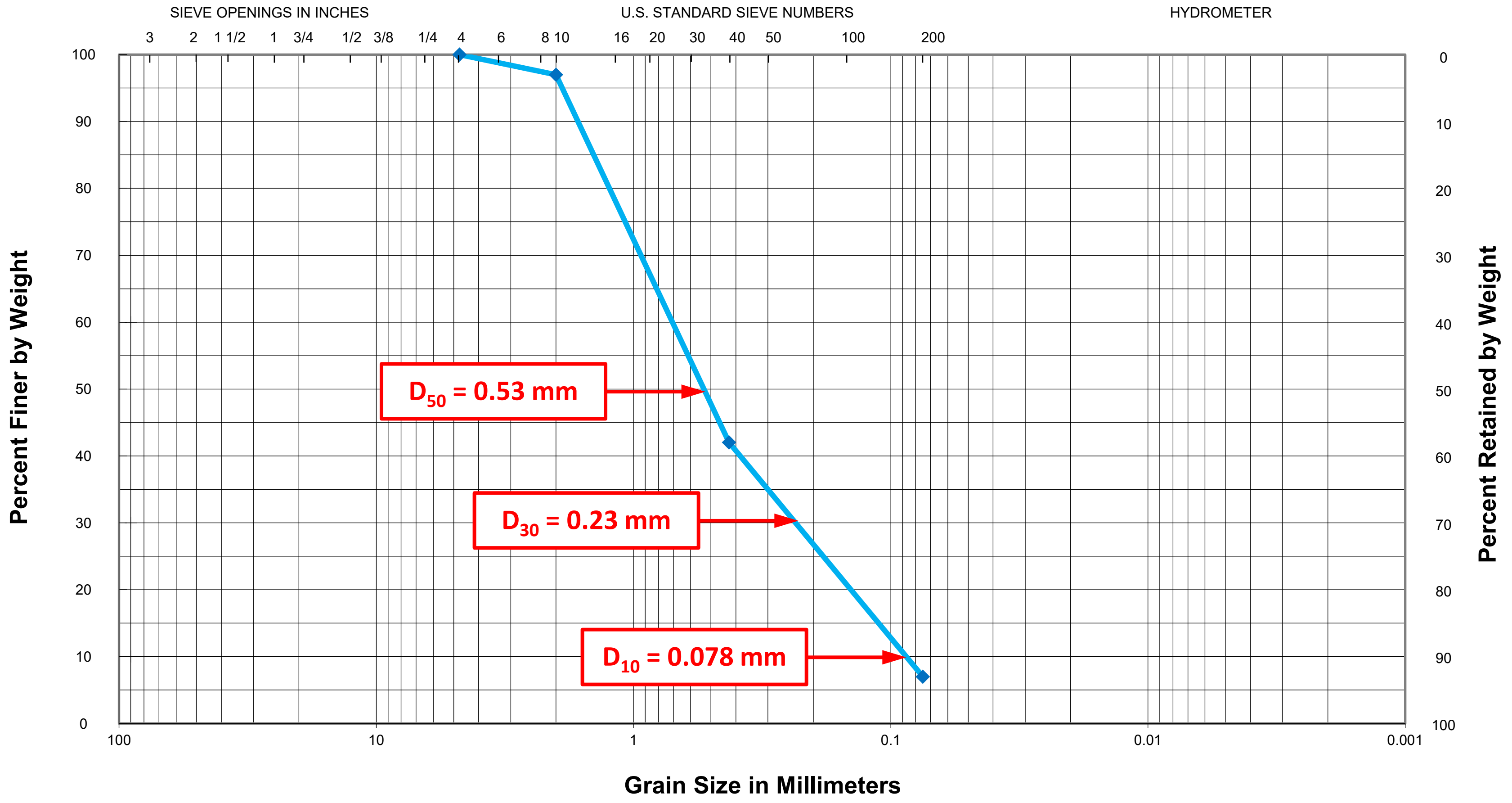
Sample: Boring B3, 54-55 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

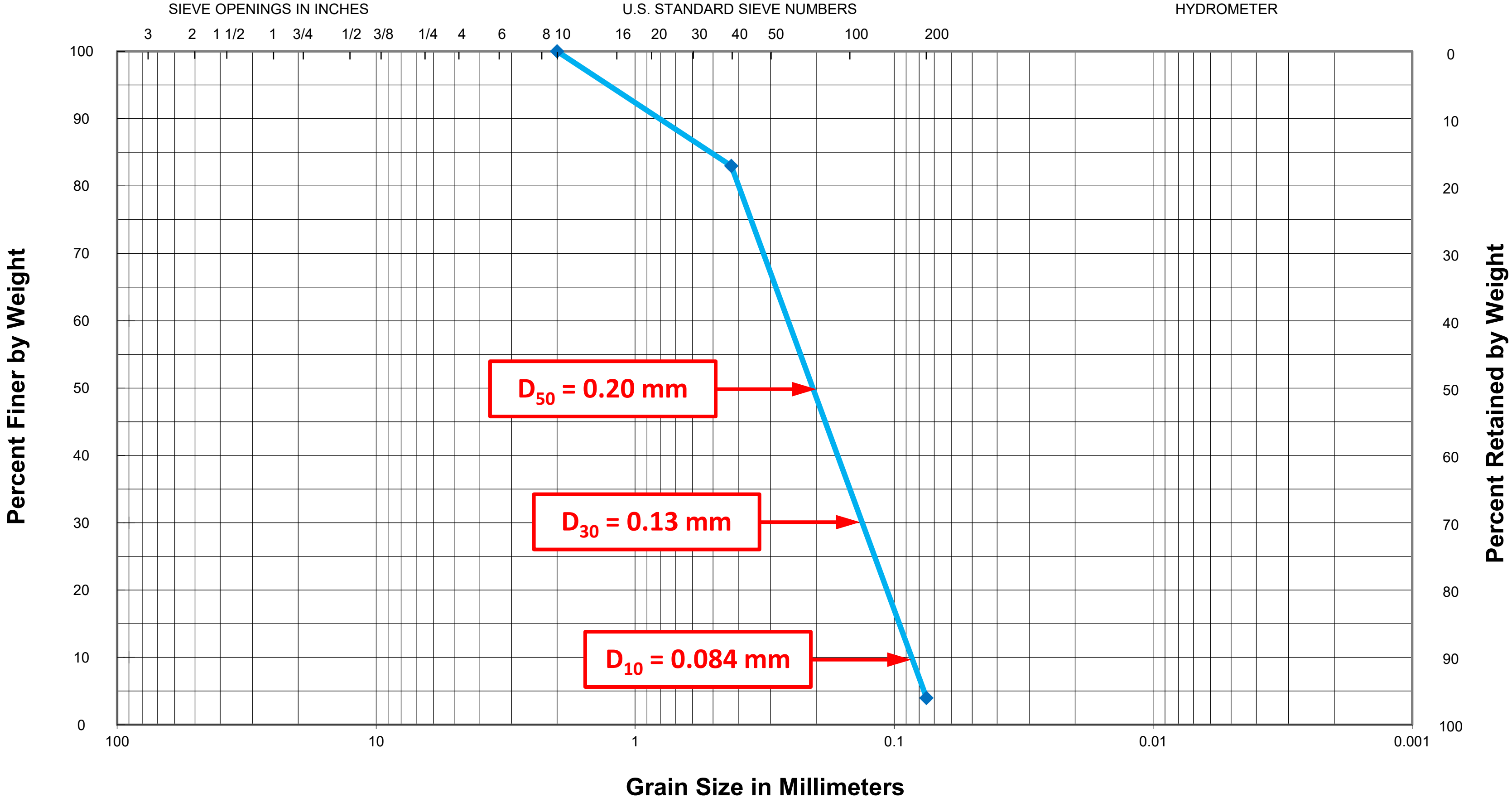
Sample: Boring B3, 69-70 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

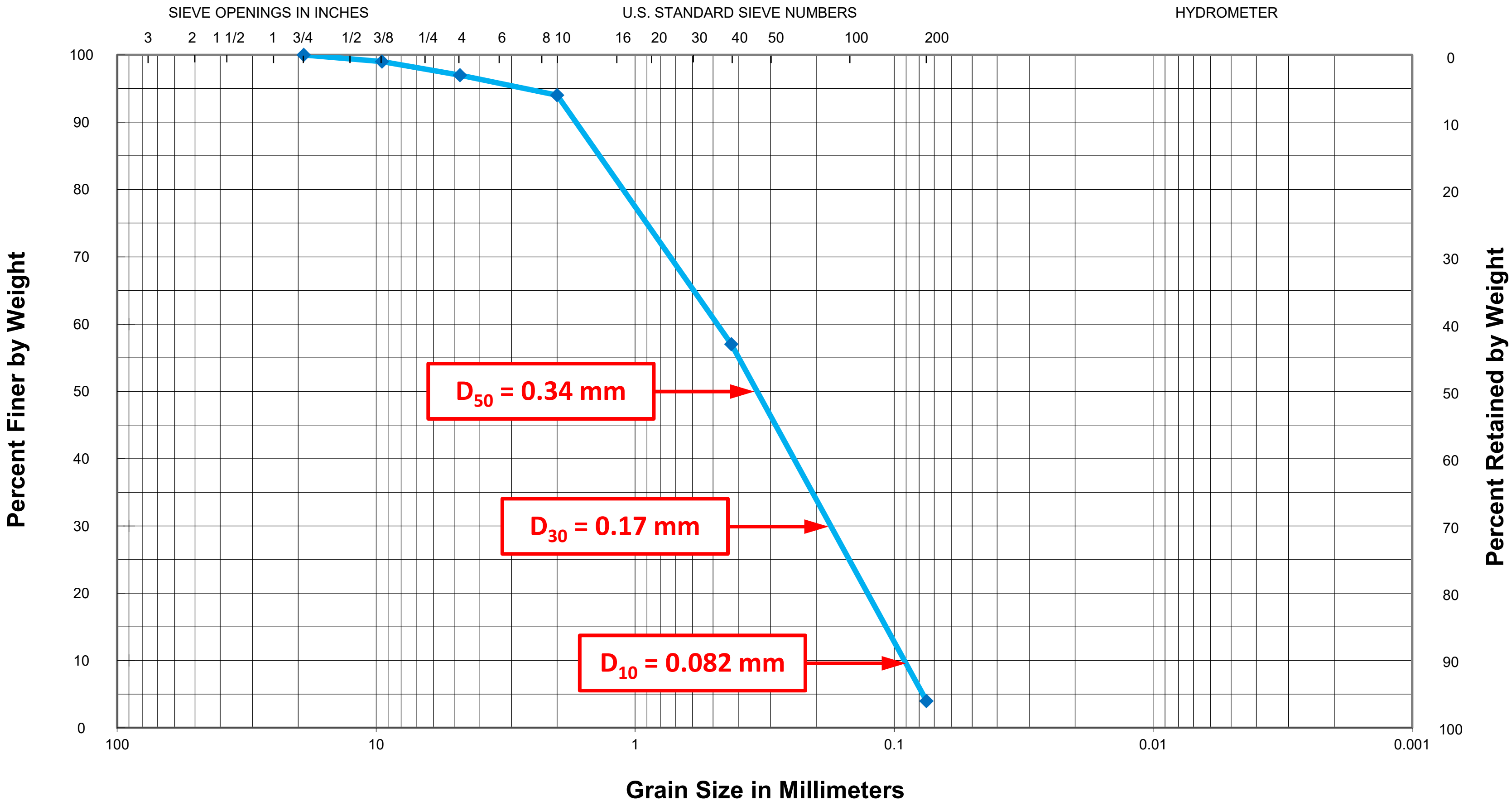
Sample: Boring B3, 69-70 ft
Description: Grayish brown fine SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.34 mm

D_{30} = 0.17 mm

D_{10} = 0.082 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

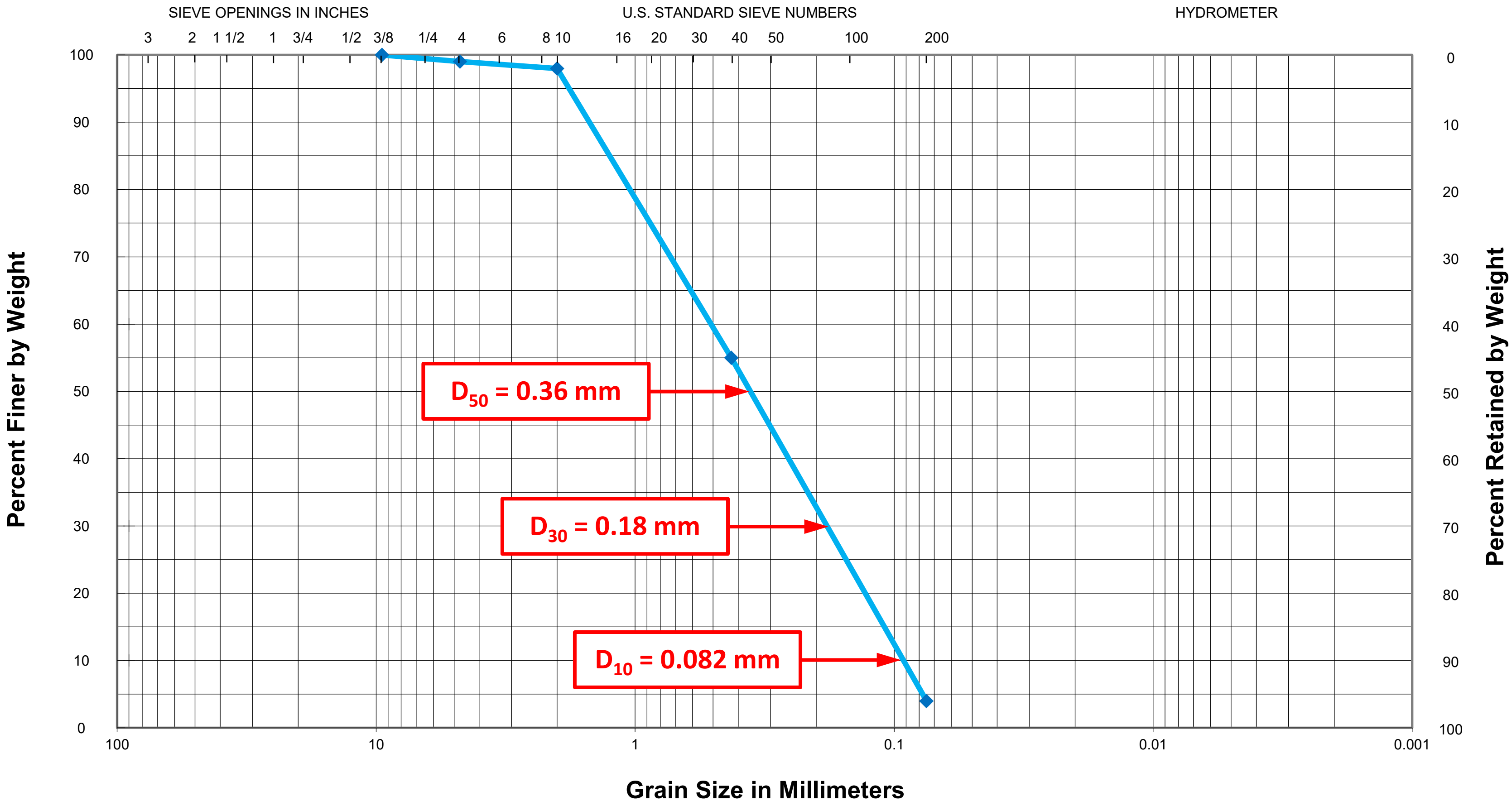
Sample: Boring B4, 19-20 ft
Description: Grayish tan fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.36 mm

D_{30} = 0.18 mm

D_{10} = 0.082 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

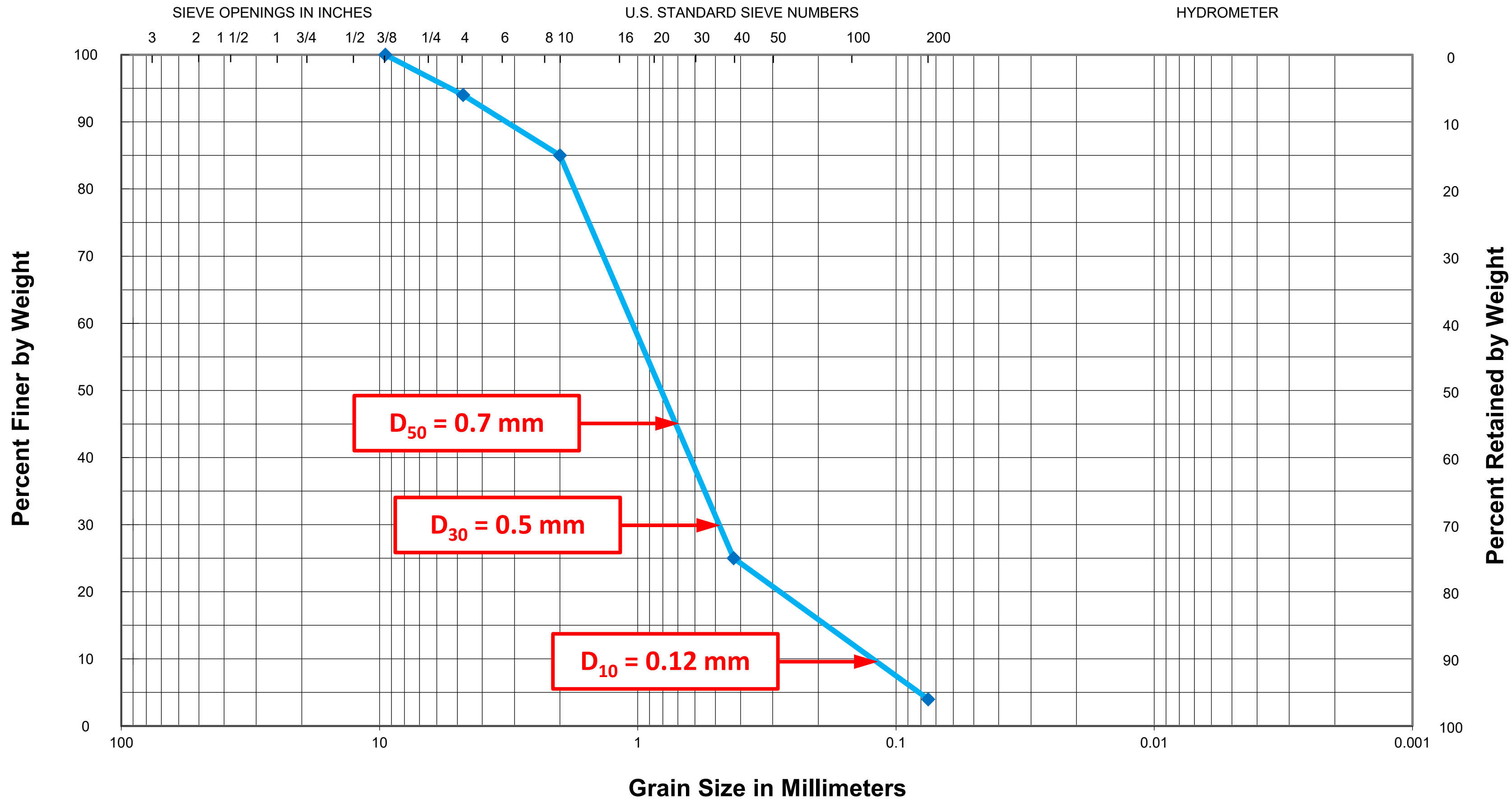
Sample: Boring B4, 39-40 ft
Description: Grayish tan fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

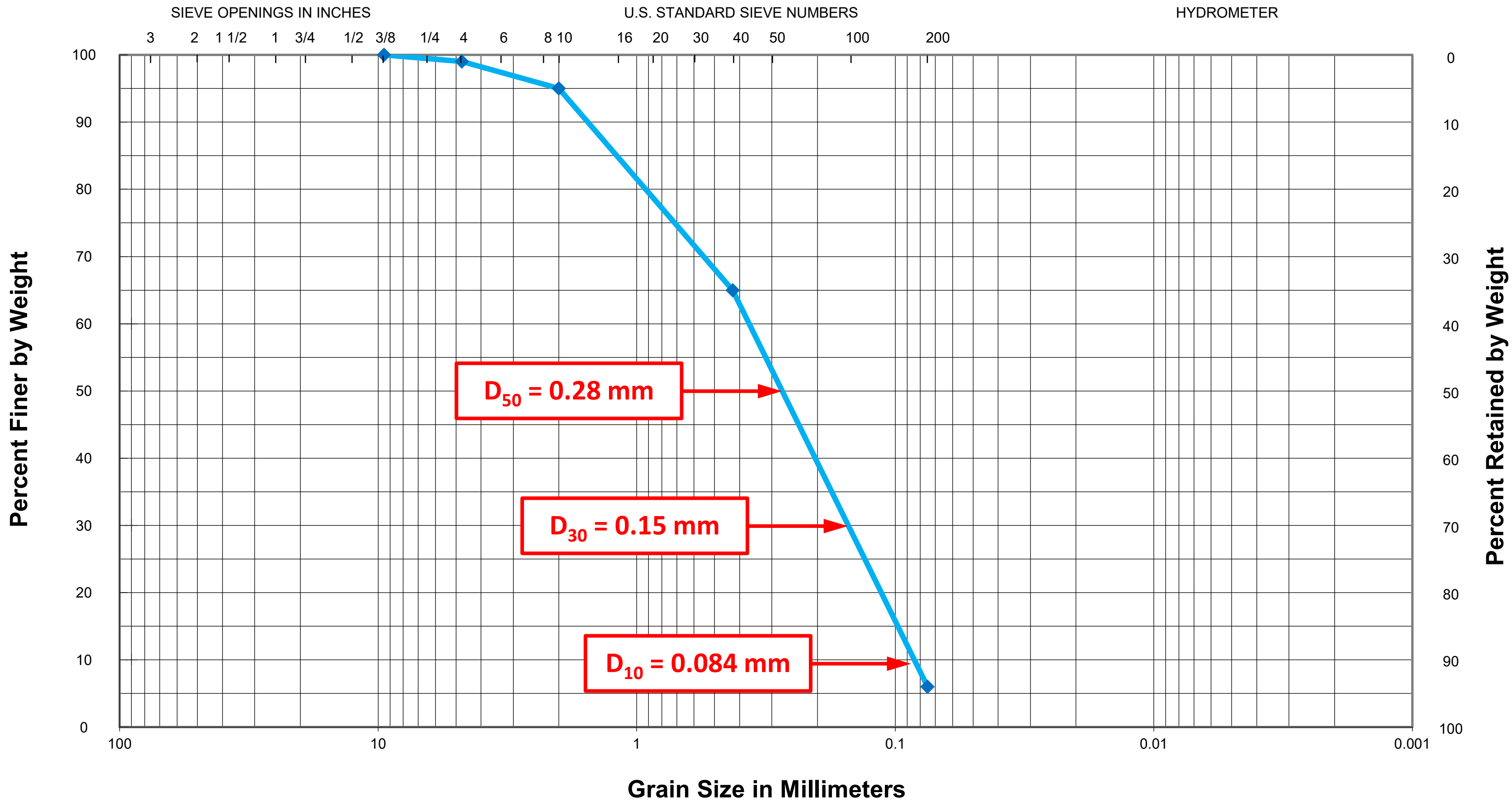
Sample: Boring B4, 64-65 ft
Description: Grayish tan fine to medium SAND w/trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

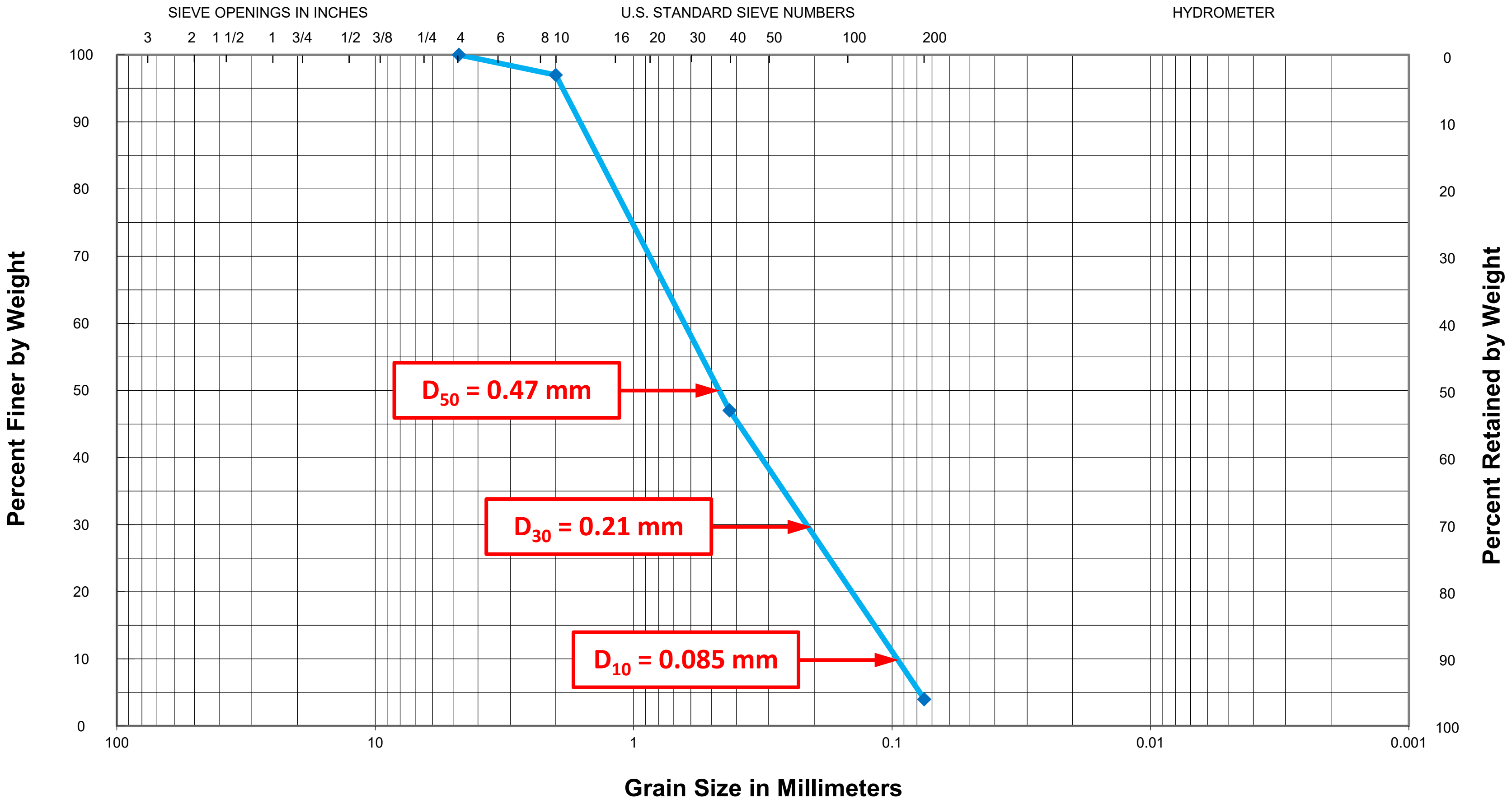
Sample: Boring B4, 89-90 ft
Description: Grayish tan fine to medium SAND, slightly silty w/trace coarse sand

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

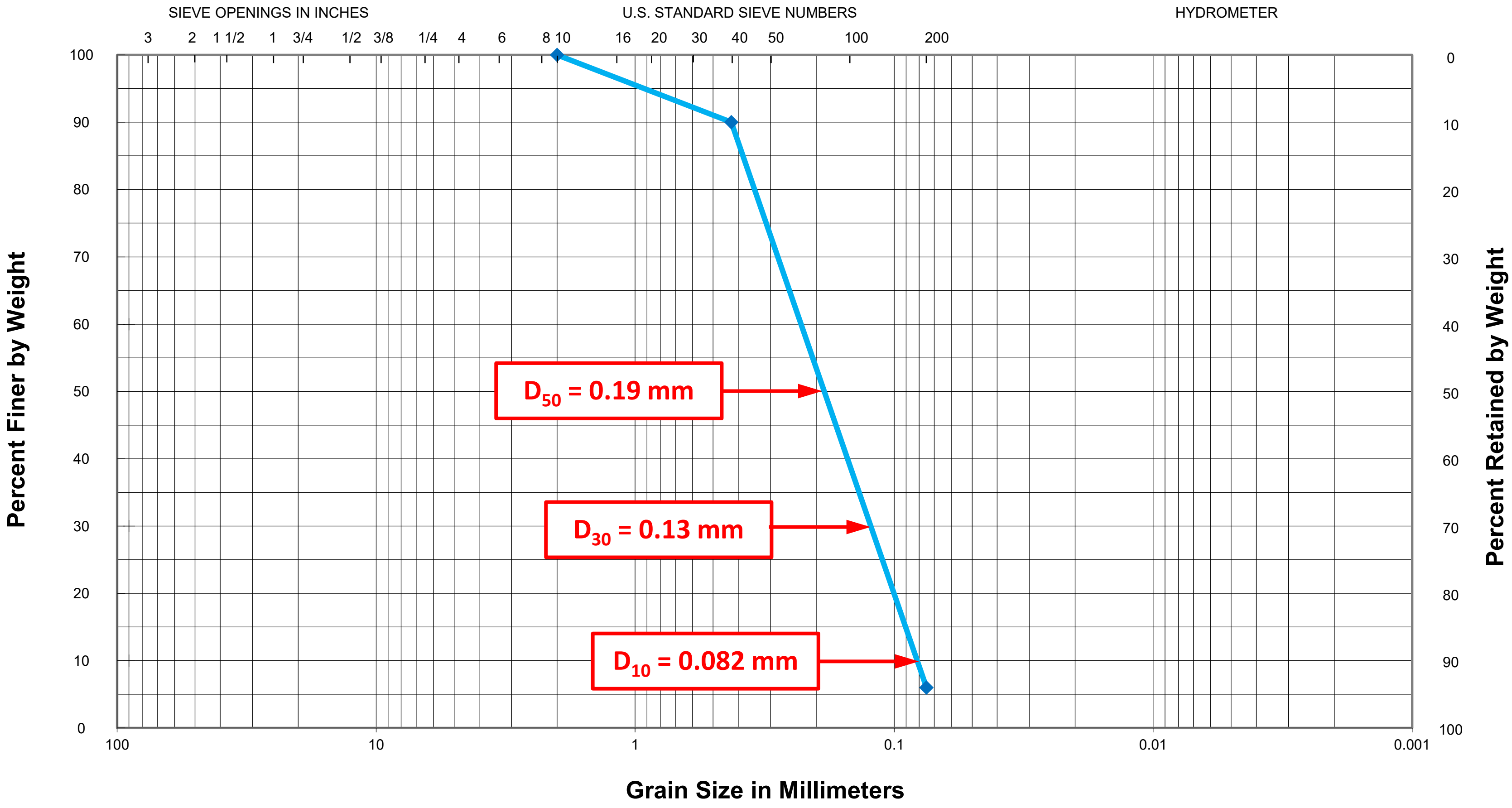
Sample: Boring B5, 44-45 ft
Description: Brownish gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

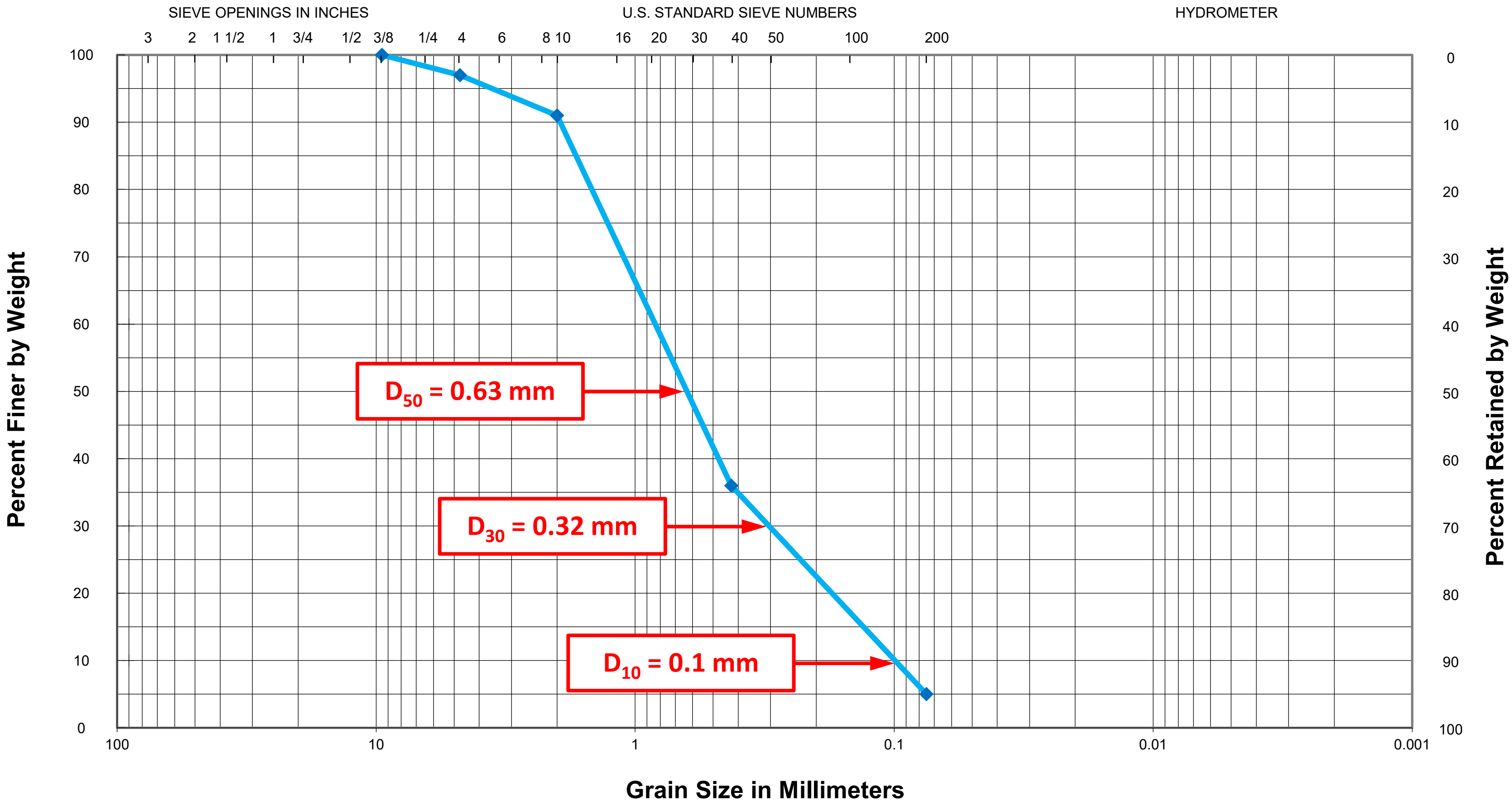
Sample: Boring B5, 64-65 ft
Description: Brownish gray fine SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.63 mm

D_{30} = 0.32 mm

D_{10} = 0.1 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring B5, 89-90 ft
Description: Brownish gray fine to medium SAND w/ trace coarse sand

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

APPENDIX D

Table 2. Summary of Site-Specific Response Results

Period	Site 2-Tyronza River	Site 5 – Righthand Chute Little River
A_s (g) (Site-adjusted PGA)	0.769	0.864
S_{DS} (g) (0.2 sec)	1.565	1.673
S_{D1} (g) (1 Sec)	1.197	1.247
Seismic Performance Zone	ZONE 4	ZONE 4

Table 4. Average Shear Wave Velocity and AASHTO Site Classification

CPT Designation	Average Shear Wave Velocity	AASHTO Site Class
SCPT-2	701	D
SCPT-5-South	709	D
SCPT-5-North	701	D
SCPT-7	712	D

Tyronza River Site:

Table 6. Site-Specific Response Accelerations Considering 5% Damping.

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	1.565
S_{D1}	1.197
S_{MS}	1.565
S_{M1}	1.200
MCE_G	0.769

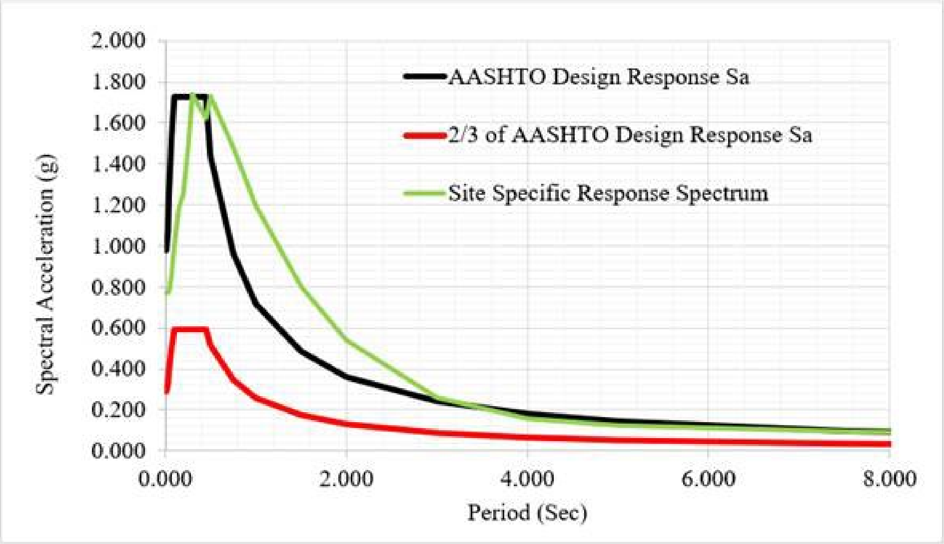
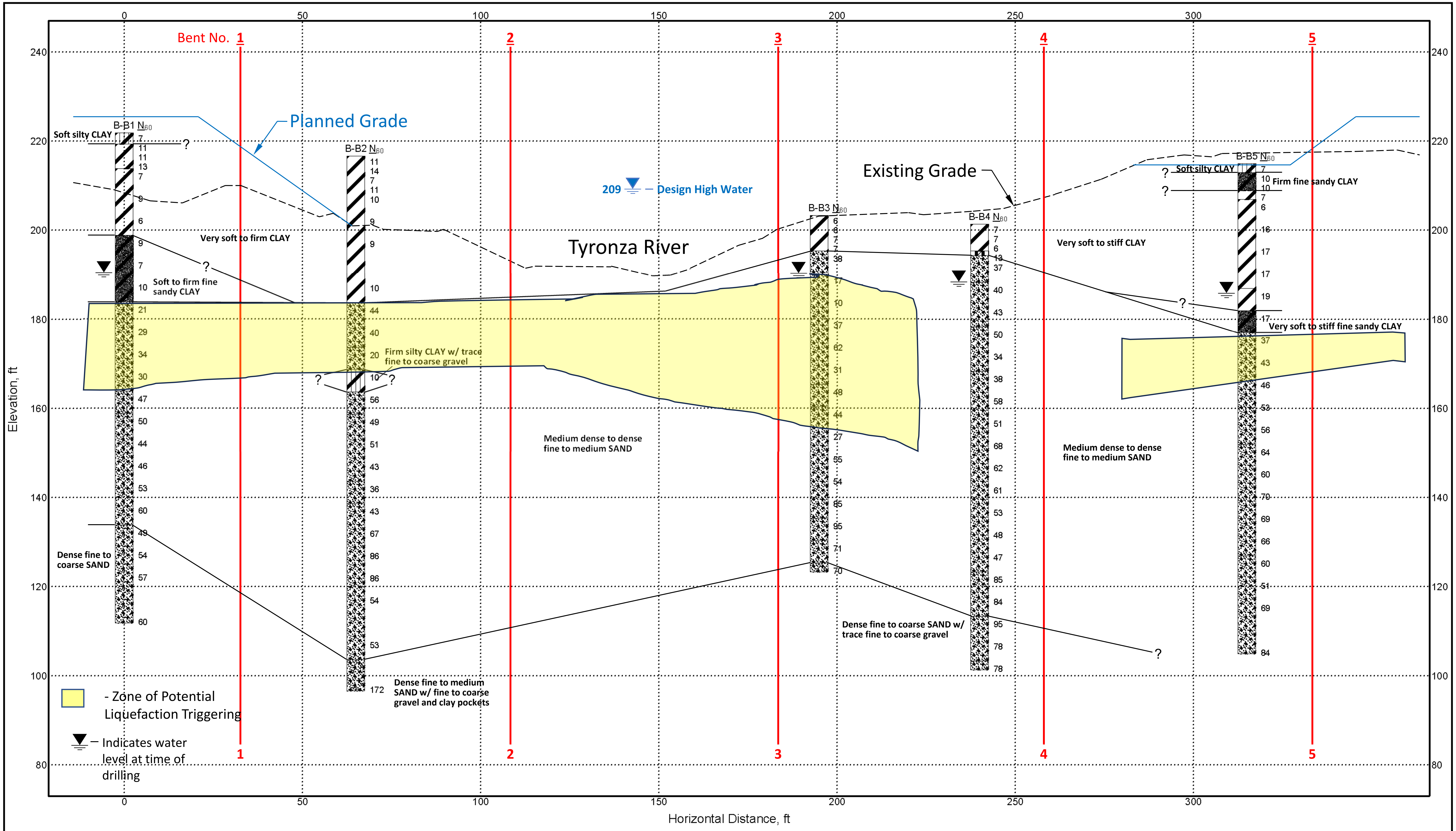


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

APPENDIX E



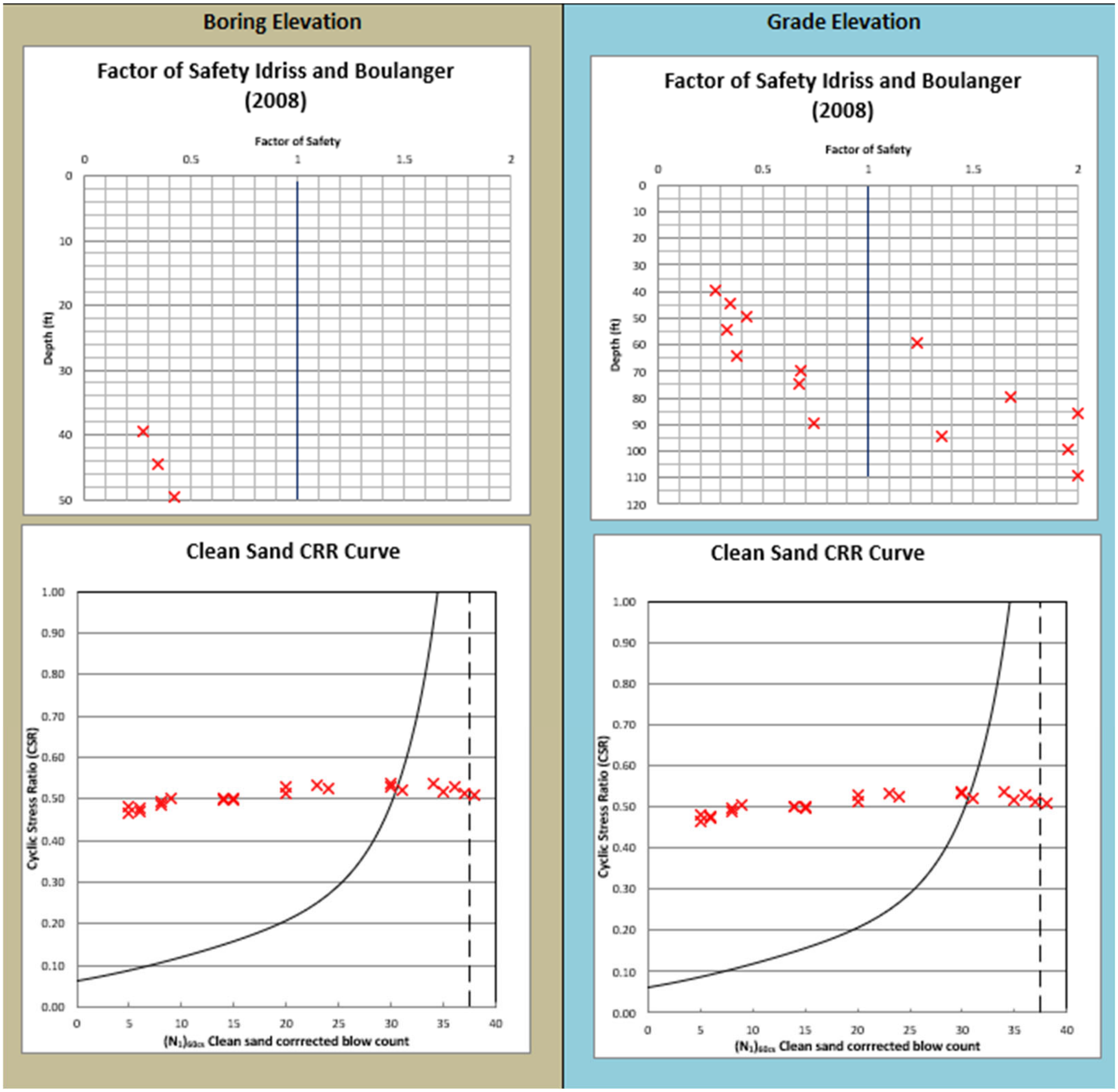
Grubbs, Hoskyn, Barton & Wyatt, LLC

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

SCALE:
1" = 25' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Project Number: 23-031



Boring B1

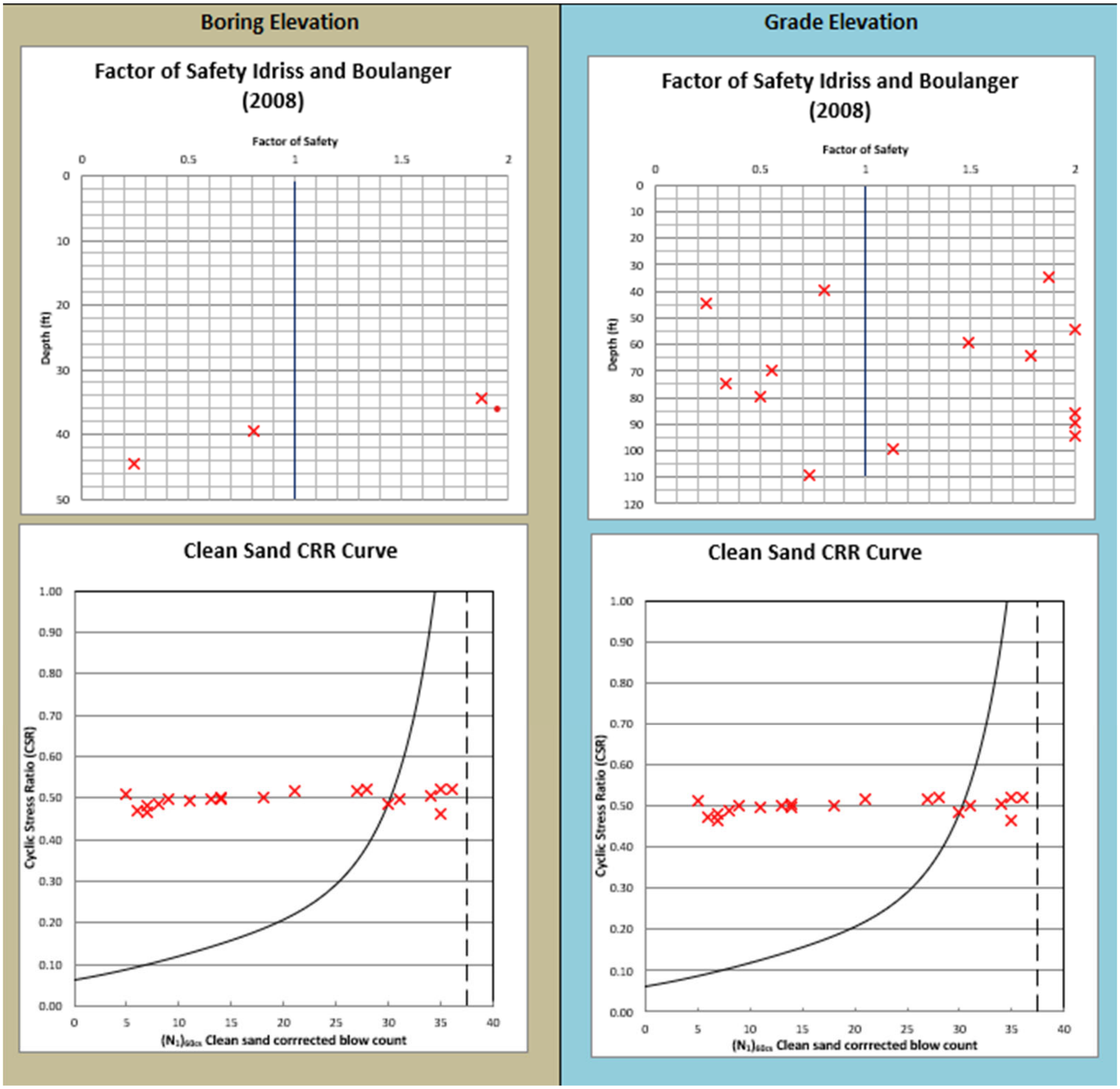
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

LIQUEFACTION ANALYSIS RESULTS

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Job No. 23-031

Plate



Boring B2

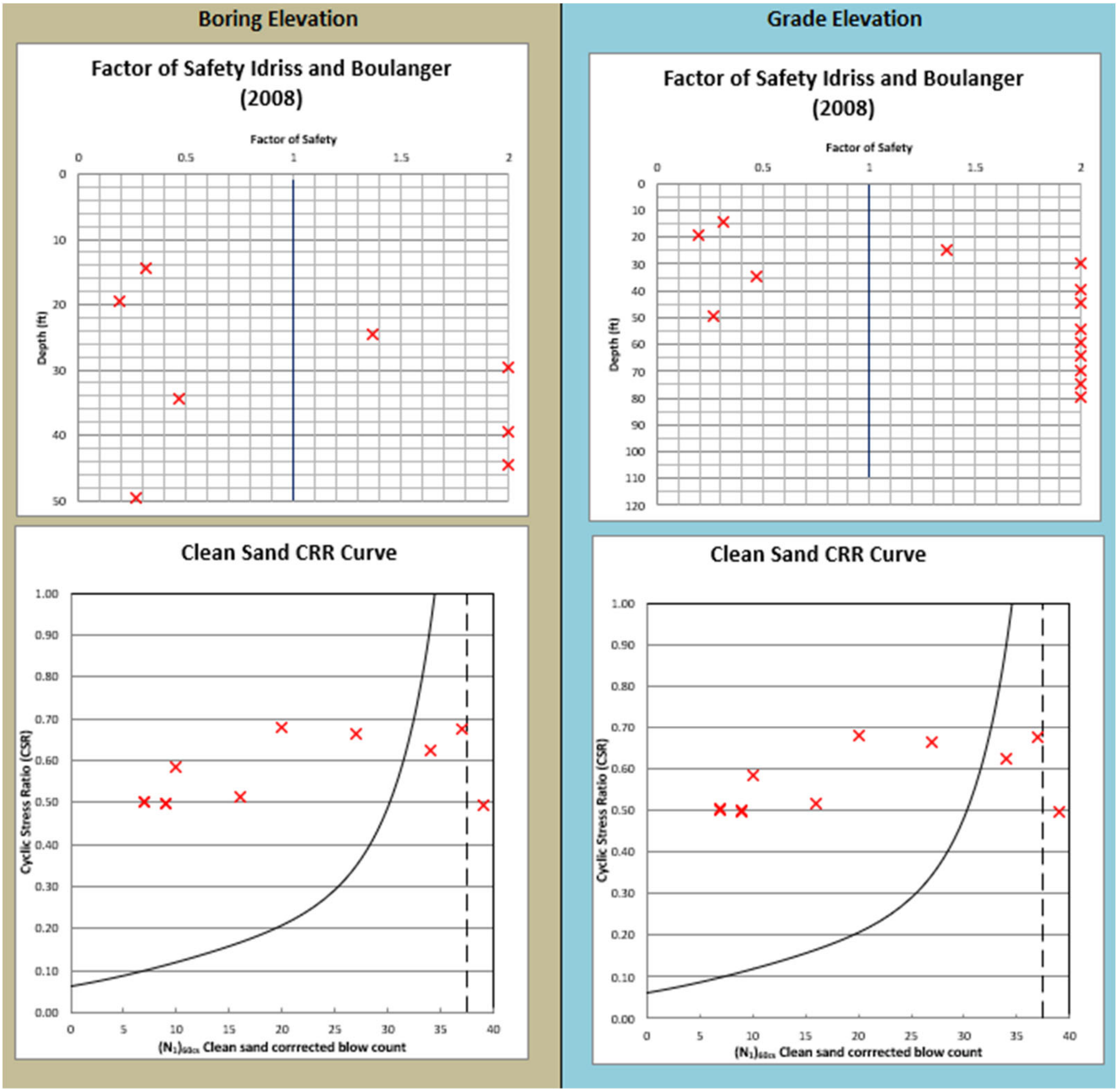
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

LIQUEFACTION ANALYSIS RESULTS

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Job No. 23-031

Plate



Boring B3

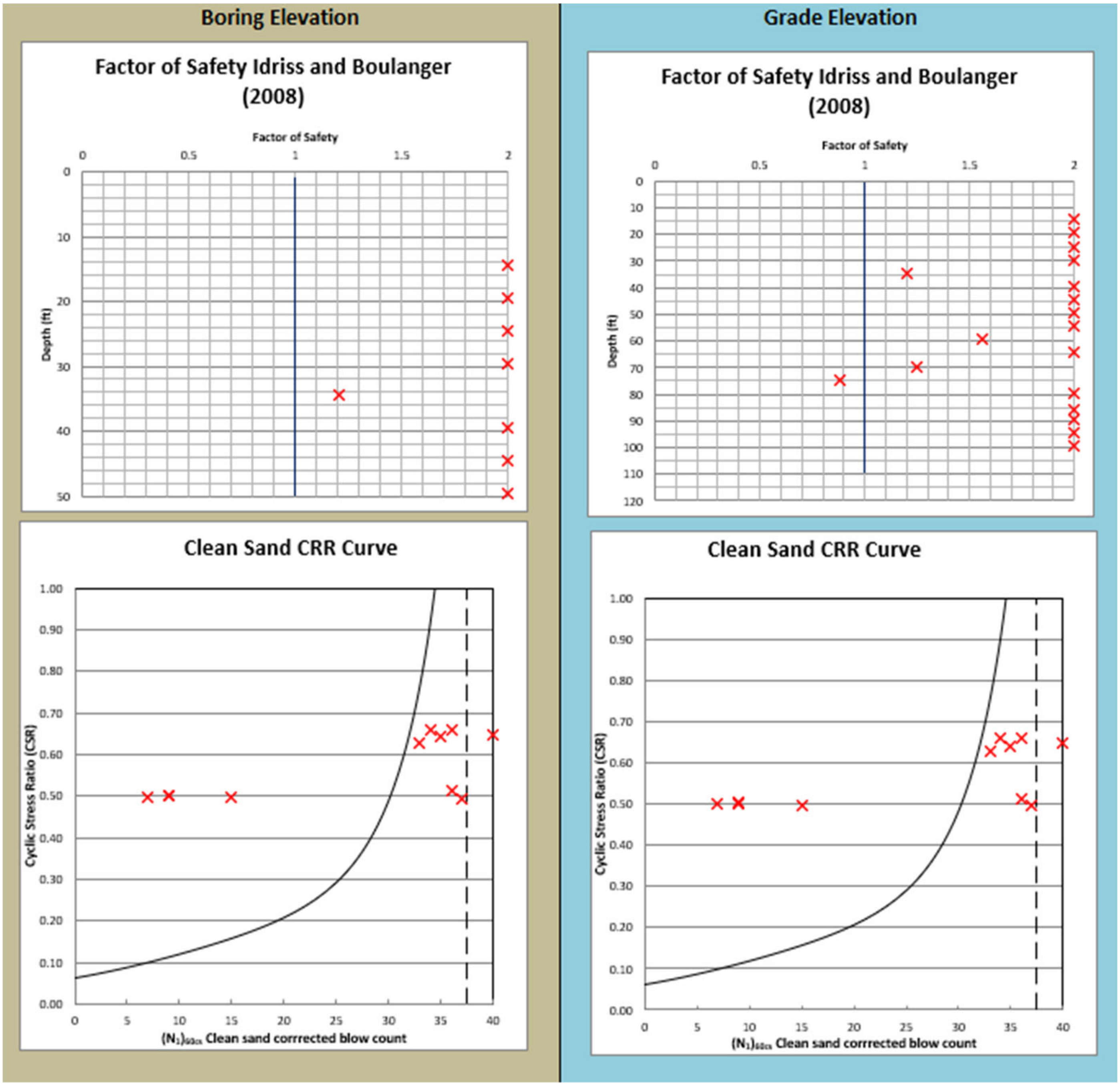
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

LIQUEFACTION ANALYSIS RESULTS

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Job No. 23-031

Plate



Boring B4

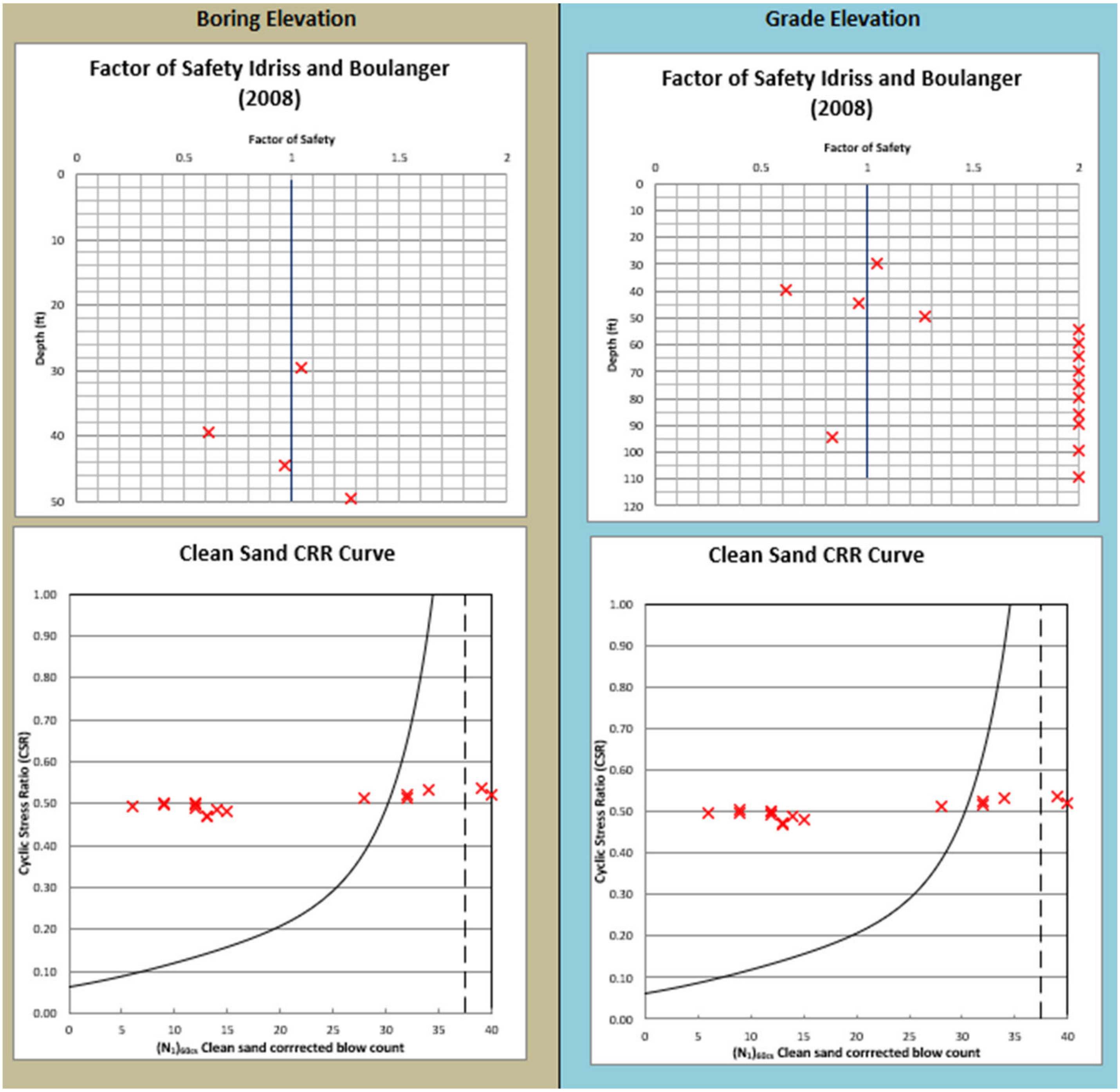
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

LIQUEFACTION ANALYSIS RESULTS

101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Job No. 23-031

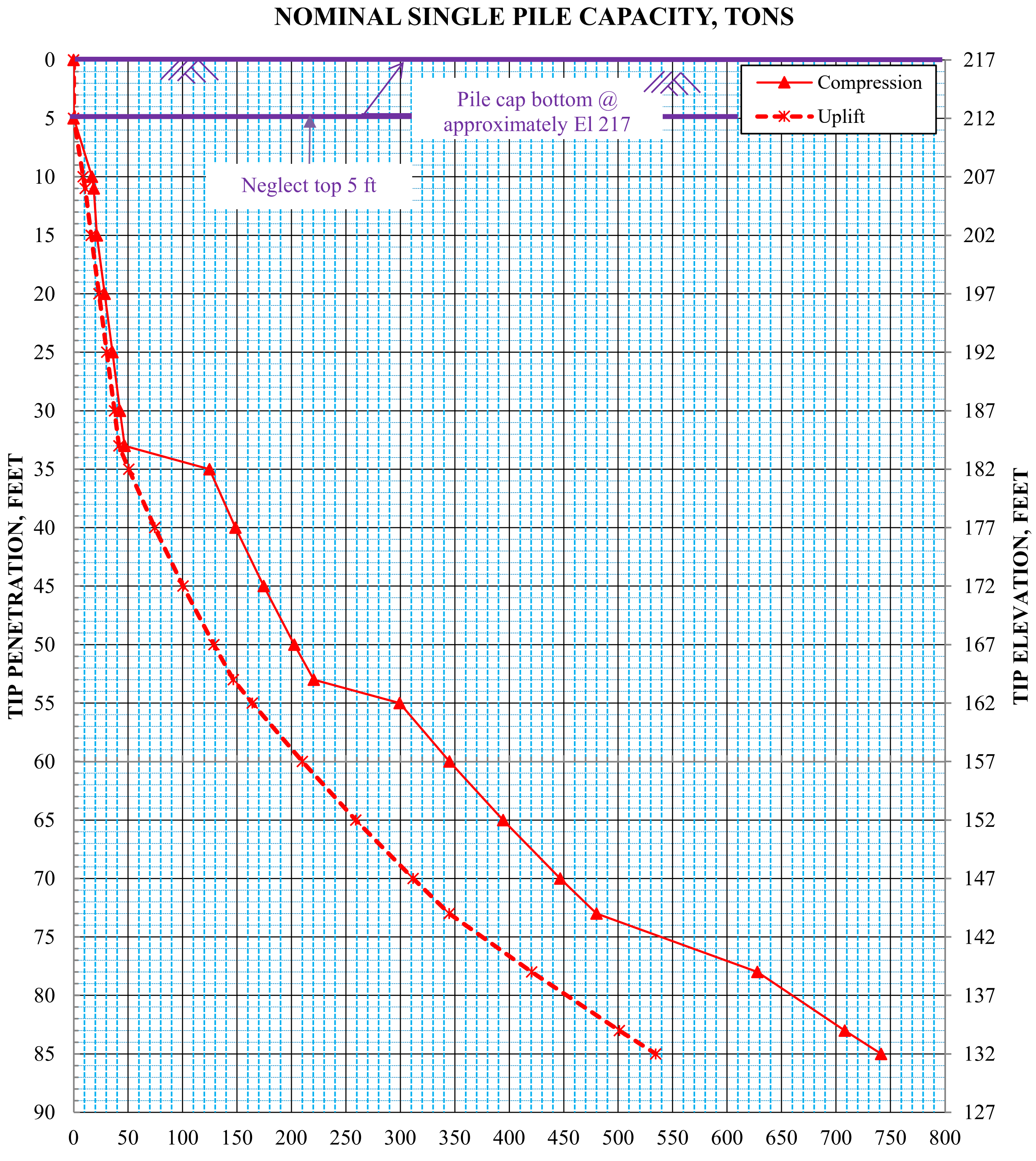
Plate



Boring B5

Grubbs, Hoskyn, Barton & Wyatt, LLC CONSULTING ENGINEERS A UES Company	**LIQUEFACTION ANALYSIS RESULTS** **101124 Hwy. 135 over Tyronza River** **Poinsett County, Arkansas**	**Job No. 23-031** **Plate**

APPENDIX F



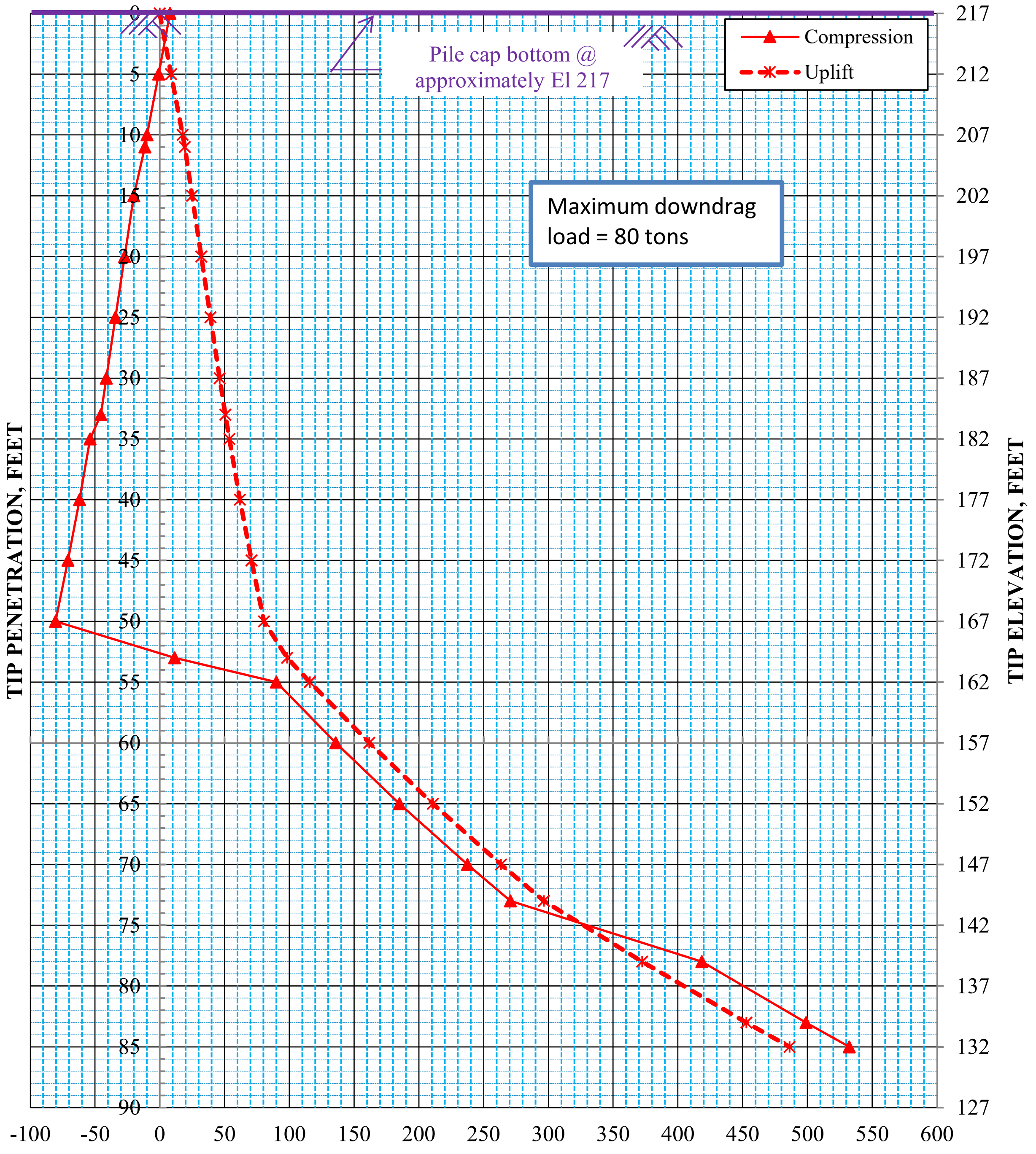
NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

NOMINAL SINGLE PILE CAPACITY, TONS

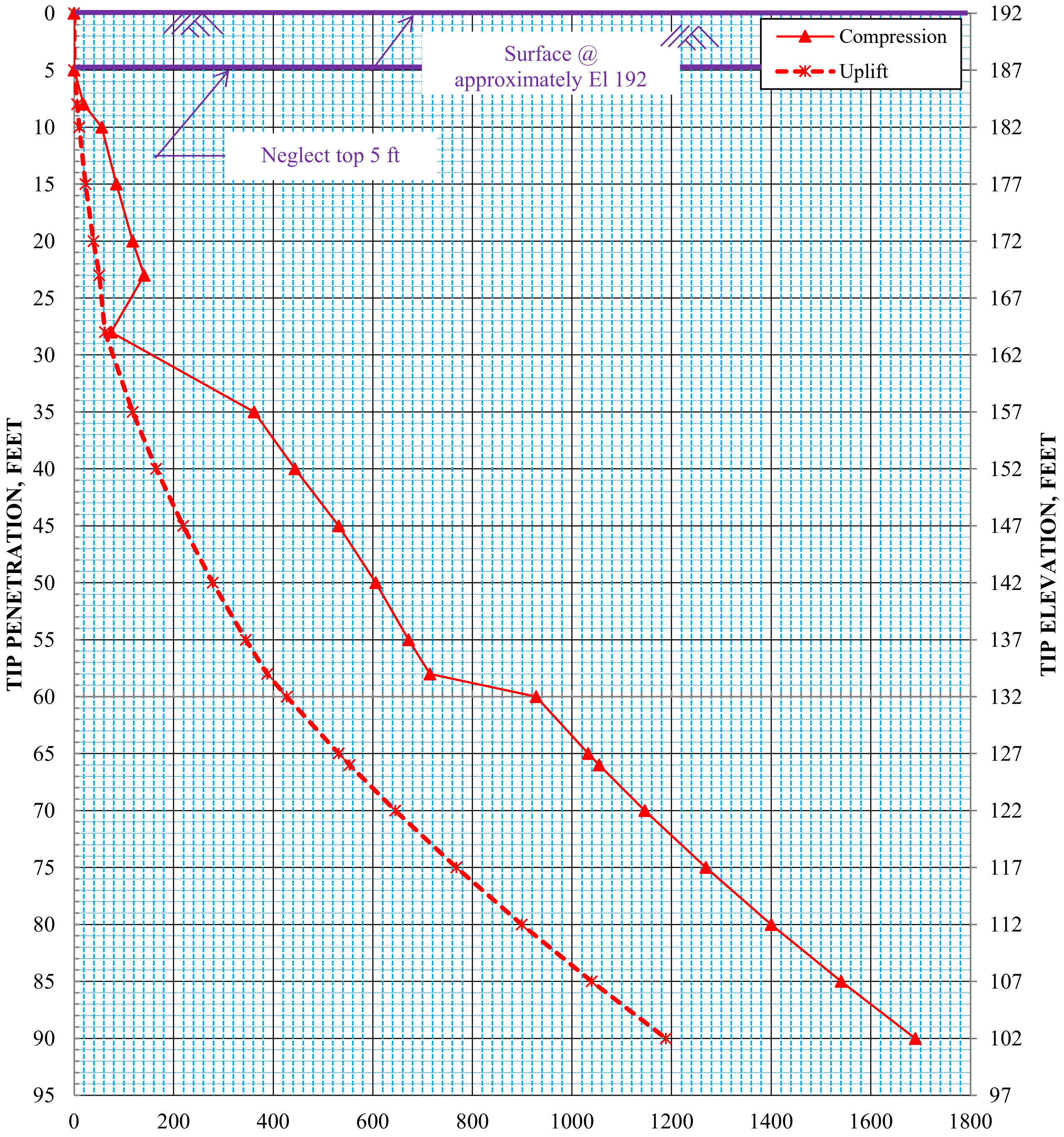


NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 167

NOMINAL SINGLE PILE CAPACITY, TONS



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 2 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom
2. No downdrag

Grubbs, Hoskyn, Barton & Wyatt, LLC
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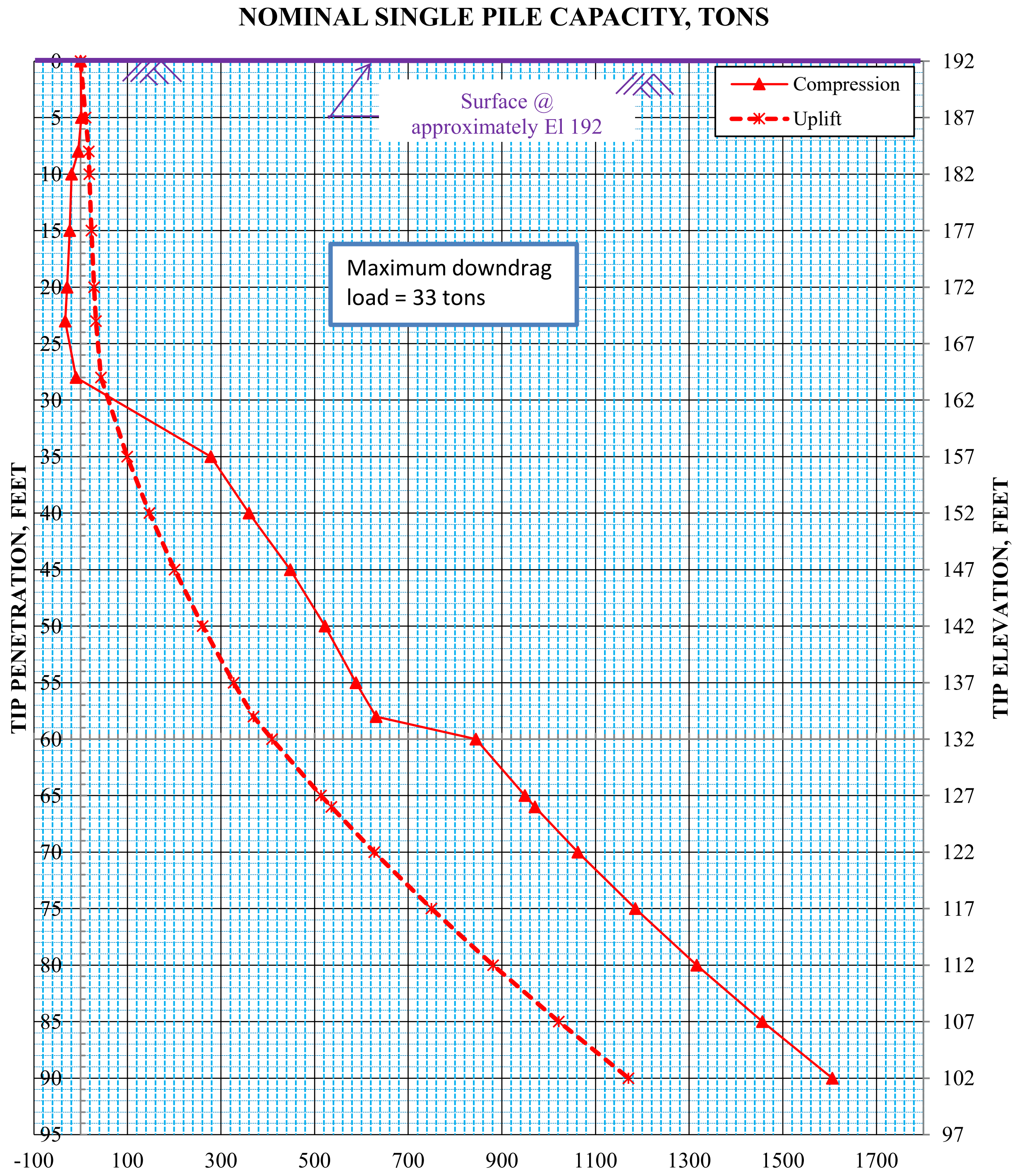

NOMINAL SINGLE PILE CAPACITY, TONS
Bent 2 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom
2. Downdrag to ±El 169

Grubbs, Hoskyn, Barton & Wyatt, LLC
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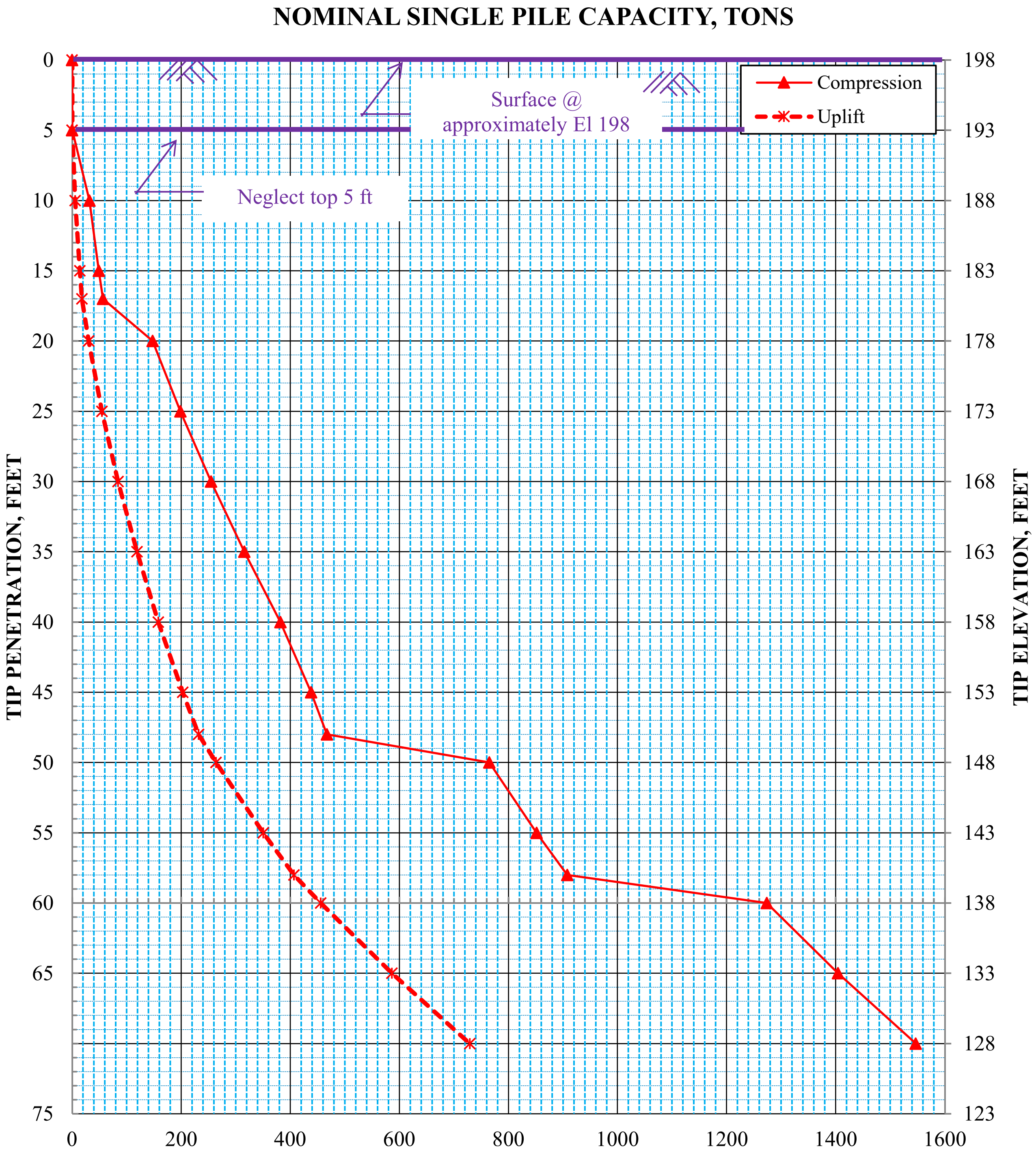
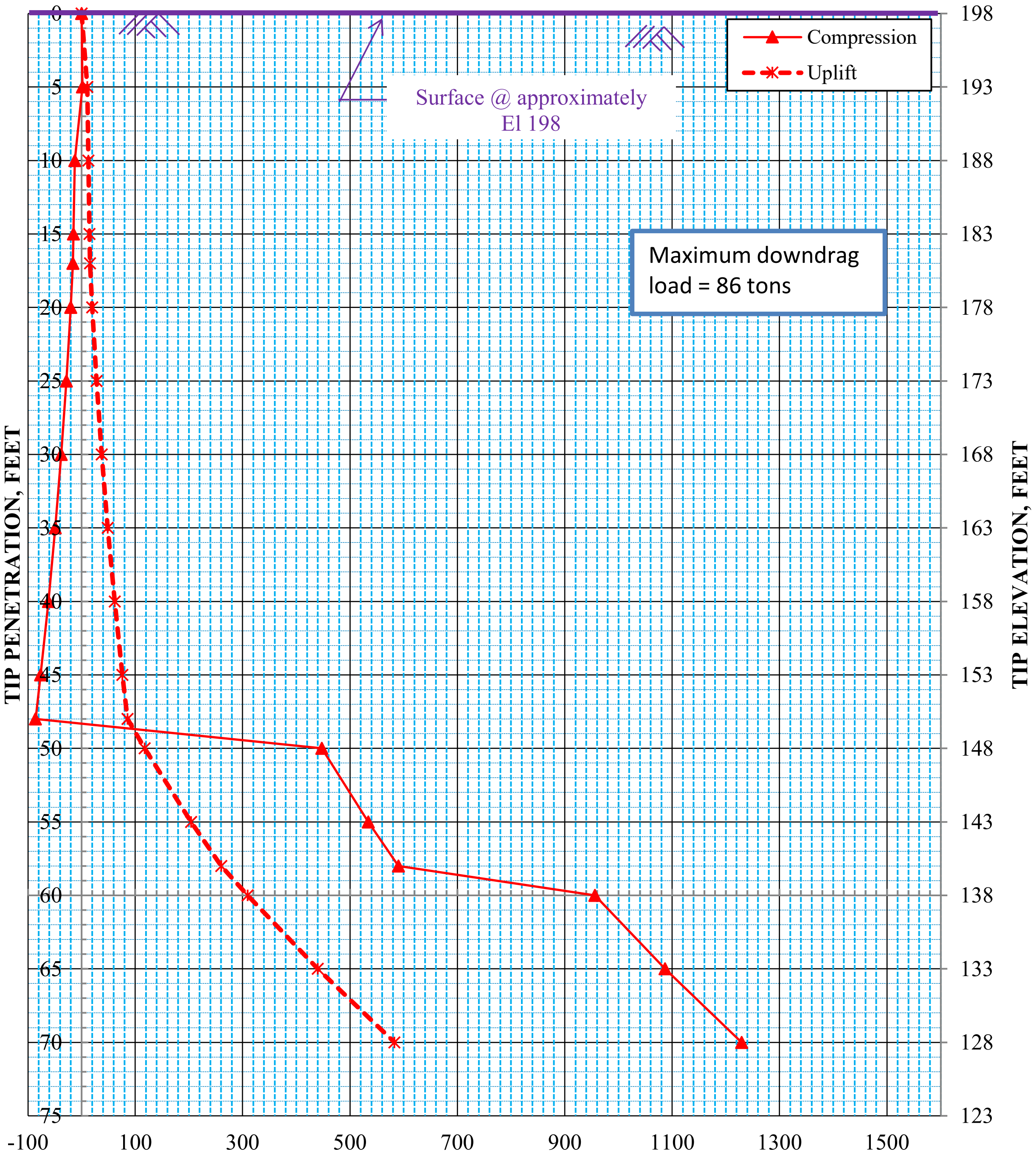


NOMINAL SINGLE PILE CAPACITY, TONS

Bent 3 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom
2. No downdrag

NOMINAL SINGLE PILE CAPACITY, TONS



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 3 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom
2. Downdrag to ±El 150

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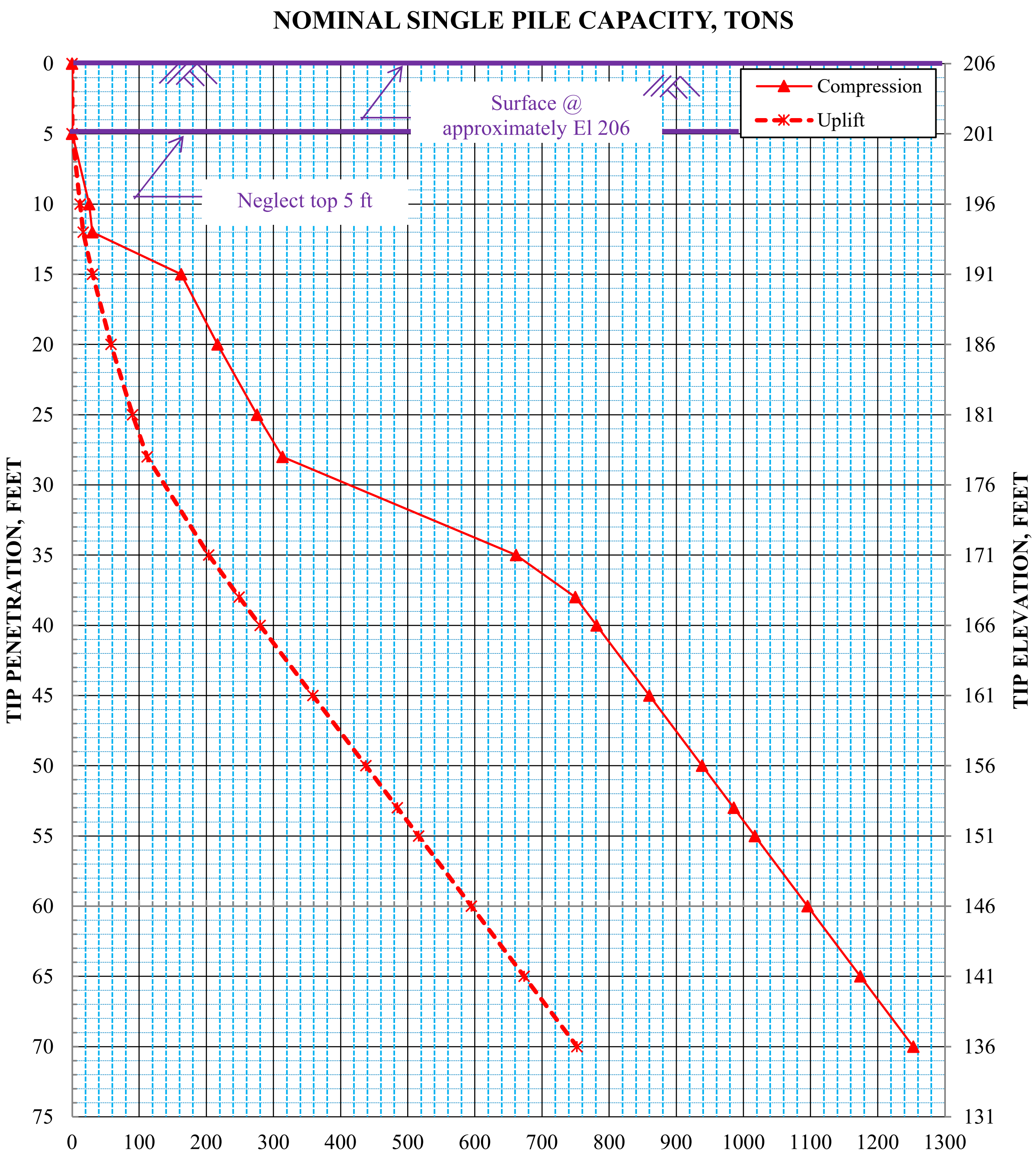

NOMINAL SINGLE PILE CAPACITY, TONS
Bent 4 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom
2. No downdrag

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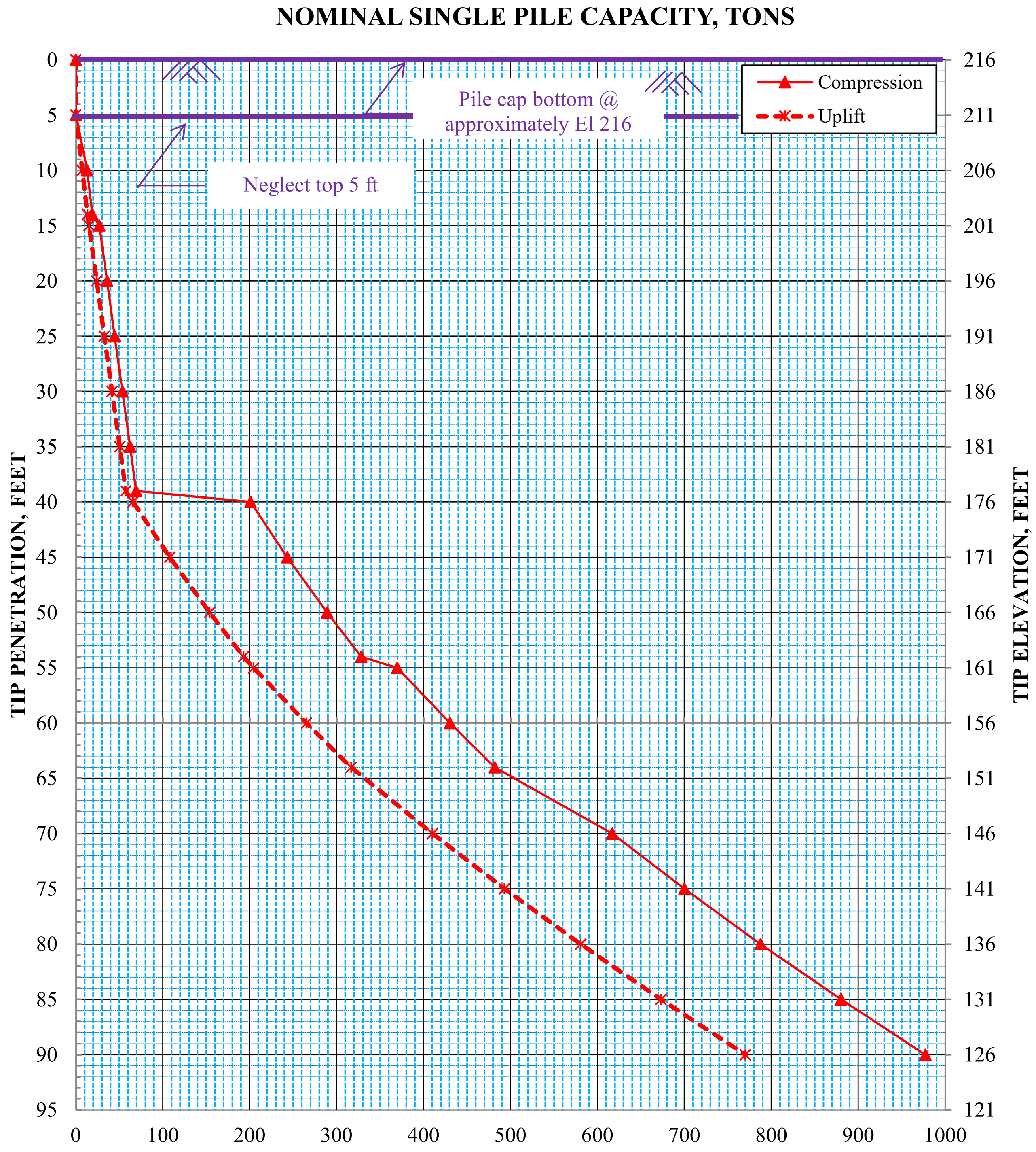
A UES Company

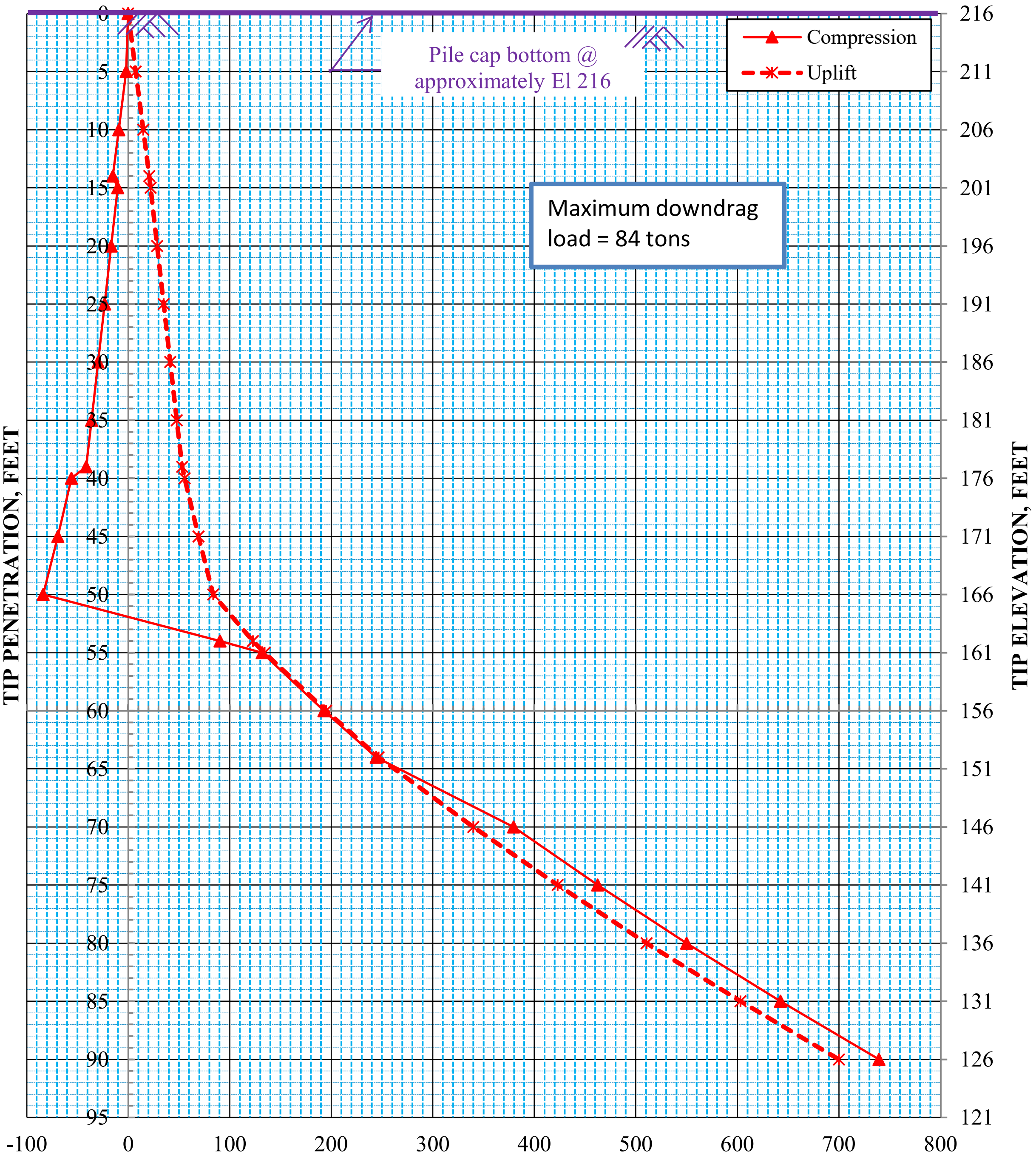


NOMINAL SINGLE PILE CAPACITY, TONS

Bent 5 (North Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag

NOMINAL SINGLE PILE CAPACITY, TONS



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 5 (North Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Tyronza River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 166

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A UES Company

APPENDIX G

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Tyronza River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft to firm CLAY	Soft CLAY	Medium dense fine to medium SAND	Medium dense to dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-11	11-33	33-53	53-73	73 and deeper
Approximate El, ft	217-206	206-184	184-164	164-144	below 144
Recommend soil type	Stiff clay without free water	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	120	115	57	63	68
Cohesion (c), lbs per sq ft	1000	650	0	0	0
Angle of internal friction (φ), °	0	0	33	36	38
Subgrade modulus (k), lbs per cu in.	300	100	55	105	125
Strain at 50% (EE50)	0.009	0.01	NA	NA	NA

Note: Pile cap bottom at ±El 217

Seismic Loading with Liquefaction
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft to firm CLAY	Soft CLAY	Medium dense fine to medium SAND (liquefiable)	Medium dense to dense fine to medium SAND	Medium dense to dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-11	11-33	33-50	50-53	53-73	73 and deeper
Approximate El, ft	217-206	206-184	184-167	167-164	164-144	below 144
Recommend soil type	Stiff clay without free water	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	120	115	57	63	63	68
Cohesion (c), lbs per sq ft	1000	650	0	0	0	0
Angle of internal friction (φ), °	0	0	11	36	36	38
Subgrade modulus (k), lbs per cu in.	300	100	20	105	105	125
Strain at 50% (EE50)	0.009	0.01	NA	NA	NA	NA

Note: Pile cap bottom at ±El 217

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Tyronza River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft CLAY**	**Medium dense silty fine SAND**	**Soft CLAY**	**Medium dense to dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-8	8-23	23-28	28-58	58 and deeper
Approximate El, ft	192-184	184-169	169-164	164-134	below 134
Recommend soil type	Soft clay	Sand (Reese)	Soft clay	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	57	53	63	68
Cohesion (c), lbs per sq ft	650	0	650	0	0
Angle of internal friction (φ), °	0	33	0	36	38
Subgrade modulus (k), lbs per cu in.	100	55	100	105	125
Strain at 50% (EE50)	0.01	NA	0.01	NA	NA

Note: Ground surface at ±El 192

Seismic Loading with Liquefaction
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft CLAY**	**Medium dense silty fine SAND (liquefiable)**	**Soft CLAY**	**Medium dense to dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-8	8-23	23-28	28-58	58 and deeper
Approximate El, ft	192-184	184-169	169-164	164-134	below 134
Recommend soil type	Soft clay	Sand (Reese)	Soft clay	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	53	57	53	63	68
Cohesion (c), lbs per sq ft	650	0	650	0	0
Angle of internal friction (φ), °	0	11	0	36	38
Subgrade modulus (k), lbs per cu in.	100	20	100	105	125
Strain at 50% (EE50)	0.01	NA	0.01	NA	NA

Note: Ground surface at ±El 192

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Tyronza River
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft CLAY**	**Loose to medium dense fine SAND**	**Medium dense fine to medium SAND**	**Dense fine to medium SAND**	**Very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-5	5-17	17-48	48-58	58 and deeper
Approximate El, ft	198-193	193-181	181-150	150-140	below 140
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	53	60	65	73
Cohesion (c), lbs per sq ft	650	0	0	68	0
Angle of internal friction (φ), °	0	30	35	38	40
Subgrade modulus (k), lbs per cu in.	100	35	80	125	130
Strain at 50% (EE50)	0.01	NA	NA	NA	NA

Note: Ground surface at ±El 198

Seismic Loading with Liquefaction
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft CLAY**	**Loose to medium dense fine SAND (liquefiable)**	**Medium dense fine to medium SAND (liquefiable)**	**Dense fine to medium SAND**	**Very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-5	5-17	17-48	48-58	58 and deeper
Approximate El, ft	198-193	193-181	181-150	150-140	below 140
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	53	60	65	73
Cohesion (c), lbs per sq ft	650	0	0	68	0
Angle of internal friction (φ), °	0	8	11	38	40
Subgrade modulus (k), lbs per cu in.	100	20	20	125	130
Strain at 50% (EE50)	0.01	NA	NA	NA	NA

Note: Ground surface at ±El 198

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Tyronza River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft CLAY**	**Medium dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-12	12-28	28 and deeper
Approximate El, ft	206-194	194-178	below 178
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	60	68
Cohesion (c), lbs per sq ft	700	0	0
Angle of internal friction (φ), °	0	35	38
Subgrade modulus (k), lbs per cu in.	100	80	125
Strain at 50% (EE50)	0.01	NA	NA

Note: Ground surface at ±El 206

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Tyronza River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 5: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft CLAY	Stiff CLAY	Medium dense to dense fine to medium SAND	Dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-14	14-39	39-54	54-64	64 and deeper
Approximate El, ft	216-202	202-177	177-162	162-152	below 152
Recommend soil type	Soft clay	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	125	63	65	68
Cohesion (c), lbs per sq ft	700	1500	0	0	0
Angle of internal friction (φ), °	0	0	36	37	38
Subgrade modulus (k), lbs per cu in.	100	500	105	115	125
Strain at 50% (EE50)	0.01	0.007	NA	NA	NA

Note: Pile cap bottom at ±El 216

Seismic Loading with Liquefaction
Bent 5: Recommended Parameters for Lateral Load Analyses Using LPILE©

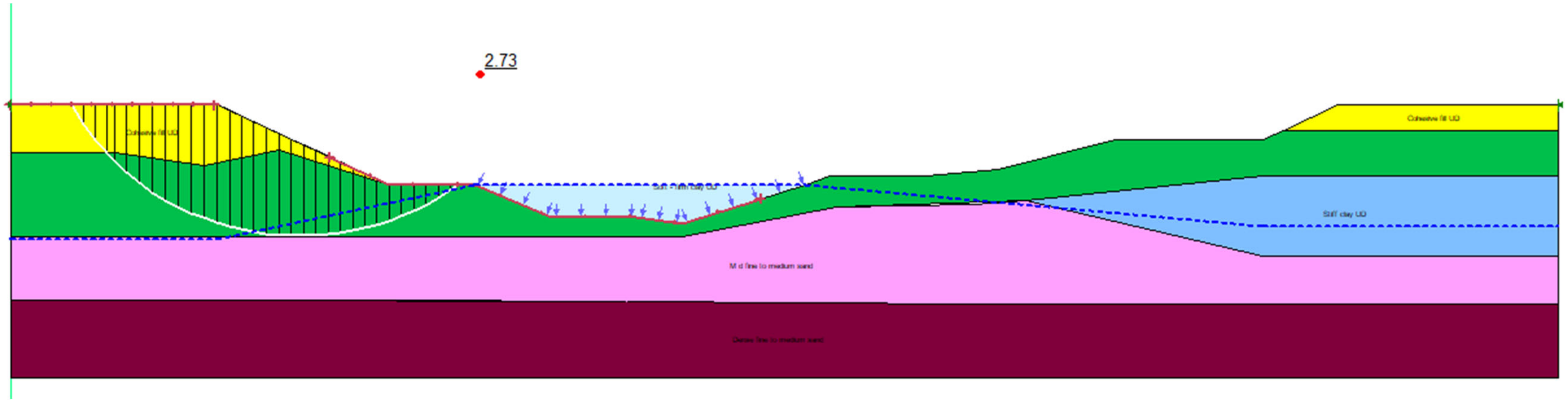
Generalized Stratigraphy	Soft CLAY	Stiff CLAY	Medium dense to dense fine to medium SAND (liquefiable)	Medium dense to dense fine to medium SAND	Dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-14	14-39	39-50	50-54	54-64	64 and deeper
Approximate El, ft	216-202	202-177	177-166	166-162	162-152	below 152
Recommend soil type	Soft clay	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	125	63	63	65	68
Cohesion (c), lbs per sq ft	700	1500	0	0	0	0
Angle of internal friction (φ), °	0	0	11	36	37	38
Subgrade modulus (k), lbs per cu in.	100	500	20	105	115	125
Strain at 50% (EE50)	0.01	0.007	NA	NA	NA	NA

Note: Pile cap bottom at ±El 216

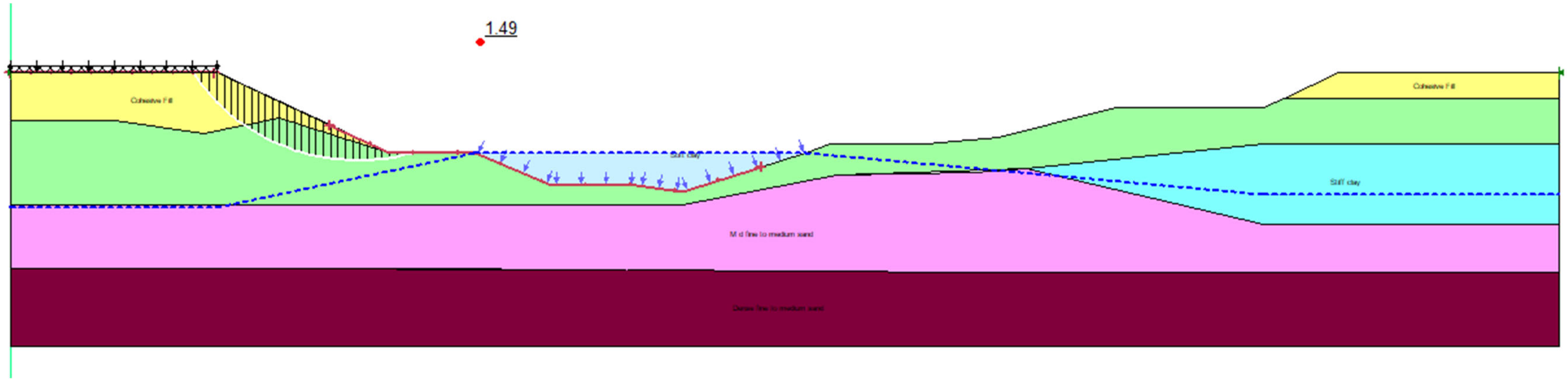
APPENDIX H

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Tyronza River
GHBW Job No. 23-031
Poinsett County, Arkansas

	Design Loading Condition	Calculated Minimum Factor of Safety
South End Slope (Bent 1) (2H:1V)	End of Construction	2.73
	Long Term	1.49
	Rapid Drawdown from El 209 to El 200	1.18
	Seismic ($k_h = A_S/2 = 0.3845$)	1.07
South Side Slope (Bent 1) (4H:1V)	End of Construction	6.53
	Long Term	3.08
	Rapid Drawdown from El 209 to Existing Grade	3.56
	Seismic ($k_h = A_S/2 = 0.3845$)	1.33
North End Slope (Bent 4) (2H:1V)	End of Construction	5.23
	Long Term	2.01
	Rapid Drawdown from El 209 to El 200	2.27
	Seismic ($k_h = A_S/2 = 0.3845$)	1.38
North Side Slope (Bent 4) (4H:1V)	End of Construction	4.78
	Long Term	2.48
	Rapid Drawdown from El 209 to Existing Grade	2.02
	Seismic ($k_h = A_S/2 = 0.3845$)	1.37



Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

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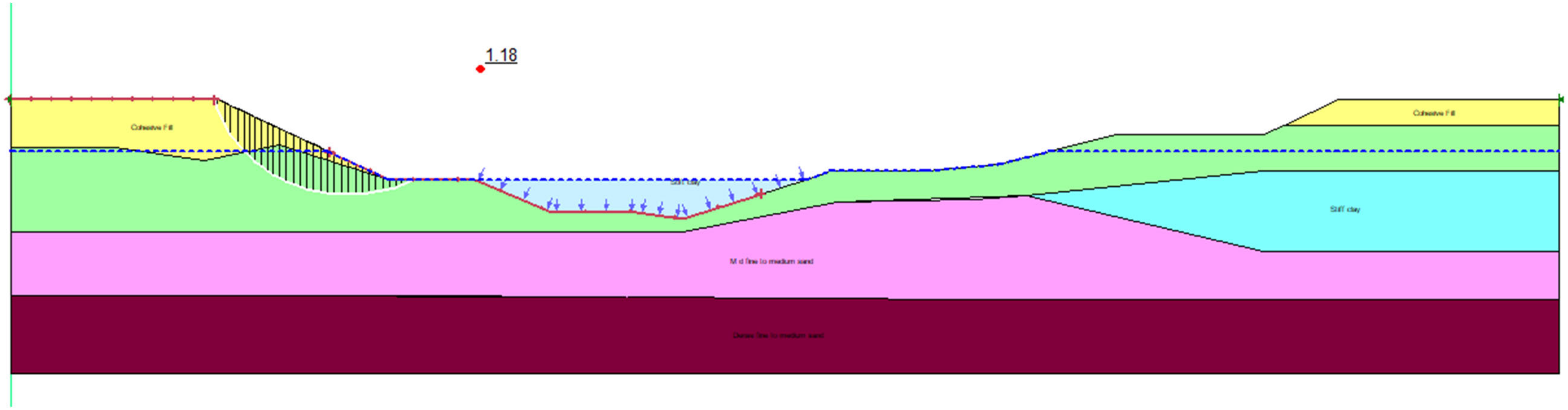
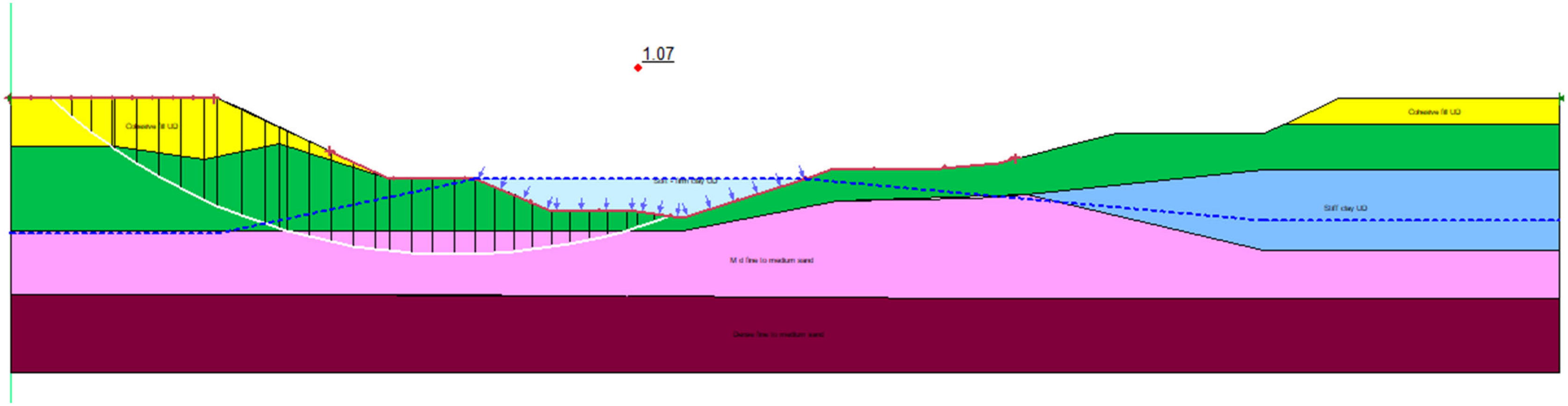


Results of Stability Analyses – Rapid Drawdown Condition from El 215 to El 213
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.3845$)
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

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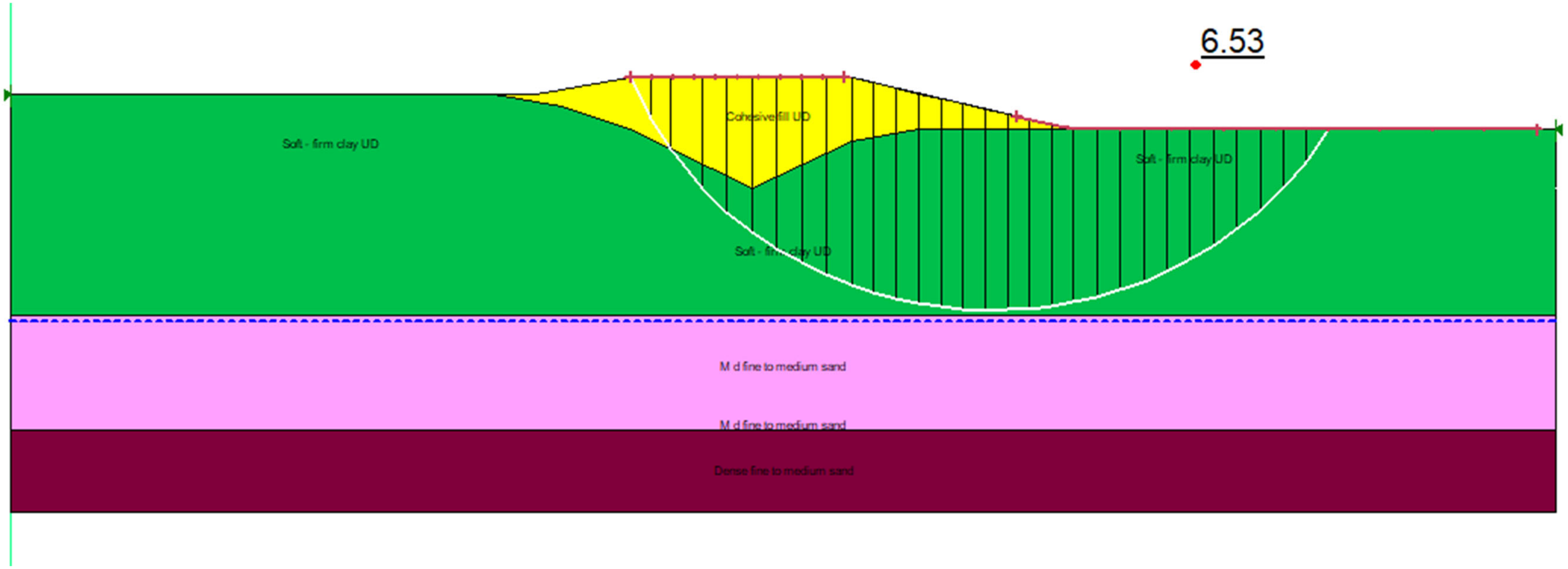
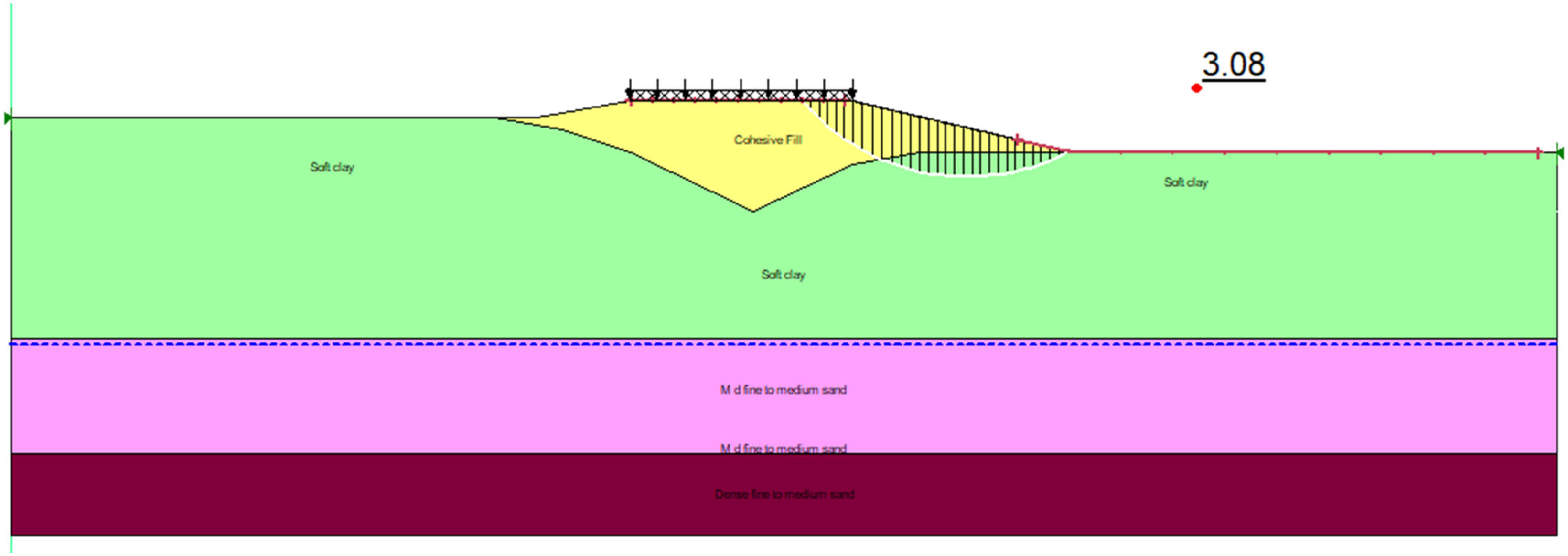
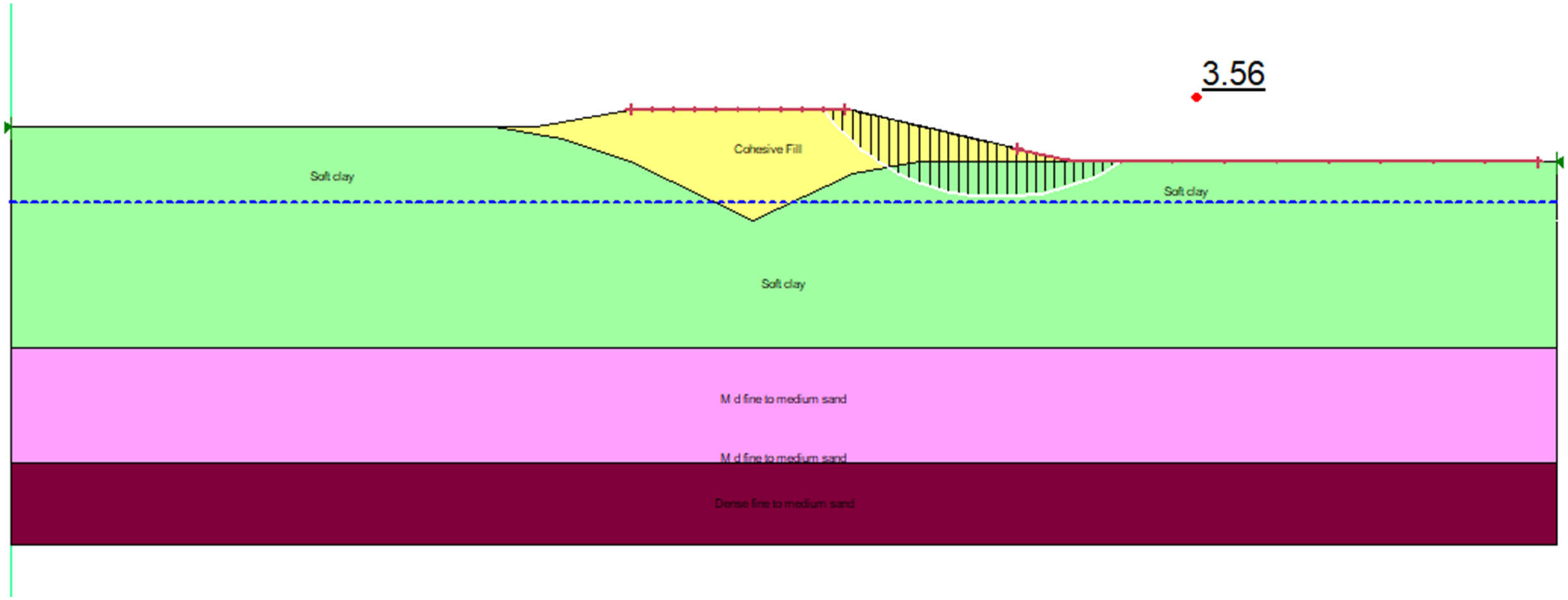


Results of Stability Analyses – End of Construction
Bent 1 Side Slope
4H:1V Slope, H=9 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Tyronza River

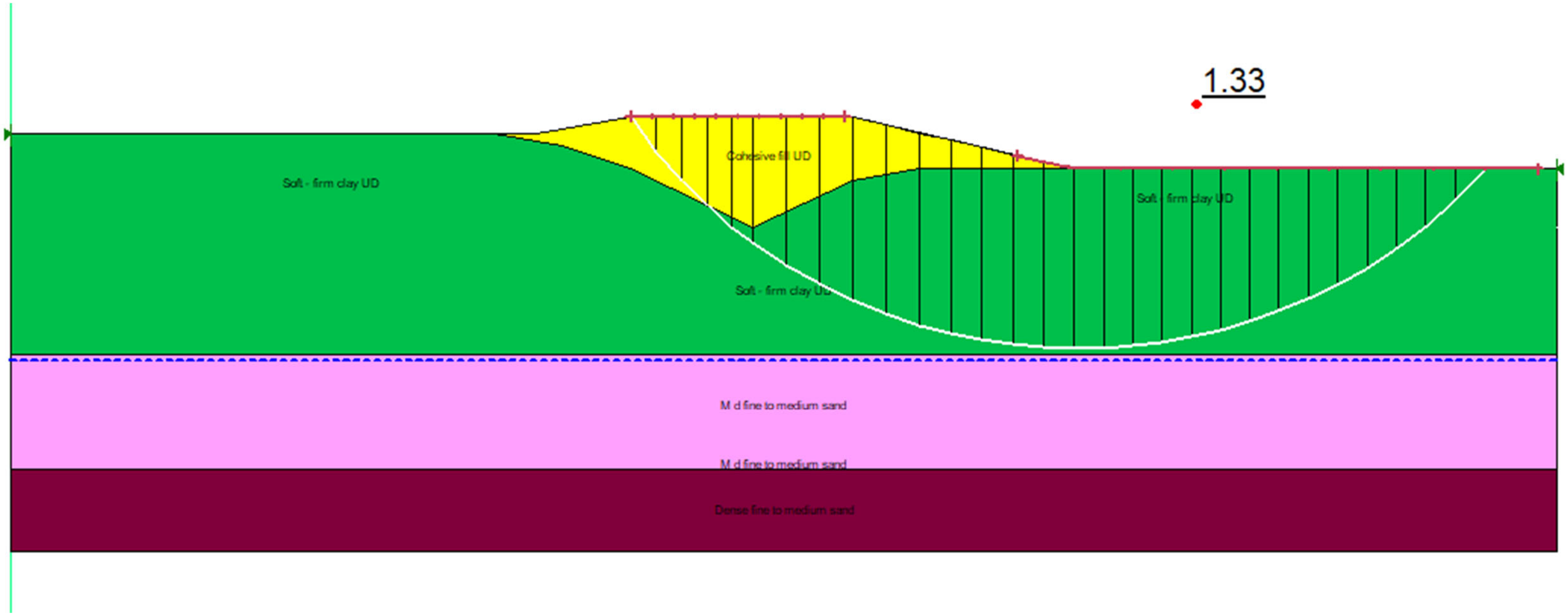
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
4H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Rapid Drawdown El 209 to Existing Grade
Bent 1 Side Slope
4H:1V Slope, H=9 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.3854$)
Bent 1 Side Slope
4H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

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CONSULTING ENGINEERS
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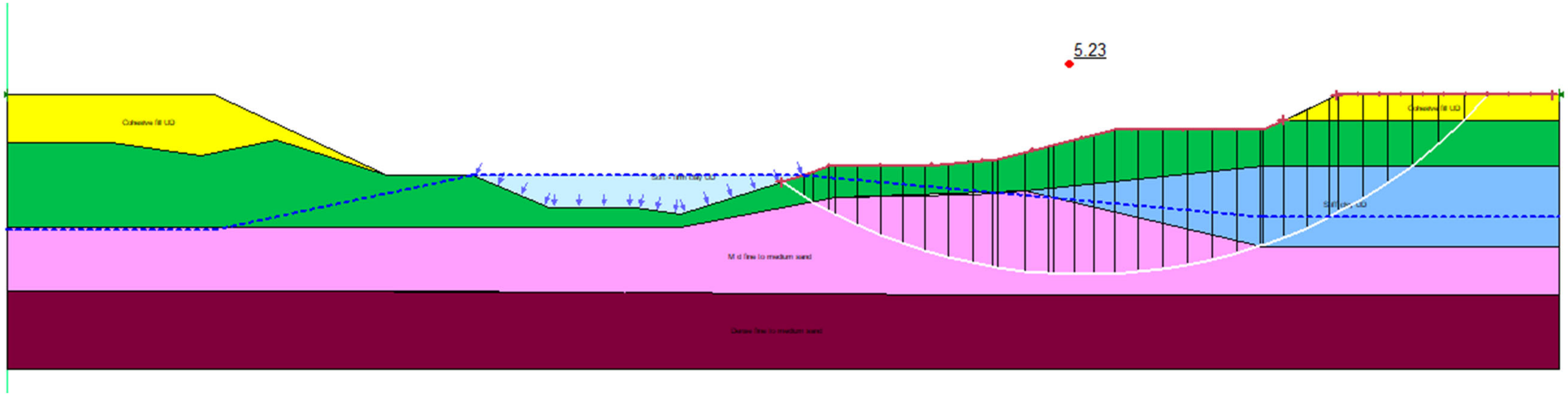
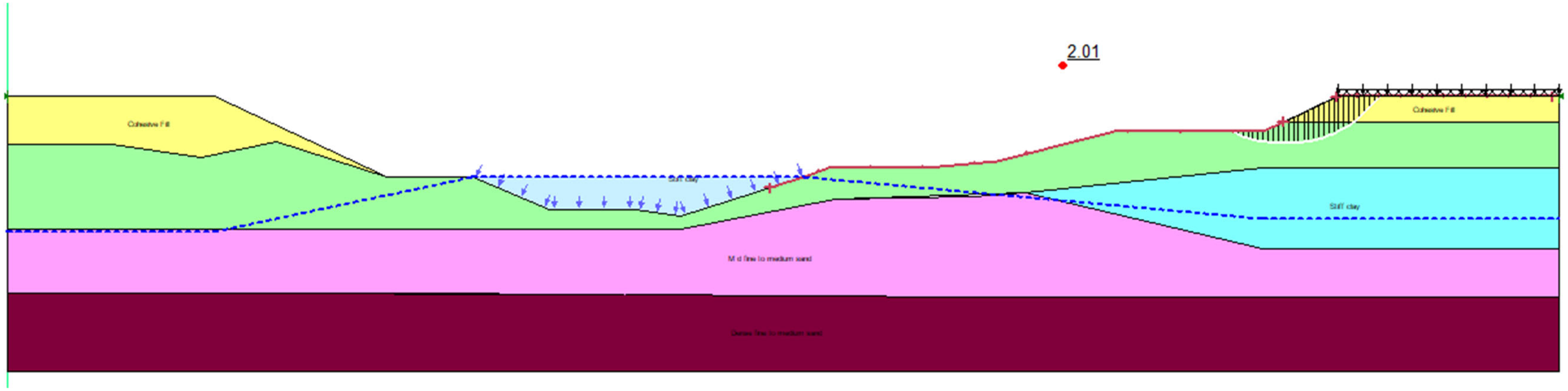


Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
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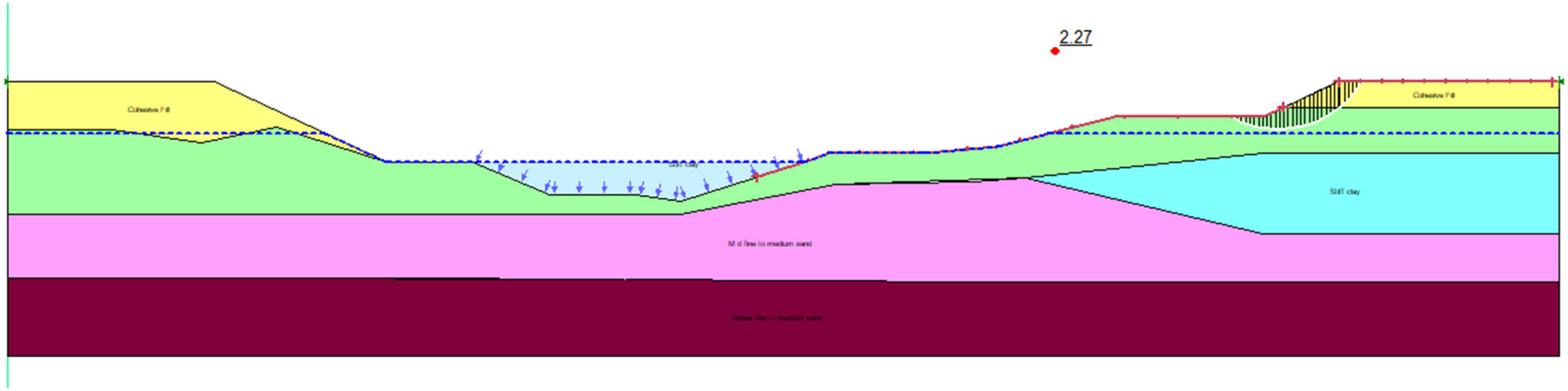
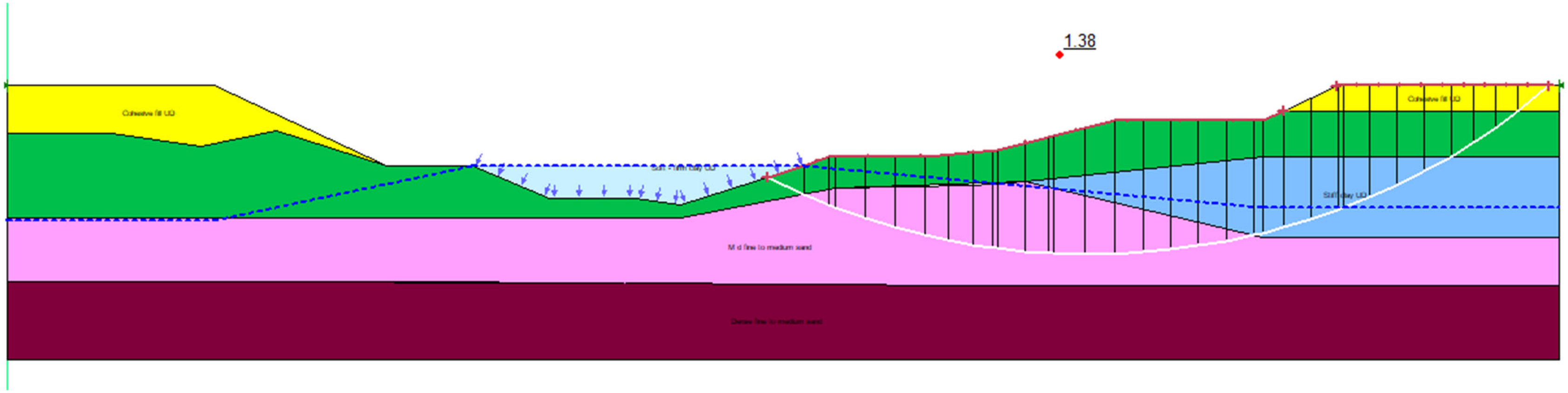
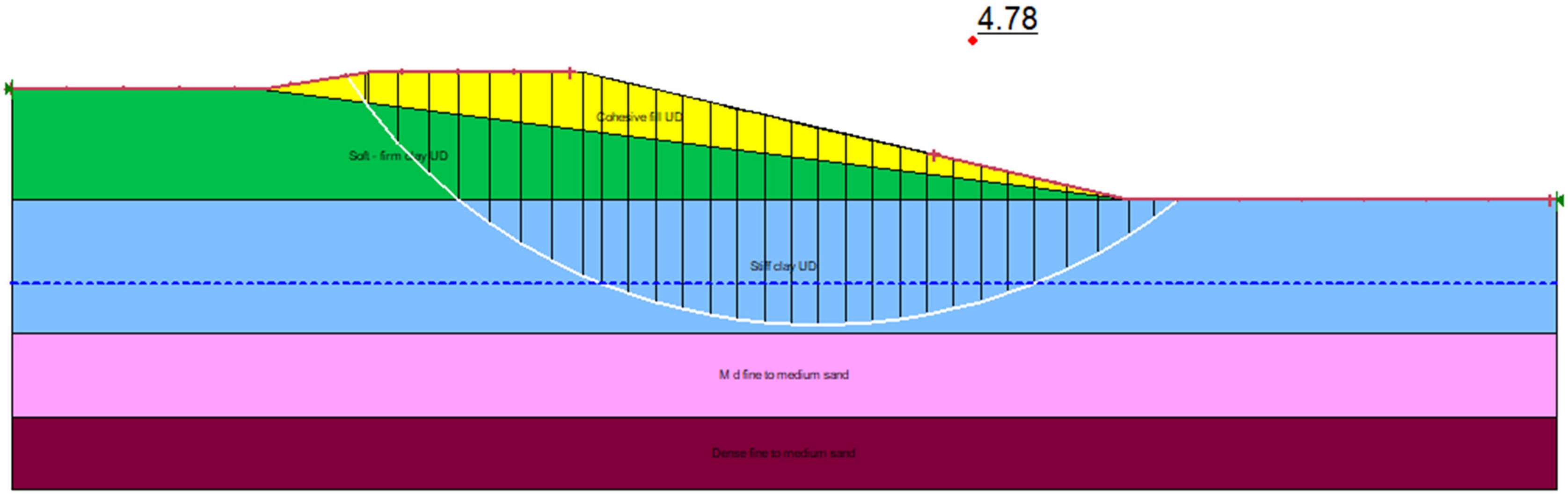


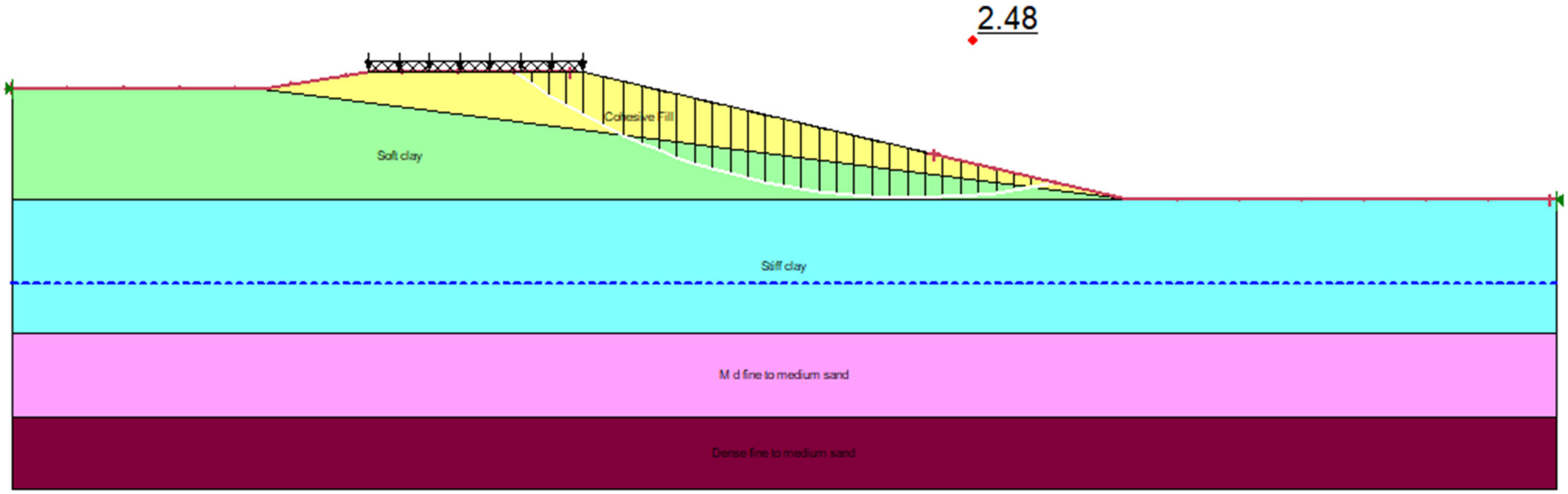
Results of Stability Analyses – Rapid Drawdown Condition, El 209 to El 200
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River



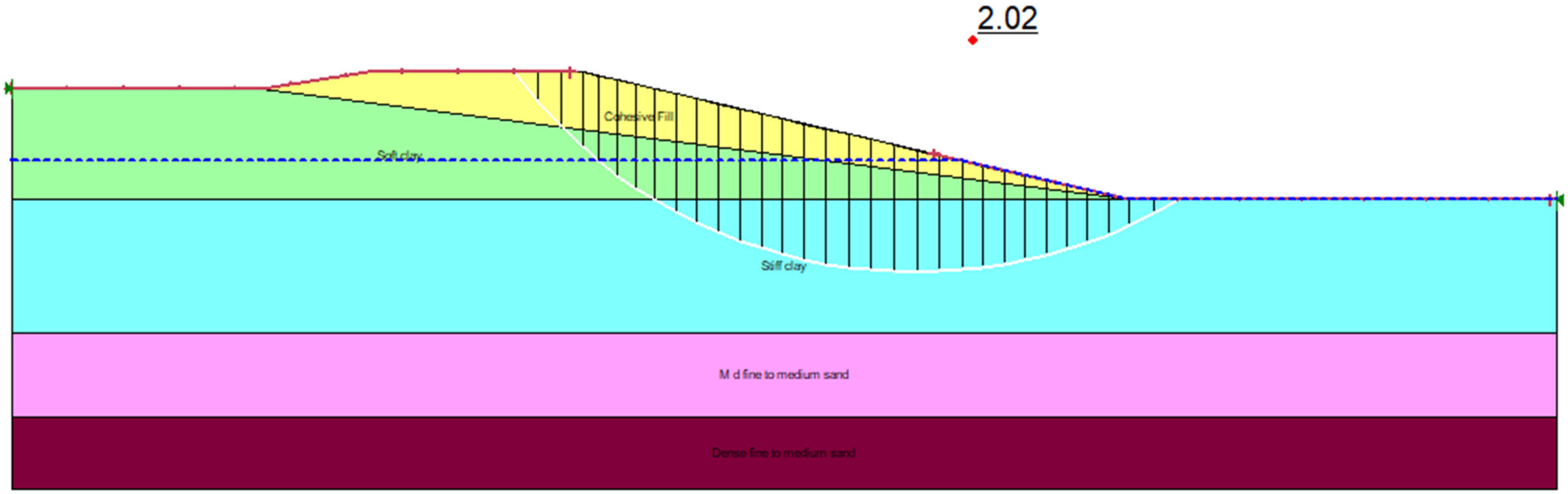
Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.3854$)
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River



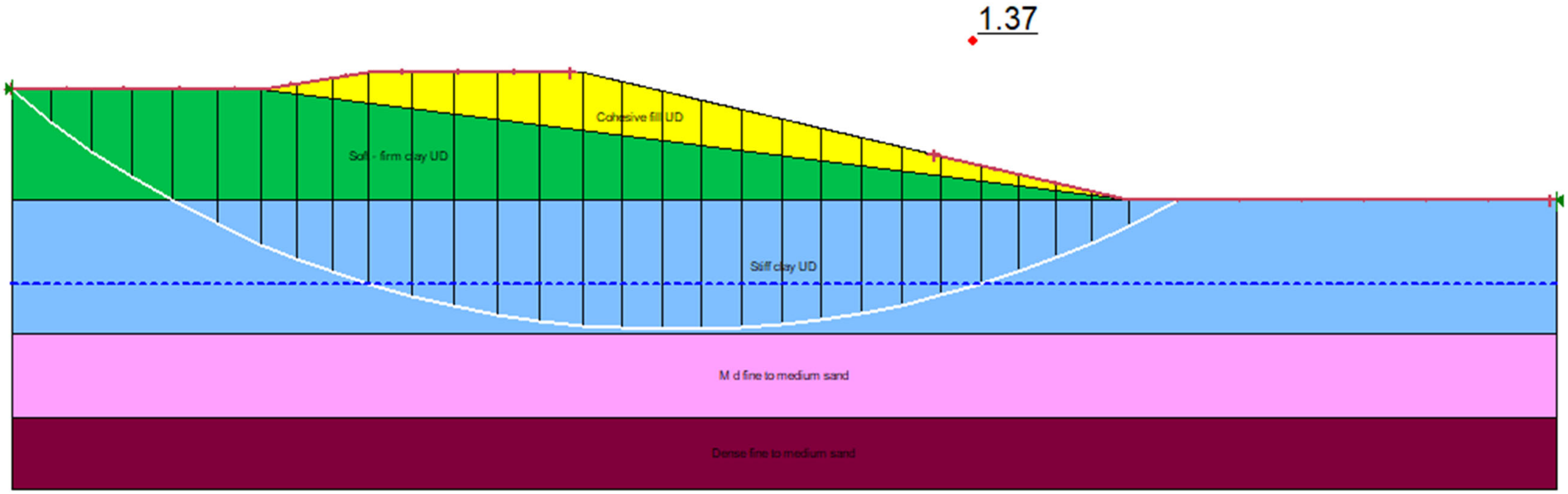
Results of Stability Analyses – End of Construction
Bent 4 Side Slope
4H:1V Slope, H=23 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
4H:1V Slope, H=23 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Rapid Drawdown Condition, El 209 to Existing Grade
Bent 4 Side Slope
4H:1V Slope, H=23 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Tyronza River



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.3845$)
Bent 4 Side Slope
4H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Tyronza River

APPENDIX I

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer’s installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX J

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, “The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project.” This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX K

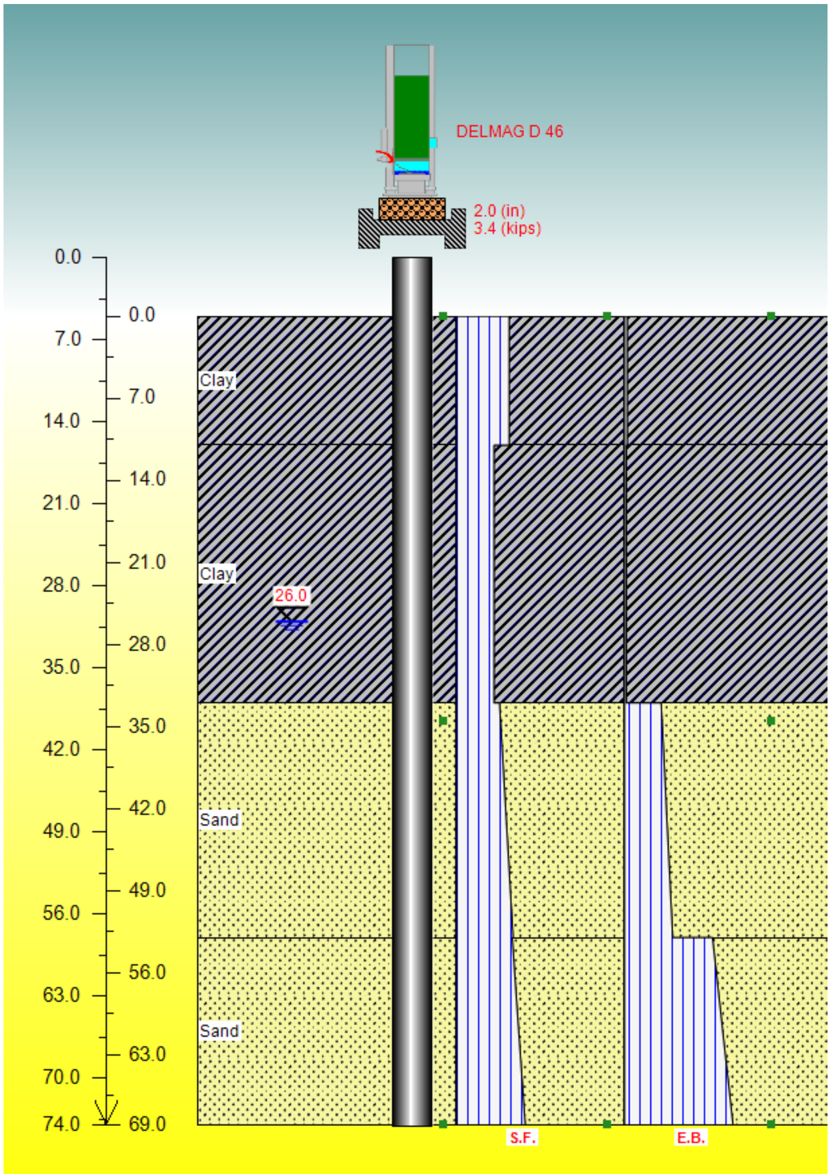
RECOMMENDED MINIMUM HAMMER ENERGY - STEEL SHELL PILES

Project: 101124 - Hwy 135
HWY. 135 OVER TYRONZA RIVER (SITE 2)
Poinsett County, Arkansas
GHBW Project No: 23-031

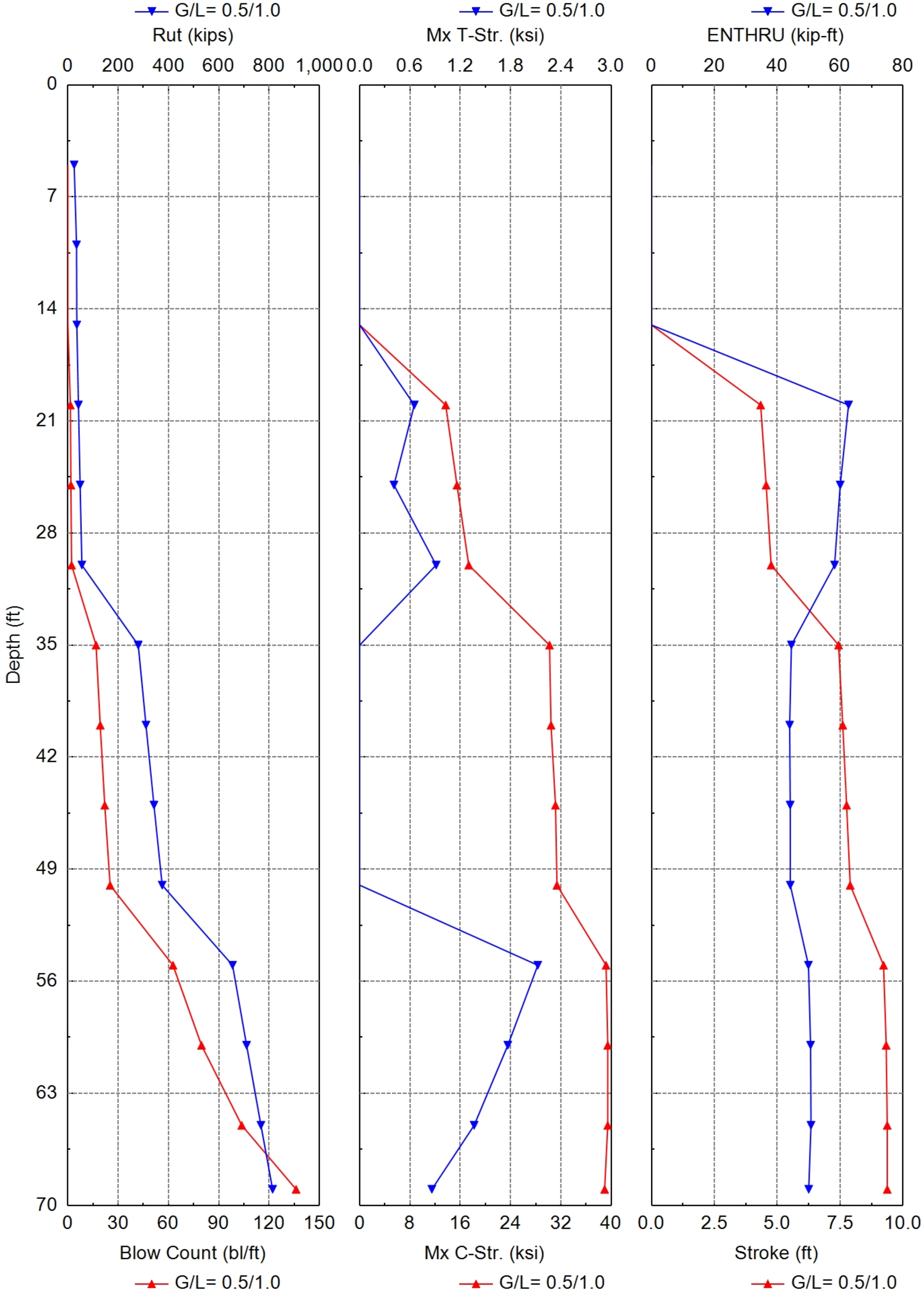
Site	Bridge	Bent	Pile Diameter (in)	Wall Thickness (in)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El.	Min Tip El.	Pile Length (ft)	Min Hammer Energy (ft-kip)	Max Comp Stress, ksi
2 - Tyronza River	B	1	18	0.50	320	217	148	69	107	39.5
		2	28	0.75	562	192	143	49	186	36.5
		3	28	0.75	562	198	138	60	186	36.2
		4	28	0.75	562	206	163	43	186	38.7
		5	18	0.50	320	216	158	58	107	36.3

Notes: 1. Driveability analyses performed utilizing GRLWEAP 2014; Pile Dynamics, Inc.
2. All piles are steel shells.

ArDOT 101124 Hwy 135 over Tyronza River
Bent 1
18-in-diameter Steel Shell Pile
Delmag D46



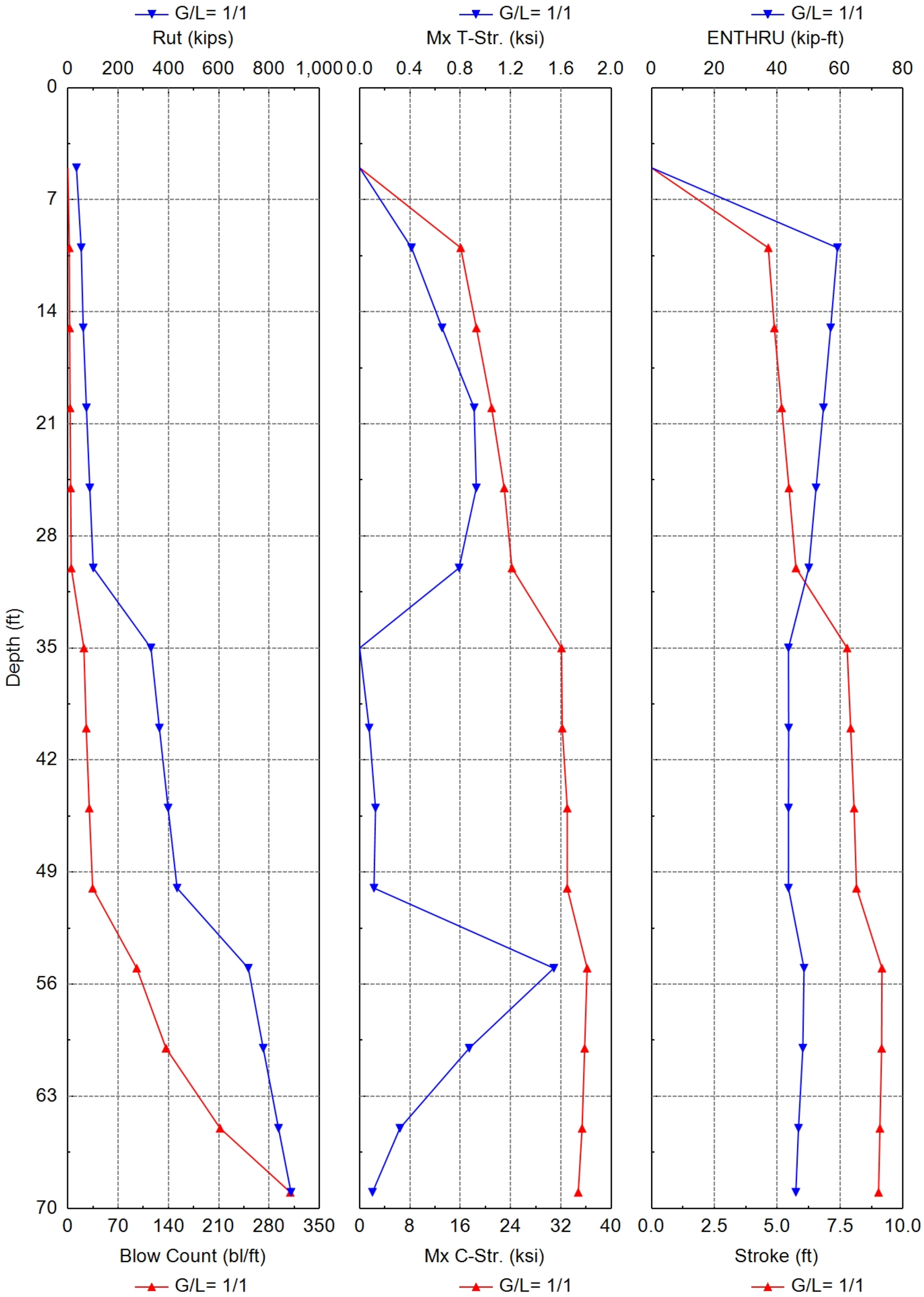
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company


G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
7
14
21
28
35
42
49
56
63
70
0
30
60
90
120
150
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	25.0	9.1	15.9	0.0	0.000	0.000	0.00	0.0	D 46
10.0	34.4	18.5	15.9	0.0	0.000	0.000	0.00	0.0	D 46
15.0	35.7	25.4	10.3	0.0	0.000	0.000	0.00	0.0	D 46
20.0	42.1	31.7	10.3	1.5	13.684	0.652	4.34	62.7	D 46
25.0	48.7	38.4	10.3	1.8	15.476	0.409	4.56	60.0	D 46
30.0	55.7	45.3	10.3	2.2	17.348	0.912	4.76	58.2	D 46
35.0	279.8	54.8	225.0	16.8	30.194	0.000	7.44	44.5	D 46
40.0	310.5	68.3	242.2	19.3	30.434	0.000	7.61	44.0	D 46
45.0	342.3	82.9	259.4	22.0	31.145	0.000	7.76	44.1	D 46
50.0	375.0	98.4	276.6	25.2	31.378	0.000	7.90	44.2	D 46
55.0	654.8	114.9	540.0	62.7	39.188	2.125	9.23	49.9	D 46
60.0	710.6	132.3	578.3	79.7	39.452	1.767	9.33	50.6	D 46
65.0	767.6	151.0	616.6	103.7	39.453	1.366	9.37	50.7	D 46
69.0	814.1	166.8	647.3	136.1	38.960	0.861	9.38	50.0	D 46

Total driving time: 48 minutes; Total Number of Blows: 1895 (starting at penetration 5.0 ft)


▼ G/L= 1/1
Rut (kips)
0 200 400 600 800 1,000
▼ G/L= 1/1
Mx T-Str. (ksi)
0.0 0.4 0.8 1.2 1.6 2.0
▼ G/L= 1/1
ENTHRU (kip-ft)
0 20 40 60 80
Depth (ft)
0 7 14 21 28 35 42 49 56 63 70
0 70 140 210 280 350
Blow Count (bl/ft)
▲ G/L= 1/1
0 8 16 24 32 40
Mx C-Str. (ksi)
▲ G/L= 1/1
0.0 2.5 5.0 7.5 10.0
Stroke (ft)
▲ G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	34.1	18.2	15.9	0.0	0.000	0.000	0.00	0.0	D 46
10.0	53.0	37.1	15.9	2.0	16.071	0.412	4.65	59.2	D 46
15.0	61.1	50.7	10.3	2.5	18.561	0.655	4.89	57.0	D 46
20.0	73.8	63.5	10.3	3.1	20.988	0.910	5.18	54.7	D 46
25.0	87.1	76.8	10.3	3.9	22.983	0.928	5.47	52.4	D 46
30.0	101.0	90.7	10.3	4.7	24.218	0.790	5.74	50.0	D 46
35.0	330.4	105.4	225.0	22.3	32.101	0.000	7.78	43.6	D 46
40.0	363.9	121.7	242.2	25.6	32.221	0.075	7.92	43.6	D 46
45.0	398.5	139.1	259.4	29.8	33.013	0.126	8.06	43.6	D 46
50.0	434.4	157.8	276.6	34.5	33.007	0.114	8.15	43.6	D 46
55.0	717.5	177.5	540.0	95.8	36.176	1.544	9.16	48.5	D 46
60.0	776.8	198.5	578.3	136.6	35.785	0.871	9.15	48.1	D 46
65.0	837.5	220.9	616.6	211.9	35.372	0.320	9.08	46.8	D 46
69.0	887.1	239.8	647.3	309.6	34.767	0.102	9.03	45.9	D 46

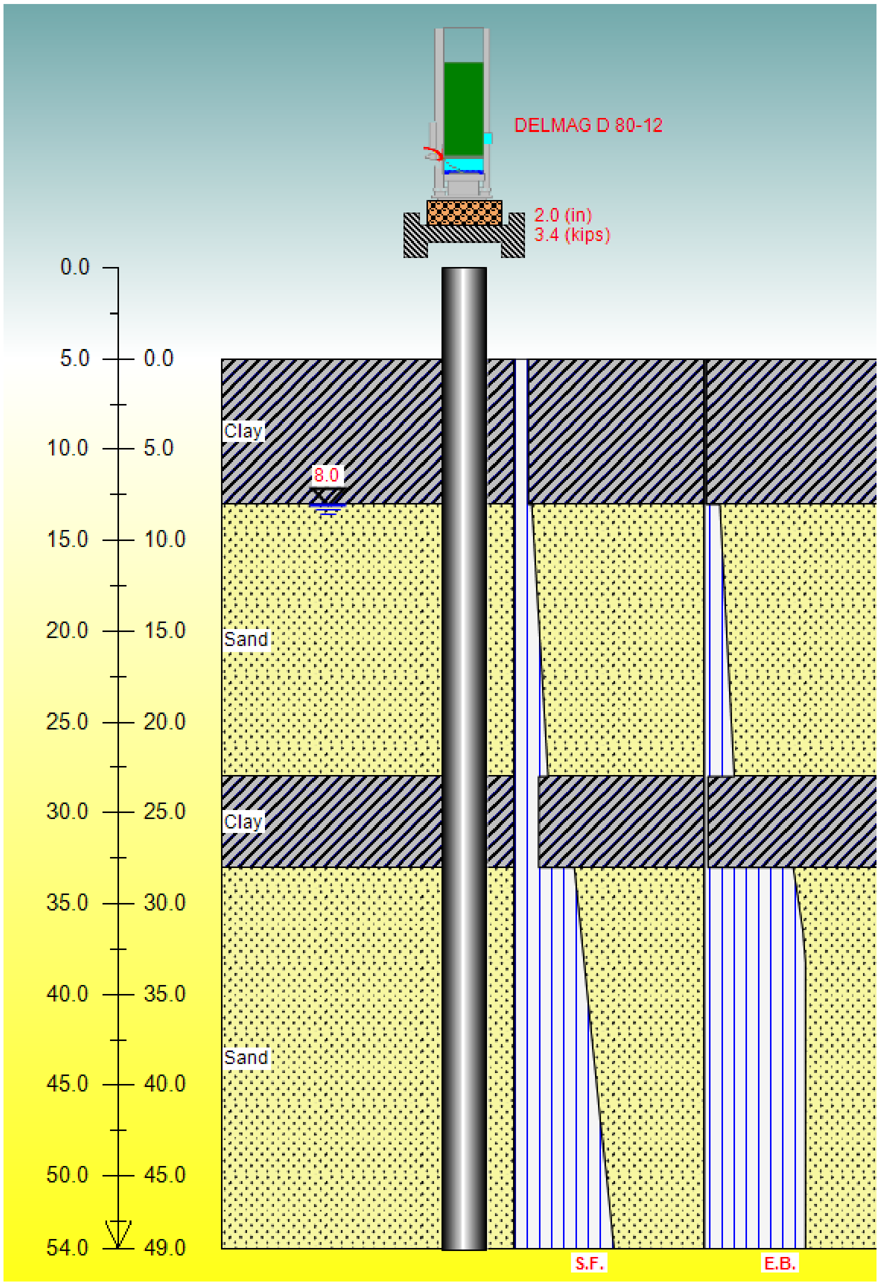
Total driving time: 85 minutes; Total Number of Blows: 3376 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Tyronza River

Bent 2

28-in-diameter Steel Shell Pile

Delmag D80-12



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CONSULTING ENGINEERS
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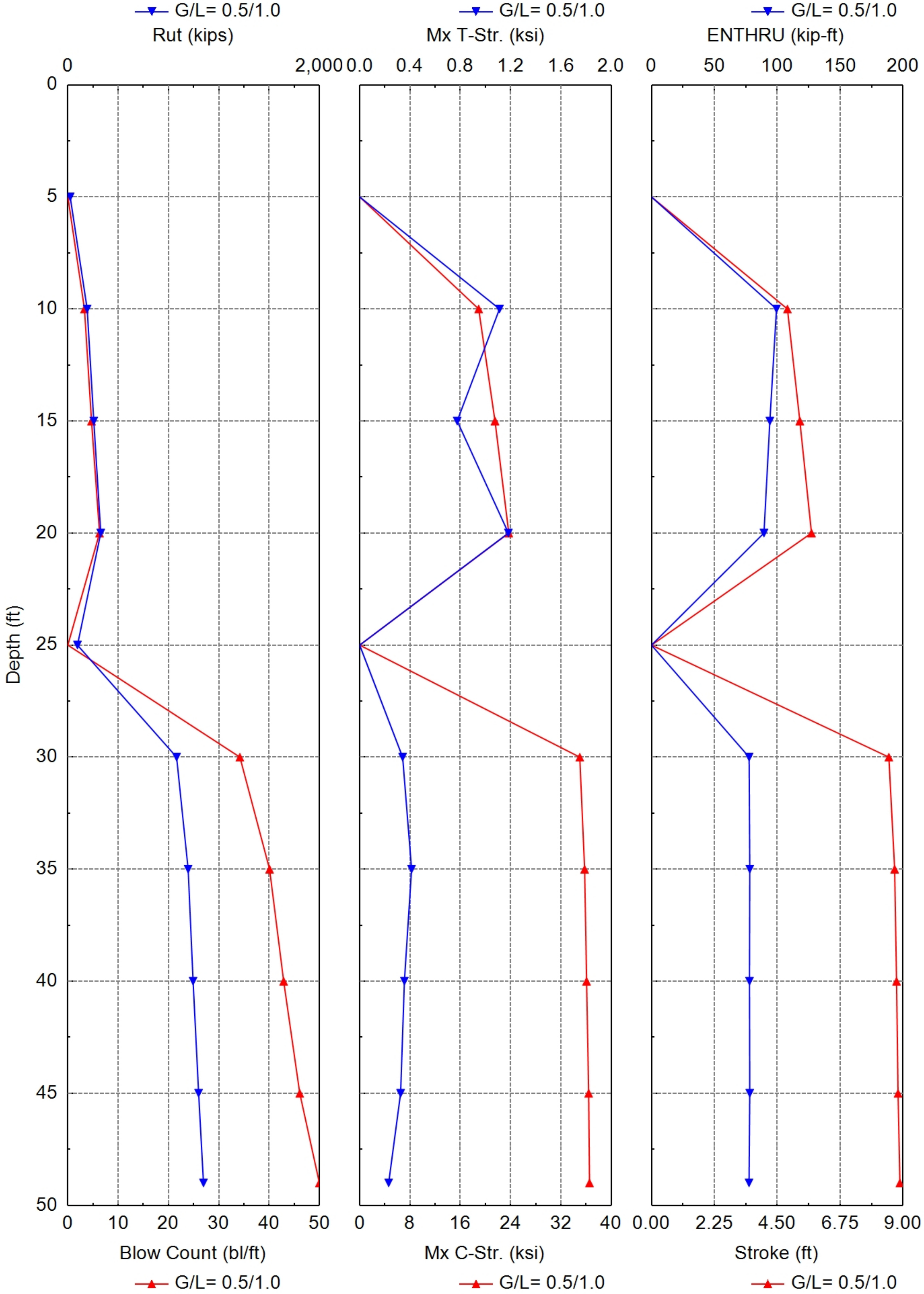
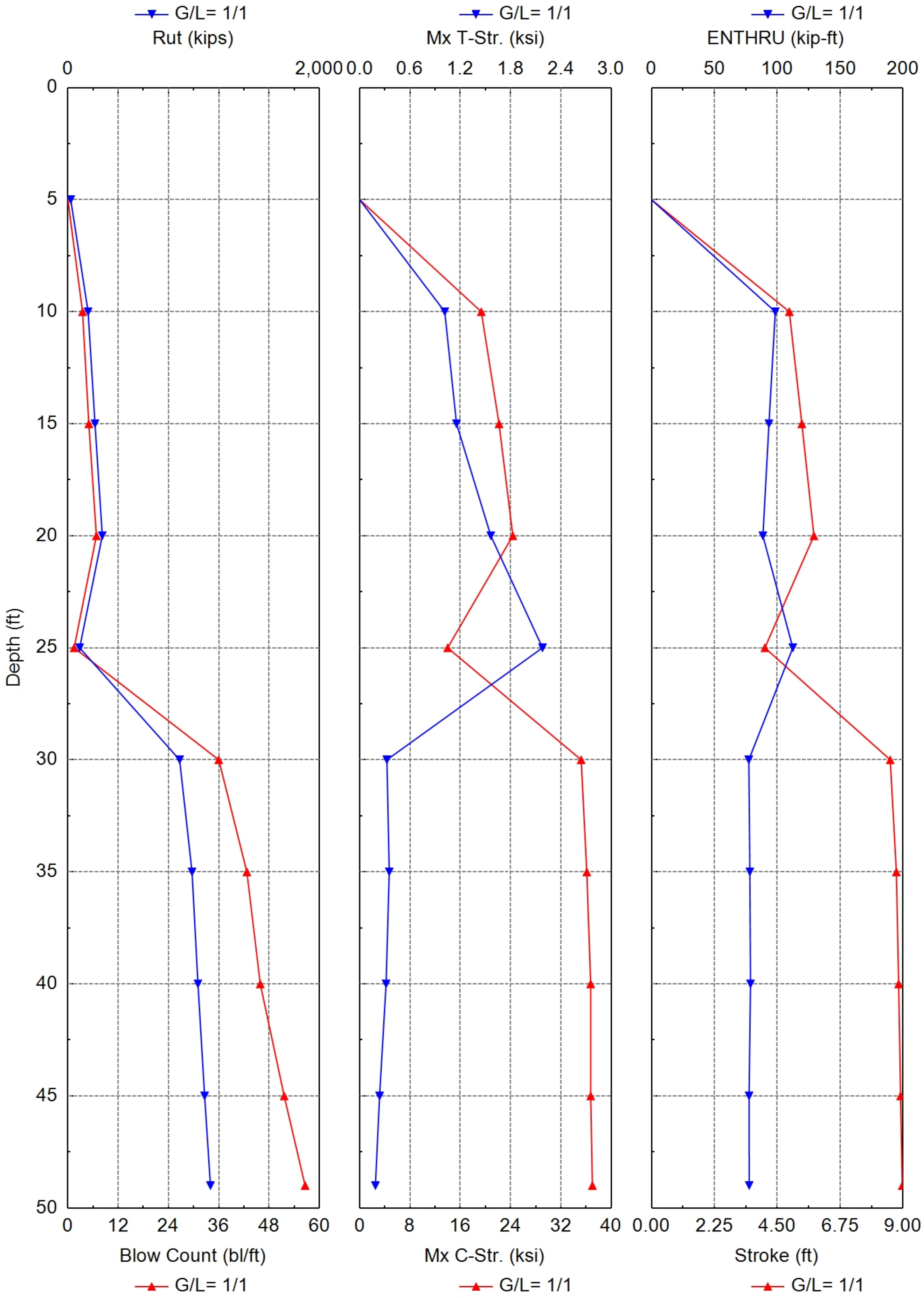

G/L= 0.5/1.0
Rut (kips)
0
2,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
0
10
20
30
40
50
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.5	4.2	14.3	0.3	0.000	0.000	10.57	0.0	D 80-12
10.0	153.5	10.4	143.1	3.3	18.927	1.112	4.87	99.4	D 80-12
15.0	206.1	21.4	184.7	4.7	21.521	0.775	5.31	94.1	D 80-12
20.0	261.4	35.2	226.2	6.3	23.713	1.183	5.73	89.4	D 80-12
25.0	76.4	47.8	28.6	0.0	0.000	0.000	0.00	0.0	D 80-12
30.0	864.5	64.6	799.9	34.2	35.004	0.342	8.49	77.7	D 80-12
35.0	956.4	99.1	857.4	40.1	35.779	0.413	8.70	78.2	D 80-12
40.0	995.7	138.4	857.4	42.9	36.088	0.356	8.77	77.9	D 80-12
45.0	1039.8	182.4	857.4	46.1	36.418	0.326	8.82	78.1	D 80-12
49.0	1078.4	221.1	857.4	50.0	36.553	0.231	8.89	77.6	D 80-12

Total driving time: 26 minutes; Total Number of Blows: 965 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
2,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 1/1
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
0
12
24
36
48
60
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer
5.0	22.7	8.4	14.3	0.0	0.000	0.000	0.00	0.0	D 80-12
10.0	161.0	17.8	143.1	3.5	19.344	1.015	4.93	98.5	D 80-12
15.0	215.7	31.1	184.7	5.0	22.166	1.155	5.38	93.4	D 80-12
20.0	273.9	47.7	226.2	6.8	24.364	1.564	5.81	88.6	D 80-12
25.0	93.6	65.1	28.6	1.5	13.993	2.181	4.06	112.4	D 80-12
30.0	888.6	88.7	799.9	36.0	35.227	0.325	8.54	77.4	D 80-12
35.0	987.5	130.1	857.4	42.7	36.126	0.353	8.76	78.2	D 80-12
40.0	1034.6	177.2	857.4	45.9	36.742	0.315	8.85	78.7	D 80-12
45.0	1087.4	230.1	857.4	51.6	36.746	0.237	8.91	77.6	D 80-12
49.0	1133.9	276.5	857.4	56.6	37.014	0.188	8.97	77.7	D 80-12

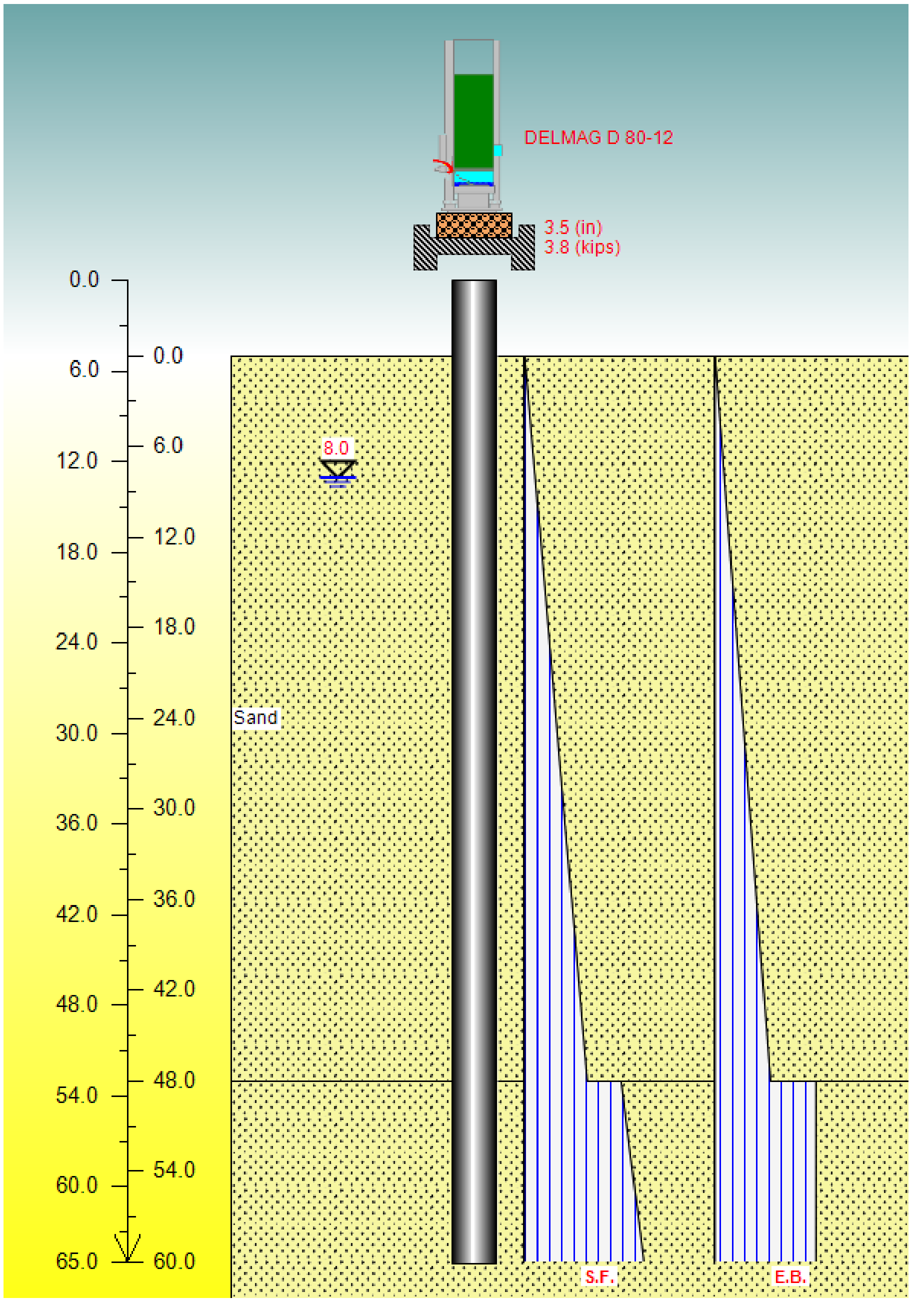
Total driving time: 26 minutes; Total Number of Blows: 1052 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Tyronza River

Bent 3

28-in-diameter Steel Shell Pile

Delmag D80-12



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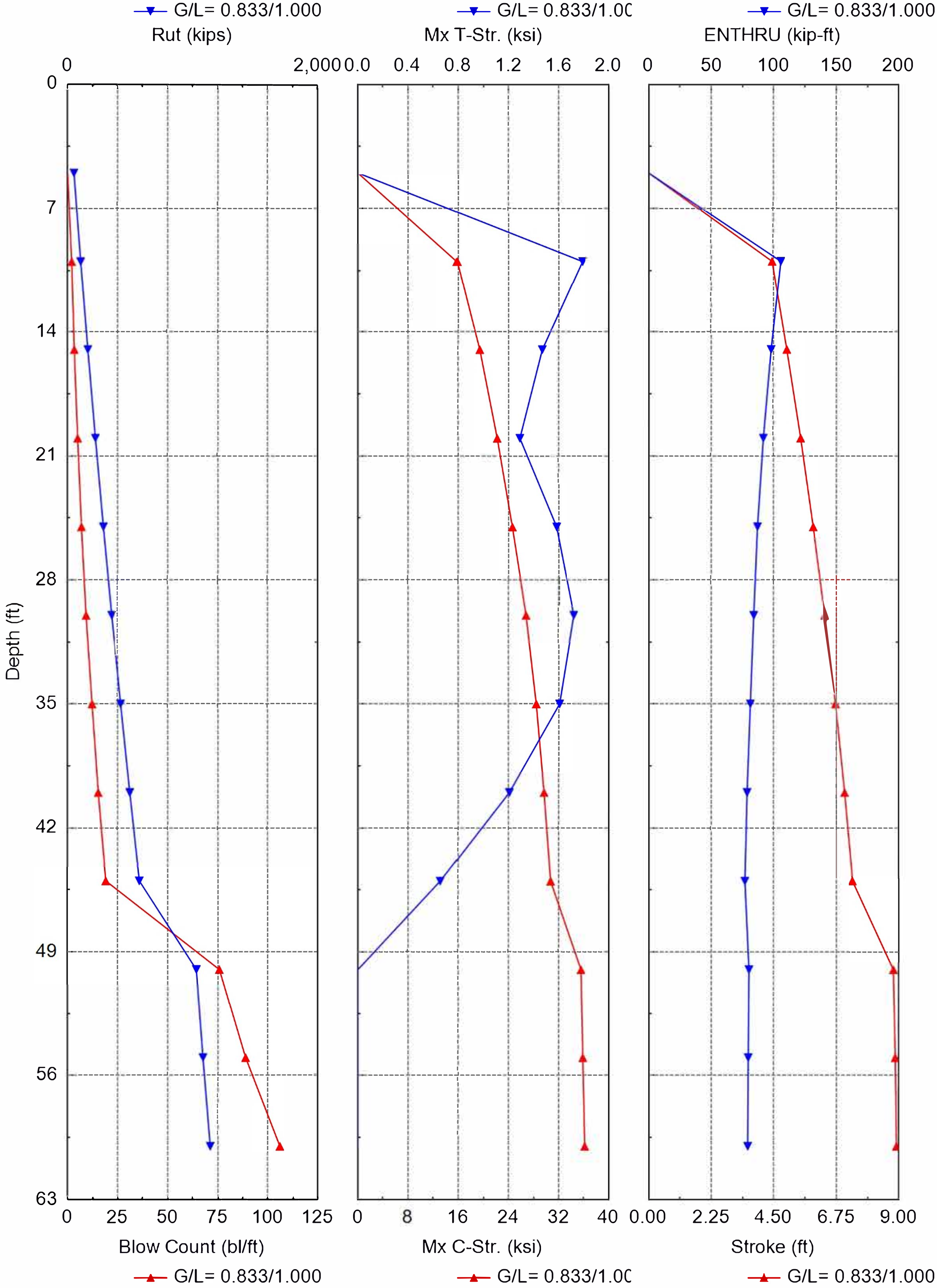
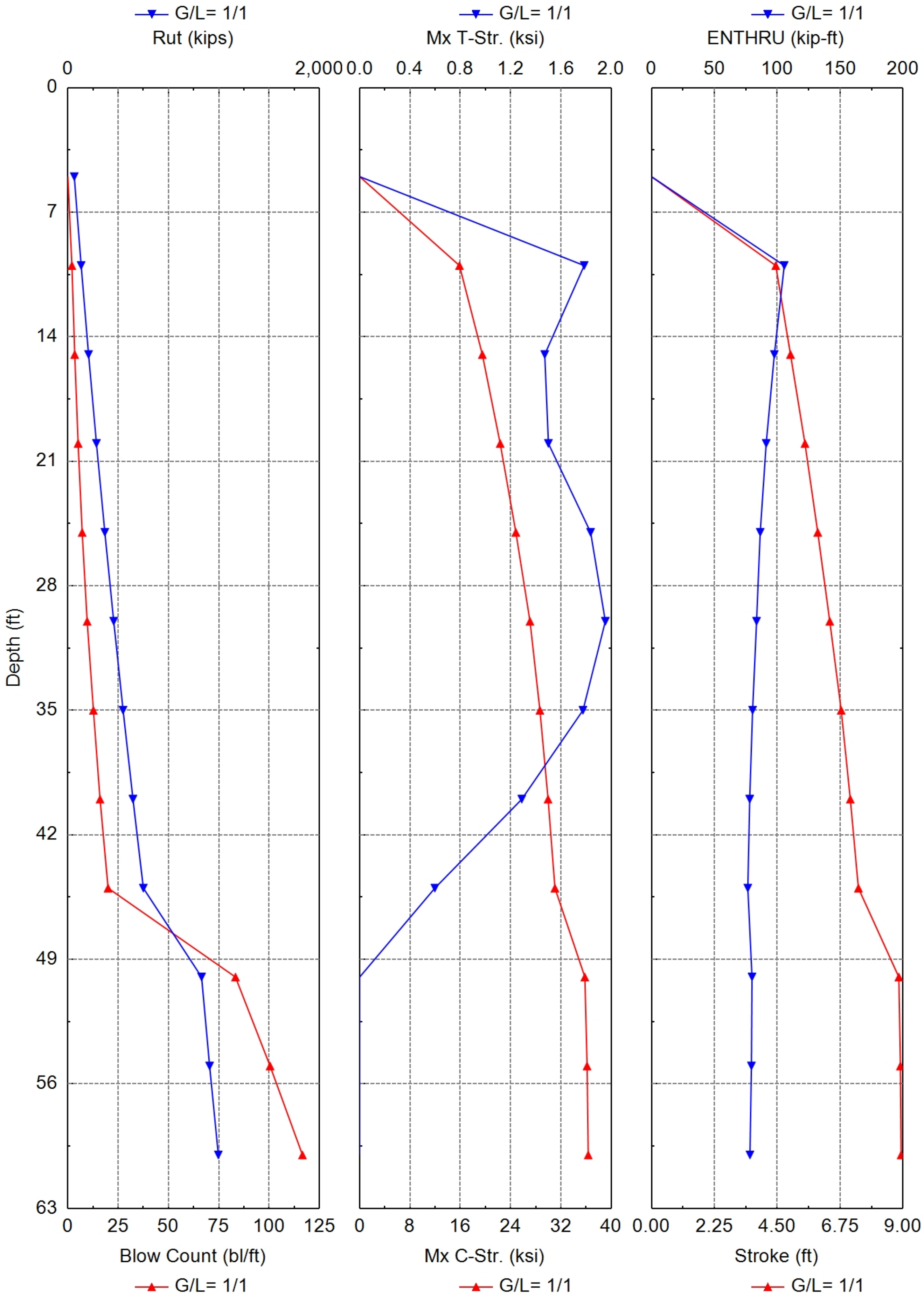

G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
7
14
21
28
35
42
49
56
63
0
25
50
75
100
125
Blow Count (bl/ft)
0
8
16
24
32
40
Mx C-Str. (ksi)
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.833/1.000
G/L= 0.833/1.00
G/L= 0.833/1.000

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	50.6	1.6	48.9	0.0	0.000	0.000	0.00	0.0	D 80-12
10.0	104.4	6.6	97.8	2.0	15.792	1.790	4.44	105.6	D 80-12
15.0	161.6	14.8	146.7	3.3	19.370	1.468	4.97	97.8	D 80-12
20.0	222.0	26.4	195.7	5.0	22.153	1.289	5.46	91.7	D 80-12
25.0	285.7	41.2	244.6	6.9	24.574	1.583	5.92	87.0	D 80-12
30.0	352.8	59.3	293.5	9.2	26.795	1.721	6.33	84.0	D 80-12
35.0	423.1	80.7	342.4	12.2	28.419	1.606	6.72	81.3	D 80-12
40.0	496.7	105.4	391.3	15.3	29.671	1.206	7.05	78.7	D 80-12
45.0	573.6	133.4	440.2	19.1	30.718	0.658	7.34	76.9	D 80-12
50.0	1029.0	171.7	857.4	75.8	35.578	0.000	8.80	80.2	D 80-12
55.0	1082.1	224.8	857.4	88.9	35.868	0.000	8.87	79.6	D 80-12
60.0	1140.0	282.7	857.4	106.0	36.174	0.000	8.91	79.1	D 80-12

Total driving time: 35 minutes; Total Number of Blows: 1452 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
2,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 1/1
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
7
14
21
28
35
42
49
56
63
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	50.9	2.0	48.9	0.0	0.000	0.000	0.00	0.0	D 80-12
10.0	105.7	7.9	97.8	2.0	15.873	1.785	4.45	105.6	D 80-12
15.0	164.5	17.8	146.7	3.4	19.511	1.472	4.98	97.7	D 80-12
20.0	227.3	31.6	195.7	5.1	22.348	1.501	5.49	91.2	D 80-12
25.0	294.0	49.4	244.6	7.1	24.841	1.836	5.95	86.5	D 80-12
30.0	364.6	71.1	293.5	9.6	27.090	1.954	6.38	83.6	D 80-12
35.0	439.2	96.8	342.4	12.7	28.668	1.776	6.79	80.4	D 80-12
40.0	517.8	126.5	391.3	16.0	29.967	1.289	7.12	78.1	D 80-12
45.0	600.3	160.1	440.2	20.0	31.060	0.598	7.41	76.6	D 80-12
50.0	1063.4	206.0	857.4	83.3	35.816	0.000	8.85	80.0	D 80-12
55.0	1127.1	269.7	857.4	100.5	36.169	0.000	8.91	79.5	D 80-12
60.0	1196.6	339.2	857.4	116.6	36.349	0.000	8.93	78.3	D 80-12

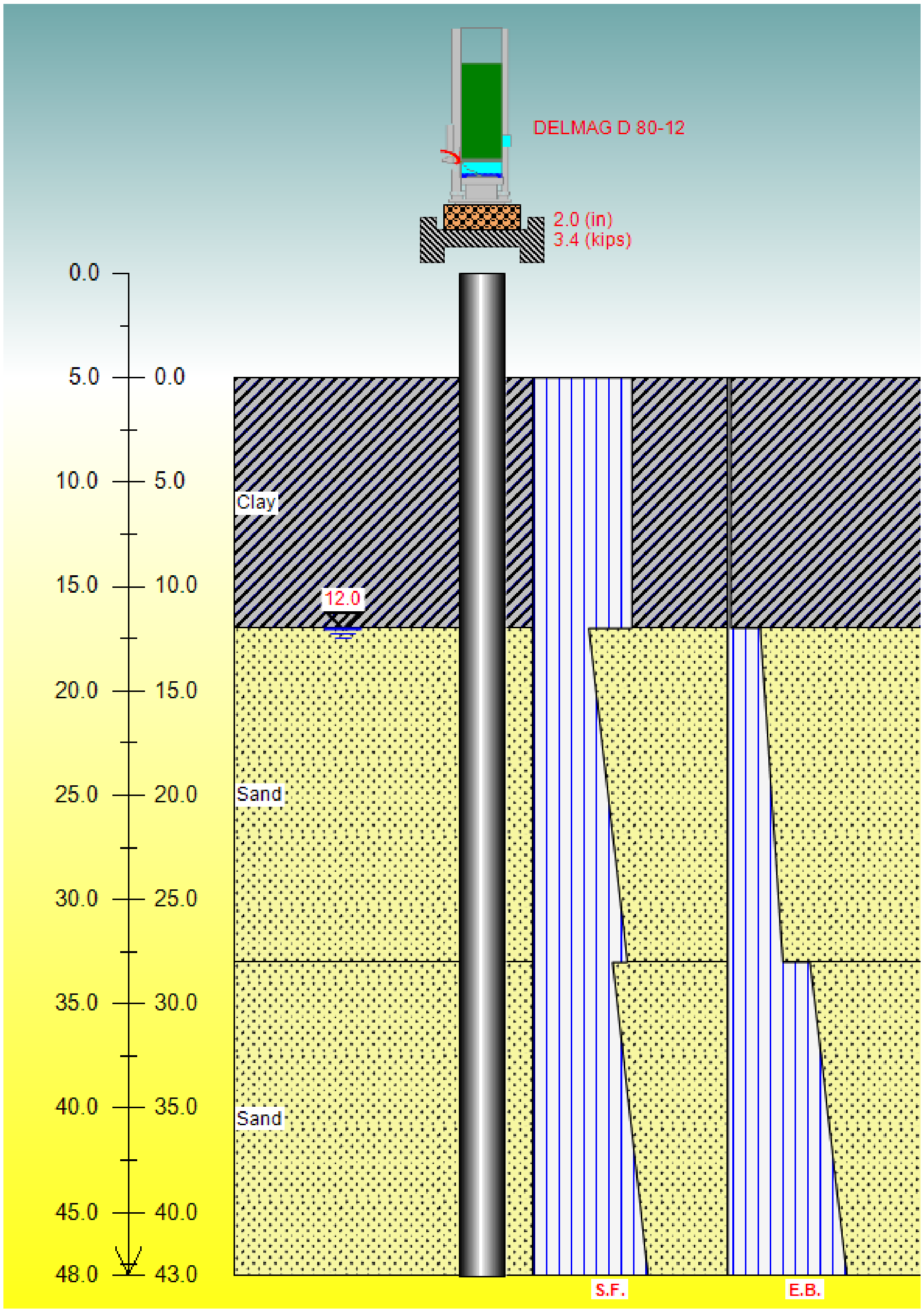
Total driving time: 39 minutes; Total Number of Blows: 1589 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Tyronza River

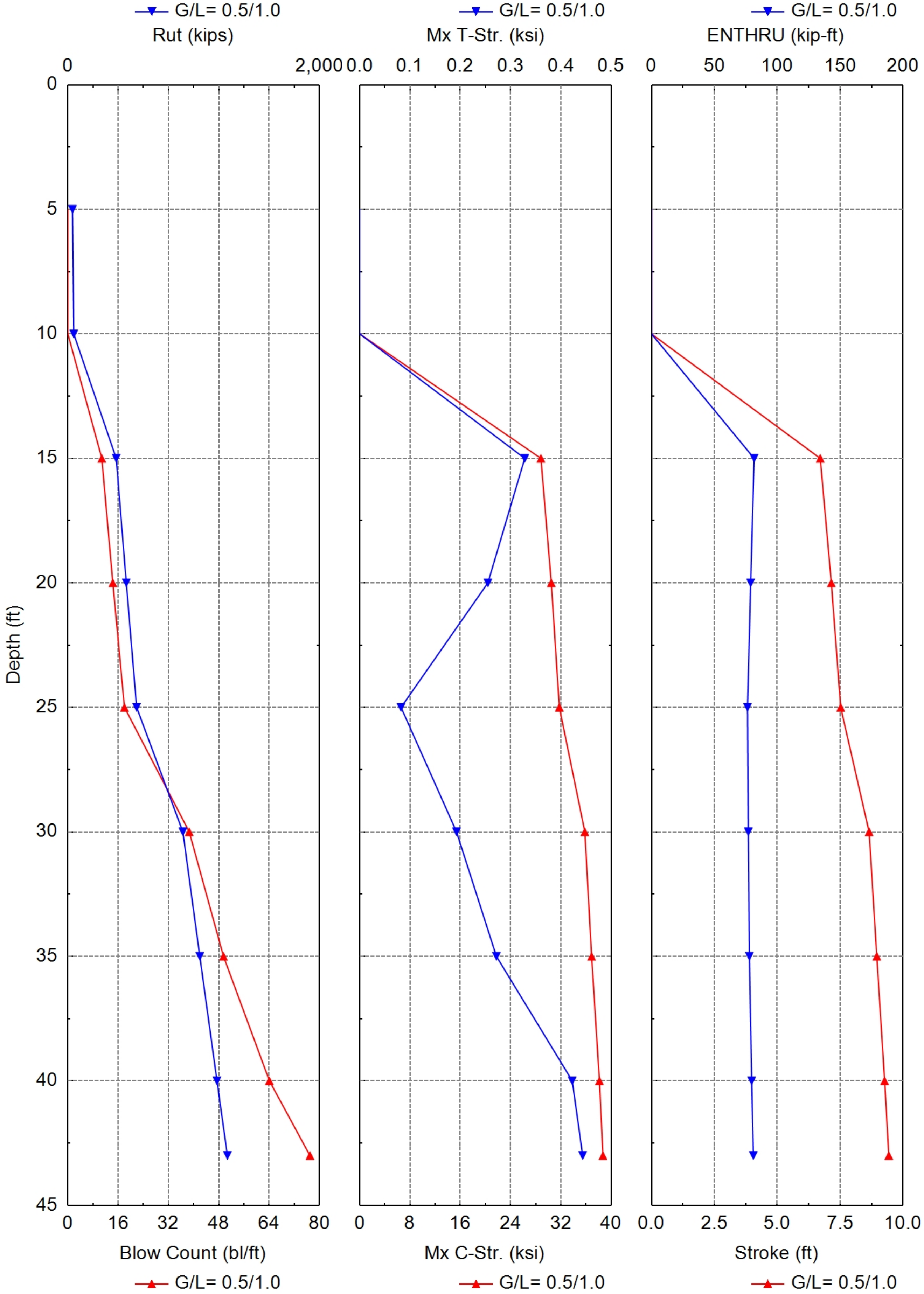
Bent 4

28-in-diameter Steel Shell Pile

Delmag D80-12



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

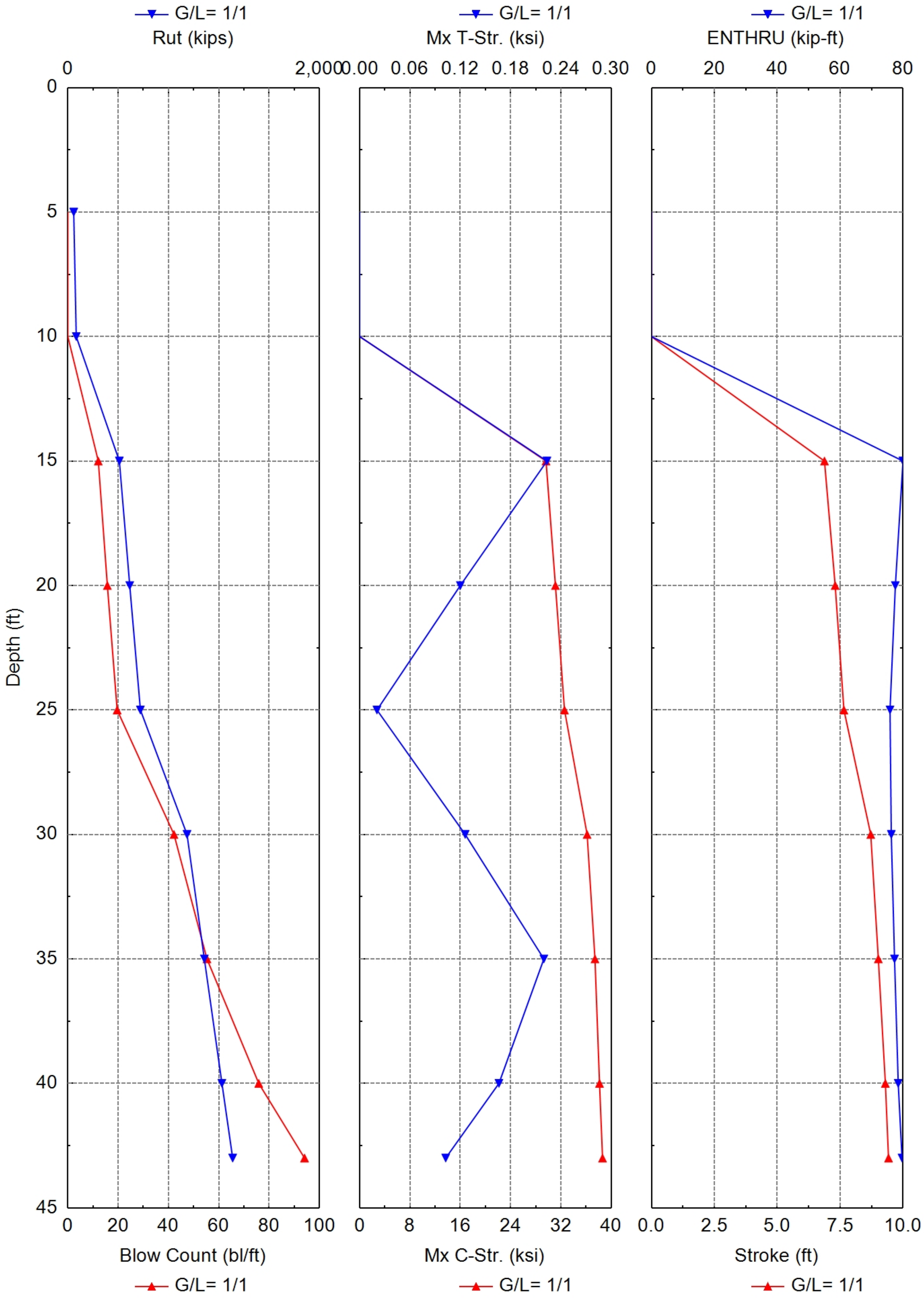

G/L= 0.5/1.0
Rut (kips)
0
2,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.1
0.2
0.3
0.4
0.5
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
16
32
48
64
80
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	36.7	9.7	26.9	0.0	0.000	0.000	0.00	0.0	D 80-12
10.0	46.7	19.8	26.9	0.0	0.000	0.000	0.00	0.0	D 80-12
15.0	384.5	29.8	354.7	10.8	28.827	0.328	6.71	81.6	D 80-12
20.0	464.7	41.3	423.3	14.3	30.477	0.255	7.15	78.9	D 80-12
25.0	546.8	54.9	491.9	18.0	31.757	0.082	7.52	76.4	D 80-12
30.0	914.5	69.5	845.0	38.6	35.812	0.193	8.65	77.0	D 80-12
35.0	1049.1	84.6	964.5	49.5	36.890	0.272	8.96	77.9	D 80-12
40.0	1185.8	101.7	1084.1	64.1	38.132	0.423	9.27	79.7	D 80-12
43.0	1268.7	112.9	1155.8	77.0	38.691	0.443	9.44	81.0	D 80-12

Total driving time: 26 minutes; Total Number of Blows: 1027 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
2,000
G/L= 1/1
Mx T-Str. (ksi)
0.00
0.06
0.12
0.18
0.24
0.30
G/L= 1/1
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
20
40
60
80
100
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	46.4	19.5	26.9	0.0	0.000	0.000	0.00	0.0	D 80-12
10.0	66.5	39.5	26.9	0.0	0.000	0.000	0.00	0.0	D 80-12
15.0	409.6	54.9	354.7	12.1	29.637	0.223	6.88	80.0	D 80-12
20.0	492.0	68.7	423.3	15.7	31.136	0.120	7.30	77.6	D 80-12
25.0	576.9	85.0	491.9	19.6	32.575	0.021	7.65	75.9	D 80-12
30.0	947.5	102.5	845.0	42.2	36.174	0.126	8.72	76.3	D 80-12
35.0	1085.2	120.6	964.5	55.3	37.422	0.220	9.02	77.3	D 80-12
40.0	1225.2	141.1	1084.1	75.9	38.146	0.166	9.30	78.5	D 80-12
43.0	1310.4	154.6	1155.8	94.1	38.629	0.103	9.43	79.7	D 80-12

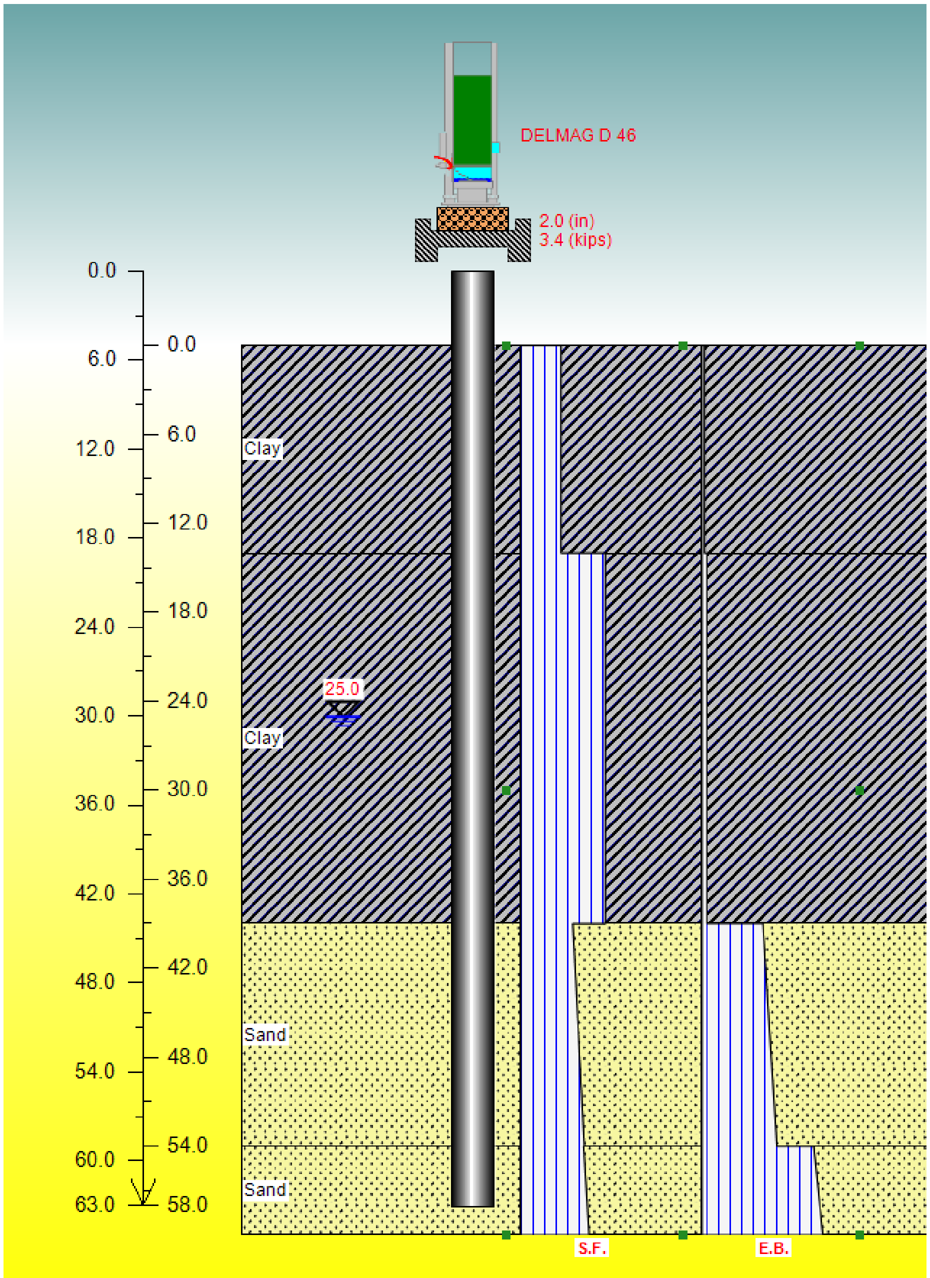
Total driving time: 30 minutes; Total Number of Blows: 1169 (starting at penetration 5.0 ft)

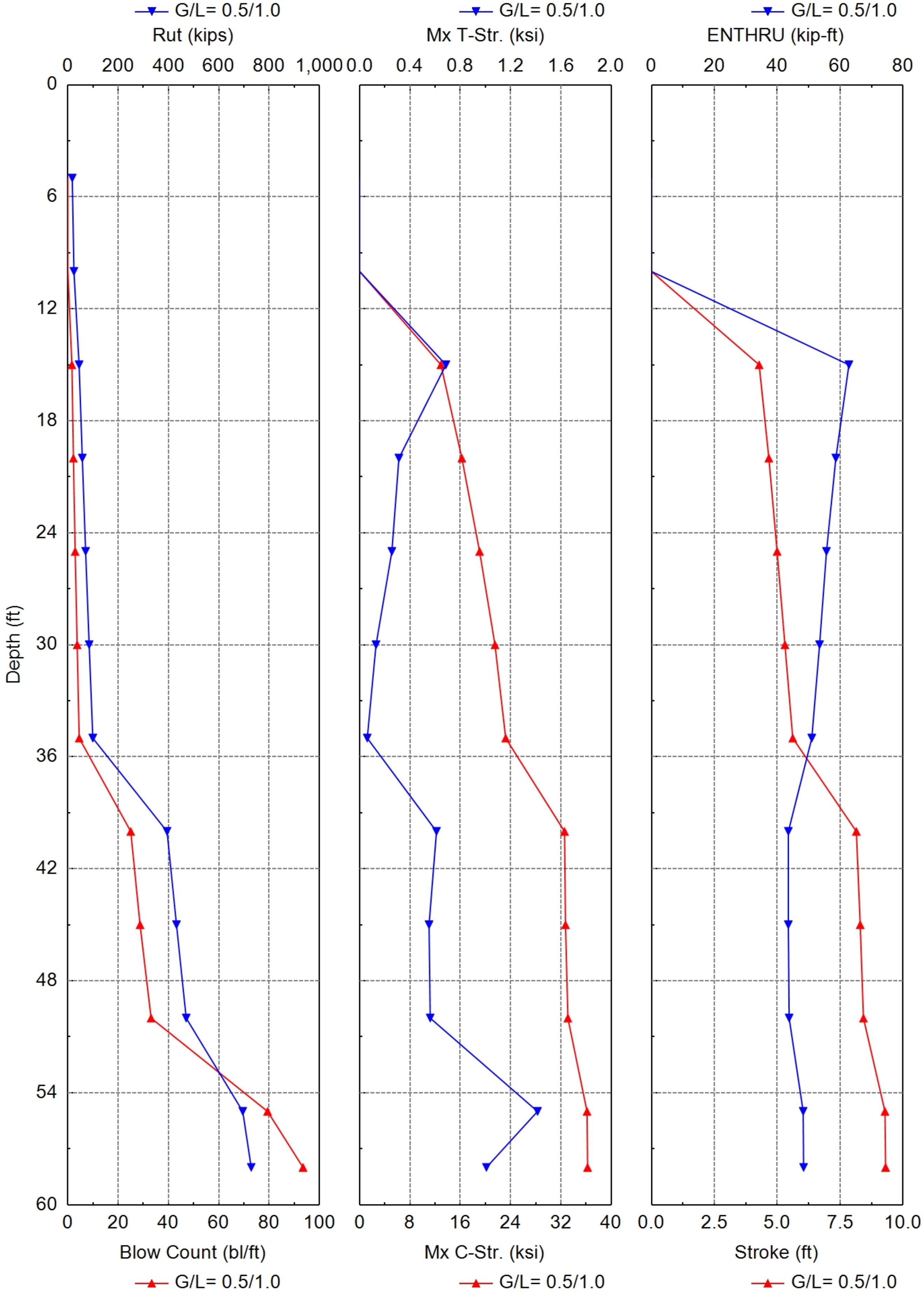
ArDOT 101124 Hwy 135 over Tyronza River

Bent 5

18-in-diameter Steel Shell Pile

Delmag D46



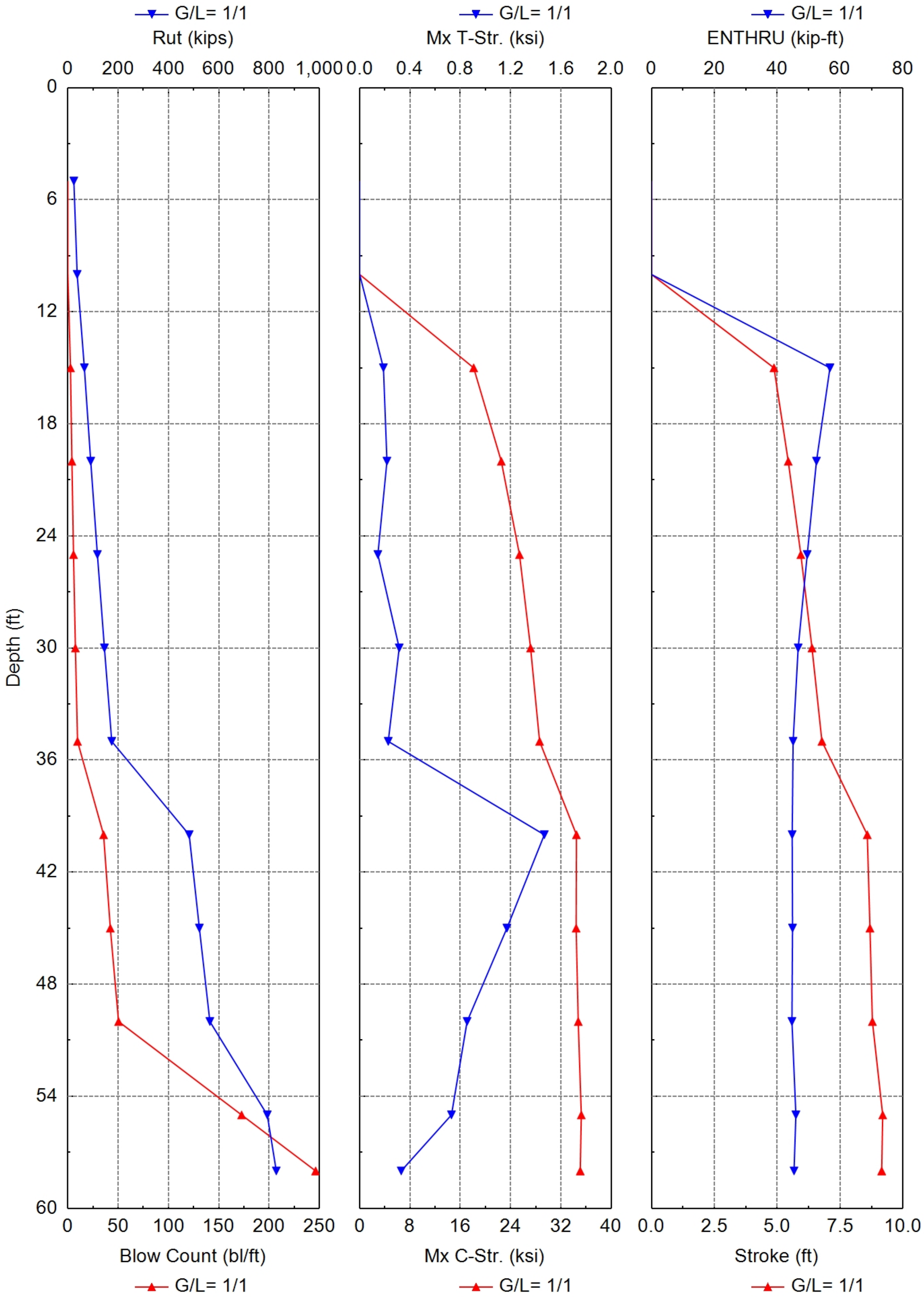

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
20
40
60
80
100
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Driveability Analysis Summary

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	17.4	6.3	11.1	0.0	0.000	0.000	0.00	0.0	D 46
10.0	24.0	12.9	11.1	0.0	0.000	0.000	0.00	0.0	D 46
15.0	44.7	20.8	23.9	1.6	12.924	0.686	4.28	62.8	D 46
20.0	57.3	33.5	23.9	2.2	16.252	0.312	4.67	58.6	D 46
25.0	70.6	46.8	23.9	2.9	19.071	0.256	4.99	55.7	D 46
30.0	84.6	60.7	23.9	3.7	21.519	0.130	5.31	53.5	D 46
35.0	99.2	75.3	23.9	4.5	23.222	0.060	5.62	51.0	D 46
40.0	394.3	90.3	304.1	25.0	32.579	0.611	8.14	43.5	D 46
45.0	431.9	105.2	326.7	28.7	32.730	0.552	8.30	43.5	D 46
50.0	470.5	121.2	349.3	33.1	33.112	0.561	8.43	43.8	D 46
55.0	695.3	138.2	557.1	79.4	36.158	1.414	9.28	48.2	D 46
58.0	728.9	148.8	580.1	93.5	36.246	1.008	9.31	48.4	D 46

Total driving time: 24 minutes; Total Number of Blows: 965 (starting at penetration 5.0 ft)


G/L= 1/1
Rut (kips)
0
200
400
600
800
1,000
G/L= 1/1
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 1/1
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
50
100
150
200
250
Blow Count (bl/ft)
G/L= 1/1
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 1/1
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 1/1

Driveability Analysis Summary
Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	23.8	12.6	11.1	0.0	0.000	0.000	0.00	0.0	D 46
10.0	37.0	25.8	11.1	0.0	0.000	0.000	0.00	0.0	D 46
15.0	65.5	41.7	23.9	2.6	18.129	0.189	4.87	56.8	D 46
20.0	90.8	67.0	23.9	4.0	22.522	0.217	5.44	52.5	D 46
25.0	117.4	93.6	23.9	5.6	25.418	0.145	5.94	49.5	D 46
30.0	145.3	121.4	23.9	7.5	27.179	0.314	6.38	46.7	D 46
35.0	174.5	150.6	23.9	9.6	28.616	0.227	6.78	45.0	D 46
40.0	482.4	178.3	304.1	35.5	34.476	1.468	8.58	44.8	D 46
45.0	522.9	196.2	326.7	42.2	34.429	1.171	8.69	44.9	D 46
50.0	564.7	215.3	349.3	50.4	34.762	0.854	8.78	44.7	D 46
55.0	792.9	235.7	557.1	172.7	35.246	0.730	9.19	45.9	D 46
58.0	828.7	248.5	580.1	246.3	35.083	0.330	9.15	45.3	D 46

Total driving time: 47 minutes; Total Number of Blows: 1847 (starting at penetration 5.0 ft)

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
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Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 13, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER DITCH NO. 1 (SITE 3)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
POINSETT COUNTY, ARKANSAS

INTRODUCTION

Submitted herewith are the final results of the geotechnical investigation performed for the Hwy. 135 over Ditch No. 1 replacement bridge in Poinsett County, Arkansas. This bridge is Site 3 of the ARDOT 110124 Hwy. 135 Strs & Apprs (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by the Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on May 9, 2023. This revised report supersedes the previous submittal of September 9, 2023.

We understand the replacement bridge will be a prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 150 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed east of the existing bridge. Site grading will include about 10 ft of fill. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, 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 in the Site 3 replacement bridge alignment were explored by drilling four (4) sample borings to 110- to 125-ft depth (Borings C1 to C4). The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
C1	122+15	CL	35.55741	-90.32252	224.4	110
C2	122+50	35 Rt	35.55754	-90.32242	224.9	110
C3	123+40	25 Lt	35.55778	-90.32259	217.9	110
C4	123+70	15 Lt	35.55786	-90.32255	223.9	125

The boring logs, presenting descriptions of the soil and rock strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 14. The centerline station and offset of the boring locations and approximate ground surface elevation, as surveyed, are also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 15.

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.

The borings were drilled with a truck-mounted SIMCO 2800 rotary-drilling rig and a track-mounted CME-55 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the appropriate energy conversion factor is shown on each boring log.

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

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

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 45 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 10 liquid and plastic (Atterberg) limit determinations and 31 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The location of 101124 Site 3 is on Hwy. 135 where the Ditch No. 1 channel crosses the highway approximately 4300 ft south of CR 86 in Poinsett County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. The channel at this location is narrow with well-defined banks. The banks are steep and lined with grass, variable sparse to thick underbrush, and occasional trees. The project locale is primarily agricultural land consisting of woods or large, flat fields and occasional residential houses. The existing two-lane roadway is on embankment. The existing pavements are in very poor condition. Surface drainage along the roadway is poor to fair and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent Alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends through exposures of Quaternary-aged Alluvium. The Alluvium is comprised of recent stream-deposited alluvial sediments which include gravel, sand, silt, clay and mixtures of all components. The thickness of the Alluvial deposits is variable. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

Seismic Conditions

In light of the results of the borings and the surface geology, a Seismic Site Class D (stiff soil profile) is considered applicable to the bridge location at Site 3 with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[2]. Given the location and AASHTO code-based values, preliminarily recommended seismic parameters are summarized below.

- Seismic Site Class D
- 1.0-sec period spectral acceleration coefficient (S_1) = 0.513
- Site amplification factor at 1.0 second (F_v) = 1.5
- 1.0-sec period spectral acceleration coefficient (S_{D1}) = 0.770
- Acceleration for a short (0.2 sec) period (S_S) = 1.815
- Site amplification factor for short period (F_a) = 1.0
- Design acceleration for a short (0.2 sec) period (S_S) = 1.815
- Peak ground acceleration (PGA) = 1.014
- Site amplification factor at PGA (F_{PGA}) = 1.0
- A_s = 1.014

Utilizing these parameters, AASHTO LRFD Seismic Bridge Design Specifications indicate that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Site 3 location of the Hwy. 135 bridge over Ditch 1.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 1.014 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Appendix D as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the analyses results are shown on the generalized subsurface profile also provided in Appendix D. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix D.

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

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

Subsurface Conditions

Based on the results of the borings, the surface and near-surface soils to 18- to 33-ft are comprised of brown, gray, tan, and reddish brown very loose to medium dense silty fine sand (SM and SP-SM) and clayey fine sand (SC and SC-SM) with interbedded very soft to stiff clay (CH) and sandy clay (CL) layers. The silty, clayey sand and clay/sandy clay exhibit low to moderate relative density or shear strength and moderate to high compressibility. The granular soils typically classify as A-3, A-4, and A-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to fair subgrade support for pavement structures.

The silty and clayey surface soil units are underlain below 18 to 33 ft to in excess of the completion depth of the borings by medium dense to very dense grayish tan and brownish gray fine to medium sand strata (SP and SP-SM). Some coarse sand, sandy clay seams, organic inclusions, and fine gravel are present at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was not encountered within the range of dry-auger drilling in the borings in April and May 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the ditch and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 3 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 16-in.-diameter steel shell piles are planned for bridge ends and 24-in.-diameter steel shell piles are planned for the interior bents. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix E. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for "static" conditions prior to a seismic event, with no liquefaction, and full soil shear strength is mobilized for the foundation soils. For the case where liquefaction occurs, the "end of earthquake" condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects. The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix F.

End Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 4) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 23 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a 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 0.507. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 214 to El 205 was assumed.

Stability analyses results are summarized and presented graphically in Appendix G. The results of the stability analyses indicate that plan configurations of the embankment end and side slopes are acceptable with respect to stability of all loading conditions evaluated. This includes stability in seismic loading. A suitable factor of safety against lateral flow was calculated for all cases.

Subgrade Support

It is understood that "standard" pavement sections for the approach roads will be developed by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-4 and A-6. These classifications correlate with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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

---

[4] Slope/W 2020; GEO-SLOPE International; 2020.

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, localized undercuts or improvement depths on the order of 2 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. in cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, localized undercutting could be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 6 to 13 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix H. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

General 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow. An example special provision for cohesive embankment fill is provided in Appendix I.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until the retaining wall, embankments, and bridge work is completed.

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

Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered. Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[5]. In the driveability analyses, the steel shell piles were assumed to be driven from the plan cap bottom elevation or existing grade. Graphical and tabulated results of these analyses are provided in Appendix J.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 66 ft-kips per blow for driving the steel shell piles at the end bents. For the intermediate bents, we recommend a hammer system capable of delivering at least 122 ft-kips per blow for driving the steel shell piles. A specific review and analysis of the pile-hammer system

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[5] GRLWEAP 2014; Pile Dynamics, Inc.

proposed by the Contractor should be performed by the Engineer or Department prior to hammer acceptance and start of pile installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. 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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment 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 attachments are included and complete this submittal.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 14	Boring Logs
Plate 15	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Liquefaction Analysis Results
Appendix E	Nominal Pile Capacity Curves
Appendix F	Lateral Load Parameters
Appendix G	Results of Stability Analyses
Appendix H	Example SP – Woven Geotextile
Appendix I	Example SP – Cohesive Embankment Fill Special Provision
Appendix J	Driveability Analysis 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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

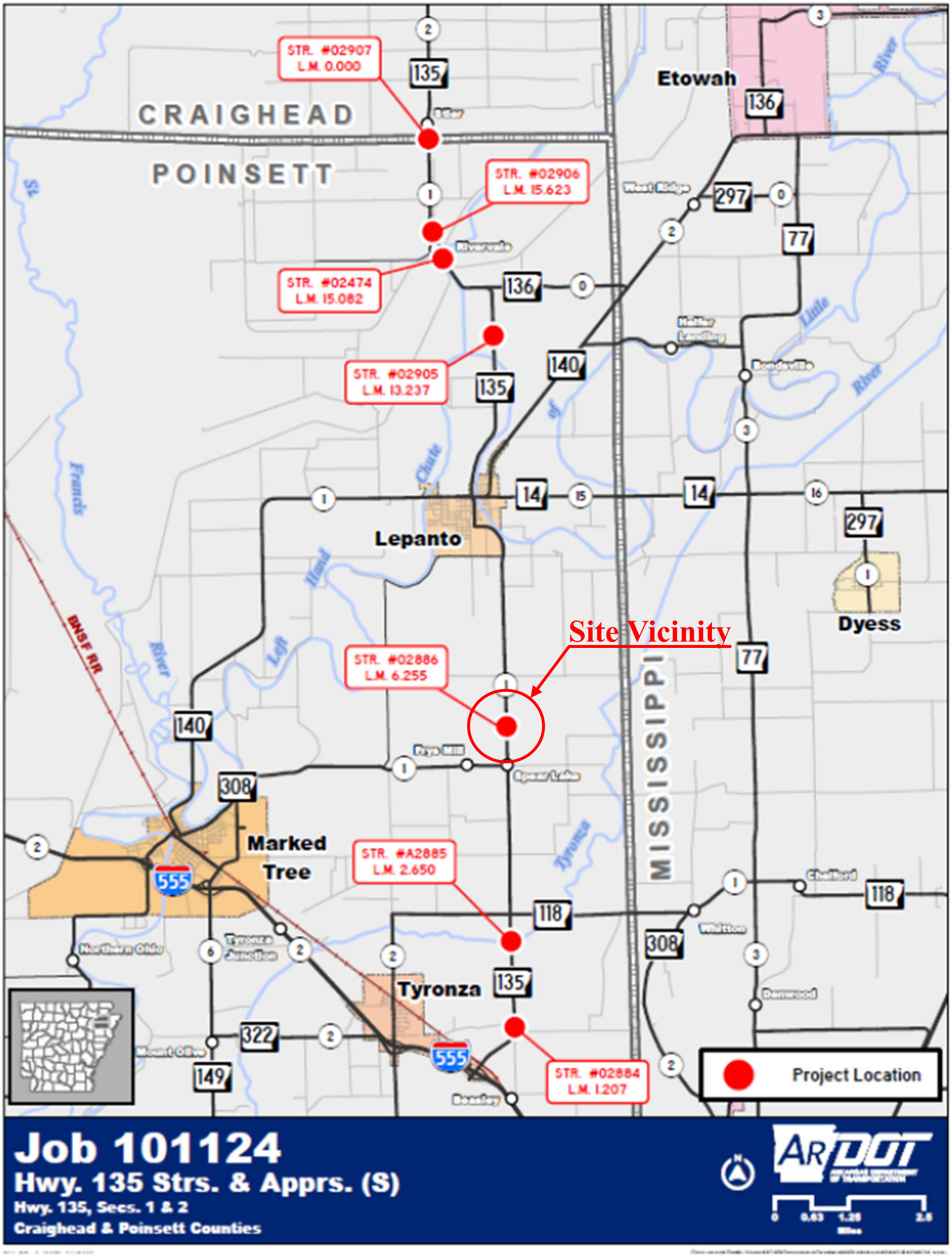
ME Wyatt

Mark E. Wyatt, P.E.
President

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

VMS/MEW:jw

Copies submitted:	Arkansas Department of Transportation		
	Attn:	Ms. Jessica Jackson, P.E.	(1-email)
	Attn:	Mr. Paul Tierney	(1-email)
	Attn:	Mr. Yongsheng Zhao, Ph.D., P.E.	(1-email)
	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

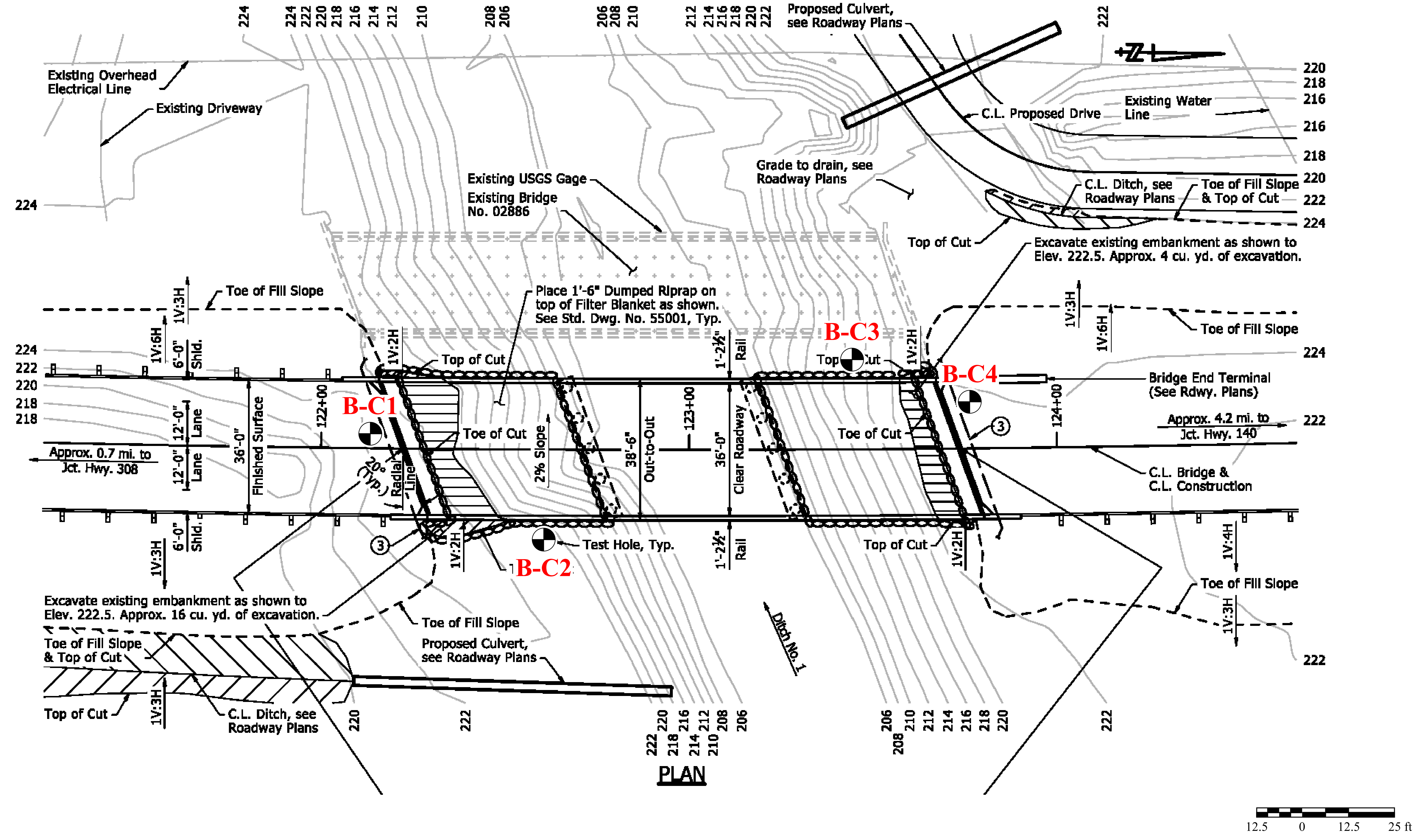


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Ditch No. 1
(Site 3/Bridge C)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORINGS
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Scale: As shown	Job No. 23-031	PLATE 2

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LOG OF BORING NO. C1

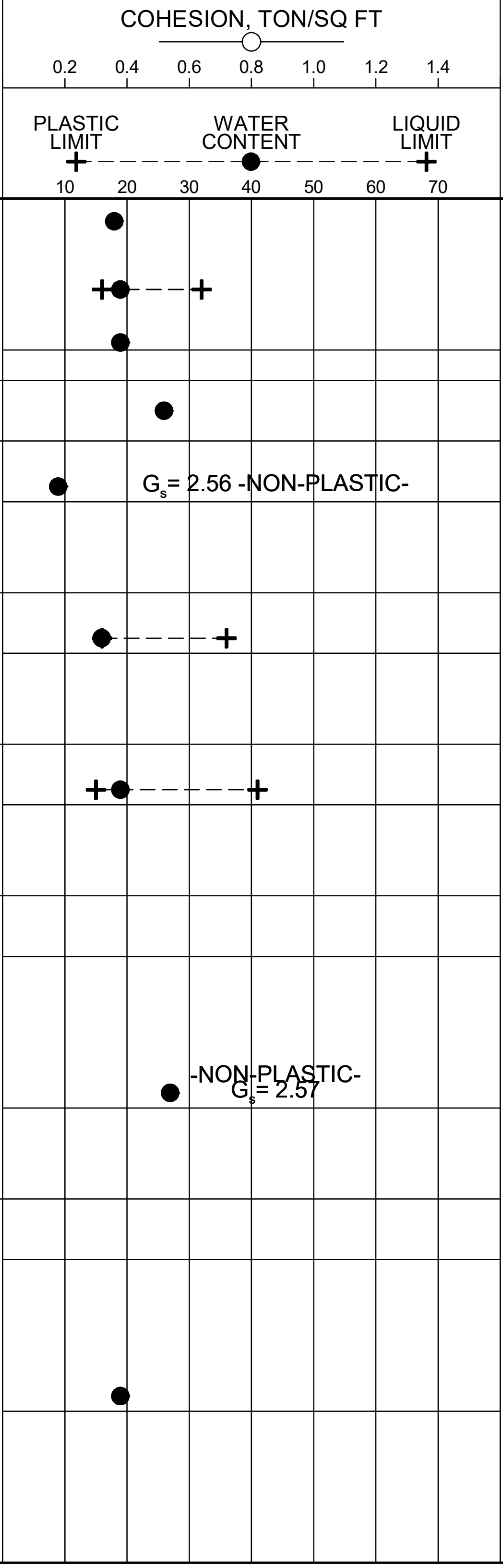
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 122+15, CL

SURF. EL: 224.4

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Loose brown clayey fine sand (SC)	9		
		7		32
5		7		
	Stiff gray, grayish brown and reddish tan clay, slightly sandy (CH) w/ferrous stains	26		
10	Medium dense gray, tan and reddish brown silty fine sand (SM)	23		38
15	Medium dense gray and tan clayey fine sand (SC)	23		
20	Stiff brownish gray and reddish tan fine sandy clay (CL) w/ferrous stains and organic inclusions	24		53
25	Medium dense brown and tan silty fine sand (SM)	37		
30	- more silt below 28 ft	21		38
35	Dense grayish tan fine to medium sand, slightly silty (SP-SM)	64		
40	- brownish gray with occasional organic inclusions below 38 ft	51		5
		44		



COMPLETION DEPTH: 110.0 ft
DATE: 4-26-23

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 4/20/2023

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

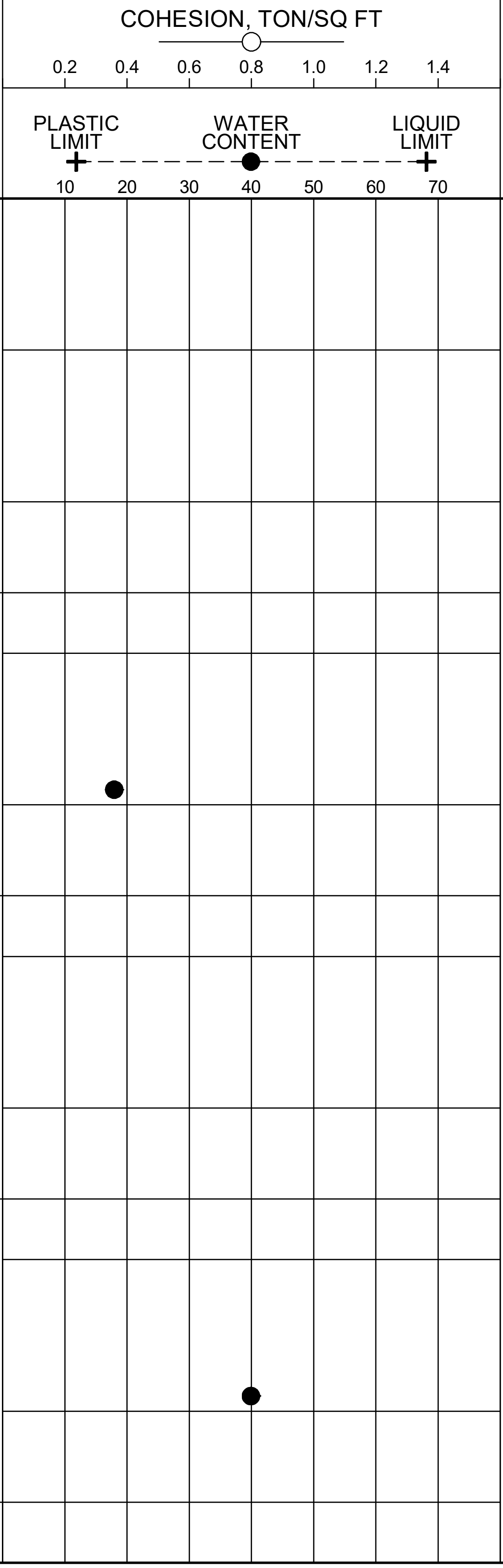
LOG OF BORING NO. C1

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 122+15, CL

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50		50		
55		55		
60	Dense grayish tan fine to medium sand (SP) w/trace coarse fine sand and gravel	48		
65	- with less coarse sand below 63 ft	72		4
70	Dense to very dense brownish gray fine sand (SP) w/organic inclusions	84		
75	- with fewer organic inclusions below 73 ft	99		
80	Dense grayish tan fine to coarse sand, slightly silty (SP-SM) w/occasional clay pockets	55		
85	- with occasional fine sandy clay pockets and seams below 83 ft	50		5
	Dense to very dense brownish gray fine sand, slightly silty (SP-SM)	87		



COMPLETION DEPTH: 110.0 ft
DATE: 4-26-23

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 4/20/2023

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Consulting Engineers

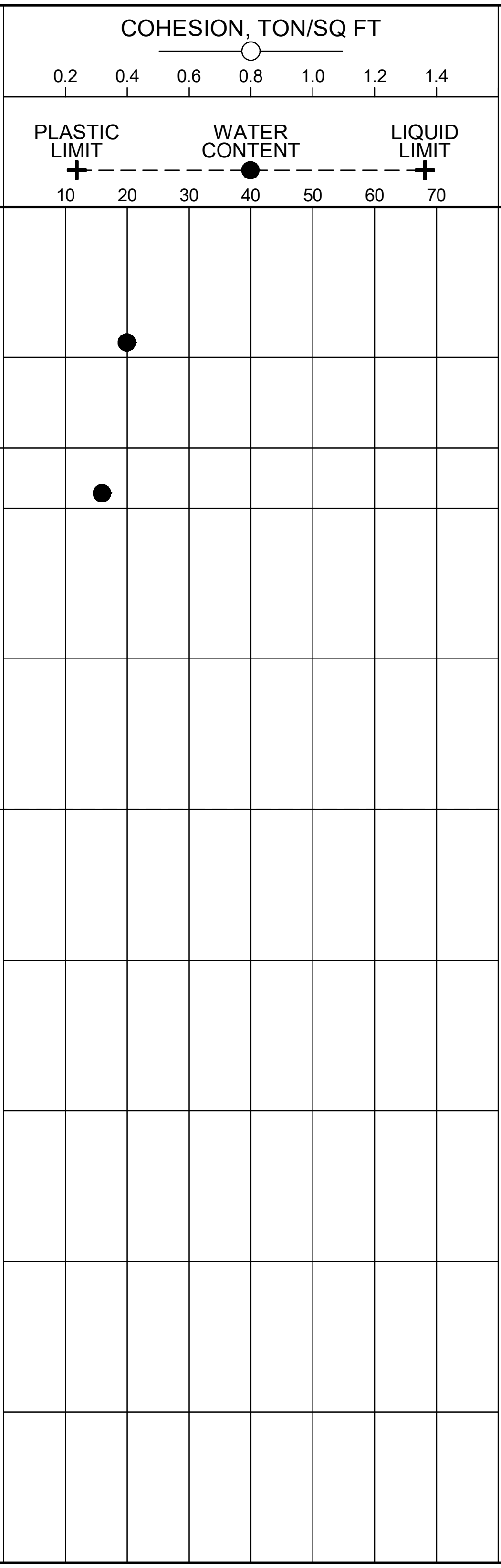
LOG OF BORING NO. C1

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 122+15, CL

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
			w/occasional organic inclusions			
			- grayish tan below 93 ft			
95		☒		84		6
			Dense grayish tan fine to medium sand, slightly silty (SP-SM)			
100		☒		58		5
105						
110		☒		61		
			NOTE: Drilled with CME-55 ECF= 1.42			
115						
120						
125						
130						



COMPLETION DEPTH: 110.0 ft
DATE: 4-26-23

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 4/20/2023

LGBNEW 23-031 BRIDGE C.GPJ 7-26-23

23-031

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

LOG OF BORING NO. C2

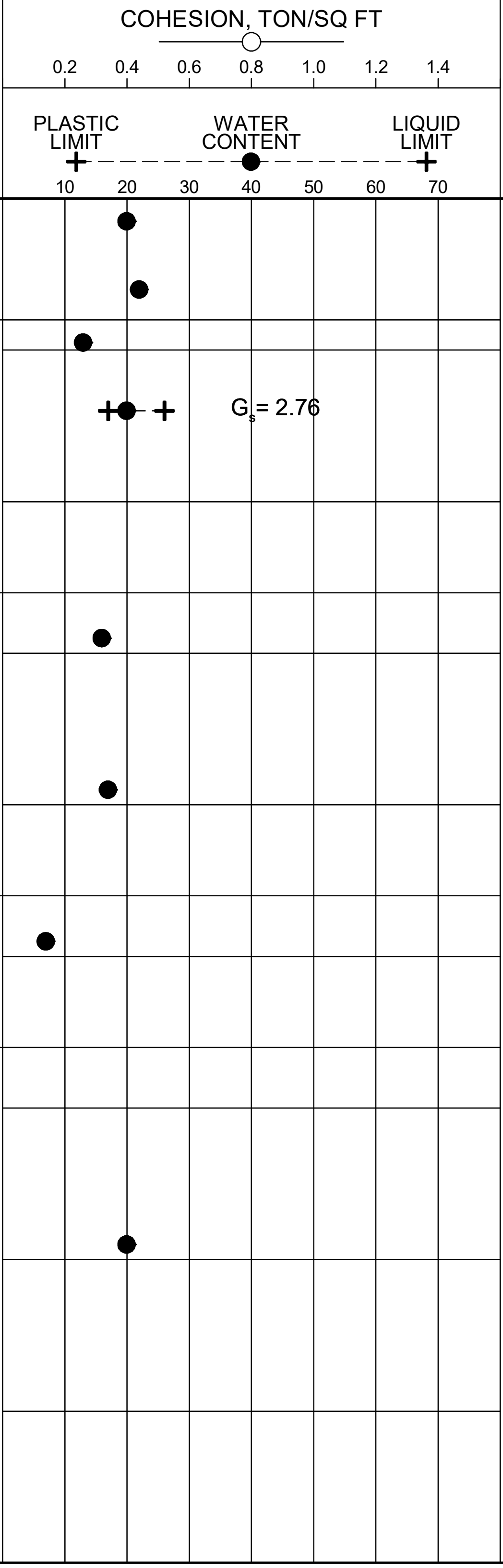
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 122+50, 35 ft Rt

SURF. EL: 224.9

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very soft to soft brown fine sandy clay (CL)	6		
	- firm with more sand below 2 ft	10		
5	Very loose to loose tan and brown silty fine sand (SM)	6		
	- loose from 6 to 8 ft - with fine sandy clay seams and layers below 6 ft	7		44
10	- medium dense below 8 ft	16		
15	Medium dense gray and reddish brown clayey fine sand (SC) w/ferrous stains	16		28
20		26		
25	Medium dense brown fine sand, slightly silty (SP-SM)	28		10
30	Medium dense grayish tan fine sand, slightly silty (SP-SM)	40		
35		41		8
40	- medium dense to dense with occasional organic inclusions at 38 to 43 ft	43		
	- medium dense from 43 to 48 ft	41		



COMPLETION DEPTH: 110.0 ft
DATE: 4-19-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/10/2023

LGBNEW 23-031 BRIDGE C.GPJ 7-26-23

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

LOG OF BORING NO. C2

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash
LOCATION: Approx Sta 122+50, 35 ft Rt

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

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50	- dense from 48 to 53 ft	60		
55	- medium dense below 53 ft	31		
60	Medium dense grayish tan fine to medium sand, slightly silty (SP-SM) w/occasional organic inclusions and trace fine gravel	26		
65	- dense below 63 ft	61		6
70	Dense brownish gray fine sand, slightly silty (SP-SM) w/occasional clay pockets and organic inclusions	67		
75		64		
80		62		
85	Medium dense to dense grayish tan fine to medium sand (SP) w/trace fine to coarse gravel	43		3
	Dense brownish gray fine sand (SP) w/occasional organic	72		

COMPLETION DEPTH: 110.0 ft
DATE: 4-19-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/10/2023

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

L O G O F B O R I N G N O. C2

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 122+50, 35 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + — WATER CONTENT ● — LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
			inclusions				
95		☒		48			
			- medium dense below 98 ft				
100		☒		33			
105							
			Medium dense brownish gray fine to coarse sand (SP) w/trace fine gravel and occasional organic inclusions				
110		☒	Medium dense grayish brown fine to medium sand (SP) w/numerous organic inclusions	33			
			NOTE: Drilled with CME-55 ECF= 1.42				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 4-19-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/10/2023

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

LOG OF BORING NO. C3

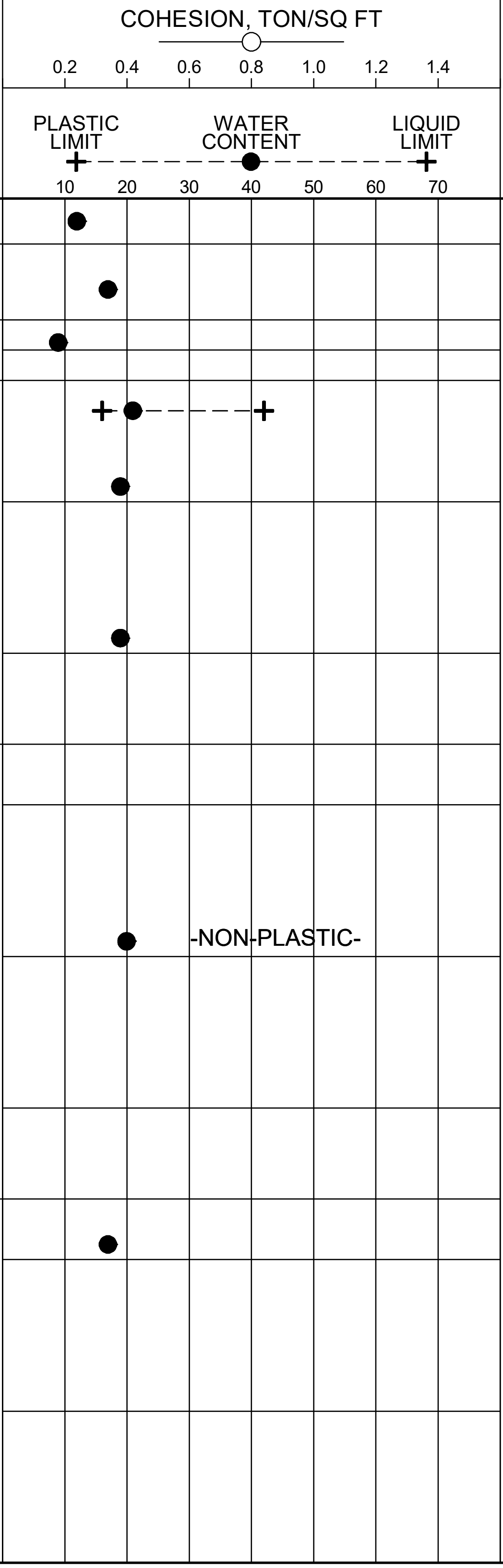
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 123+40, 25 ft Lt

SURF. EL: 217.9

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Loose brown clayey fine sand (SC) (fill)	7		
	Very loose to loose tan and gray clayey fine sand, silty (SC-SM) w/occasional decayed organics	6		
5	Loose tan, reddish tan and brownish gray fine sand, slightly silty (SP-SM)	7		
	Firm gray, brown and reddish tan fine sandy clay (CL) w/occasional ferrous nodules and stains	10		53
10		11		
	- stiff below 13 ft			
15		23		
	Medium dense brown fine sand, slightly silty (SP-SM)			
20		23		
25		37		7
	- dense below 28 ft			
30		37		
35	Medium dense grayish tan fine to medium sand, slightly silty (SP-SM)	48		7
	- dense at 38 to 43 ft			
40		41		
	- medium dense below 43 ft			
		57		



COMPLETION DEPTH: 110.0 ft
DATE: 5-4-23

DEPTH TO WATER
IN BORING: Dry to 10 ft
DATE: 5/2/2023

23-031

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

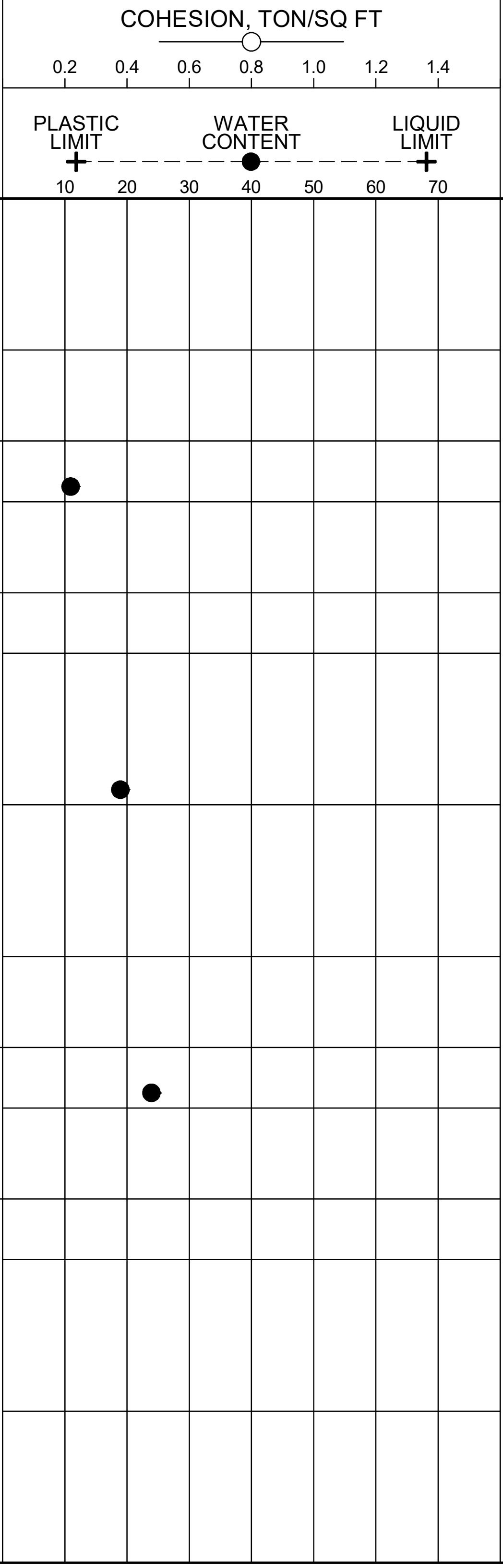
LOG OF BORING NO. C3

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 123+40, 25 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50	- with organic inclusions below 48 ft	41		
55	Medium dense grayish tan fine to coarse sand, slightly silty (SP-SM) w/a little fine to coarse gravel	34		7
60	Medium dense to dense grayish tan fine to medium sand, slightly silty (SP-SM)	43		
65	- dense with organic inclusions below 63 ft	65		5
70		57		
75	Dense to very dense gray silty fine sand (SM) w/occasional organic inclusions	105		18
80	Dense gray fine sand, slightly silty (SP-SM) w/occasional organic inclusions	45		
85		57		
		71		



COMPLETION DEPTH: 110.0 ft
DATE: 5-4-23

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 5/2/2023

23-031

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

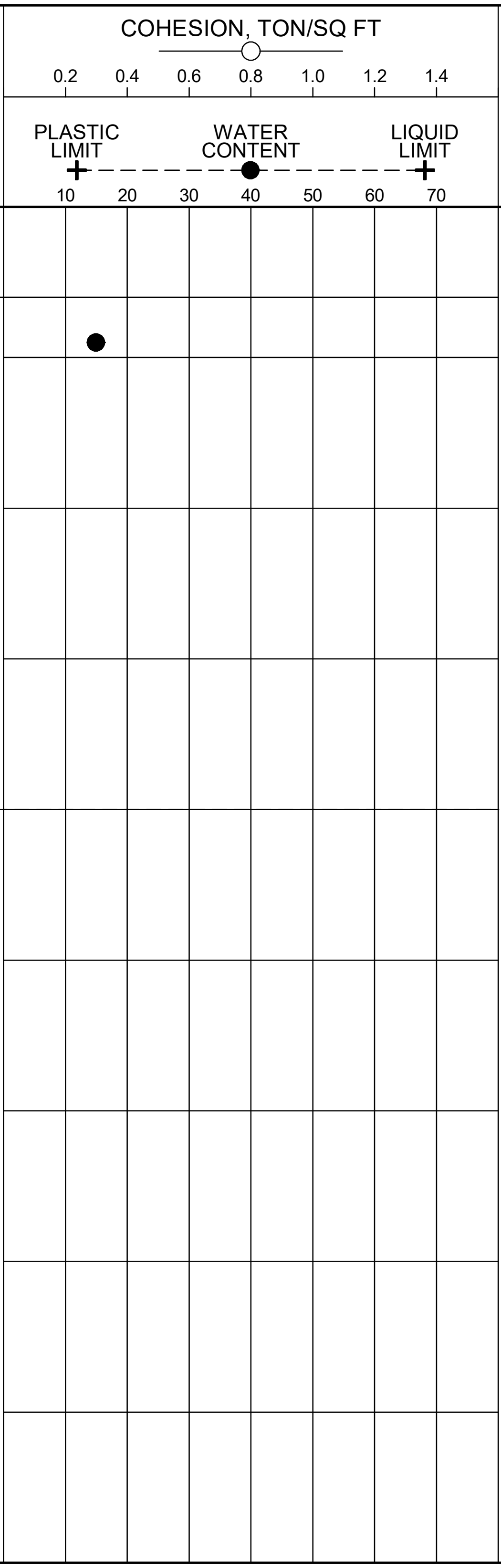
LOG OF BORING NO. C3

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 123+40, 25 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
95		☒	Dense to very dense grayish tan fine to medium sand, slightly silty (SP-SM) w/occasional organic inclusions and trace fine gravel	74		7
100		☒		92		
105						
110		☒		107		
			NOTE: Drilled with CME-55 ECF= 1.42			
115						
120						
125						
130						



COMPLETION DEPTH: 110.0 ft
DATE: 5-4-23

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 5/2/2023

LGBNEW 23-031 BRIDGE C.GPJ 7-26-23

23-031

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

LOG OF BORING NO. C4

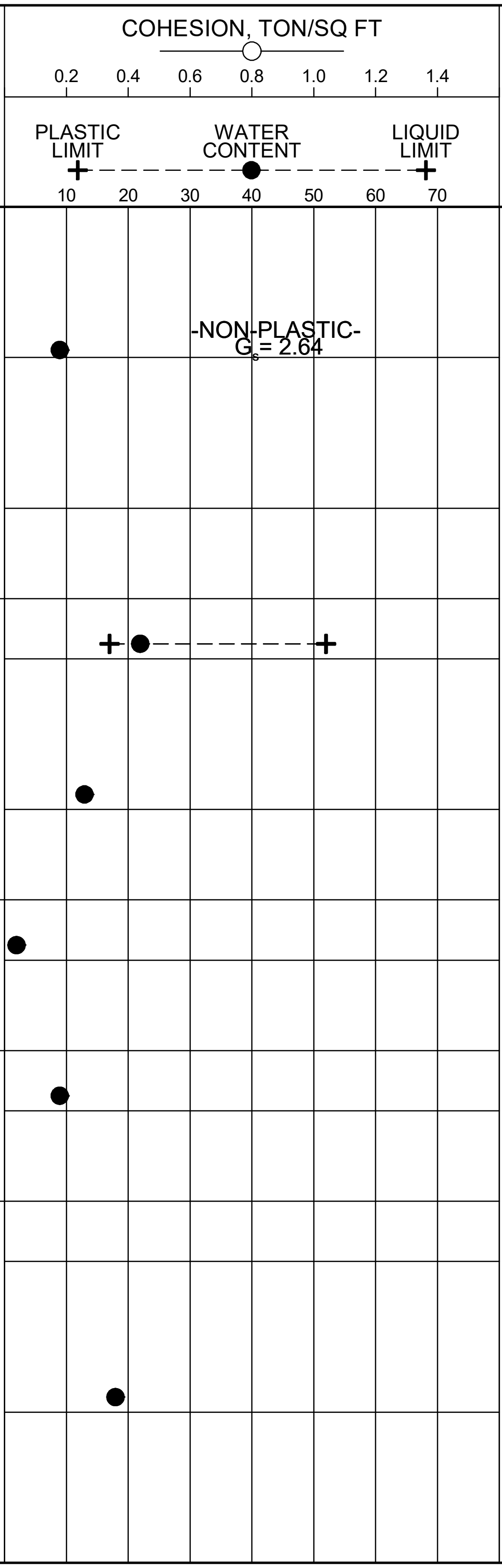
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 123+70, 15 ft Lt

SURF. EL: 223.9

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Loose tan and brown fine sand, slightly silty (SP-SM) w/fine sandy clay seams	6		
		5		
5		8		8
	- with occasional organic inclusions below 6 ft - medium dense below 8 ft	8		
10		15		
15	Medium dense gray and reddish tan clayey fine sand (SC) w/ferrous stains	17		49
20	- silty below 18 ft	32		
25	Medium dense tan fine sand (SP)	26		3
30	Dense brown silty fine sand (SM)	55		22
35	Dense brownish gray fine to medium sand (SP)	45		
40		71		2
		57		



COMPLETION DEPTH: 125.0 ft
DATE: 4-27-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/13/2023

LGBNEW 23-031 BRIDGE C.GPJ 7-26-23

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

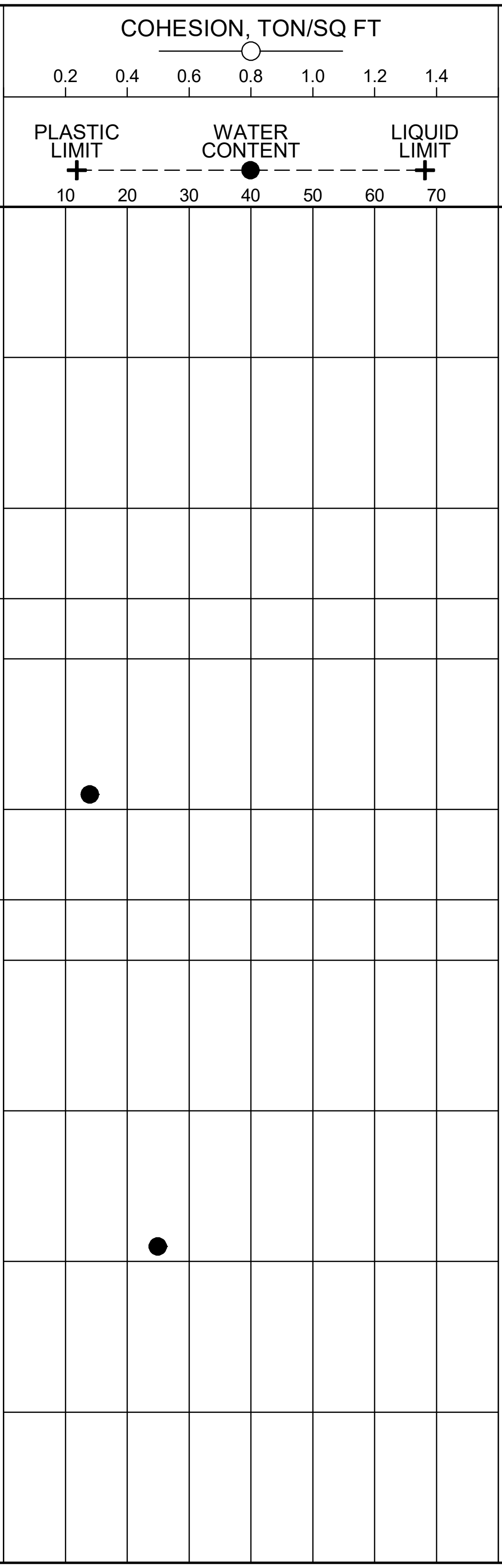
LOG OF BORING NO. C4

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 123+70, 15 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50		48		
55	- with occasional organic inclusions below 53 ft	37		
60	Dense brownish gray fine to coarse sand (SP)	61		
65	- tan with less coarse sand and trace fine gravel below 63 ft	37		4
70	Dense brownish gray silty fine sand (SM) w/occasional organic inclusions	42		
75		37		
80		68		14
85		58		
		74		



COMPLETION DEPTH: 125.0 ft
DATE: 4-27-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/13/2023

23-031

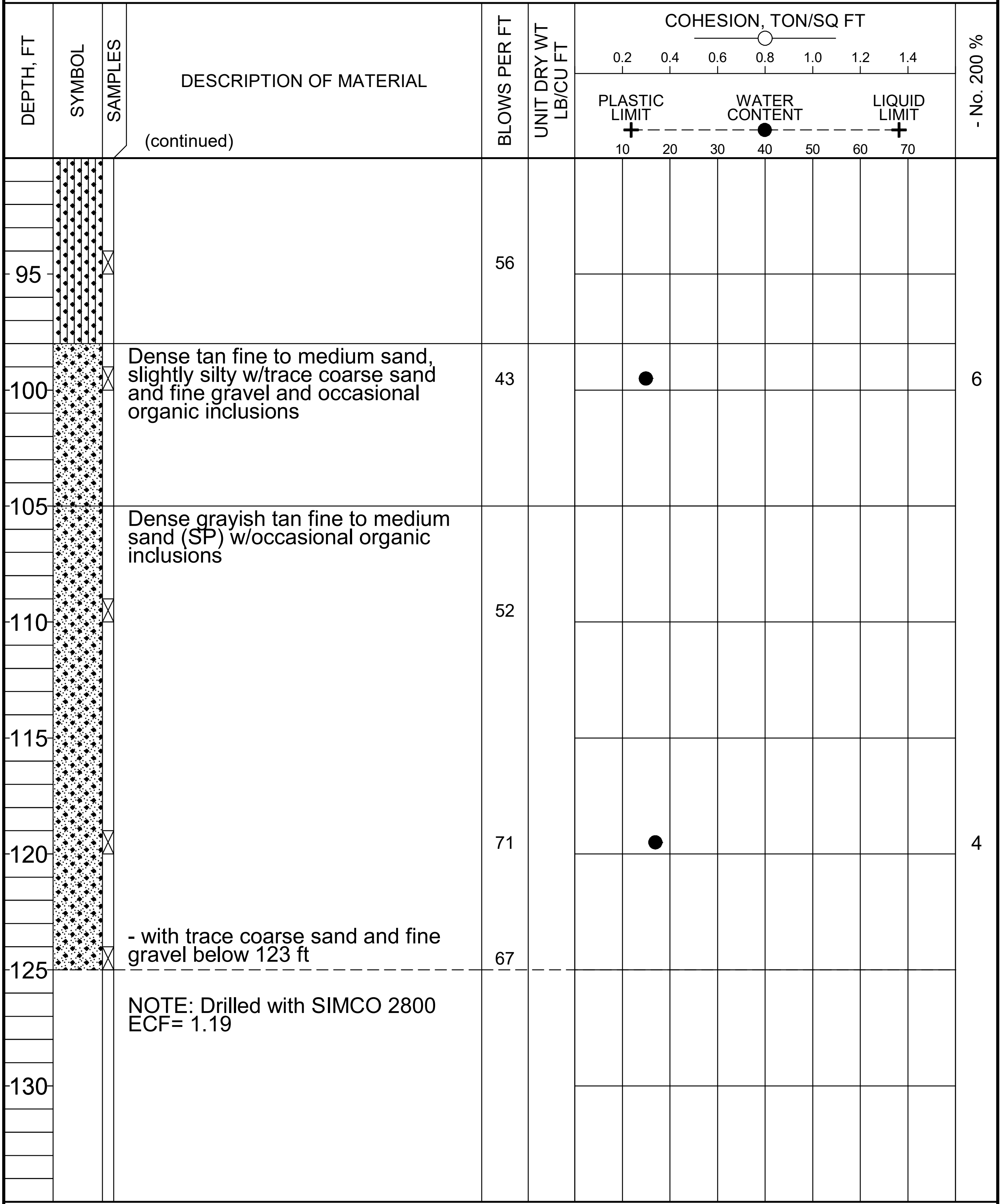
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

L O G O F B O R I N G N O. C4

101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 123+70, 15 ft Lt



DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
95		☒		56		
100		☒	Dense tan fine to medium sand, slightly silty w/trace coarse sand and fine gravel and occasional organic inclusions	43		6
105			Dense grayish tan fine to medium sand (SP) w/occasional organic inclusions			
110		☒		52		
120		☒		71		4
125		☒	- with trace coarse sand and fine gravel below 123 ft	67		

NOTE: Drilled with SIMCO 2800
ECF= 1.19

COMPLETION DEPTH: 125.0 ft
DATE: 4-27-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 4/13/2023

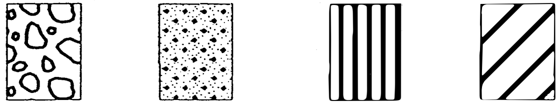
LGBNEW 23-031 BRIDGE C.GPJ 7-26-23

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

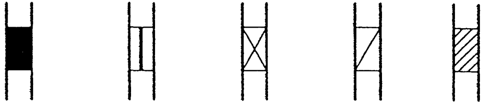
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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.

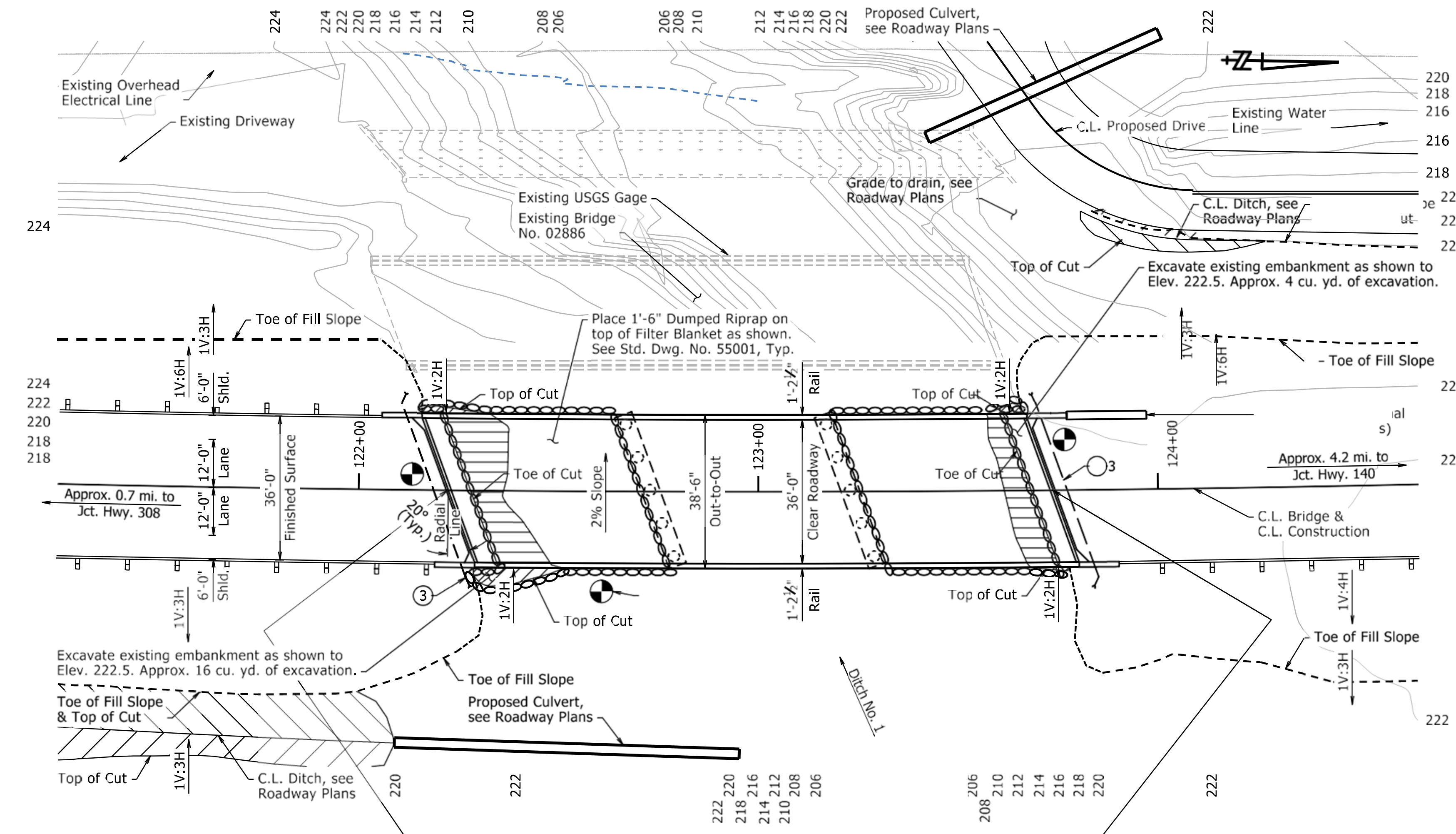
Terms used on this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No.3-357, Waterways Experiment Station, March 1953

KEY 9-26-02

APPENDIX A

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	98	183
		XXXXX		LAYOUT		XXXXX

For R/W Data, See Roadway Plans.

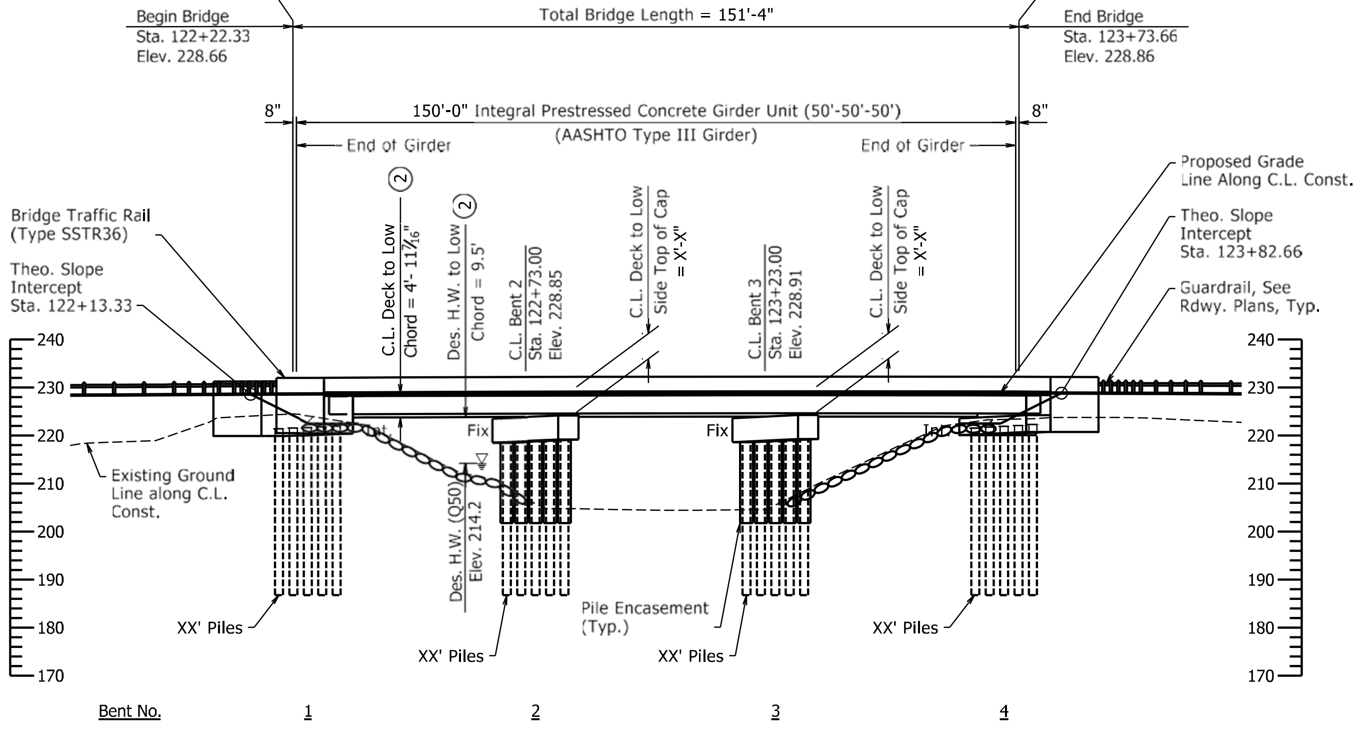


PLAN

Begin Bridge
Sta. 122+22.33
Elev. 228.66

Total Bridge Length = 151'-4"

End Bridge
Sta. 123+73.66
Elev. 228.86



ELEVATION

Note: For Soil Borings, see Dwg. No. XXXXX.

Notes:

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

③ 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".

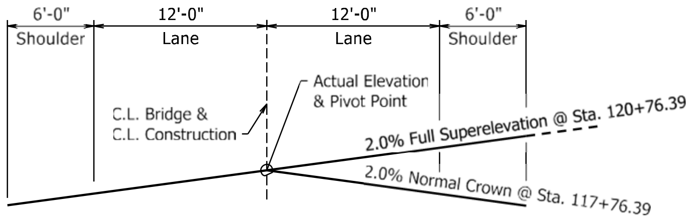
HORIZONTAL CURVE DATA

Along C.L. Const.

P.I. = 122+98.51
△ = 7°45'15" LT.
D = 0°45'00"
T = 517.74'
L = 1033.89'
P.C. = 117+80.77
P.T. = 128+14.66

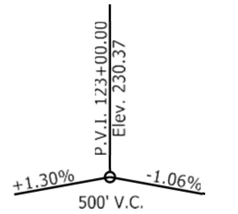
Note: C.L. Construction is on a 00°45'00" curve left. The longitudinal lines of the deck shall be constructed on curves concentric with C.L. Construction. C.L. Girders are parallel to Bridge Chord between the Begin Bridge Station and the End Bridge Station. Bents shall be constructed along lines skewed from perpendicular lines to the C.L. of Bridge Chord. Skew is measured at C.L. Bent for Intermediate Bents and at bridge ends for End Bents.

Note: Stations shown are along C.L. Construction. Elevations shown are actual top of deck elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on actual top of deck elevation at C.L. Bridge.



SUPERELEVATION TRANSITION SKETCH

(Looking Ahead)



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along
C.L. Construction

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

SEISMIC ZONE: XX S_{D1}= XX SITE CLASS: XX

SEISMIC OPERATIONAL CLASS: OTHER

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 6,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

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

STEEL SHELL PILING: Piling in Bents 1 and 4 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. Piling in Bents 2 and 3 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. All piling shall be driven with an approved air, steam, or diesel hammer to a minimum tip elevation of ______ or lower at Bents 1 and 4 and to a minimum tip elevation of ______ or lower at Bents 2 and 3. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (__" Dia.)".

PREBORING: Preboring is required for all piling at Bents 1 and 4. Prebored holes shall have a diameter 6" greater than the diameter of the pile for a depth of 10' below the bottom of the cap. The void space around the pile after completion of driving shall be backfilled with sand or pea gravel. The Contractor shall be responsible for keeping prebored holes free of debris prior to backfilling which may require the use of temporary casings or other approved methods. Any related cost for backfilling and temporary casing will not be paid for directly, but shall be considered subsidiary to the item "Preboring".

For Additional General Notes, see Dwg. No. XXXXX.

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	① NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	1170	213.6	214.2
BASE	100	1280	213.9	214.5
EXTREME	500	1520	214.6	215.3
OVERTOPPING	>500	---	---	---

① Unconstricted water surface elevation without structure or roadway approaches.

② Proposed Low Bridge Chord Elev. = 223.7 feet at Station 122+21.66

100 yr. backwater elevation for existing structure = 214.5 feet
Drainage Area = 18.6 sq. miles
Historical H.W. Elev. = N/A

SITE 3/BRIDGE C

PRELIMINARY
SUBJECT TO
REVISION

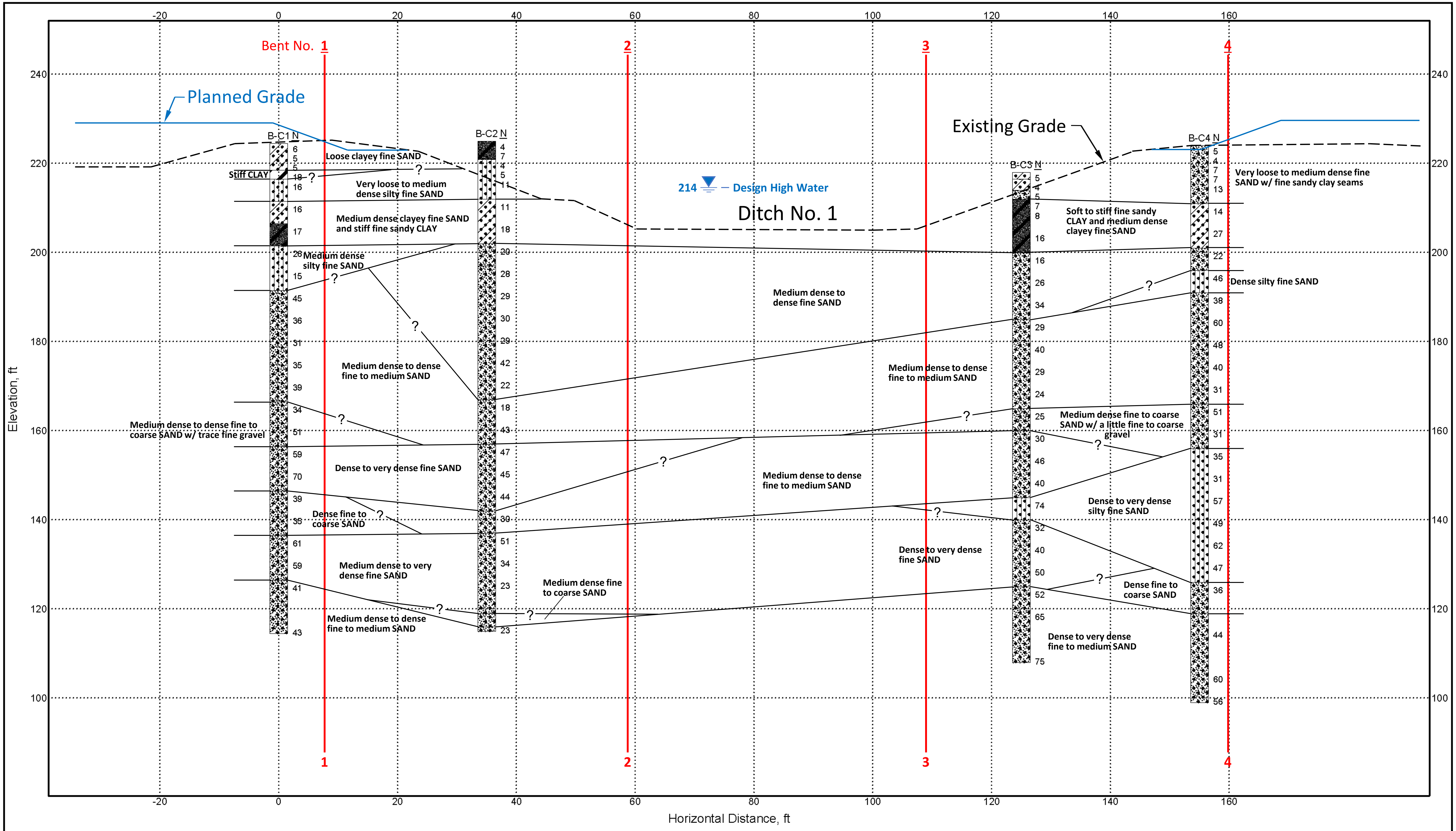
SHEET 1 OF 1
LAYOUT OF BRIDGE
HWY. 135 OVER DITCH NO. 1
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: MLC DATE: 11-16-22 FILENAME: b101124x3_l1.dgn
CHECKED BY: CAW DATE: 12-06-22 SCALE: 1" = 20'
DESIGNED BY: MLC DATE: 11-02-22
BRIDGE NO. XXXXX DRAWING NO. XXXXX

February 2023

USER: bc5100
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x3_l1.dgn SCALE: 1:40
PLOTTED: 1/26/2023 15:35

APPENDIX B



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

SCALE:
1'' = 15' Horizontal
1'' = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Ditch No. 1 (Site 3)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
C1	2.5-3.5	19	32	16	16	---	---	---	---	100	---	---	32	SC	A-6
C1	9-10	9	NON-PLASTIC			100	100	100	100	100	100	100	38	SM	A-4
C1	14-15	16	36	16	20	---	---	---	---	---	---	---	---	SC	A-6
C1	19-20	19	41	15	26	---	---	---	---	100	---	---	53	CL	A-7-6
C1	29-30	27	NON-PLASTIC			100	100	100	100	100	100	100	38	SM	A-4
C1	39-40	19	---	---	---	100	100	100	100	100	100	78	5	SM-SP	A-3
C1	64-65	18	---	---	---	100	100	100	99	96	93	30	4	SW	A-1-b
C1	84-85	40	---	---	---	100	100	100	98	95	65	16	4	SW	A-1-b
C1	94-95	20	---	---	---	100	100	100	100	95	94	87	6	SM-SP	A-3
C1	99-100	16	---	---	---	100	100	100	99	98	96	32	5	SM-SW	A-1-b
C2	6.5-7.5	20	26	17	9	100	100	100	100	100	100	99	44	SC	A-4
C2	14-15	16	---	---	---	100	100	100	100	100	99	92	28	SC	A-6
C2	24-25	7	---	---	---	---	---	---	---	100	---	---	10	SM-SP	A-3
C2	34-35	20	---	---	---	100	100	100	100	100	100	96	8	SM-SP	A-3
C2	64-65	12	---	---	---	100	100	100	92	87	81	27	6	SM-SW	A-1-b
C2	84-85	14	---	---	---	100	100	94	88	84	79	16	3	SW	A-1-b
C3	6.5-7.5	21	42	16	26	---	---	---	---	100	---	---	53	CL	A-7-6
C3	24-25	20	NON-PLASTIC			---	---	---	---	100	---	---	7	SM-SP	A-3
C3	34-35	17	---	---	---	100	100	100	99	98	97	59	7	SM-SP	A-3
C3	54-55	11	---	---	---	100	83	83	81	74	66	19	7	SM-SW	A-1-b
C3	64-65	19	---	---	---	100	100	100	99	98	97	51	5	SM-SP	A-3
C3	74-75	24	---	---	---	---	---	---	---	100	---	---	18	SM	A-2-4
C3	94-95	15	---	---	---	100	100	100	100	98	94	33	7	SM-SW	A-1-b

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS

PLATE

SUMMARY of CLASSIFICATION TEST RESULTS

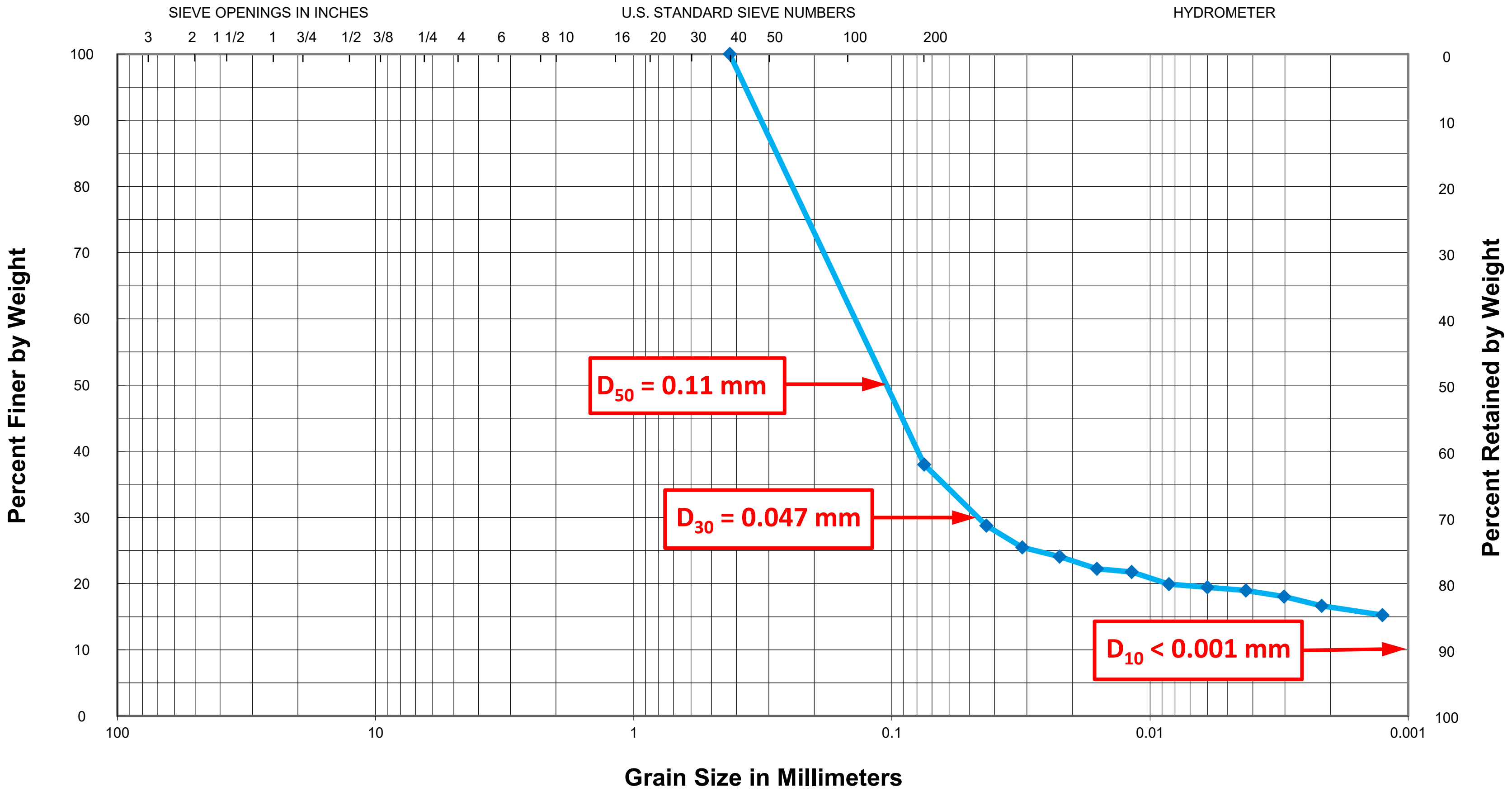
PROJECT: 101124 Hwy. 135 over Ditch No. 1 (Site 3)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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**		
C4	4.5-5.5	9	NON-PLASTIC			100	100	100	100	100	100	82	8	SM-SP	A-3
C4	14-15	22	52	17	35	---	---	---	---	100	---	---	49	SC	A-2-7
C4	24-25	2	---	---	---	---	---	---	---	100	---	---	3	SP	A-3
C4	29-30	9	---	---	---	---	---	---	---	100	---	---	22	SM	A-2-4
C4	39-40	18	---	---	---	100	100	100	100	100	100	53	2	SP	A-3
C4	64-65	14	---	---	---	100	100	100	93	88	84	19	4	SW	A-1-b
C4	79-80	25	---	---	---	100	100	100	100	100	100	96	14	SM	A-2-4
C4	99-100	15	---	---	---	---	---	---	---	90	---	---	6	SW	A-1-b
C4	119-120	17	---	---	---	100	100	100	100	100	99	40	4	SP	A-1-b

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

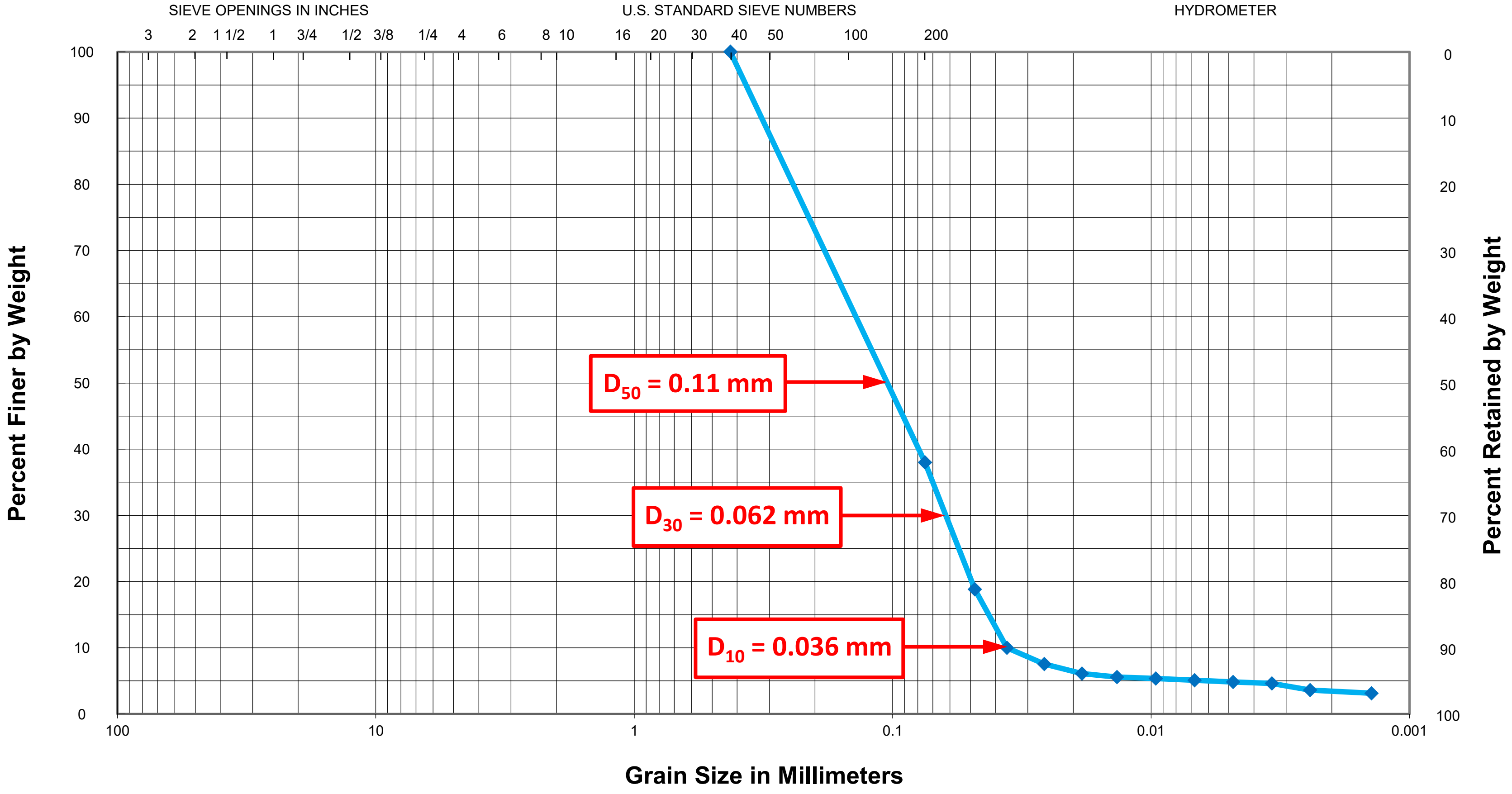
Sample: Boring C1, 9-10 ft; NON-PLASTIC
Description: Gray, tan, and reddish brown silty fine SAND

USCS Classification = SM
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

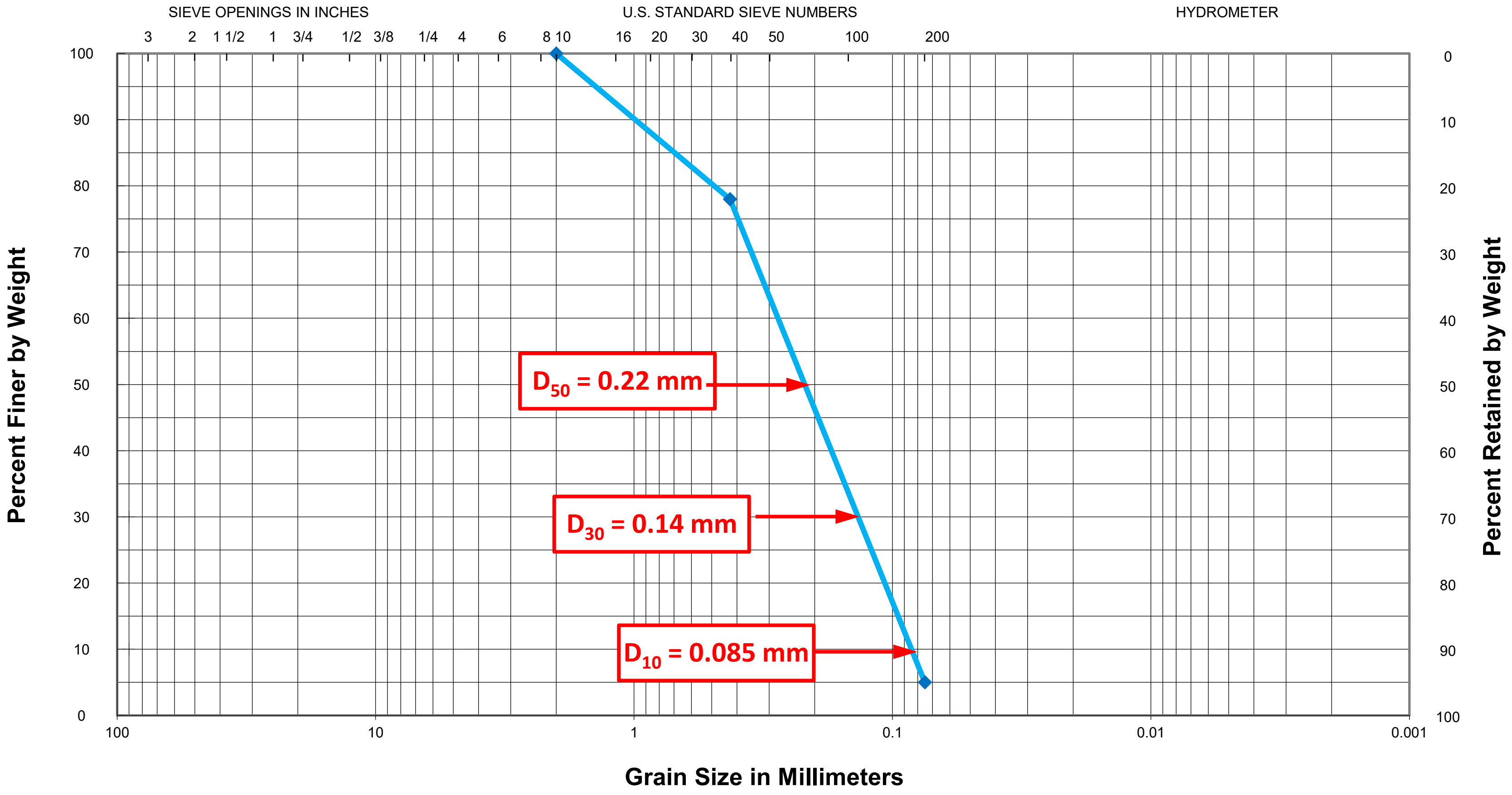
Sample: Boring C1, 29-30 ft; NON-PLASTIC
Description: Brown and tan silty fine SAND

USCS Classification = SM
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.22 mm

D_{30} = 0.14 mm

D_{10} = 0.085 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring C1, 39-40 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

D_{50} = 0.11 mm

23-031

GRAIN SIZE CURVE

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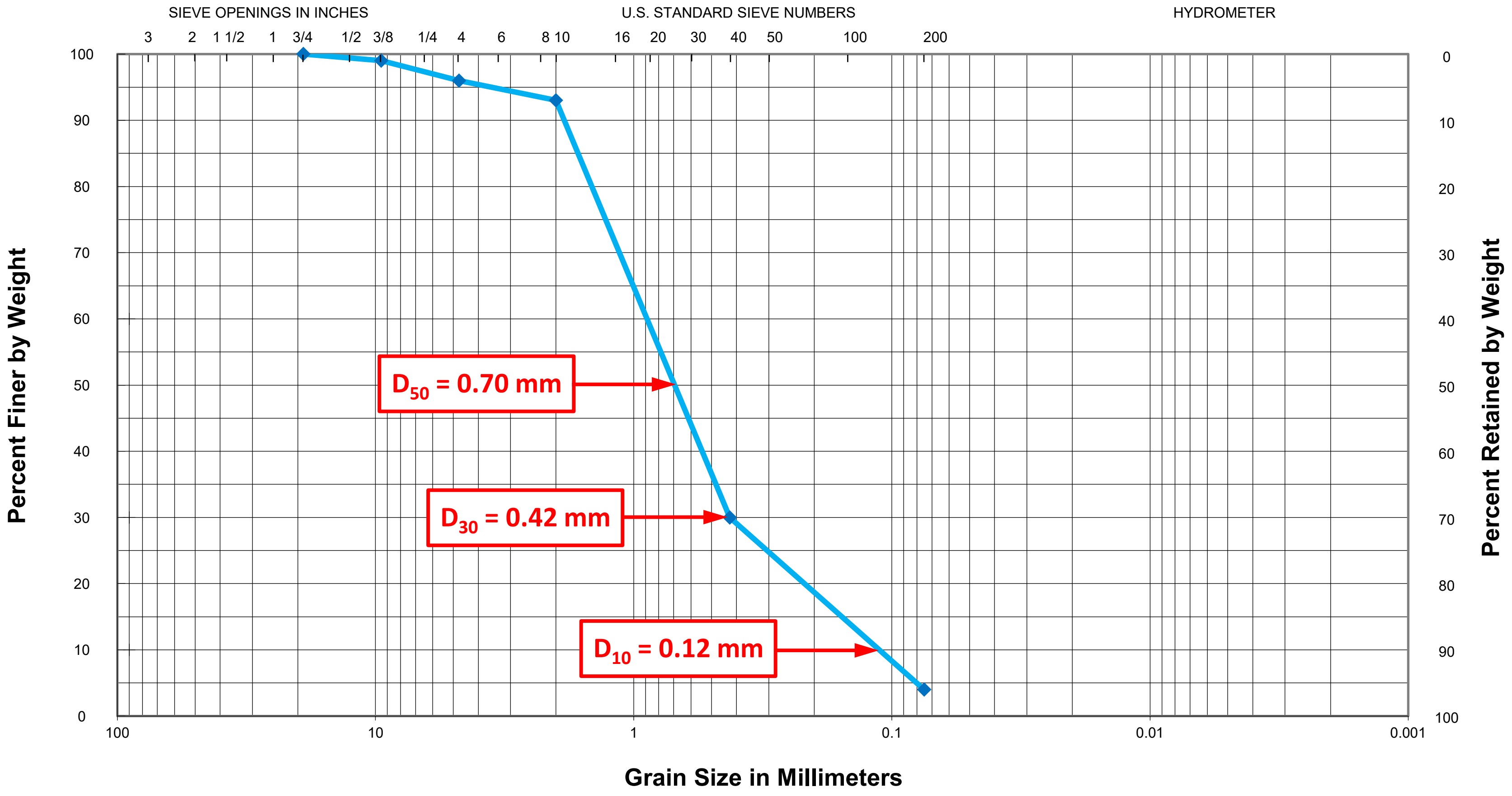
A UES Company



D_{50} = 0.70 mm

D_{30} = 0.42 mm

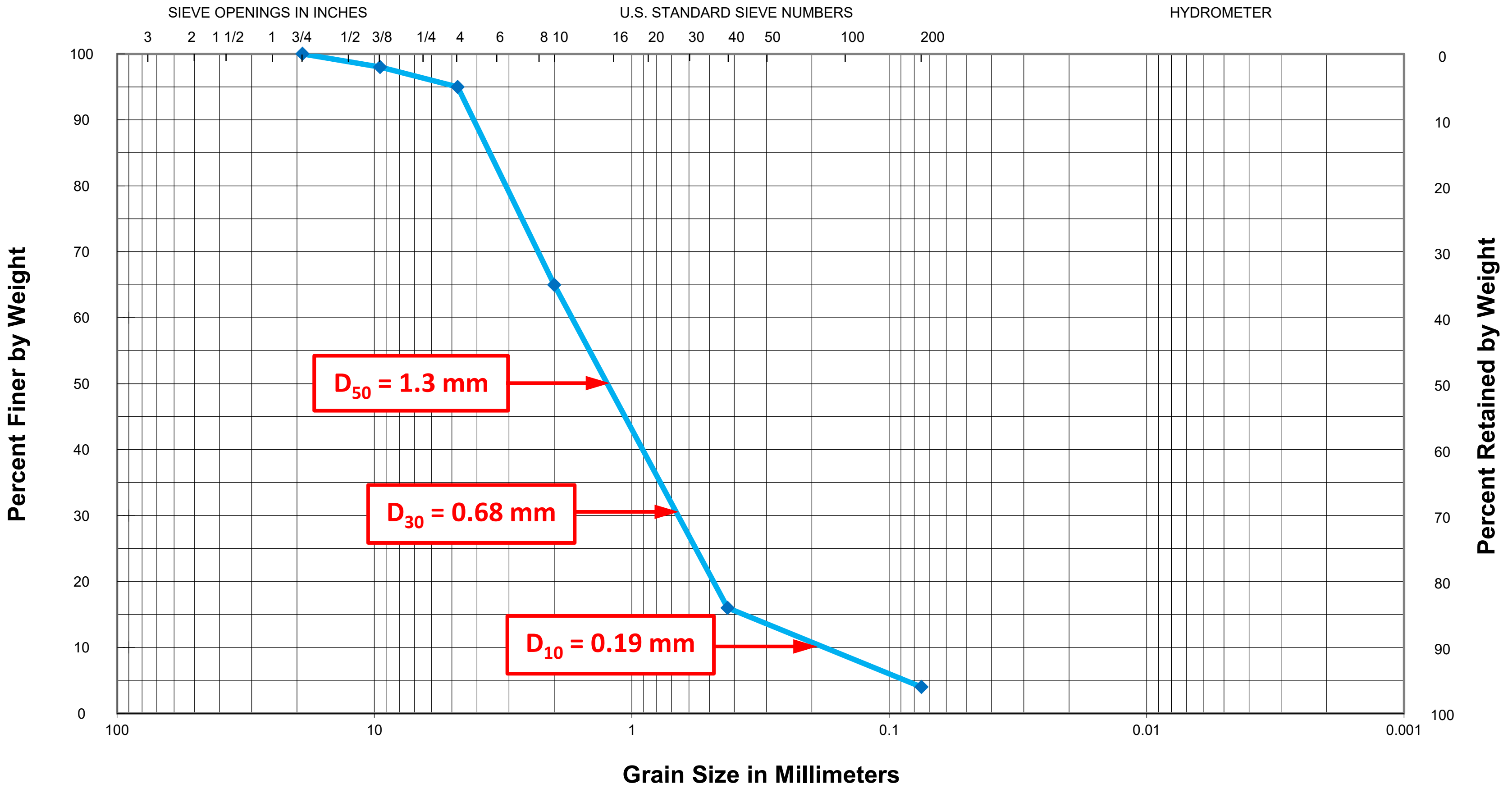
D_{10} = 0.12 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring C1, 64-65 ft
Description: Grayish tan fine to medium SAND

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

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

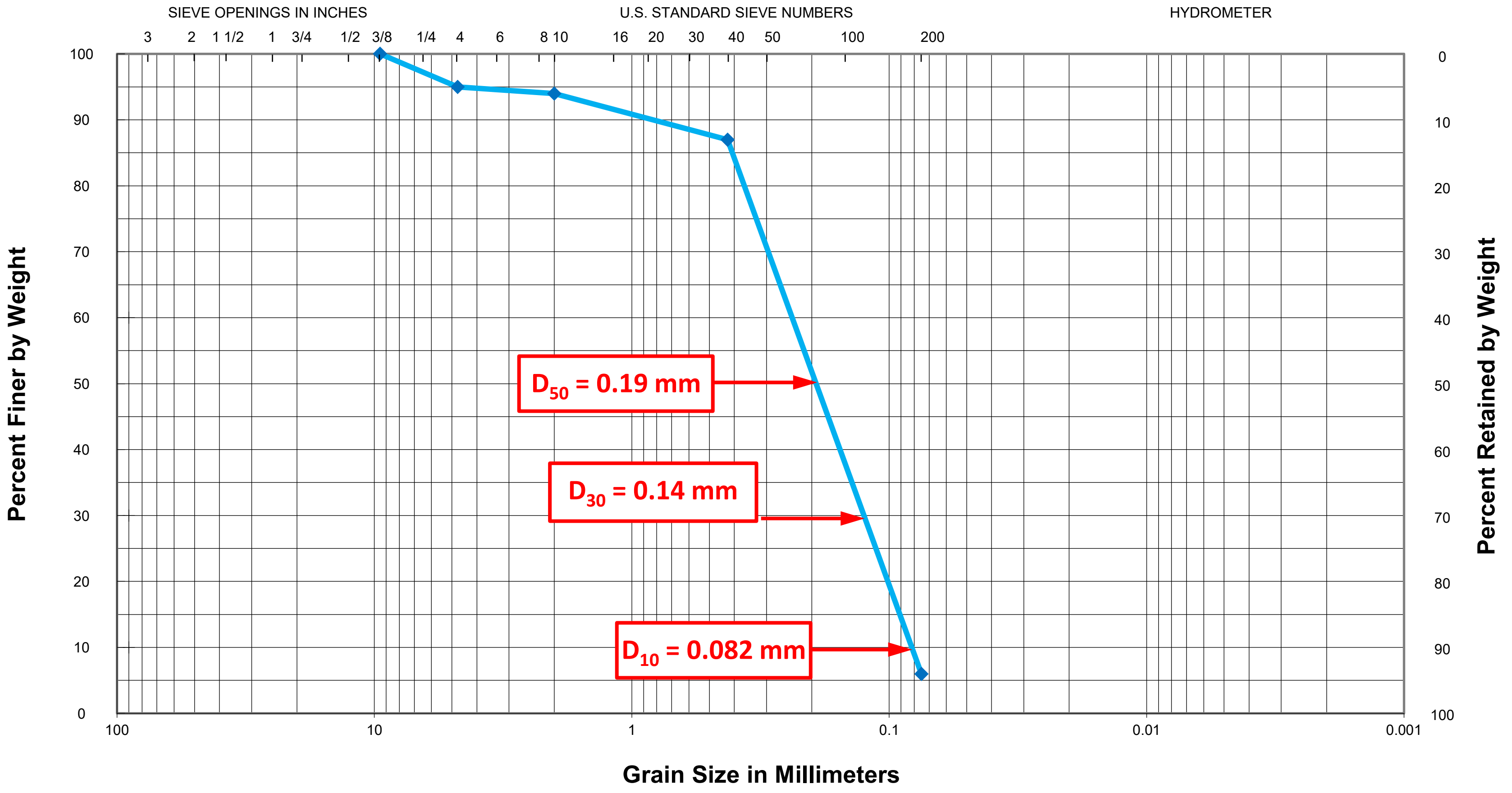
Sample: Boring C1, 84-85 ft
Description: Grayish tan fine to coarse SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

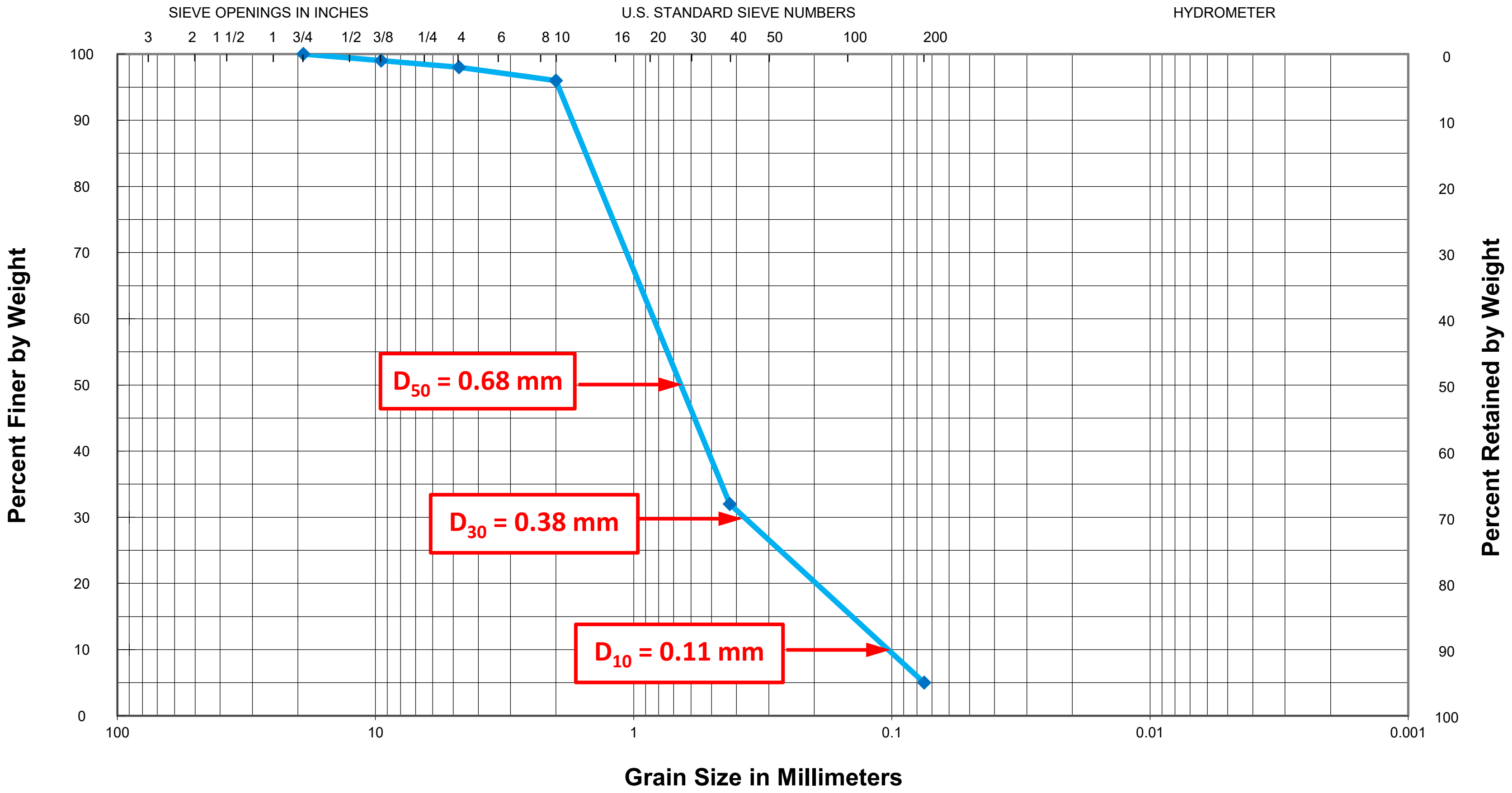
Sample: Boring C1, 94-95 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.68 mm

D_{30} = 0.38 mm

D_{10} = 0.11 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

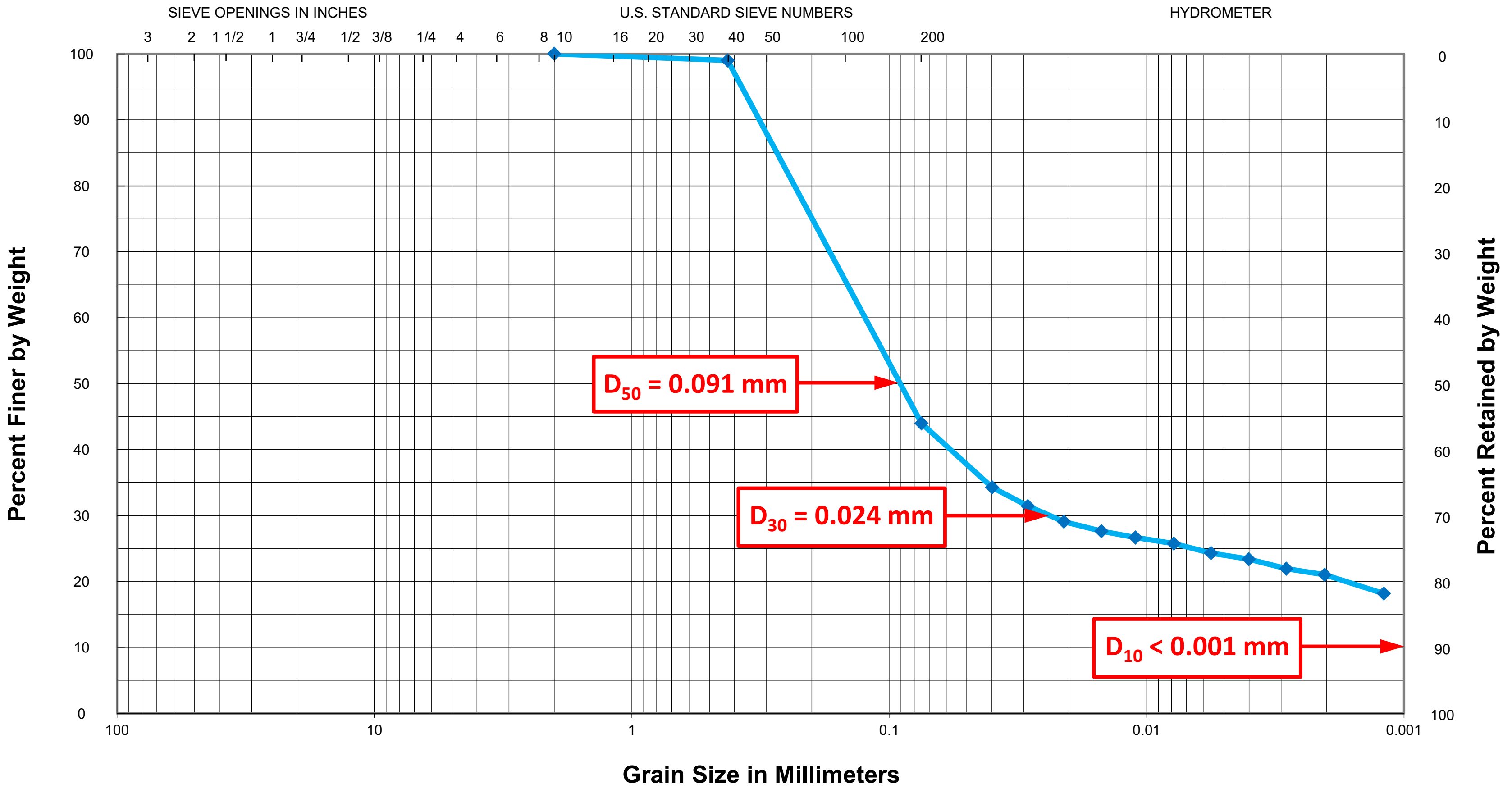
Sample: Boring C1, 99-100 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.091 mm

D_{30} = 0.024 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

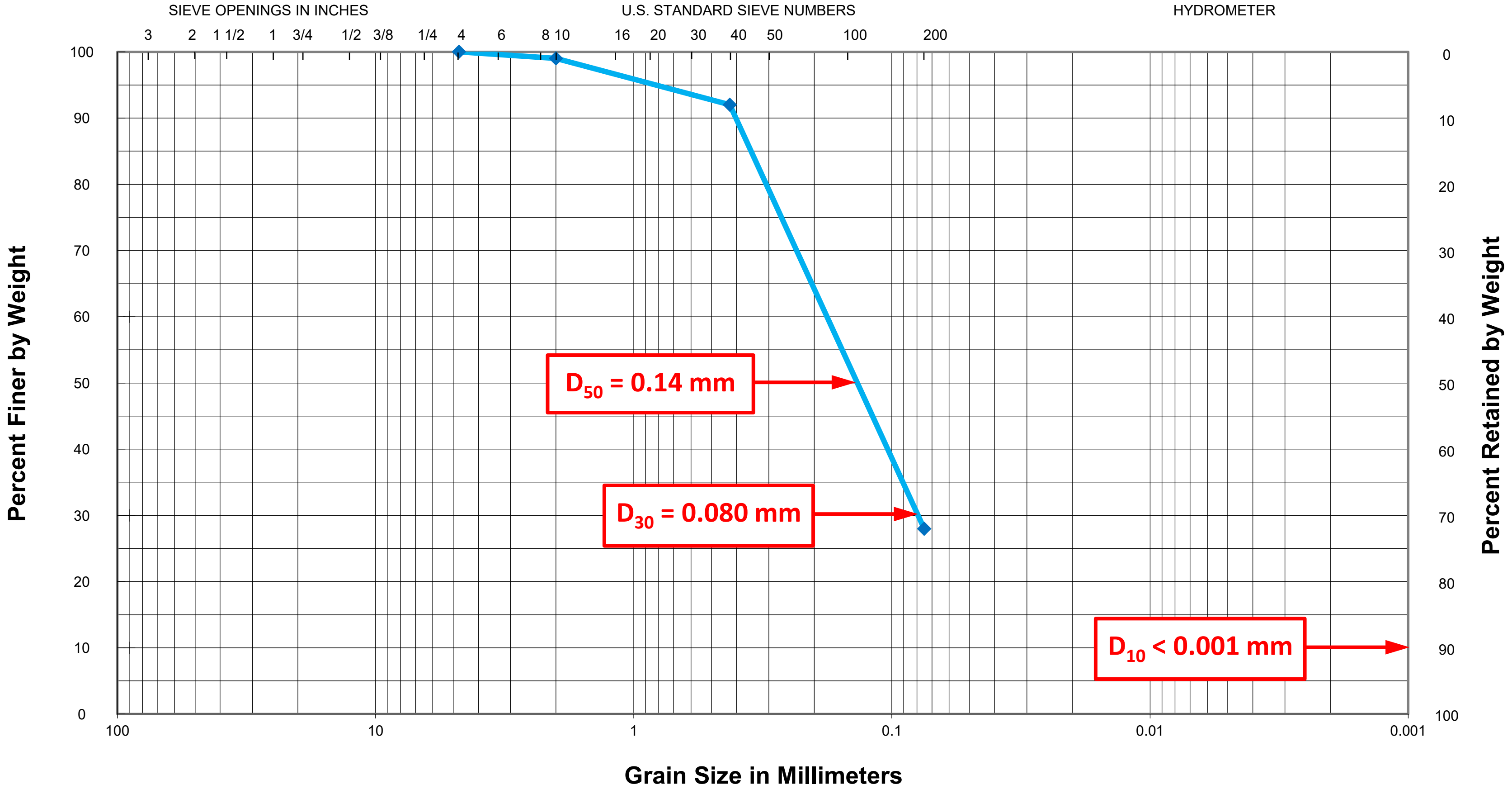
Sample: Boring C2, 6.5-7.5 ft; LL=26, PL=17, PI=9
Description: Tan and brown silty fine SAND w/ fine sandy clay seams and layers

USCS Classification = SC
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

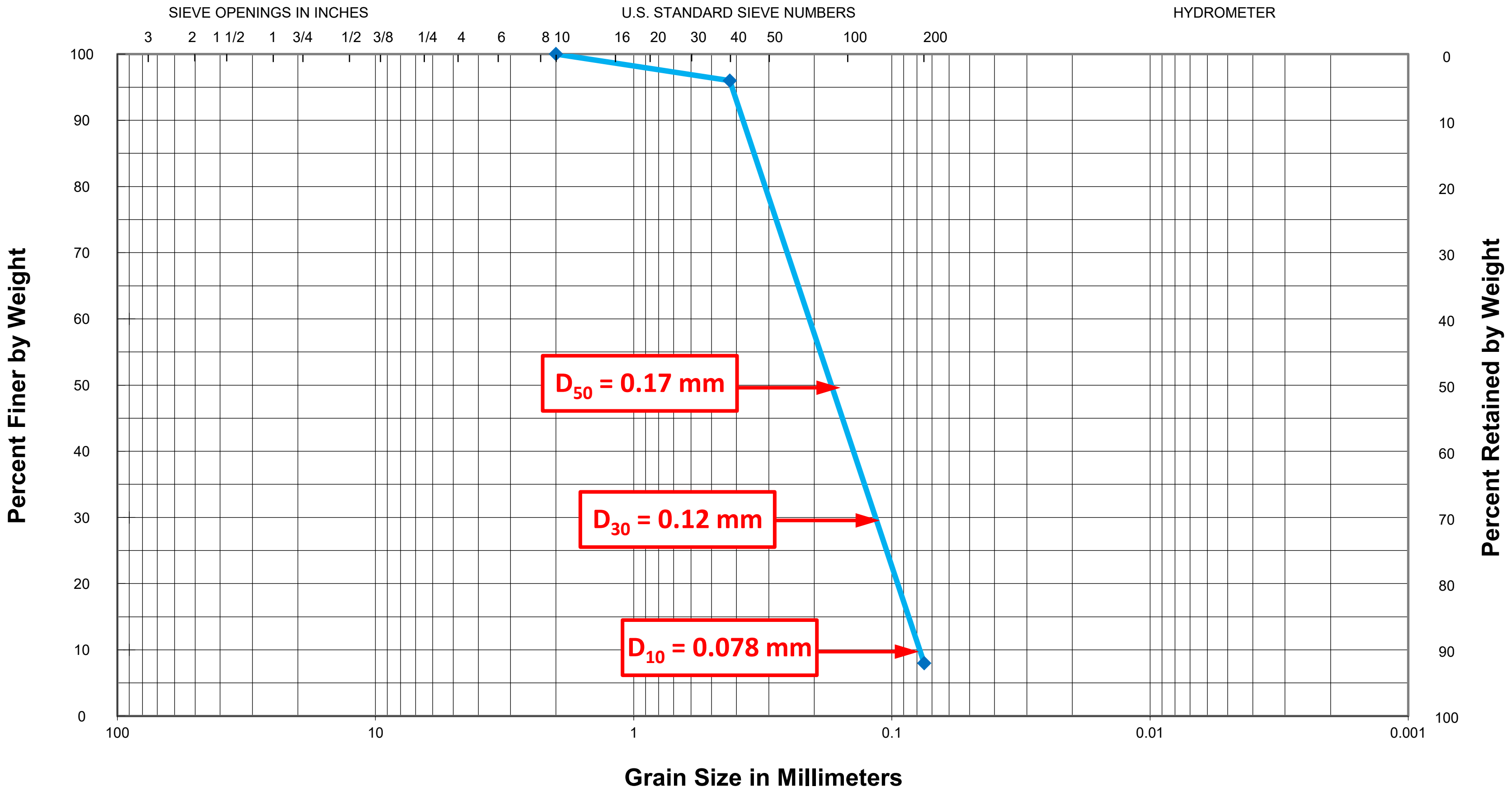
Sample: Boring C2, 14-15 ft
Description: Gray and reddish brown clayey fine SAND

USCS Classification = SC
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

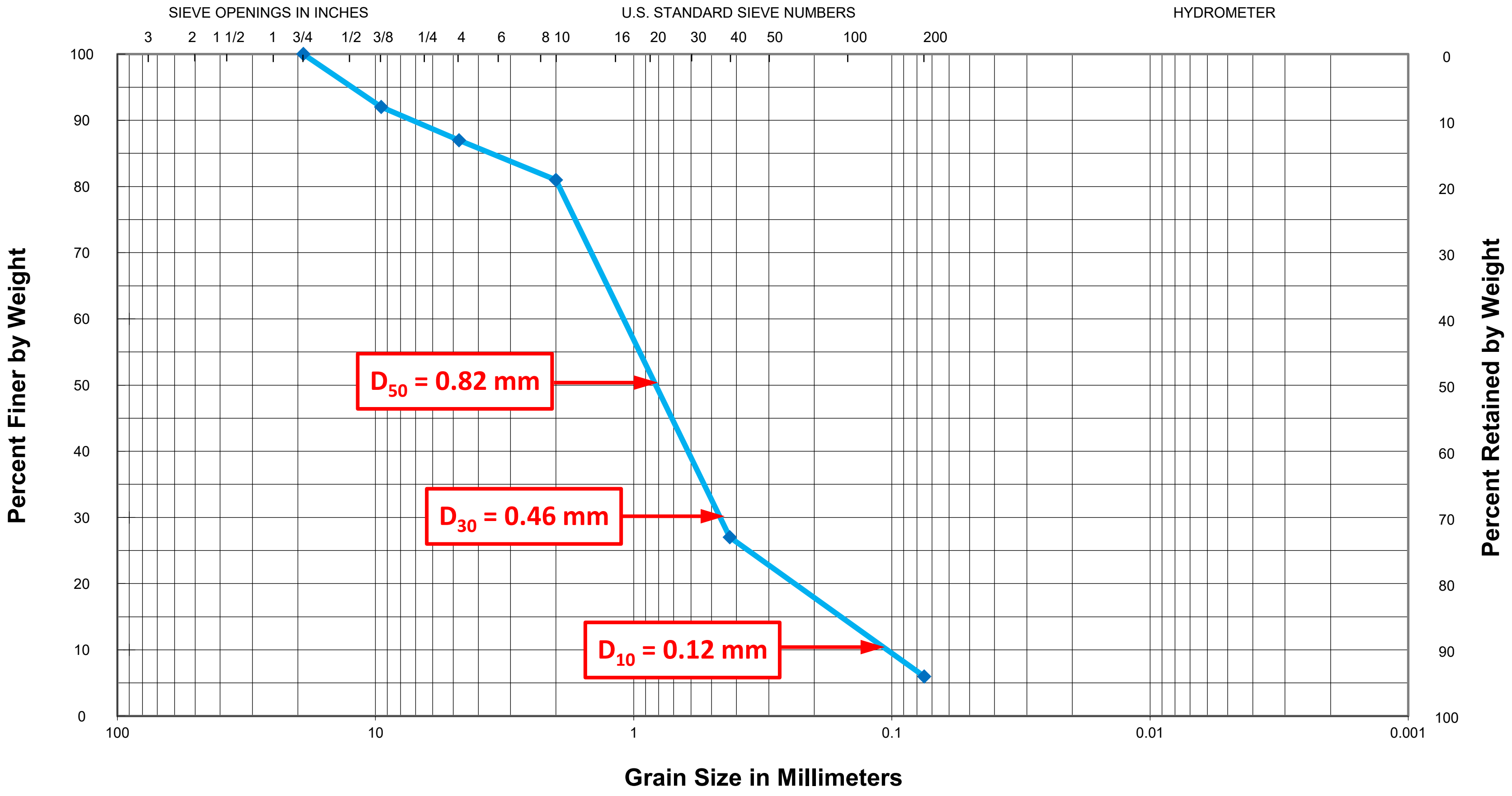
Sample: Boring C2, 34-35 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.82 mm

D_{30} = 0.46 mm

D_{10} = 0.12 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

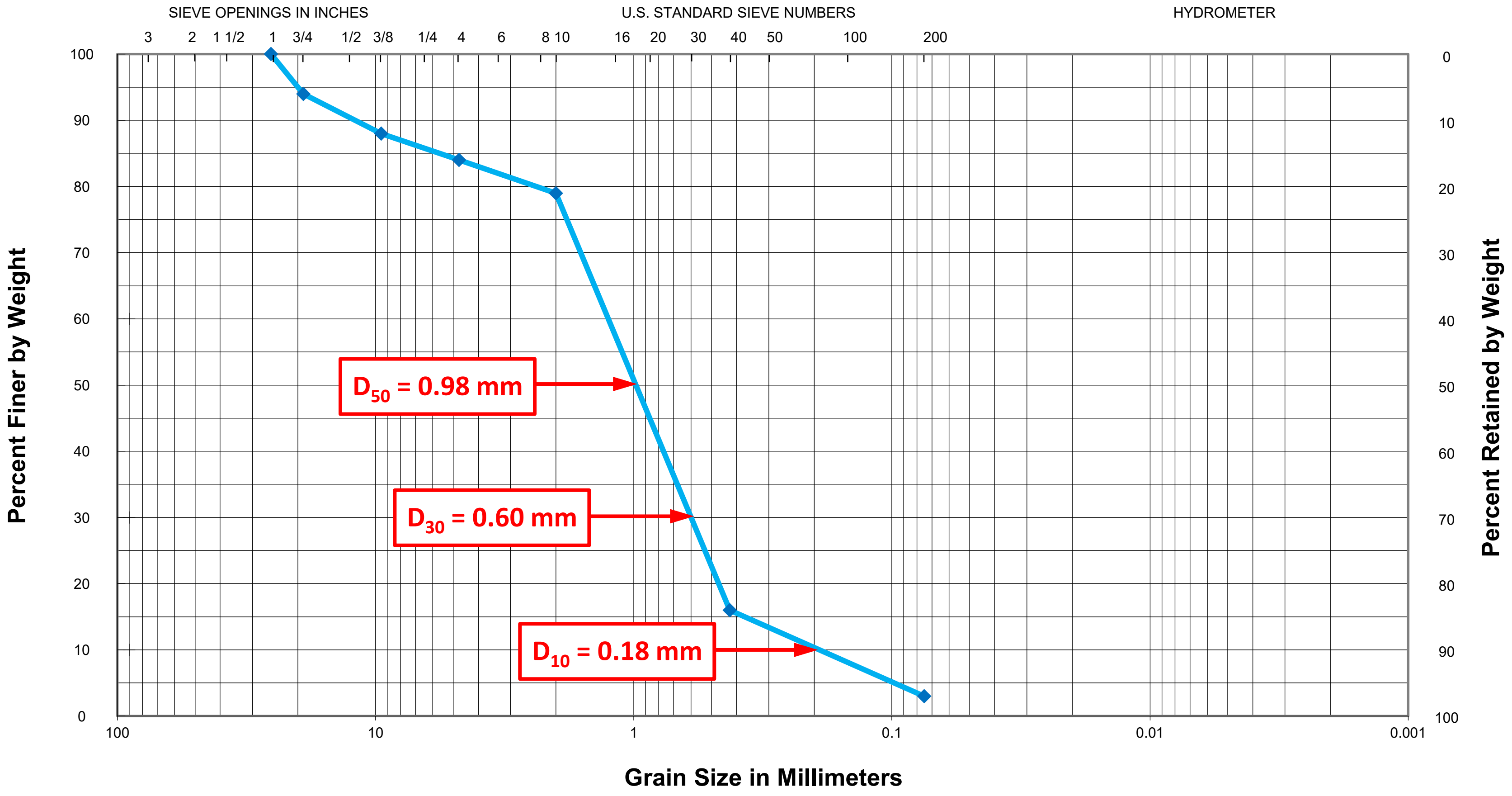
Sample: Boring C2, 64-65 ft
Description: Grayish tan fine to medium SAND, slightly silty w/ trace fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.98 mm

D_{30} = 0.60 mm

D_{10} = 0.18 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

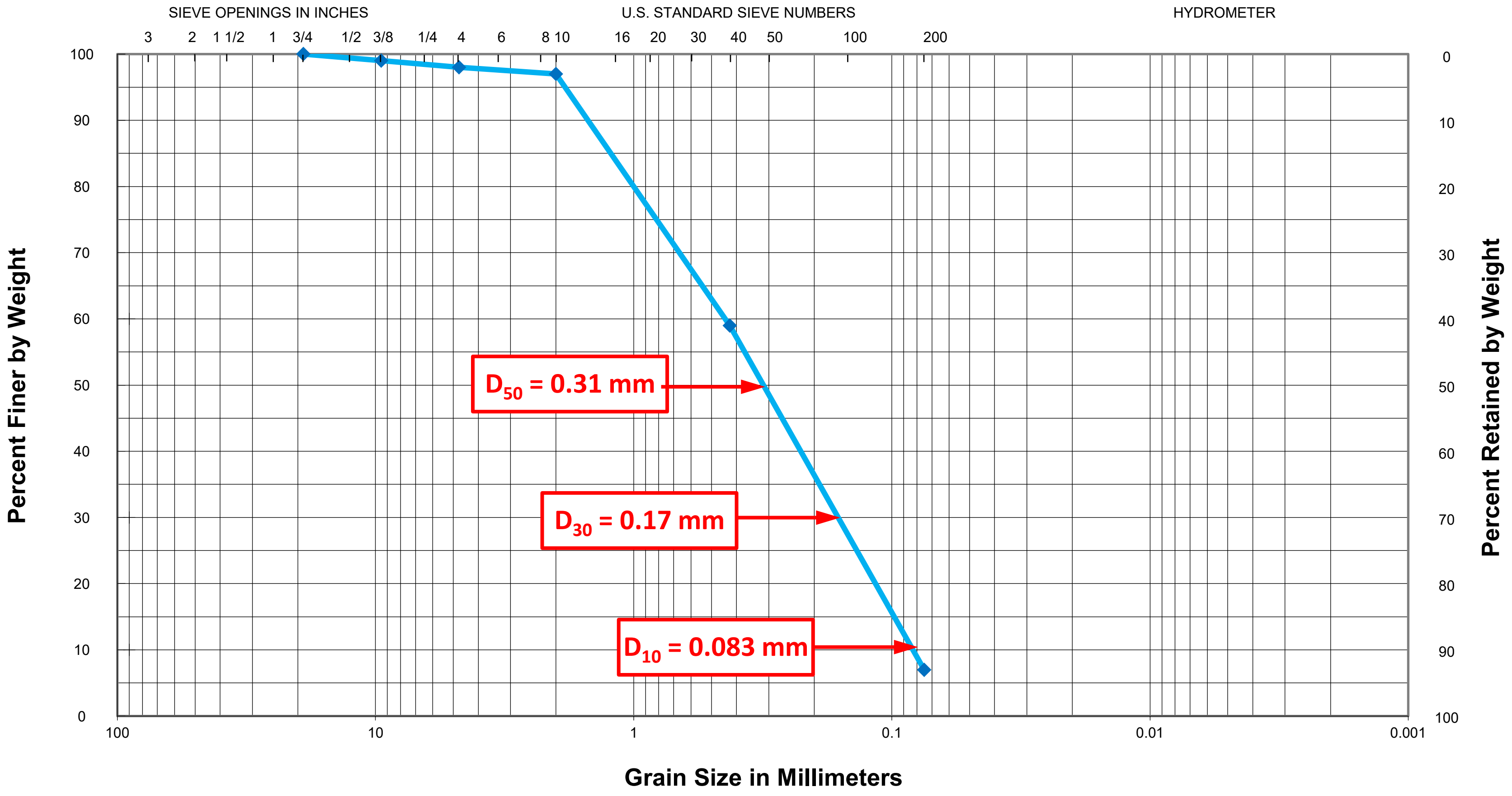
Sample: Boring C2, 84-85 ft
Description: Grayish tan fine to medium SAND w/ trace fine to coarse gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

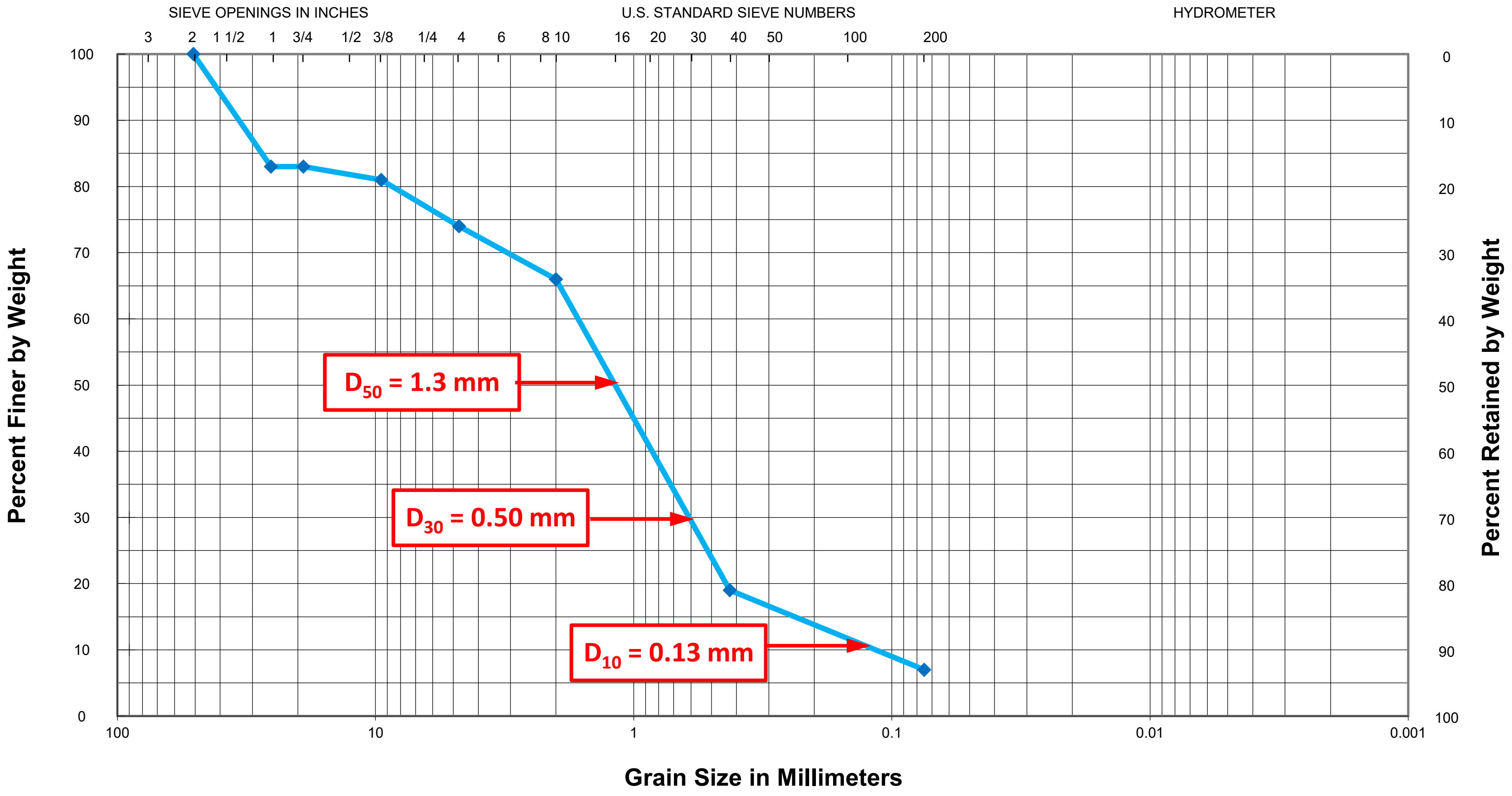
Sample: Boring C3, 34-35 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 1.3 mm

D_{30} = 0.50 mm

D_{10} = 0.13 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

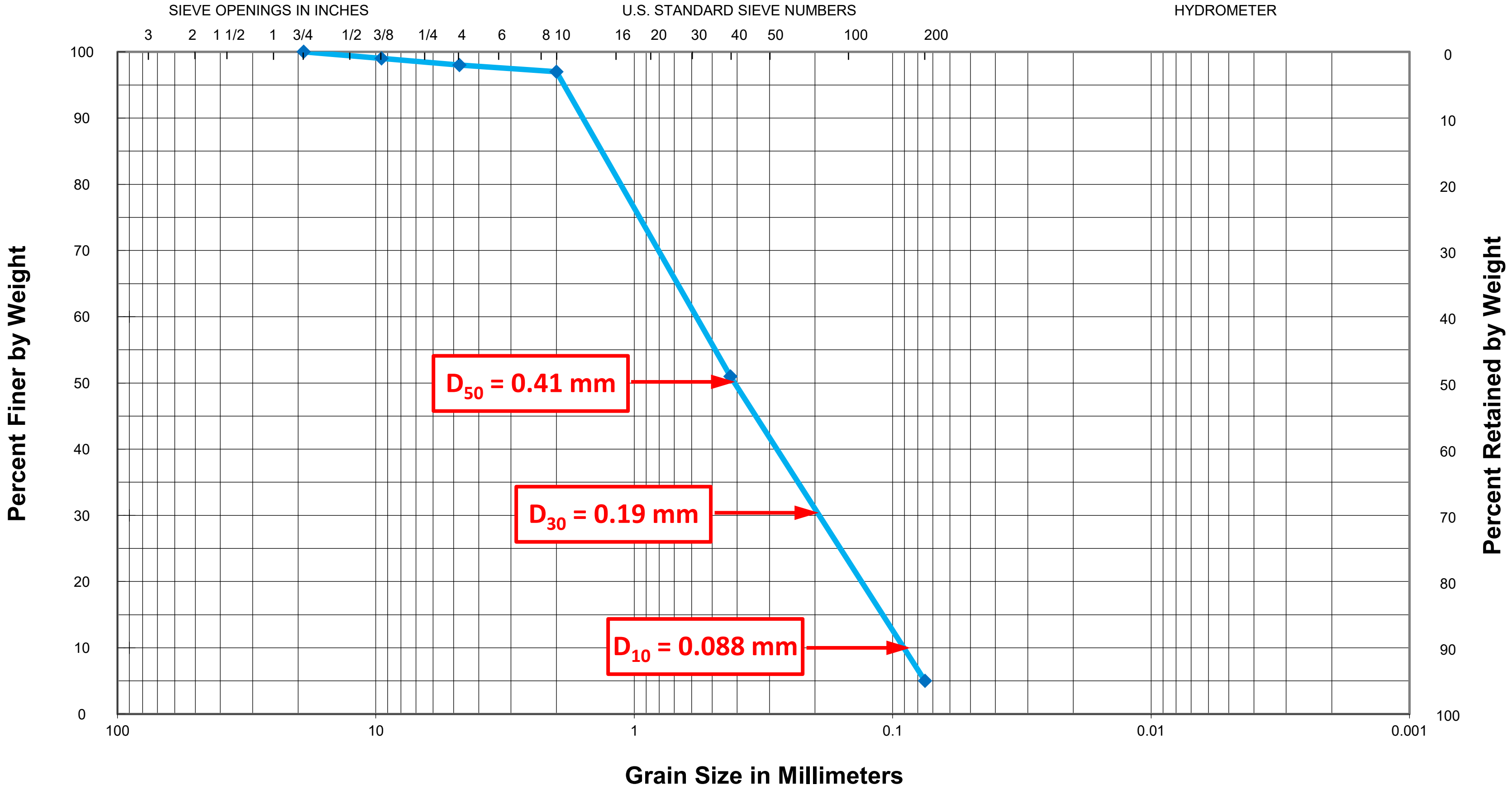
Sample: Boring C3, 54-55 ft
Description: Grayish tan fine to coarse SAND, slightly silty w/ a little fine to coarse gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

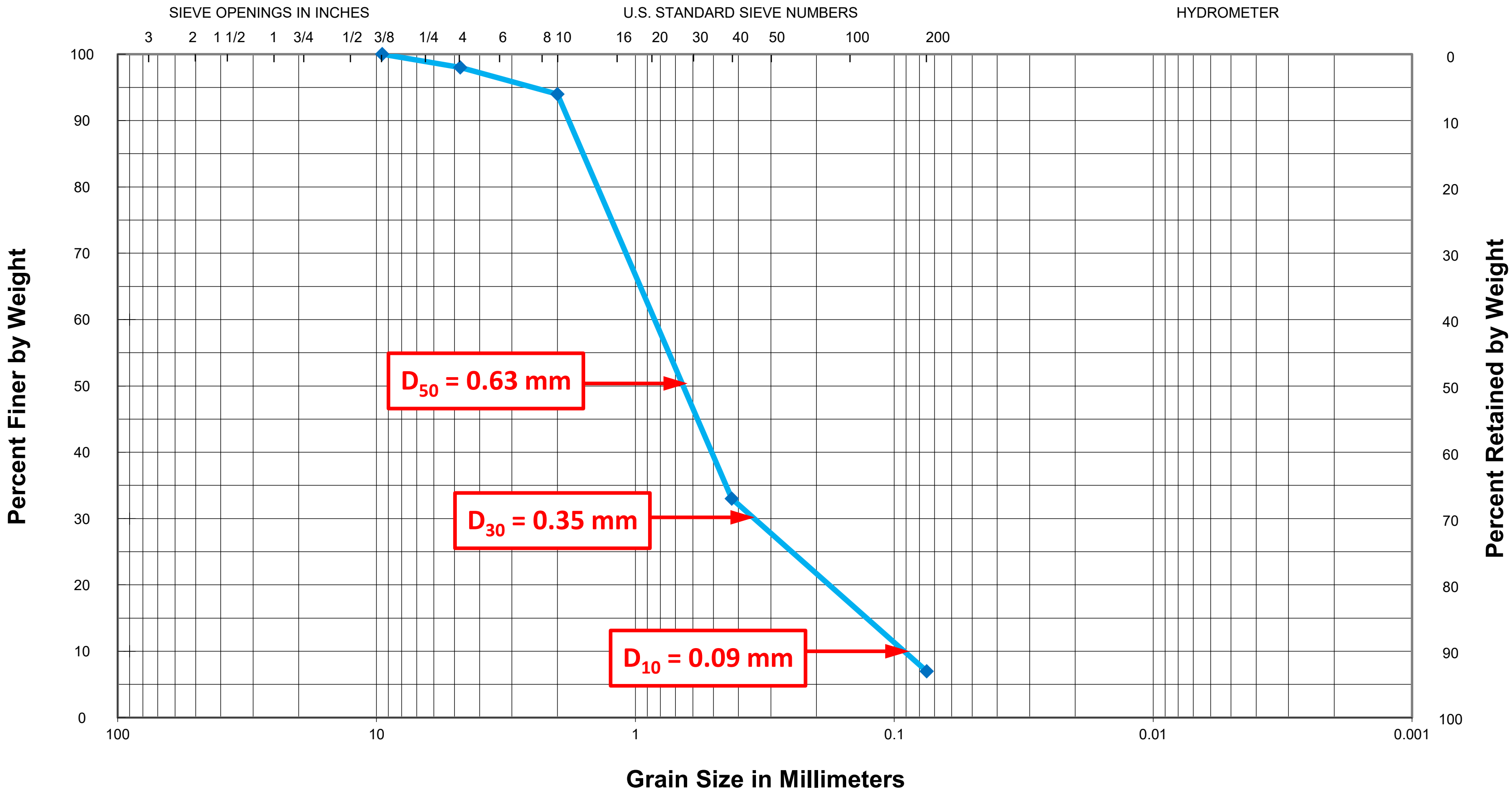
Sample: Boring C3, 64-65 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

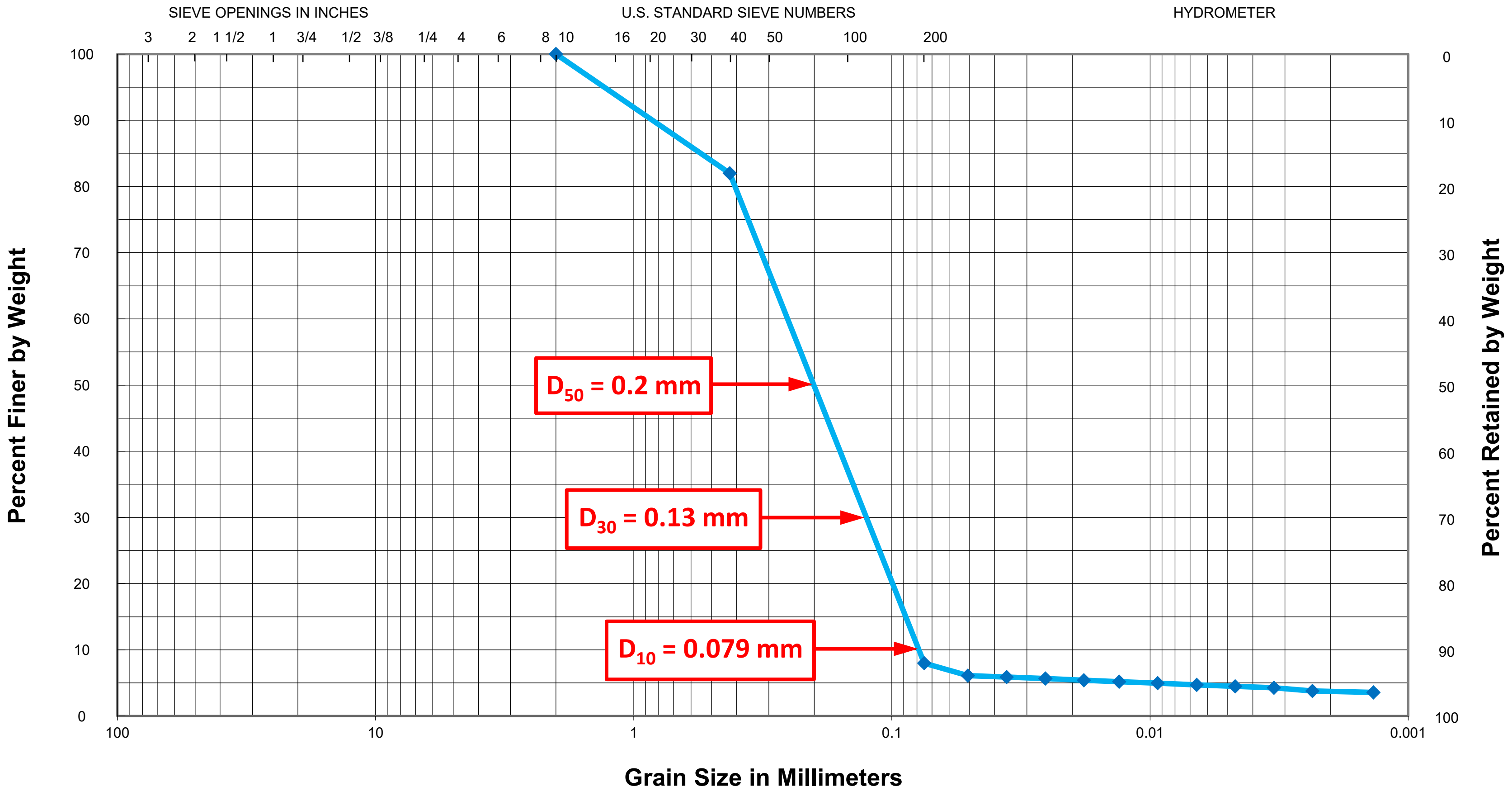
Sample: Boring C3, 94-95 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

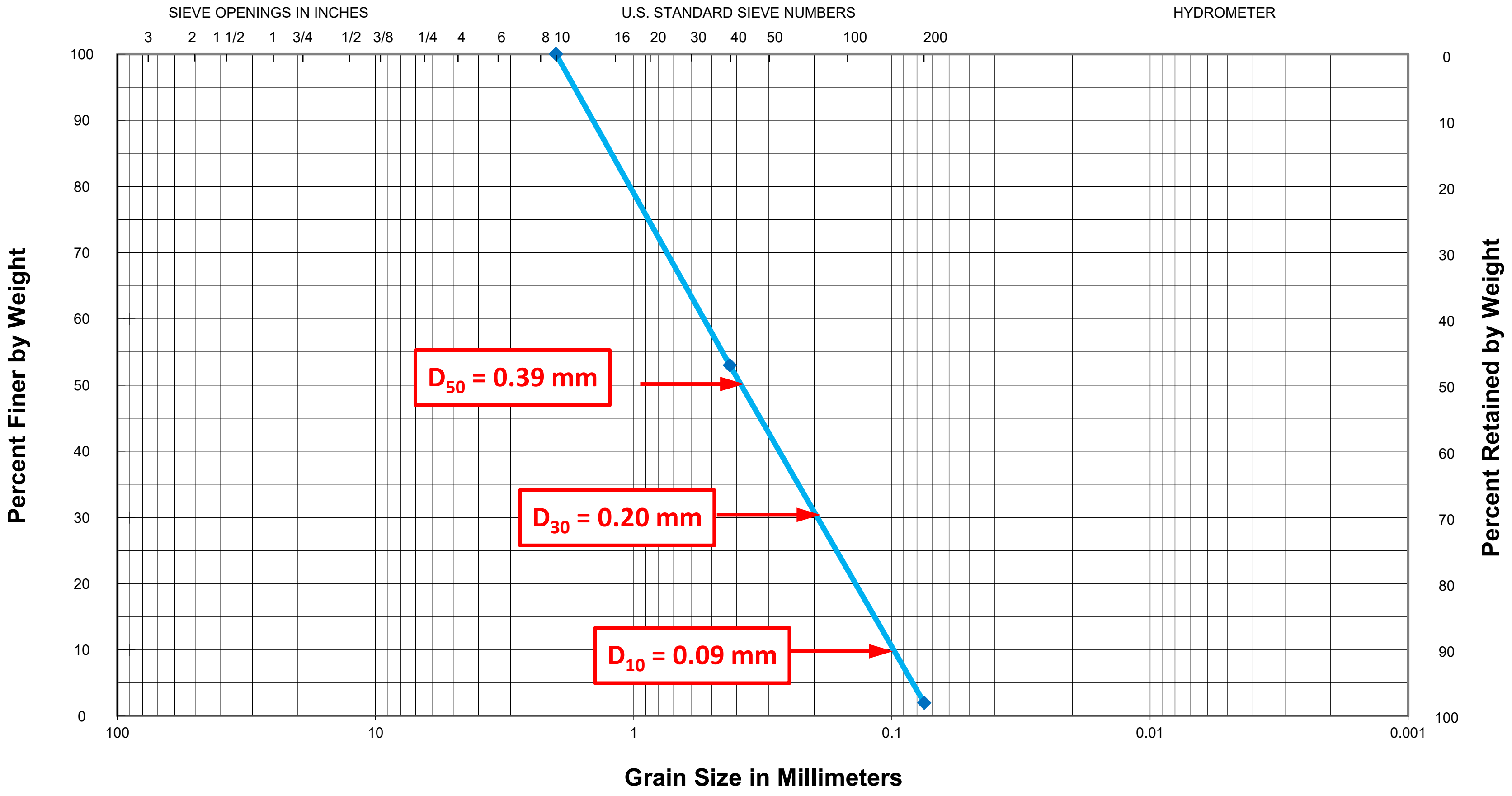
Sample: Boring C4, 4.5-5.5 ft; NON-PLASTIC
Description: Tan and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

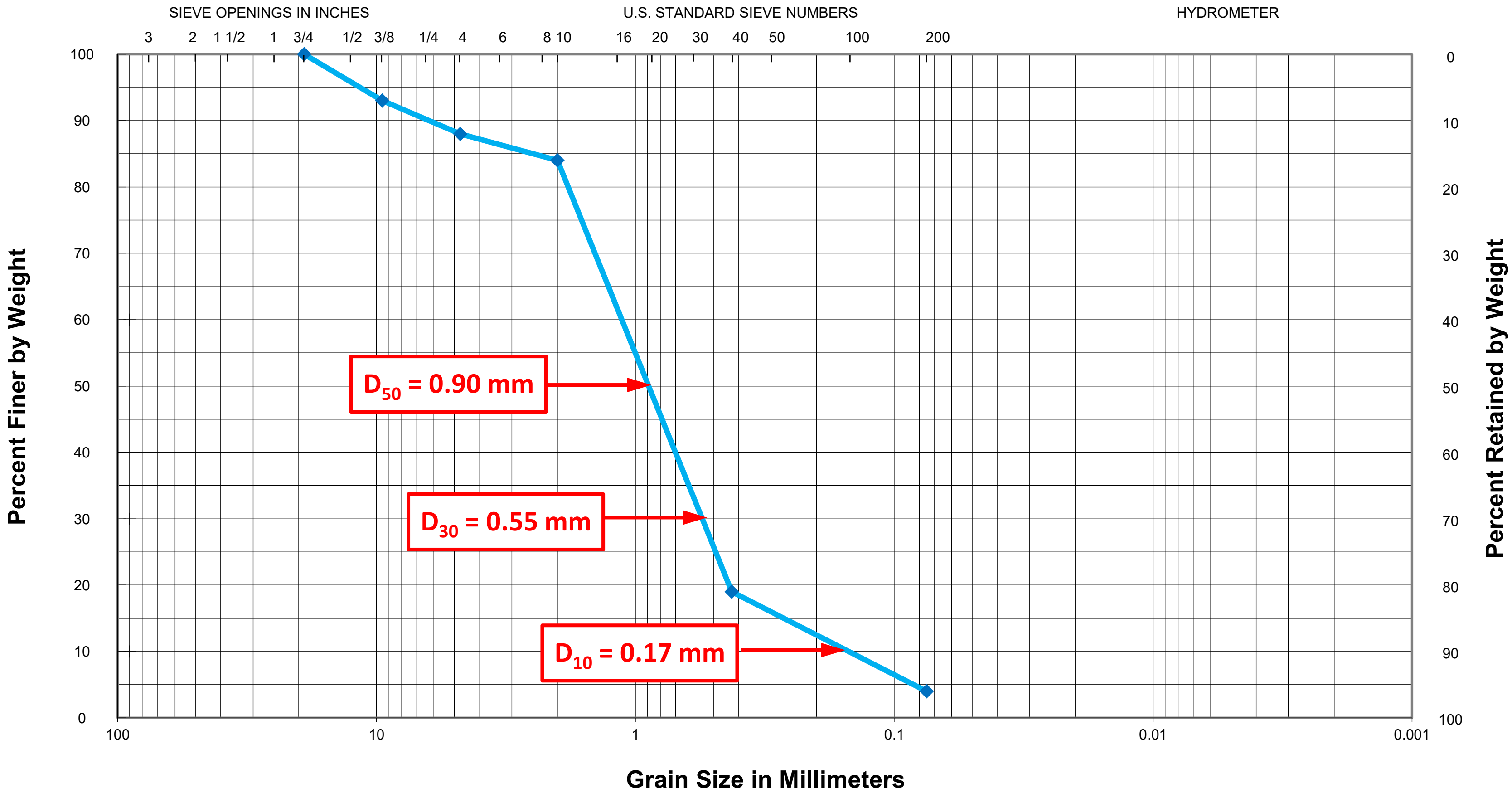
Sample: Boring C4, 39-40 ft
Description: Brownish gray fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.90 mm

D_{30} = 0.55 mm

D_{10} = 0.17 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

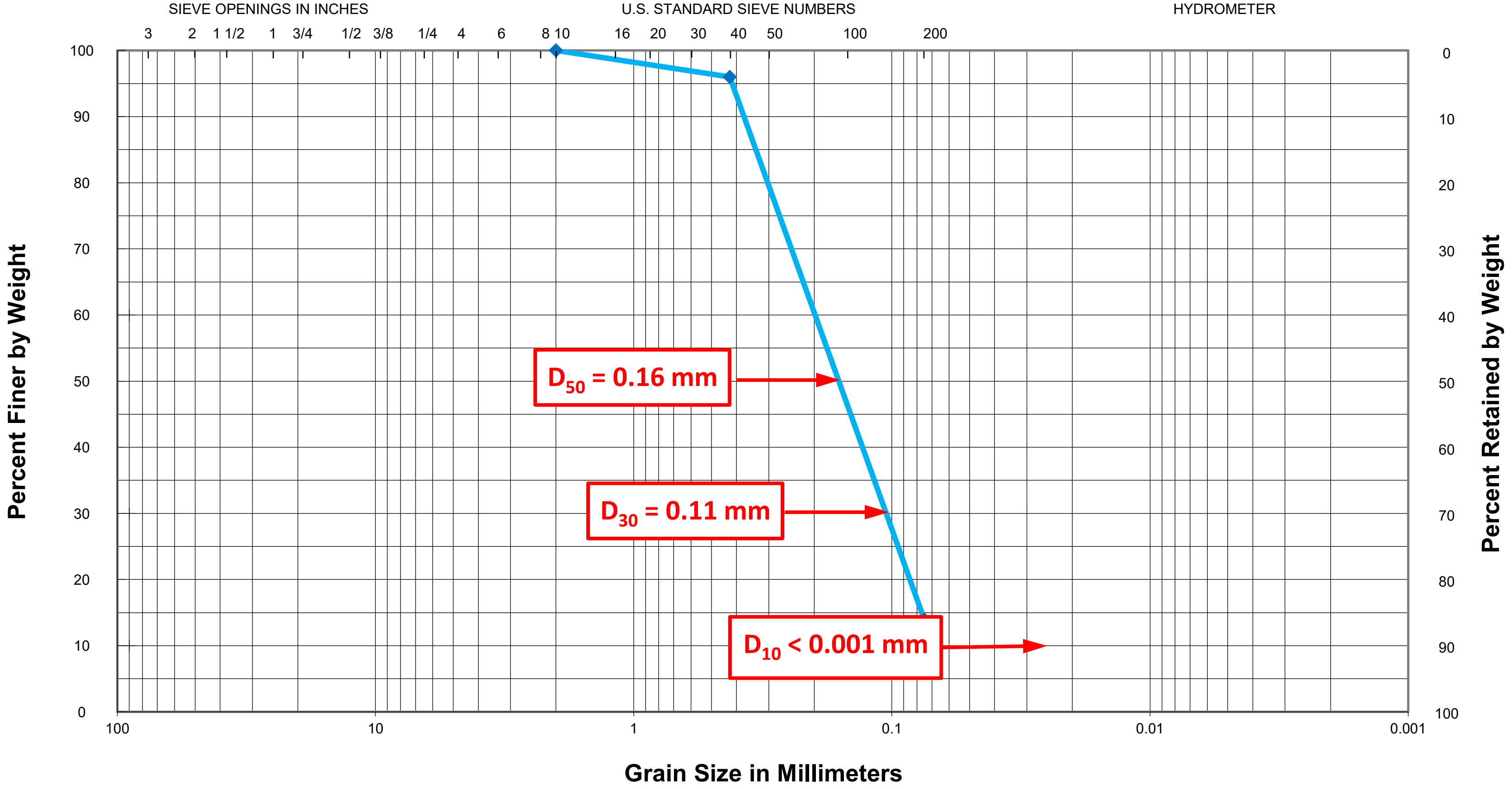
Sample: Boring C4, 64-65 ft
Description: Tan fine to medium SAND w/ trace fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

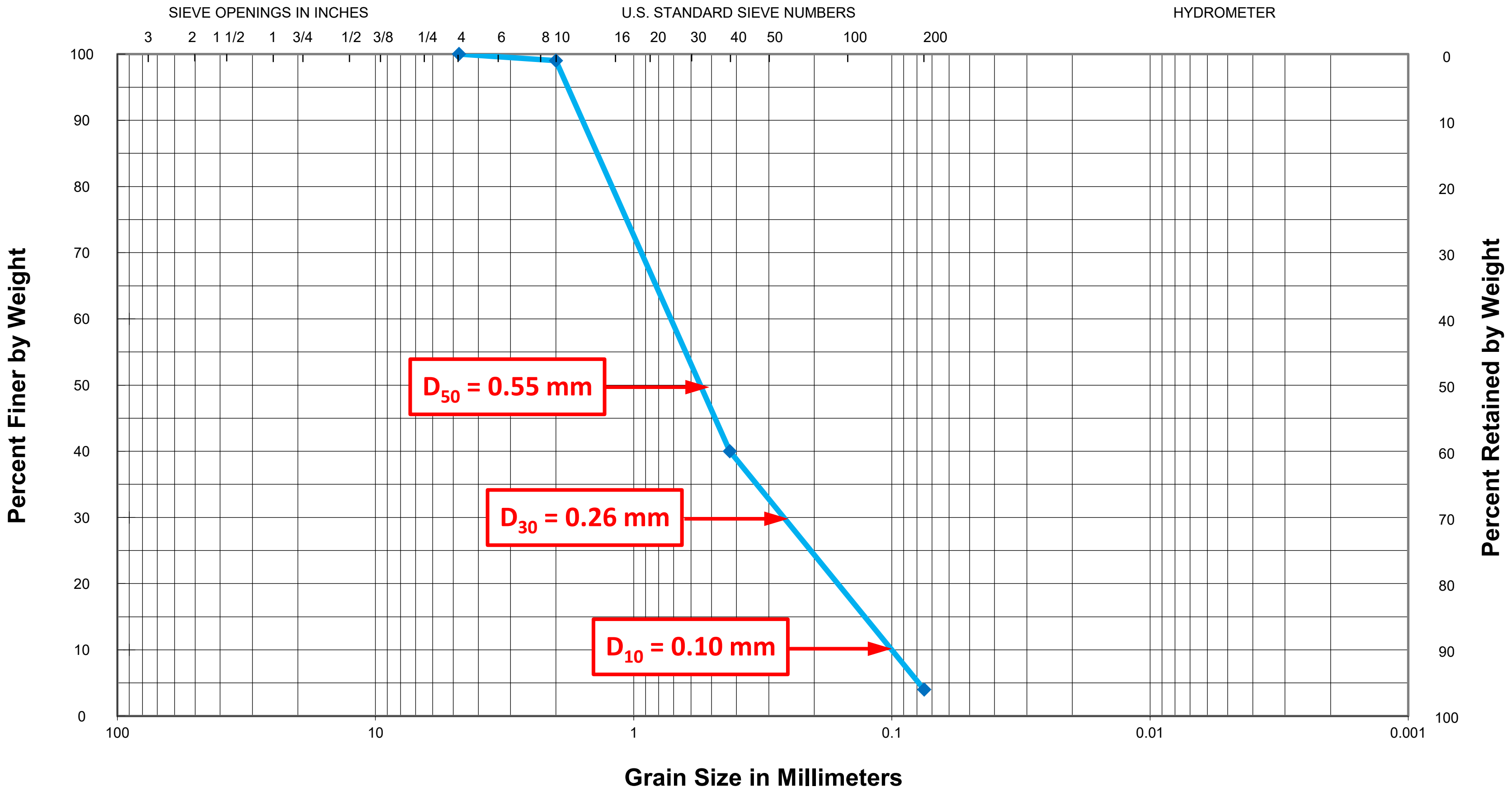
Sample: Boring C4, 79-80 ft
Description: Brownish gray silty fine SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.55 mm

D_{30} = 0.26 mm

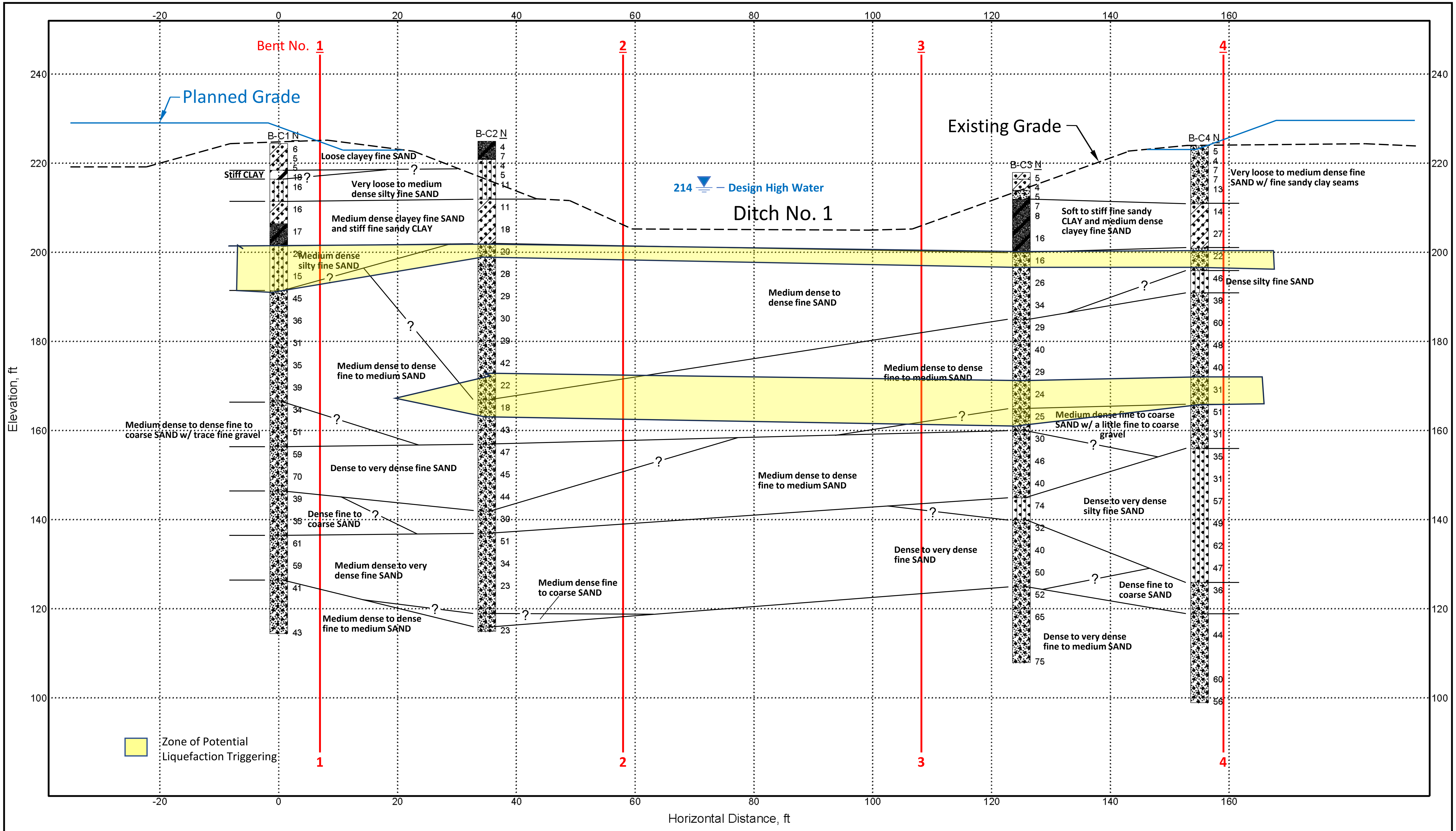
D_{10} = 0.10 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring C4, 119-120 ft
Description: Grayish tan fine to medium SAND

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

APPENDIX D



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

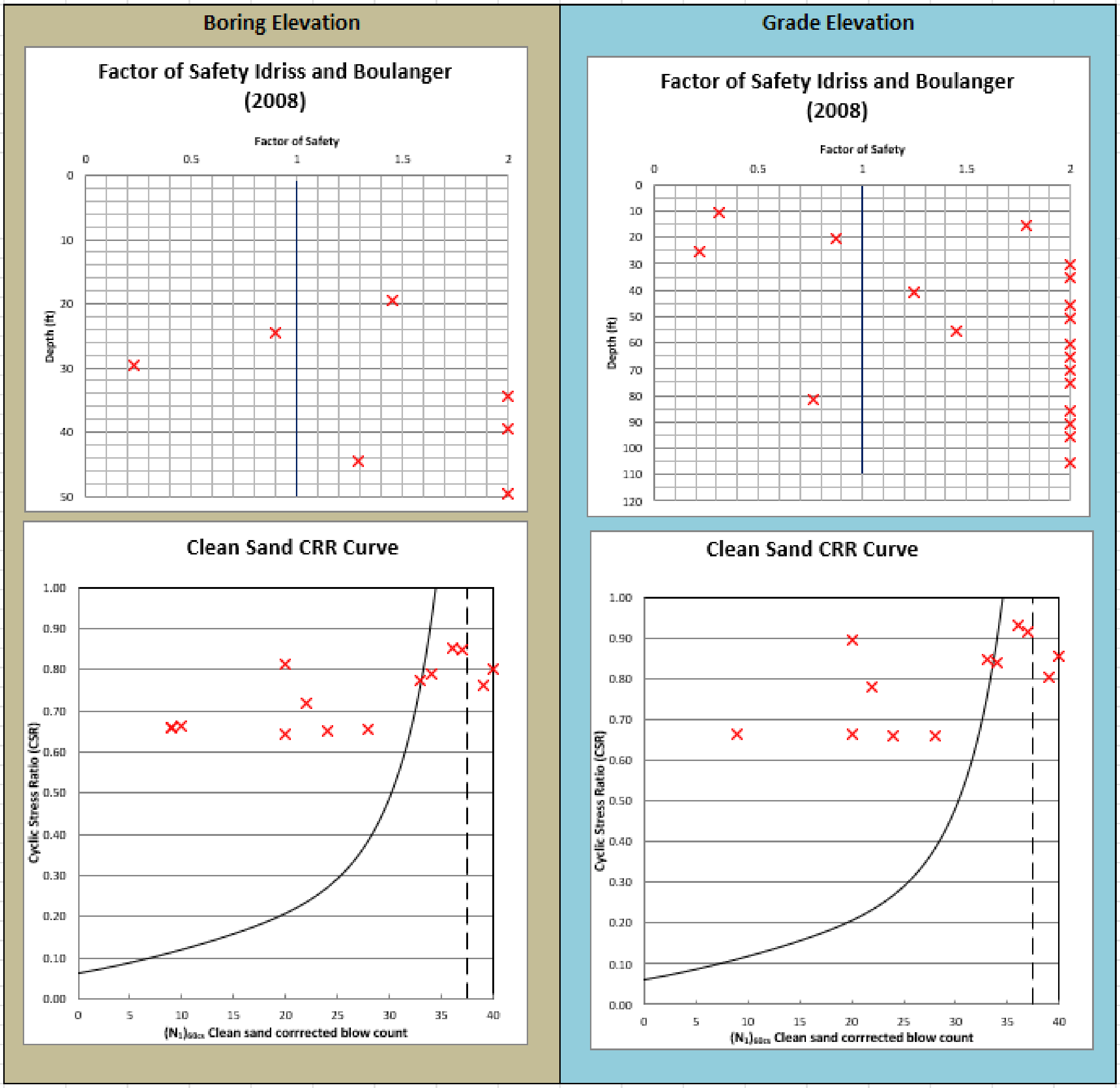
SCALE:
1" = 15' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Project Number: 23-031

Liquefaction Analysis Results

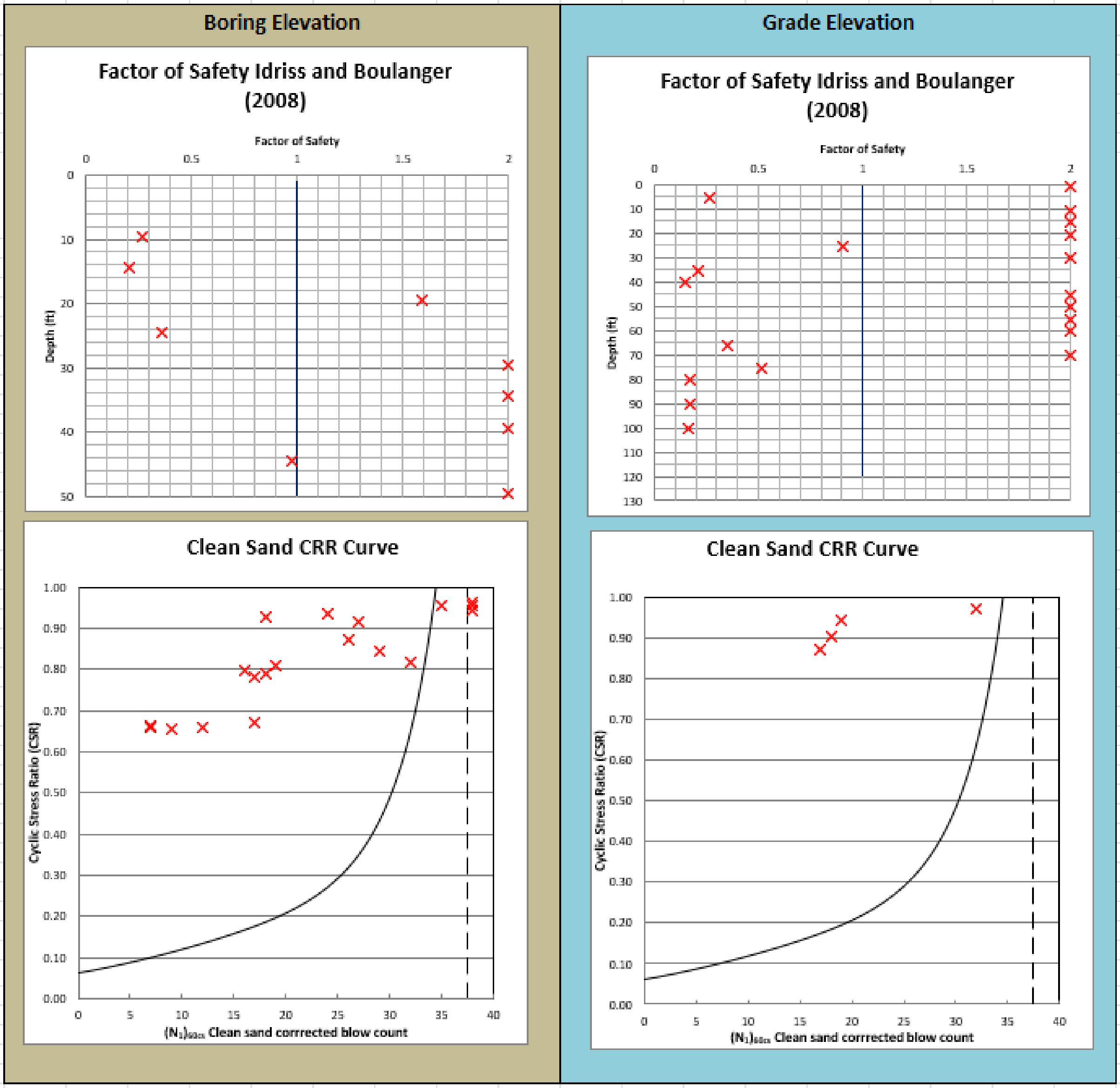
ARDOT 101124 Hwy 135 over Ditch No. 1
Bent 1 / Boring C1
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

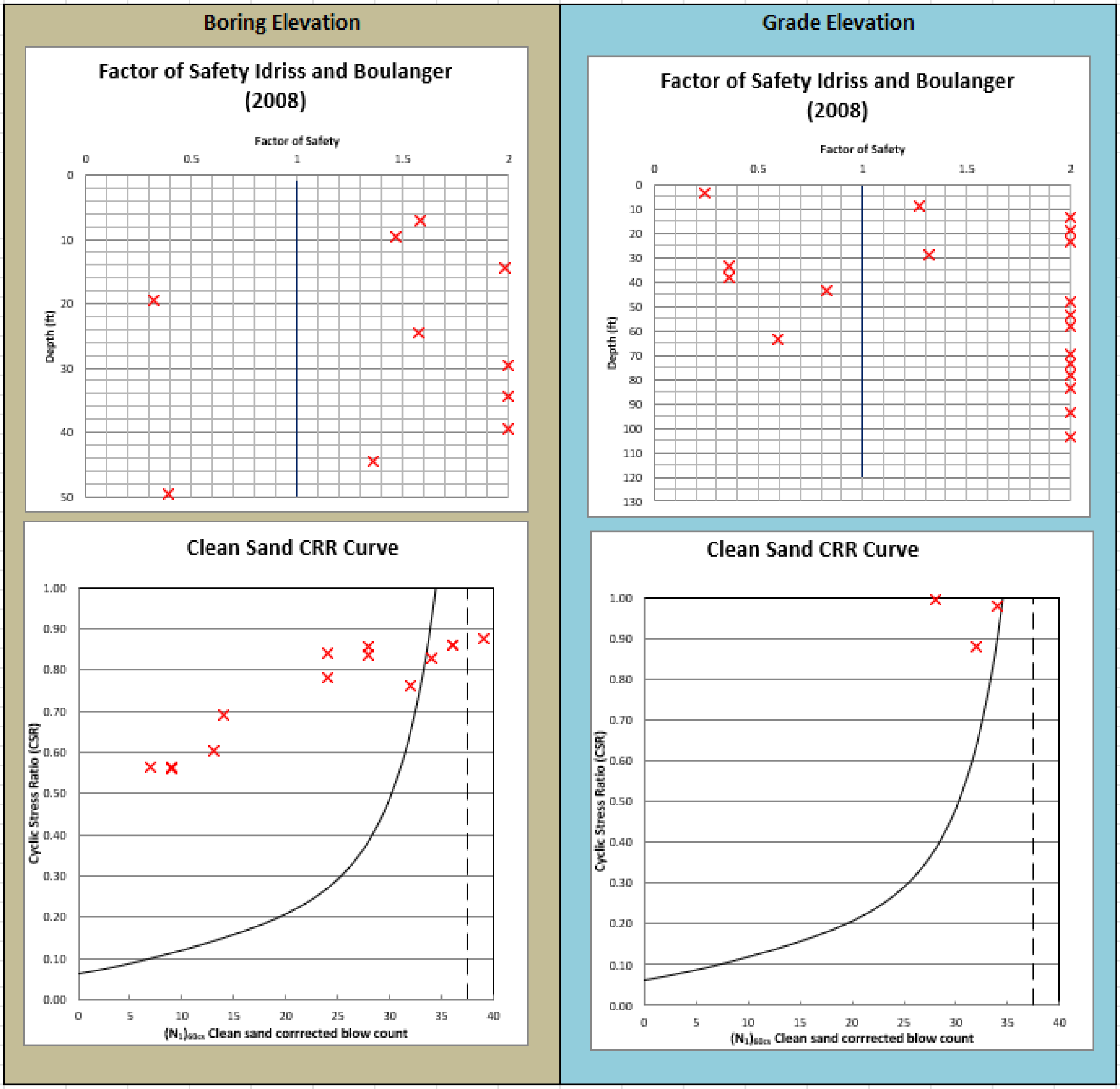
ARDOT 101124 Hwy 135 over Ditch No. 1
Bent 2 / Boring C2
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Ditch No. 1
Bent 3 / Boring C3
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

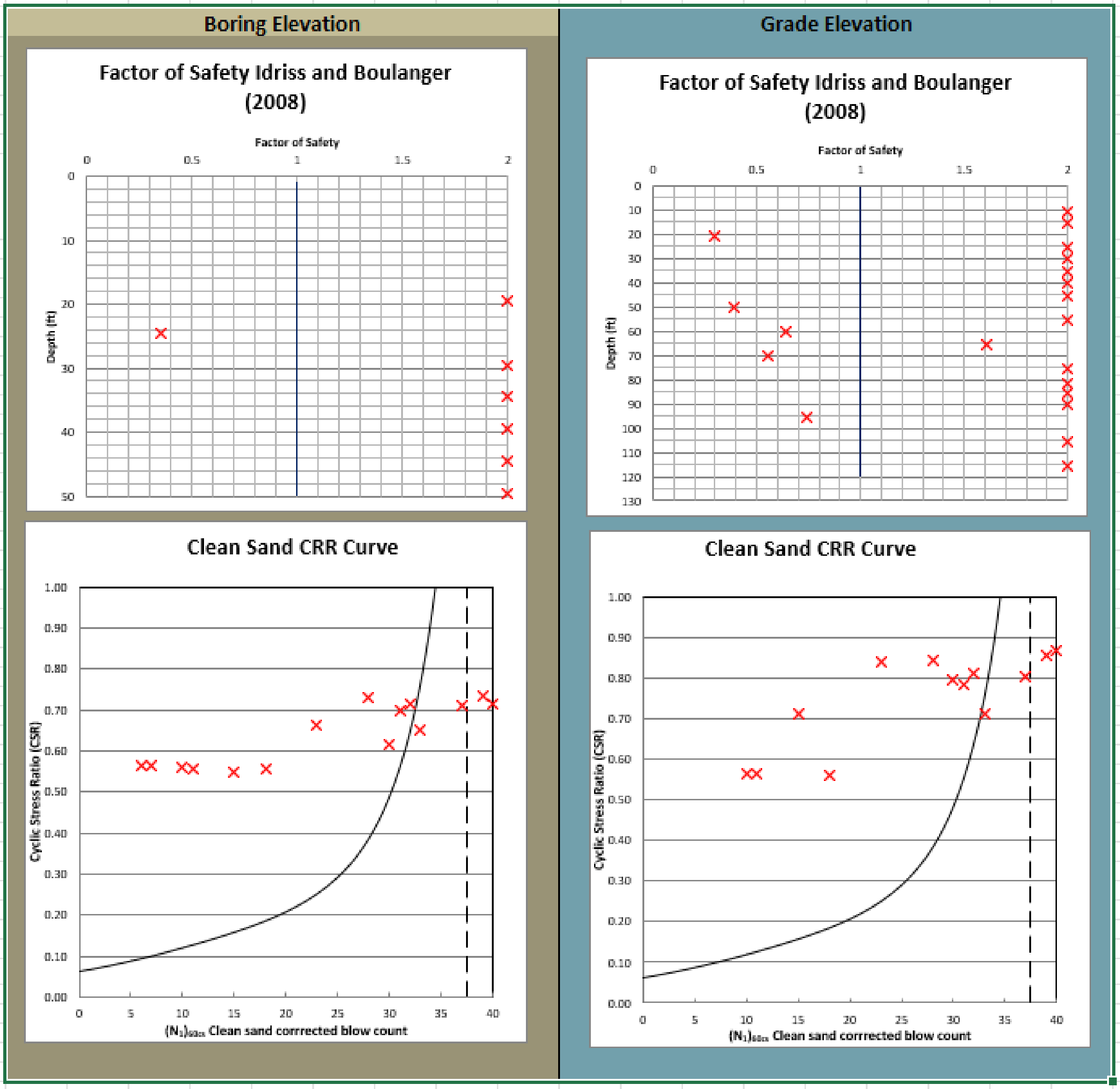
Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Ditch No. 1

Bent 4 / Boring C4

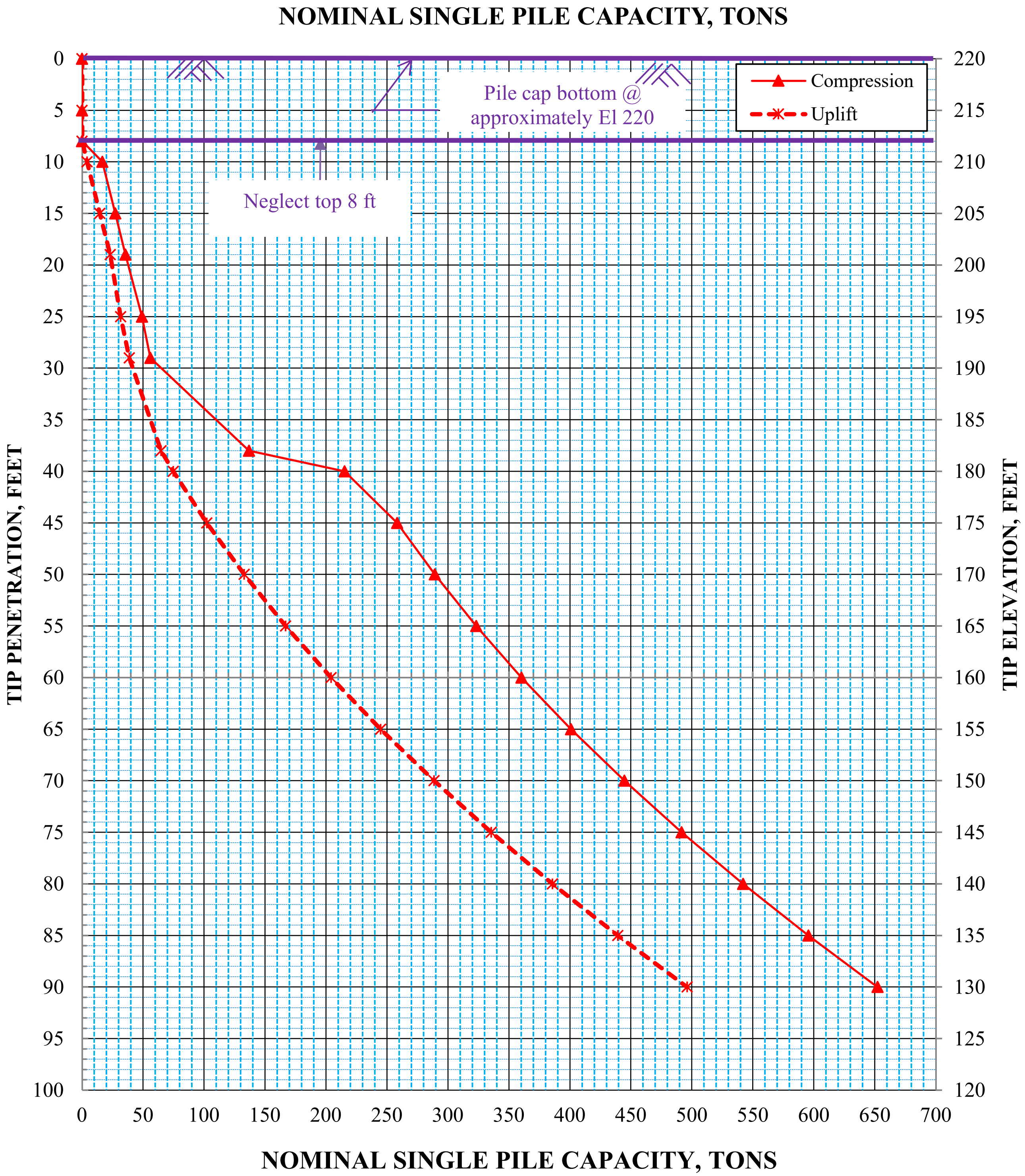
GHBW Job No. 23-031

Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

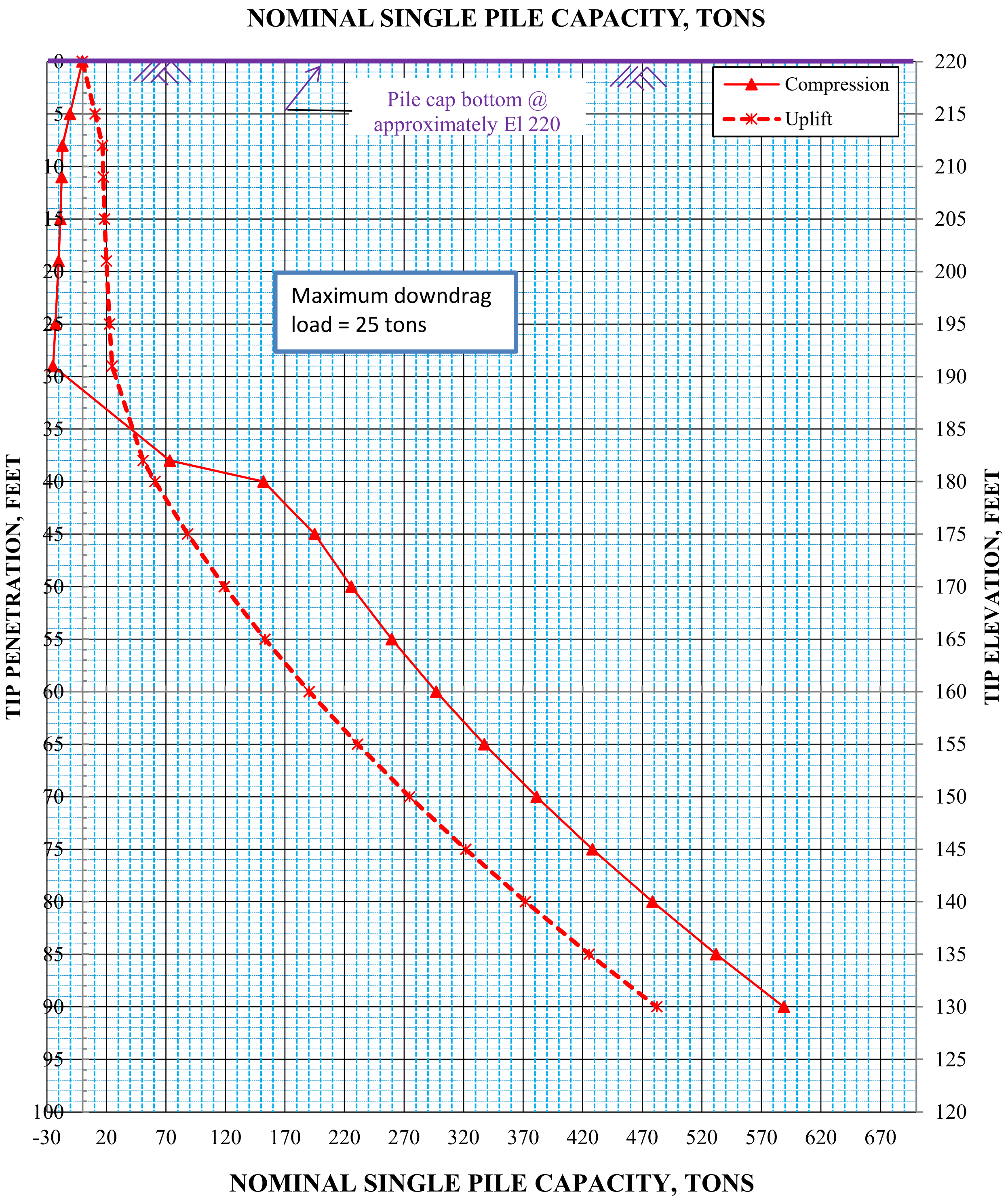
APPENDIX E



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

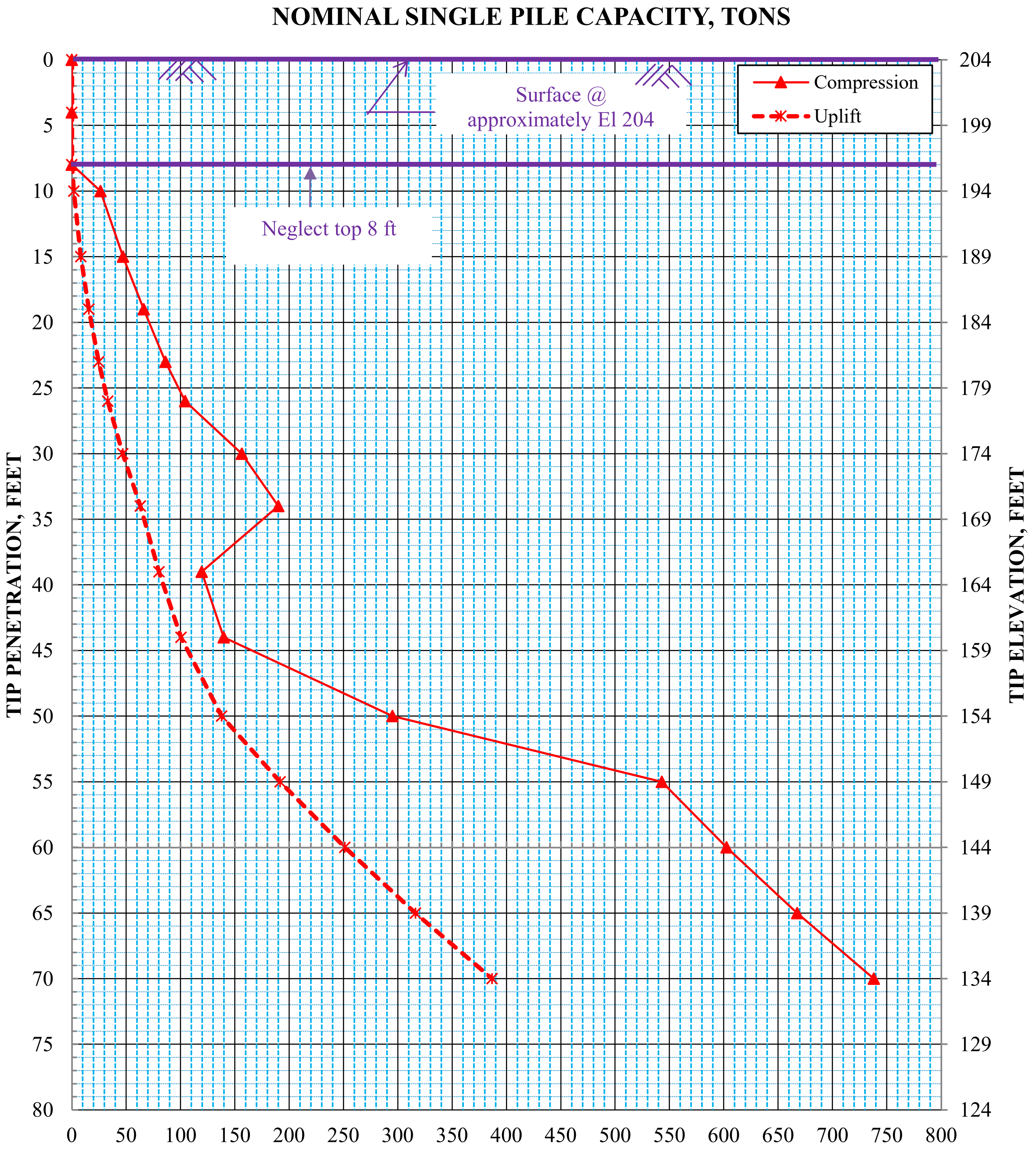
Notes: 1. Driven from cap bottom elevation
2. No downdrag



Bent 1 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 47
Poinsett County, Arkansas

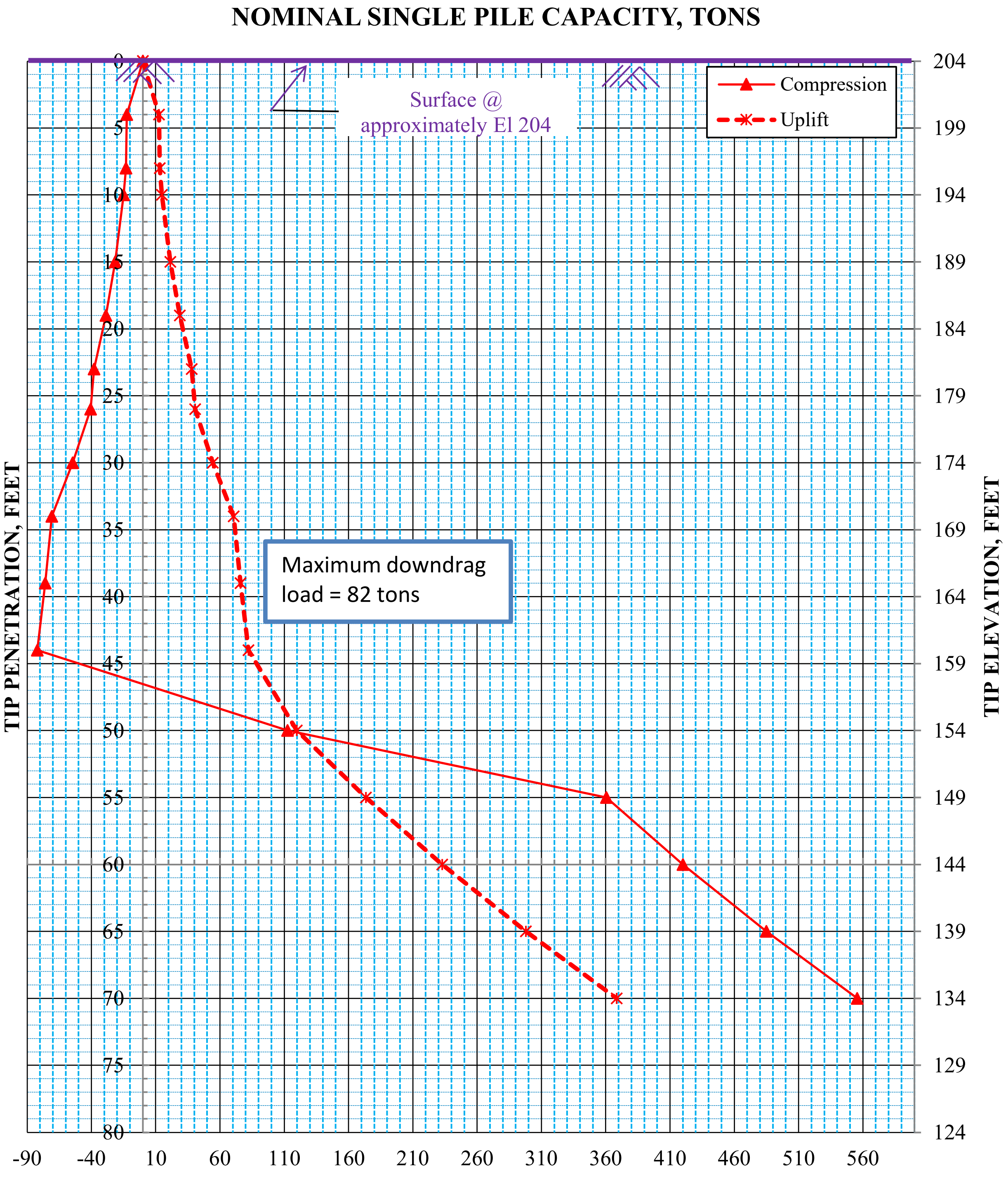
Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 191

Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 2 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

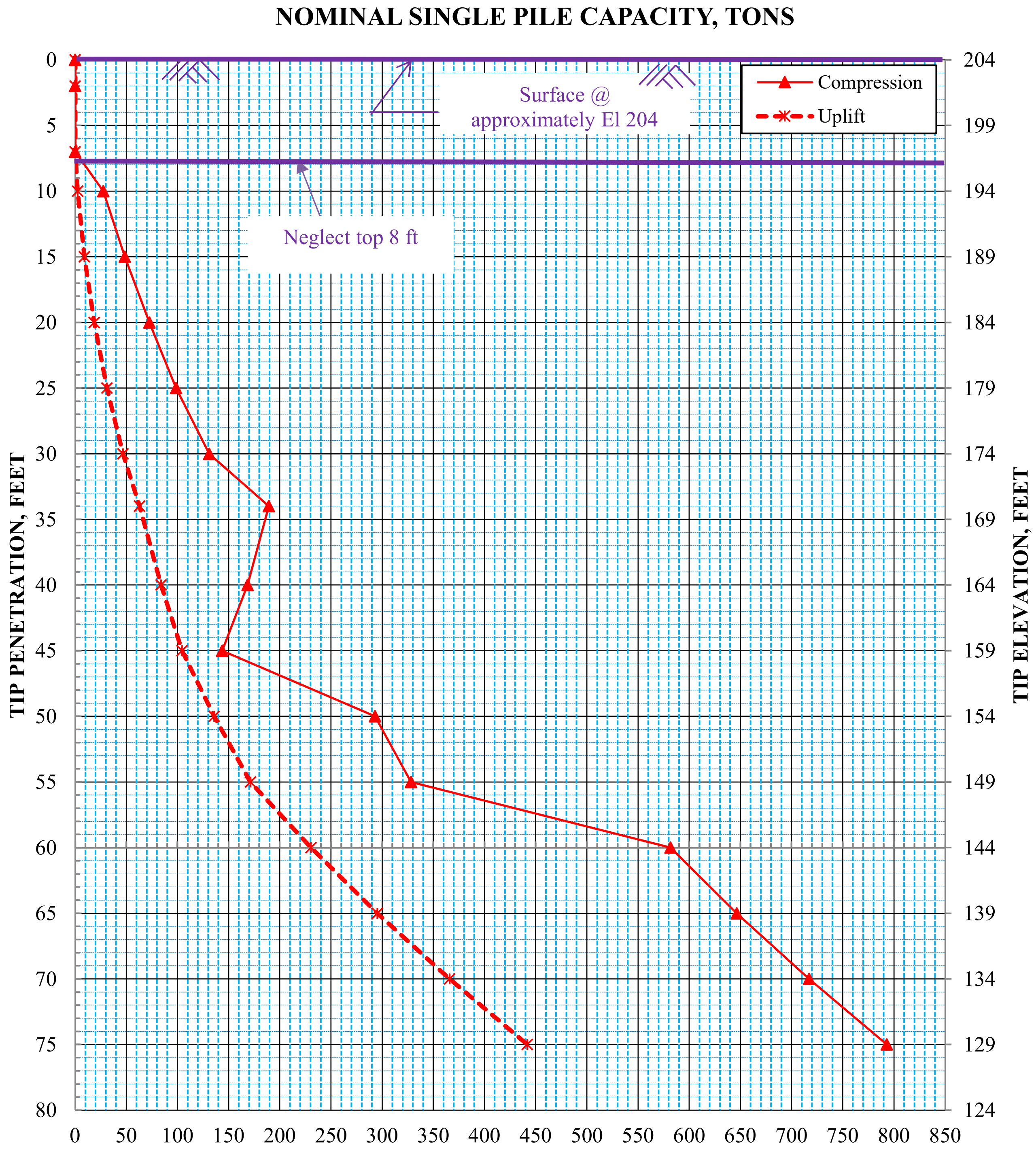
Notes: 1. Driven from channel bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

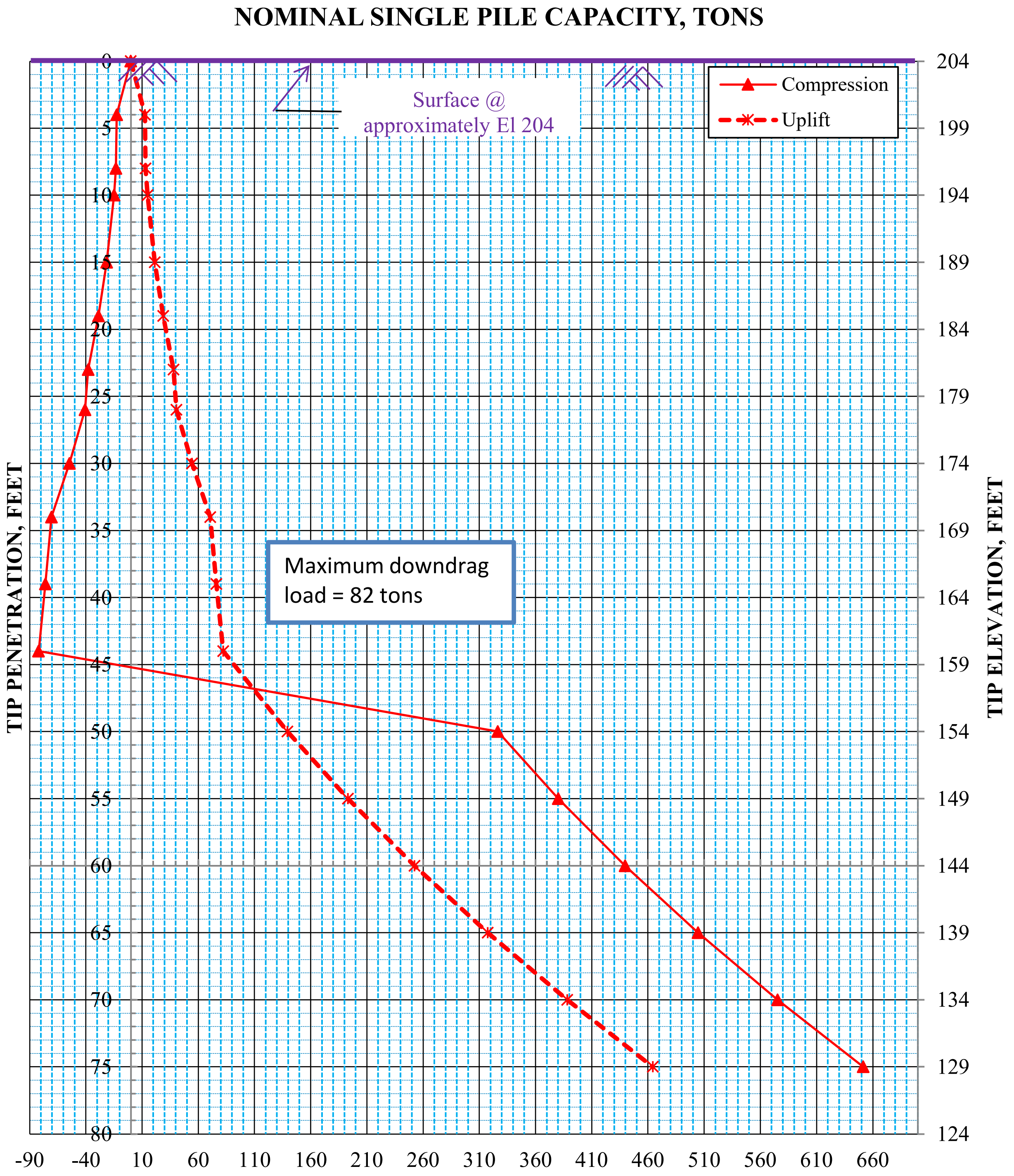
Bent 2 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 160



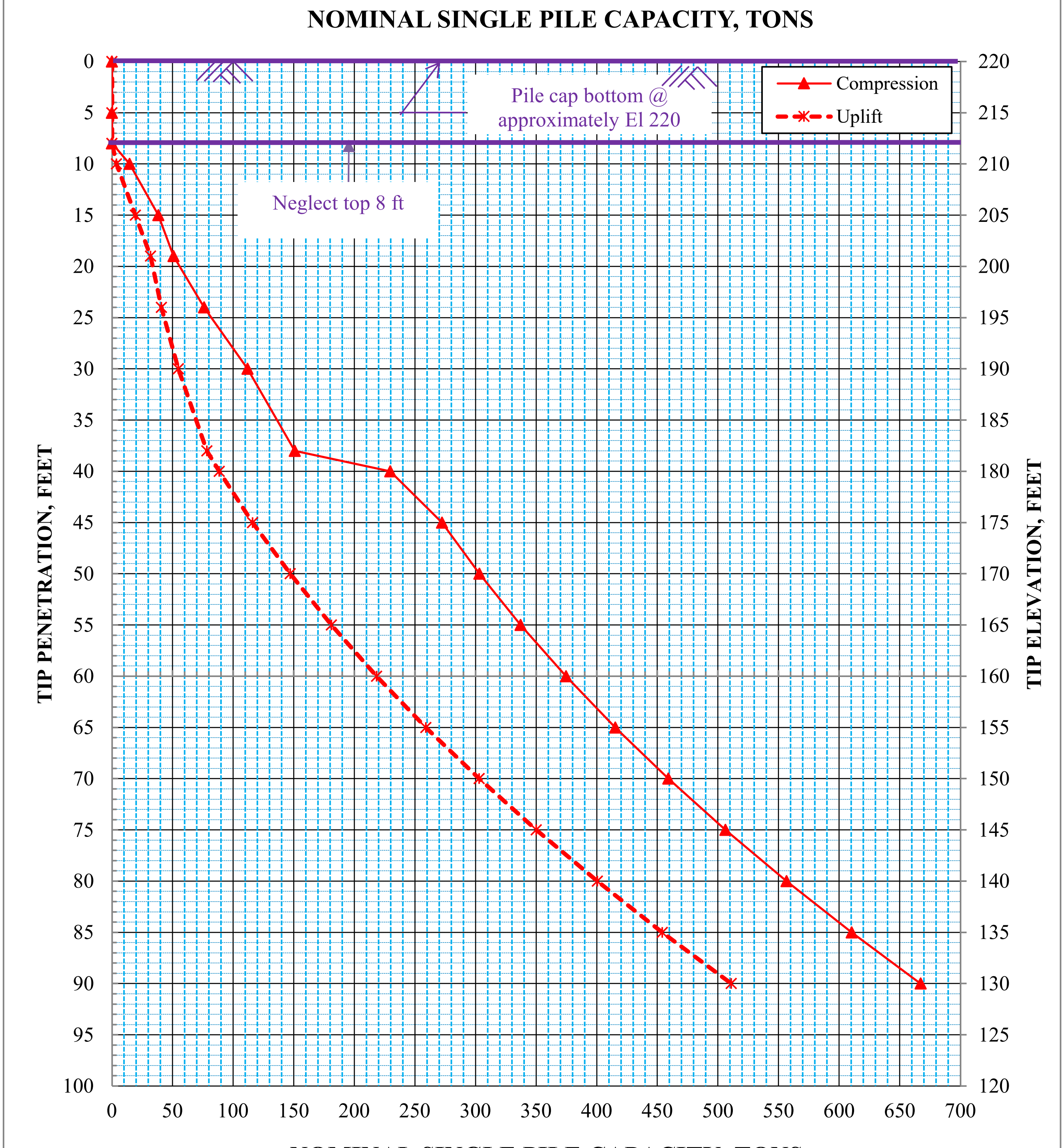
Bent 3 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 3 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

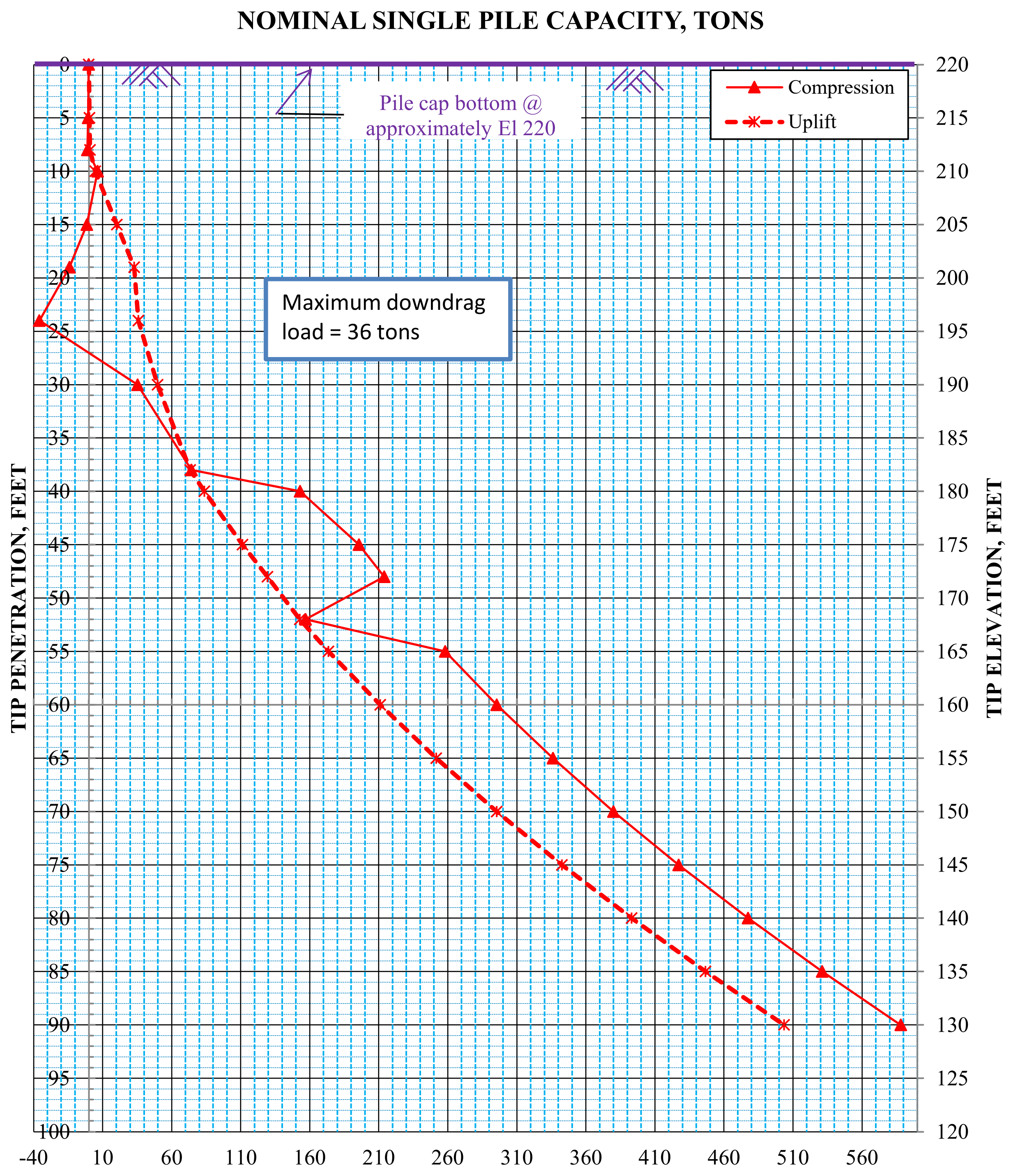
Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 160



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (North Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 1
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (North Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Ditch No. 47
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 196

APPENDIX F

SUMMARY OF LATERAL LOAD PARAMETERS
101124 Hwy. 135 over Ditch 1 (Site 3 / Bridge C)

PROJECT: Project: 101124 - Hwy 135 over Ditch No. 1 - Bent 1
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty, clayey fine SAND**	**Medium dense silty fine SAND**	**Dense fine SAND**
Depth below pile cap bottom, ft	0-8	8-19	19-29	29 and deeper
Approximate El, ft	220-212	212-201	201-191	below 191
Recommend soil type	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	59	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0
Angle of internal friction (φ), °	0	32	32	38
Subgrade modulus (k), lbs per cu in.	500	60	60	125
Strain at 50% (EE50)	0.007	NA	NA	NA

Note: Pile cap bottom at ±El 220

Seismic Loading with Liquefaction

Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty, clayey fine SAND (liquifiable)**	**Medium dense silty fine SAND (liquifiable)**	**Dense fine SAND**
Depth below pile cap bottom, ft	0-8	8-19	19-29	29 and deeper
Approximate El, ft	220-212	212-201	201-191	below 191
Recommend soil type	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	59	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0
Angle of internal friction (φ), °	0	8	8	38
Subgrade modulus (k), lbs per cu in.	500	20	20	125
Strain at 50% (EE50)	0.007	NA	NA	NA

Note: Pile cap bottom at ±El 220

SUMMARY OF LATERAL LOAD PARAMETERS
101124 Hwy. 135 over Ditch 1 (Site 3 / Bridge C)

PROJECT: Project: 101124 - Hwy 135 over Ditch No. 1 - Bent 2
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND**	**Medium dense to dense fine SAND**	**Medium dense silty fine SAND**	**Dense fine SAND**	**Medium dense fine SAND**	**Medium dense to very dense fine to medium SAND**
Depth below surface grade, ft	0-4	4-8	8-23	23-26	26-34	34-44	44 and deeper
Approximate El, ft	204-200	200-196	196-181	181-178	178-170	170-160	below 160
Recommend soil type	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	48	59	59	68	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0	0	0	0
Angle of internal friction (φ), °	0	32	34	32	35	32	38
Subgrade modulus (k), lbs per cu in.	500	60	60	60	125	60	125
Strain at 50% (EE50)	0.007	NA	NA	NA	NA	NA	NA

Note: Ground surface at ±El 204

Seismic Loading with Liquefaction

Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND (liquifiable)**	**Medium dense to dense fine SAND**	**Medium dense silty fine SAND (liquifiable)**	**Dense fine SAND**	**Medium dense fine SAND (liquifiable)**	**Medium dense to very dense fine to medium SAND**
Depth below surface grade, ft	0-4	4-8	8-23	23-26	26-34	34-44	44 and deeper
Approximate El, ft	204-200	200-196	196-181	181-178	178-170	170-160	below 160
Recommend soil type	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	48	59	59	68	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0	0	0	0
Angle of internal friction (φ), °	0	8	34	11	35	8	38
Subgrade modulus (k), lbs per cu in.	500	20	60	60	125	20	125
Strain at 50% (EE50)	0.007	NA	NA	NA	NA	NA	NA

Note: Ground surface at ±El 204

SUMMARY OF LATERAL LOAD PARAMETERS
101124 Hwy. 135 over Ditch 1 (Site 3 / Bridge C)

PROJECT: Project: 101124 - Hwy 135 over Ditch No. 1 - Bent 3
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND**	**Medium dense to dense fine SAND**	**Medium dense silty fine SAND**	**Dense fine SAND**
Depth below surface grade, ft	0-3	3-7	7-34	34-44	44 and deeper
Approximate El, ft	204-201	201-197	197-170	170-160	below 160
Recommend soil type	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	48	59	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0	0
Angle of internal friction (φ), °	0	32	34	32	38
Subgrade modulus (k), lbs per cu in.	500	60	60	60	125
Strain at 50% (EE50)	0.007	NA	NA	NA	NA

Note: Ground surface at ±El 204

Seismic Loading with Liquefaction

Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND (liquifiable)**	**Medium dense to dense fine SAND**	**Medium dense silty fine SAND (liquifiable)**	**Dense fine SAND**
Depth below surface grade, ft	0-3	3-7	7-34	34-44	44 and deeper
Approximate El, ft	204-201	201-197	197-170	170-160	below 160
Recommend soil type	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	48	59	59	68
Cohesion (c), lbs per sq ft	2000	0	0	0	0
Angle of internal friction (φ), °	0	8	34	32	38
Subgrade modulus (k), lbs per cu in.	500	20	60	20	125
Strain at 50% (EE50)	0.007	NA	NA	NA	NA

Note: Ground surface at ±El 204

SUMMARY OF LATERAL LOAD PARAMETERS
101124 Hwy. 135 over Ditch 1 (Site 3 / Bridge C)

PROJECT: Project: 101124 - Hwy 135 over Ditch No. 1 - Bent 4
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose to medium dense silty fine SAND**	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND**	**Dense fine SAND**	**Medium dense fine SAND**	**Dense fine SAND**
Depth below pile cap bottom, ft	0-8	8-19	19-24	24-48	48-52	52 and deeper
Approximate El, ft	220-212	212-201	201-196	196-172	172-168	below 168
Recommend soil type	Sand (Reese)	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	59	56	68	68	68
Cohesion (c), lbs per sq ft	0	1750	0	0	0	0
Angle of internal friction (φ), °	28	0	34	35	32	38
Subgrade modulus (k), lbs per cu in.	20	500	60	125	60	125
Strain at 50% (EE50)	NA	0.007	NA	NA	NA	NA

Note: Pile cap bottom at ±El 220

Seismic Loading with Liquefaction

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

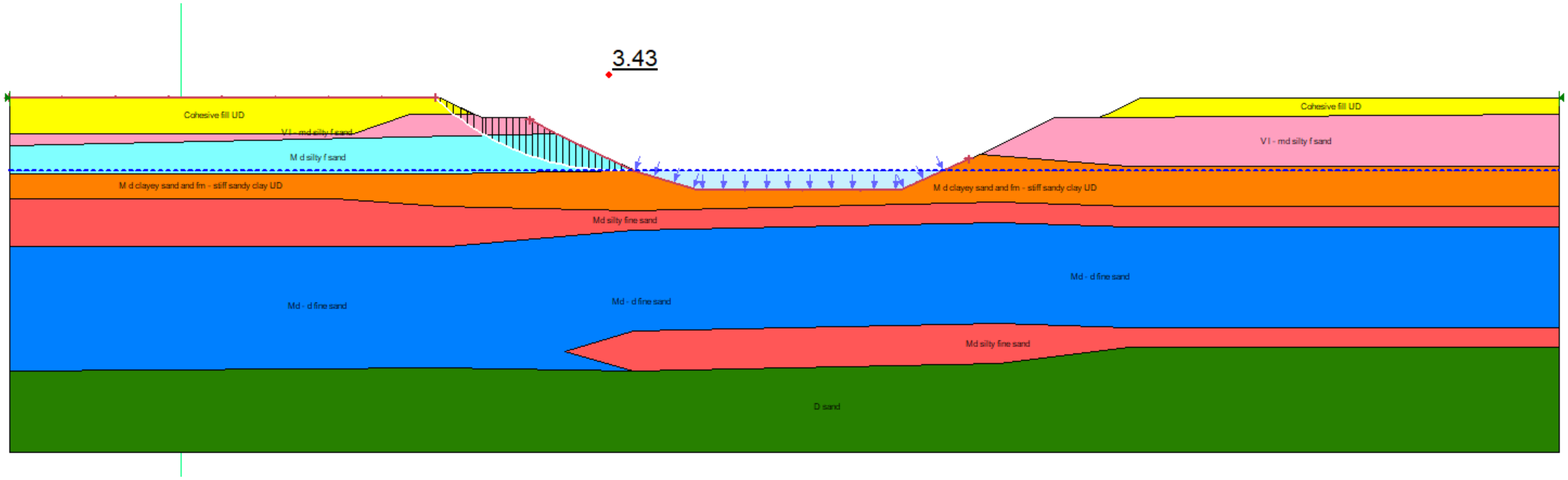
Generalized Stratigraphy	**Loose to medium dense silty fine SAND**	**Stiff fine sandy CLAY**	**Medium dense silty fine SAND (liquifiable)**	**Dense fine SAND**	**Medium dense fine SAND (liquifiable)**	**Dense fine SAND**
Depth below pile cap bottom, ft	0-8	8-19	19-24	24-48	48-52	52 and deeper
Approximate El, ft	220-212	212-201	201-196	196-172	172-168	below 168
Recommend soil type	Sand (Reese)	Stiff clay with free water	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	59	56	68	68	68
Cohesion (c), lbs per sq ft	0	1750	0	0	0	0
Angle of internal friction (φ), °	28	0	34	35	20	38
Subgrade modulus (k), lbs per cu in.	20	500	60	125	20	125
Strain at 50% (EE50)	NA	0.007	NA	NA	NA	NA

Note: Pile cap bottom at ±El 220

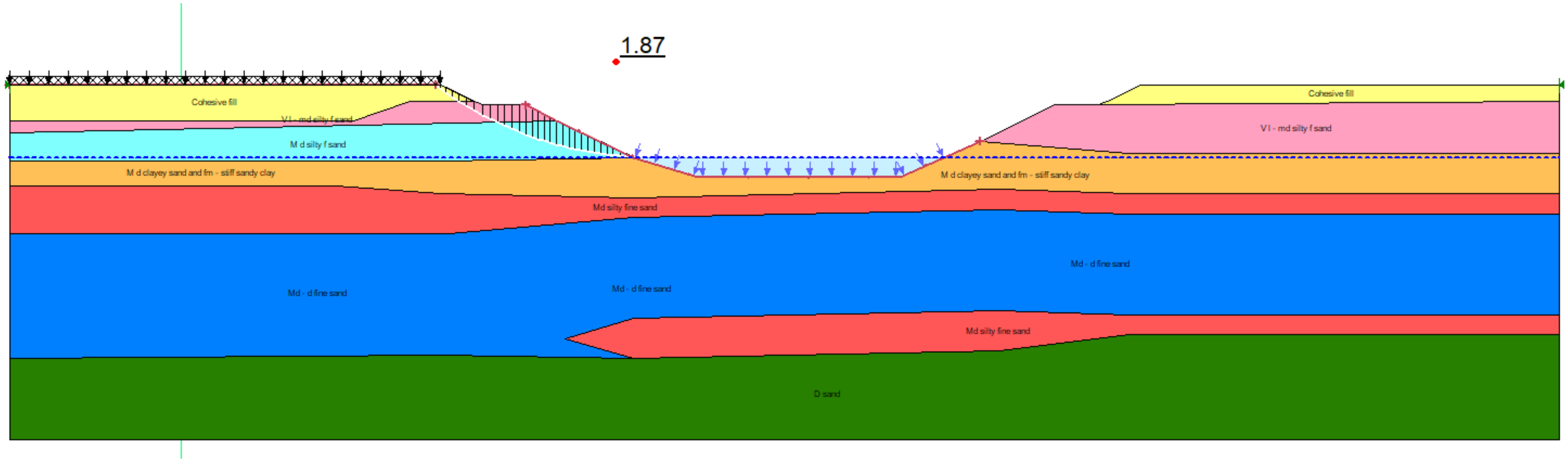
APPENDIX G

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Ditch No. 1
GHBW Job No. 23-031
Poinsett County, Arkansas

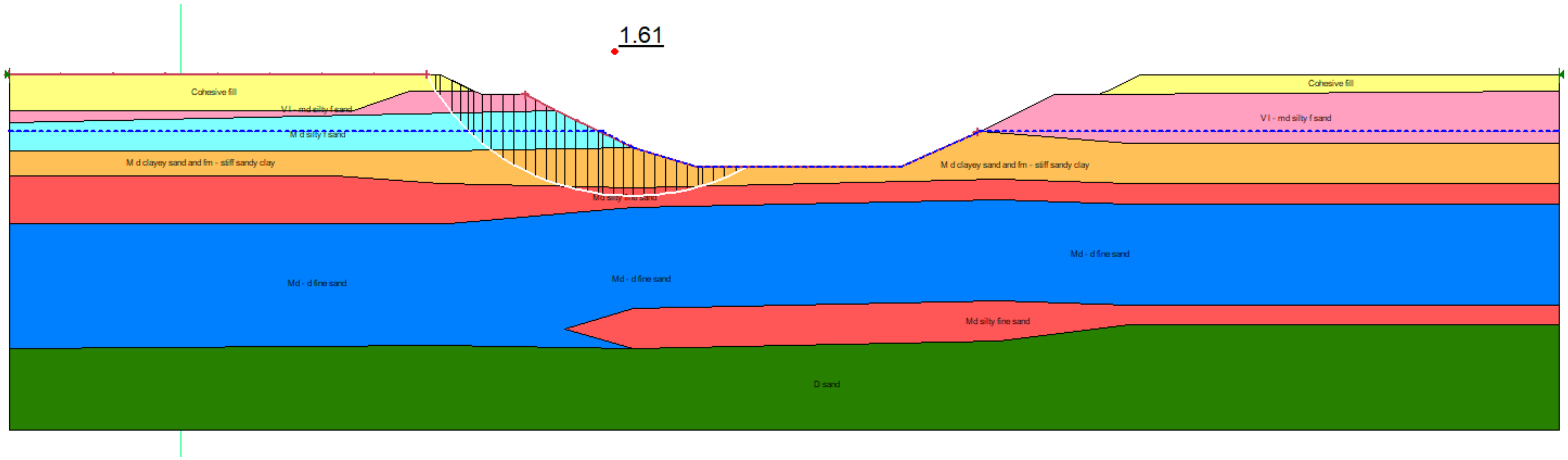
Embankment Slope	Design Loading Condition	Calculated Minimum Factor of Safety
South End Slope (Bent 1) (2H:1V)	End of Construction	3.43
	Long Term	1.87
	Rapid Drawdown from El 214 to El 205	1.61
	Seismic ($k_h = A_S/2 = 0.507$)	1.08
South Side Slope (Bent 1) (3H:1V)	End of Construction	7.58
	Long Term	5.22
	Rapid Drawdown from El 214 to Existing Grade	6.30
	Seismic ($k_h = A_S/2 = 0.507$)	1.38
North End Slope (Bent 4) (2H:1V)	End of Construction	3.48
	Long Term	1.92
	Rapid Drawdown from El 214 to El 205	1.53
	Seismic ($k_h = A_S/2 = 0.507$)	1.09
North Side Slope (Bent 4) (3H:1V)	End of Construction	7.00
	Long Term	2.39
	Rapid Drawdown from El 214 to Existing Grade	3.02
	Seismic ($k_h = A_S/2 = 0.507$)	1.35



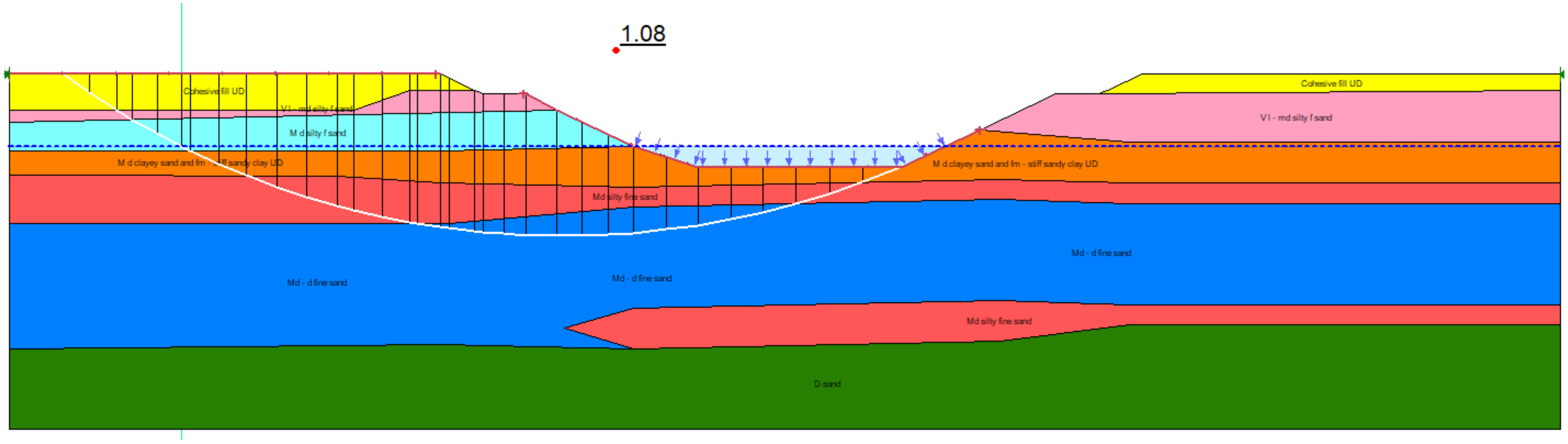
Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



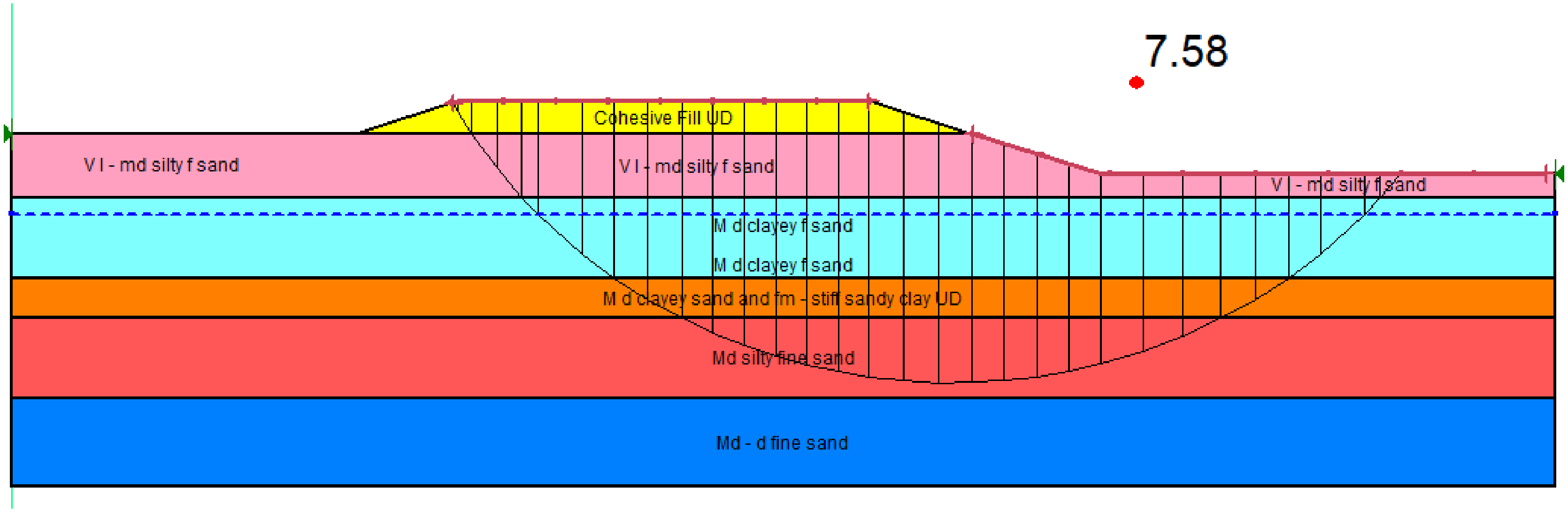
Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Rapid Drawdown Condition from El 214 to El 205
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.507$)
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – End of Construction
Bent 1 Side Slope
3H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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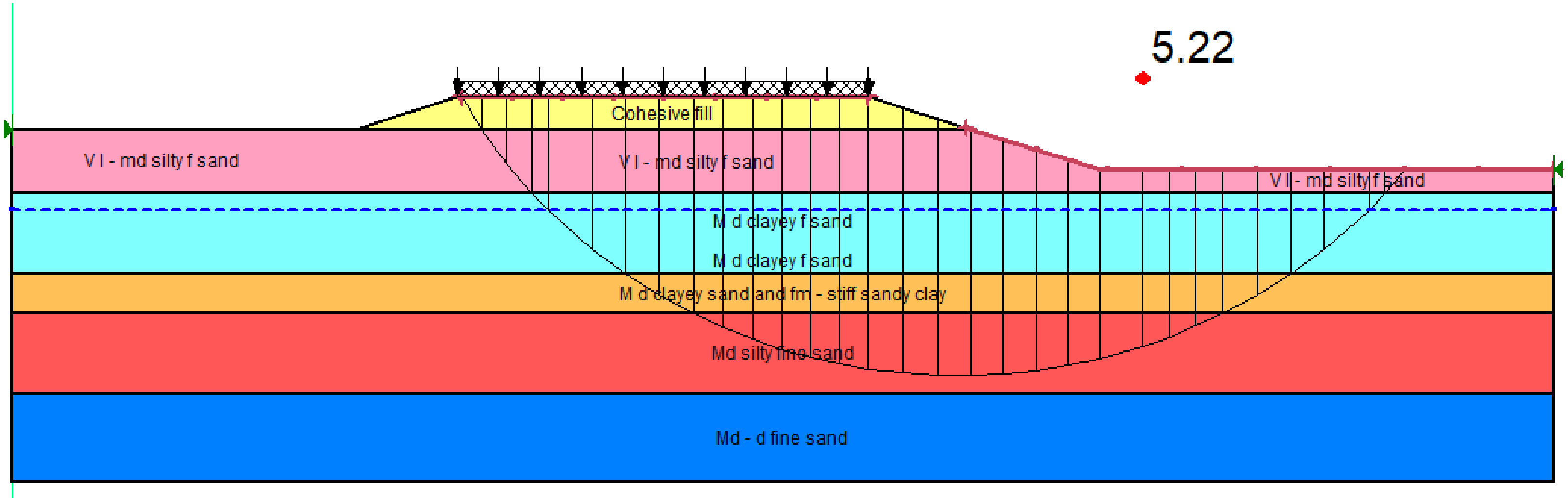
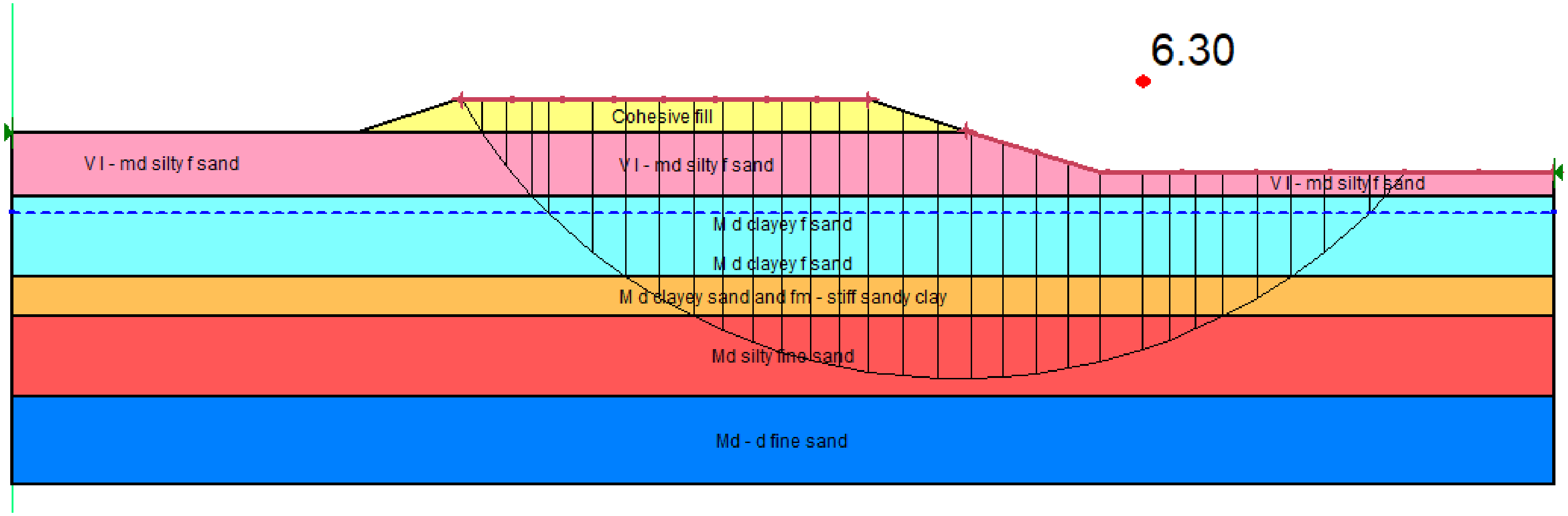
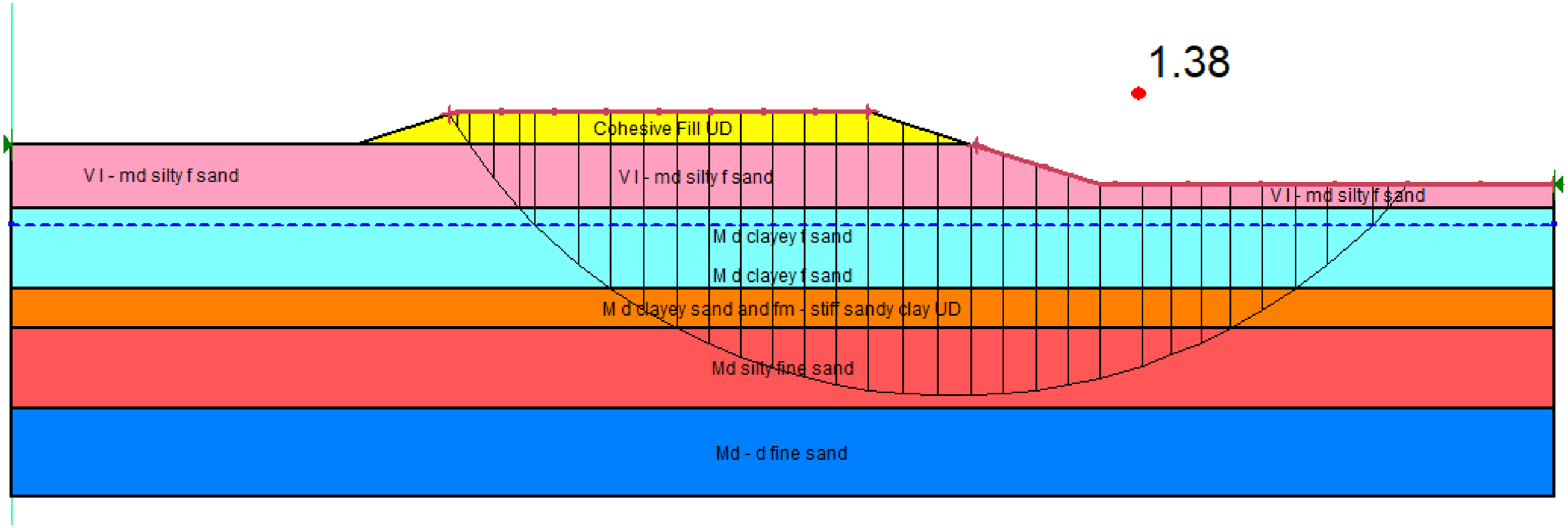


Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
3H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Rapid Drawdown El 214 to Existing Grade
Bent 1 Side Slope
3H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.507$)
Bent 1 Side Slope
3H:1V Slope, H=9 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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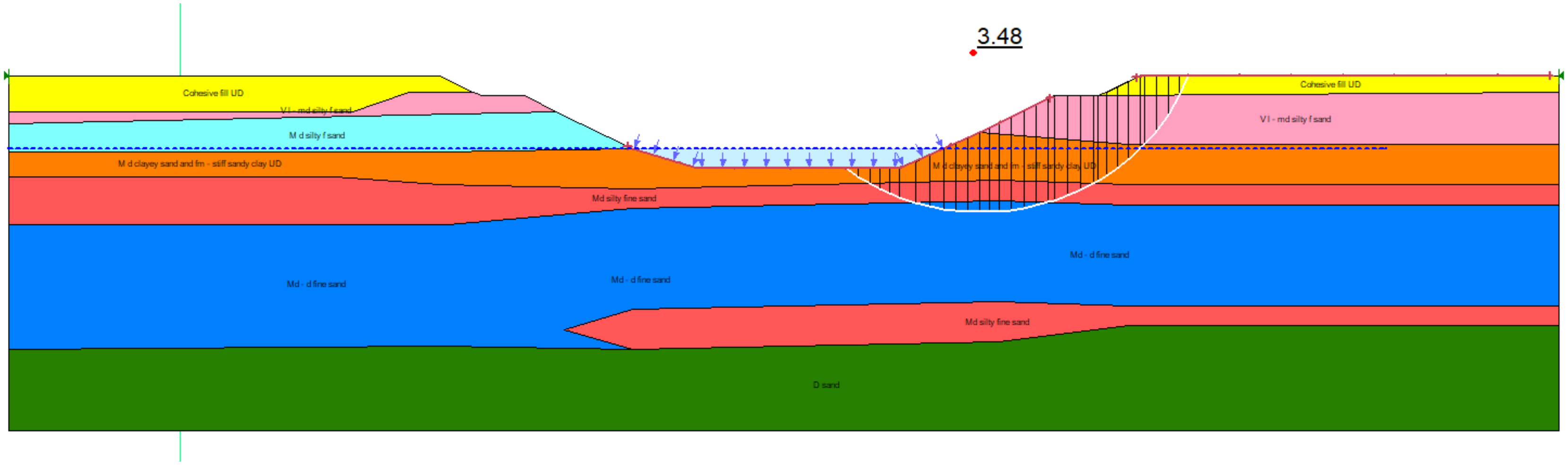
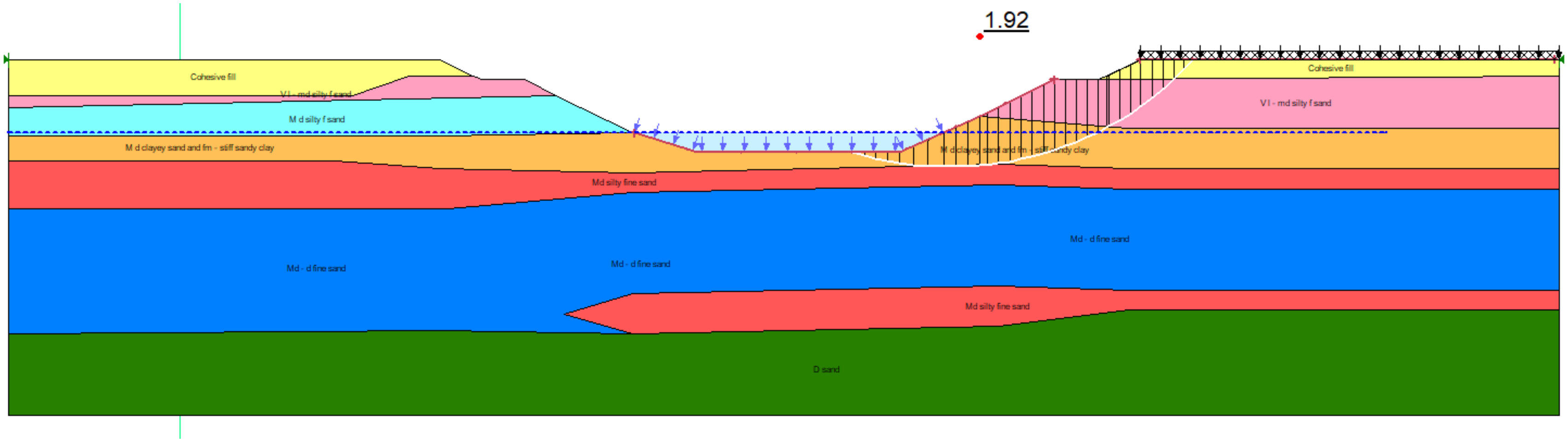
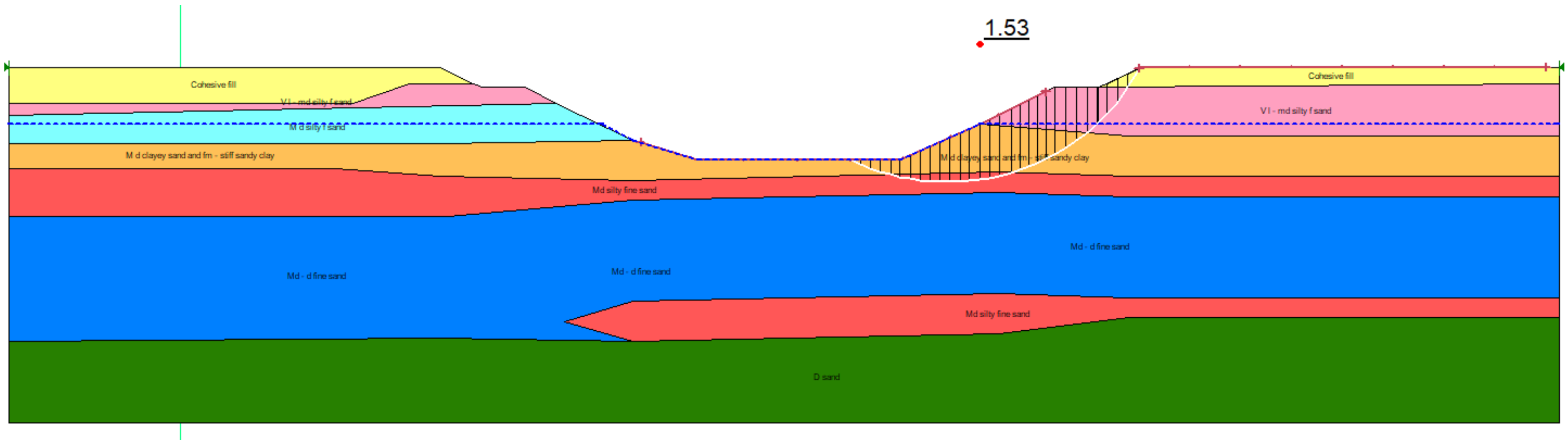


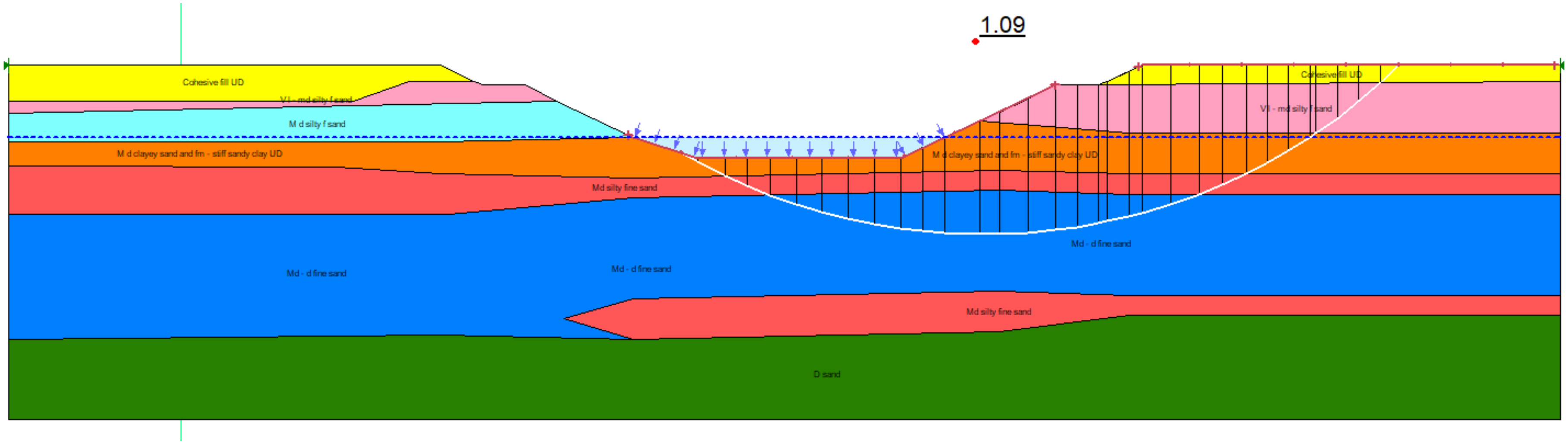
Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Rapid Drawdown Condition, El 214 to El 205
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1



Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.507)
Bent 4 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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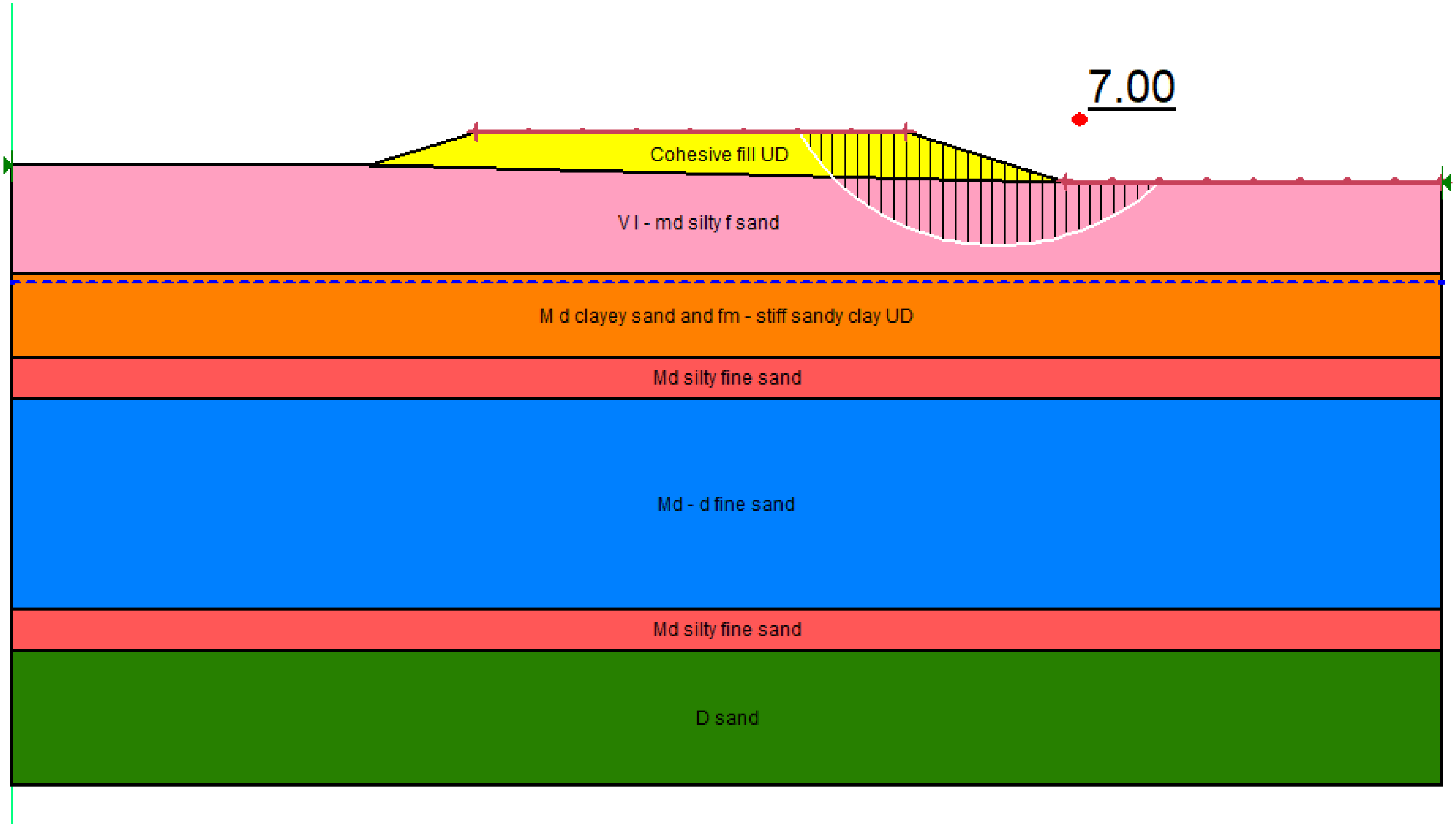

Results of Stability Analyses – End of Construction
Bent 4 Side Slope
3H:1V Slope, H=6 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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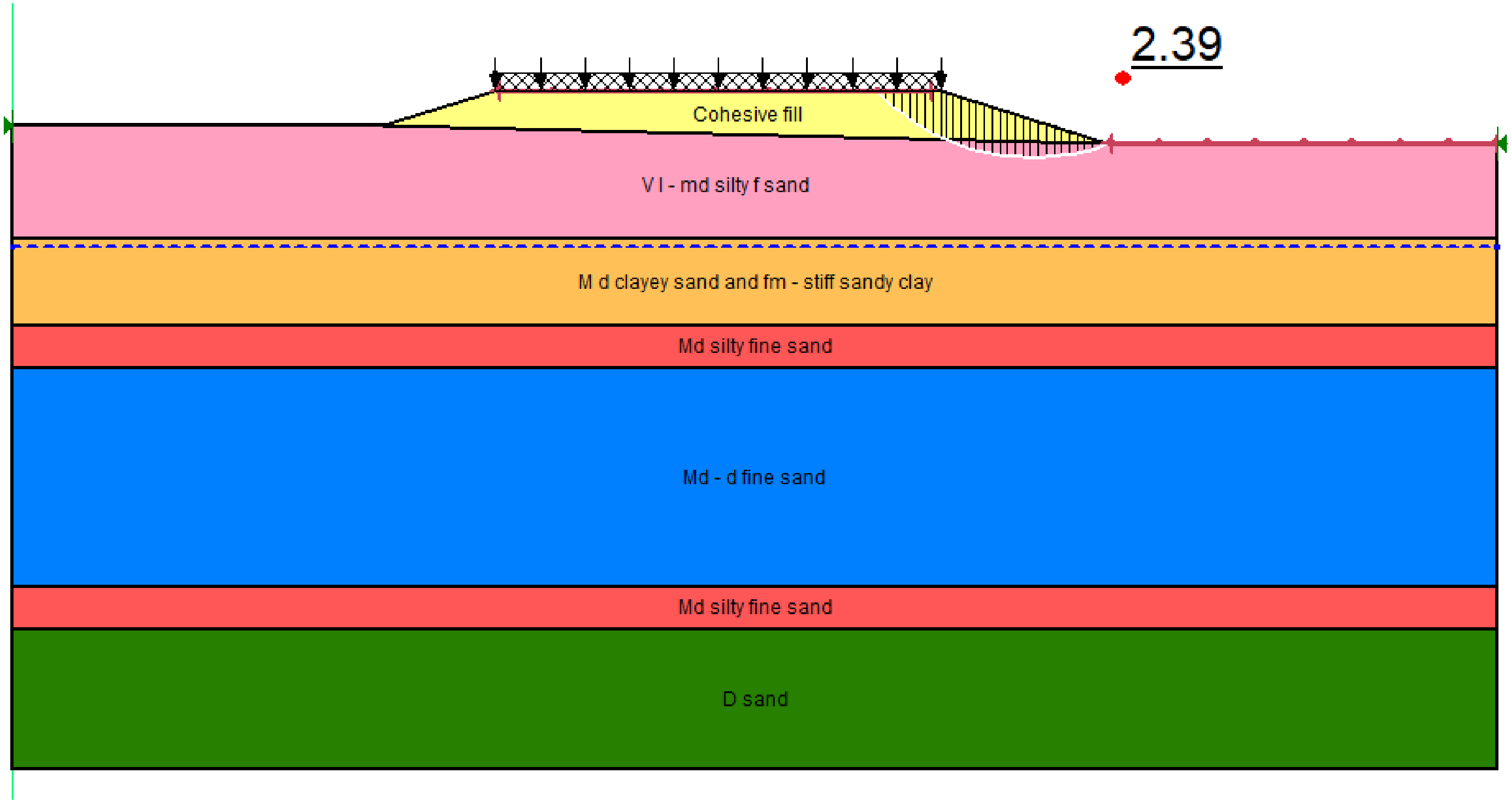

Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
3H:1V Slope, H=6 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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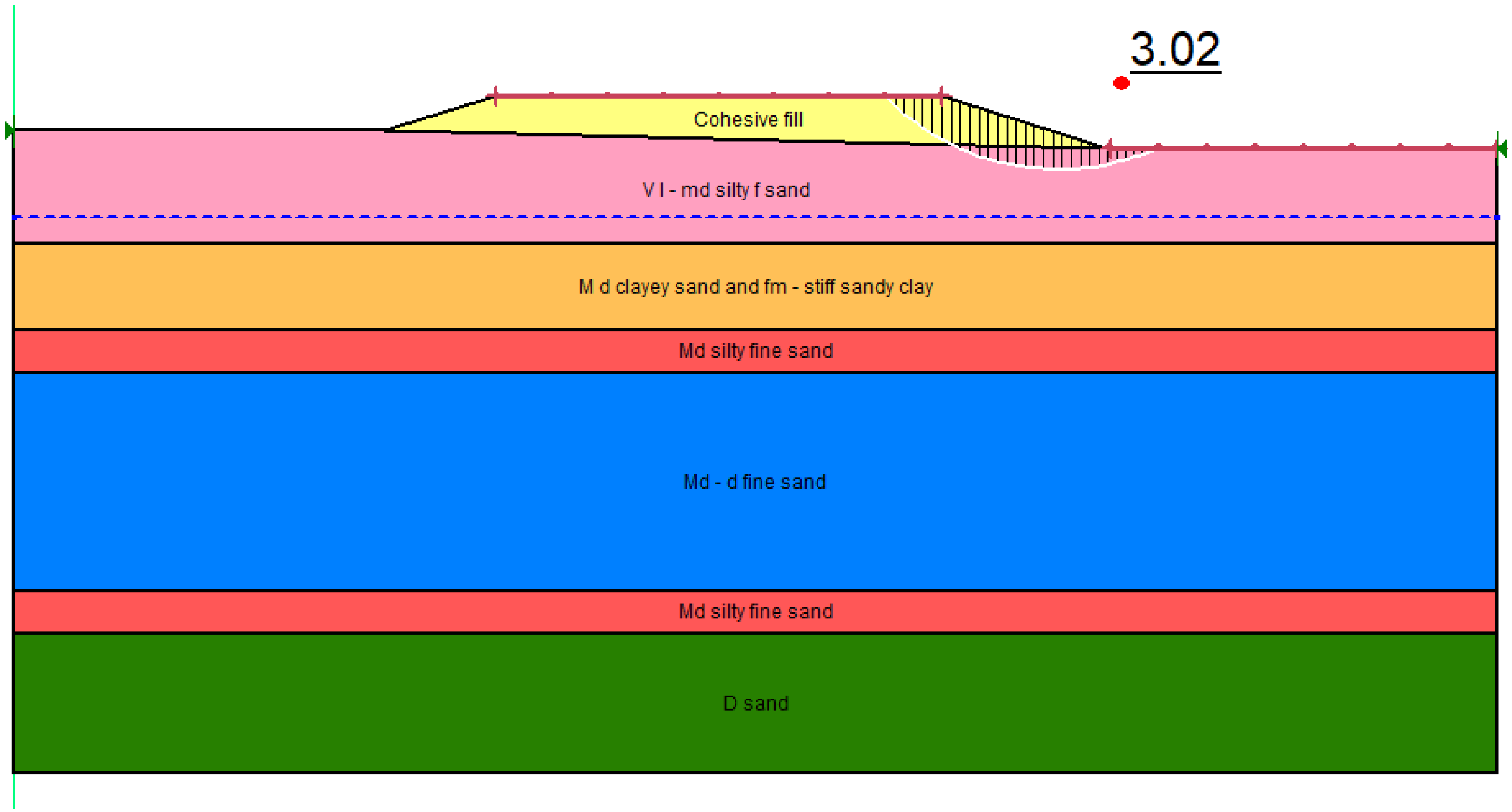

Results of Stability Analyses – Rapid Drawdown Condition, El 214 to Existing Grade
Bent 4 Side Slope
3H:1V Slope, H=6 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

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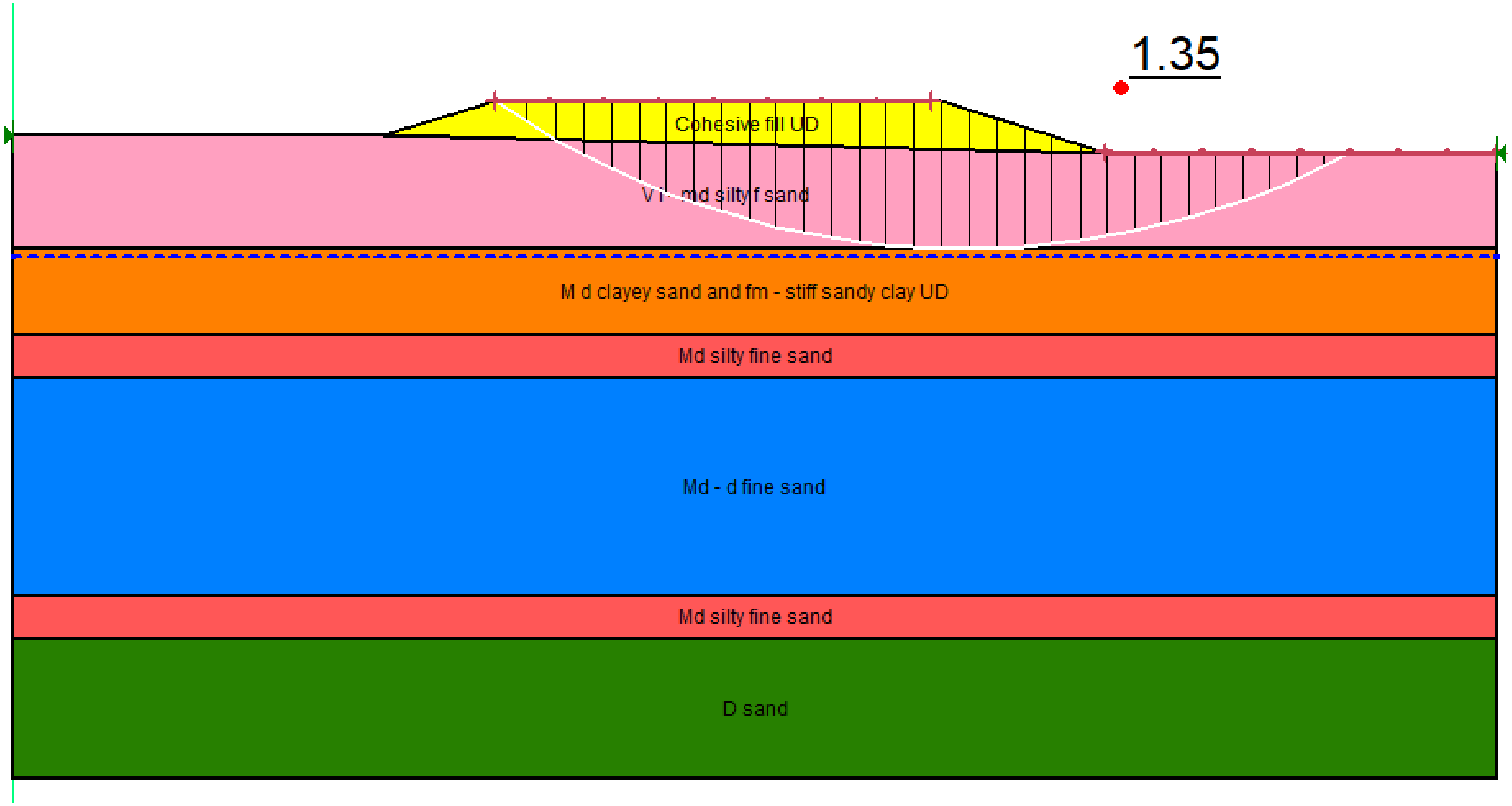


Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.507)
Bent 4 Side Slope
3H:1V Slope, H=6 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 1

APPENDIX H

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer’s installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX I

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, “The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project.” This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX J

SUMMARY of DRIVEABILITY ANALYSIS RESULTS

Project: 101124 - Hwy 135

Poinsett County, Arkansas

GHBW Project No: 23-031

Site	Bridge	Bent	Pile Diameter (in.)	Wall Thickness (in.)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El, ft	Min Tip El, ft	Pile Length, ft	Minimum Hammer Energy (ft-kip)	Max Comp Stress, ksi
Site 3 - Ditch No. 1	C	1	16	0.75	270	220	167	53	66	35.5
		2	24	0.50	455	208	150	58	122	35.7
		3	24	0.50	450	206	146	60	122	35.7
		4	16	0.75	230	220	161	59	66	34.5

Notes: 1. Driveability analyses performed utilizing GRLWEAP 2014; Pile Dynamics, Inc.

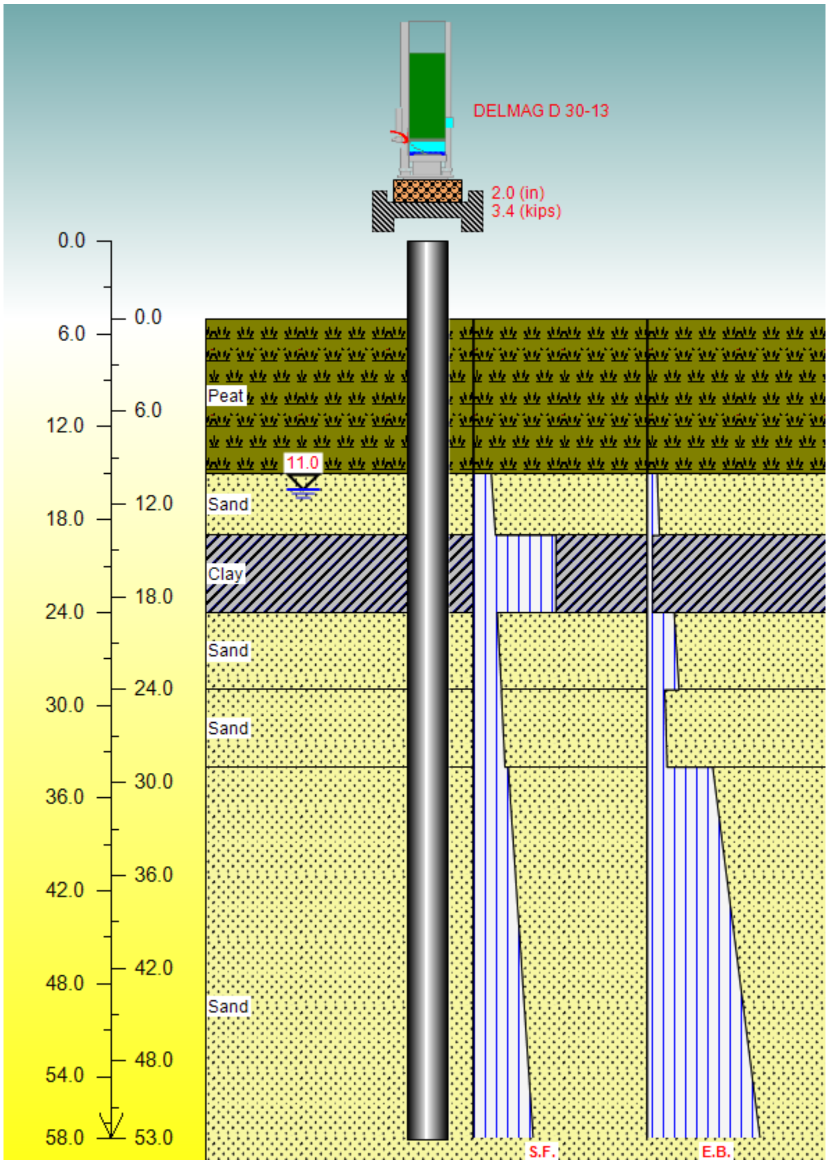
2. All piles are steel shells.

ArDOT 101124 Hwy 135 over Ditch No. 1

Bent 1

16-in-diameter Steel Shell Pile

Delmag D30-13



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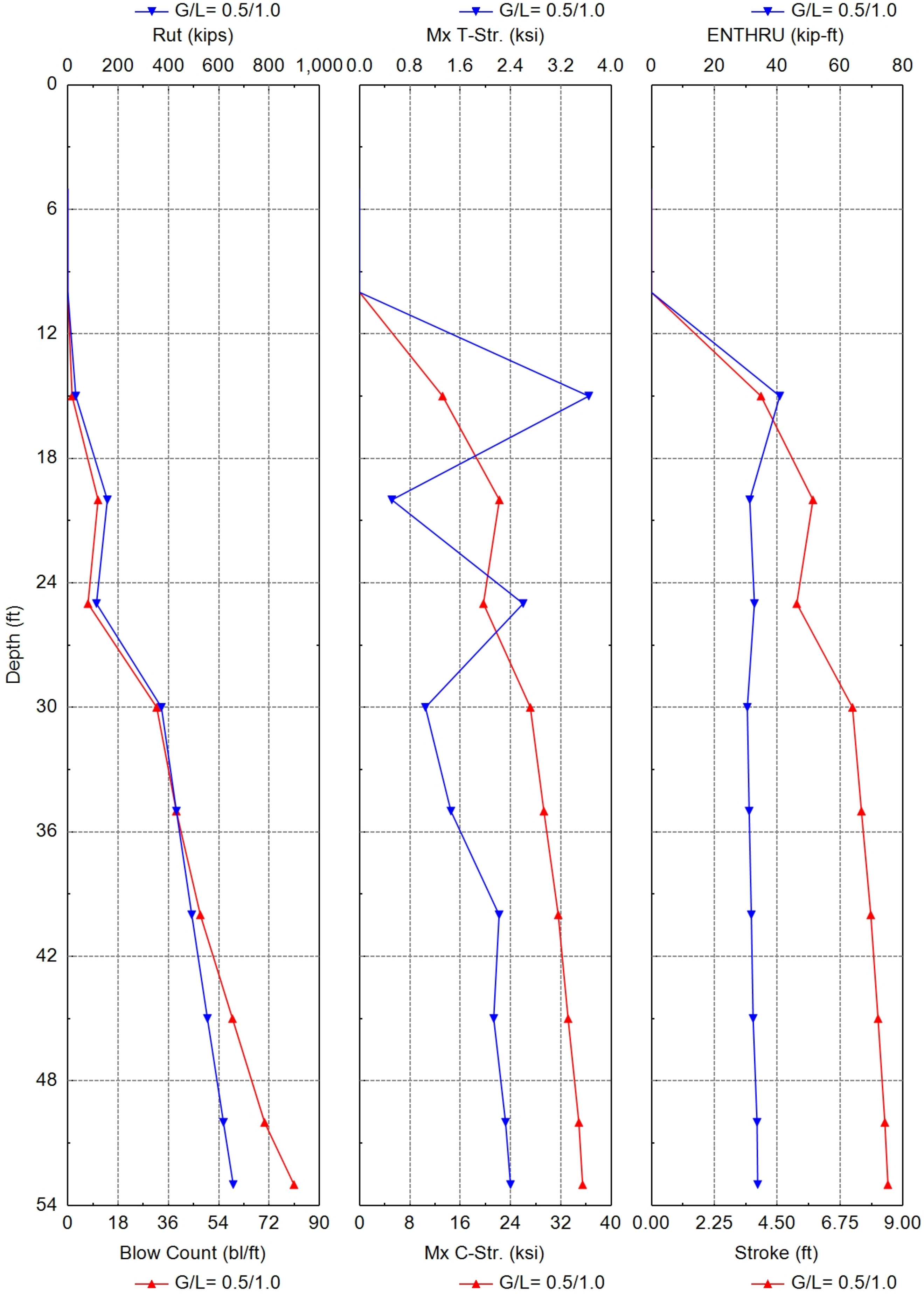

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
0
18
36
54
72
90
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.03	0.0	D 30-13
10.0	0.0	0.0	0.0	0.3	0.000	0.000	10.03	0.0	D 30-13
15.0	31.5	6.4	25.1	1.5	13.188	3.644	3.92	40.8	D 30-13
20.0	157.0	17.7	139.3	10.8	22.219	0.512	5.78	31.3	D 30-13
25.0	113.7	24.3	89.3	7.2	19.685	2.600	5.20	32.7	D 30-13
30.0	372.0	32.1	339.9	31.8	27.142	1.046	7.19	30.5	D 30-13
35.0	431.7	41.8	389.9	38.8	29.297	1.451	7.51	31.1	D 30-13
40.0	492.7	52.7	439.9	47.4	31.581	2.218	7.85	31.8	D 30-13
45.0	555.0	65.0	490.0	58.9	33.145	2.132	8.11	32.3	D 30-13
50.0	618.6	78.6	540.0	70.4	34.861	2.320	8.35	33.6	D 30-13
53.0	657.5	87.4	570.0	80.9	35.456	2.399	8.46	33.8	D 30-13

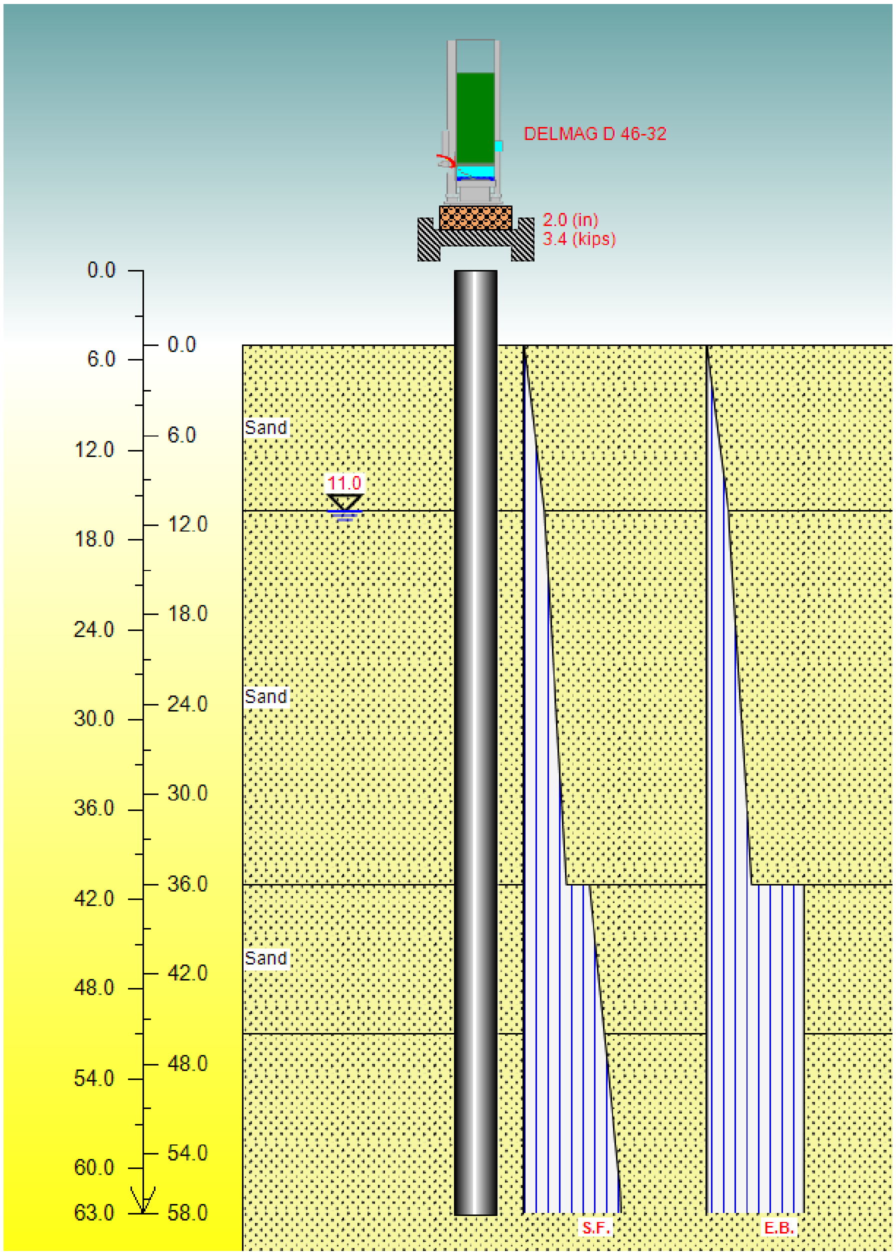
Total driving time: 32 minutes; Total Number of Blows: 1386 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Ditch No. 1

Bent 2

24-in-diameter Steel Shell Pile

Delmag D46-32



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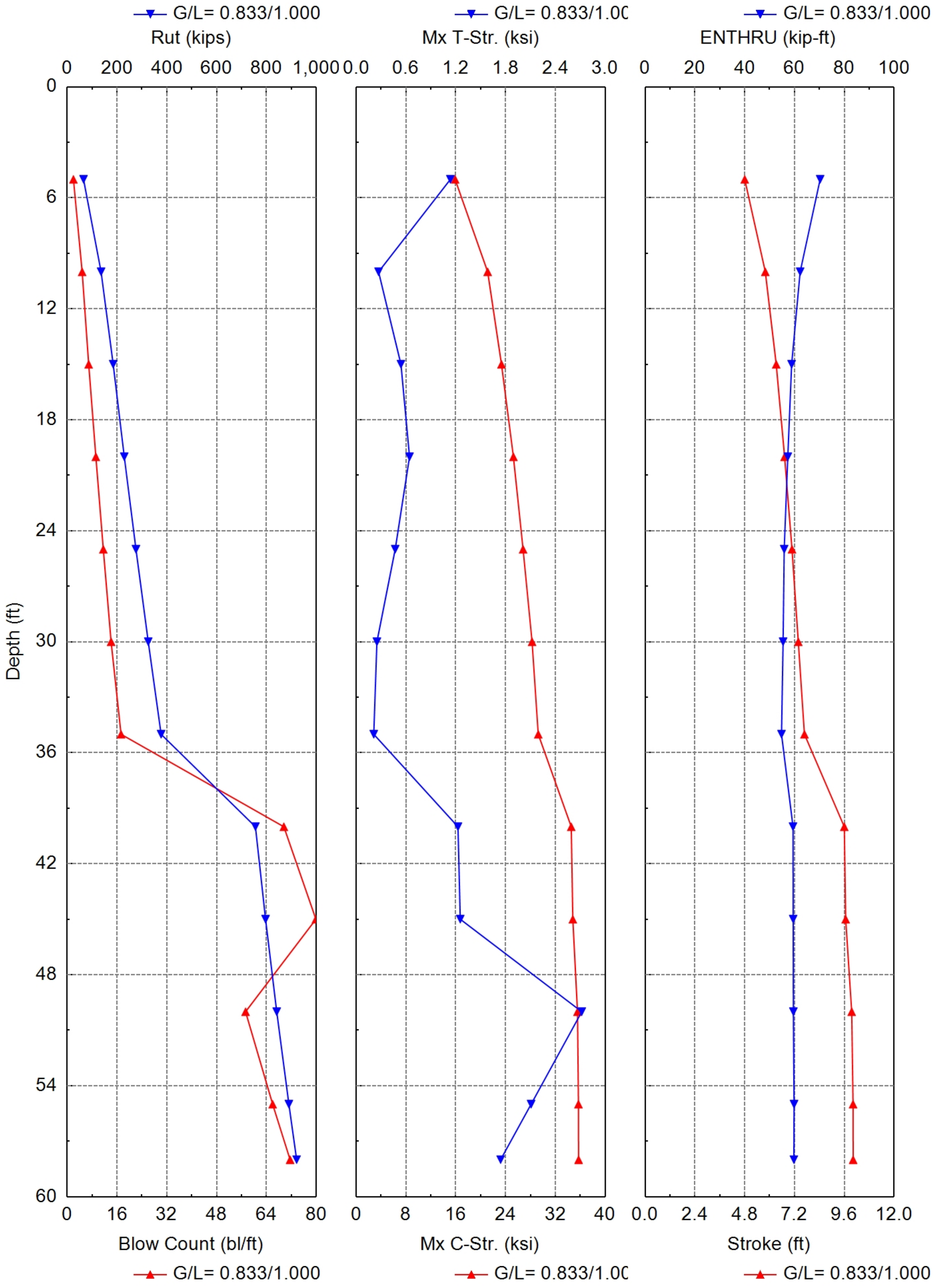
A UES Company


G/L= 0.833/1.000
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
20
40
60
80
100
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
16
32
48
64
80
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	65.6	2.5	63.1	2.0	15.898	1.136	4.80	70.3	D 46-32
10.0	136.1	9.9	126.2	4.8	21.128	0.270	5.80	62.3	D 46-32
15.0	184.7	21.5	163.2	6.9	23.347	0.540	6.32	58.9	D 46-32
20.0	229.2	35.5	193.7	9.2	25.263	0.643	6.72	57.3	D 46-32
25.0	276.1	51.9	224.2	11.6	26.816	0.471	7.08	55.9	D 46-32
30.0	325.4	70.7	254.7	14.1	28.261	0.251	7.38	55.4	D 46-32
35.0	377.1	91.9	285.2	17.3	29.251	0.212	7.68	54.8	D 46-32
40.0	755.9	126.0	629.9	69.6	34.551	1.227	9.60	59.4	D 46-32
45.0	796.6	166.7	629.9	79.9	34.805	1.254	9.67	59.5	D 46-32
50.0	841.3	211.4	629.9	57.3	35.553	2.718	9.96	59.6	D 46-32
55.0	890.2	260.3	629.9	66.0	35.708	2.108	10.02	59.8	D 46-32
58.0	921.3	291.4	629.9	71.6	35.732	1.741	10.03	59.8	D 46-32

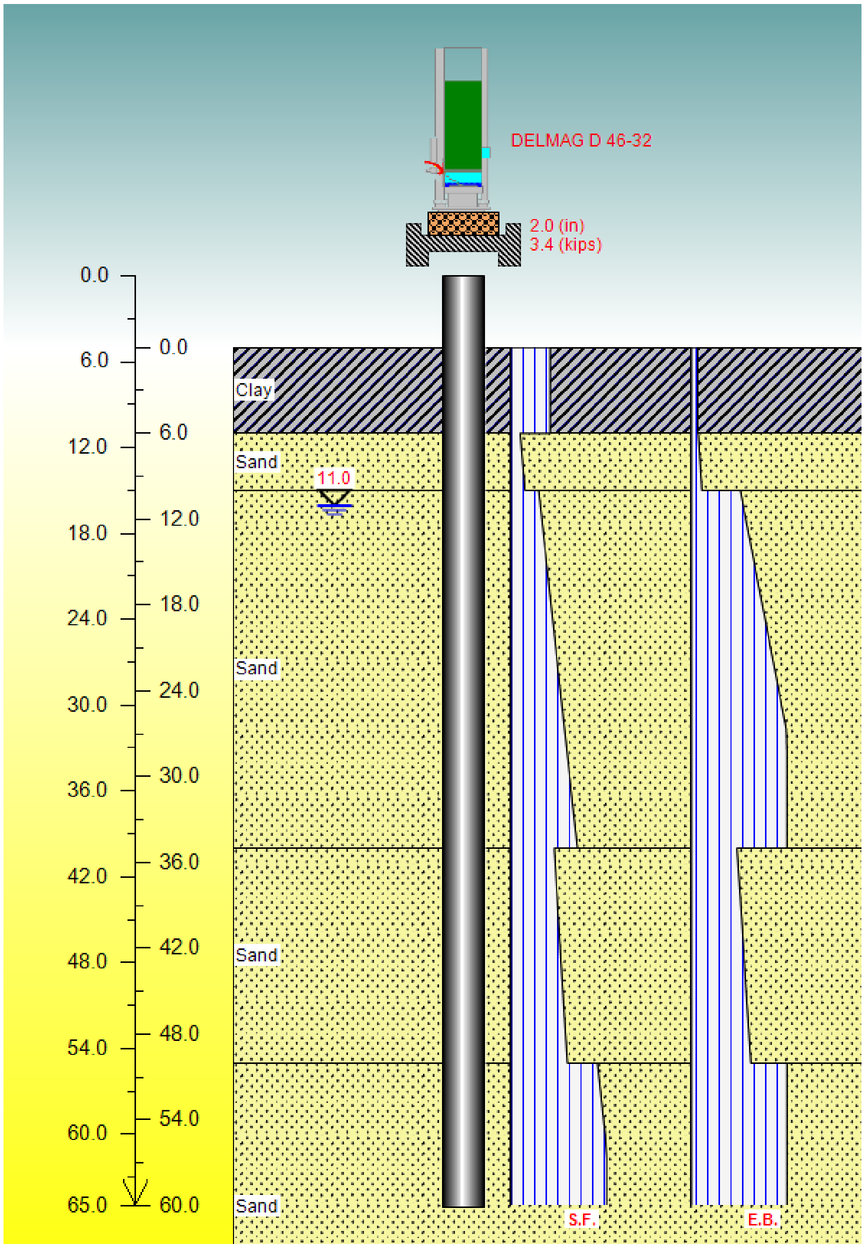
Total driving time: 44 minutes; Total Number of Blows: 1728 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Ditch No. 1

Bent 3

24-in-diameter Steel Shell Pile

Delmag D46-32



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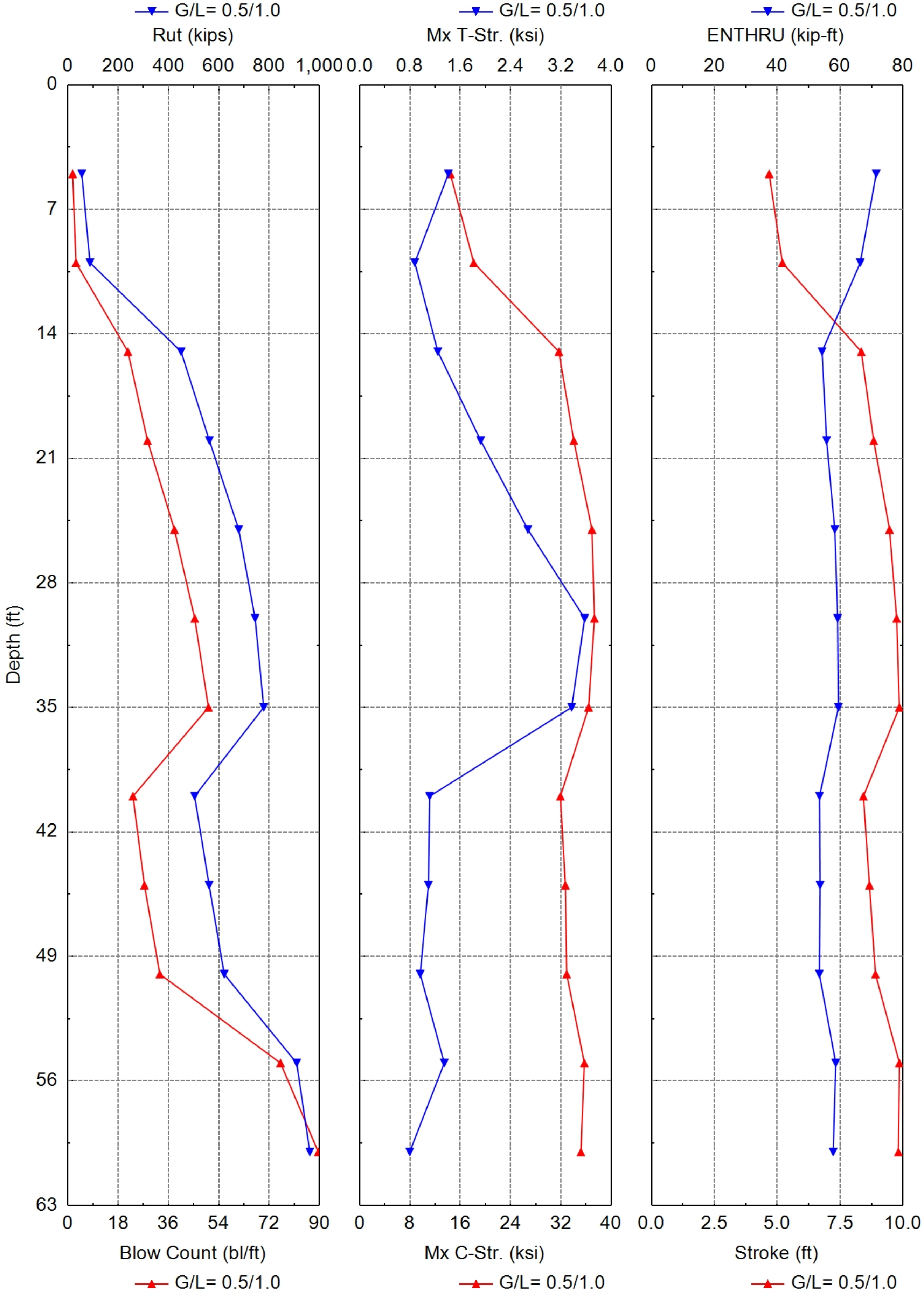

G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
7
14
21
28
35
42
49
56
63
0
18
36
54
72
90
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	55.4	12.8	42.6	1.7	14.417	1.411	4.68	71.5	D 46-32
10.0	88.0	20.4	67.7	2.9	18.139	0.877	5.21	66.4	D 46-32
15.0	450.2	37.5	412.6	21.5	31.681	1.243	8.34	54.3	D 46-32
20.0	562.6	59.0	503.6	28.5	34.054	1.924	8.84	55.7	D 46-32
25.0	679.3	84.7	594.6	38.1	36.942	2.675	9.47	58.3	D 46-32
30.0	744.5	114.6	629.9	45.4	37.337	3.578	9.75	59.2	D 46-32
35.0	778.7	148.9	629.9	50.3	36.413	3.372	9.86	59.5	D 46-32
40.0	503.9	173.6	330.3	23.3	31.928	1.113	8.43	53.4	D 46-32
45.0	561.6	200.8	360.8	27.4	32.713	1.094	8.67	53.6	D 46-32
50.0	621.7	230.3	391.3	32.9	32.924	0.963	8.91	53.4	D 46-32
55.0	909.7	279.8	629.9	76.1	35.737	1.346	9.86	58.6	D 46-32
60.0	962.0	332.1	629.9	89.6	35.189	0.796	9.82	57.8	D 46-32

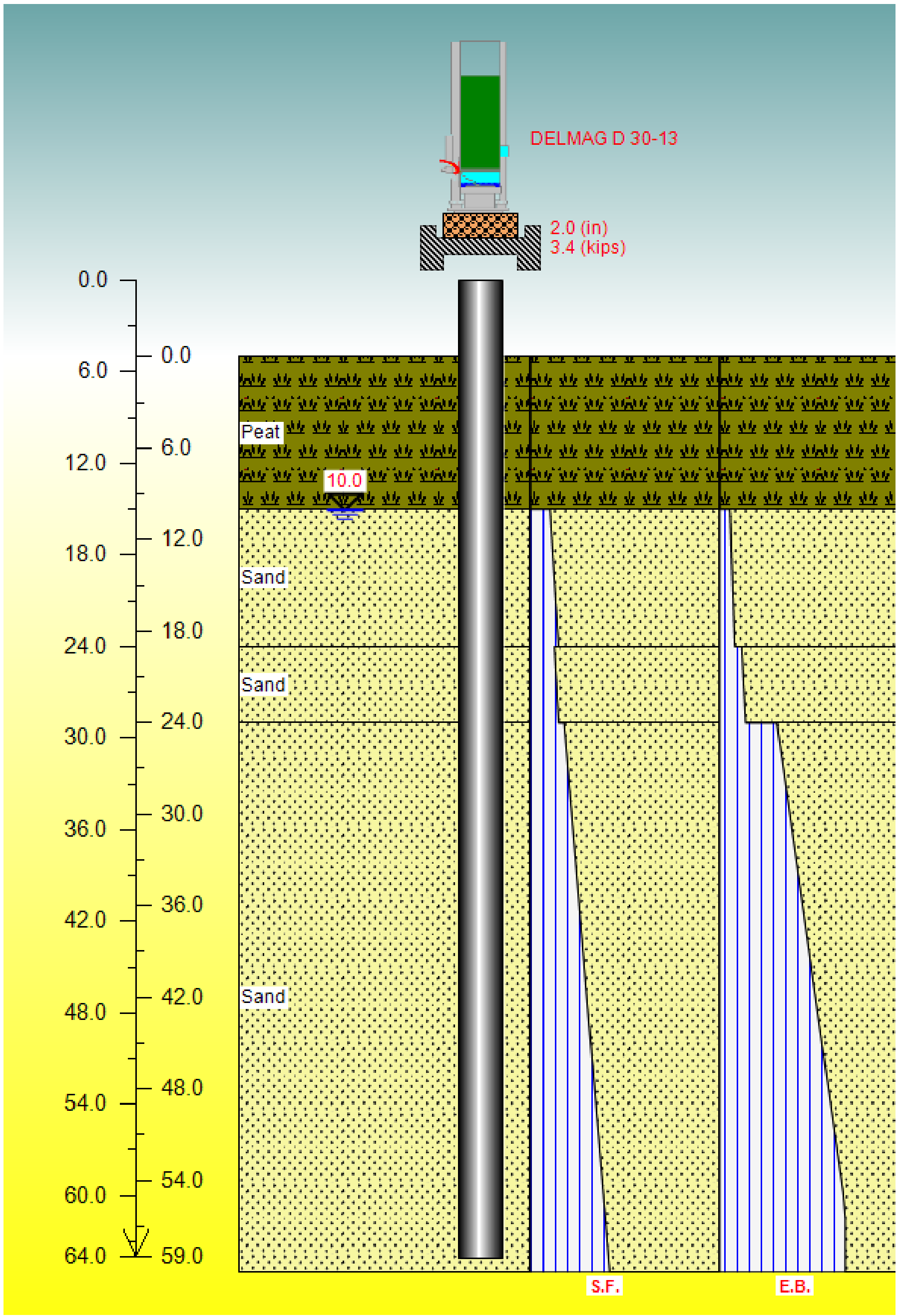
Total driving time: 50 minutes; Total Number of Blows: 1960 (starting at penetration 5.0 ft)

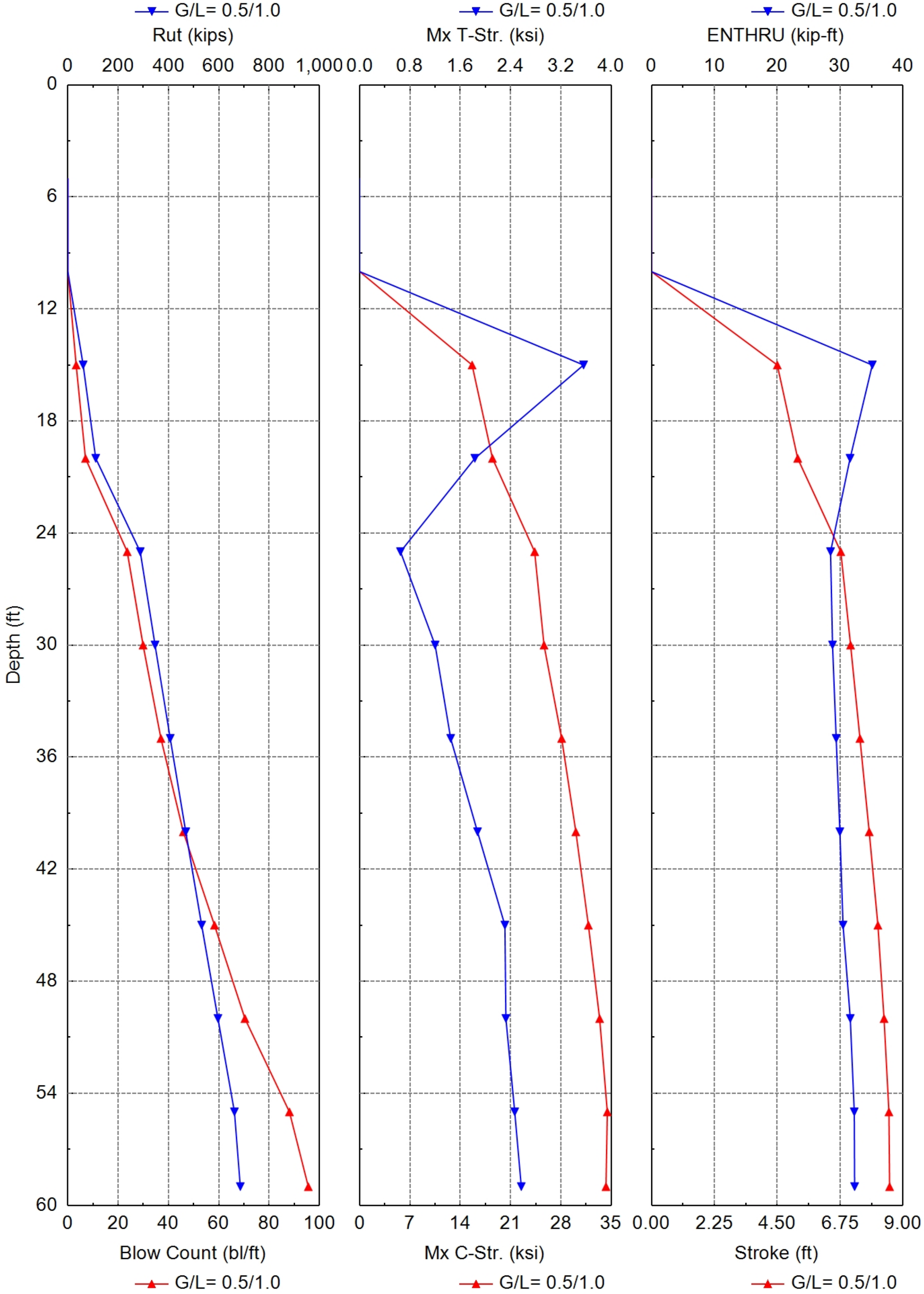
ArDOT 101124 Hwy 135 over Ditch No. 1

Bent 4

16-in-diameter Steel Shell Pile

Delmag D30-13




G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
10
20
30
40
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
20
40
60
80
100
Blow Count (bl/ft)
G/L= 0.5/1.0
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.03	0.0	D 30-13
10.0	0.0	0.0	0.0	0.3	0.000	0.000	10.03	0.0	D 30-13
15.0	60.7	3.0	57.7	3.3	15.647	3.561	4.50	35.1	D 30-13
20.0	110.8	7.0	103.8	7.0	18.481	1.832	5.23	31.6	D 30-13
25.0	287.8	13.4	274.4	23.6	24.350	0.649	6.77	28.5	D 30-13
30.0	346.6	22.2	324.4	29.9	25.652	1.200	7.12	28.8	D 30-13
35.0	406.9	32.5	374.4	37.0	28.108	1.448	7.46	29.4	D 30-13
40.0	468.7	44.3	424.5	45.9	30.084	1.874	7.79	30.0	D 30-13
45.0	532.0	57.5	474.5	58.3	31.829	2.310	8.10	30.5	D 30-13
50.0	596.7	72.2	524.5	70.4	33.396	2.325	8.32	31.6	D 30-13
55.0	662.3	88.4	574.0	88.1	34.465	2.466	8.50	32.3	D 30-13
59.0	685.6	102.4	583.2	95.6	34.276	2.571	8.52	32.3	D 30-13

Total driving time: 47 minutes; Total Number of Blows: 1966 (starting at penetration 5.0 ft)

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Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 15, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
ARDOT 101124 HWY. 135 OVER DITCH No. 12
SITE 4 / BOX CULVERT
POINSETT COUNTY, ARKANSAS

INTRODUCTION

Submitted herewith are the results of the geotechnical investigation performed for the Hwy. 135 over Ditch No. 12 box culvert planned in Poinsett County, Arkansas. This box culvert is Site 4 of the ARDOT 110124 Hwy. 135 Strs. & Apprs. (S) project. ARDOT Job 110124 geotechnical investigation was authorized by the Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023.

We understand the reinforced concrete box culvert will replace the existing highway bridge. The box will be a quadruple 12-ft by 10-ft reinforced concrete structure with a total length of approximately 60 feet. Simple slopes will be utilized at the box culvert with slopes at approximate 3-horizontal to 1-vertical (3H:1V) configurations. Site grading is expected to be minor with existing grades utilized to the extent possible. The maximum embankment height is understood to be about 12 feet.

SUBSURFACE INVESTIGATION

Subsurface conditions at the Hwy. 135 over Ditch No. 12 location were investigated by drilling one (1) sample boring (Boring D1) to a depth of 40 ft below existing grades. The project vicinity is shown on Plate 1 of Attachment 1. The approximate boring location is shown on the Plan of Boring, Plate 2 of Attachment 1. The subsurface conditions encountered in the boring, and the results of field and laboratory tests, are shown on the boring log, Plate 3. The surveyed ground surface elevation is also shown on the log, as well as GPS coordinates. A key to the terms and symbols used on the log is presented on Plate 4.

LABORATORY TESTING

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

The laboratory testing program included three (3) natural water content determinations performed to develop information on *in-situ* soil water content for the boring. The results of these tests are plotted on the log as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field visual classification and to evaluate soil plasticity, two (2) liquid and plastic limit (Atterberg limits) determinations and three (3) sieve analyses were performed on selected representative samples. The Atterberg limits are plotted on the logs as pluses inter-connected with a dashed line using the water content scale. The percentage of soil passing through the No. 200 Sieve is noted in the "- No. 200 %" column on the appropriate log forms. Classification test results, along with soil classification by the Unified Soil Classification System and AASHTO designations, are summarized in Attachment 2. Grain-size distribution curves are also provided in Attachment 2.

SEISMIC CONDITIONS

Based on the results of the boring drilled at this location and the surface geology, a Seismic Site Class D (stiff soil profile) is considered fitting for the Hwy 135 Site 4 location with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Eighth Edition 2017[1].

Given the site location and AASHTO code-based values, recommended seismic parameters are summarized below.

- Seismic Site Class D
- 1.0-sec period spectral acceleration coefficient (S_1) = 0.549
- Site amplification factor at 1.0 second (F_v) = 1.5
- 1.0-sec period spectral acceleration coefficient (S_{D1}) = 0.823
- Acceleration for a short (0.2 sec) period (S_S) = 1.883
- Site amplification factor for short period (F_a) = 1.0

---

[1] AASHTO LRFD Bridge Design Specifications, 8th Edition; AASHTO; 2017.

- Peak ground acceleration (PGA) = 1.047
- Site amplification factor at PGA (F_{PGA}) = 1.0
- A_s = 1.047

Utilizing these parameters, Table 3.10.6-1[2] indicates that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Hwy. 135 bridge over Ditch No. 12 site.

LIQUEFACTION POTENTIAL

Liquefaction analyses were performed to evaluate the liquefaction potential of the foundation soils in the box culvert alignment. The analyses were performed utilizing the results of the boring drilled at the box culvert and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 1.047 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Attachment 3 as plots of calculated factors of safety against liquefaction potential. Liquefaction in these zones would result in immediate liquefaction settlement during a seismic event. Liquefaction settlement values on the order of 1 to 2 in. were calculated based on the results of the liquefaction analyses.

SUBSURFACE CONDITIONS

Based on the results of the boring performed at Site 4, the surface soils to 4-ft depth are comprised of soft to stiff reddish brown and gray fine sandy clay embankment fill. The embankment fill contains minor amounts of fine to coarse gravel and asphalt fragments. The fill has poor compaction and exhibits low shear strength and high compressibility. These soils typically classify as A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with very poor subgrade support for pavement structures.

Below the existing embankment fill is soft to firm gray and brown clay and fine sandy clay extending to 19 ft below existing grades. The clay and fine sandy clay exhibit low shear strength, moderate to low plasticity, and high compressibility.

The clayey soil units are underlain below 18 ft by medium dense brown and brownish gray fine sand and silty fine sand. Some medium to coarse sand is present at depth. These granular units

---

[2] AASHTO LRFD Bridge Design Specification, AASHTO; 2012

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

exhibit medium relative density and low compressibility. Relative density typically increases with depth.

Groundwater was encountered at 18.8 ft in June 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the ditch and nearby surface water features.

ANALYSES and RECOMMENDATIONS

Reinforced Concrete Box Culverts

We recommend the box culvert foundation loads be supported on the culvert bottom mat or continuous footings. However, it will be necessary to support footings on a minimum thickness of select granular fill to develop suitable bearing and to limit the settlement potential.

The foundation loads of the box culvert can be supported on a mat or continuous footings founded in compacted select granular fill. All mats or footings should be underlain by a minimum of 3 ft of select granular fill. Granular fill should consist of stone backfill (ARDOT Standard Specifications Section 207), Select Granular Backfill (AASHTO M 43 Size 57), or alternates approved by the Engineer or Department. Where clean crushed stone backfill (Section 207 stone backfill or AASHTO M 43 clean stone) is used, the stone should be fully incapsulated by a geotextile filter fabric complying with ARDOT Subsection 625.02, Type 2. The culvert foundation depths must be adequate to resist scour or must be protected from scour.

Foundation undercuts should have a minimum width determined by a 1-horizontal to 2-vertical (1H:2V) projection from the footing edge to the undercut bottom. Where site conditions warrant mass undercut, footings may be founded in the compacted undercut backfill.

Foundation recommendations for the RCB culvert are summarized below.

- Bearing Stratum: select granular backfill
- Maximum nominal bearing pressure (q_{ult}): 3500 lbs per sq ft
- Recommended resistance factor (φ_b): 0.45
- Factored bearing pressure (q_r): 1580 lbs per sq ft
- Maximum nominal sliding resistance (tan δ): 0.40
- Sliding resistance factor (φ_τ): 0.80

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

Liquefaction settlement values on the order of 1 to 2 in. have been calculated. Where seismic settlement is a design consideration, ground improvement or deep foundations may be considered. Recommendations for ground improvement or deep foundations can be provided upon request.

A minimum width of 24 in. is recommended for continuous footings. All culvert bottom and foundation excavations should be observed by the Engineer or Department to verify suitable bearing. Post-construction total and differential settlement of foundations supported as recommended is expected to be less than 1 inch.

Lateral Earth Pressures on Culvert Walls

It is anticipated that culvert walls and any wingwalls will be backfilled with either unclassified borrow or select granular fill. Unclassified borrow is expected to be locally available soils which could be silty, sandy clay or silty fine sand. Select granular fill should comply with ARDOT Standard Specifications Section 302 for SM-1 or Select Granular Backfill (AASHTO M 43 No. 57).

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

- Total unit weight (γ) for unclassified backfill: 125 lbs per cu ft
- Angle of internal friction (ϕ) for unclassified backfill: 20°
- Equivalent fluid pressure for unclassified backfill:
  - At-rest condition for walls that are fixed against rotation, backfilled with unclassified borrow, and fully drained: 85 lbs per sq ft per ft depth.
  - At-rest condition for walls that are fixed against rotation, backfilled with unclassified borrow, and no provision for internal drainage: 105 lbs per sq ft per ft depth.
- Angle of internal friction (ϕ) for SM-1 backfill: 32°
- Total unit weight (γ) for SM-1: 125 lbs per cu ft
- Equivalent fluid pressure for SM-1 backfill:
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and fully drained: 60 lbs per sq ft per ft depth.
  - At-rest condition for walls that are fixed against rotation, backfilled with SM-1 or clean granular backfill, and no provision for internal drainage: 92 lbs per sq ft per ft depth.

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

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

Stability Analyses

The box culvert replacement project includes new box culvert end embankments at each box culvert end. Plan box culvert embankment configurations are expected to be 3-horizontal to 1-vertical (3H:1V) slope configurations. The embankment heights are expected to be a maximum of 12 feet.

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 2021[4] and a 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.

For the analyses of the seismic condition, a horizontal seismic acceleration coefficient (k_h) of one-half the peak acceleration (A_s) was used, a value of 0.5235. For evaluating the rapid drawdown condition, a water surface elevation drop from El 224 to El 215 was assumed. The results of the stability analyses of the end slopes are summarized in the table provided in Attachment 5. These results indicate acceptable stability for all cases evaluated.

The new box culvert end configurations will include some additional embankment fill. We recommend the use of cohesive fill for the embankments within at least 100 ft of the box culvert ends. An example special provision is provided in Attachment 6.

---

[4] Slope/W 2021; GEOSLOPE Ltd.

CONSTRUCTION CONSIDERATIONS

Earthwork

Site grading and site preparation at the Site 4 RCB location should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. in cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. The zone of organic surface soils should be completely stripped in the embankment footprint areas and at least 5 ft beyond the projected plan of the box culvert. All saturated and organic soils at the box bottom grade should be mucked out and replaced with suitable materials.

The mat bottom should be constructed on select granular fill. A minimum of 3 ft of select granular fill has been recommended below the box. All undercuts and foundation excavations should be observed by the Engineer.

General fill and backfill for embankments 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow.

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

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. Diversion of the ditch will be required to allow construction in the dry. Use of sumps is likely to be required to maintain suitable subgrade conditions during the work.

Density and water content of all earthwork should be maintained until box construction and embankments are completed.

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

Groundwater was encountered at 18.8-ft depth in June 2023. The ditch channel will contain varying amounts of water. In addition, shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvement, and all foundation, culvert, and embankment 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 attachments are included and complete this submittal.

Attachment 1	Site Vicinity Map, Plans of Borings, Preliminary Boring Logs, Key to Terms and Symbols
Attachment 2	Laboratory Test Results
Attachment 3	Liquefaction Analysis Results
Attachment 4	Stability Analysis 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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

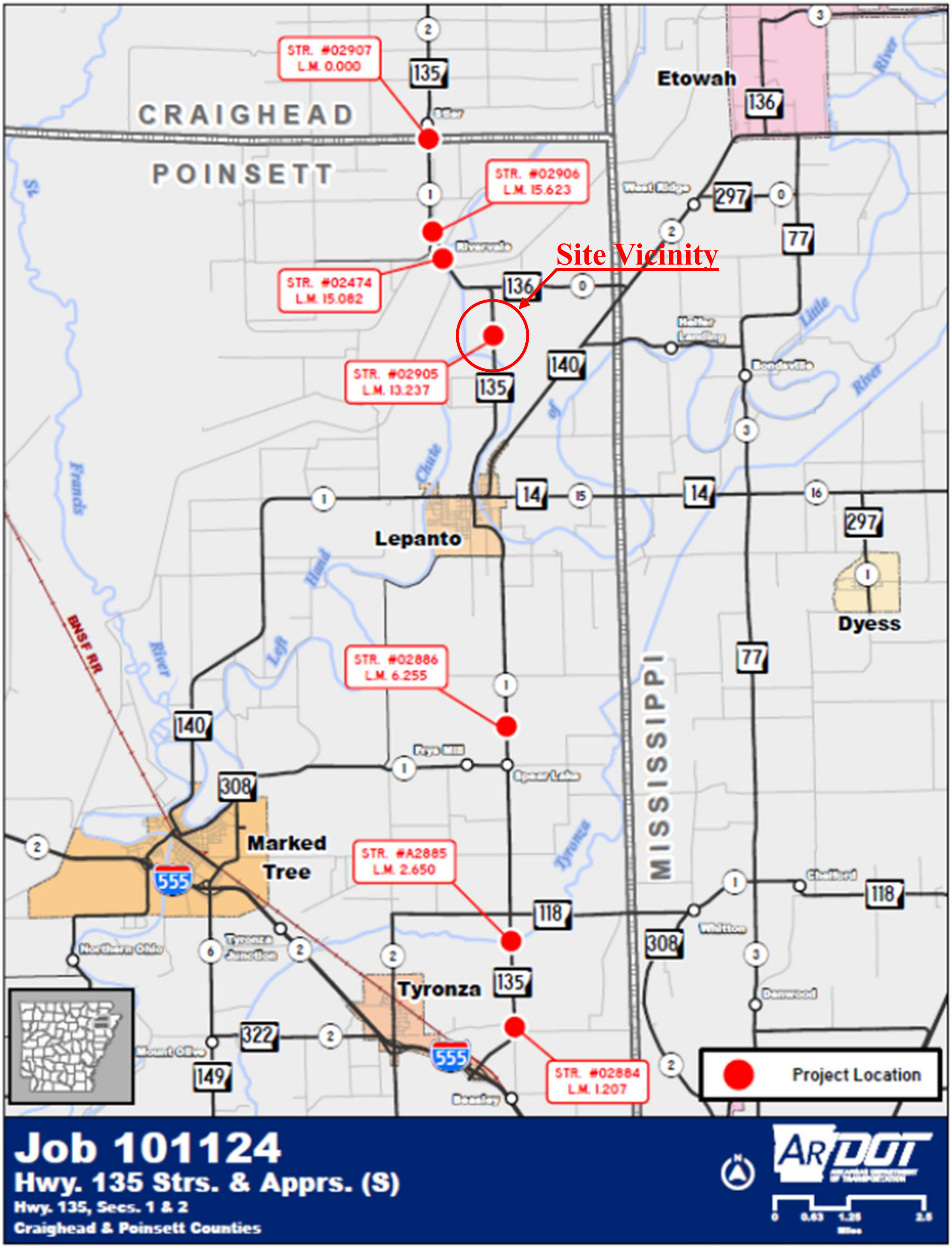
Mark E. Wyatt, P.E.
President

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

VMS/MEW:jw

Copies submitted:	Arkansas Department of Transportation		
	Attn:	Ms. Jessica Jackson, P.E.	(1-email)
	Attn:	Mr. Paul Tierney	(1-email)
	Attn:	Mr. Yongsheng Zhao, Ph.D., P.E.	(1-email)
	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

ATTACHMENT 1



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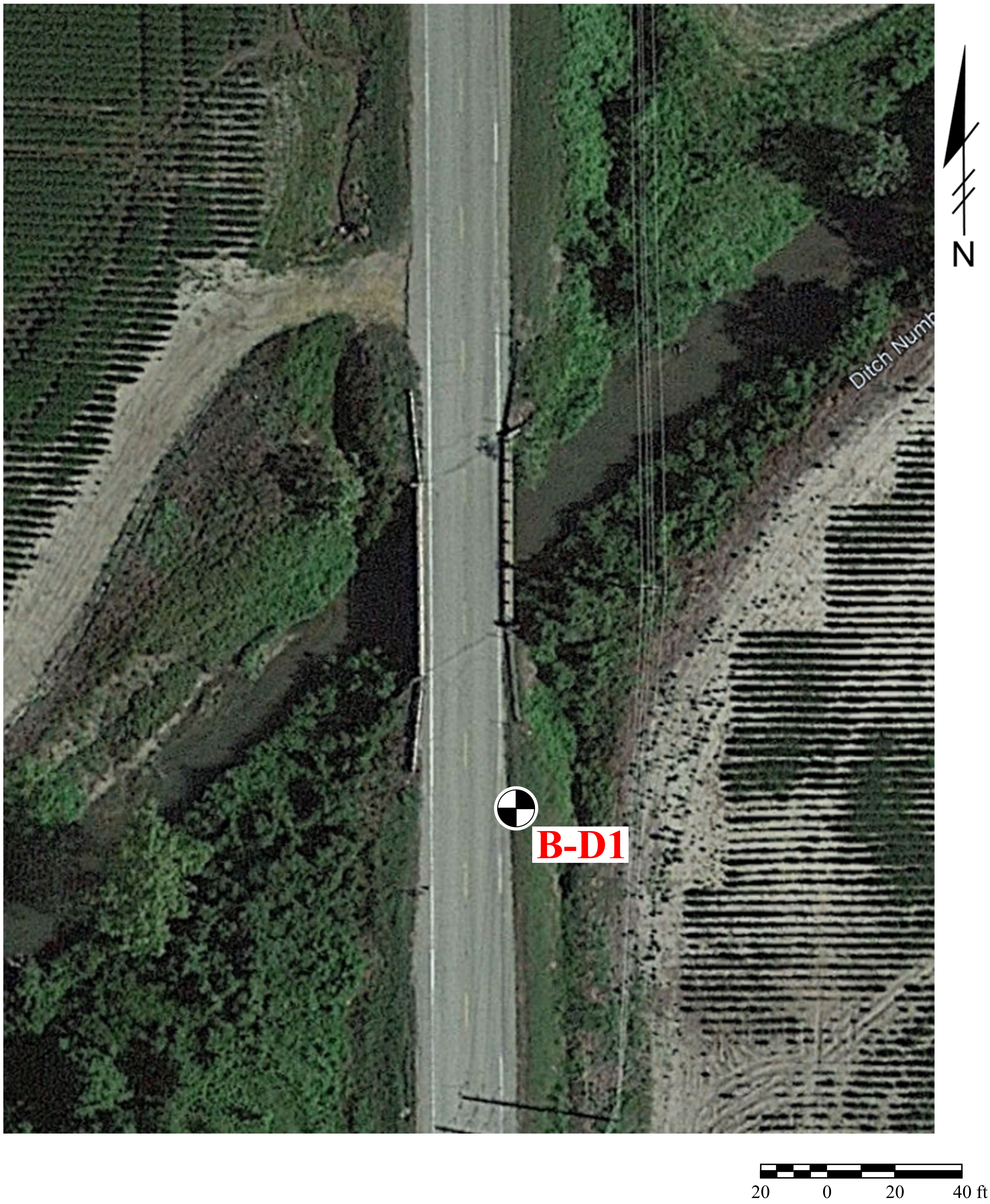
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 RCB
at Ditch No. 12 (Site 4)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORING
101124 Hwy. 135 RCB at Ditch No. 12
(Site 4)
Poinsett County, Arkansas

Scale: As Shown

Job No. 23-031

Plate 2

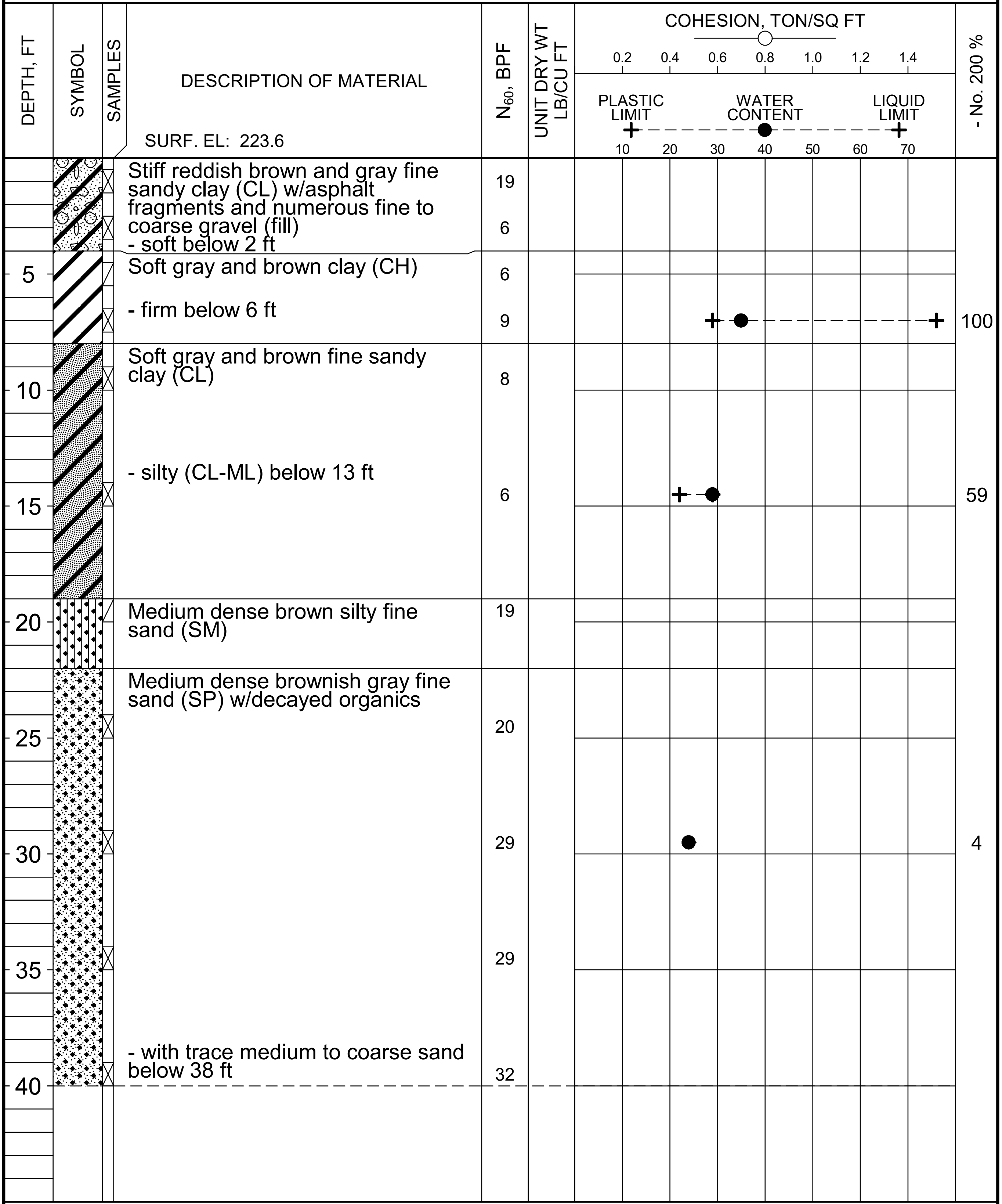
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. D1

101124 Hwy. 135 over Ditch No.12
Poinsett County, Arkansas

TYPE: HSA to 23 ft /Wash

LOCATION: 35.65230° N, -90.32336° E



DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Stiff reddish brown and gray fine sandy clay (CL) w/asphalt fragments and numerous fine to coarse gravel (fill) - soft below 2 ft	19 6		
Soft gray and brown clay (CH) - firm below 6 ft	6 9		100
Soft gray and brown fine sandy clay (CL) - silty (CL-ML) below 13 ft	8 6		59
Medium dense brown silty fine sand (SM)	19		
Medium dense brownish gray fine sand (SP) w/decayed organics - with trace medium to coarse sand below 38 ft	20 29 29 32		4

COMPLETION DEPTH: 40.0 ft
DATE: 6-15-23

DEPTH TO WATER
IN BORING: 18.8 ft

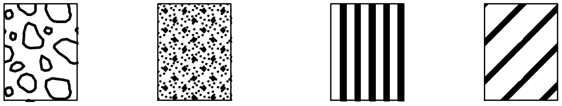
DATE: 6/15/2023

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

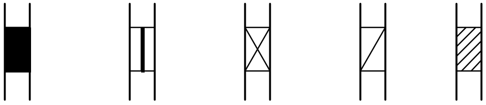
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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

ATTACHMENT 2

SUMMARY of CLASSIFICATION TEST RESULTS

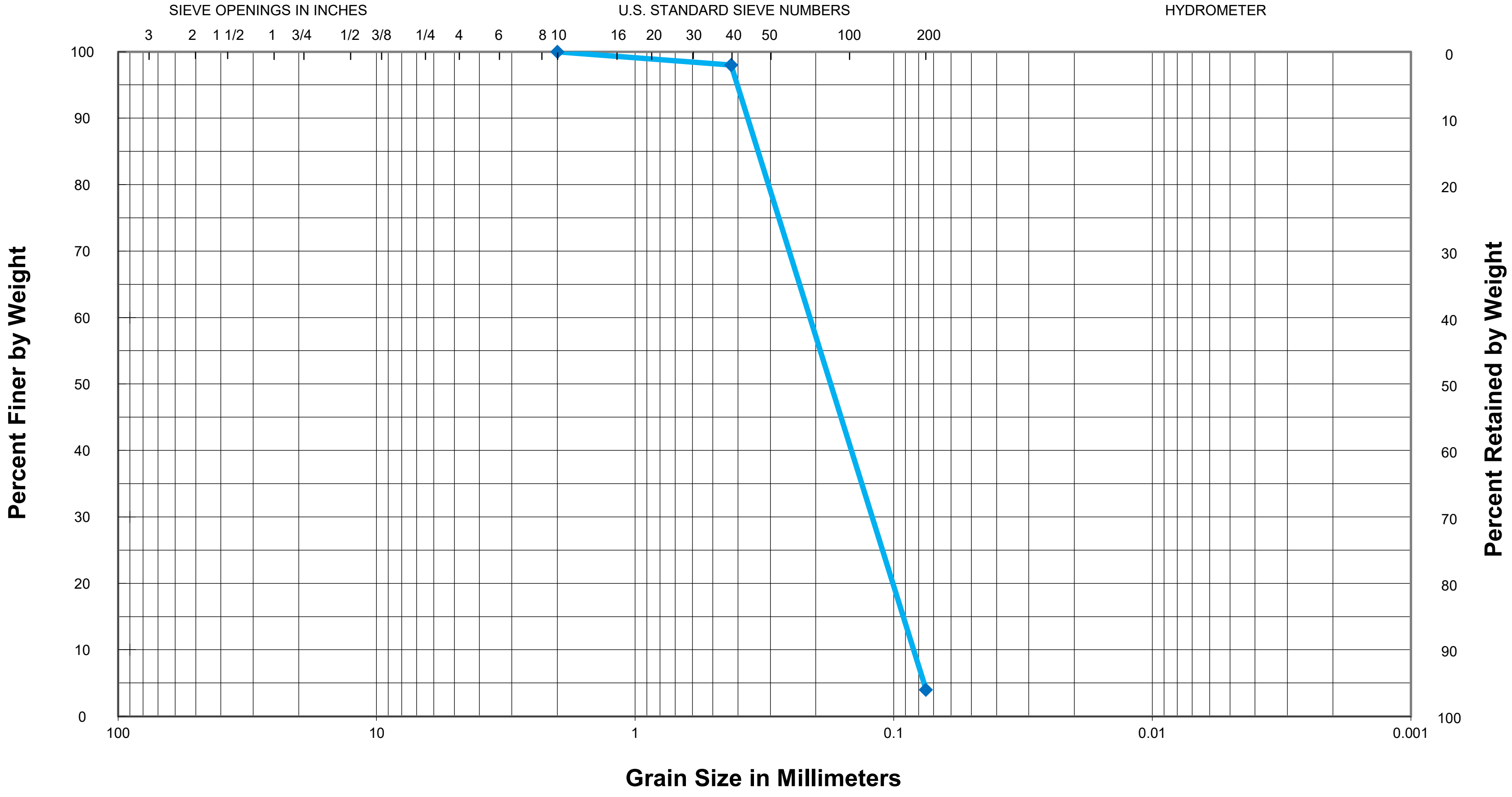
PROJECT: 101124 Hwy. 135 over Ditch No. 12
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

BORING No.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			PERCENT PASSING #200	USCS CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX			
D1	6.5-7.5	35	76	29	47	100	CH	A-7-6
D1	14-15	29	29	22	7	59	CL-ML	A-4
D1	29-30	24	---	---	---	4	SP	A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

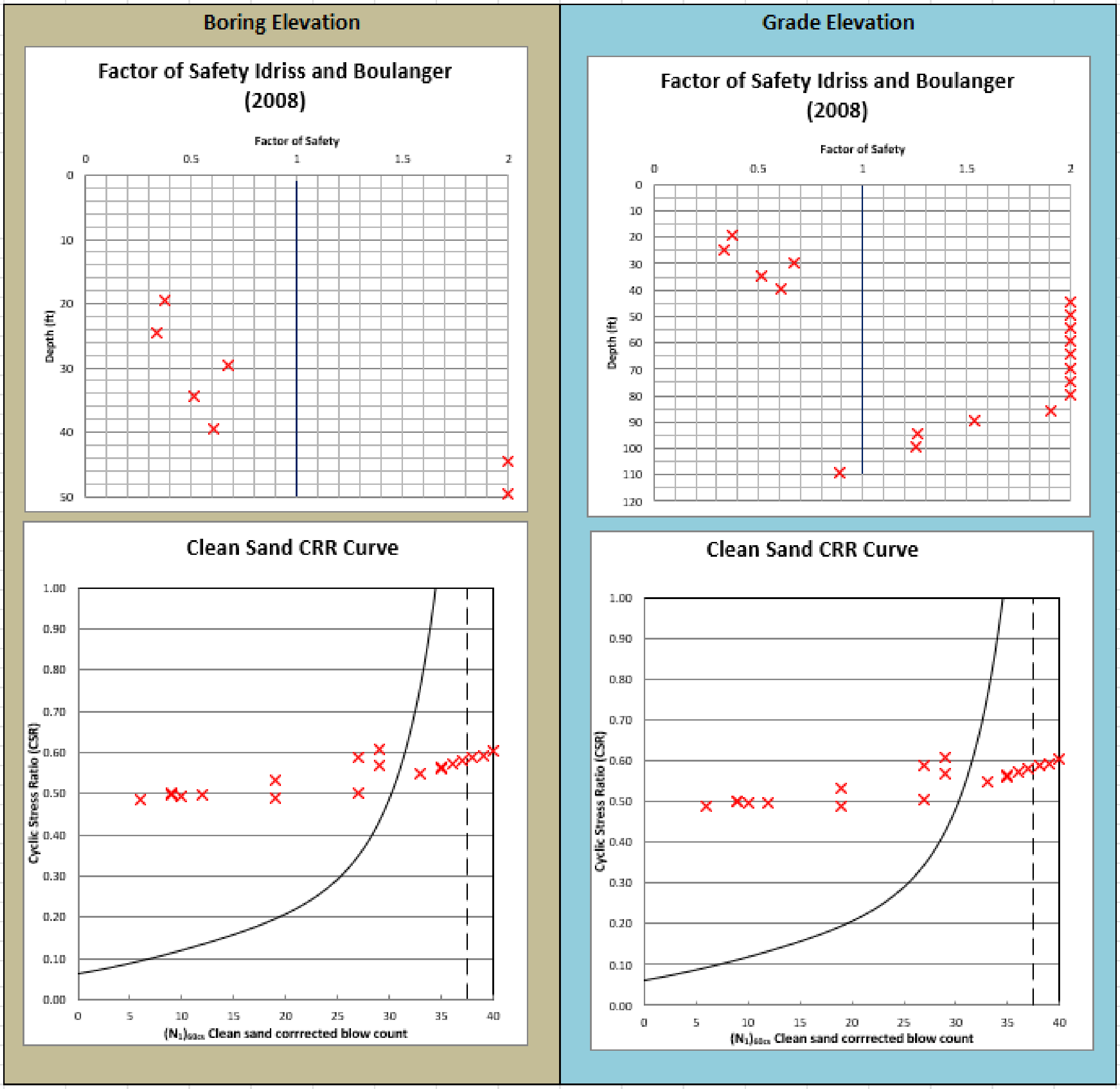
Sample: Boring D1, 29-30 ft
Description: Brownish gray fine SAND

USCS Classification = SP
AASHTO Classification = A-3

ATTACHMENT 3

Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Ditch No. 12
Boring D1
GHBW Job No. 23-031
Poinsett County, Arkansas

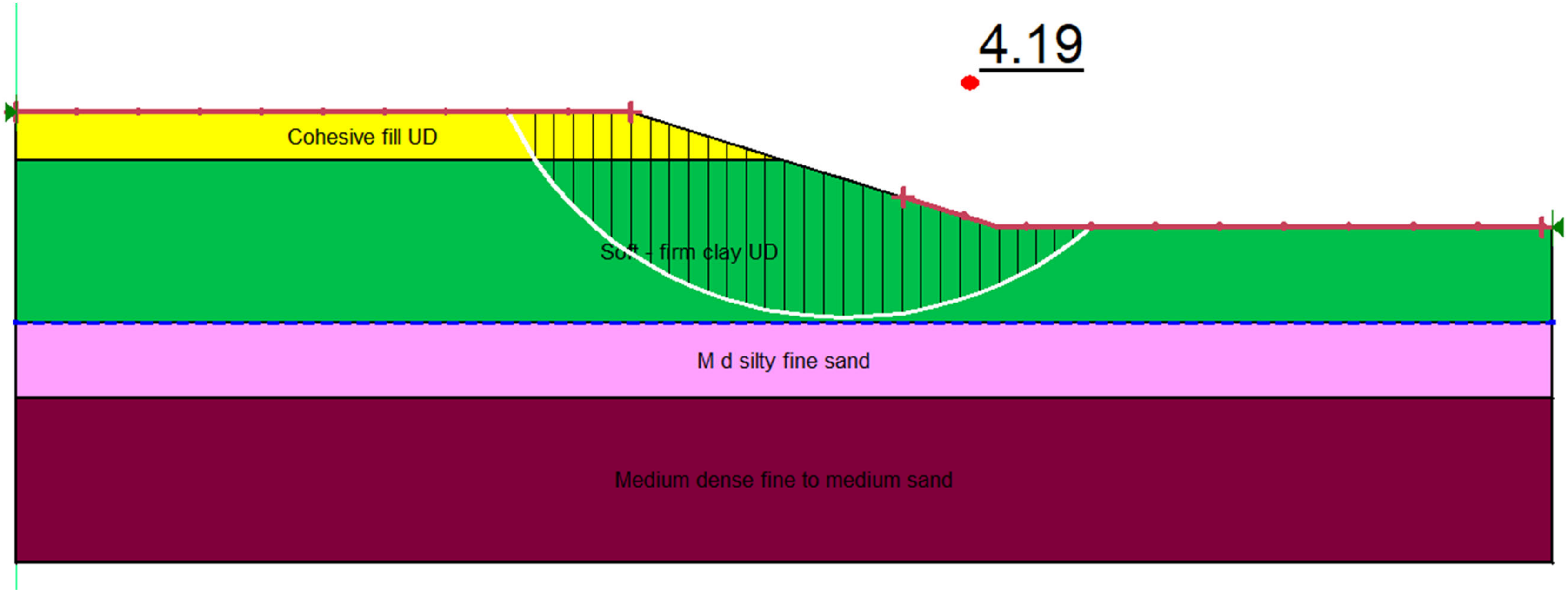


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

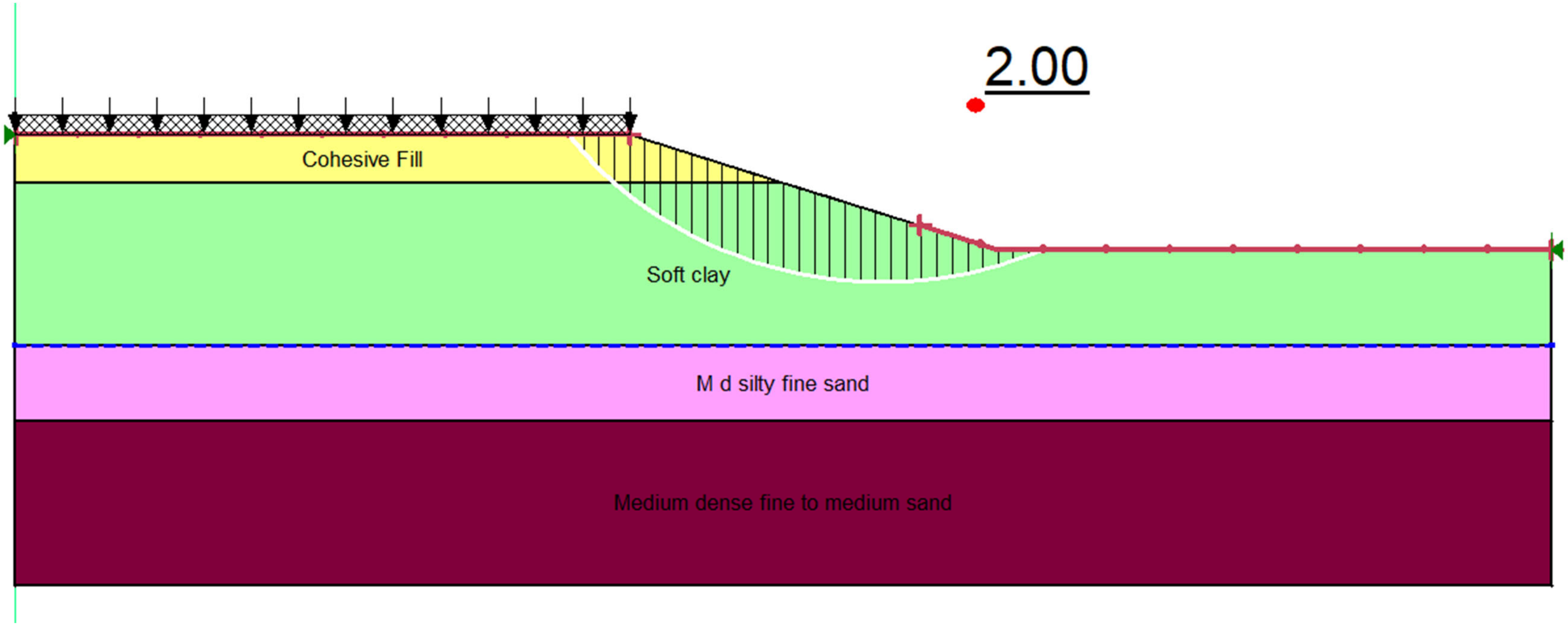
ATTACHMENT 4

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Ditch No. 12 (Site 4)
GHBW Job No. 23-031
Poinsett County, Arkansas

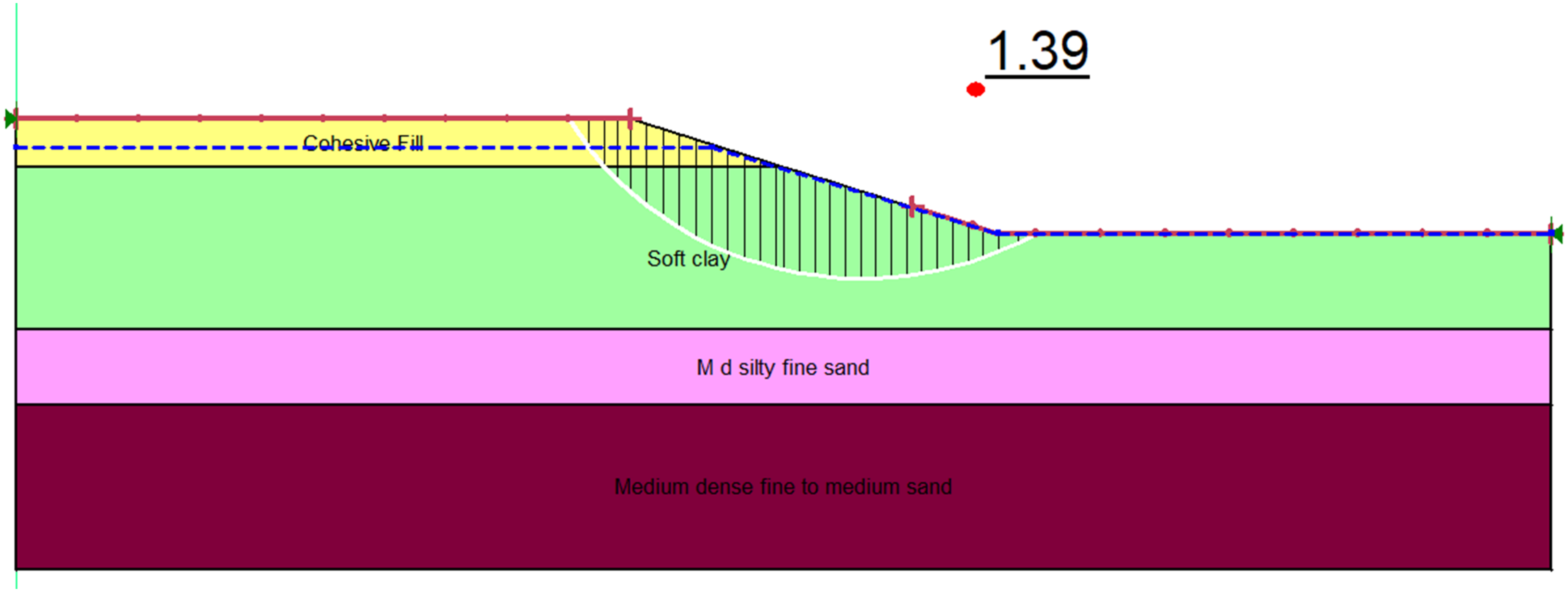
	Design Loading Condition	Calculated Minimum Factor of Safety
Box Culvert (3H:1V)	End of Construction	4.19
	Long Term	2.00
	Rapid Drawdown from El 224 to El 215	1.38
	Seismic (k_h =A_S/2 = 0.5235)	1.15



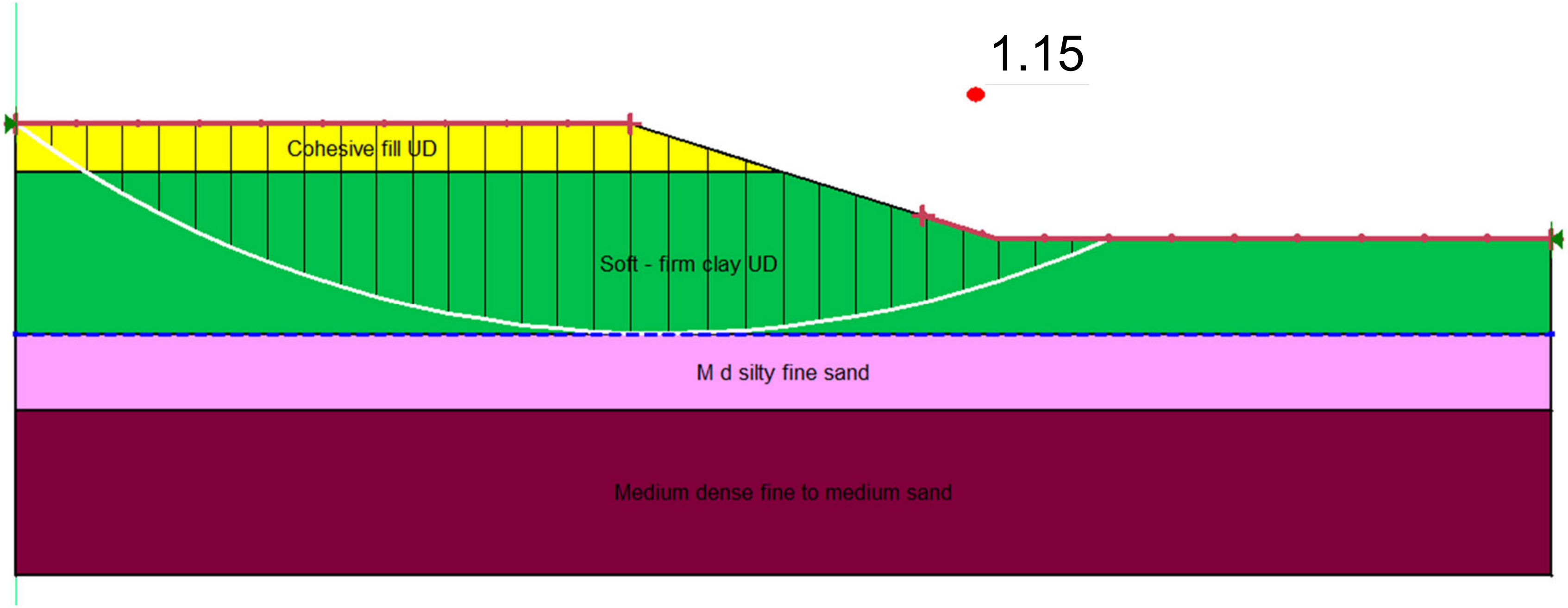
Results of Stability Analyses – End of Construction
Box Culvert
3H:1V Slope, H=12 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Ditch No. 12



Results of Stability Analyses – Long Term Condition
Box Culvert
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 12



Results of Stability Analyses – Rapid Drawdown Condition from El 224 to El 215
Box Culvert
3H:1V Slope, H=12 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Ditch No. 12



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.5235$)
Box Culvert
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Ditch No. 12

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 18, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER RIGHT HAND CHUTE OF LITTLE RIVER (SITE 5)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
POINSETT COUNTY, ARKANSAS

INTRODUCTION

This report provides the final results of the geotechnical investigation performed for the Hwy. 135 over Right Hand Chute of Little River replacement bridge in Poinsett County, Arkansas. This bridge is Site 5 of the ARDOT 110124 Hwy. 135 Strs & Apprs (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by the Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on May 31, 2023. Additional pile capacities and recommendations for ground improvement were submitted on August 14, 2023 and August 18, 2023, respectively.

We understand the replacement bridge will be a prestressed concrete girder unit with eight (8) bents, seven (7) spans, and a total length of approximately 667 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed east of the existing bridge. Site grading will include about 10 ft of fill. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, 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 in the Site 5 replacement bridge alignment were explored by drilling nine (9) sample borings to 100- to 130-ft depth (Borings E1 to E9). The bridge end borings, Borings E1 and E9, were offset south and north of the existing flood control levee to avoid drilling through the earth structure. These borings were backfilled with cement-bentonite grout after completion. One (1) boring drilled from the existing bridge deck, Boring E5, was abandoned when refusal on riprap was encountered at 4-ft depth. The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
E1	319+50	20 Lt	35.671390	90.337669	233.8	130
E2	321+05	15 Lt	35.671773	90.337937	219.6	111
E3	321+20	25 Rt	35.671858	90.337853	220.3	110

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
E4	322+00	20 Lt	35.671984	90.338113	218.3	110
E5	322+95	30 Lt	35.672182	90.338321	234.1	4.5
E6	324+50	30 Lt	35.672546	90.338600	234.2	100
E7	326+20	20 Lt	35.672963	90.338858	219.1	110
E8	326+20	25 Rt	35.673025	90.338737	221.1	110
E9	327+60	30 Lt	35.673275	90.339143	233.8	110

The boring logs, presenting descriptions of the soil strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 27. The centerline station and offset of the boring locations and approximate ground surface elevation, as surveyed, are also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 28.

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.

The borings were drilled with a truck-mounted CME-55 HTX rotary-drilling rig and a track-mounted Diedrich D-50 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the appropriate energy conversion factor is shown on each boring log.

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

The borings were advanced using dry-auger drilling procedures to the extent possible to facilitate groundwater observations. Observations regarding groundwater are noted in the lower portion of each log and are discussed in subsequent sections of this report. The boreholes were backfilled after obtaining final water level readings. Borings E1 and E9 were backfilled with cement-bentonite grout after completion.

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 58 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 6 liquid and plastic (Atterberg) limit determinations and 57 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The location of 101124 Site 5 is on Hwy. 77 where the Right Hand Chute of the Little River channel crosses the highway alignment just south of Highway 87 in Poinsett County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. The channel at this location is broad with variable well-defined to shallow banks. An existing weir is located downstream (southwest) of the new bridge alignment. A flood control levee is located on each side of the channel at the bridge location. The banks are fairly short and covered with grass, variable sparse to thick underbrush, and occasional trees. The project locale is primarily agricultural land consisting of woods or large, flat fields. Several houses are located behind the levee north of the bridge. The existing two-lane roadway is on an embankment and is several feet higher than the adjacent terrain. The existing bridge deck and pavements are in poor condition. Surface drainage along the roadway is poor to fair and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends through exposures of Quaternary Terrace Deposits and Alluvium. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The Alluvium is comprised of recent stream-deposited alluvial sediments which include gravel, sand, silt, clay and mixtures of all components. The thickness of the Terrace and Alluvial deposits is variable. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

Seismic Conditions

A Site-Specific Ground Motion Response Analysis was performed for the 110124 project. The site-specific ground motion response analyses were performed by Geotechnology in accordance with Section 3.4.3.2 of the 2022 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition. Three (3) sites were analyzed for shear wave velocities: Sites 2, 5, and 7. The site-specific results from Site 5 were utilized in the current analysis.

Shear wave velocity profiles were developed for the Site-Specific Ground Motion Response Analysis. Summary results from the analysis are provided in Appendix D. An average shear wave velocity in the top 100 ft of subsurface soil was calculated to be 705 ft per second. In light of the shear wave velocity profile and the results of the borings, a Seismic Site Class D (stiff soil profile) is considered fitting for the Site 5 bridge location.

Based on the results of the site-specific seismic hazard analysis, design earthquake spectral response acceleration of 0.864g for PGA, 1.673g for S_{DS}, 1.247g for S_{D1} and 7.7 for Design Earthquake Moment Magnitude (M_w) were determined. These calculated design seismic accelerations utilizing the site-specific procedure are 67 percent or greater of the corresponding

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

counterparts as determined using the code-based procedure. A plot of design response spectra, showing the design earthquake spectral response accelerations versus period for both code-based and site-specific values, is also included in Appendix D. The design response spectra developed based on the results of the site-specific procedure are considered suitable for use in structural design.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[2] in 2008. A design PGA value of 0.864 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Appendix E as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the analyses results are shown on the generalized subsurface profile also provided in Appendix E. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix E.

Subsurface Conditions

Based on the results of the borings, the surface soils are locally comprised of existing embankment fill extending to 13 to 23 ft below existing grades (see Borings E1 and E9). The embankment fill consists of loose to medium dense gray, dark gray, brown, and reddish brown silty fine sand and clayey fine sand (SM and SC) and firm to stiff gray and reddish brown clay and fine sandy clay (CH and CL). The silty, clayey sand and clay/sandy clay exhibit low to moderate relative density or shear strength and moderate to high compressibility. The fill soils typically classify as A-2-4, A-6, and A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to fair subgrade support for pavement structures.

Below the fill or at the surface to 17- to 38-ft is brown, gray, dark gray, and brownish gray very loose to medium dense silty fine sand (SM and SP-SM), clayey fine sand (SC), and fine sandy silt (ML) with interbedded very soft to soft clay (CH) and silty clay (CL) layers. The silty, clayey

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[2] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

sand and clay/silty clay exhibit low to moderate relative density or shear strength and moderate to high compressibility. The granular soils typically classify as A-2-6, A-3, A-4, and A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to fair subgrade support for pavement structures.

The silty and clayey surface soil units are underlain below 17 to 38 ft to in excess of the completion depth of the borings by medium dense to very dense gray, brown, grayish brown and brownish gray fine to medium sand strata (SP and SP-SM). Some coarse sand, sandy clay seams, organic inclusions, and fine gravel are present at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was encountered in the borings at 4.7 to 28 ft depth in in May and June 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the river and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 5 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Additionally, stability of the northern embankment end slope is not expected to be adequate for the seismic condition. Lateral spread would also occur during some seismic events. Consequently, ground improvement will be warranted to mitigate deficient slope stability and prevent lateral spread during seismic events.

Recommendations for piling and ground improvement are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 24-in.-diameter steel shell piles are planned for bridge ends and 30-in.-diameter steel shell piles are planned for the interior bents. We also understand that piling at Bents 2, 3, and 4 will have isolation casing driven to El 192.6 prior to the steel shells being driven. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix F. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for “static” conditions prior to a seismic event, with no liquefaction, and full soil shear strength is mobilized for the foundation soils. For the case where liquefaction occurs, the “end of earthquake” condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects. The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix G.

End Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 8) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 33 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[3] and a 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 0.432. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value from the site-specific seismic hazard analysis. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 231 to El 214 was assumed.

Given the anticipated liquefaction triggering with concomitant reduced shear strength and lateral spread in the liquefied zone, ground improvement will be required to develop sufficient stability in a seismic event. A minimum factor of safety against sliding of 1.05 is required for the seismic condition. Stability analyses were performed assuming ground improvement at bridge ends, as discussed in the Ground Improvement section of this report.

Stability analyses results are summarized and presented graphically in Appendix H. The results of the stability analyses indicate that plan configurations of the embankment end slopes are acceptable with respect to stability of all loading conditions evaluated. This includes stability in seismic loading.

Ground Improvement

The results of liquefaction analyses indicate significant risk of liquefaction triggering in the loose to medium dense fine to medium sand at relatively shallow depth. The zone of liquefaction adversely impacts the stability of the north bridge end embankment during a seismic

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

event as determined by stability analyses. Analyses indicate that stability at the north bridge end in the seismic loading condition would not be adequate without ground improvement. Analyses also indicate that stability at the south bridge end in the seismic loading condition will be acceptable without ground improvement.

The use of compaction piles is recommended for ground improvement at the north bridge end. The compaction piles will mitigate the liquefaction potential by densifying the surrounding granular soils and reinforcing the soil mass with stiffened elements. This will serve to both increase the resistance to liquefaction and to improve stability during seismic loading. With ground improvement, adequate north bridge end embankment slope stability and resistance to lateral spread during seismic loading are anticipated during seismic loading.

The concept for ground improvement was developed by evaluation of compaction piles at various spacings. The assumption of ground improvement was to provide densification through a sufficient depth of potential liquefaction triggering to provide adequate stability during seismic loading. The liquefaction analyses results and stability analyses were used to develop a minimum plan penetration and tip elevation for compaction piles. For evaluation of the general case of mitigating the liquefaction potential and improving stability for the seismic case, stability analyses were performed. Multiple iterations were performed until a minimum calculated factor of safety of 1.05 had been developed for the seismic case.

Displacement piles are recommended for ground improvement to maximize the effect of densification. Based on economic considerations, untreated timber piles complying with ARDOT Standard Specifications Section 818 are recommended. Other displacement pile types or sizes could be used if approved by the Engineer.

For ground improvement at the Site 6 north bridge end, the following are recommended.

- Untreated timber piles (nominal 14-in. butt, 10-in. tip), spaced at 8 ft on center each direction.
- Piling extending in a zone extending as shown on the conceptual layout drawing provided in Appendix I.
- Plan tip elevation varies. Piles driven to practical refusal may be terminated at shallower depths.
- Pile length: 40 feet.

The concept for compaction pile ground improvement is shown on the drawings included in Appendix I. Some field adjustment of the pile layout is considered acceptable. However, location adjustments in excess of the specified tolerance should be approved by the Engineer or Department.

We recommend that timber piles be driven with a pile hammer capable of delivering at least 12,500 ft lbs per blow. Where compaction piles are driven to practical refusal, we recommend that driving be terminated and the compaction pile accepted. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

Subgrade Support

It is understood that "standard" pavement sections for the approach roads will be developed by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-4 and A-6. These classifications correlate with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, localized undercuts or improvement depths on the order of 2 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. in cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, localized undercutting could be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 13 to 18 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix J. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

General 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. Given the high seismic zone, we recommend that <u>new</u> embankment fill consist of cohesive borrow. An example special provision for cohesive embankment fill is provided in Appendix K.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until the retaining wall, embankments, and bridge work is completed.

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

Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered. Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[4]. In the driveability analyses, the steel shell piles were assumed to be driven from the plan cap bottom elevation or existing grade. Graphical and tabulated results of these analyses are provided in Appendix L.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 125 ft-kips per blow for driving the steel shell piles at the end bents. For intermediate bents 3 through 7, we recommend a hammer system capable of delivering at least 212 ft-kips per blow for driving the steel shell piles. A hammer system capable of delivering at least 248 ft-kips per blow is recommended for driving the steel shell piles at Bent 2. 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 installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. 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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

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[4] GRLWEAP 2014; Pile Dynamics, Inc.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment 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 attachments are included and complete this submittal.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 27	Boring Logs
Plate 28	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Selected Results - Site-Specific Ground Motion Response Analysis
Appendix E	Liquefaction Analysis Results
Appendix F	Nominal Pile Capacity Curves
Appendix G	Lateral Load Parameters
Appendix H	Results of Stability Analyses
Appendix I	Conceptual Ground Improvement Plan
Appendix J	Example SP – Woven Geotextile
Appendix K	Example SP – Cohesive Embankment Fill Special Provision
Appendix L	Driveability Analysis 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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

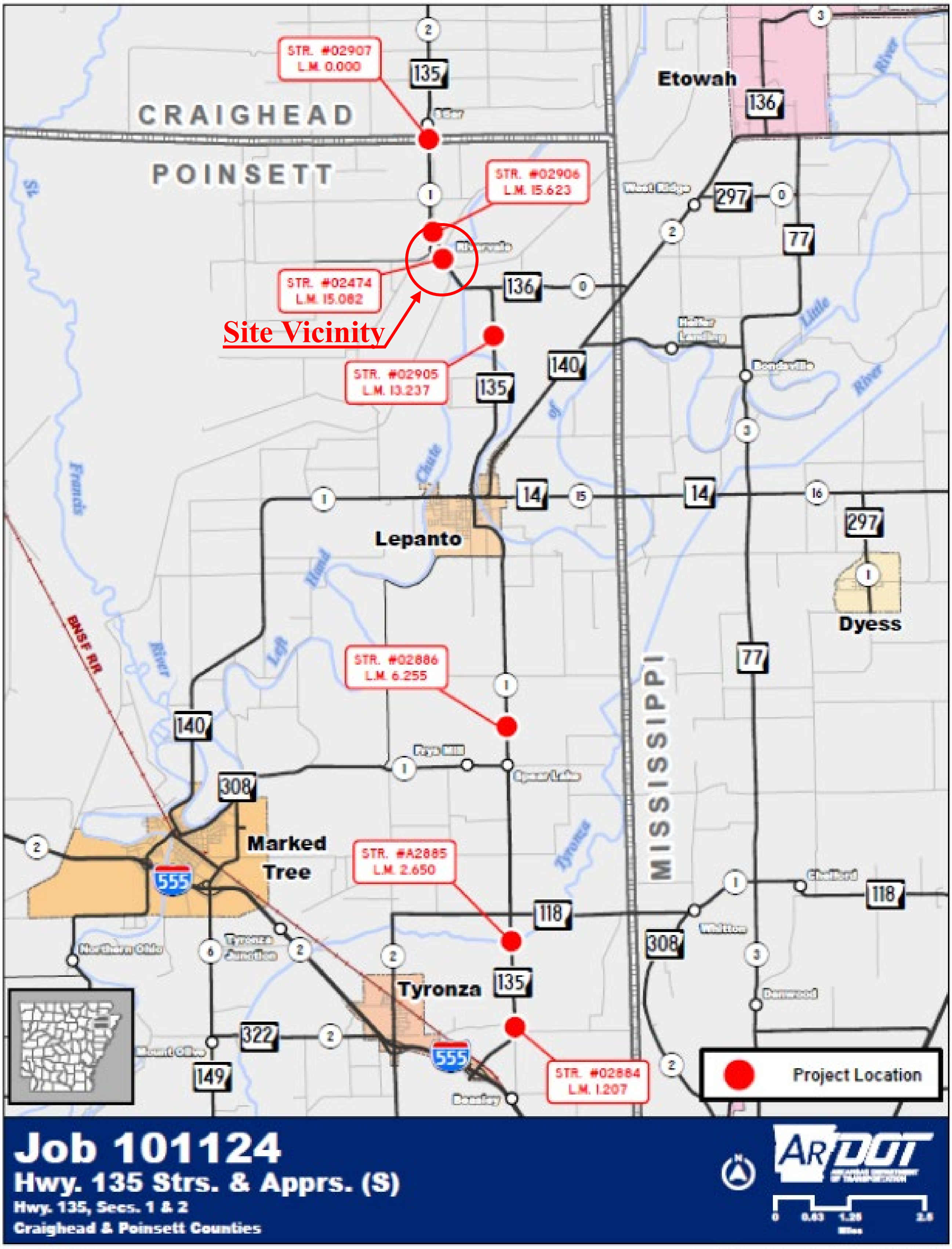
Velleta M. Scott, P.E.
Sr. Project Engineer

Mark E. Wyatt, P.E.
President

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

VMS/MEW:jw

Copies submitted:	Arkansas Department of Transportation		
	Attn:	Ms. Jessica Jackson, P.E.	(1-email)
	Attn:	Mr. Paul Tierney	(1-email)
	Attn:	Mr. Yongsheng Zhao, Ph.D., P.E.	(1-email)
	Crafton Tull & Associates, Inc.		
	Attn:	Mr. Mike Burns, P.E.	(1-email)
	Attn:	Mr. Chuck Wipf, P.E.	(1-email)

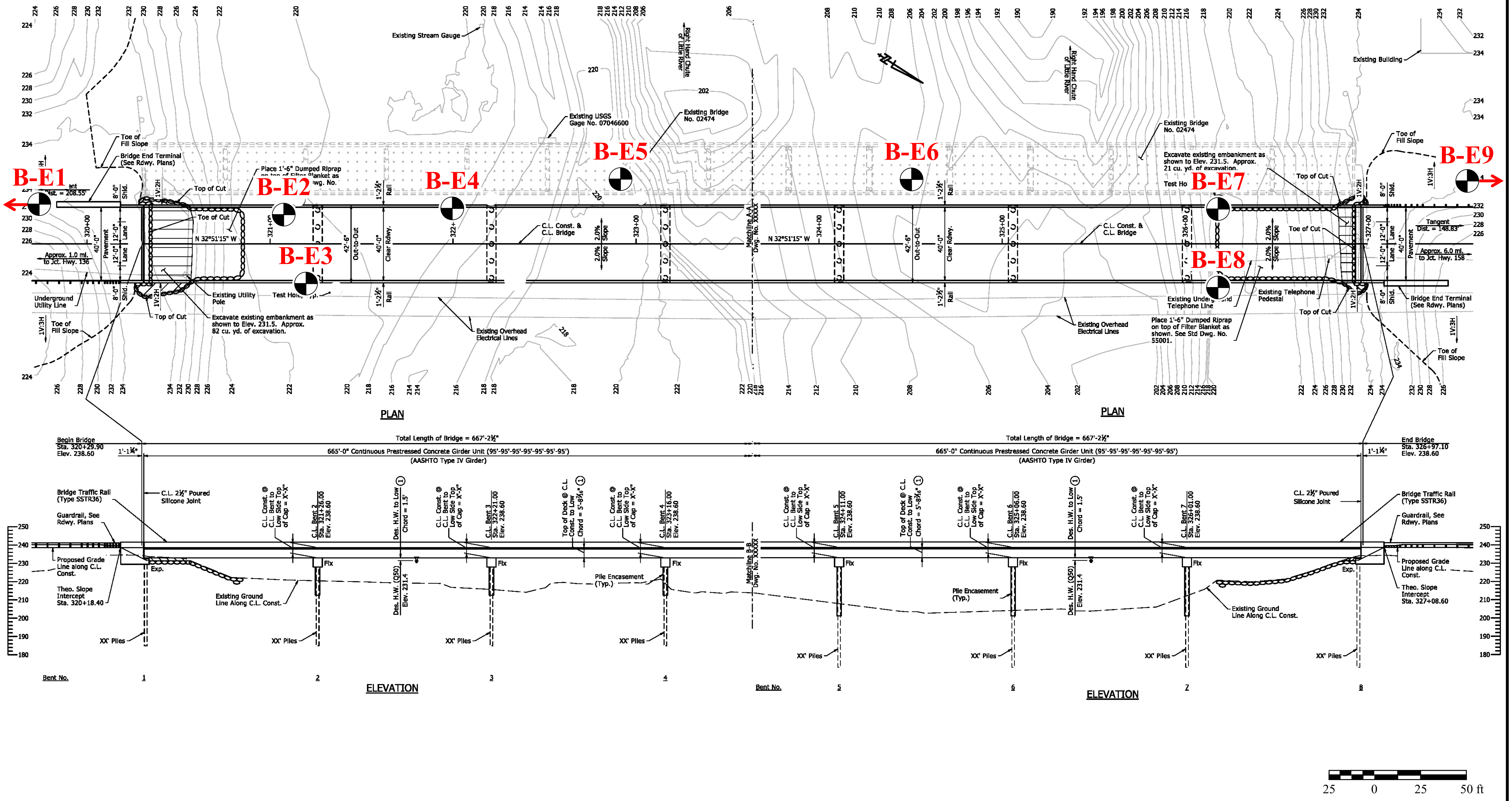


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Right Hand Chute of Little River (Site 5/Bridge E)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORINGS

101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Scale: As shown

Job No. 23-031

PLATE 2

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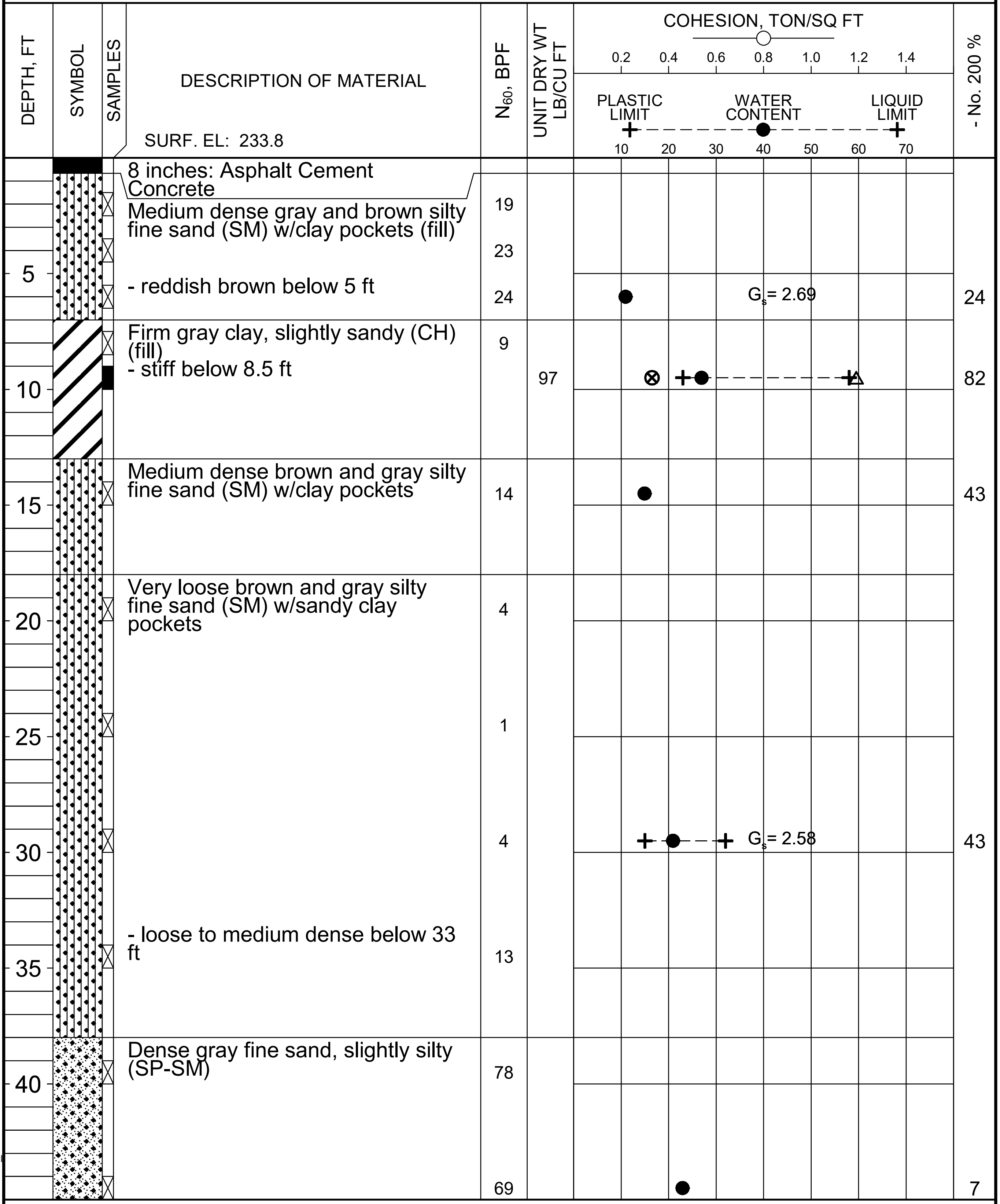
LOG OF BORING NO. E1

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 30 ft /Wash
LOCATION: Approx Sta 319+50, 20 ft Lt



SURF. EL: 233.8

DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
8 inches: Asphalt Cement Concrete			
Medium dense gray and brown silty fine sand (SM) w/clay pockets (fill)	19 23		
- reddish brown below 5 ft	24		24
Firm gray clay, slightly sandy (CH) (fill)	9		
- stiff below 8.5 ft		97	82
Medium dense brown and gray silty fine sand (SM) w/clay pockets	14		43
Very loose brown and gray silty fine sand (SM) w/sandy clay pockets	4 1 4		 43
- loose to medium dense below 33 ft	13		
Dense gray fine sand, slightly silty (SP-SM)	78 69		 7

COMPLETION DEPTH: 130.0 ft
DATE: 5-31-23

DEPTH TO WATER
IN BORING: Dry to 30 ft
DATE: 5/30/2023

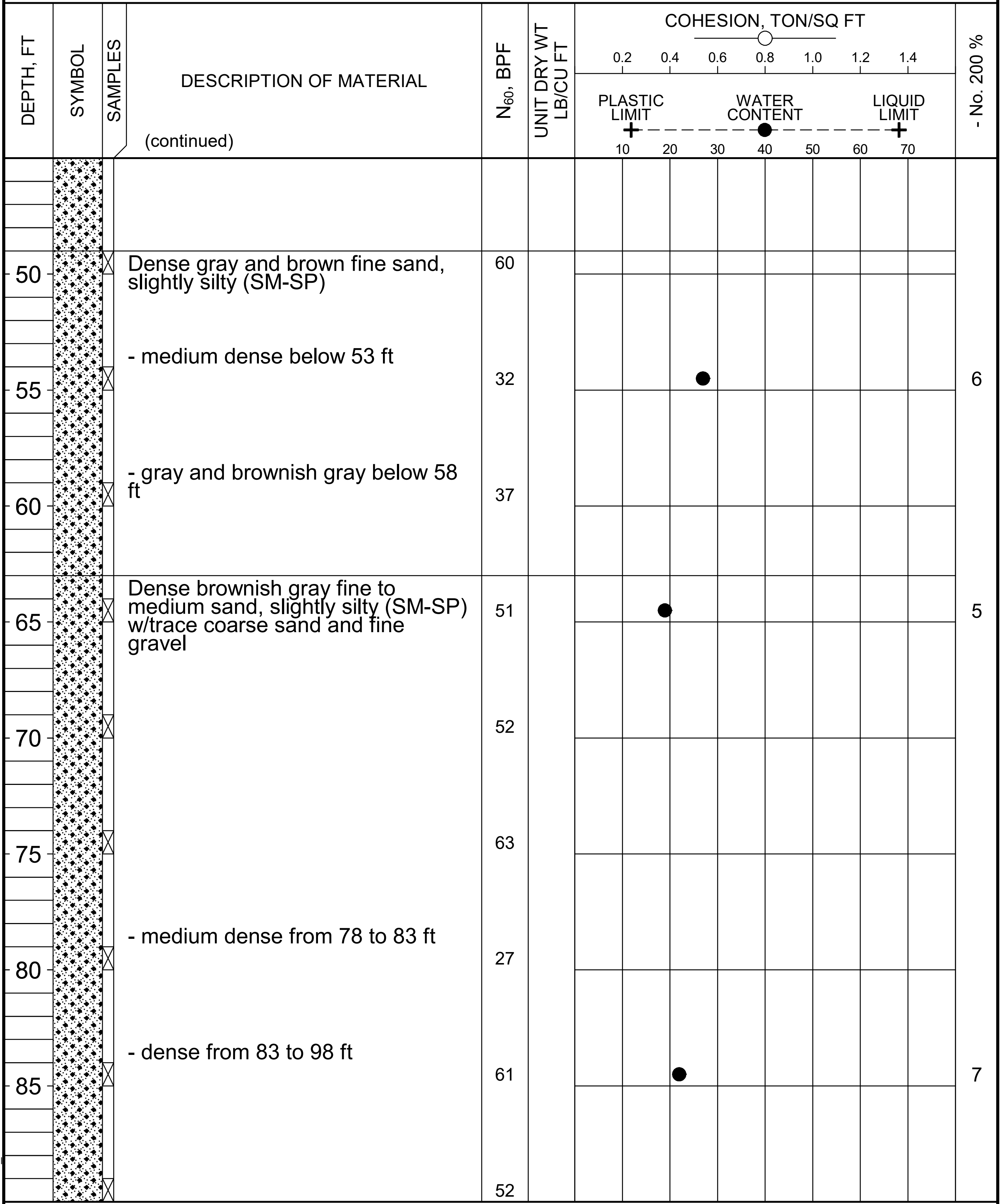
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. E1

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Approx Sta 319+50, 20 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	Dense gray and brown fine sand, slightly silty (SM-SP)	60		
55	- medium dense below 53 ft	32		6
60	- gray and brownish gray below 58 ft	37		
65	Dense brownish gray fine to medium sand, slightly silty (SM-SP) w/trace coarse sand and fine gravel	51		5
70		52		
75		63		
80	- medium dense from 78 to 83 ft	27		
85	- dense from 83 to 98 ft	61		7
		52		

COMPLETION DEPTH: 130.0 ft
DATE: 5-31-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

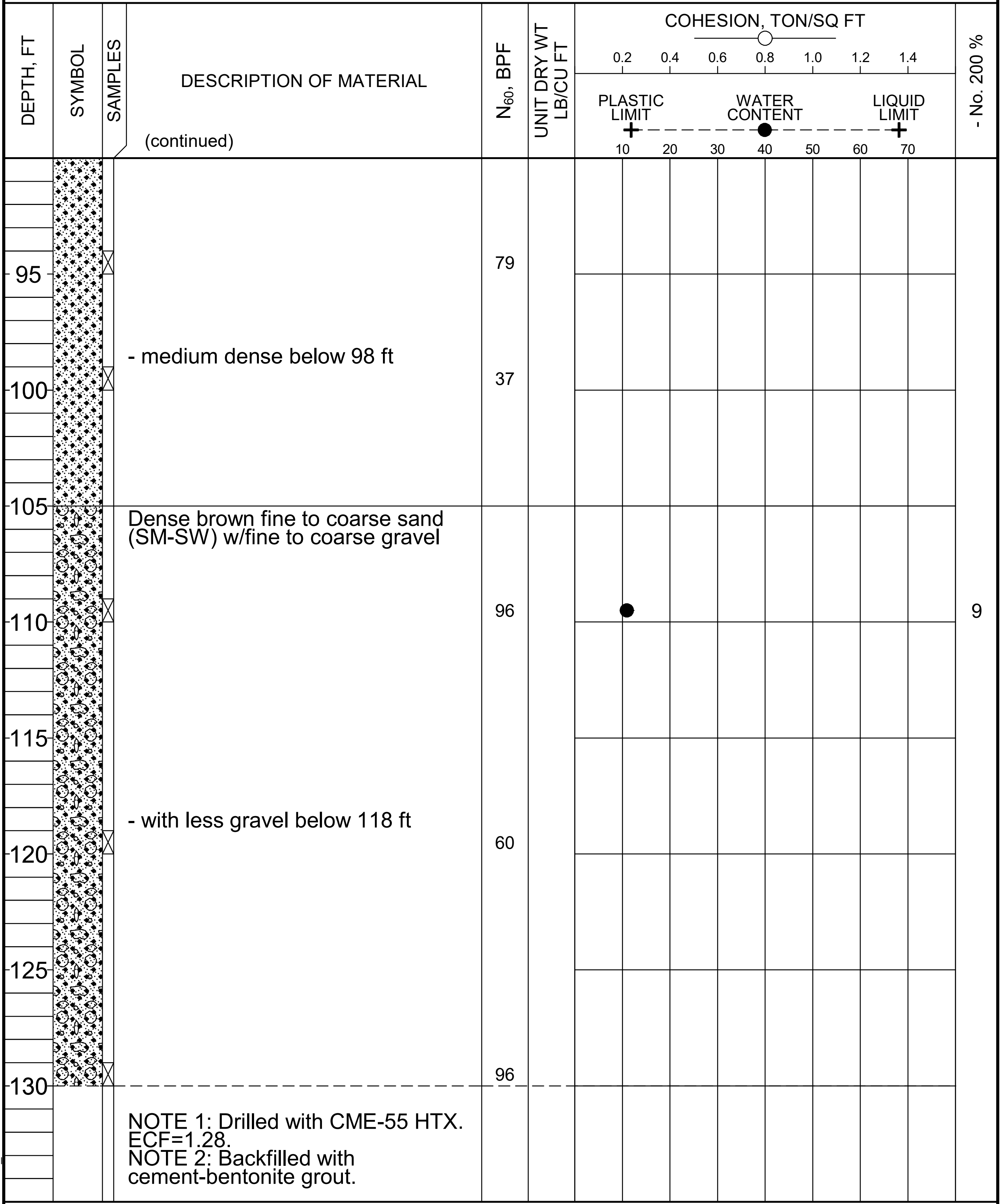
DATE: 5/30/2023

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

L O G O F B O R I N G N O. E1

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 30 ft /Wash
LOCATION: Approx Sta 319+50, 20 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
95		79		
100	- medium dense below 98 ft	37		
105	Dense brown fine to coarse sand (SM-SW) w/fine to coarse gravel			
110		96		9
115				
120	- with less gravel below 118 ft	60		
125				
130		96		

NOTE 1: Drilled with CME-55 HTX. ECF=1.28.
NOTE 2: Backfilled with cement-bentonite grout.

COMPLETION DEPTH: 130.0 ft
DATE: 5-31-23

DEPTH TO WATER
IN BORING: Dry to 30 ft

DATE: 5/30/2023

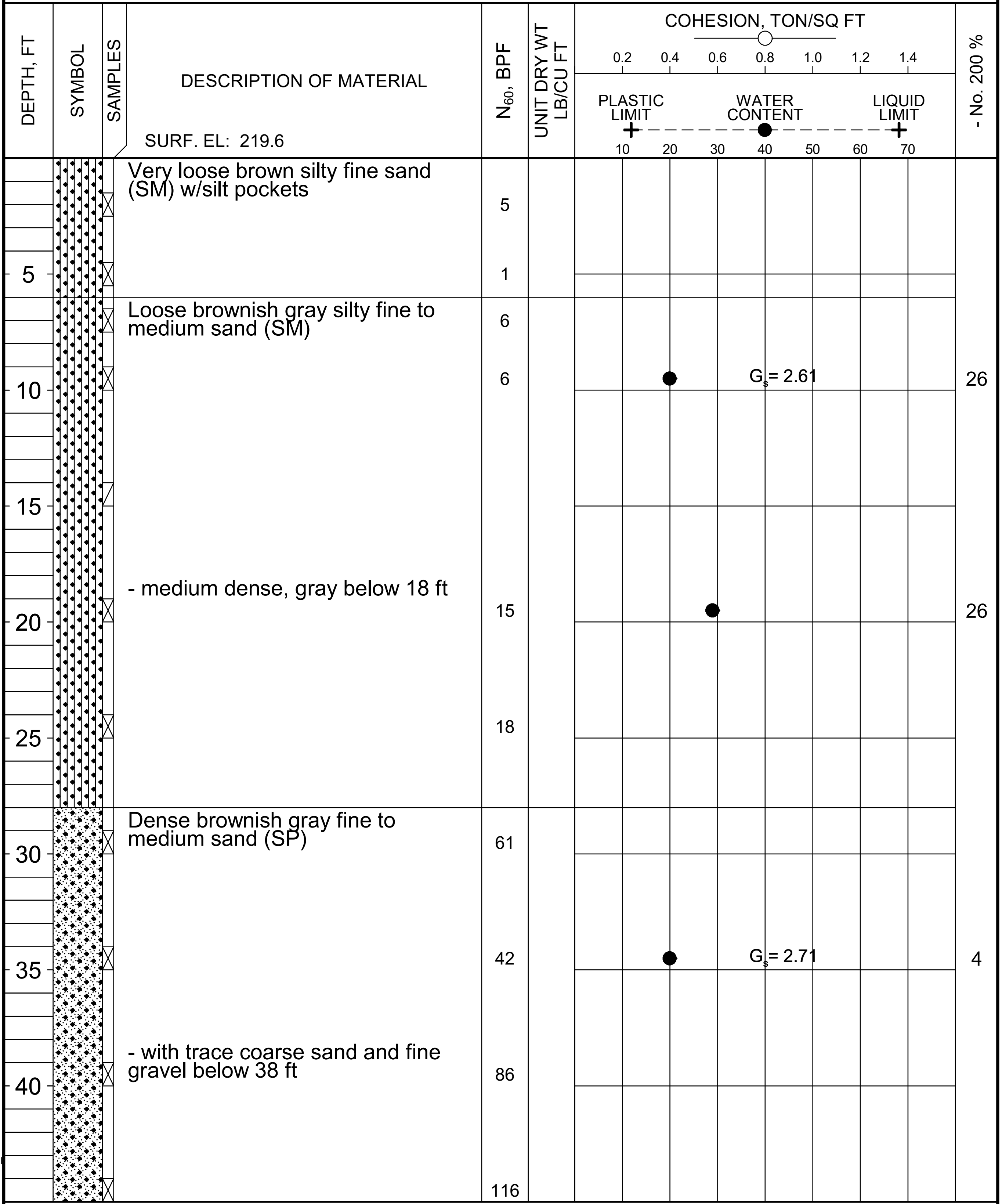
LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

LOG OF BORING NO. E2

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash
LOCATION: Approx Sta 321+05, 15 ft Lt



SURF. EL: 219.6

DEPTH, FT	DESCRIPTION OF MATERIAL	N₆₀, BPF	UNIT DRY WT LB/CU FT	COHESION / WATER CONTENT	- No. 200 %
	Very loose brown silty fine sand (SM) w/silt pockets	5			
5		1			
	Loose brownish gray silty fine to medium sand (SM)	6			
10		6		G_s = 2.61	26
15					
20	- medium dense, gray below 18 ft	15			26
25		18			
30	Dense brownish gray fine to medium sand (SP)	61			
35		42		G_s = 2.71	4
40	- with trace coarse sand and fine gravel below 38 ft	86			
		116			

COMPLETION DEPTH: 111.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 5/23/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

LOG OF BORING NO. E2

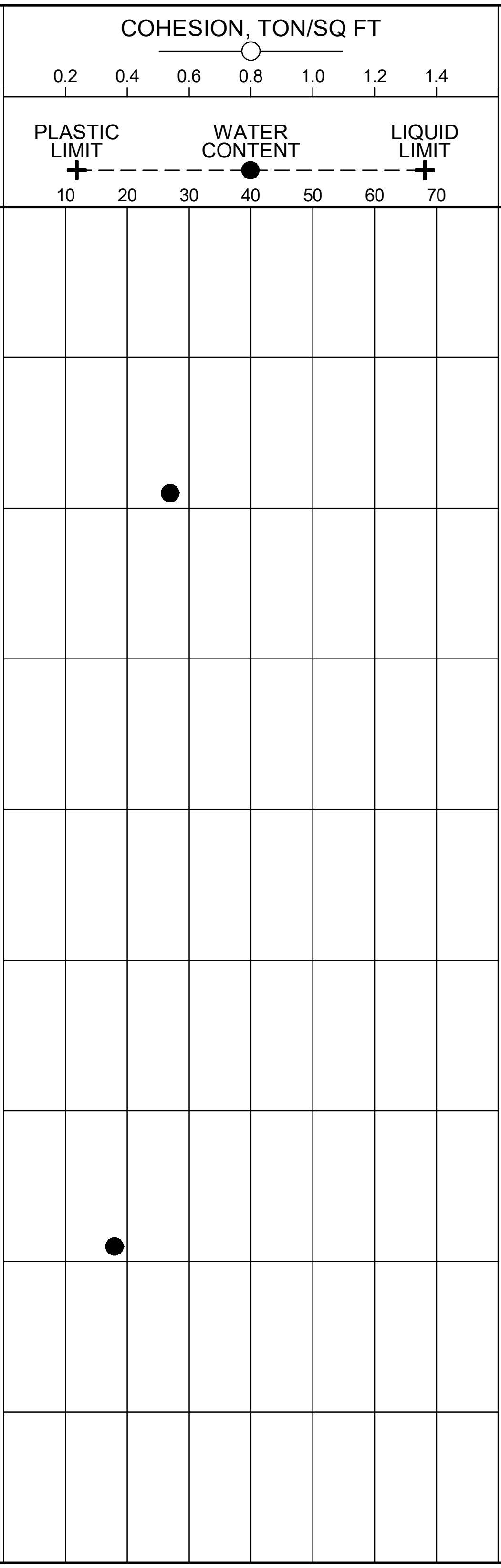
101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 321+05, 15 ft Lt

DESCRIPTION OF MATERIAL (continued)

DEPTH, FT	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	128		
55	154		3
60	56		
65	116		
70	77		
75	42		
80	44		3
85	73		
	81		



COMPLETION DEPTH: 111.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 5/23/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

LOG OF BORING NO. E2

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 321+05, 15 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
95		☒		154			
100		☒	Dense silty fine to medium sand (SM) w/trace fine gravel	46			
105							
110		☒		52			
115			NOTE: Drilled with CME-55 HTX ECF= 1.28				
120							
125							
130							

COMPLETION DEPTH: 111.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 13 ft

DATE: 5/23/2023

23-031

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

LOG OF BORING NO. E3

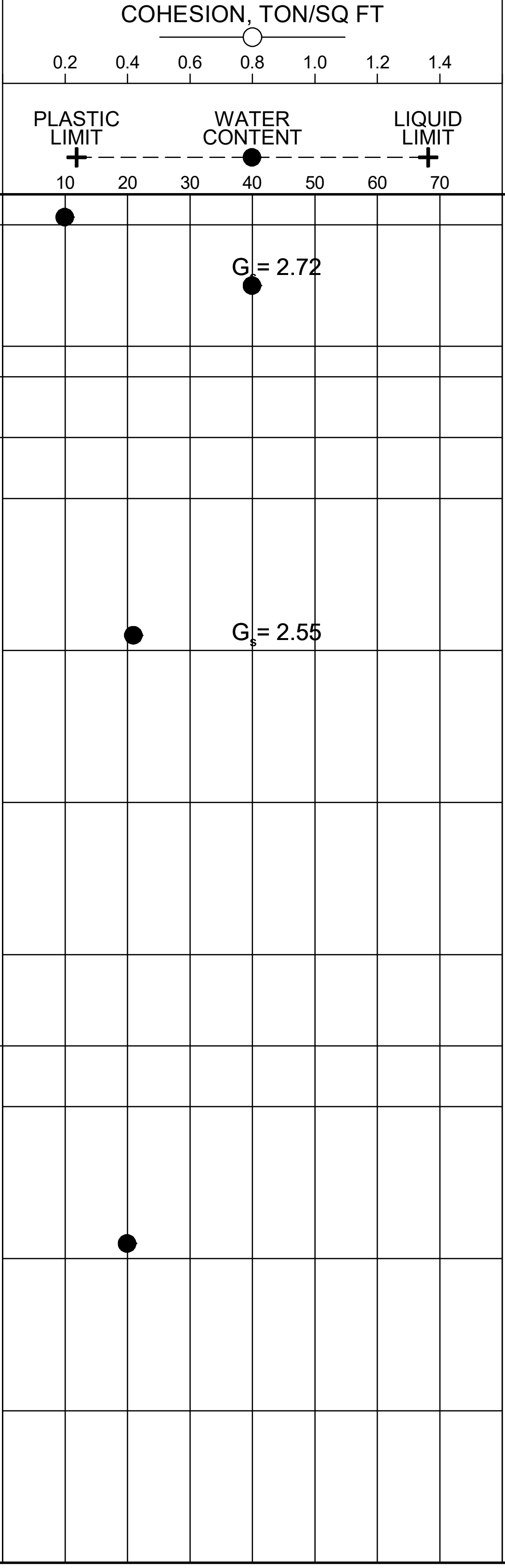
101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 321+20, 25 ft Rt

SURF. EL: 220.3

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	☒	Loose brown and gray clayey fine sand (SC)	10		31
	☒	Firm dark brown clay, slightly sandy (CH)	8		81
5	☒		6		
	☒	Soft brown and gray silt, slightly sandy (ML)	5		
10	☒	Loose to medium dense gray and brown fine sand, slightly silty (SM-SP)	13		
		- loose at 13 to 23 ft			
15	☒		10		11
20	☒		12		
		- medium dense below 23 ft			
25	☒		14		
30	☒	Medium dense gray fine sand, slightly silty (SP-SM)	37		
		- dense below 33 ft			
35	☒		49		6
		- gray and brown below 38 ft			
40	☒		59		
	☒		63		



COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 12.8 ft

DATE: 5/31/2023

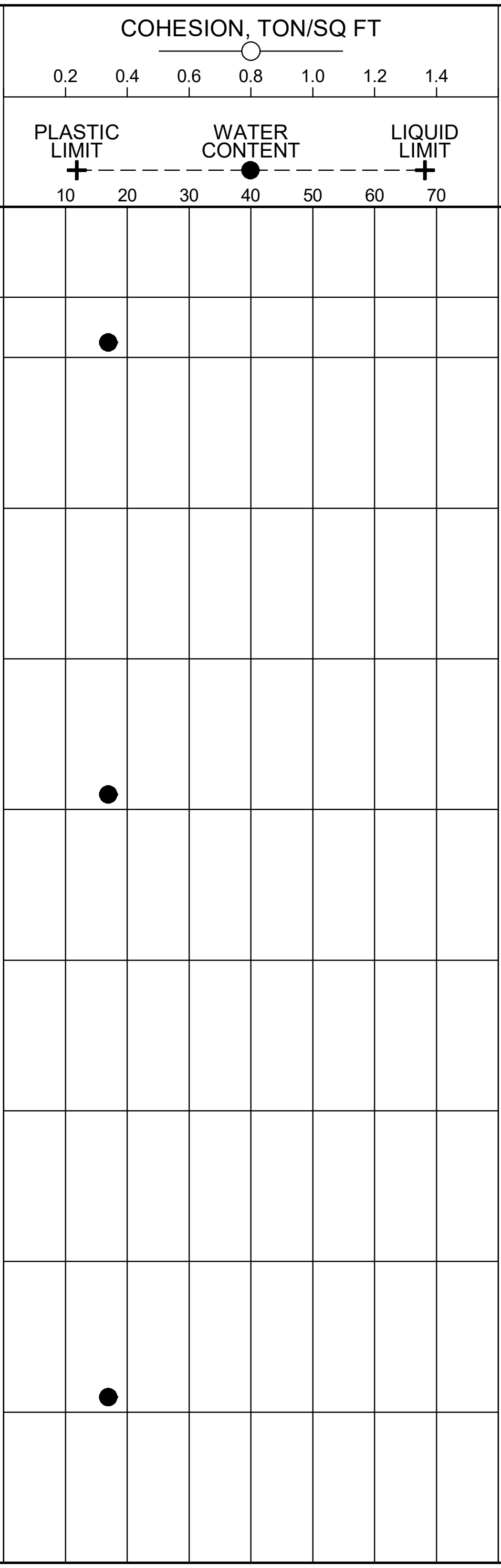
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

L O G O F B O R I N G N O. E3

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash
LOCATION: Approx Sta 321+20, 25 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	Dense to very dense gray and brown fine to medium sand, slightly silty (SM-SP) w/trace coarse sand and fine gravel	86		5
55		128		
60	- dense with less silt (SP) below 58 ft	61		
65		55		2
70		47		
75	- gray below 74ft	46		
80		44		
85		42		4
	- with some fine to coarse gravel	42		



COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 12.8 ft

DATE: 5/31/2023

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

LOG OF BORING NO. E3

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 321+20, 25 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
			below 89 ft				
95		☒	- with fine to coarse gravel below 94 ft	49			
100		☒		60			
105							
110		☒		77			
			NOTE: Drilled with Diedrich D-50 ECF= 1.43				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 12.8 ft

DATE: 5/31/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

23-031

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

LOG OF BORING NO. E4

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Approx Sta 322+00, 20 ft Lt

SURF. EL: 218.3

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	PLASTIC LIMIT	WATER CONTENT	LIQUID LIMIT	- No. 200 %
	Very soft brown clay (CH) w/fine sand pockets	4			21		
	Soft brown clayey silt, sandy (CL-ML)	9		19	31	23	73
5	Loose brownish gray fine sand (SP)	7		-NON-PLASTIC-	20		2
	- very loose to loose at 6 to 8 ft	6					
	- very loose below 8 ft	4					
10							
15	Medium dense brownish gray fine sand, slightly silty (SP-SM)	30					
	- dense below 18 ft						
20		89			23		6
25		143					
30		96					
35	Dense brownish gray fine to medium sand (SP) w/trace coarse sand and fine gravel	73					
40		90					
		79					

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

COMPLETION DEPTH: 110.0 ft
DATE: 5-26-23

DEPTH TO WATER
IN BORING: 4.7 ft

DATE: 5/25/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

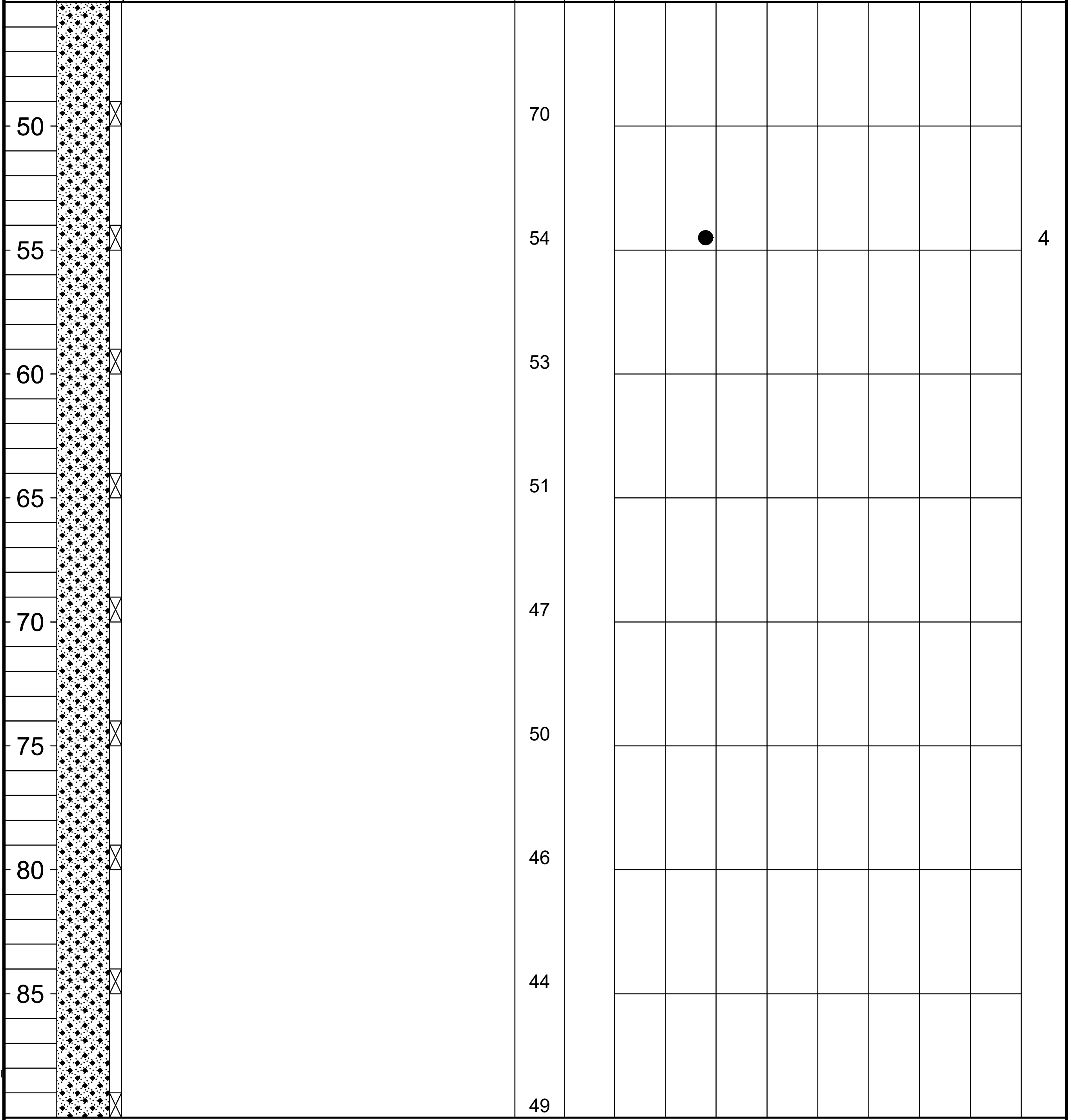
LOG OF BORING NO. E4

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Approx Sta 322+00, 20 ft Lt

DESCRIPTION OF MATERIAL (continued)



DEPTH, FT	N₆₀, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	70		
55	54		4
60	53		
65	51		
70	47		
75	50		
80	46		
85	44		
	49		

COMPLETION DEPTH: 110.0 ft
DATE: 5-26-23

DEPTH TO WATER
IN BORING: 4.7 ft

DATE: 5/25/2023

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

LOG OF BORING NO. E4

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Approx Sta 322+00, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N₆₀, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
95		☒		54			
100		☒		60			
105			Dense to very dense brownish gray fine sand, slightly silty (SP-SM)				
110		☒		79		●	6
			NOTE 1: Drilled with Diedrich D-50 ECF= 1.43				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 5-26-23

DEPTH TO WATER
IN BORING: 4.7 ft

DATE: 5/25/2023

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

LOG OF BORING NO. E5

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger

LOCATION: Approx Sta 322+95, 30 ft Lt

SURF. EL: 220±

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2 0.4 0.6 0.8 1.0 1.2 1.4) / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT (10 20 30 40 50 60 70)	- No. 200 %
0 – 4.5	▨		Very loose brownish gray silty fine sand w/organics				
4.5			- refusal on riprap at 4.5 ft				
			NOTE 1: Drilled through bridge deck. NOTE 2: Drilled with CME-SSHTX. NOTE 3: 18.1 ft deck to mudline. NOTE 4: Set 20 ft HW casing. NOTE 5: Boring abandoned at 4.5 ft.				

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

COMPLETION DEPTH: 4.5 ft
DATE: 6-7-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/7/2023

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

LOG OF BORING NO. E6

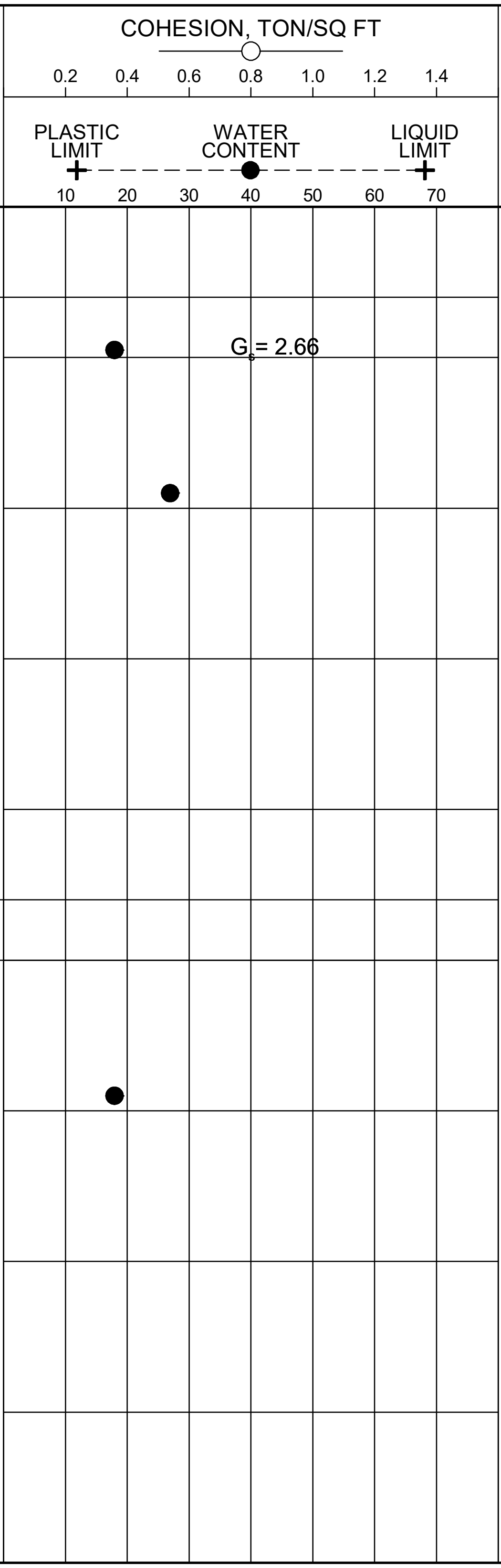
101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 324+40, 30 ft Lt

SURF. EL: 204±

DEPTH, FT	DESCRIPTION OF MATERIAL	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Loose dark gray and brown fine clayey fine sand (SC) w/organics and fine gravel			
	Very loose brown and dark gray silty fine sand, slightly silty (SM-SP) w/organics	5		
5		3		5
10	- gray and brown below 9 ft - medium dense with less silt (SP) from 9 to 14 ft	18		4
15	- very dense from 14 to 19 ft	73		
20	- with fine gravel layers below 20 ft	22		
25	Medium dense to dense gray and brown medium sand (SP) w/trace coarse sand and occasional organic inclusions Medium dense gray and brown fine to medium sand (SP) w/trace coarse sand and fine gravel	49		
30		35		3
35	- dense from 34 to 38 ft	59		
40	- medium dense below 38 ft	35		
		17		



COMPLETION DEPTH: 100.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 18.5 ft

DATE: 6/6/2023

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

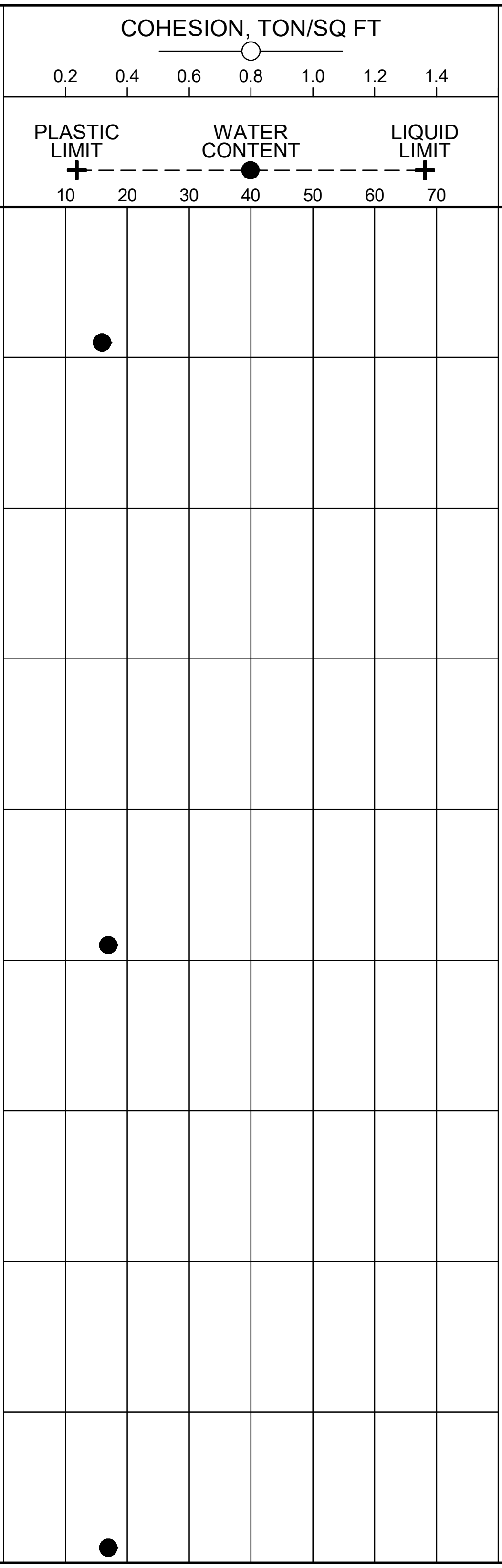
LOG OF BORING NO. E6

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 324+40, 30 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		20		3
55	- dense, slightly silty (SM-SP) below 54 ft	27		
60		49		
65		52		
70		60		5
75	- with a little fine gravel at 74 to 79 ft	44		
80		50		
85		52		
		55		5



COMPLETION DEPTH: 100.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 18.5 ft

DATE: 6/6/2023

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

LOG OF BORING NO. E6

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 324+40, 30 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
95		☒		65			
100		☒		192			
105			NOTE: Drilled with CME-55 HTX ECF=1.28				
110							
115							
120							
125							
130							

COMPLETION DEPTH: 100.0 ft
DATE: 6-5-23

DEPTH TO WATER
IN BORING: 18.5 ft

DATE: 6/6/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

LOG OF BORING NO. E7

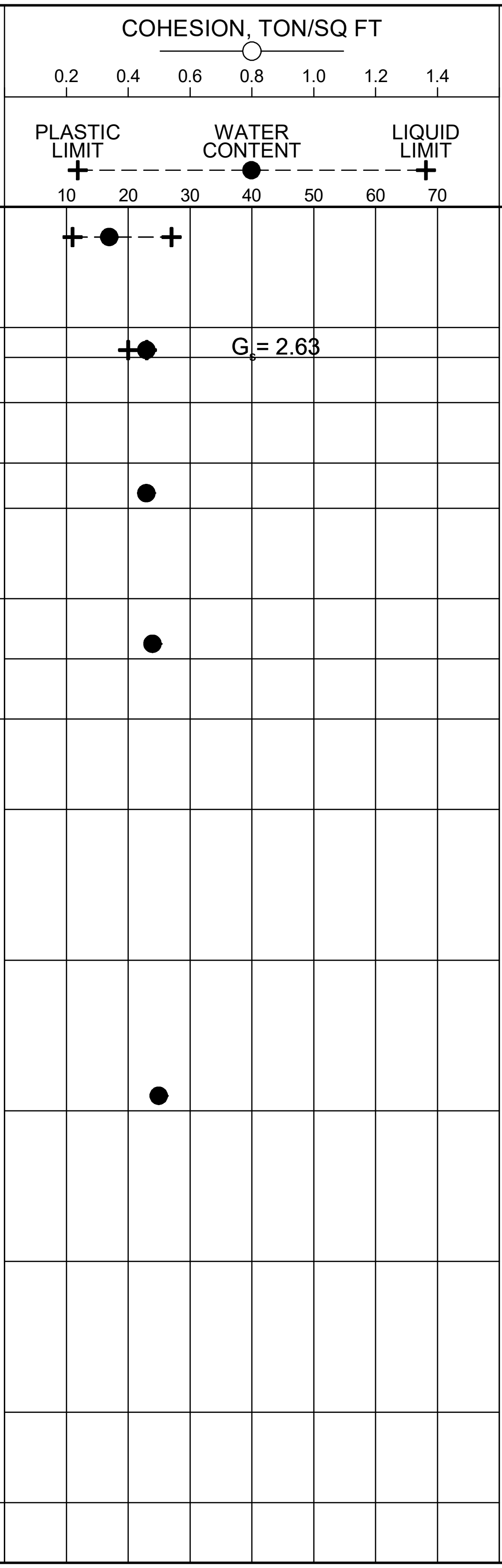
101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 20 ft Lt

SURF. EL: 219.1

DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Loose brown clayey fine sand (SC)	7 11		42
Firm gray silt (ML) w/silty fine sand seams and layers	9		51
Soft brown fine sandy clay (CL)	4		
Very loose gray fine sand, slightly silty (SM-SP) w/clay seams and layers	6		10
Loose brownish gray silty fine sand (SM)	9		20
Medium dense brown fine sand (SP)	24 23 20 24 29		3
Medium dense brownish gray and brown fine to medium sand (SP)	37		



DEPTH, FT: 5 10 15 20 25 30 35 40

COMPLETION DEPTH: 110.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 10.5 ft

DATE: 5/22/2023

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

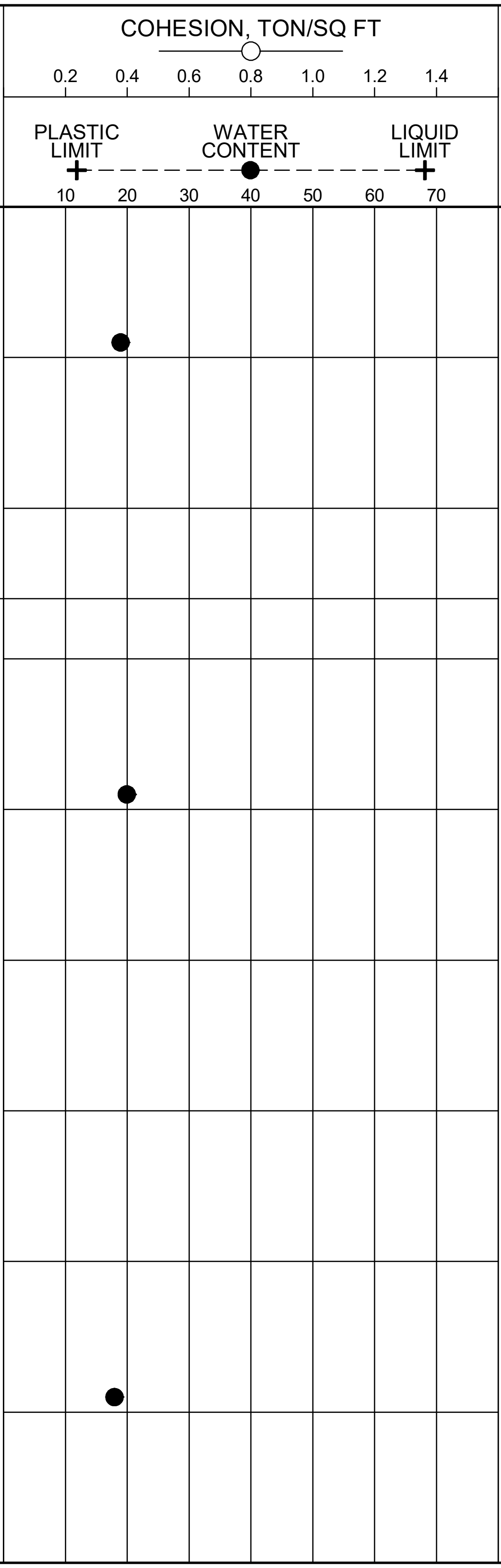
LOG OF BORING NO. E7

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
			w/organic inclusions			
50		☒		30		4
55		☒		39		
			Medium dense grayish brown fine to medium sand (SP) w/trace coarse sand			
60		☒		37		
65		☒		36		4
			- dense below 68 ft			
70		☒		43		
75		☒		49		
80		☒		60		
			- slightly silty (SM-SW) below 83 ft			
85		☒		90		6
		☒		66		



COMPLETION DEPTH: 110.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 10.5 ft

DATE: 5/22/2023

23-031

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

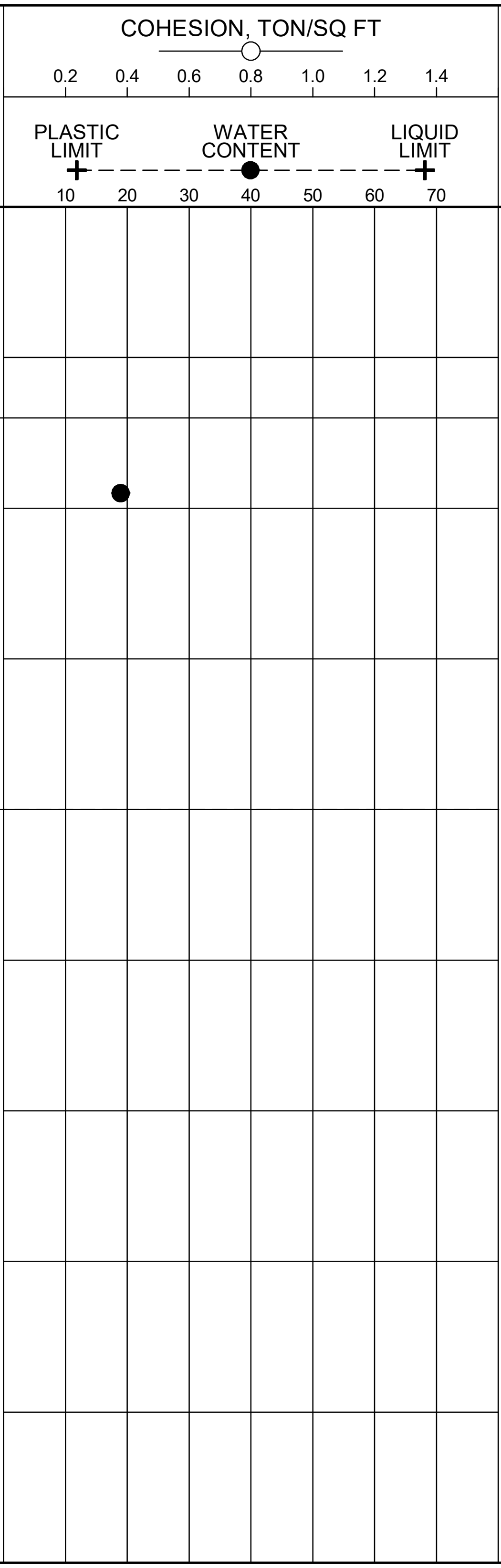
LOG OF BORING NO. E7

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N₆₀, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
95		☒		59		
100		☒	Dense grayish brown fine to medium sand, slightly silty (SM-SP) w/trace coarse sand	51		7
105						
110		☒		60		
115			NOTE: Drilled with Diedrich D-50 ECF=1.43			
120						
125						
130						



COMPLETION DEPTH: 110.0 ft
DATE: 5-23-23

DEPTH TO WATER
IN BORING: 10.5 ft

DATE: 5/22/2023

LGBNEW 23-031 BRIDGE E.GPJ 7-28-23

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

LOG OF BORING NO. E8

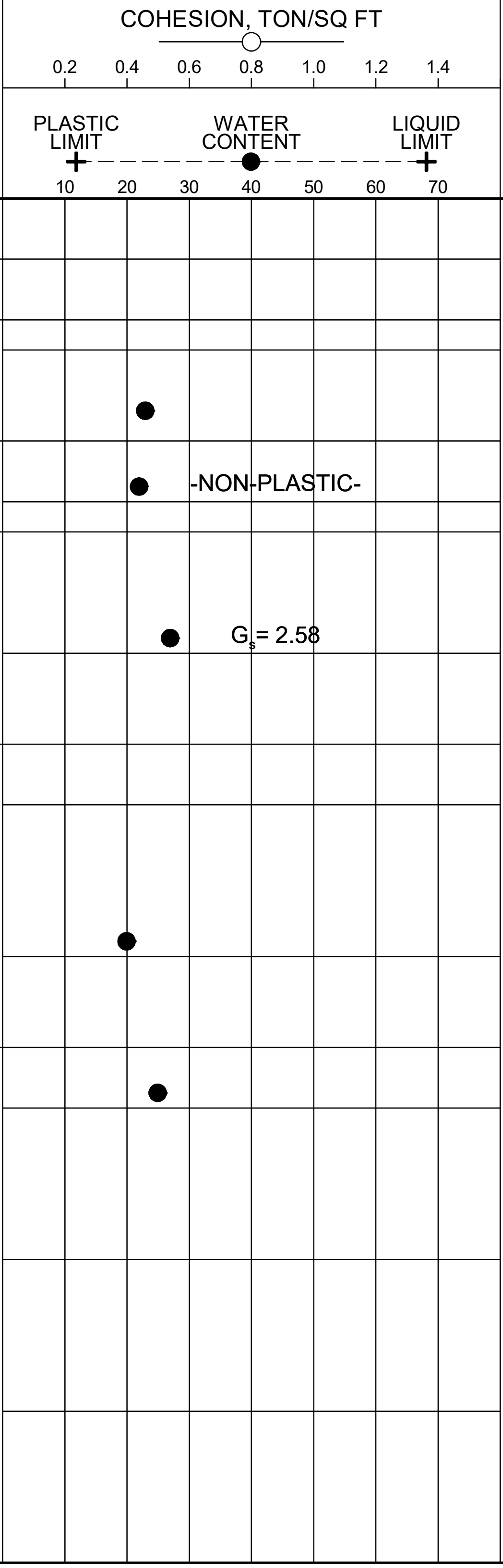
101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 25 ft Rt

SURF. EL: 220±

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown silty clay, (CL) slightly sandy	9		
	Loose brown silty fine sand (SM) w/clay pockets	7		
5	Loose tan and brownish gray silty fine sand (SM)	13		
	- very loose below 6 ft	3		35
10	Loose grayish brown fine sandy SILT (ML) w/silty clay pockets and occasional organic inclusions	7		63
15	Loose brownish gray silty fine sand (SM)	11		20
20	Very loose grayish brown silty fine sand (SM)	6		
25	- loose below 23 ft	11		19
30	Medium dense brownish gray fine sand, slightly silty (SM-SP)	16		5
35		19		
40		40		
		37		



COMPLETION DEPTH: 110.0 ft
DATE: 5-24-23

DEPTH TO WATER
IN BORING: 13.2 ft

DATE: 5/23/2023

23-031

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

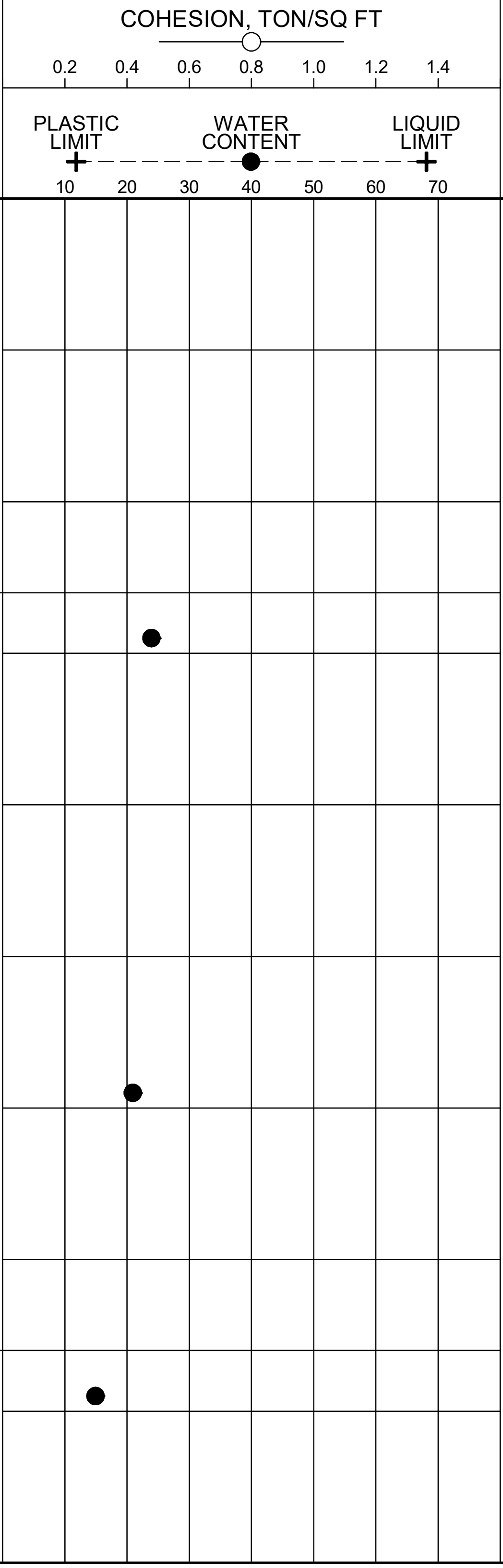
LOG OF BORING NO. E8

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 25 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		40		
55		41		
60	Medium dense brownish gray fine to medium sand (SP)	33		3
65		24		
70		30		
75	- dense, slightly silty (SM-SP) at 73 to 78 ft	44		5
80	- medium dense below 78 ft	37		
85	Medium dense grayish brown fine to medium sand, slightly silty (SM-SW) w/trace coarse sand and fine gravel	41		6
	- dense below 88 ft	47		



COMPLETION DEPTH: 110.0 ft
DATE: 5-24-23

DEPTH TO WATER
IN BORING: 13.2 ft

DATE: 5/23/2023

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

L O G O F B O R I N G N O. E8

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 326+20, 25 ft Rt

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	- No. 200 %
95		☒		64			
100		☒		59			
105							
110		☒		67			
115							
120							
125							
130							

NOTE: Drilled with Diedrich D-50
ECF= 1.43.

COMPLETION DEPTH: 110.0 ft
DATE: 5-24-23

DEPTH TO WATER
IN BORING: 13.2 ft

DATE: 5/23/2023

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

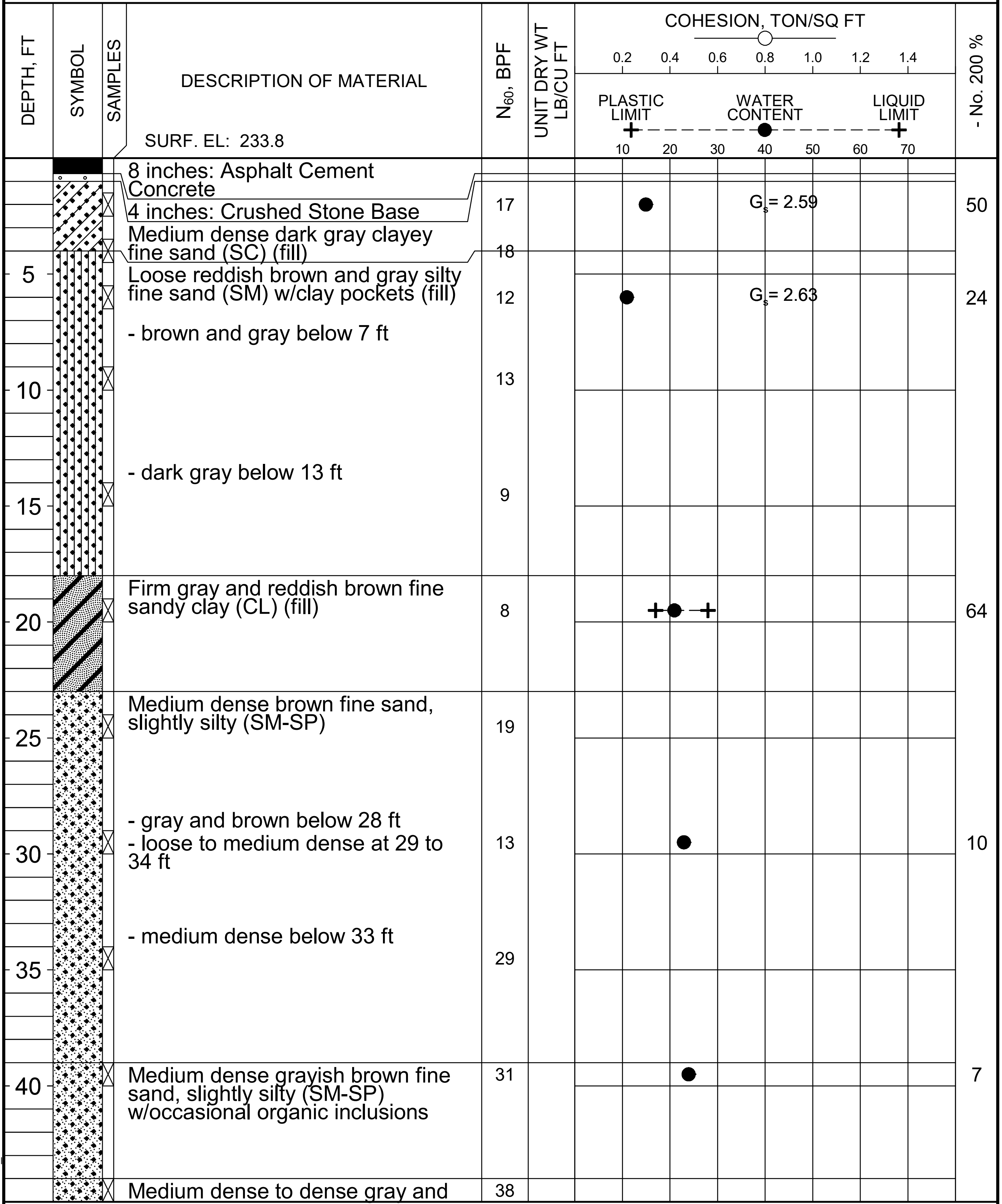
LOG OF BORING NO. E9

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 327+60, 30 ft Lt

SURF. EL: 233.8



DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
8 inches: Asphalt Cement Concrete			
4 inches: Crushed Stone Base	17		50
Medium dense dark gray clayey fine sand (SC) (fill)	18		
Loose reddish brown and gray silty fine sand (SM) w/clay pockets (fill)	12		24
- brown and gray below 7 ft	13		
- dark gray below 13 ft	9		
Firm gray and reddish brown fine sandy clay (CL) (fill)	8		64
Medium dense brown fine sand, slightly silty (SM-SP)	19		
- gray and brown below 28 ft - loose to medium dense at 29 to 34 ft	13		10
- medium dense below 33 ft	29		
Medium dense grayish brown fine sand, slightly silty (SM-SP) w/occasional organic inclusions	31		7
Medium dense to dense gray and	38		

COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 28 ft

DATE: 6/1/2023

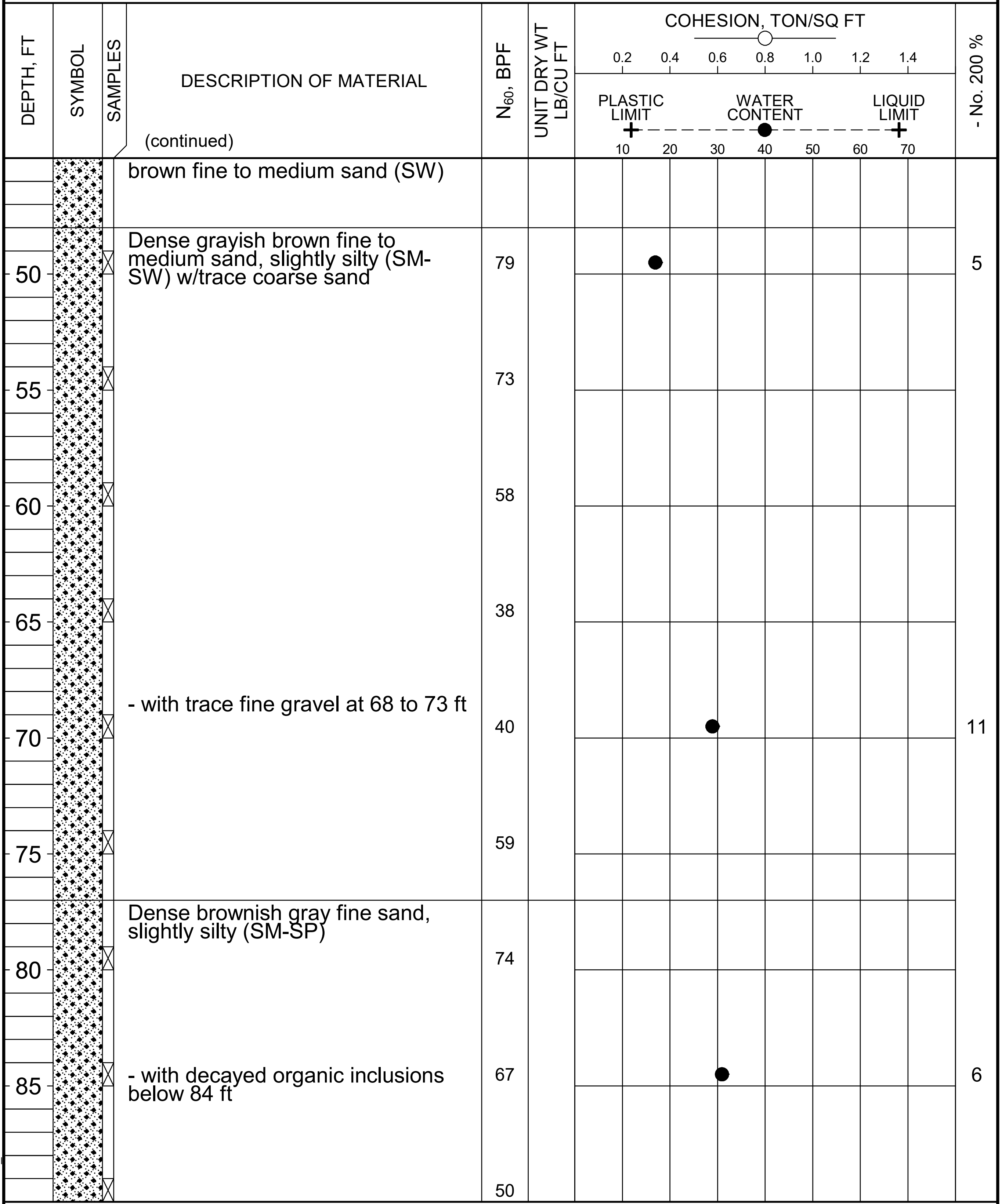
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. E9

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 327+60, 30 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N₆₀, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	brown fine to medium sand (SW)			
50	Dense grayish brown fine to medium sand, slightly silty (SM-SW) w/trace coarse sand	79		5
55		73		
60		58		
65		38		
70	- with trace fine gravel at 68 to 73 ft	40		11
75		59		
80	Dense brownish gray fine sand, slightly silty (SM-SP)	74		
85	- with decayed organic inclusions below 84 ft	67		6
		50		

COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 28 ft

DATE: 6/1/2023

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

LOG OF BORING NO. E9

101124 Hwy. 135 over Rt Hand Chute of Little River
Poinsett Co., Arkansas

TYPE: Auger to 35 ft /Wash

LOCATION: Approx Sta 327+60, 30 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
95		☒	Dense brown and gray fine to medium sand (SP) w/trace coarse sand and fine gravel	77			
100		☒		59			
105							
			- dense to very dense below 107 ft				
110		☒		128			
			NOTE 1: Drilled with CME-55 HTX ECF= 1.28 NOTE 2: Backfilled with cement-bentonite grout.				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-1-23

DEPTH TO WATER
IN BORING: 28 ft

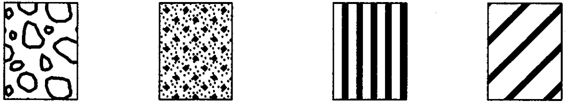
DATE: 6/1/2023

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

(SHOWN IN SYMBOLS COLUMN)

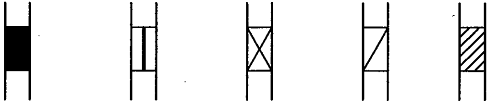


Gravel Sand Silt Clay

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

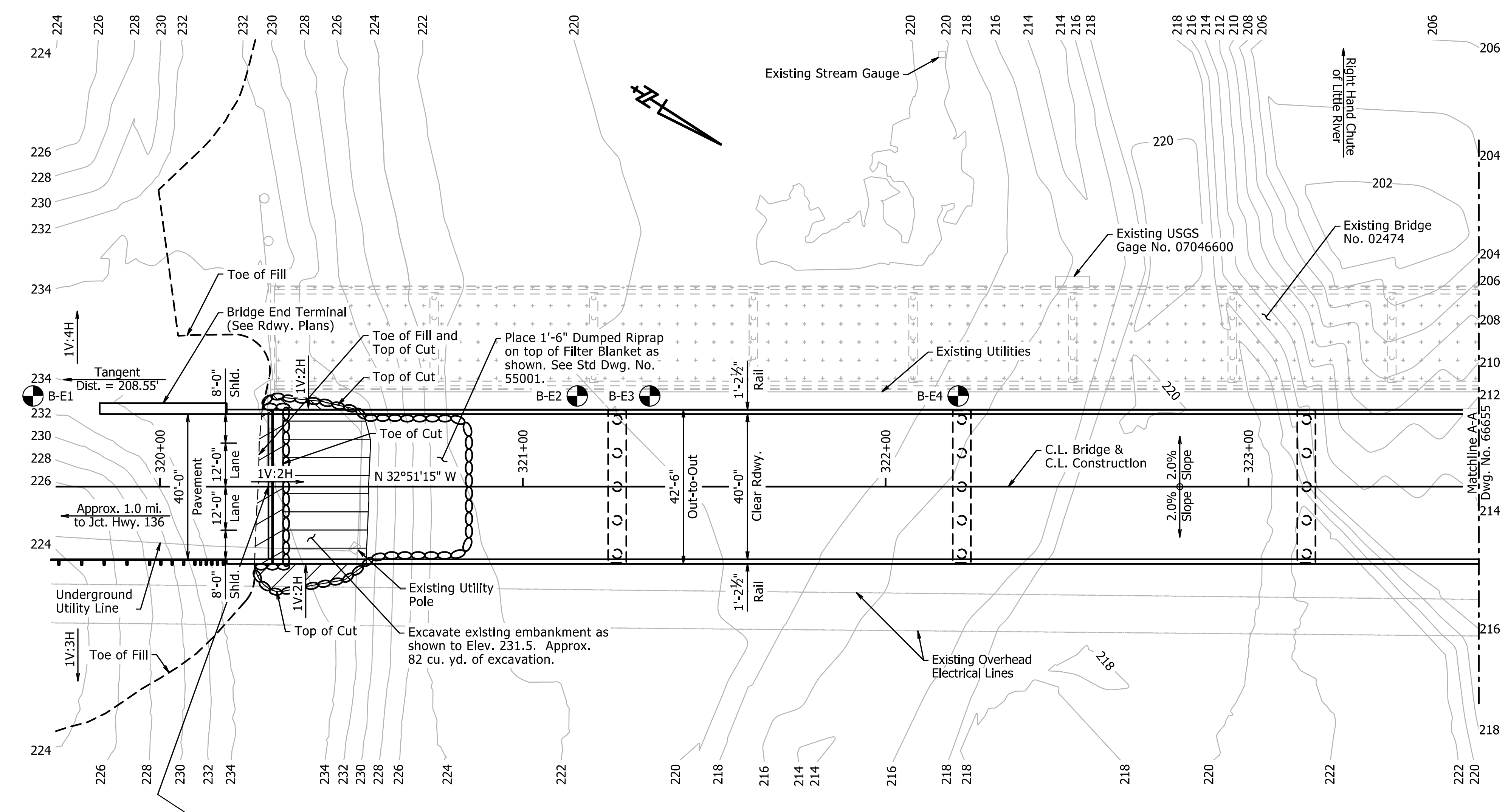
SLICKENSIDED - having inclined planes of weakness that are slick and glossy in appearance.
FISSURED - containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
LAMINATED - composed of thin layers of varying color and texture.
INTERBEDDED - composed of alternate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

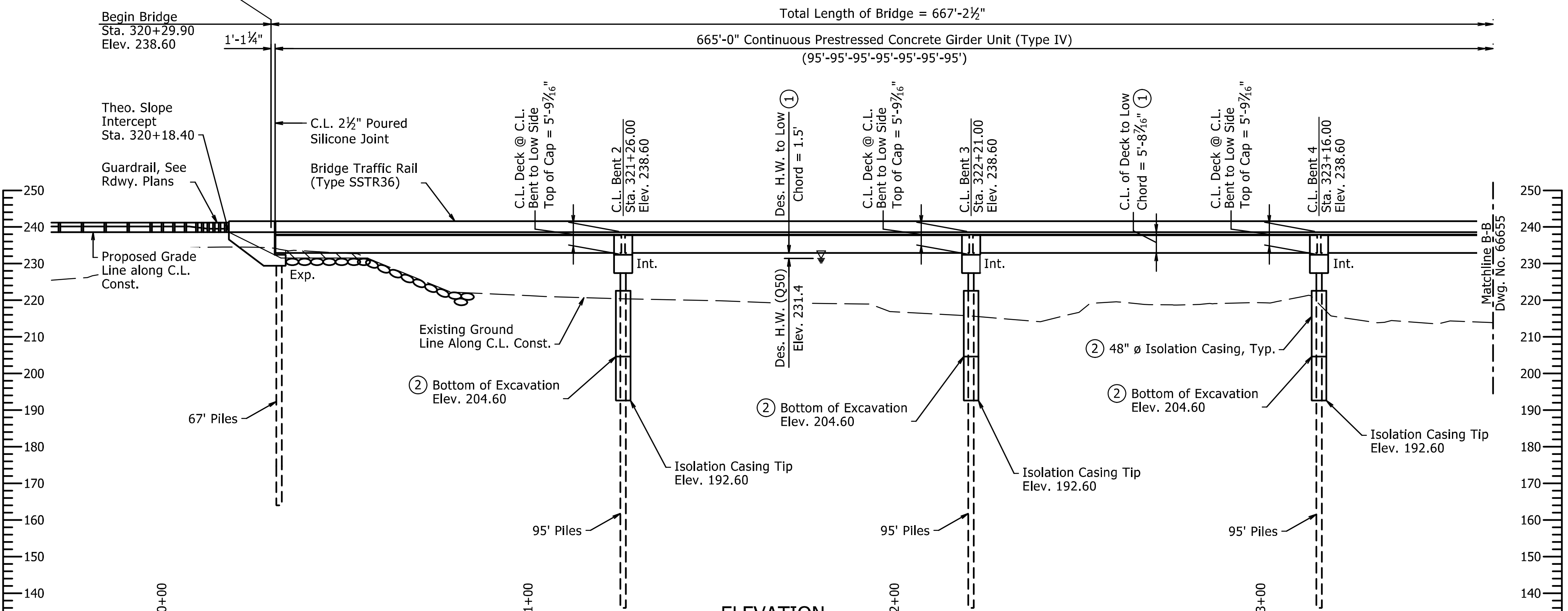
KEY 7-28-11

APPENDIX A

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	208	356
		07651		LAYOUT		66654



PLAN



ELEVATION

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

SEISMIC ZONE: 4 S_{D1}=1.247 SITE CLASS: D

SEISMIC OPERATIONAL CLASS: OTHER

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 8,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

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

STEEL SHELL PILING: Piling in Bents 1 and 8 shall be 24" diameter concrete filled steel shell piles and shall be driven to meet the requirements of the "PILE BEARING TABLE" on Dwg. No. 66655. The 24" diameter piles shall have a nominal wall thickness of ¾". Piling in Bents 2 thru 7 shall be 30" diameter concrete filled steel shell piles and shall be driven to meet the requirements of the "PILE BEARING TABLE" on Dwg. No. 66655. The 30" diameter piles shall have a nominal wall thickness of ¾". All piling shall be driven with an approved air, steam, or diesel hammer to the minimum tip elevation as specified in the "PILE BEARING TABLE" on Dwg. No. 66655. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g). No payment shall be made for test piles.

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (30" Dia.)".

DRIVING SYSTEM: The driving system approval and the ultimate bearing capacity determination for piling shall be based on the requirements of Subsection 805.09(b), "Method B - Wave Equation Analysis (WEAP)" and SP "PILE DRIVING SYSTEM". See the "PILE BEARING TABLE" for the estimated minimum rated hammer energy required to overcome the anticipated driving resistance for all piles at each bent. If the Contractor elects to use water jetting or other approved methods to obtain the minimum tip elevations shown while driving only to the required minimum ultimate bearing capacity, the minimum rated hammer energy required will be lower and shall be accounted for in the driving system chosen by the Contractor.

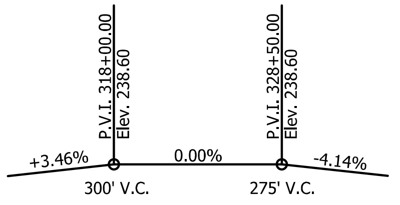
For Additional General Notes see Dwg. No. 66655.

For R/W Data, See Roadway Plans.

For Soil Boring Information, see Dwg. Nos. 66656 & 66657.

① See "HYDRAULIC DATA" table on Dwg. No. 66656.

② See SP Job No. 101124 "ISOLATION CASING".



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along
C.L. Construction

Note:
Use Type Special Approach Gutters and Type C2 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. 66675 & 55040C2, respectively. Eliminate or modify Type Special Approach Gutter curb section to fit bridge end terminal. No additional payment will be made for this work.

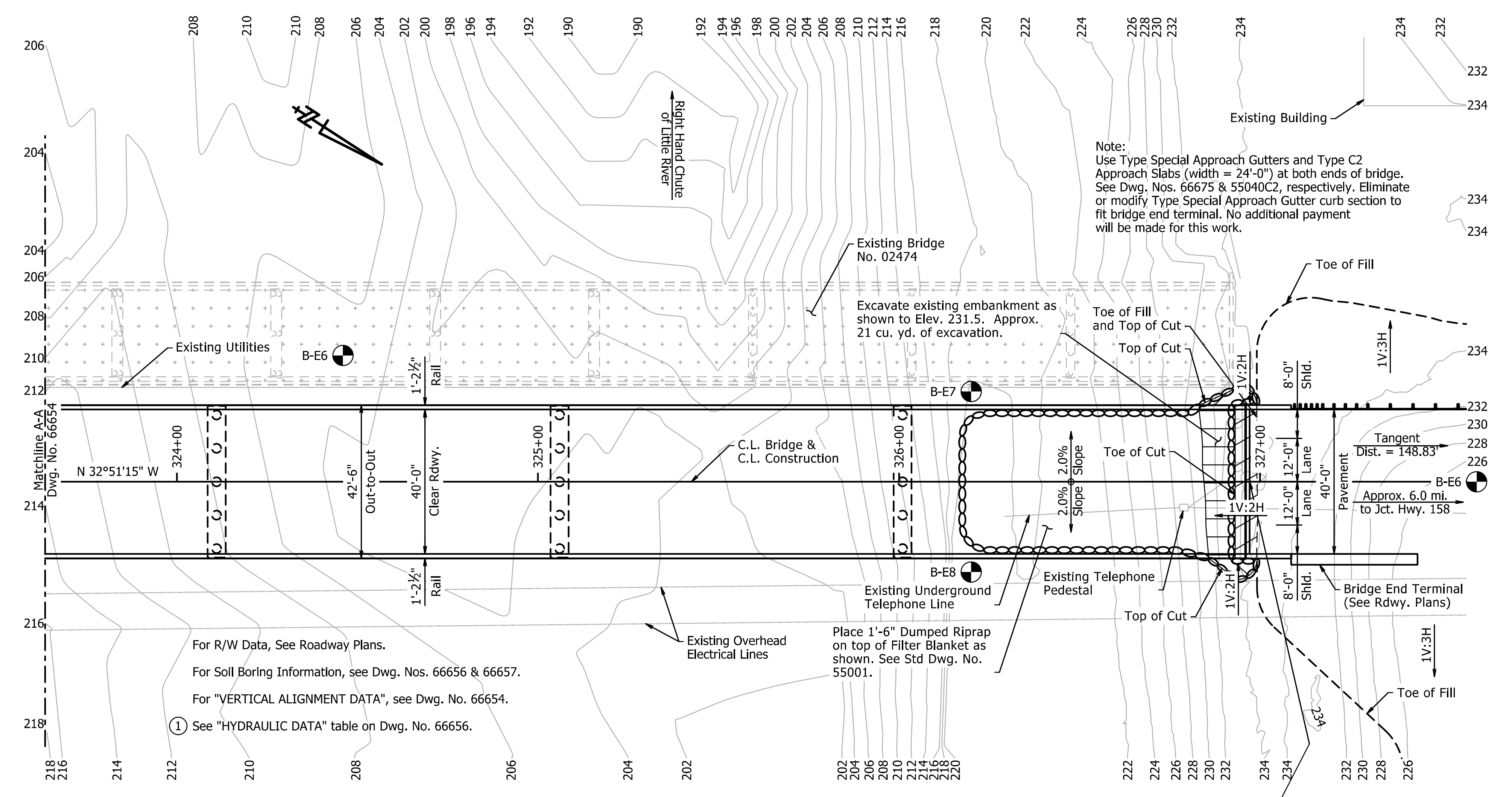
Note:
Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "ROUNDING DETAIL" on Dwg. No. 66667.

No. 11856
CHARLES A. WIPF

SHEET 1 OF 4
LAYOUT OF BRIDGE
HWY. 135 OVER RIGHT HAND CHUTE LITTLE RIVER
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: JRF DATE: 10-06-2022 FILENAME: b101124x5_l1.dgn
CHECKED BY: CAW DATE: 11-07-2022 SCALE: 1" = 20'
DESIGNED BY: JRF DATE: 10-04-2022
BRIDGE NO. 07651 DRAWING NO. 66654

USER: CTAUSER
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x5_l1.dgn SCALE: 40.0000 ' / in.
PLOTTED: 9/8/2023 10:42:52 AM



Note:
Use Type Special Approach Gutters and Type C2 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. 66675 & 55040C2, respectively. Eliminate or modify Type Special Approach Gutter curb section to fit bridge end terminal. No additional payment will be made for this work.

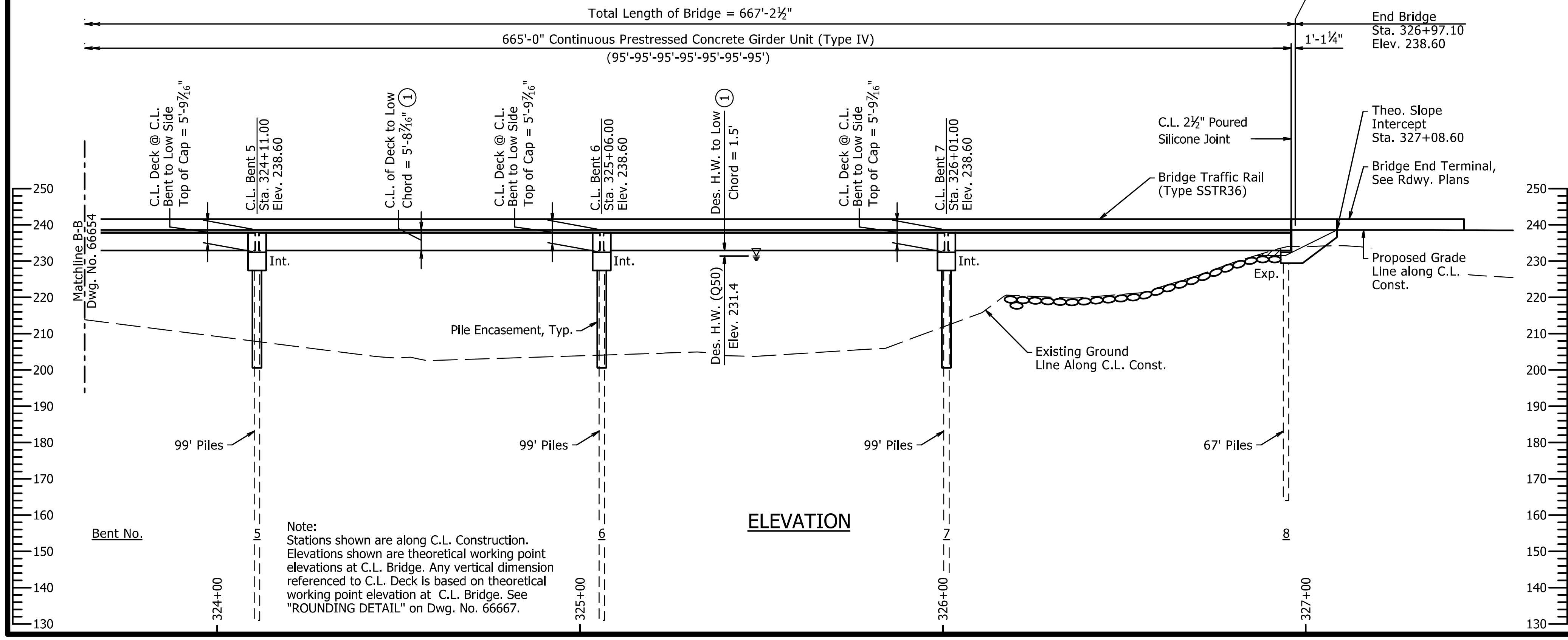
For R/W Data, See Roadway Plans.

For Soil Boring Information, see Dwg. Nos. 66656 & 66657.

For "VERTICAL ALIGNMENT DATA", see Dwg. No. 66654.

① See "HYDRAULIC DATA" table on Dwg. No. 66656.

PLAN



Note:
Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "ROUNDING DETAIL" on Dwg. No. 66667.

ELEVATION

GENERAL NOTES (CONT'D.)

PILE ENCASEMENT: Pile encasement for Bents 5 thru 7 shall extend from bottom of cap to 3' below natural or finished ground. See Std. Dwg. No. 55021 & Dwg. No. 66666 for additional information. Piles at Bents 2 thru 4 shall be covered by 48" Dia. Isolation Casings in accordance with the Job 101124 SP "ISOLATION CASING". See Dwg. No. 66665 for additional details.

BRIDGE DECK: The concrete bridge deck shall be given a tine finish as specified for final finishing in Subsection 802.19 for Class 5 Tined Bridge Roadway Surface Finish.

PROTECTIVE SURFACE TREATMENT: Class 2 Protective Surface Treatment shall be applied to the roadway surface and to the roadway face and top of the Bridge Traffic Rail in accordance with Section 803.

DETAIL DRAWINGS:	DRAWING NO(S).
End Bents	66659-66662
Intermediate Bents	66663-66665
Elastomeric Bearings	66624
665' Prestressed Concrete Girder Unit	66667-66674
Concrete Filled Steel Shell Piling	55021 & 66666
Type Special Approach Gutters	66675
Type C2 Approach Slabs	55040C2
Bridge Traffic Rail	55070

EXISTING BRIDGE: Existing Bridge No. 02474 (Log Mile 15.09) is 28.7' wide (24' clear roadway) and 662' long and consists of steel I-beam spans (15 spans total) supported by concrete piles. The existing bridge is located approximately 41' downstream from the proposed new bridge. Plans of the existing structure, if available, may be obtained upon request to the Construction Contract Development Section of the Program Management Division.

REMOVAL AND SALVAGE: After the new bridge is open to traffic, the Contractor shall remove Existing Bridge No. 02474, including existing riprap, in accordance with Section 205. Removal of existing riprap will not be paid for directly but shall be considered subsidiary to the item "Removal of Existing Bridge Structure (Site No._)". All material from the existing bridge shall become the property of the Contractor except the following:

1.The existing USGS stream gage shall remain the property of the USGS. The Contractor shall remove and store the stream gage on site in a manner approved by the Engineer. The Contractor shall notify the USGS 7 business days in advance of removing the existing stream gage. Contact information is as follows:

USGS
3918 Central Avenue
Memphis, TN 38111
901-678-7287

2. The existing utilities attached to the bridge shall remain the property of the Ritter Communications. The Contractor shall remove and store the utility items on site in a manner approved by the Engineer. The Contractor shall notify the Ritter Communications 7 business days in advance of removing the existing utility items. Contact information is as follows:

Rich Busby
2400 Ritter Dr.
Jonesboro, AR 72401
870-336-3434

This work shall be considered incidental to the item "Removal of Existing Bridge Structure (Site No._)".

MAINTENANCE OF TRAFFIC: See Roadway Plans.

PILE BEARING TABLE

BENTS	REQUIRED MINIMUM ULTIMATE BEARING CAPACITY (TONS)	MIN. TIP ELEVATION	ANTICIPATED DRIVING RESISTANCE AT MIN. TIP (TONS)	ESTIMATED MIN. RATED HAMMER ENERGY (FT. LBS. PER BLOW)
1	428	164	780	125,000
2	856	136	1280	248,000
3	856	136	1025	212,000
4	856	136	1005	212,000
5	863	131	1085	212,000
6	863	131	1195	212,000
7	863	131	945	212,000
8	428	164	640	125,000

Note: Required minimum ultimate bearing capacity corresponds to the minimum post driving capacity to be obtained after an allowance for water jetting or any other methods employed to facilitate pile installation.

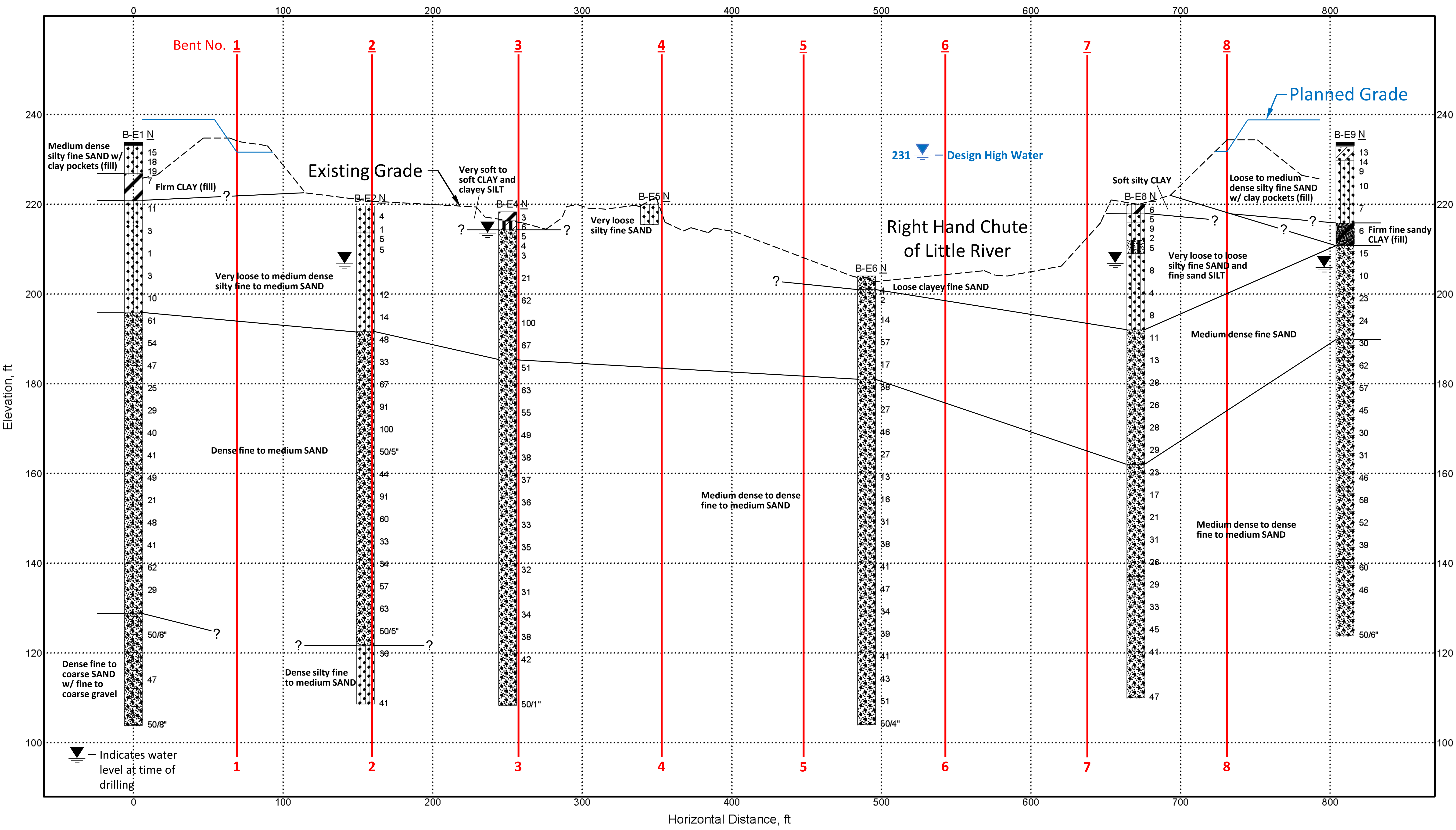
Anticipated Driving Resistance corresponds to the resistance to be overcome to achieve minimum tip elevation without any water jetting or other methods employed to facilitate pile installation.

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 11856
CHARLES A. WIPF

SHEET 2 OF 4
LAYOUT OF BRIDGE
HWY. 135 OVER RIGHT HAND CHUTE LITTLE RIVER
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: JRF DATE: 10-11-2022 FILENAME: b101124x5_l2.dgn
CHECKED BY: CAW DATE: 11-07-2022 SCALE: 1" = 20'
DESIGNED BY: JRF DATE: 10-04-2022
BRIDGE NO. 07651 DRAWING NO. 66655

APPENDIX B



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

SCALE:
1" = 60' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over RHC of Little River
Poinsett Co., Arkansas
Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Right Hand Chute of Little River (Site 5)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
E1	5.5-6.5	11	---	---	---	100	100	100	100	100	100	95	24	SM	A-2-4
E1	9.5-10	27	58	23	35	---	---	---	---	100	---	---	82	CH	A-7-6
E1	14-15	15	---	---	---	100	100	100	100	100	100	96	43	SM	A-4
E1	29-30	21	32	15	17	100	100	100	100	100	96	90	43	SC	A-6
E1	44-45	23	---	---	---	100	100	100	100	100	100	90	7	SM-SP	A-3
E1	54-55	27	---	---	---	100	100	100	100	100	100	94	6	SM-SP	A-3
E1	64-65	19	---	---	---	100	100	100	100	99	96	36	5	SM-SW	A-1-b
E1	84-85	22	---	---	---	100	100	100	100	100	99	84	7	SM-SP	A-3
E1	109-110	11	---	---	---	100	100	85	74	60	49	23	9	SM-SW	A-1-a
E2	9-10	21	---	---	---	100	100	100	100	100	100	79	26	SM	A-2-4
E2	19-20	29	---	---	---	---	---	---	---	100	---	---	26	SM	A-2-4
E2	34-35	20	---	---	---	100	100	100	100	100	100	52	4	SP	A-3
E2	54-55	27	---	---	---	100	100	100	100	100	100	47	4	SP	A-1-b
E2	79-80	18	---	---	---	100	100	100	100	95	89	23	3	SW	A-1-b
E3	0.5-1.5	10	---	---	---	---	---	---	---	100	---	---	31	SC	A-2-6
E3	2.5-3.5	40	---	---	---	100	100	100	100	100	100	97	81	CH	A-7-6
E3	14-15	21	---	---	---	100	100	100	100	100	100	81	11	SM-SP	A-2-4
E3	34-35	20	---	---	---	100	100	100	100	100	100	93	6	SM-SP	A-3
E3	49-50	17	---	---	---	100	100	100	98	97	95	49	5	SM-SP	A-1-b
E3	64-65	17	---	---	---	100	100	100	100	98	95	49	2	SP	A-1-b
E3	84-85	17	---	---	---	100	100	100	100	100	98	47	4	SP	A-1-b
E4	2.5-3.5	31	23	19	4	---	---	---	---	100	---	---	73	ML-CL	A-4

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Right Hand Chute of Little River (Site 5)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
E4	6.5-7.5	20	NON-PLASTIC			---	---	---	---	100	---	---	2	SP	A-3
E4	19-20	23	---	---	---	100	100	100	100	100	100	94	6	SM-SP	A-3
E4	54-55	18	---	---	---	100	100	100	100	95	92	47	4	SP	A-1-b
E4	109-110	23	---	---	---	100	100	100	100	100	100	95	6	SM-SP	A-3
E6	4.5-5.5	18	---	---	---	100	100	100	100	100	100	80	5	SM-SP	A-3
E6	9-10	27	---	---	---	100	100	100	100	100	100	88	4	SP	A-3
E6	29-30	18	---	---	---	100	100	100	100	99	98	41	3	SP	A-1-b
E6	49-50	16	---	---	---	100	100	100	100	97	94	48	3	SP	A-1-b
E6	69-70	17	---	---	---	100	100	100	95	93	90	45	5	SM-SP	A-1-b
E6	89-90	17	---	---	---	100	100	100	98	95	92	42	5	SM-SP	A-1-b
E7	0.5-1.5	17	27	16	11	---	---	---	---	99	---	---	42	SC	A-6
E7	4.5-5.5	23	23	20	3	100	100	100	100	100	100	97	51	ML	A-4
E7	9-10	23	---	---	---	100	100	100	100	100	100	96	10	SM-SP	A-3
E7	14-15	24	---	---	---	---	---	---	---	100	---	---	20	SM	A-2-4
E7	29-30	25	---	---	---	100	100	100	100	100	100	95	3	SP	A-3
E7	49-50	19	---	---	---	100	100	100	100	97	95	64	4	SP	A-3
E7	64-65	19	---	---	---	100	100	100	99	98	97	32	4	SW	A-1-b
E7	84-85	18	---	---	---	100	100	100	99	98	97	28	6	SM-SW	A-1-b
E7	99-100	19	---	---	---	100	100	100	98	95	94	51	7	SM-SP	A-3

SUMMARY of CLASSIFICATION TEST RESULTS

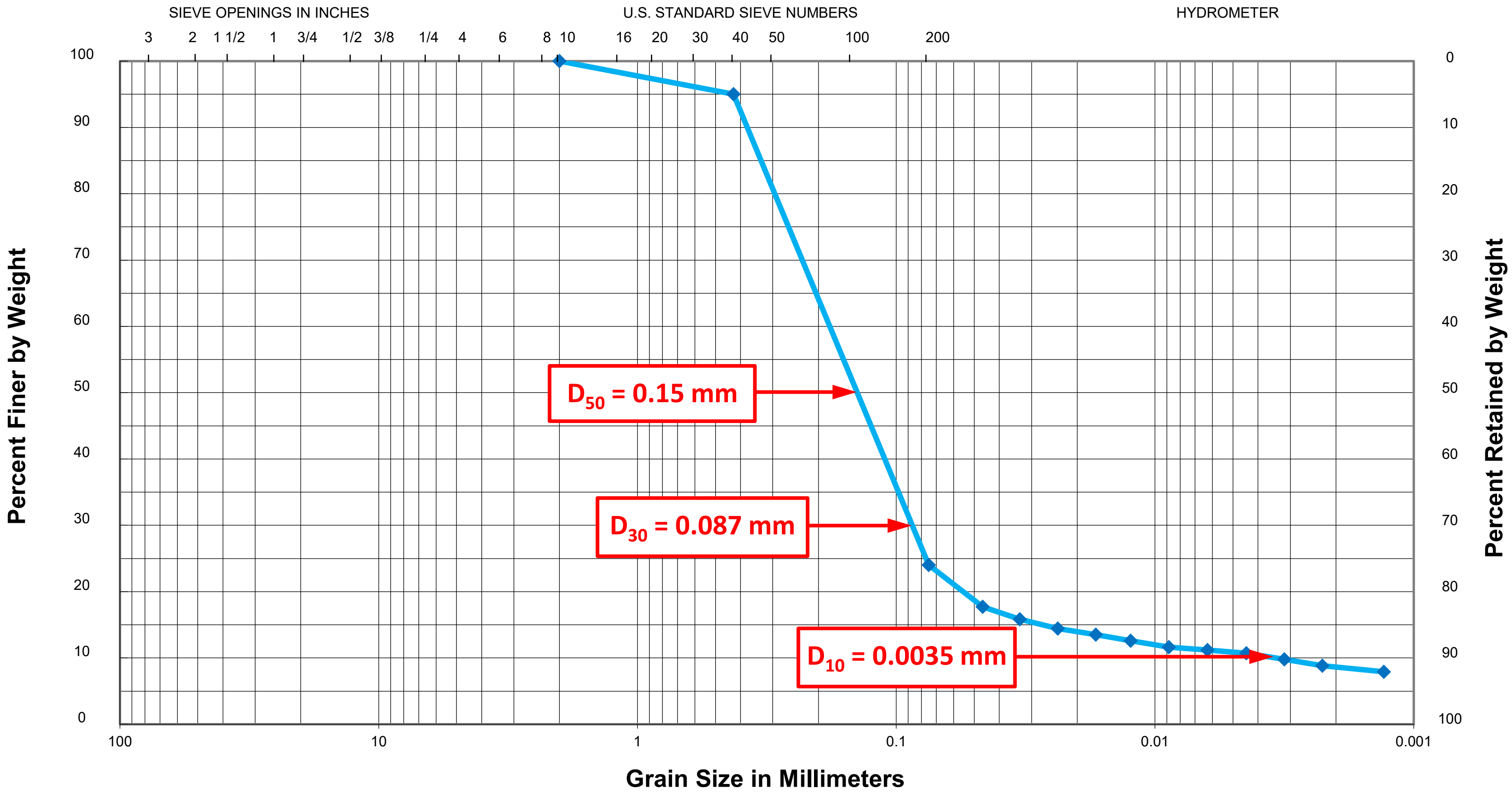
PROJECT: 101124 Hwy. 135 over Right Hand Chute of Little River (Site 5)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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**		
E8	6.5-7.5	23	---	---	---	---	---	---	---	100	---	---	35	SC	A-2-6
E8	9-10	22	NON-PLASTIC			---	---	---	---	100	---	---	63	ML	A-4
E8	14-15	27	---	---	---	100	100	100	100	100	100	96	20	SM	A-2-4
E8	24-25	20	---	---	---	---	---	---	---	100	---	---	19	SM	A-2-4
E8	29-30	25	---	---	---	100	100	100	100	100	100	91	5	SM-SP	A-3
E8	59-60	24	---	---	---	100	100	100	100	100	100	78	3	SP	A-3
E8	74-75	21	---	---	---	100	100	100	100	100	100	71	5	SM-SP	A-3
E8	84-85	15	---	---	---	100	100	92	91	89	84	30	6	SM-SW	A-1-b
E9	1.5-2.5	15	---	---	---	100	100	100	100	100	100	92	50	SC	A-6
E9	5.5-6.5	11	---	---	---	100	100	100	100	100	100	96	24	SM	A-2-4
E9	19-20	21	28	17	11	---	---	---	---	99	---	---	64	CL	A-6
E9	29-30	23	---	---	---	100	100	100	100	100	100	93	10	SM-SP	A-3
E9	39-40	24	---	---	---	100	100	100	100	100	100	85	7	SM-SP	A-3
E9	49-50	17	---	---	---	100	100	100	100	100	100	35	5	SM-SW	A-1-b
E9	69-70	28	---	---	---	100	100	100	97	95	93	56	11	SM-SP	A-2-4
E9	84-85	31	---	---	---	100	100	100	100	100	100	95	6	SM-SP	A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.15 mm

D_{30} = 0.087 mm

D_{10} = 0.0035 mm

<table>
<tr><th colspan="2">GRAVEL</th><th colspan="3">SAND</th><th rowspan="2">SILT</th><th rowspan="2">CLAY</th></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

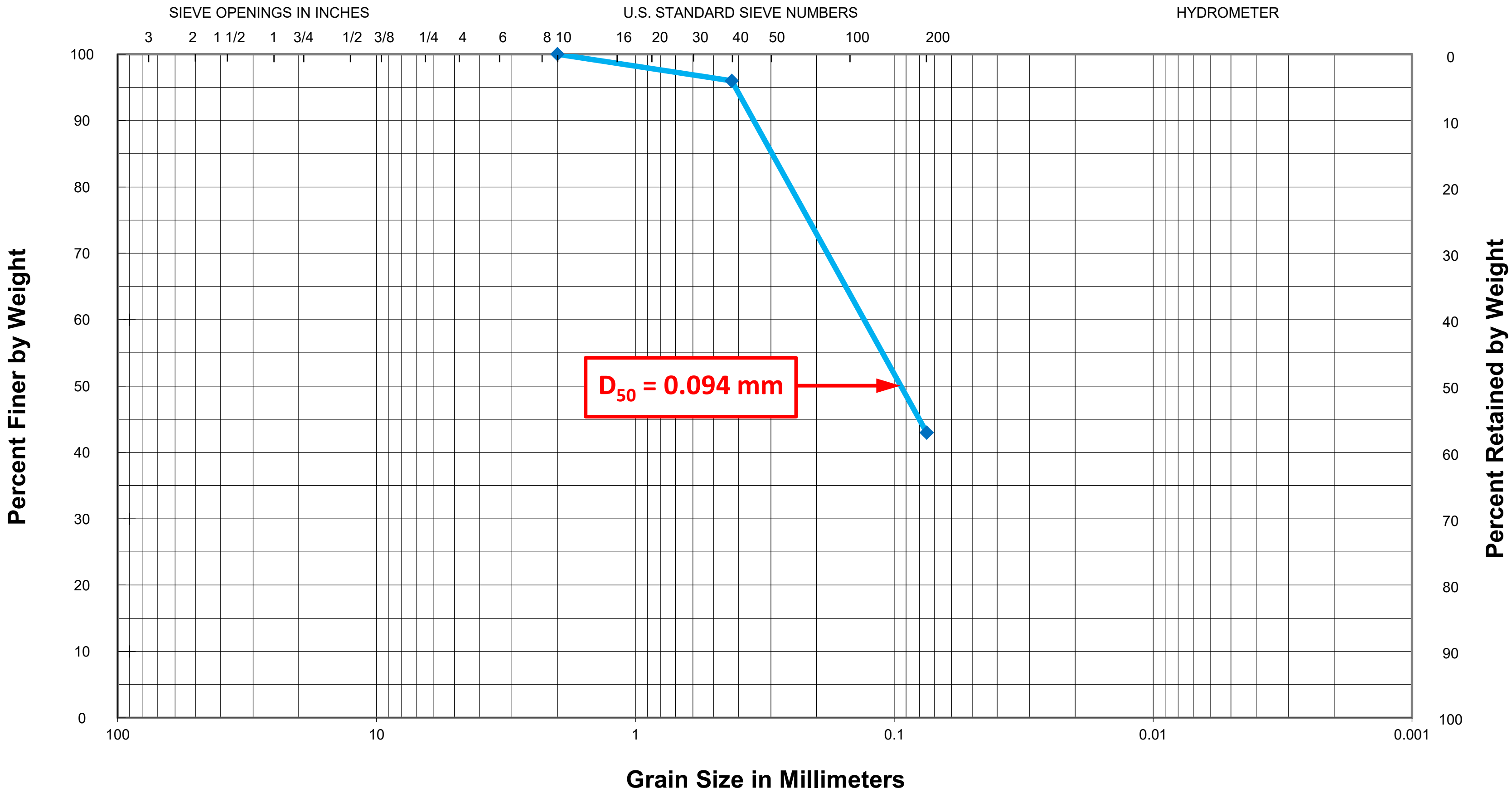
Sample: Boring E1, 5.5-6.5 ft
Description: Reddish brown silty fine SAND w/ clay pockets (fill)

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

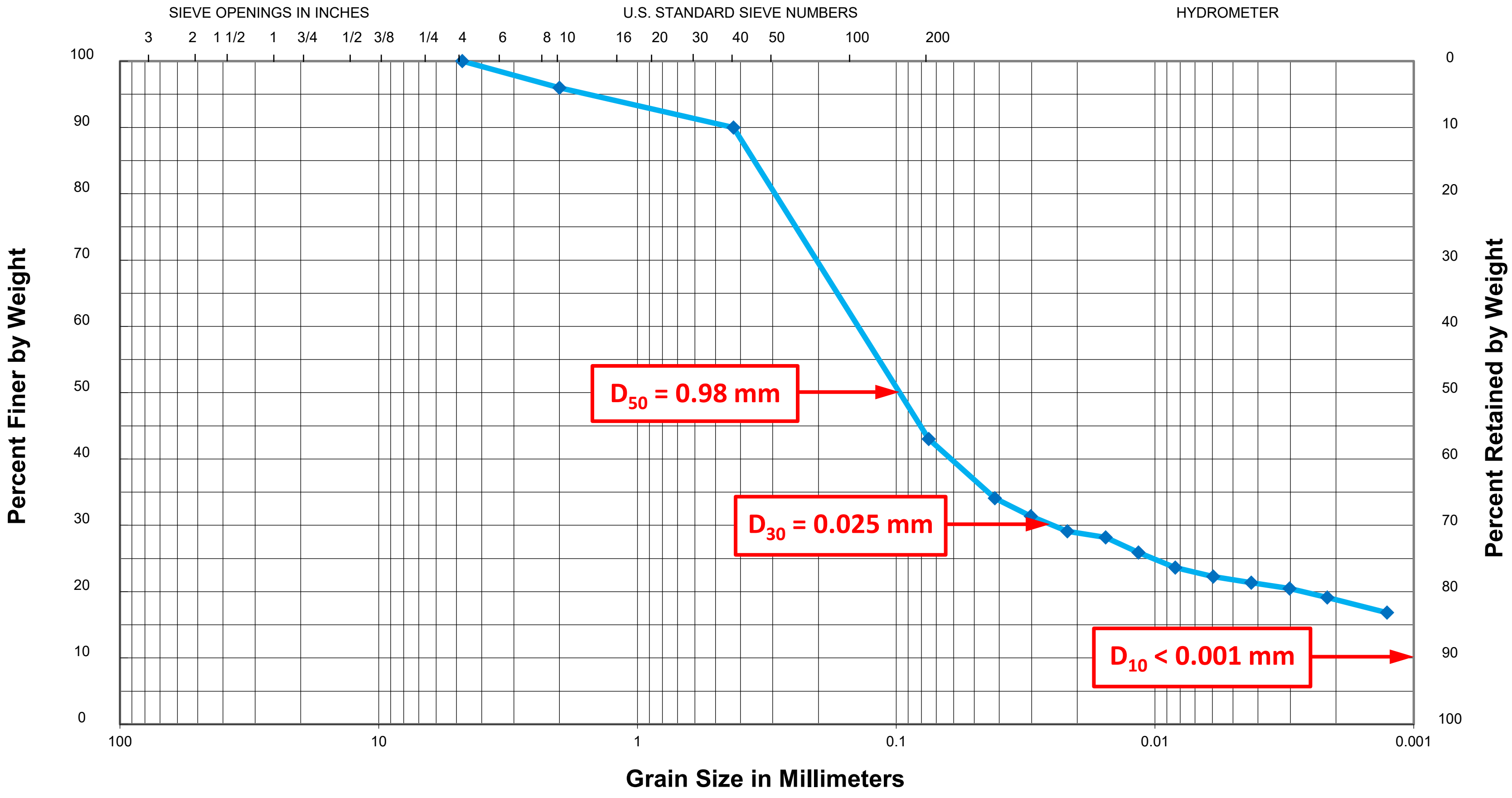
Sample: Boring E1, 14-15 ft
Description: Brown and gray silty fine SAND w/ clay pockets

USCS Classification = SM
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.98 mm

D_{30} = 0.025 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

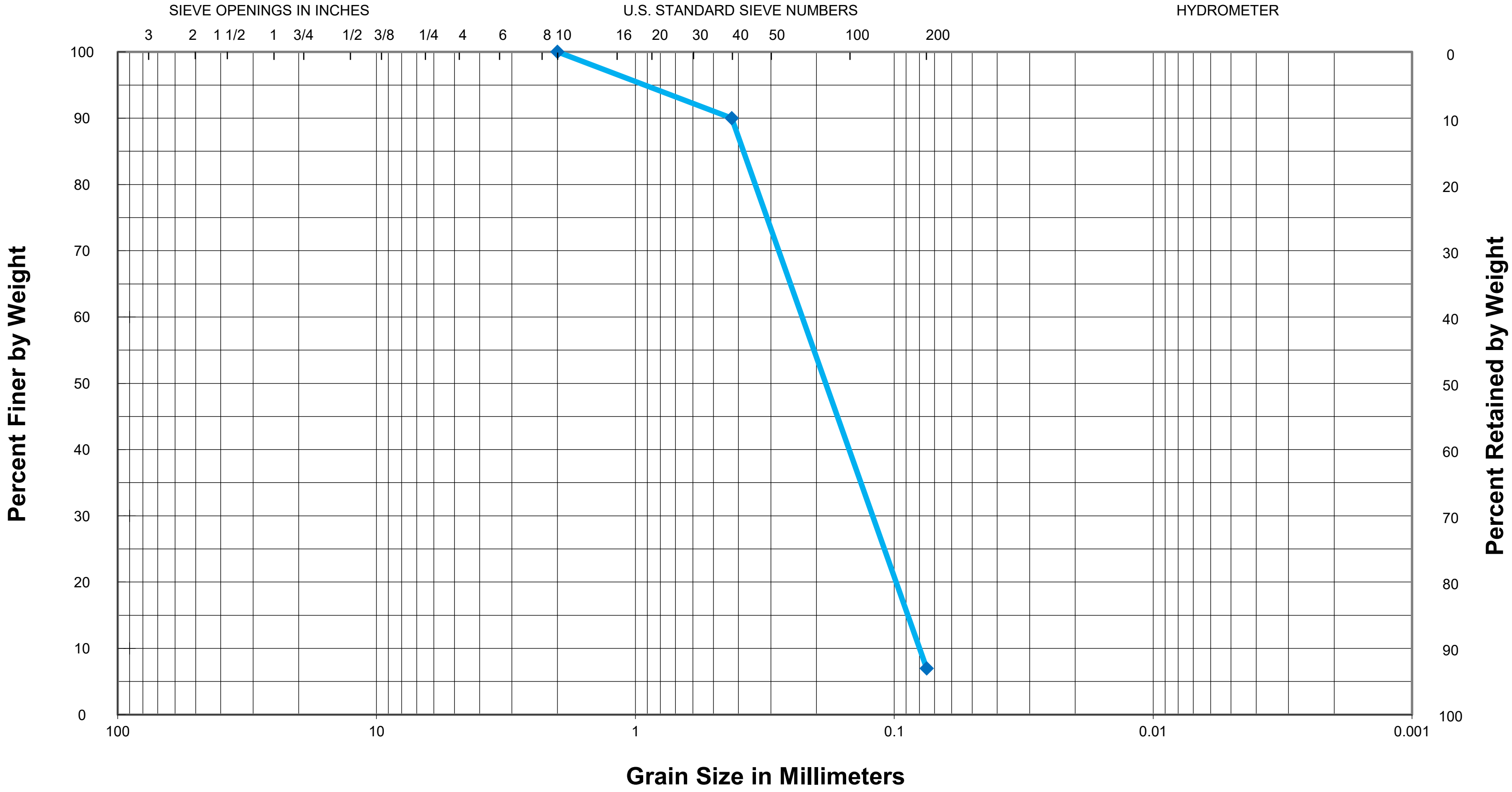
Sample: Boring E1, 29-30 ft; LL = 32, PL = 15, PI = 17
Description: Brown and gray silty fine SAND w/ sandy clay pockets

USCS Classification = SC
AASHTO Classification = A-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

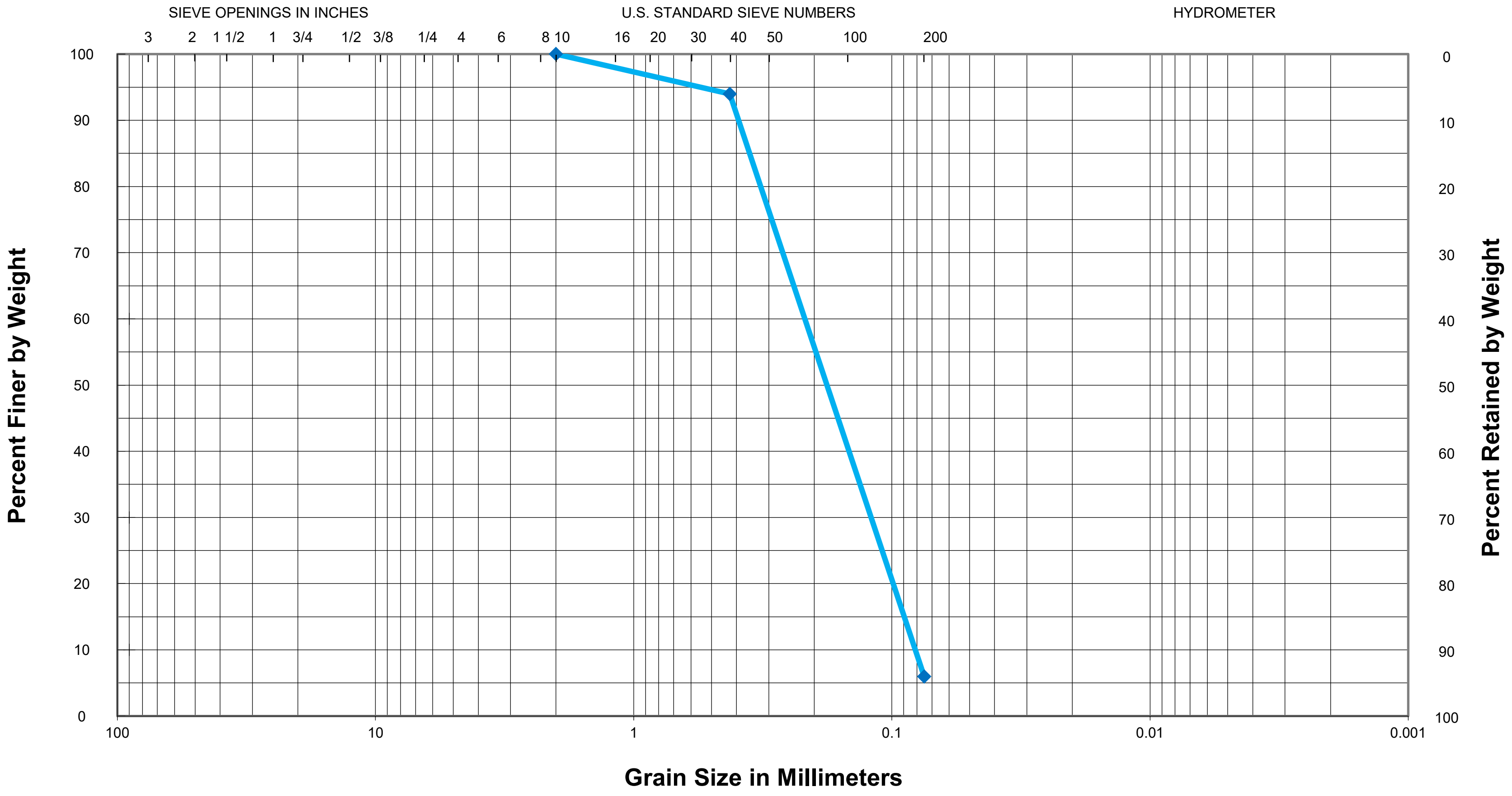
Sample: Boring E1, 44-45 ft
Description: Gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

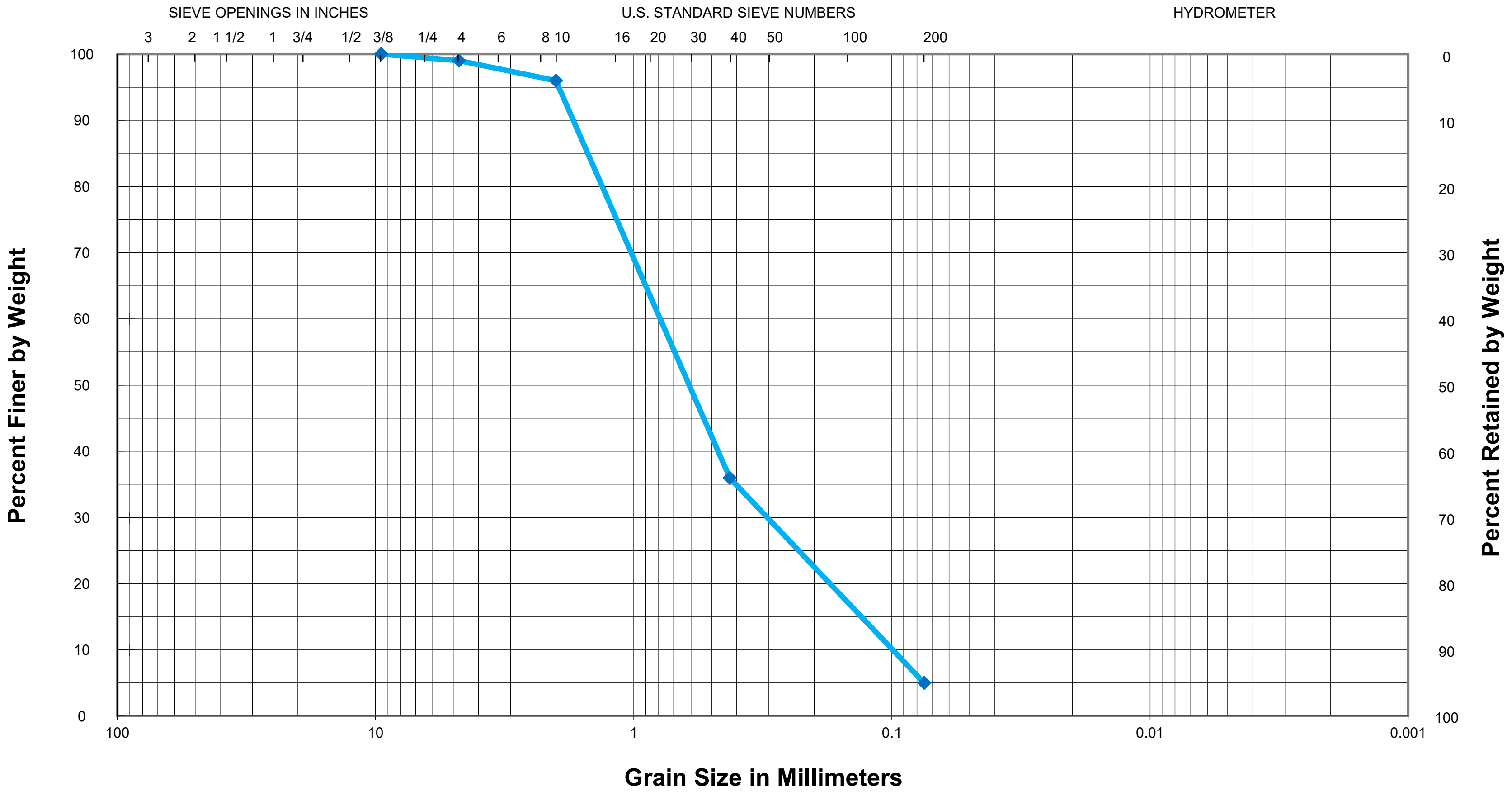
Sample: Boring E1, 54-55 ft
Description: Gray and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

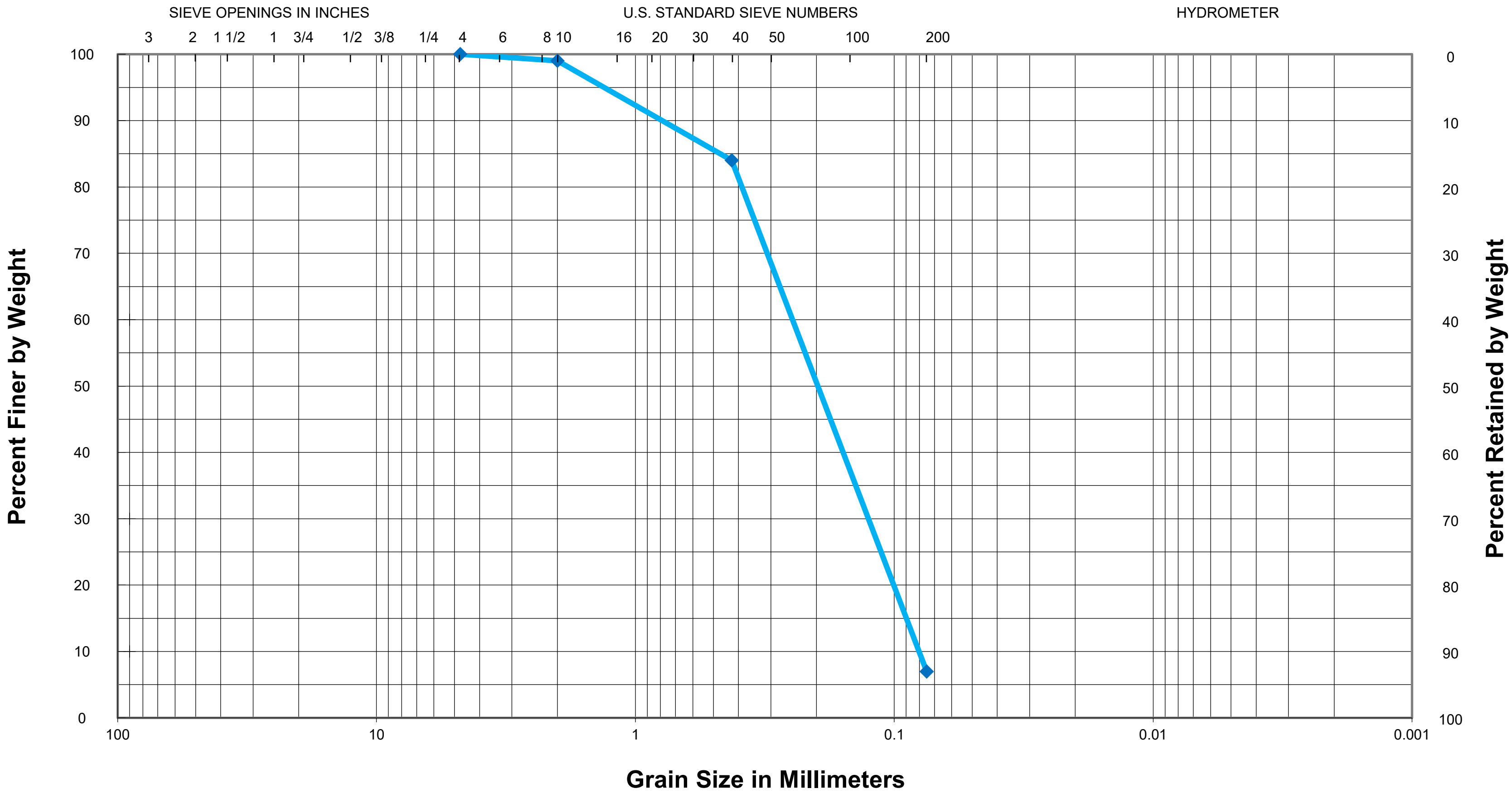
Sample: Boring E1, 64-65 ft
Description: Brownish gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

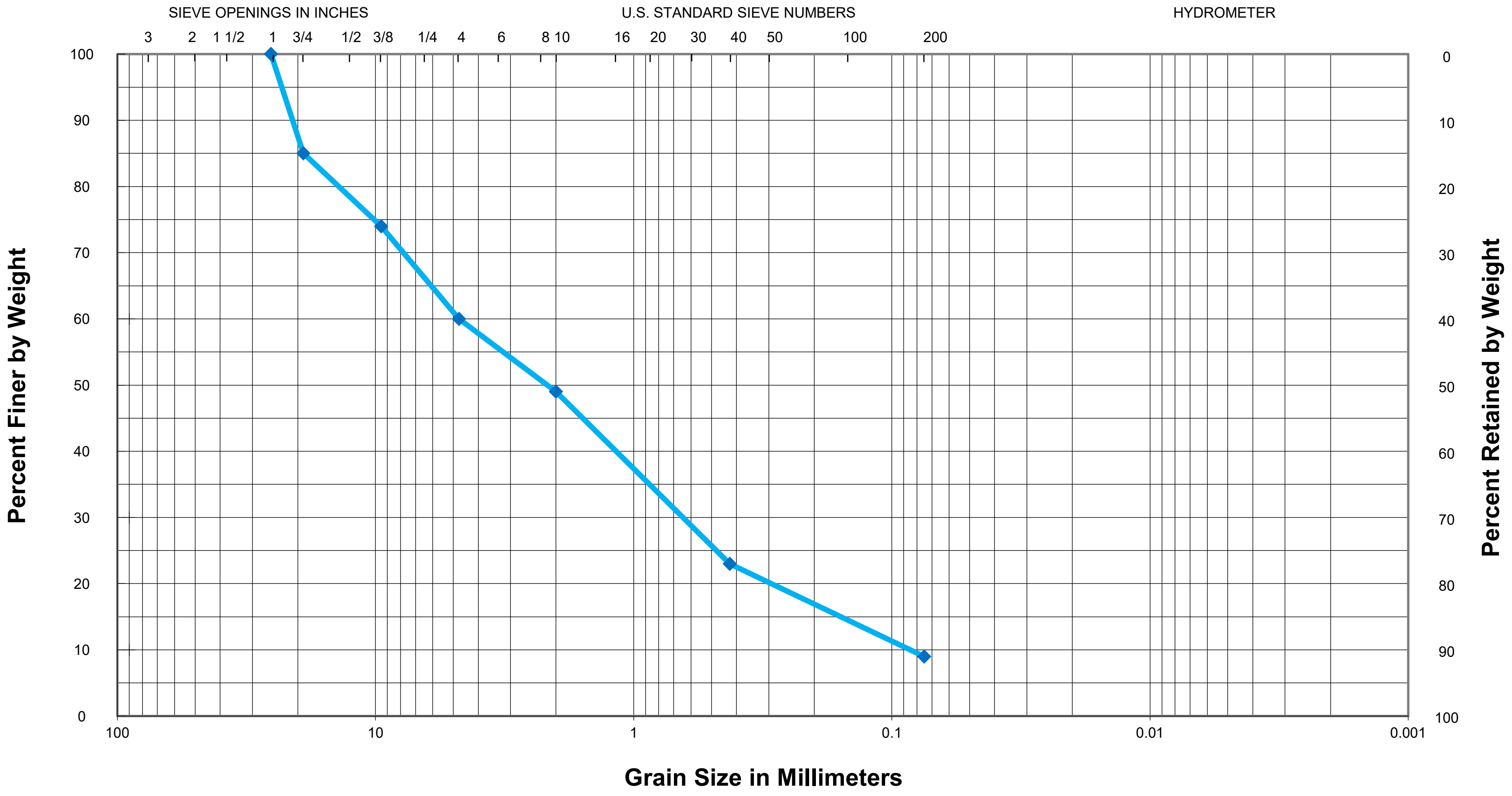
Sample: Boring E1, 84-85 ft
Description: Brownish gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

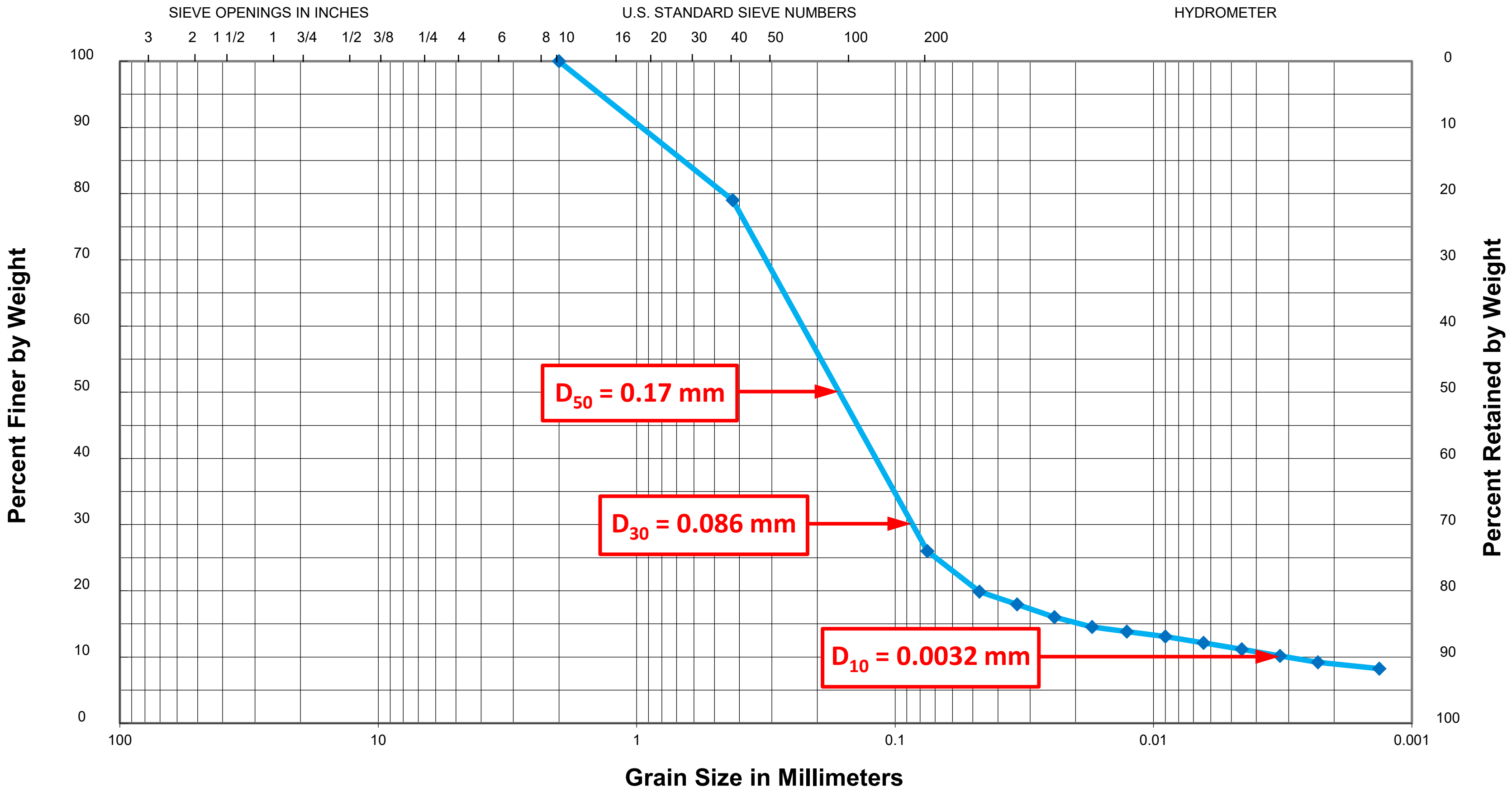
Sample: Boring E1, 109-110 ft
Description: Brown fine to coarse SAND w/ fine to coarse gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.17 mm

D_{30} = 0.086 mm

D_{10} = 0.0032 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

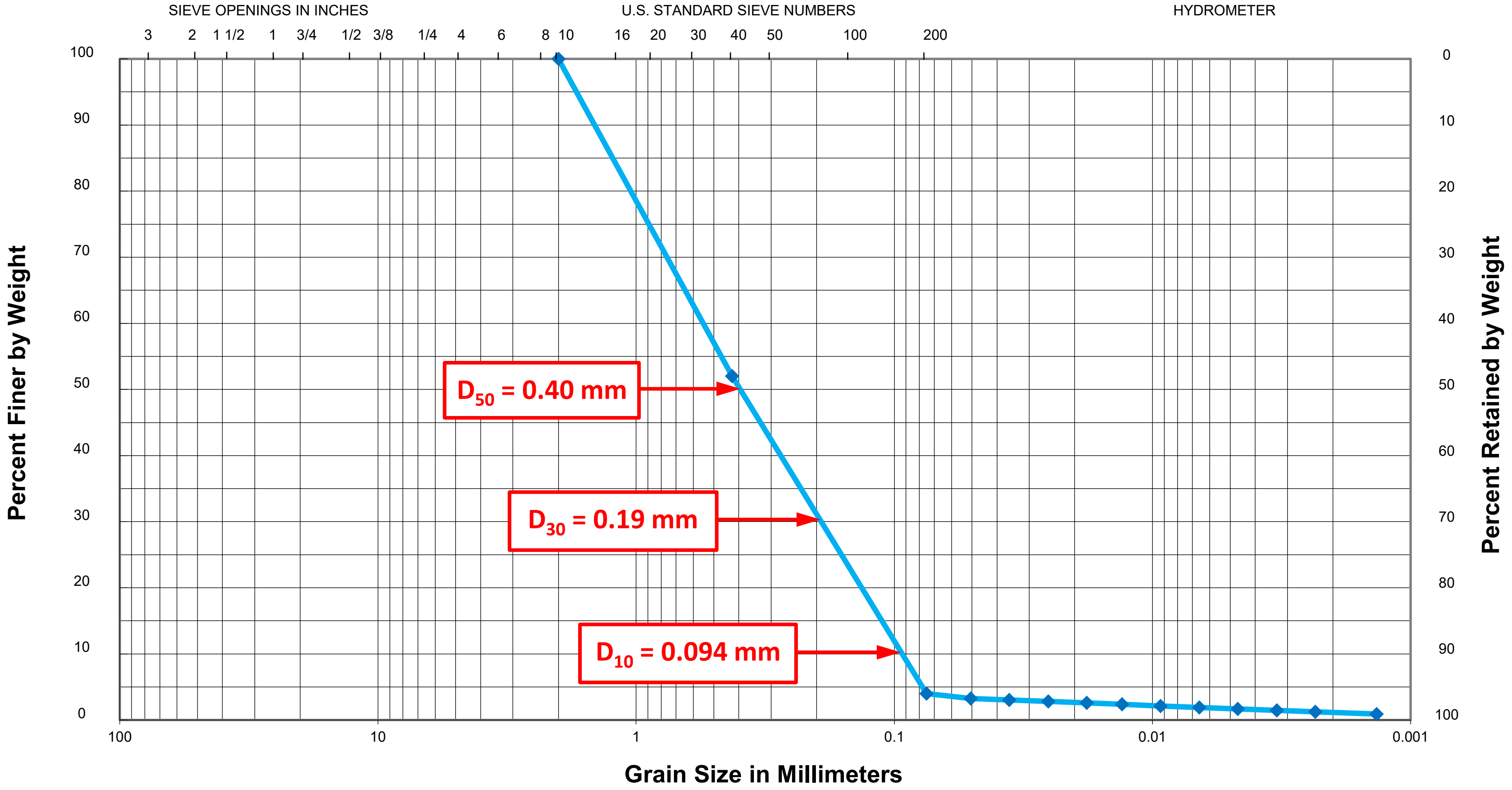
Sample: Boring E2, 9-10 ft
Description: Brownish gray silty fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

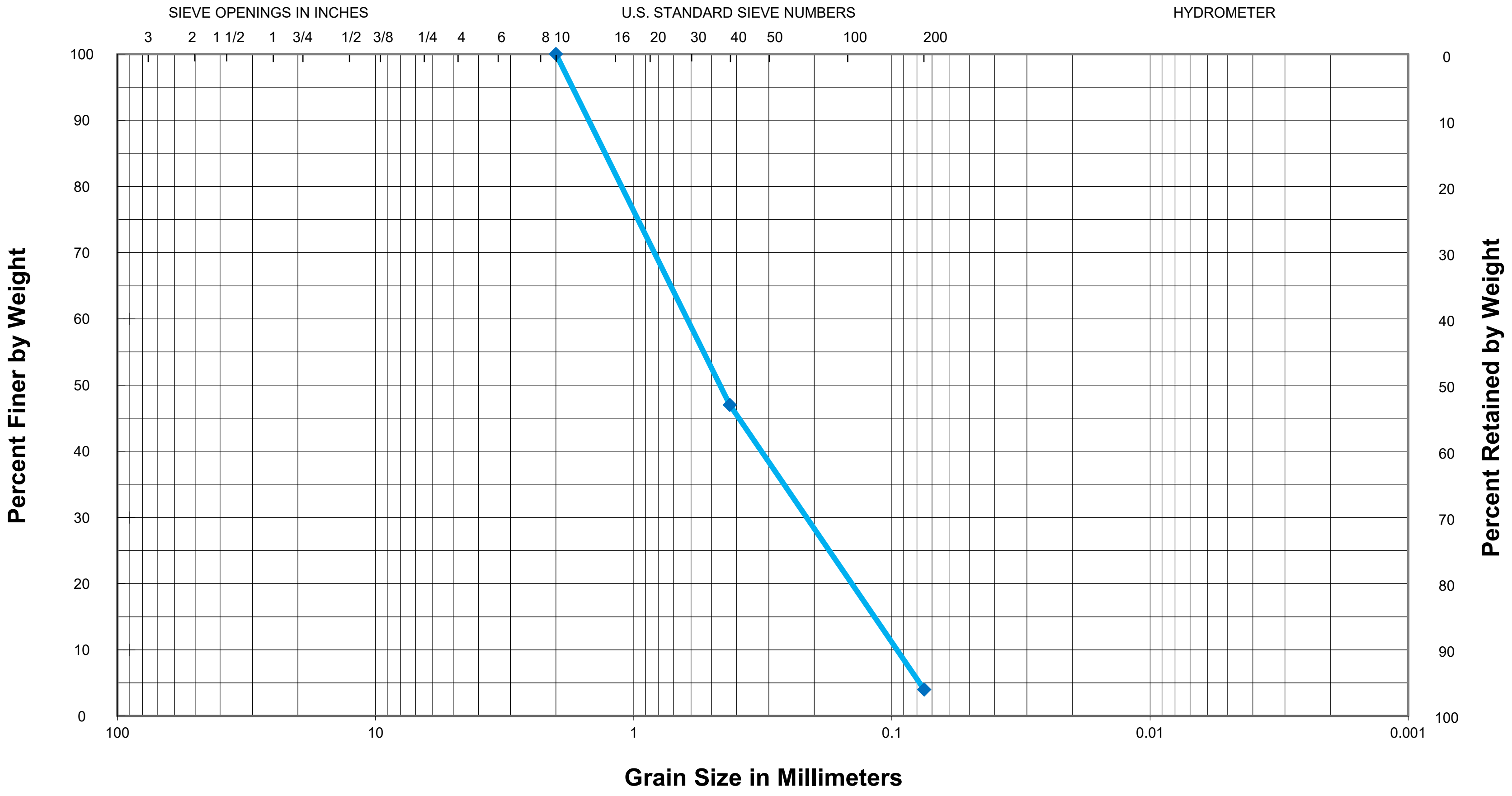
Sample: Boring E2, 34-35 ft
Description: Brownish gray fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

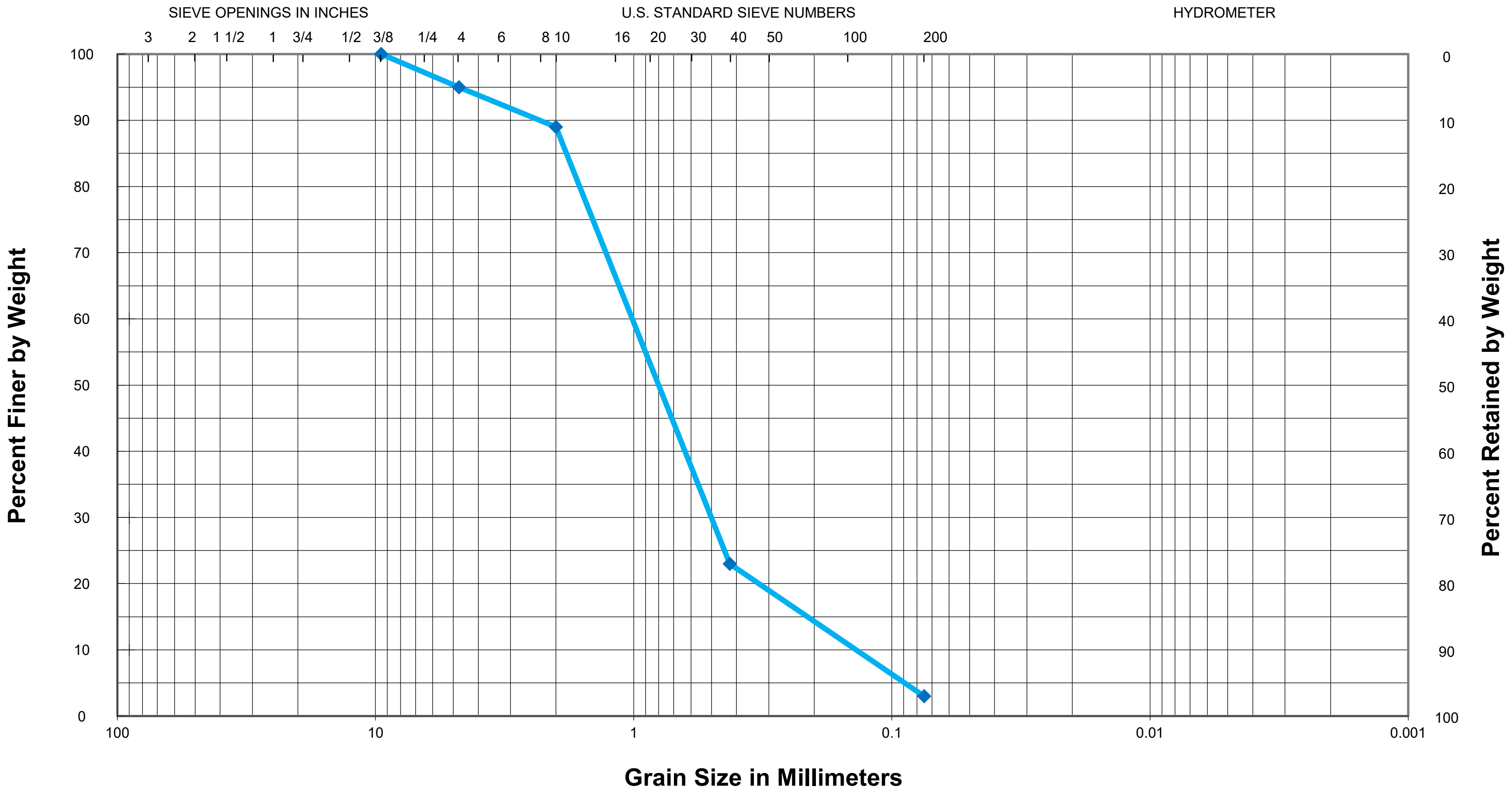
Sample: Boring E2, 54-55 ft
Description: Brownish gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

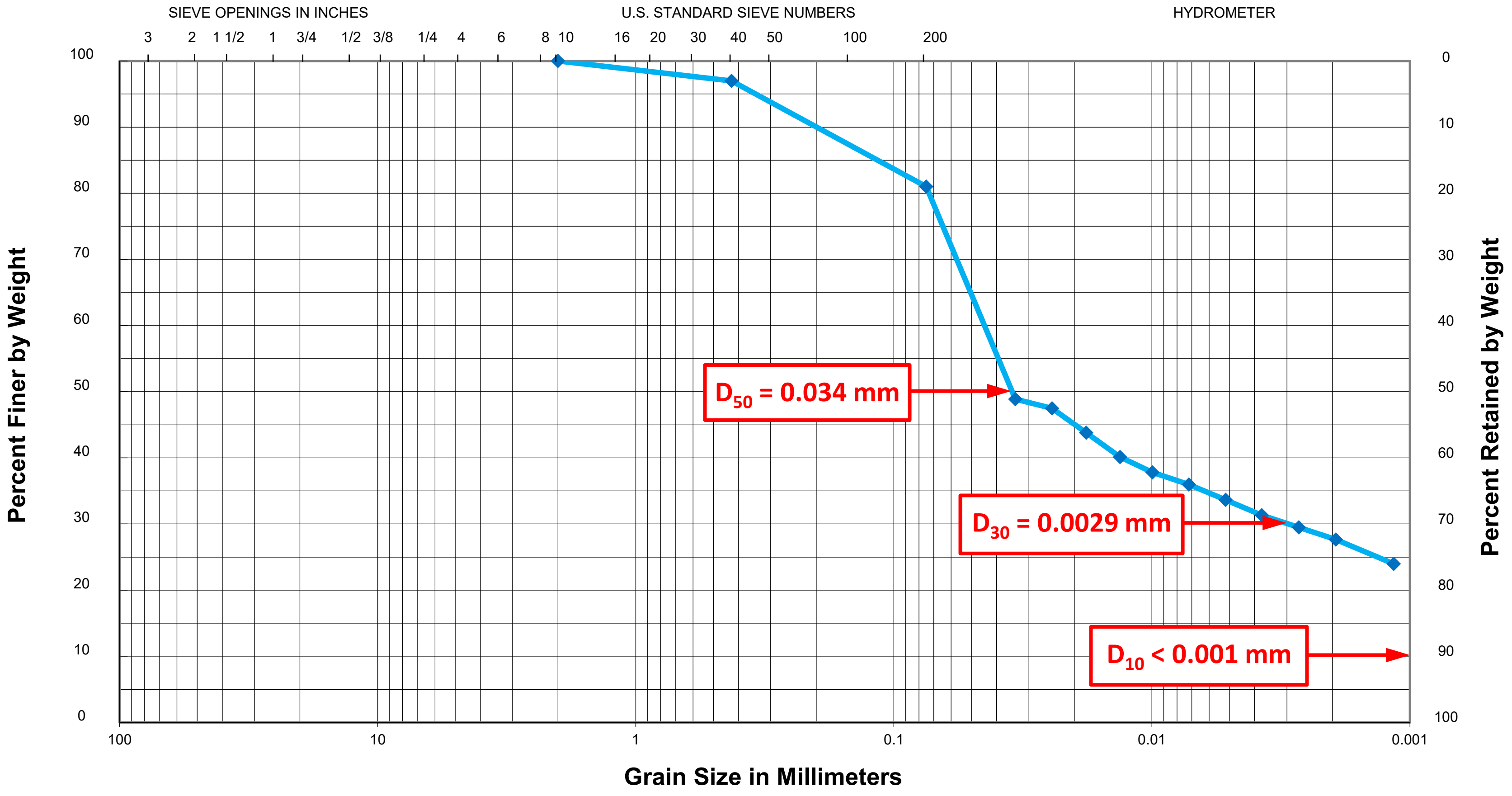
Sample: Boring E2, 79-80 ft
Description: Brownish gray fine to medium SAND w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.034 mm

D_{30} = 0.0029 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

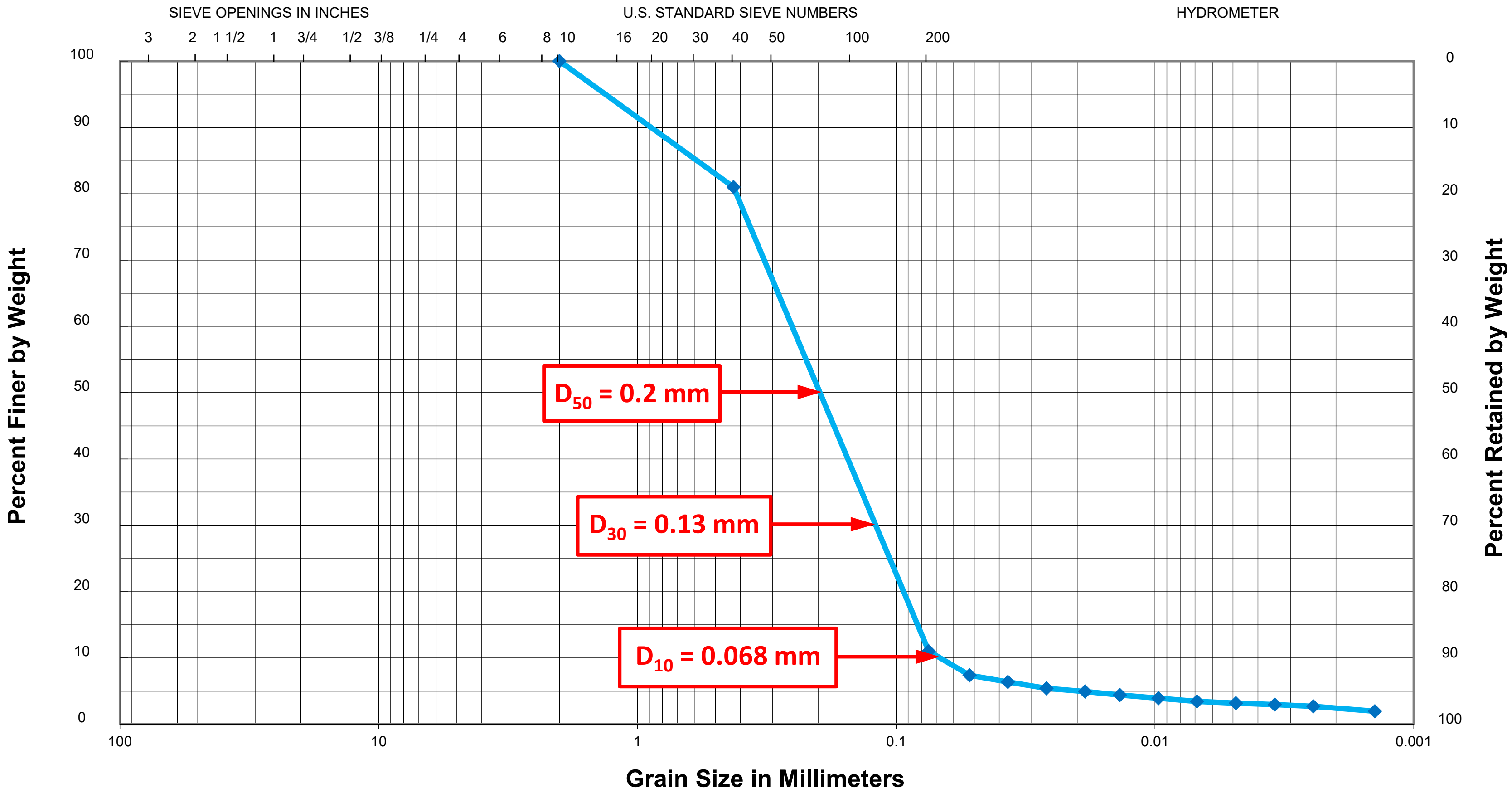
Sample: Boring E3, 2.5-3.5 ft
Description: Dark brown CLAY, slightly sandy

USCS Classification = CH
AASHTO Classification = A-7-6

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

23-031

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring E3, 14-15 ft
Description: Gray and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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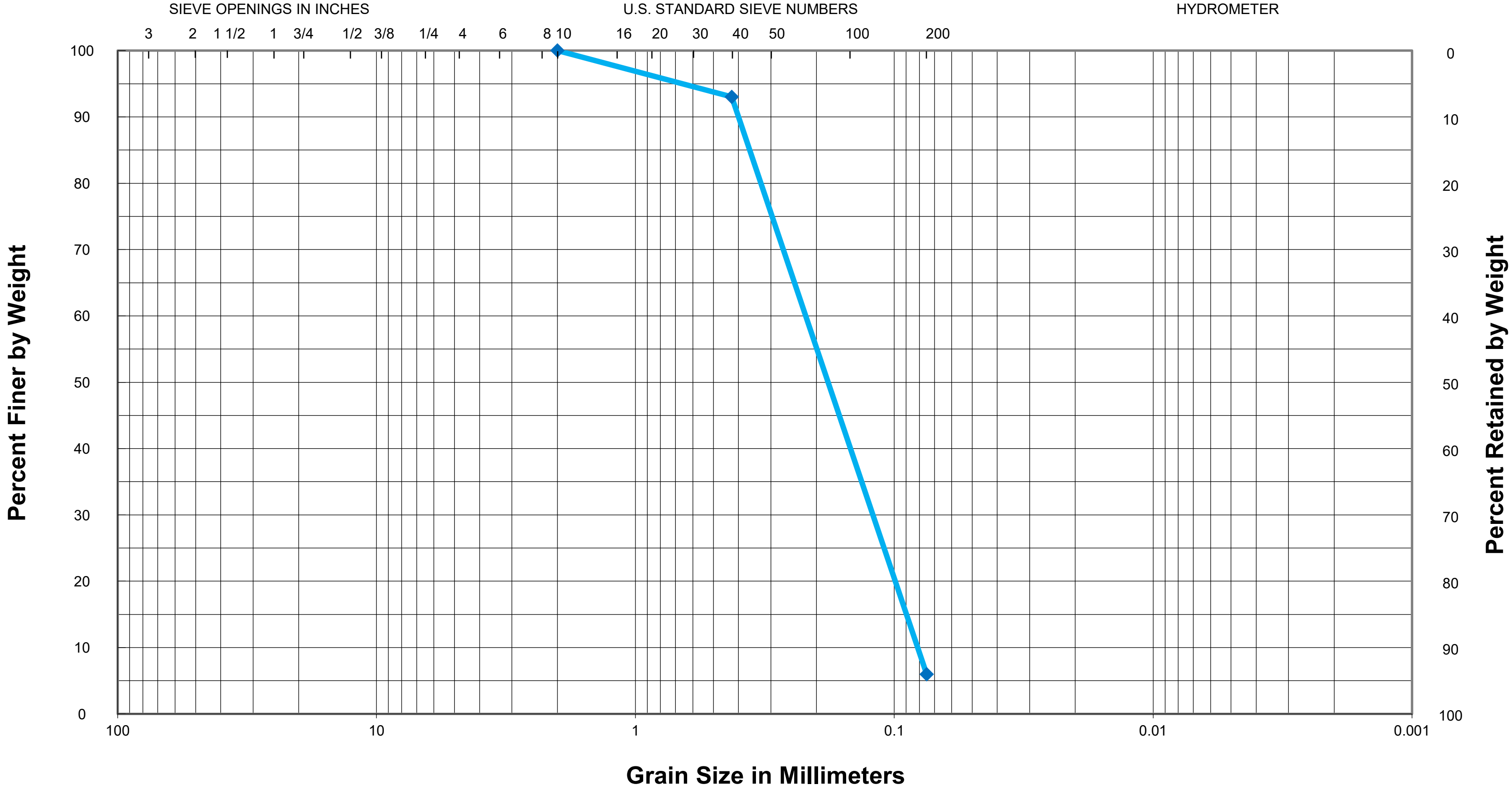
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E3, 34-35 ft
Description: Gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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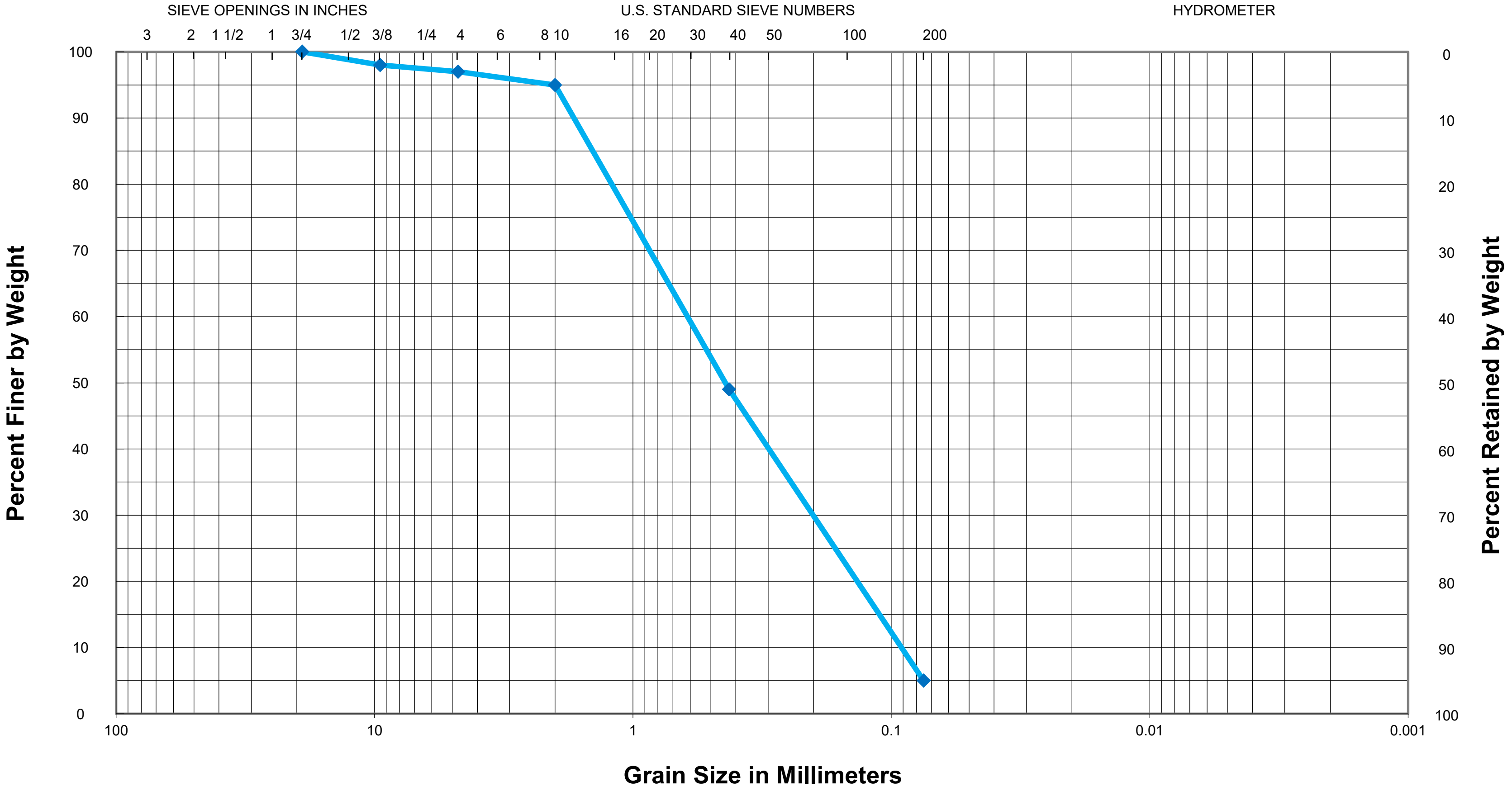
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E3, 49-50 ft
Description: Gray and brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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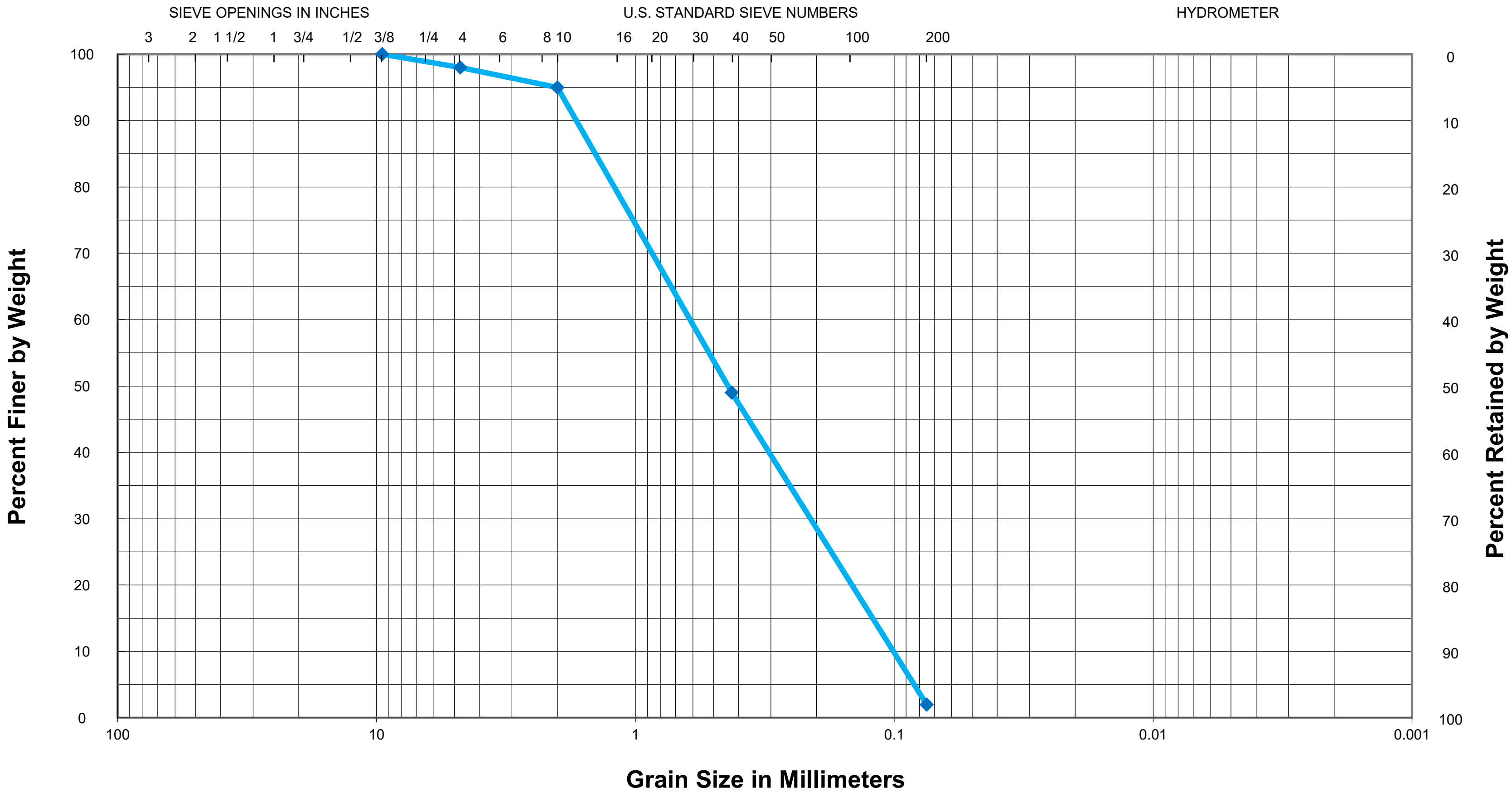
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E3, 64-65 ft
Description: Gray and brown fine to medium SAND

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

23-031

GRAIN SIZE CURVE

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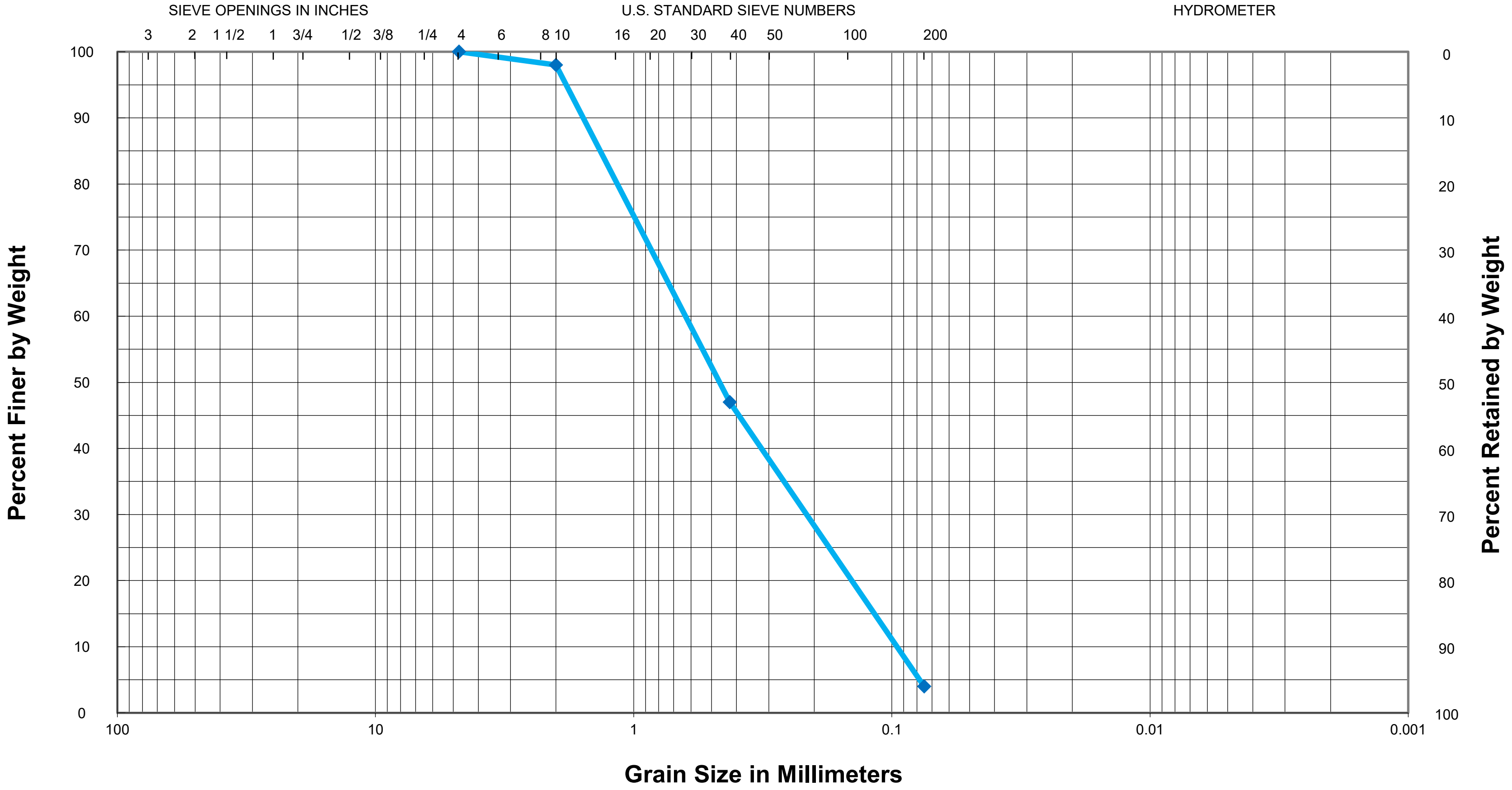
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E3, 84-85 ft
Description: Gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

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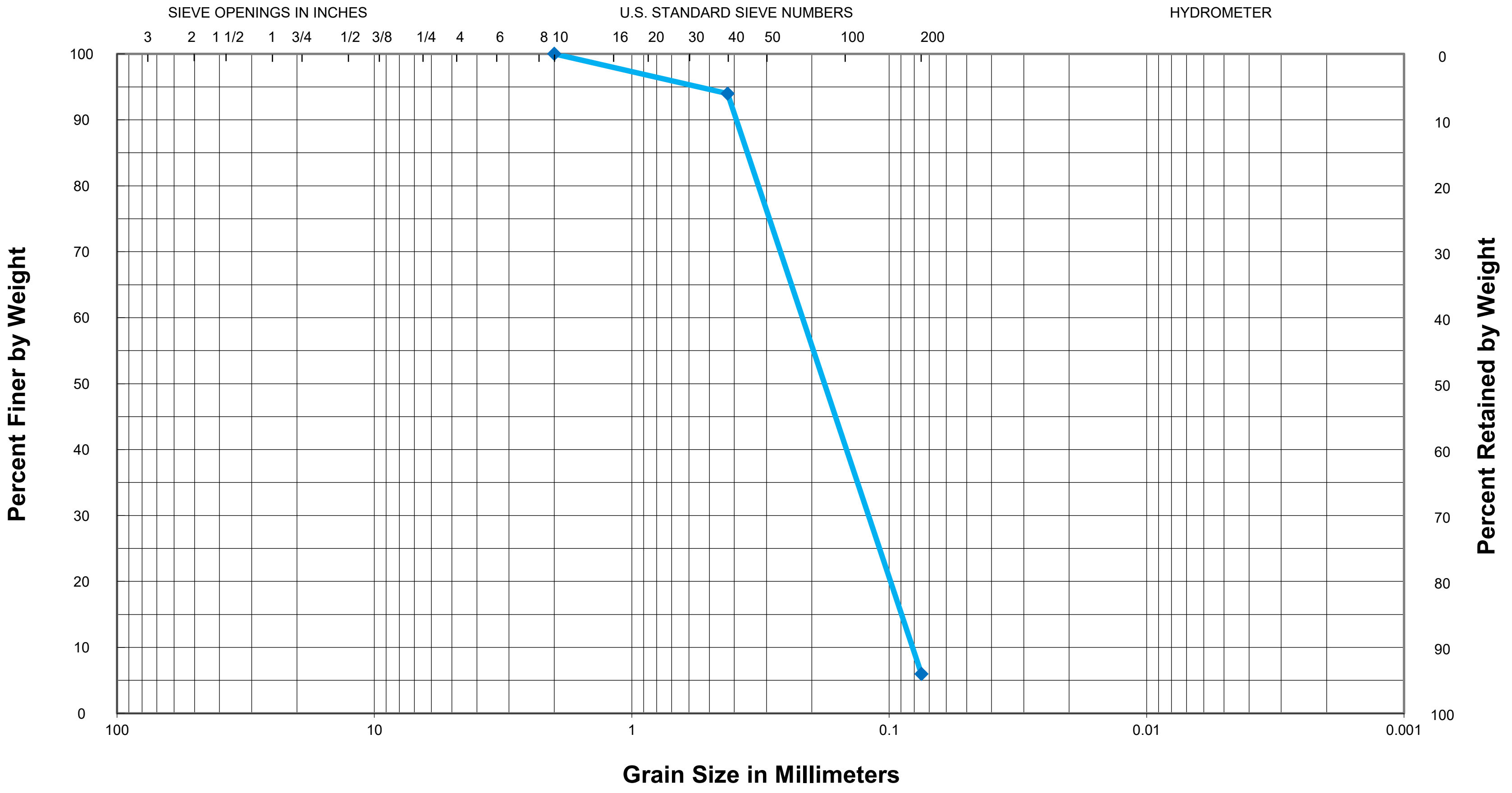
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E4, 19-20 ft
Description: Brownish gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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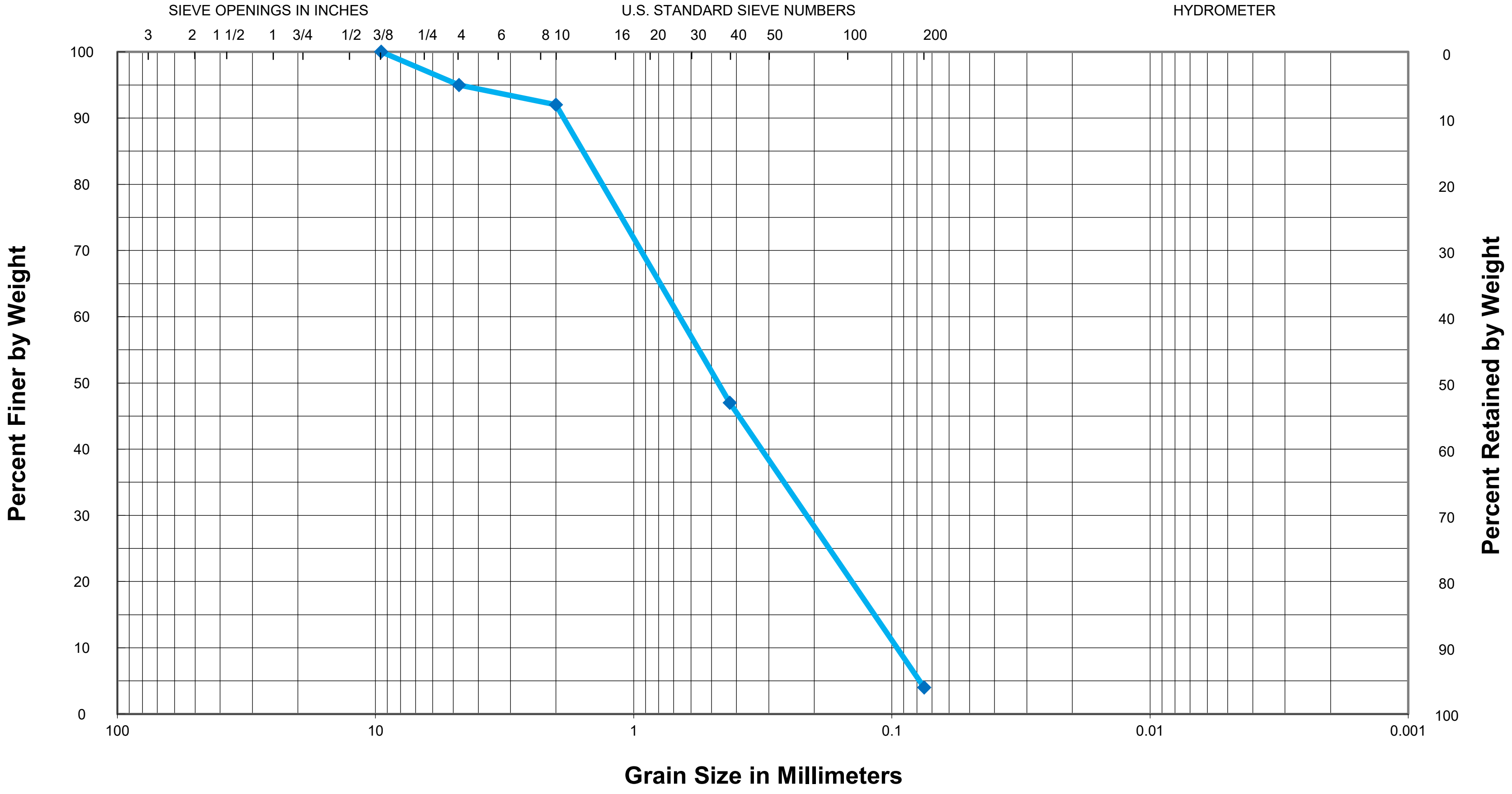
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E4, 54-55 ft
Description: Brownish gray fine to medium SAND w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

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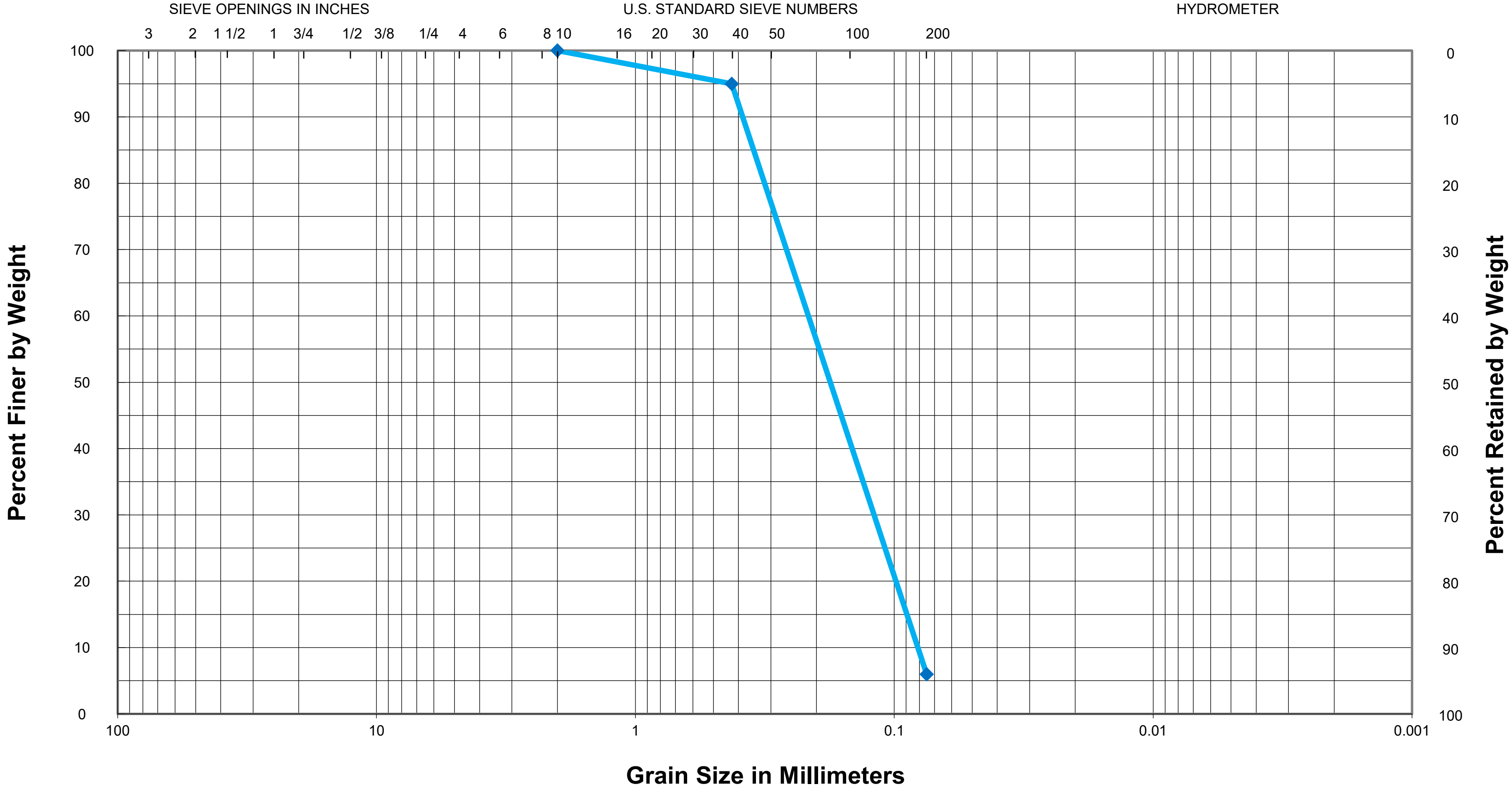
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

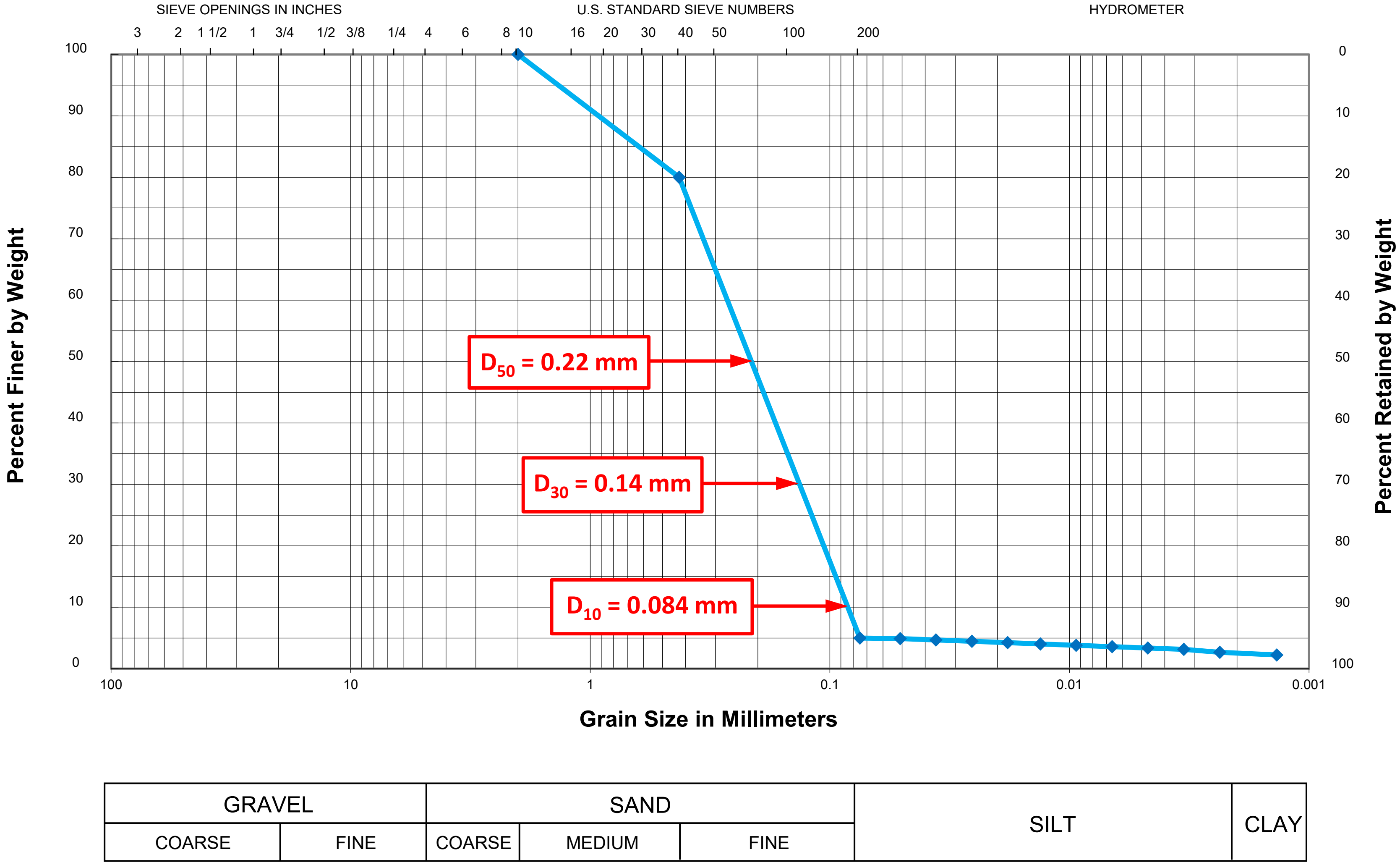
Sample: Boring E4, 109-110 ft
Description: Brownish gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.22 mm

D_{30} = 0.14 mm

D_{10} = 0.084 mm

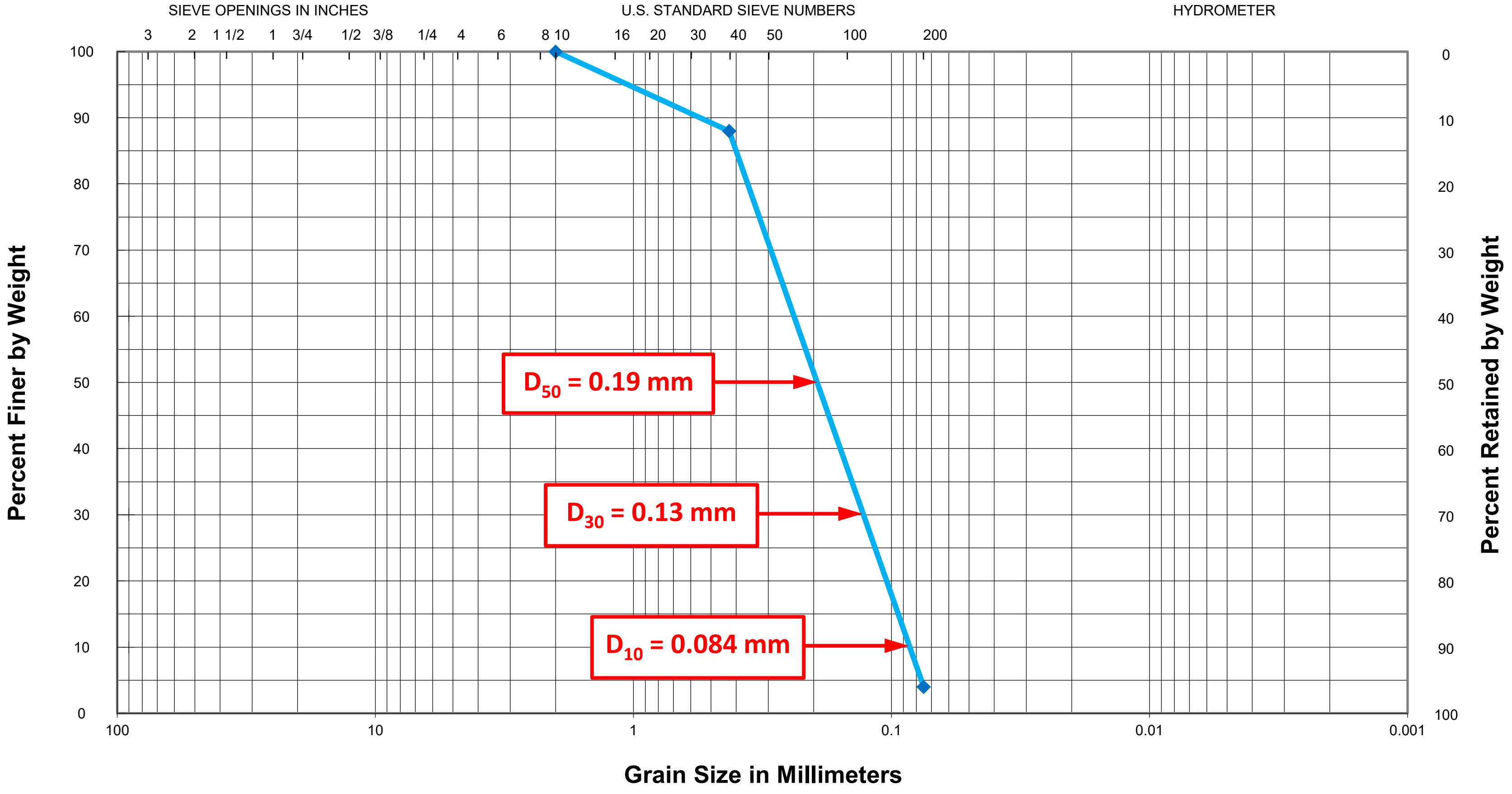
Sample: Boring E6, 4.5-5.5 ft
Description: Brown and dark gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E6, 9-10 ft
Description: Brown and dark gray fine SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

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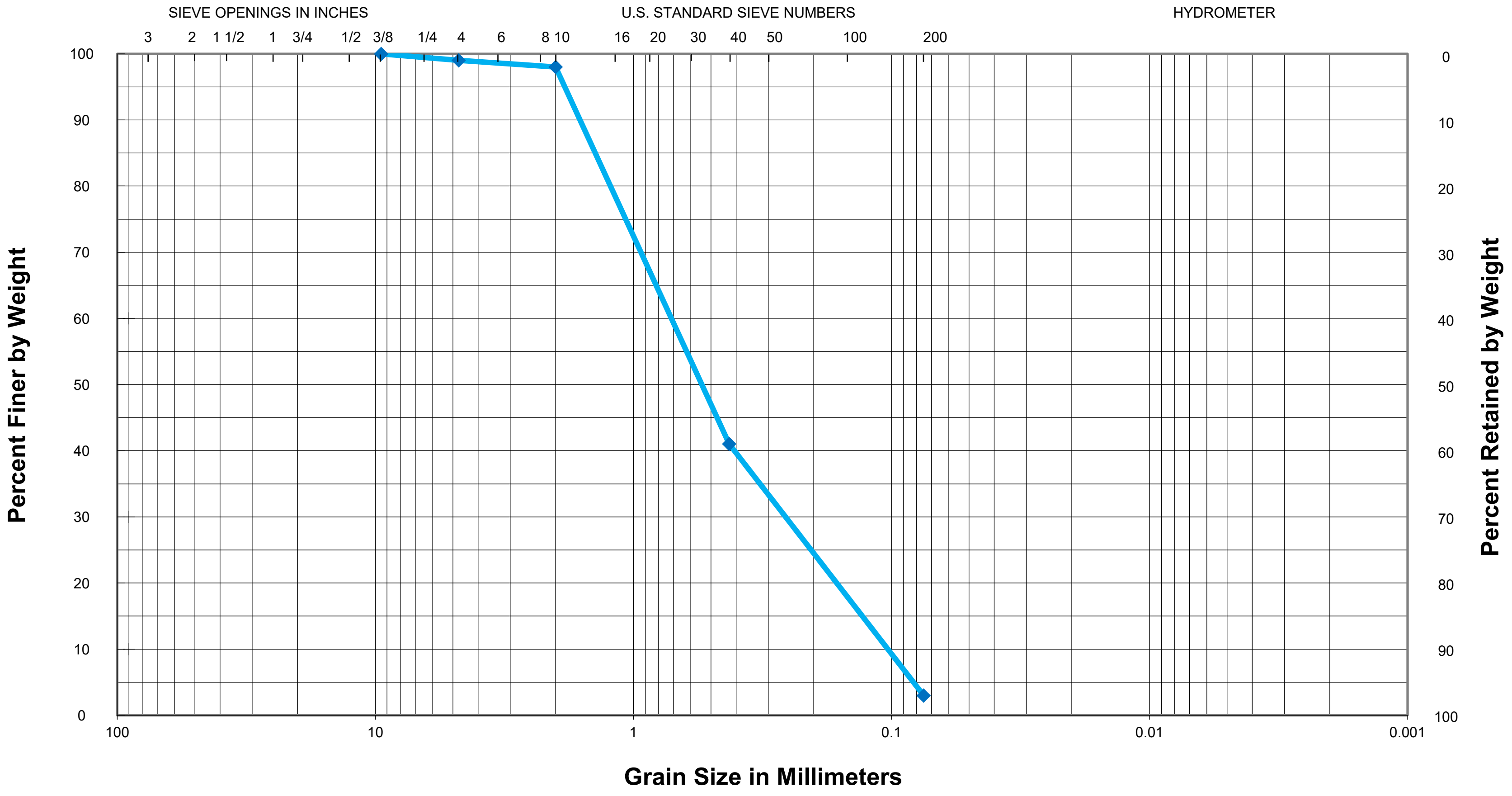
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E6, 29-30 ft
Description: Gray and brown fine to medium SAND

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

23-031

GRAIN SIZE CURVE

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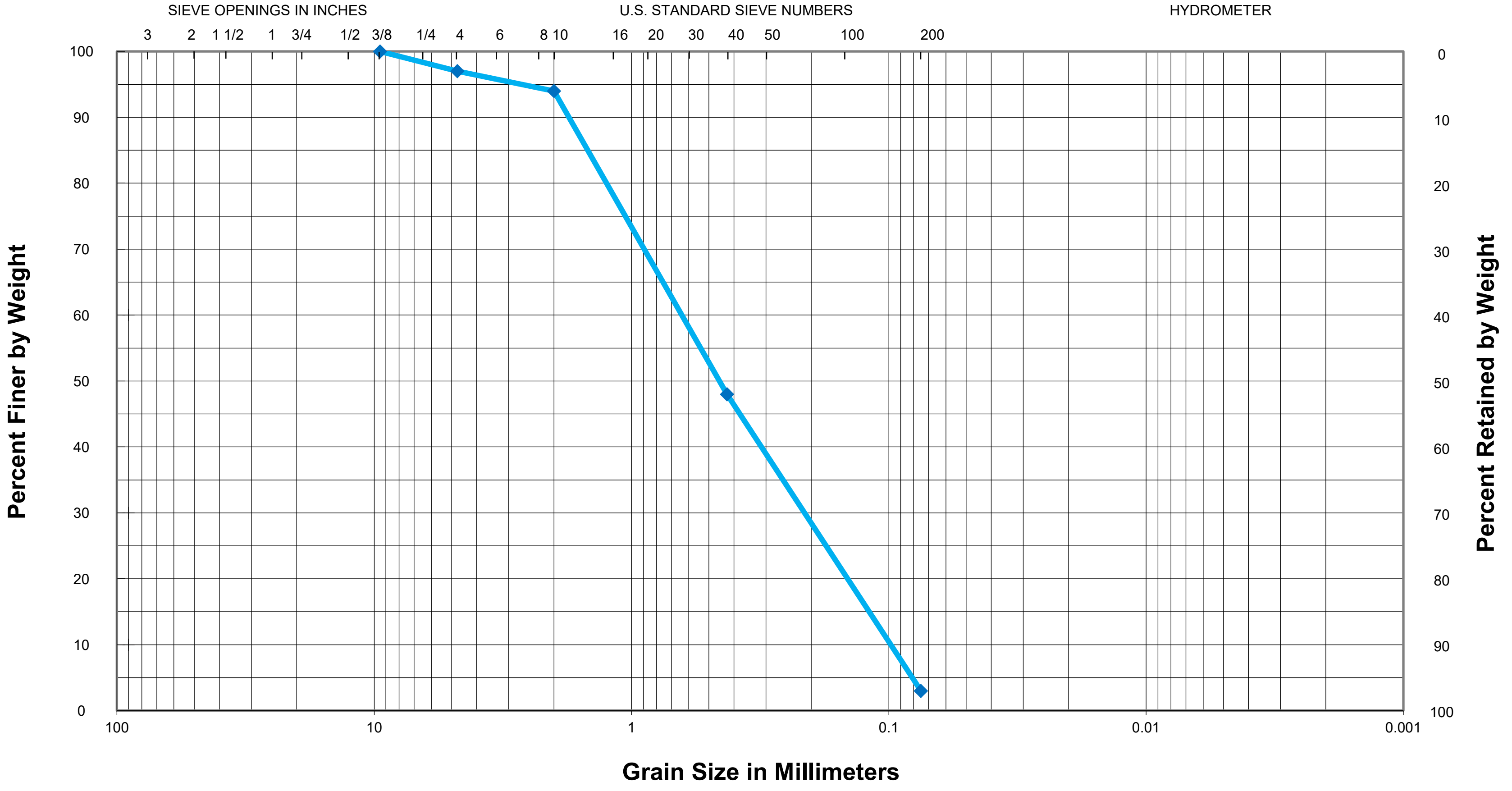
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

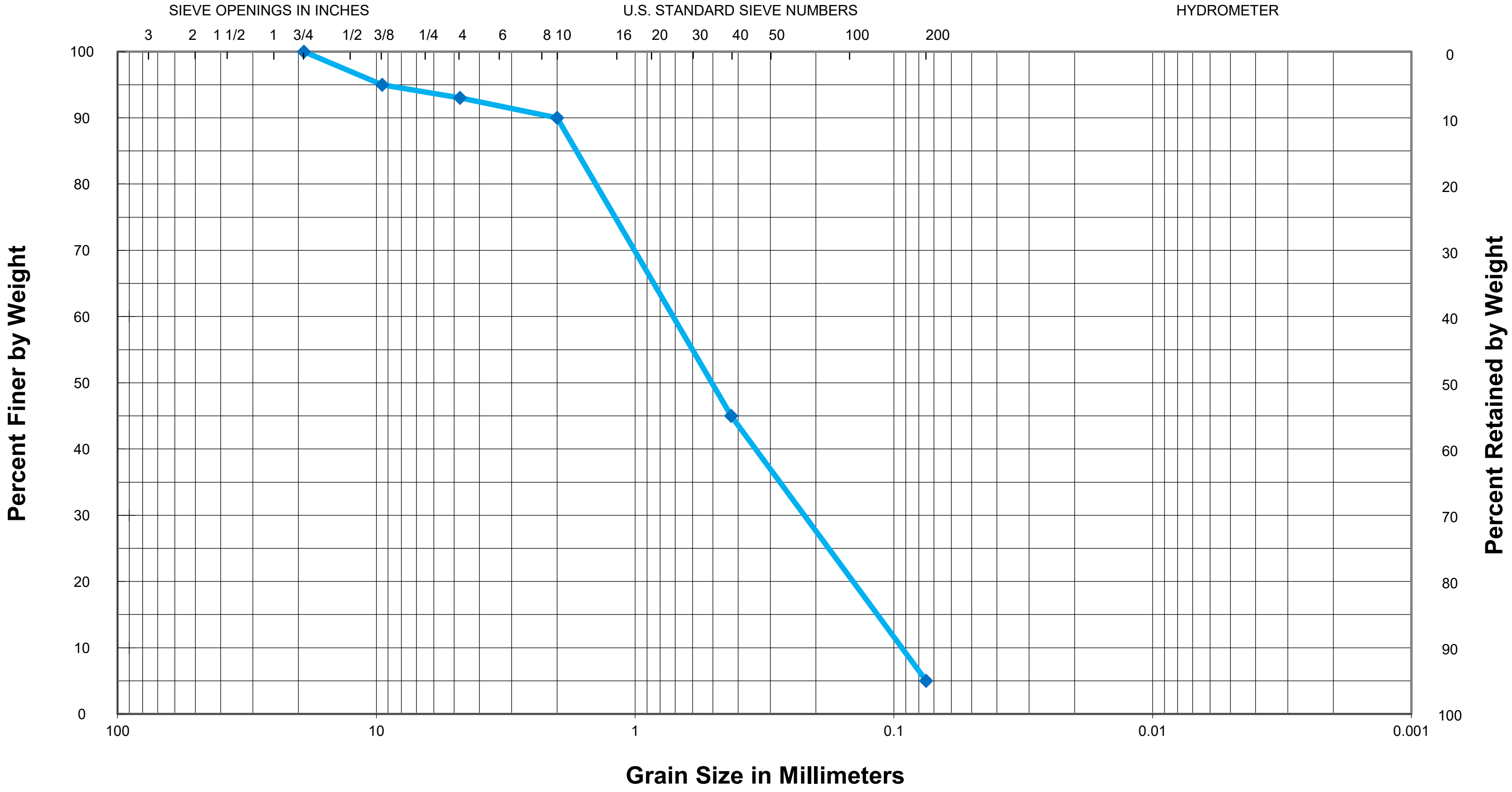
Sample: Boring E6, 49-50 ft
Description: Gray and brown fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

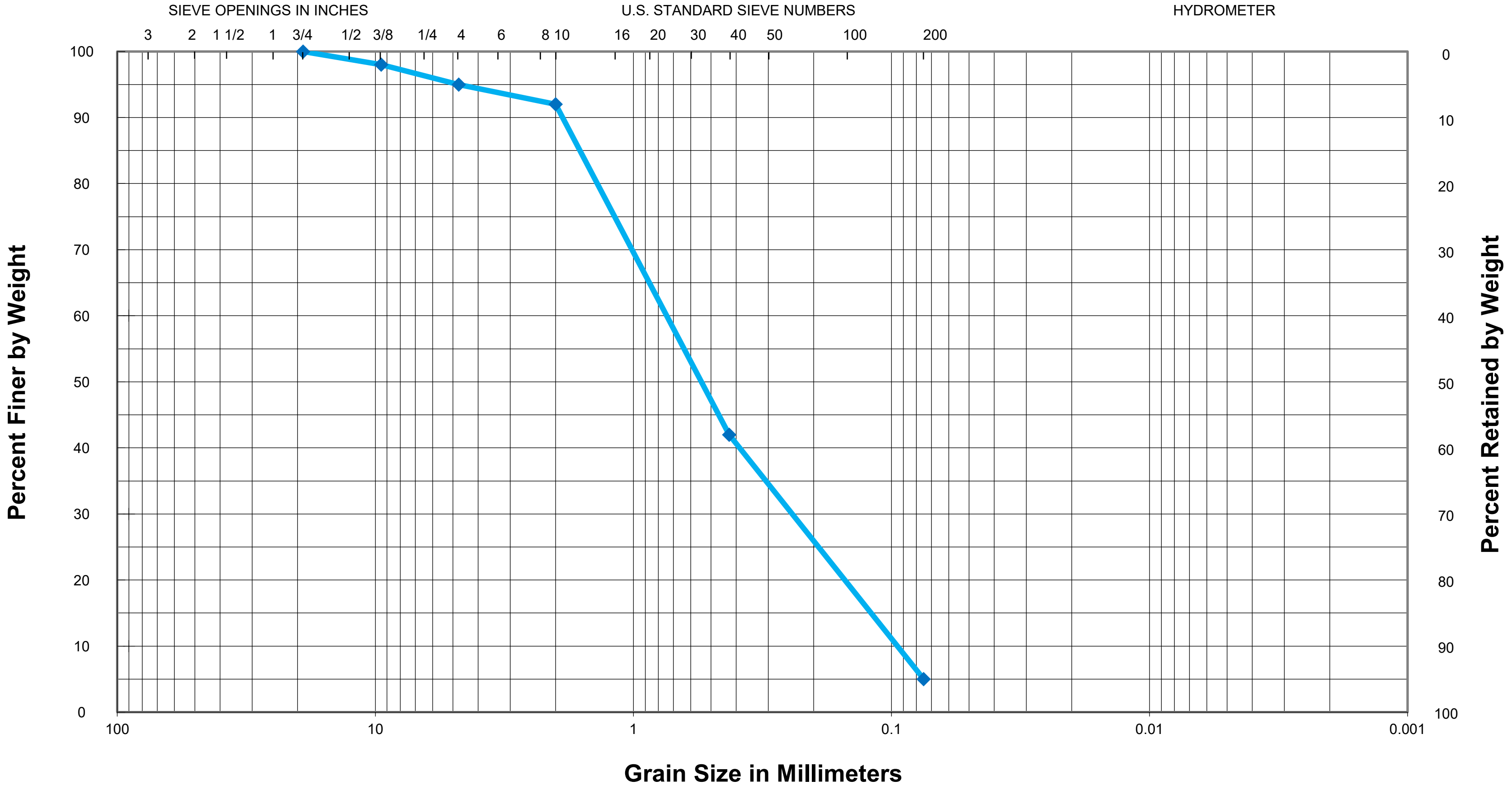
Sample: Boring E6, 69-70 ft
Description: Gray and brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

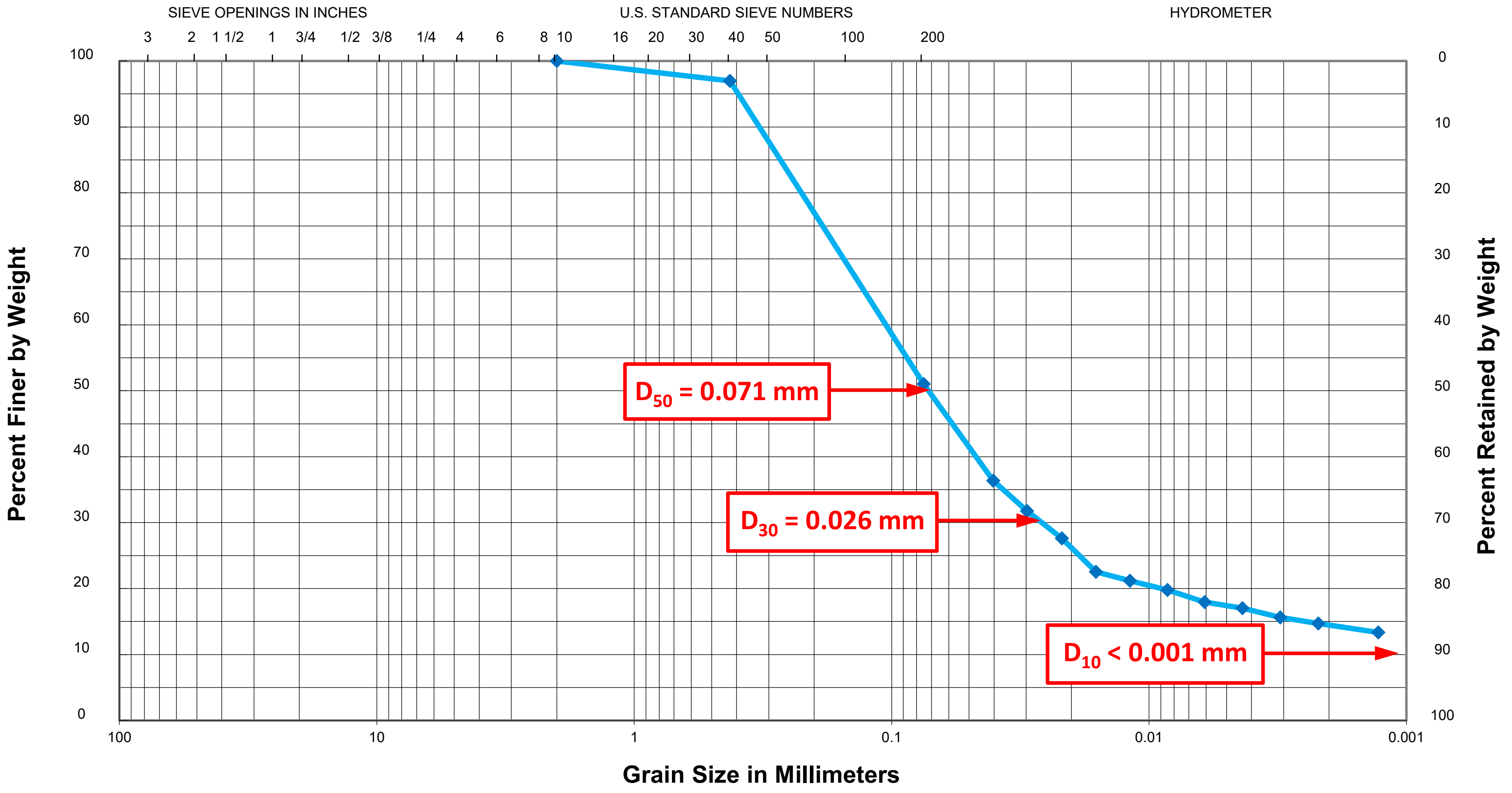
Sample: Boring E6, 89-90 ft
Description: Gray and brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

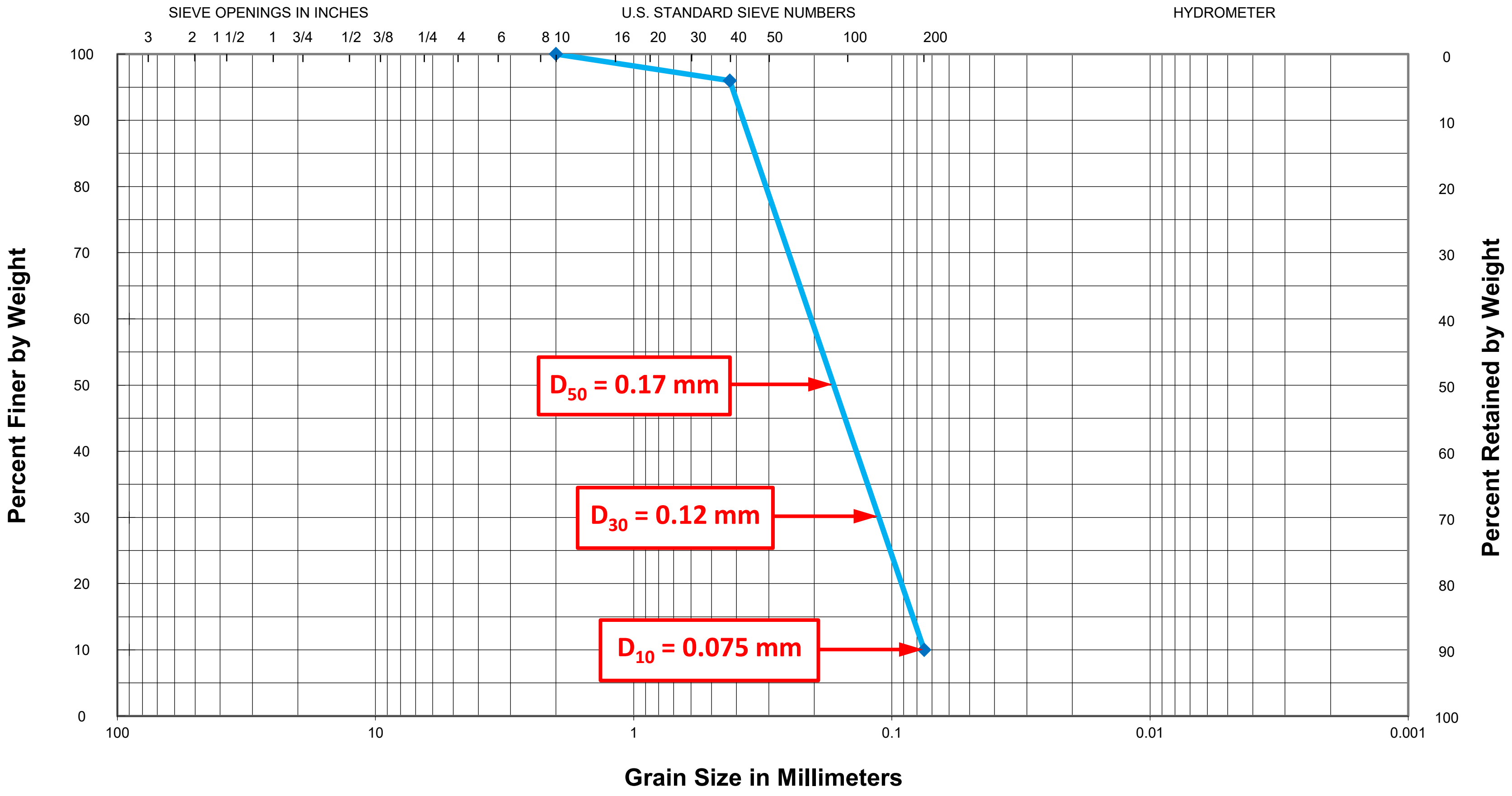
Sample: Boring E7, 4.5-5.5 ft; LL = 23, PL = 20, PI = 3
Description: Gray SILT w/ silty fine sand seams and layers

USCS Classification = ML
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.17 mm

D_{30} = 0.12 mm

D_{10} = 0.075 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

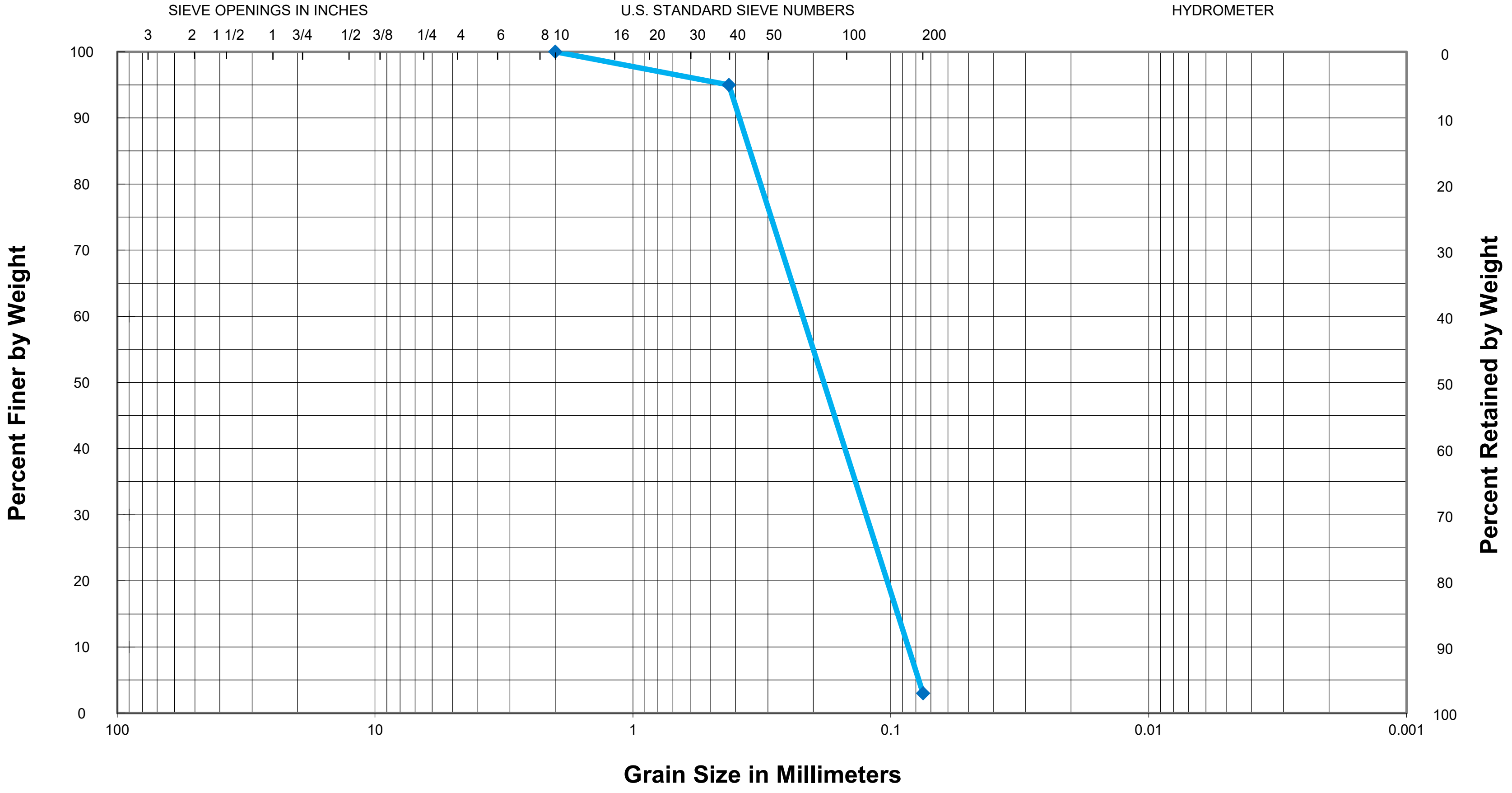
Sample: Boring E7, 9-10 ft
Description: Gray fine SAND, slightly silty w/ clay seams and layers

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

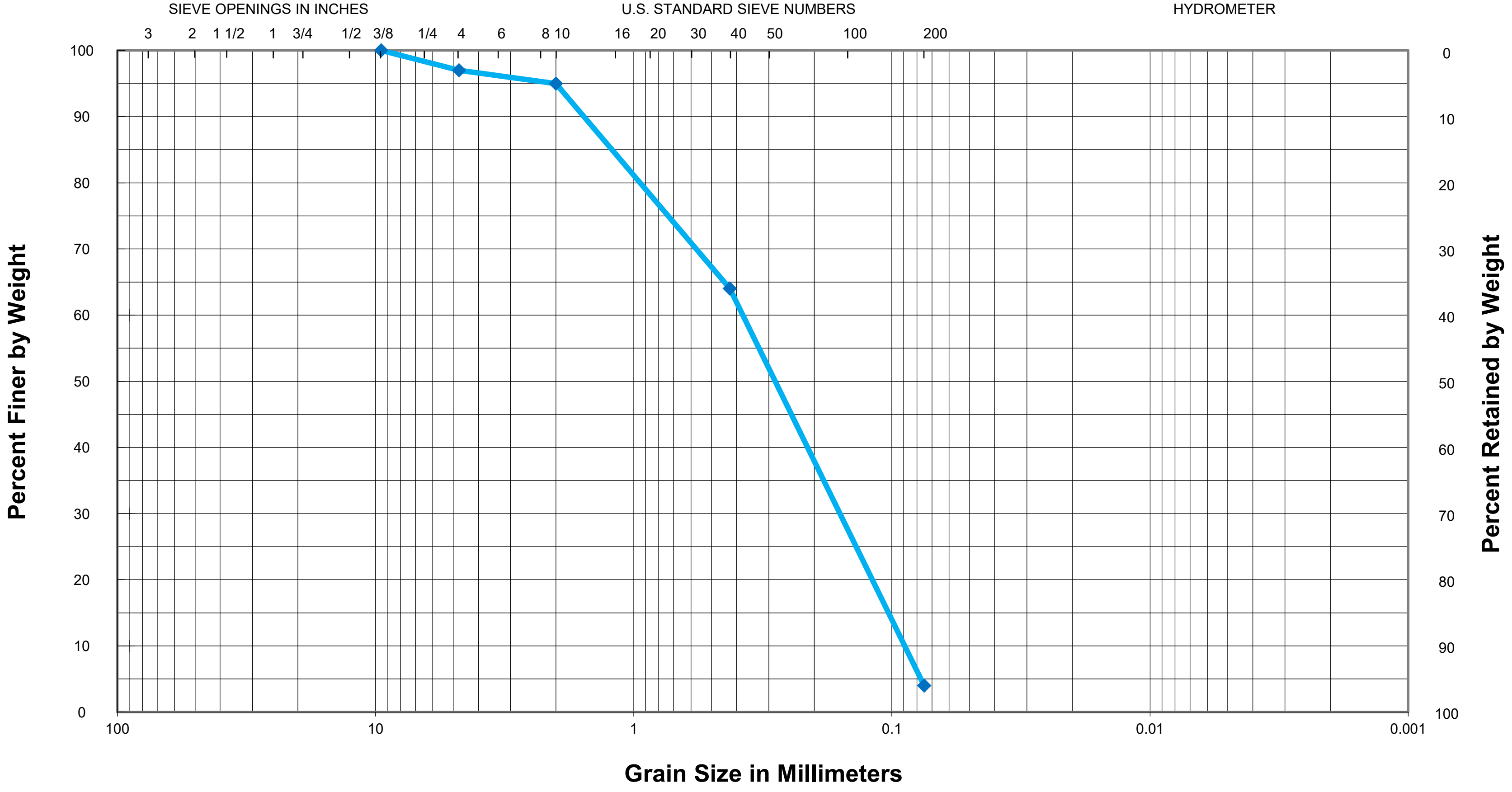
Sample: Boring E7, 29-30 ft
Description: Brown fine SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

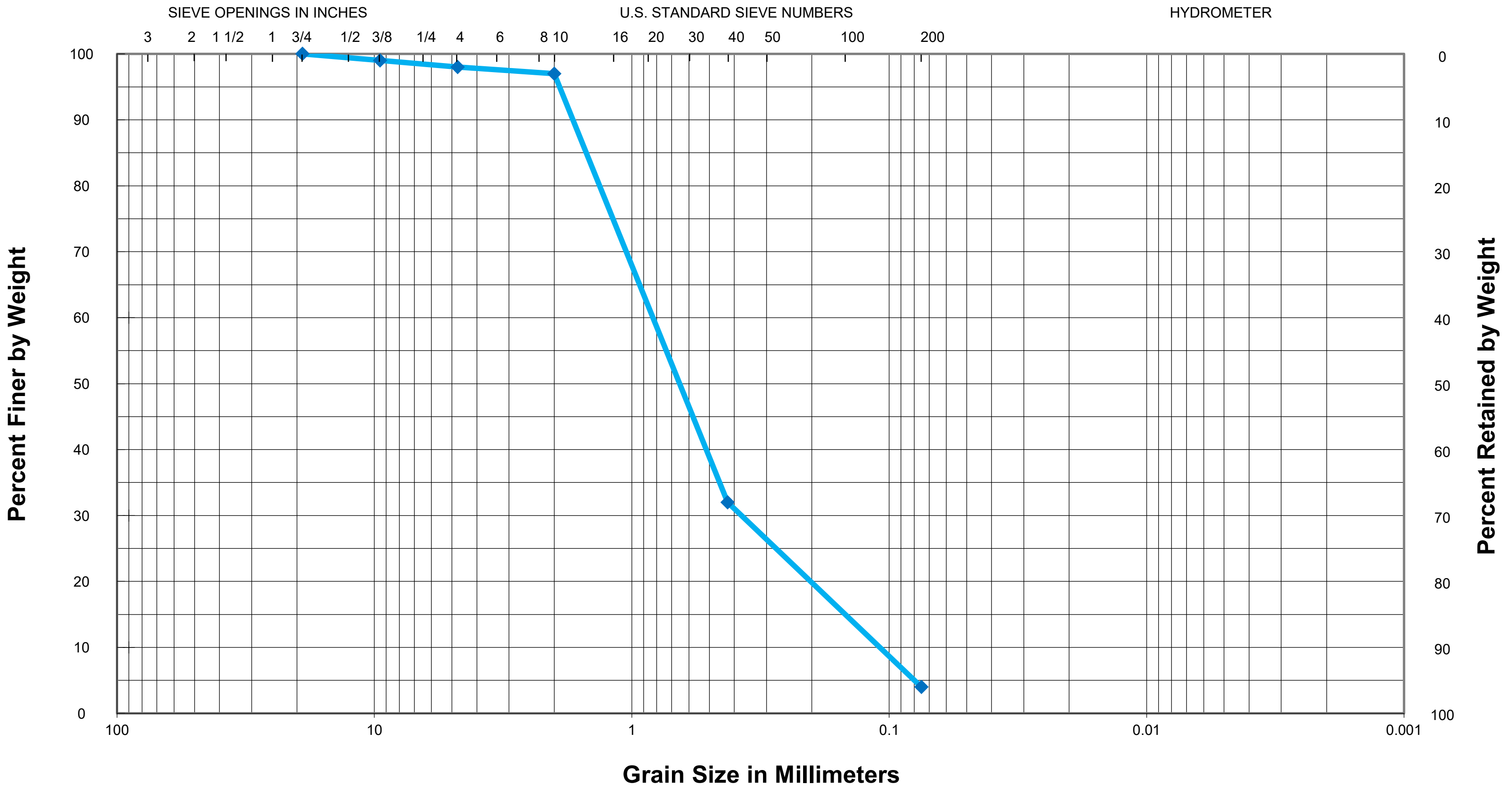
Sample: Boring E7, 49-50 ft
Description: Brownish gray and brown fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

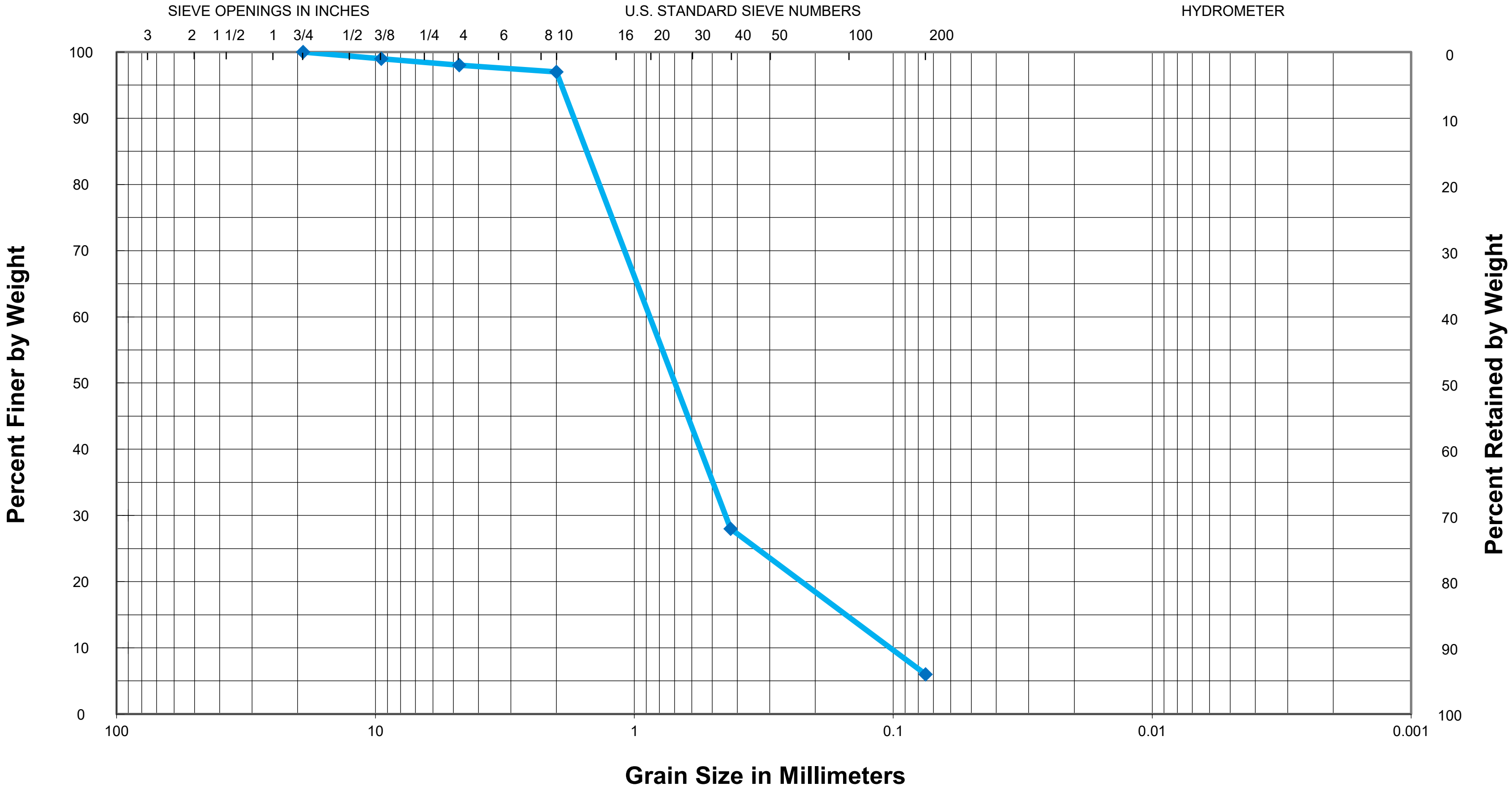
Sample: Boring E7, 64-65 ft
Description: Grayish brown fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

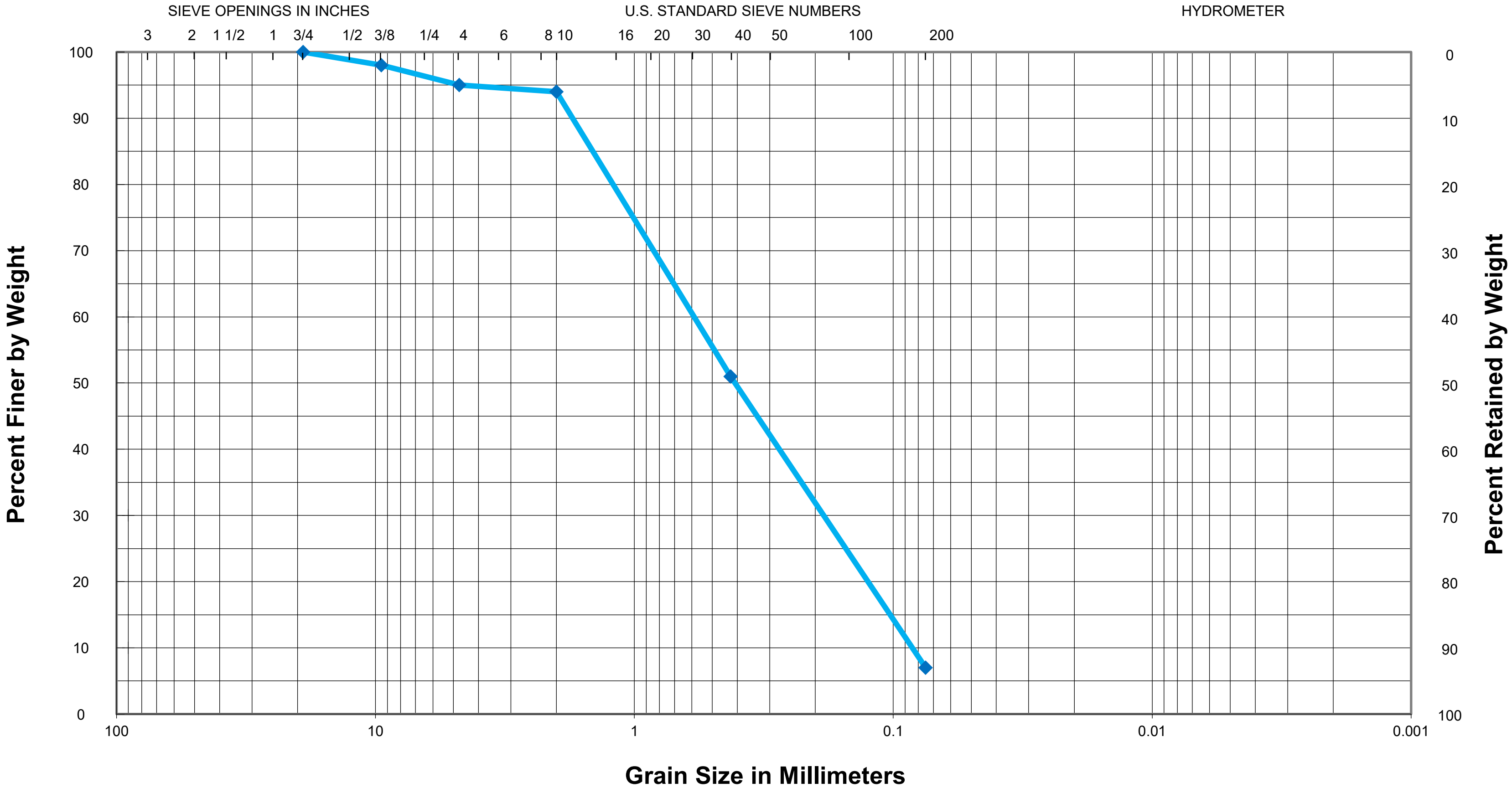
Sample: Boring E7, 84-85 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

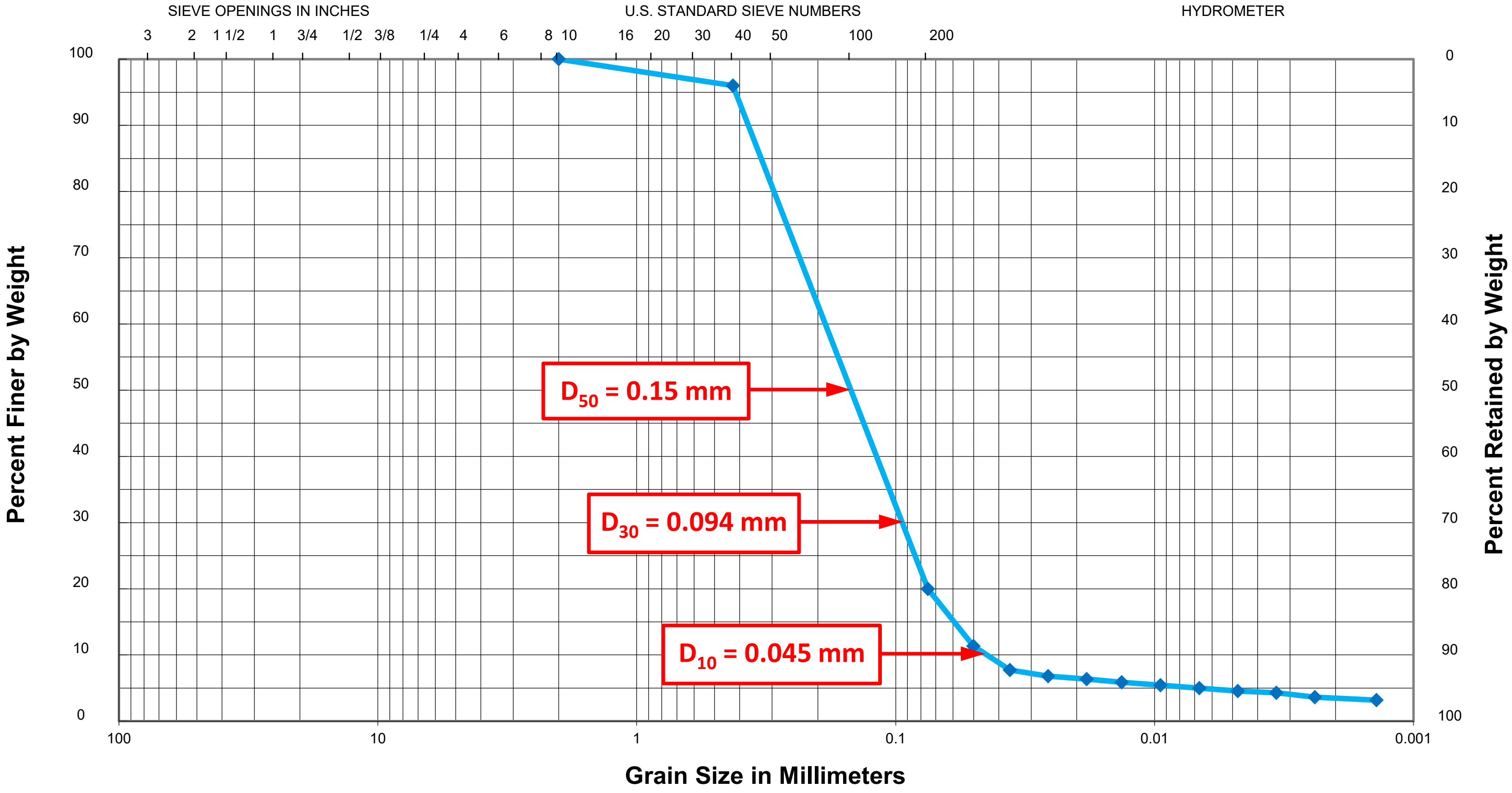


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E7, 99-100 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

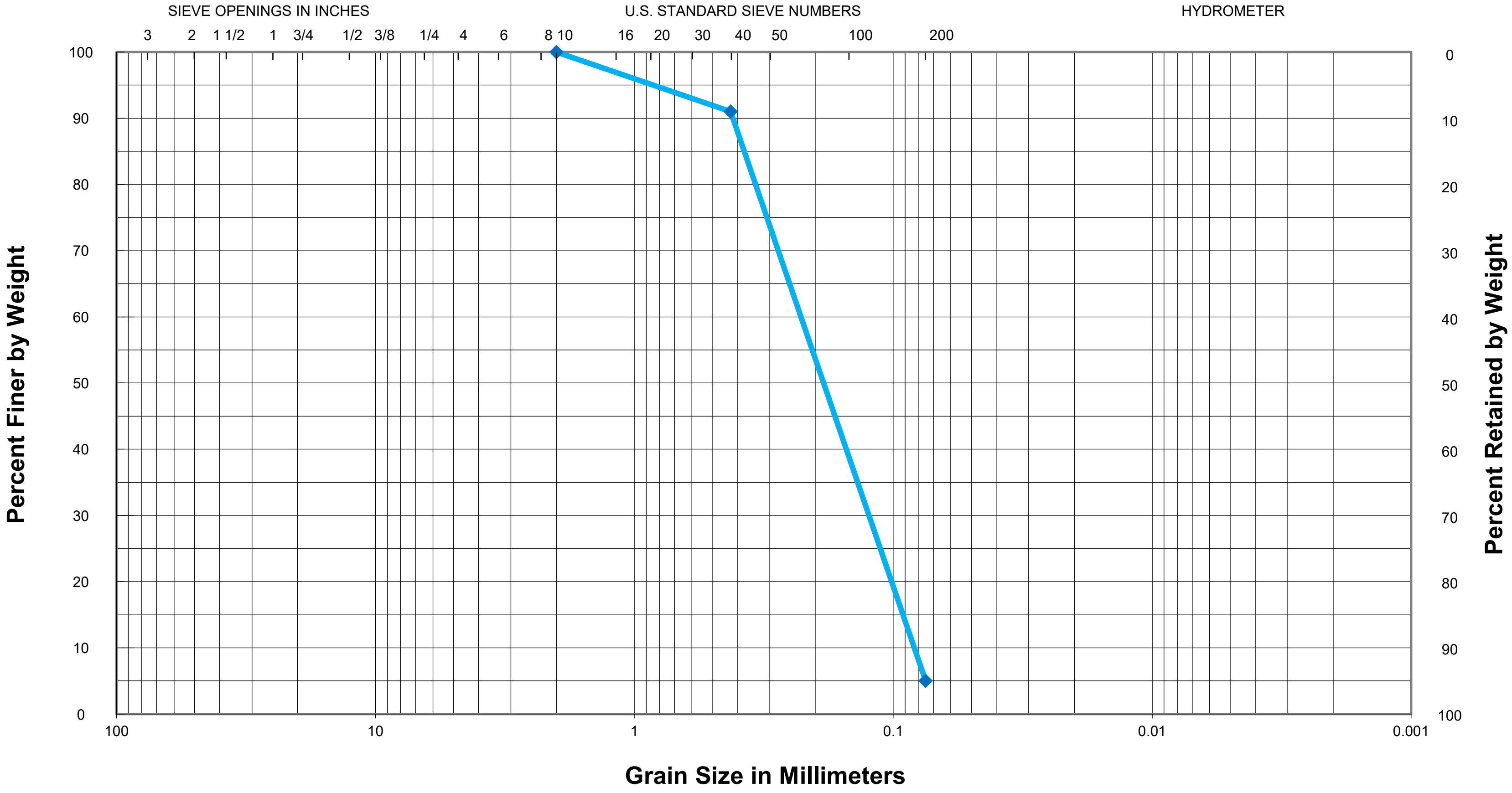
Sample: Boring E8, 14-15 ft
Description: Brownish gray silty fine SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

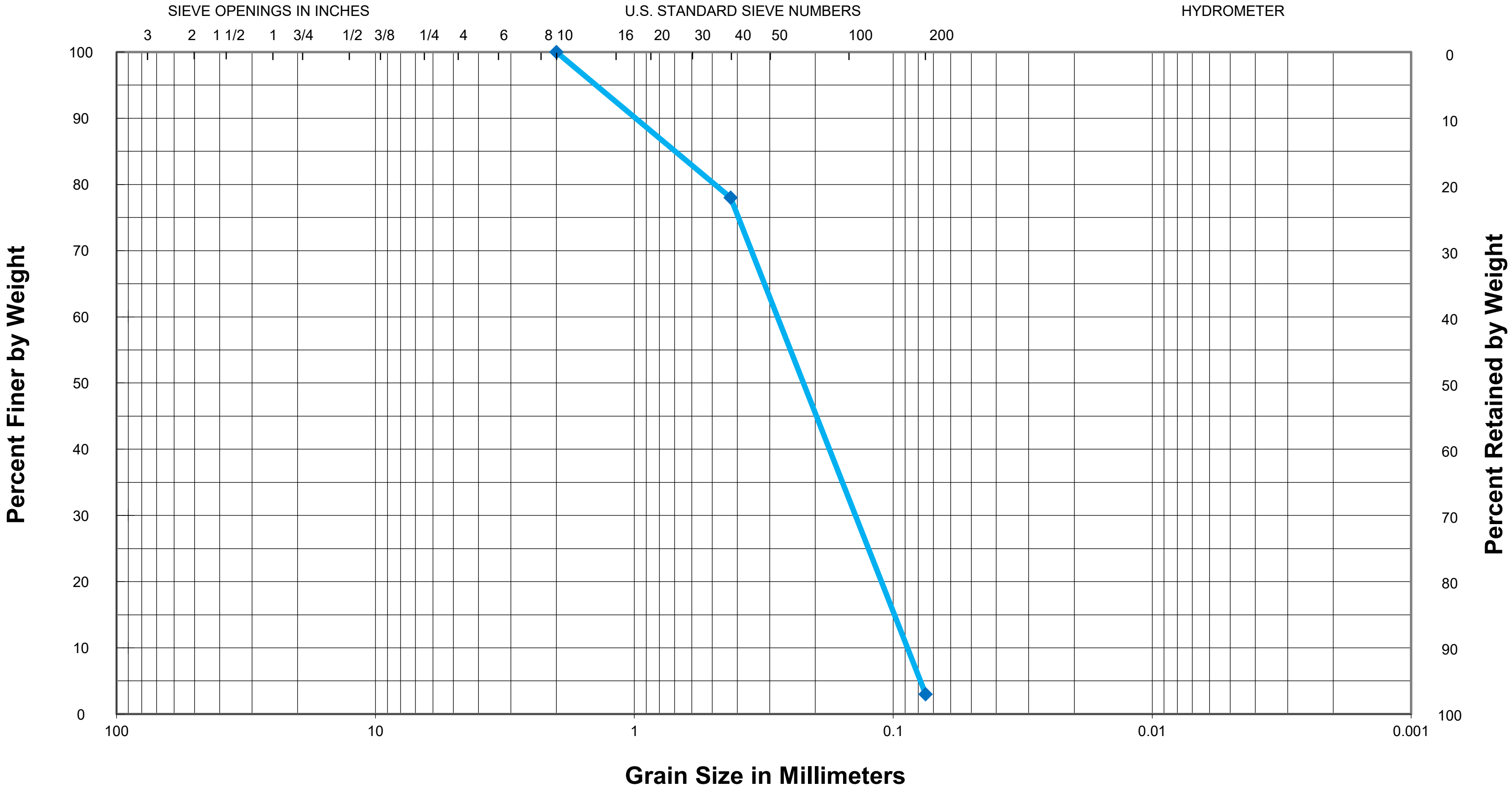
Sample: Boring E8, 29-30 ft
Description: Brownish gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

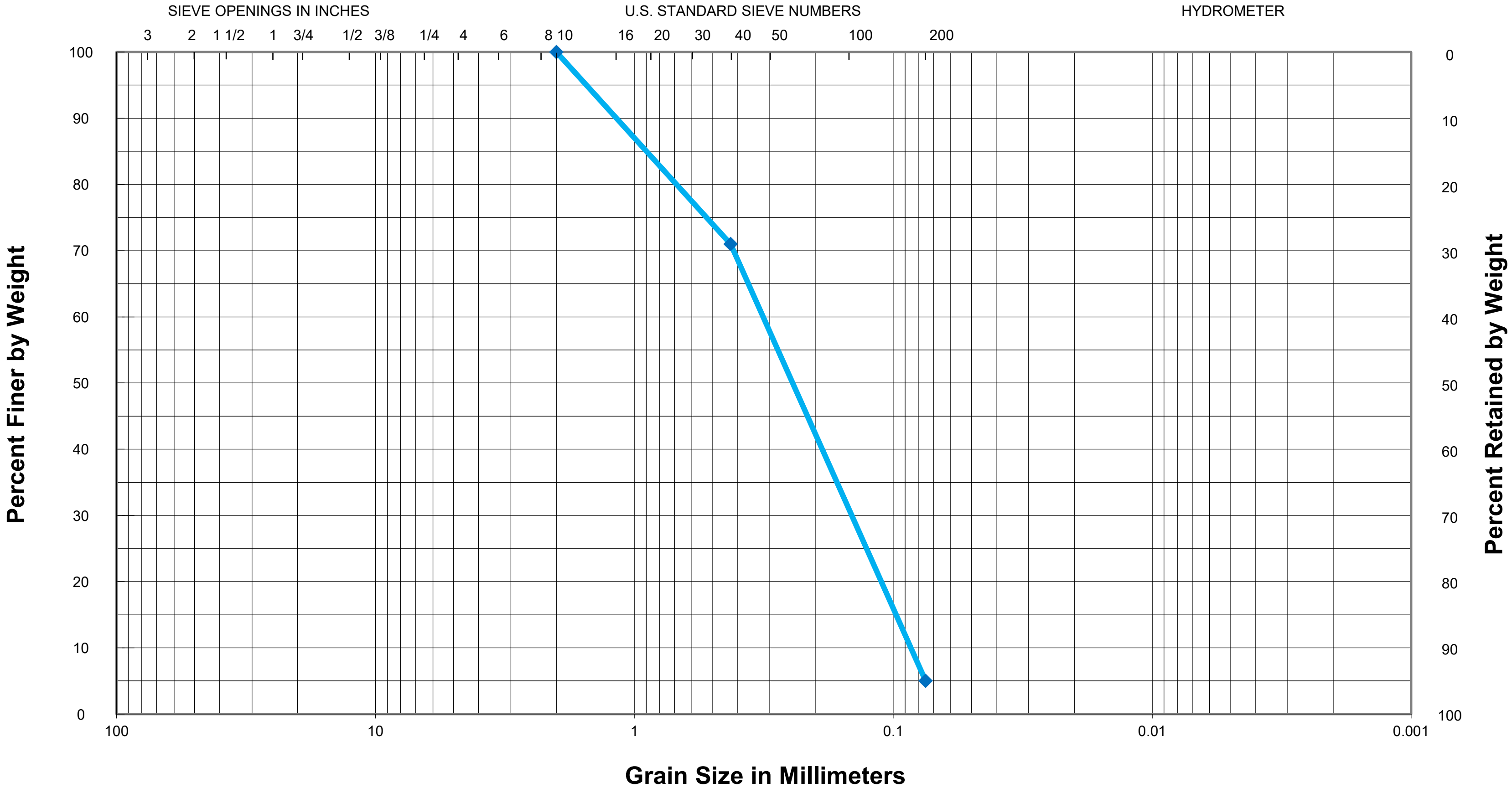
Sample: Boring E8, 59-60 ft
Description: Brownish gray fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

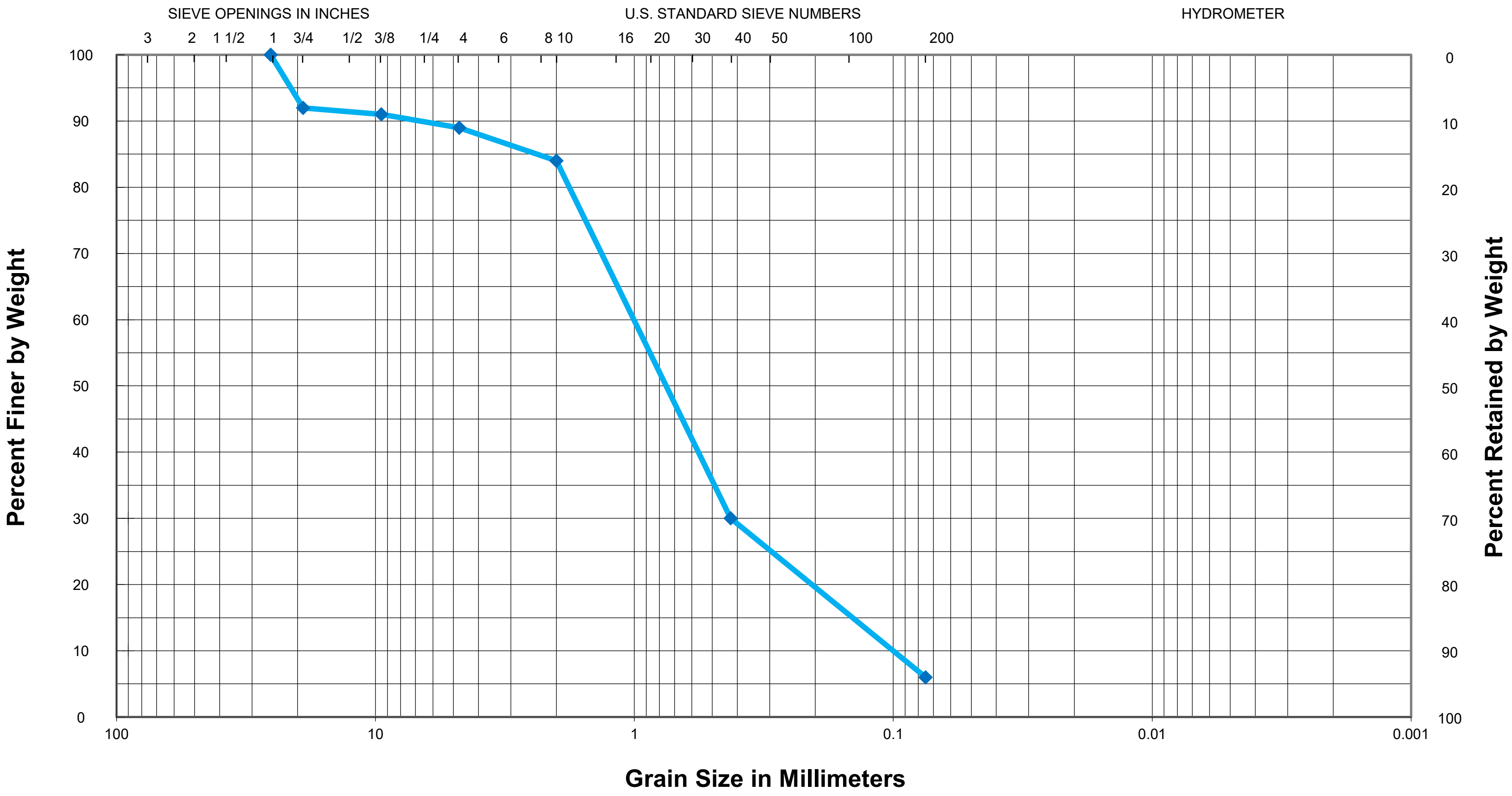
Sample: Boring E8, 74-75 ft
Description: Brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

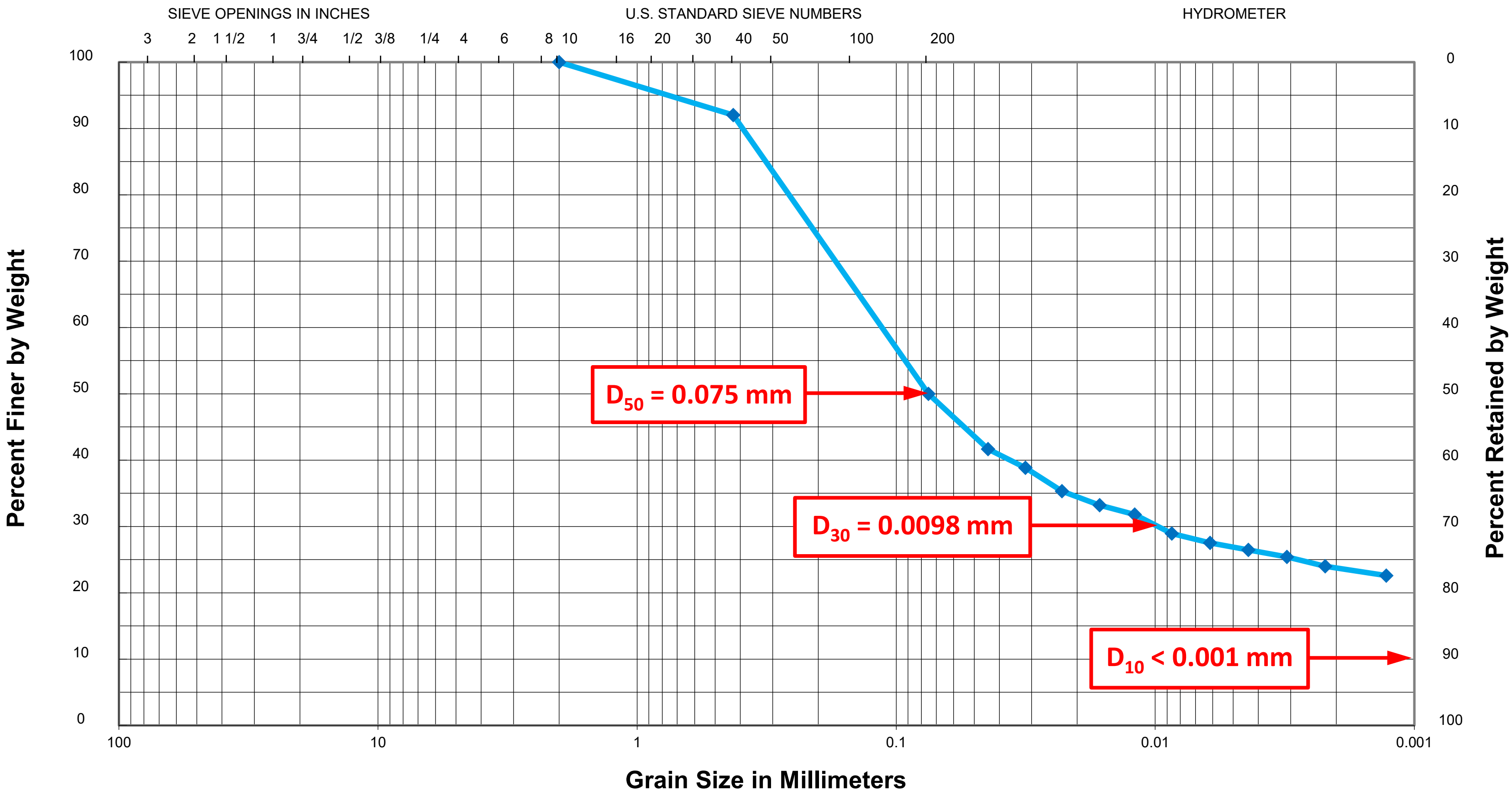
Sample: Boring E8, 84-85 ft
Description: Grayish brown fine to medium SAND, slightly silty w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.075 mm

D_{30} = 0.0098 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

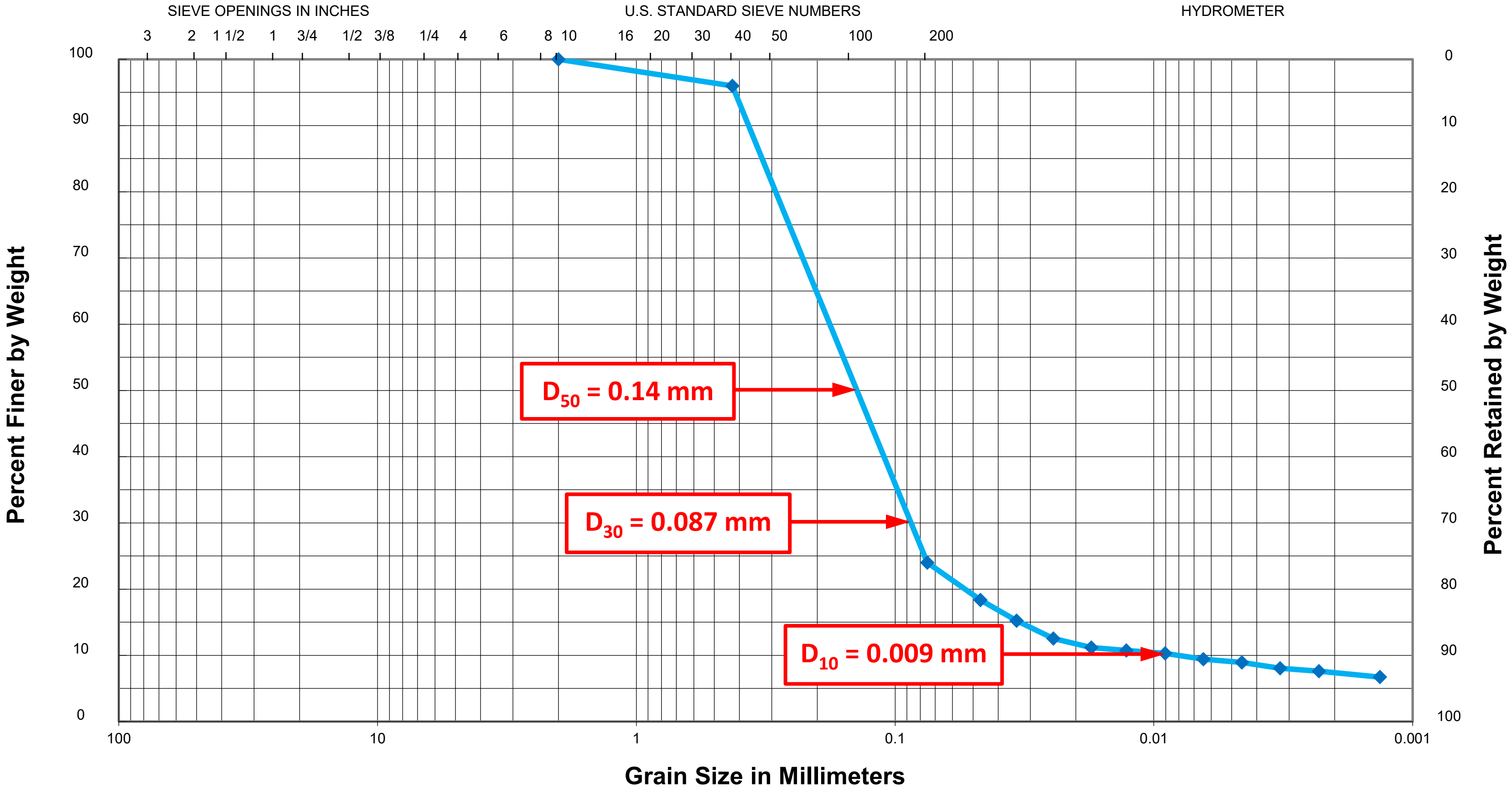
Sample: Boring E9, 1.5-2.5 ft
Description: Dark gray clayey fine SAND (fill)

USCS Classification = SC
AASHTO Classification = A-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



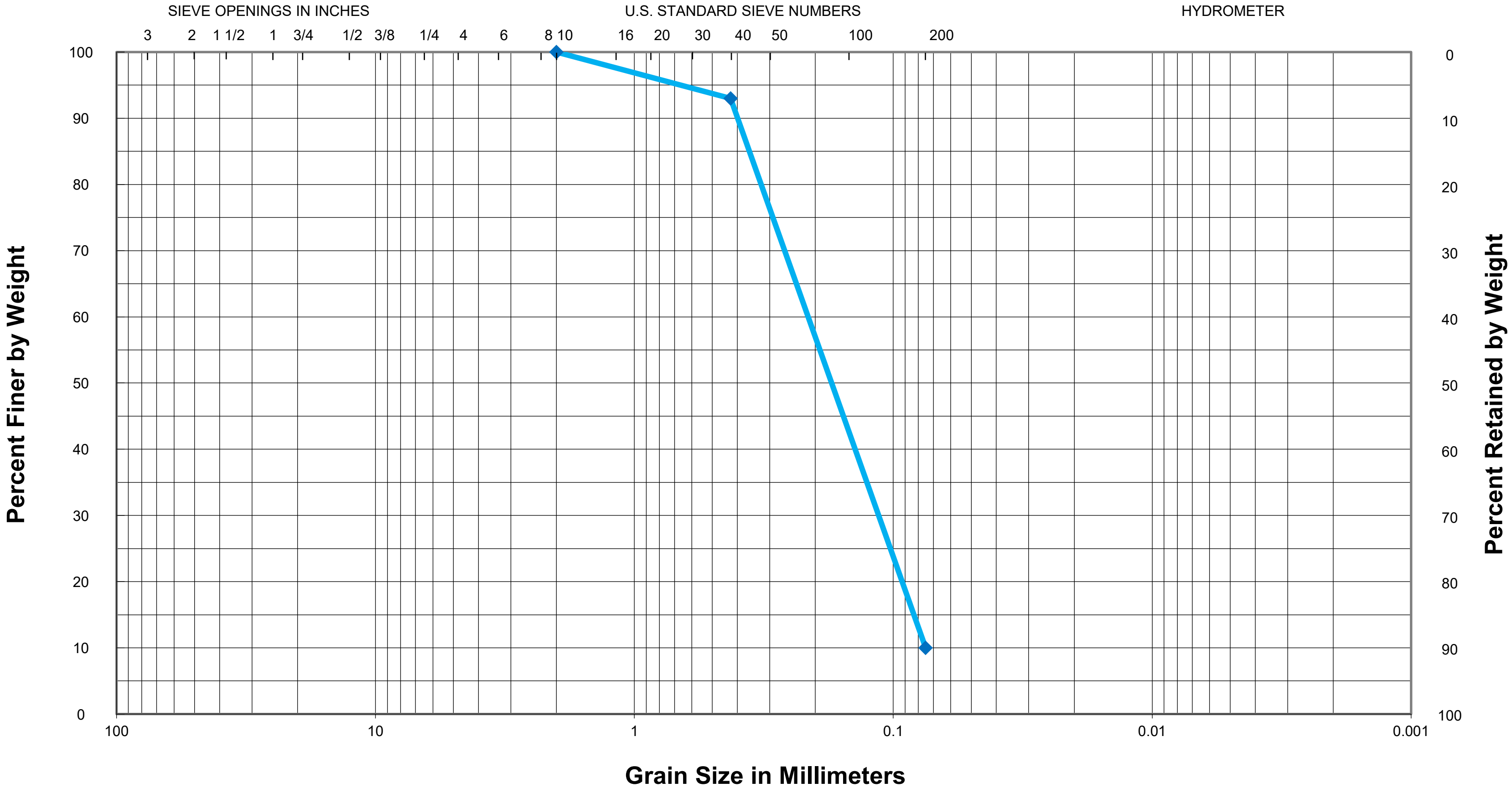
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring E9, 5.5-6.5 ft
Description: Reddish brown and gray silty fine SAND w/ clay pockets (fill)

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

23-031

GRAIN SIZE CURVE



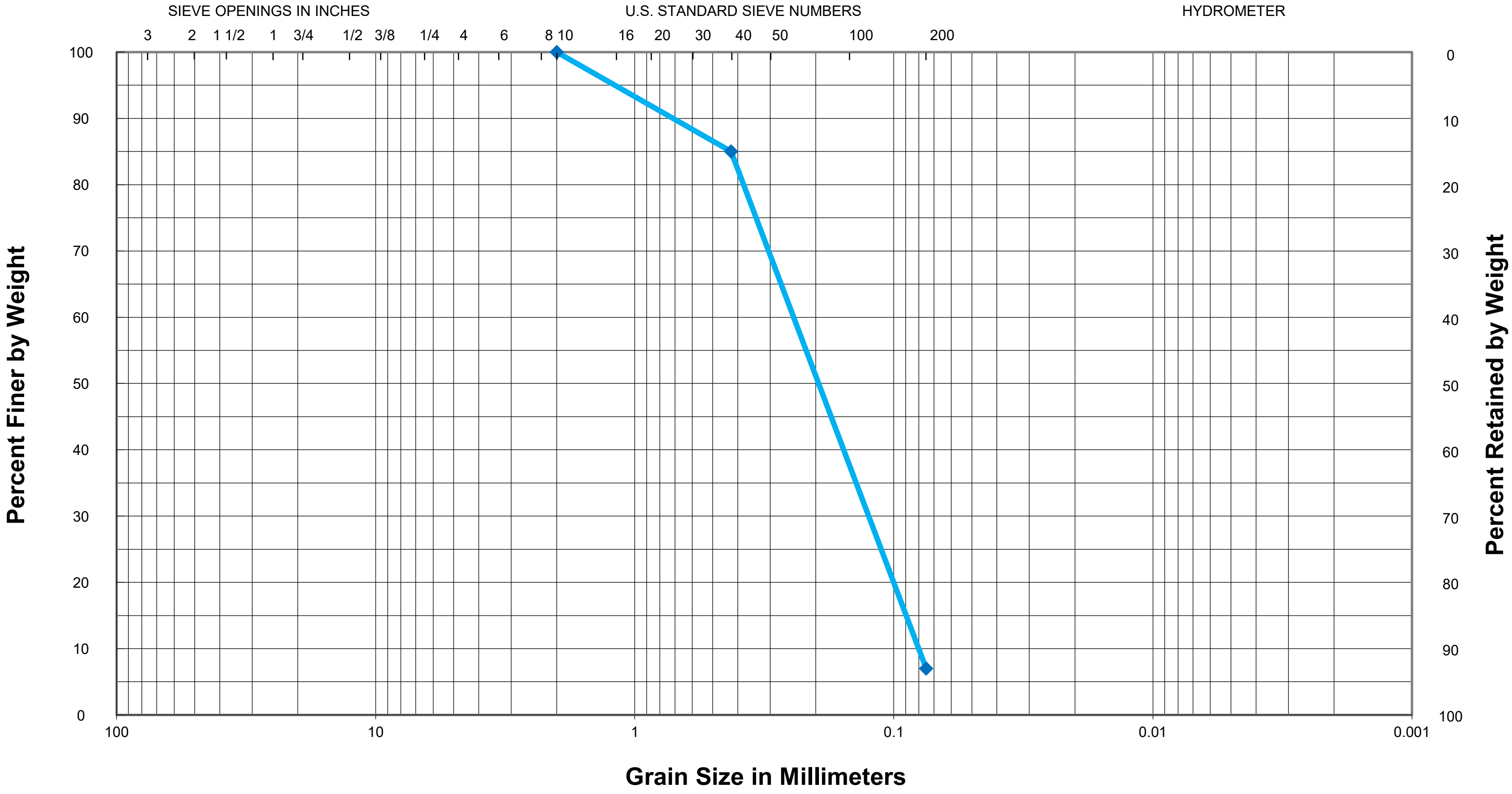
<table>
<tr><th colspan="2">GRAVEL</th><th colspan="3">SAND</th><th rowspan="2">SILT OR CLAY</th></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Sample: Boring E9, 29-30 ft
Description: Gray and brown fine SAND, slightly silty

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

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

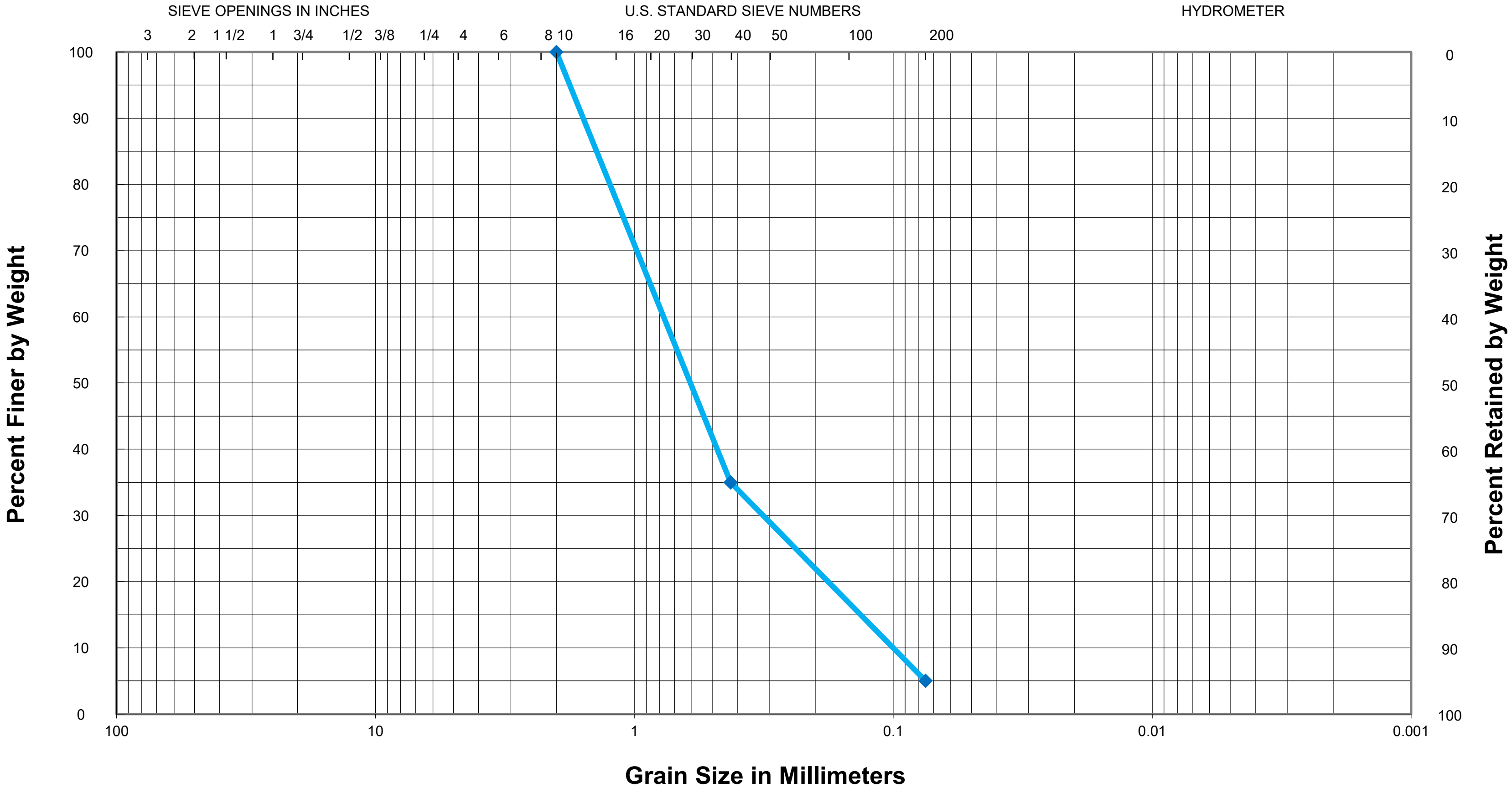
Sample: Boring E9, 39-40 ft
Description: Grayish brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



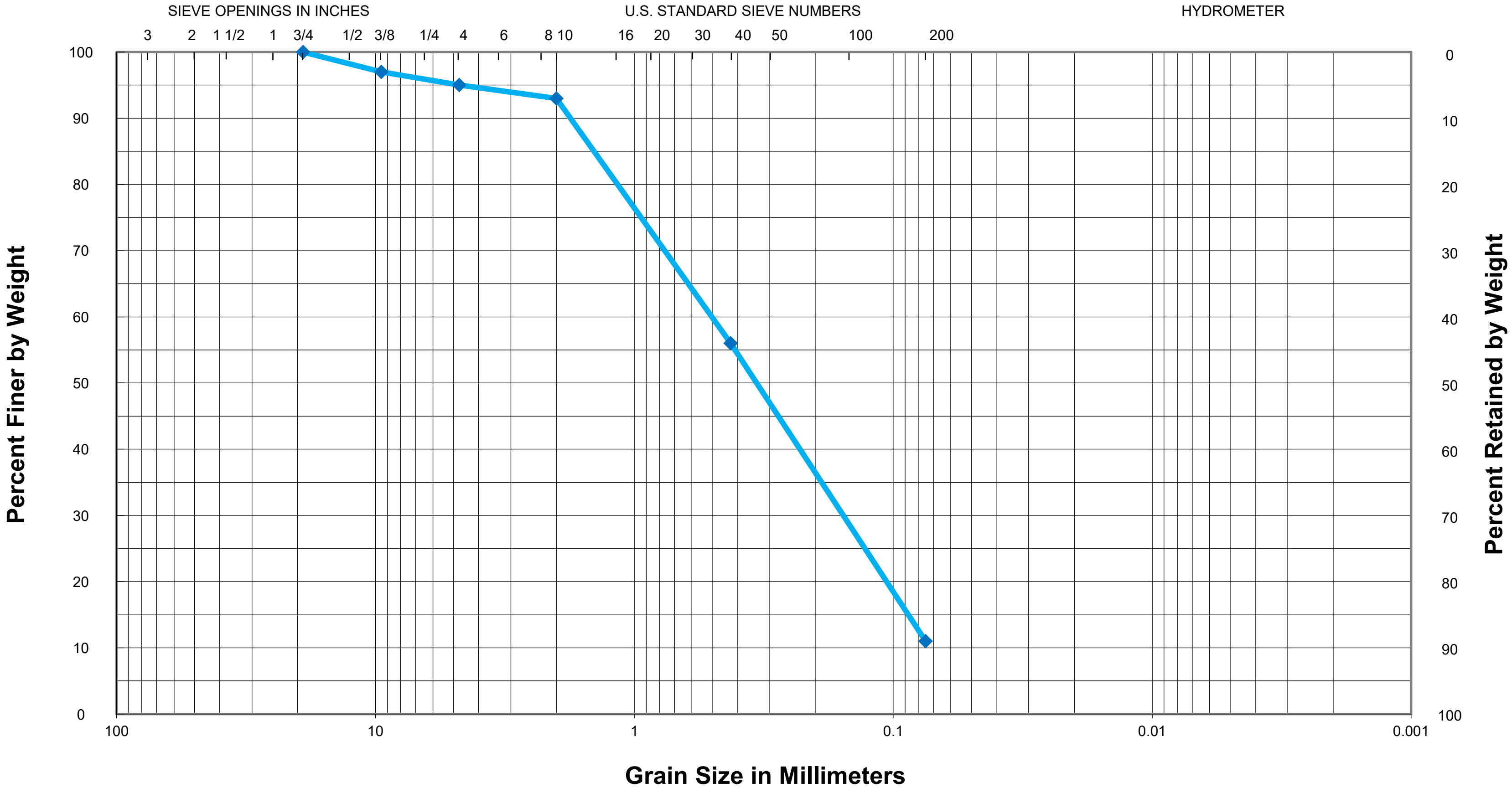
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E9, 49-50 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

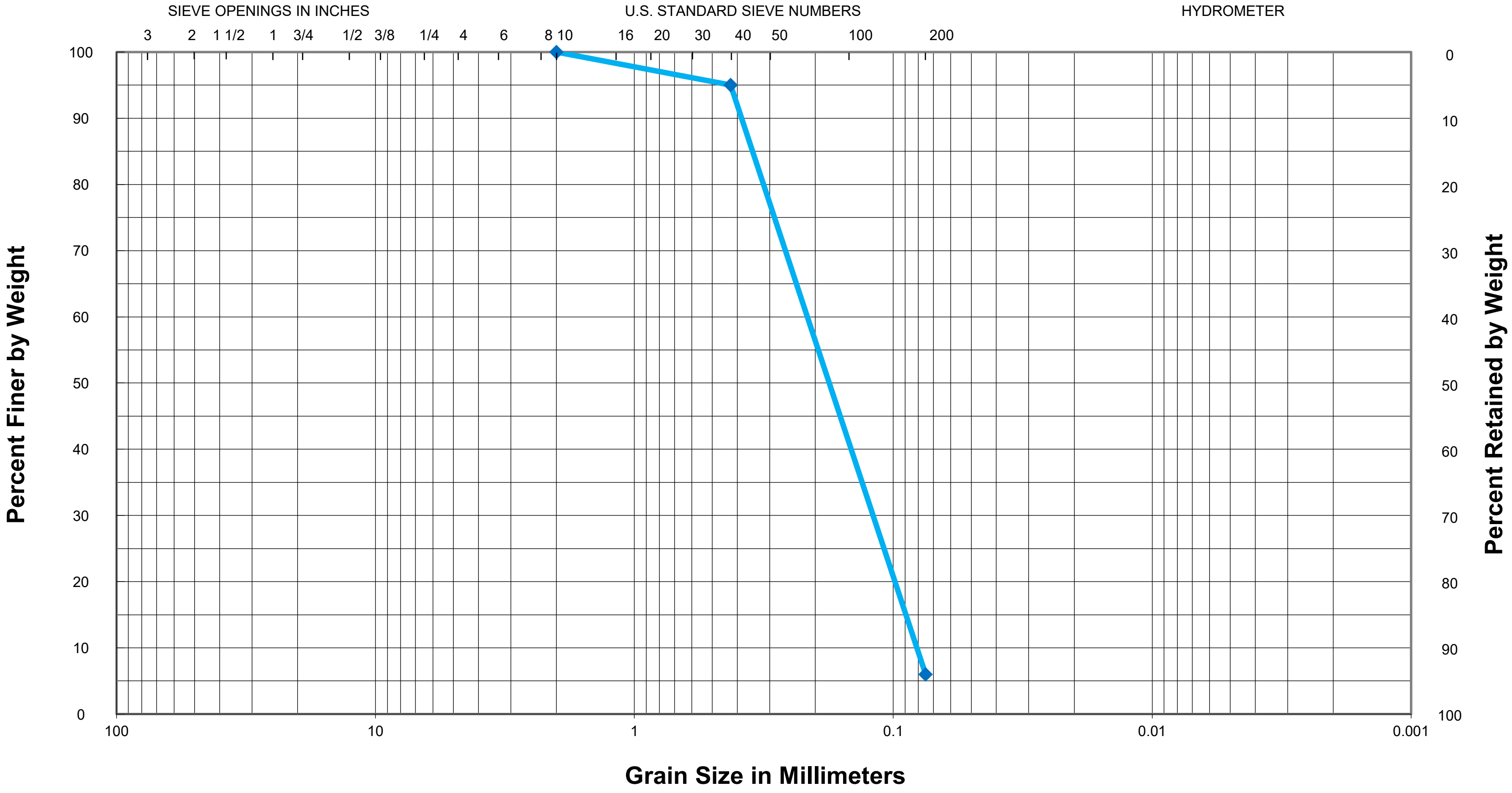
Sample: Boring E9, 69-70 ft
Description: Dark gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring E9, 84-85 ft
Description: Brownish gray fine SAND, slightly silty

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

APPENDIX D

Table 2. Summary of Site-Specific Response Results

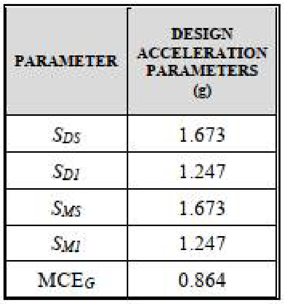
Period	Site 2-Tyronza River	Site 5 – Righthand Chute Little River
A_s (g) (Site-adjusted PGA)	0.769	0.864
S_{DS} (g) (0.2 sec)	1.565	1.673
S_{D1} (g) (1 Sec)	1.197	1.247
Seismic Performance Zone	ZONE 4	ZONE 4

Table 4. Average Shear Wave Velocity and AASHTO Site Classification

CPT Designation	Average Shear Wave Velocity	AASHTO Site Class
SCPT-2	701	D
SCPT-5-South	709	D
SCPT-5-North	701	D
SCPT-7	712	D

Right Hand Chute Little River Site:

Table 6. Site-Specific Response Accelerations Considering 5% Damping.



PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	1.673
S_{D1}	1.247
S_{MS}	1.673
S_{M1}	1.247
MCE_G	0.864

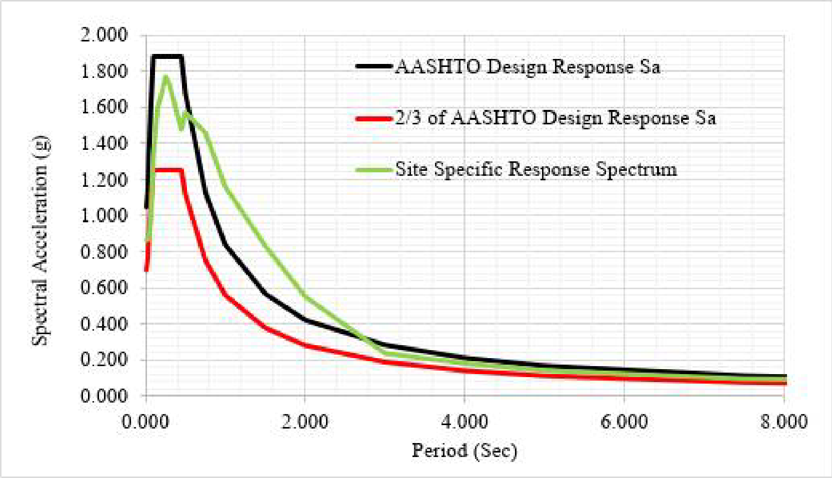
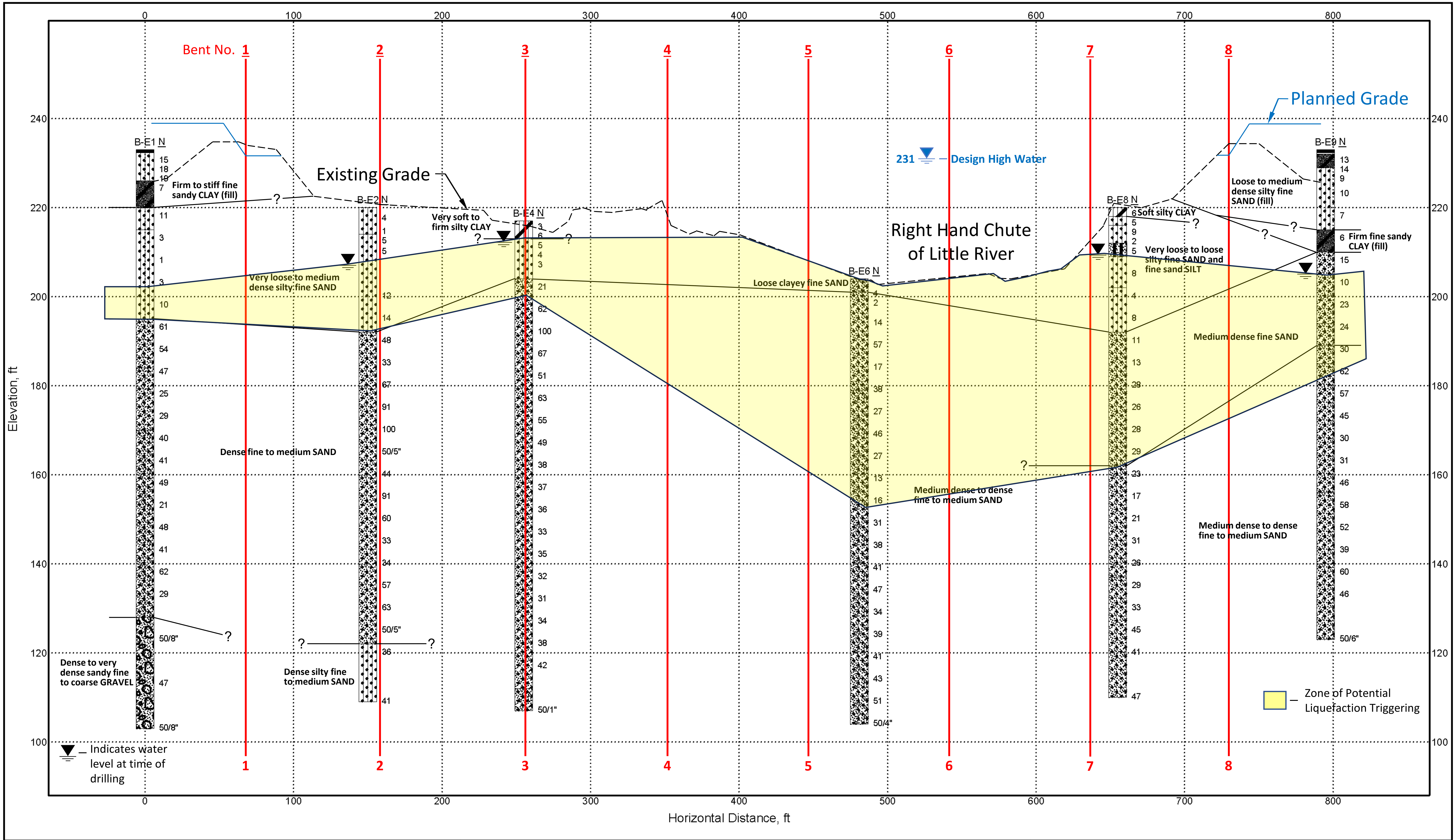


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

APPENDIX E



Grubbs, Hoskyn, Barton & Wyatt, LLC

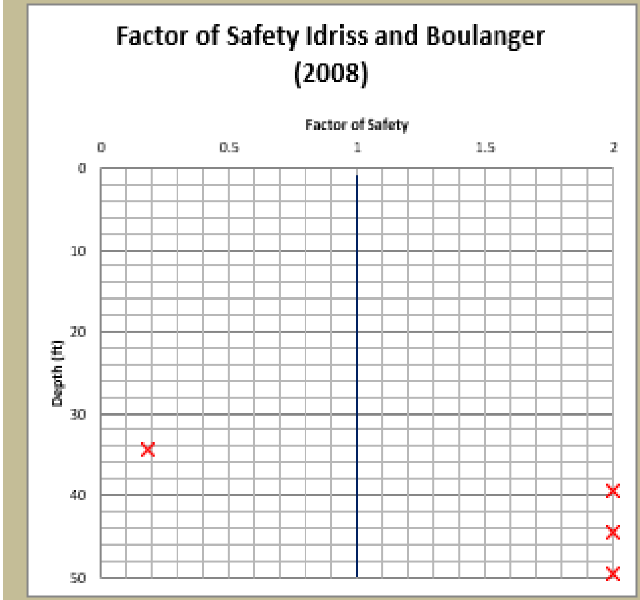
NOTES:
1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface approximate.

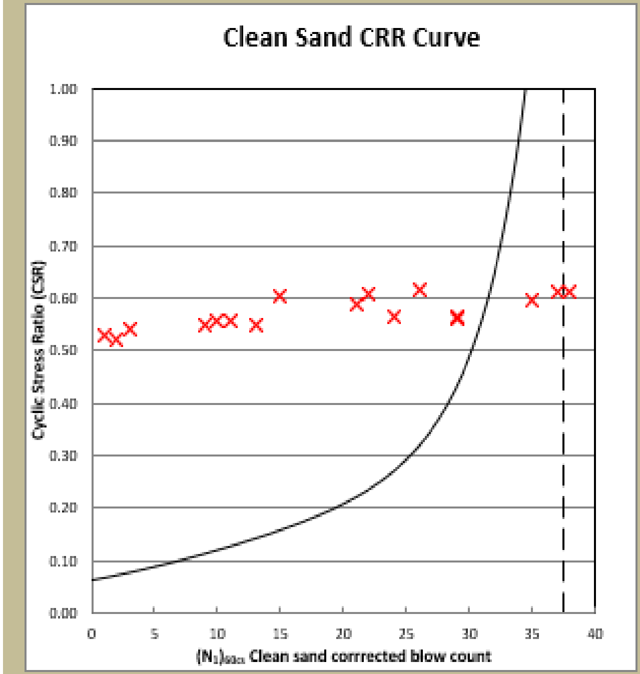
SCALE:
1'' = 60' Horizontal
1'' = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over RHC of Little River
Poinsett Co., Arkansas

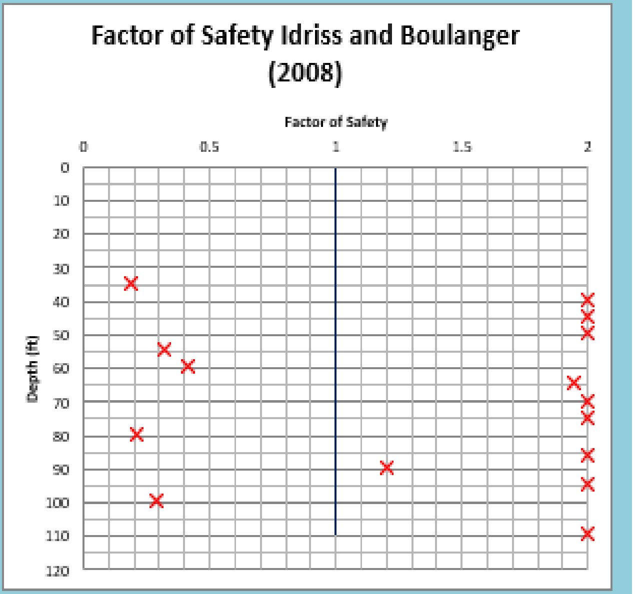
Project Number: 23-031

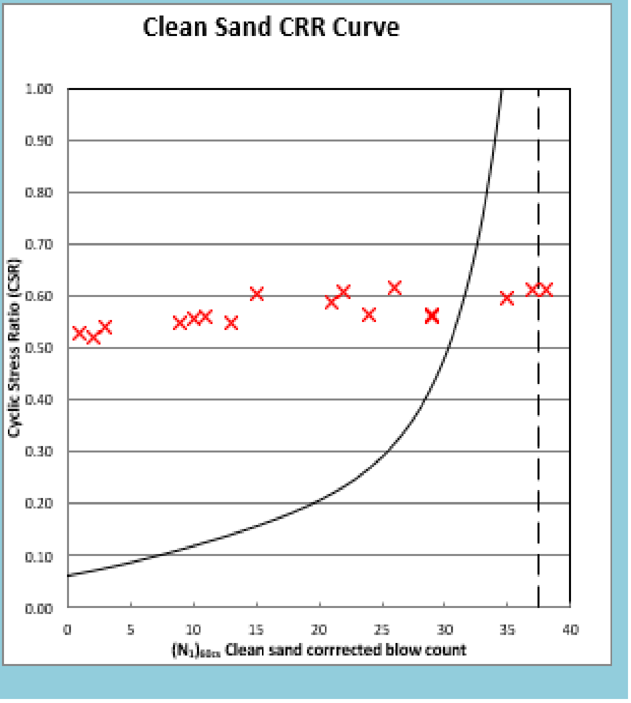
Boring Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Grade Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Boring E1

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A UES Company

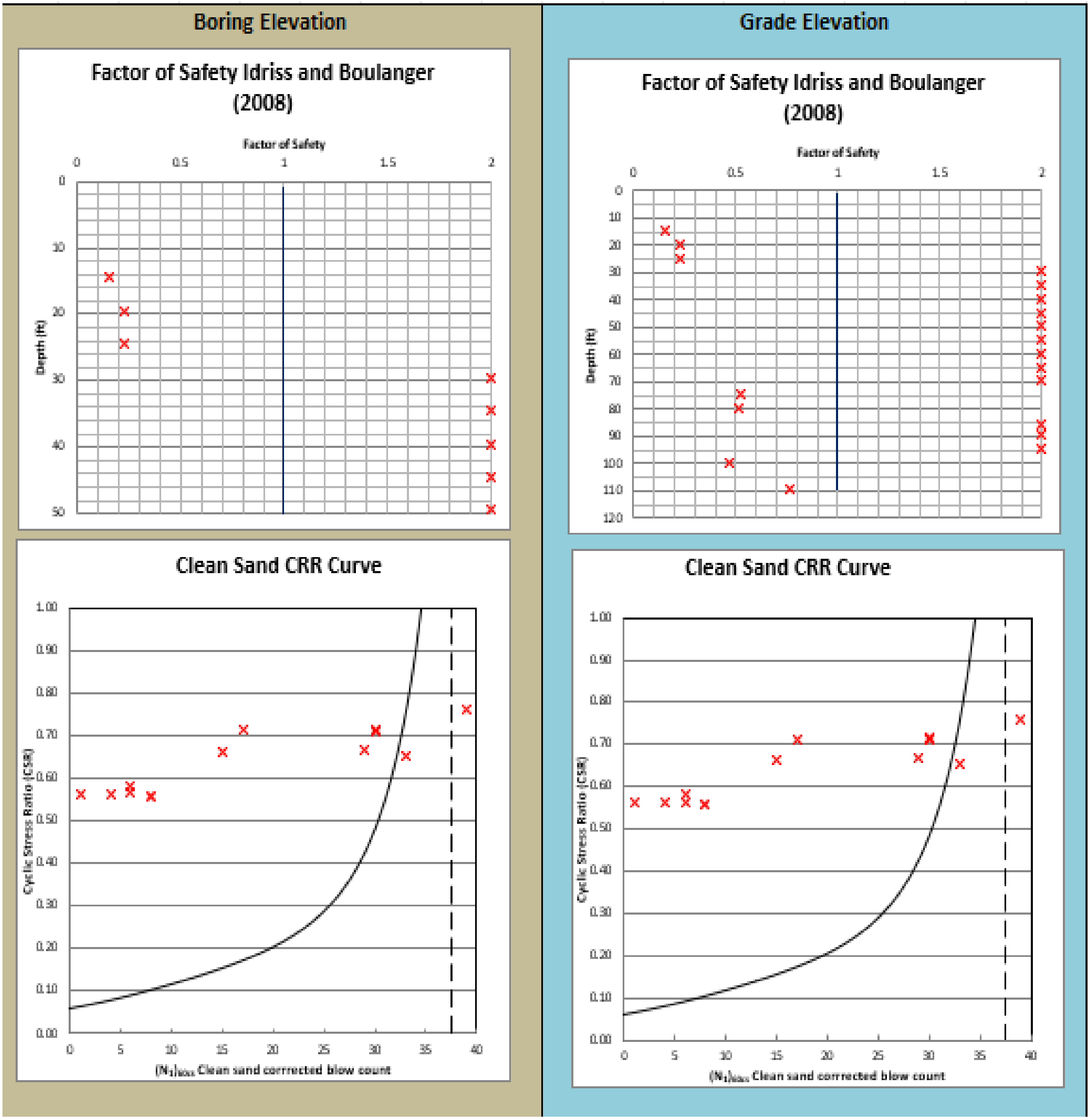
LIQUEFACTION ANALYSIS RESULTS

101124 Hwy. 135 over Right Hand Chute of Little River

Poinsett County, Arkansas

Job No. 23-031

Plate



Boring E2 (Bent 2)

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LIQUEFACTION ANALYSIS RESULTS

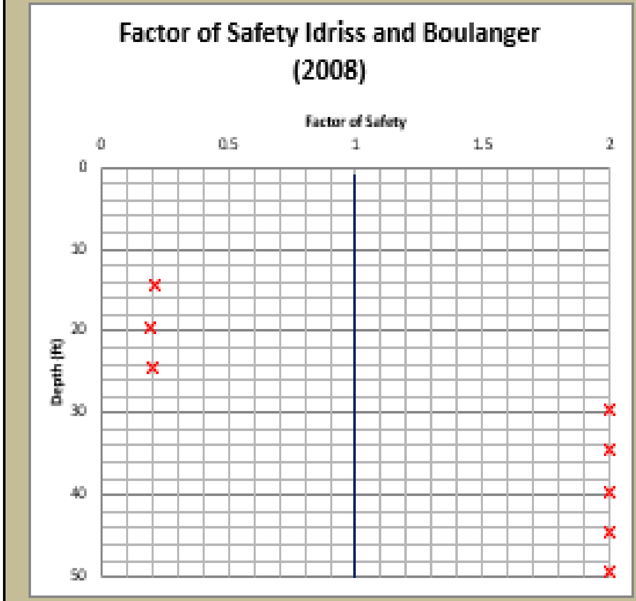
101124 Hwy. 135 over Right Hand Chute of Little River

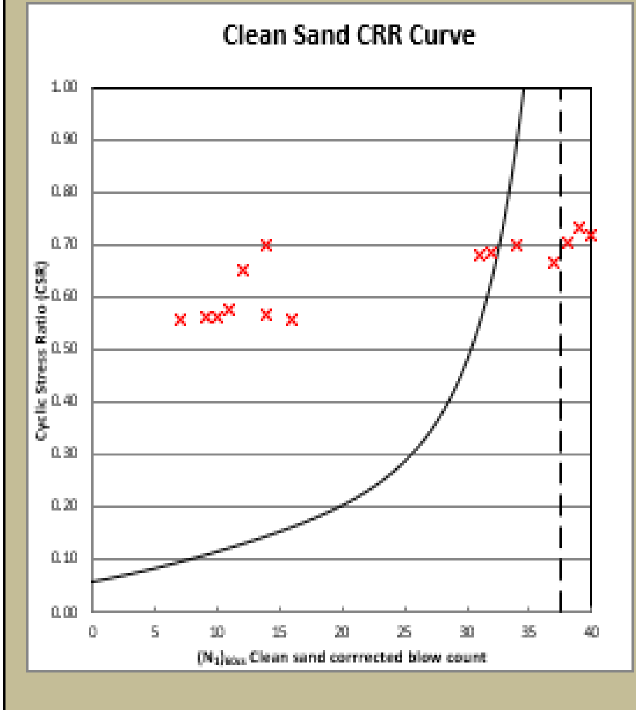
Poinsett County, Arkansas

Job No. 23-031

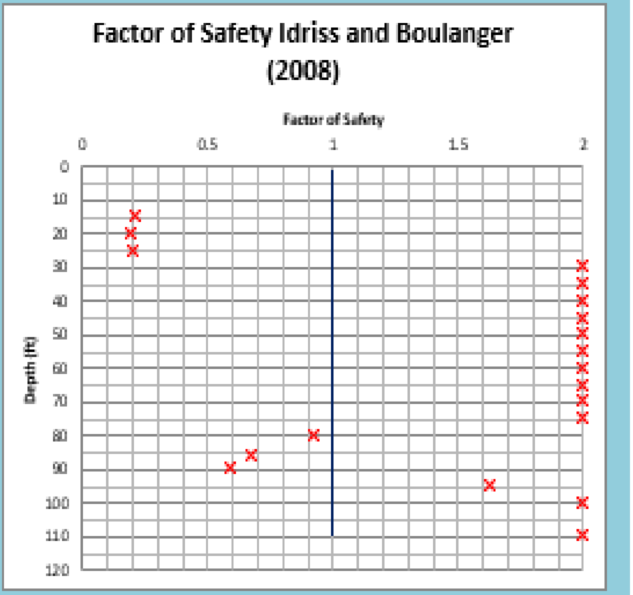
Plate

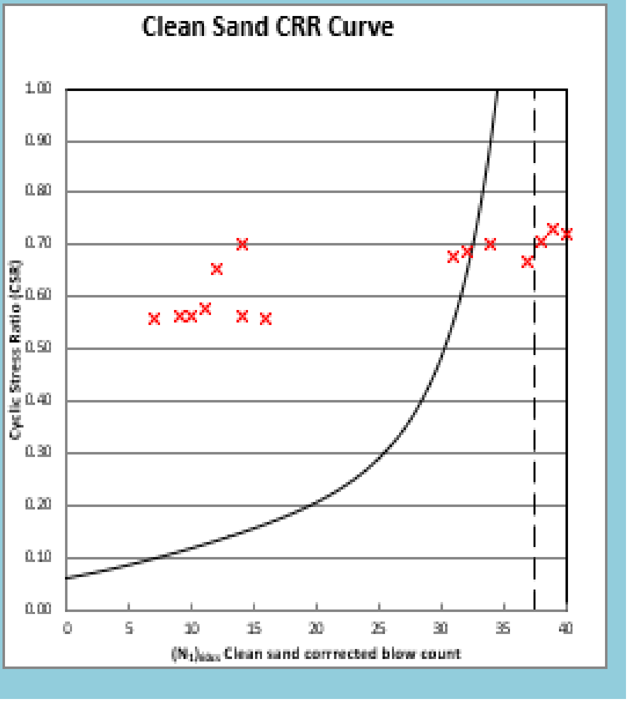
Boring Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Grade Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Boring E3 (Bent 2)

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LIQUEFACTION ANALYSIS RESULTS

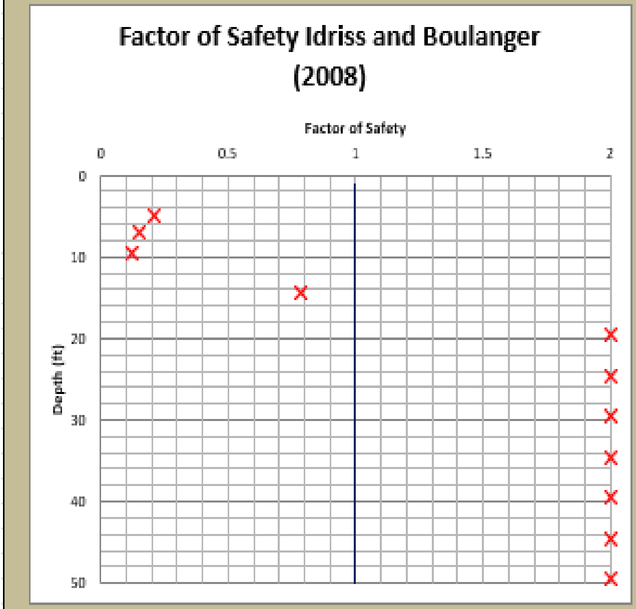
101124 Hwy. 135 over Right Hand Chute of Little River

Poinsett County, Arkansas

Job No. 23-031

Plate

Boring Elevation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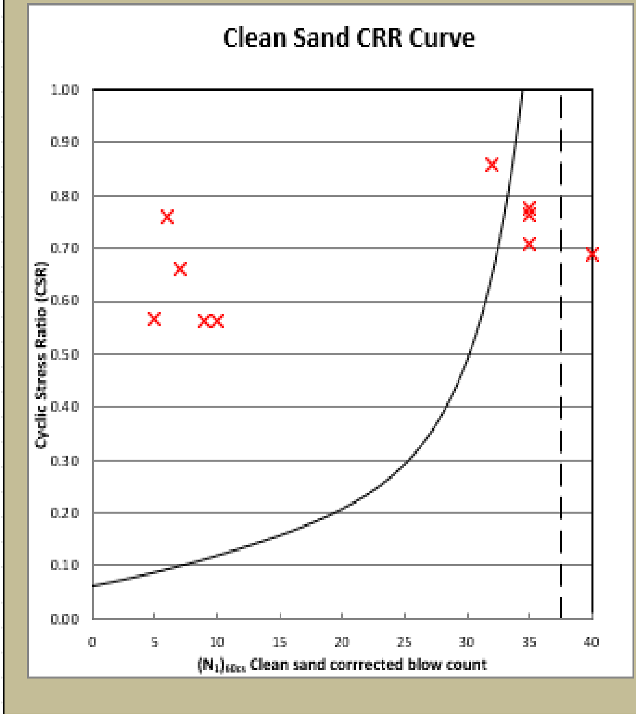
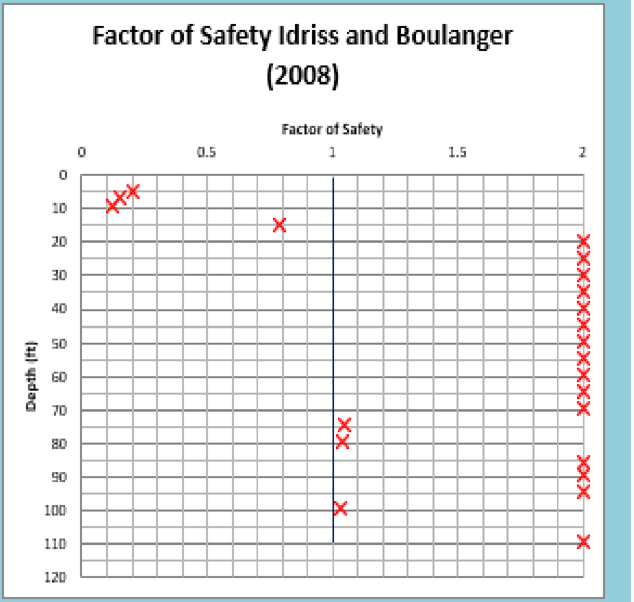
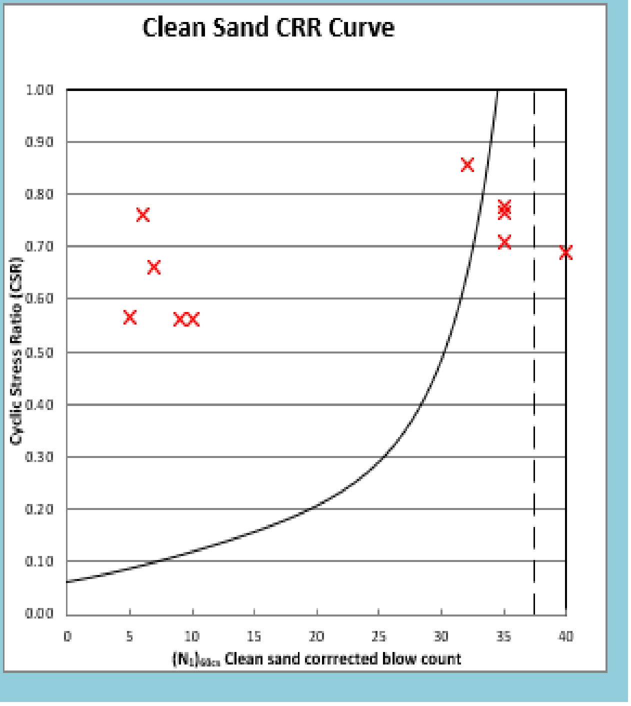

Clean Sand CRR Curve

Grade Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Boring E4 (Bent 3)

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LIQUEFACTION ANALYSIS RESULTS

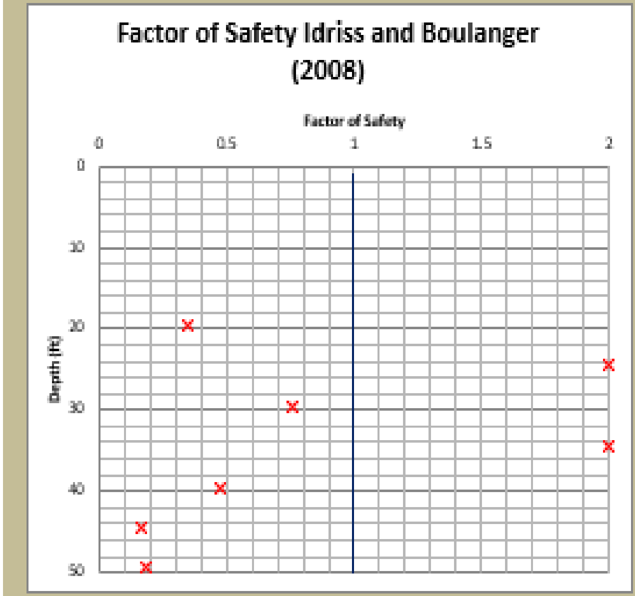
101124 Hwy. 135 over Right Hand Chute of Little River

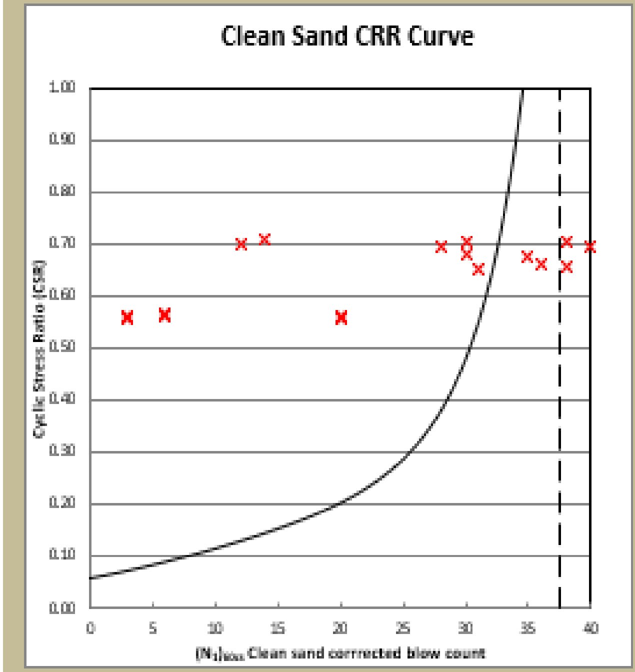
Poinsett County, Arkansas

Job No. 23-031

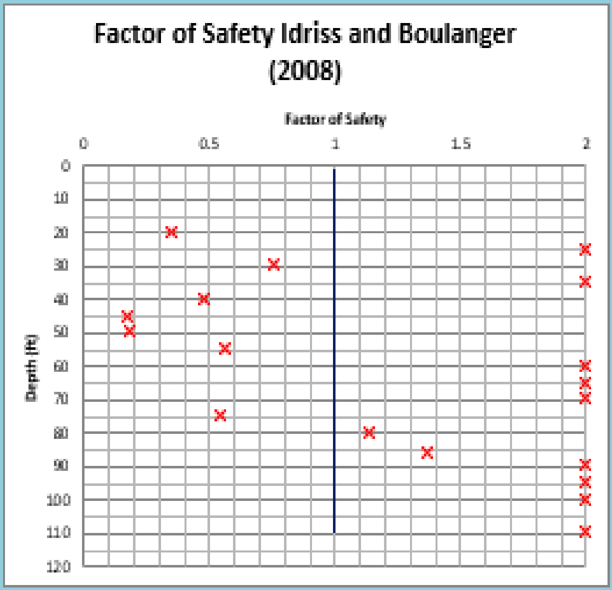
Plate

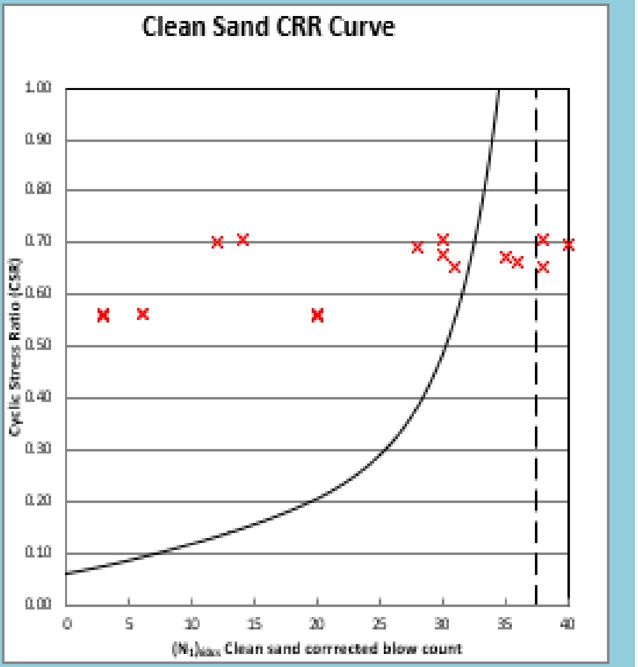
Boring Elevation





Grade Elevation





Boring E6

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A UES Company

LIQUEFACTION ANALYSIS RESULTS

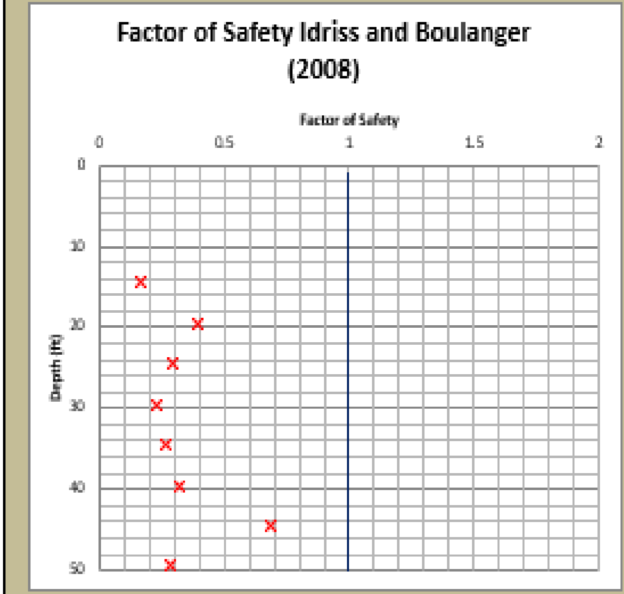
101124 Hwy. 135 over Right Hand Chute of Little River

Poinsett County, Arkansas

Job No. 23-031

Plate

Boring Elevation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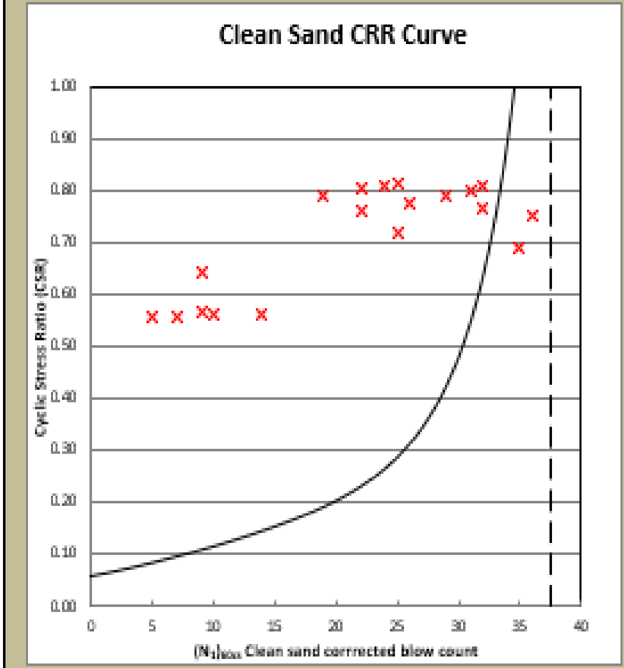
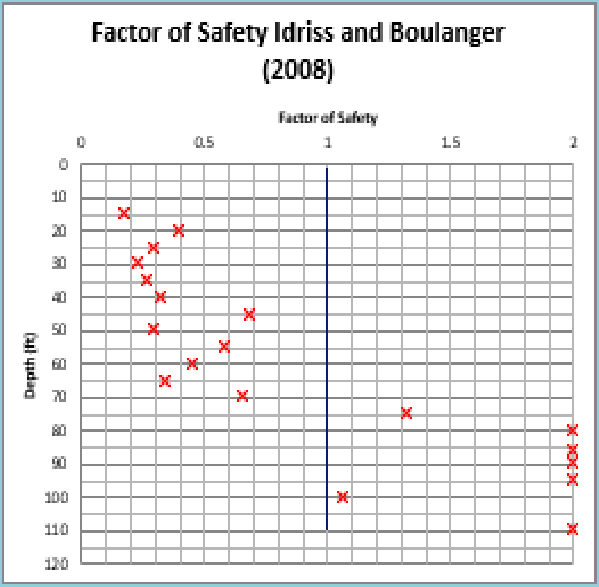

Clean Sand CRR Curve

Grade Elevation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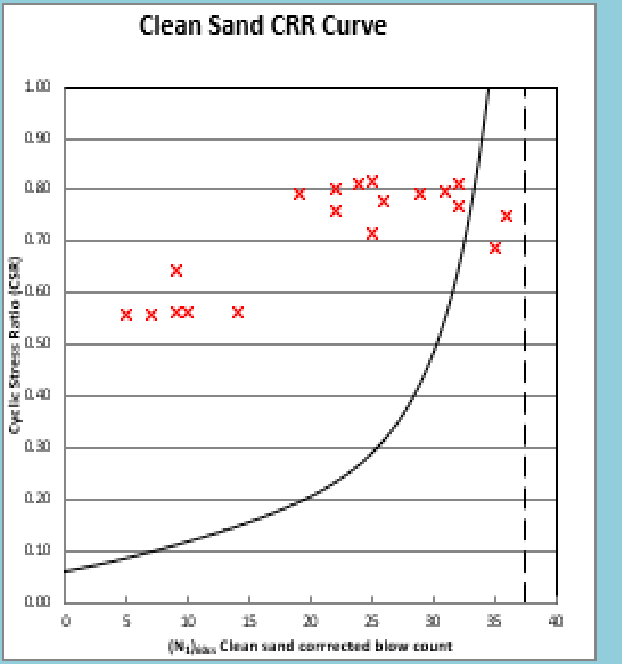
Clean Sand CRR Curve

Boring E7

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LIQUEFACTION ANALYSIS RESULTS

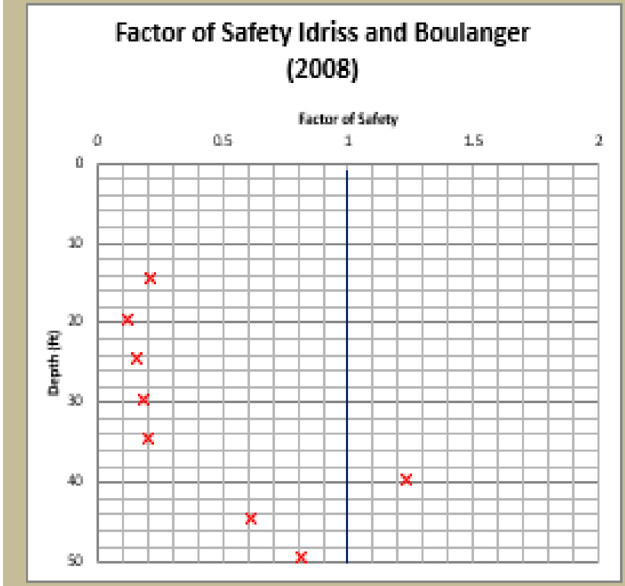
101124 Hwy. 135 over Right Hand Chute of Little River

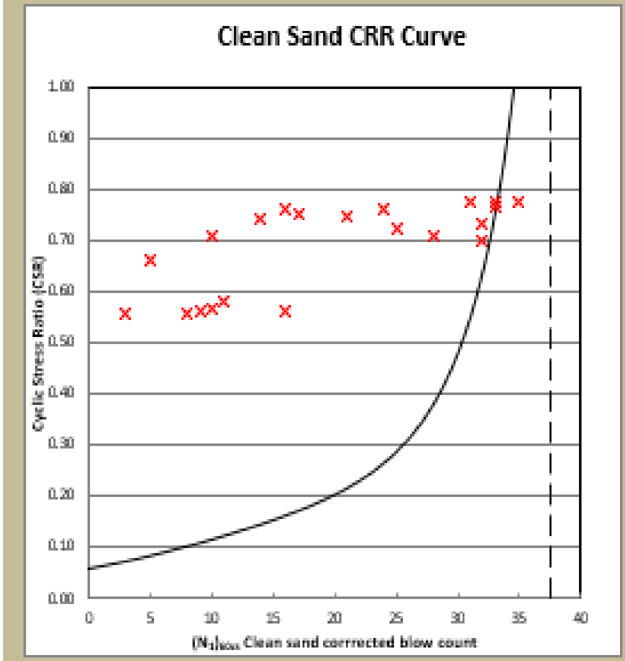
Poinsett County, Arkansas

Job No. 23-031

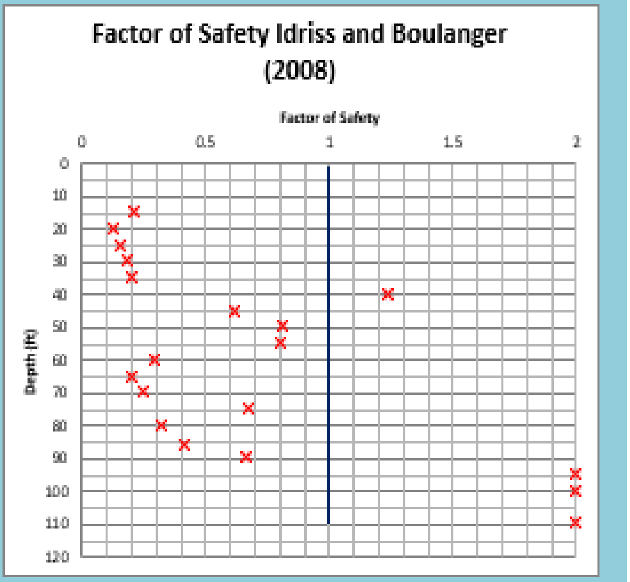
Plate

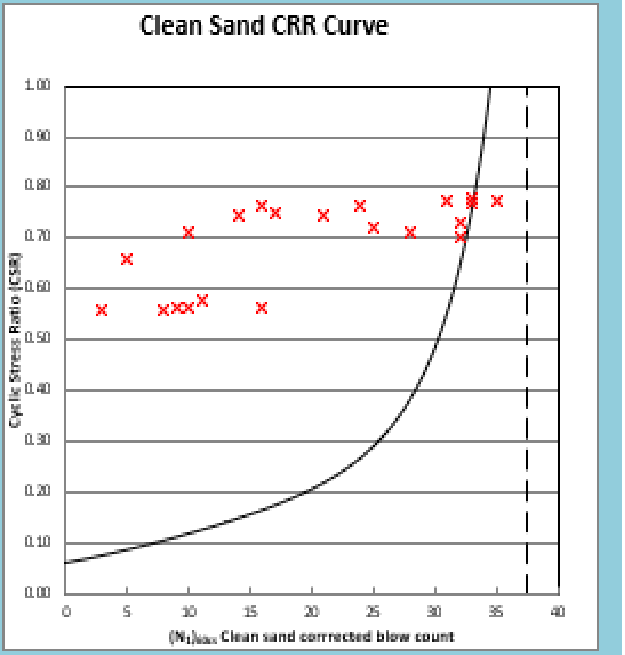
Boring Elevation





Grade Elevation





Boring E8

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LIQUEFACTION ANALYSIS RESULTS

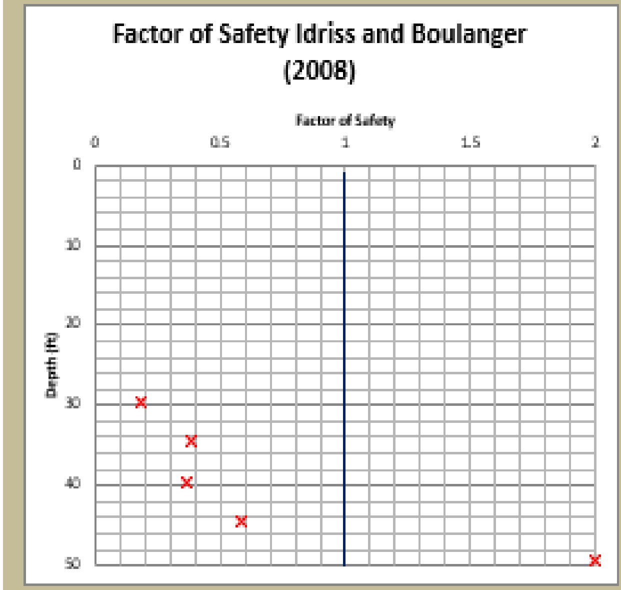
101124 Hwy. 135 over Right Hand Chute of Little River

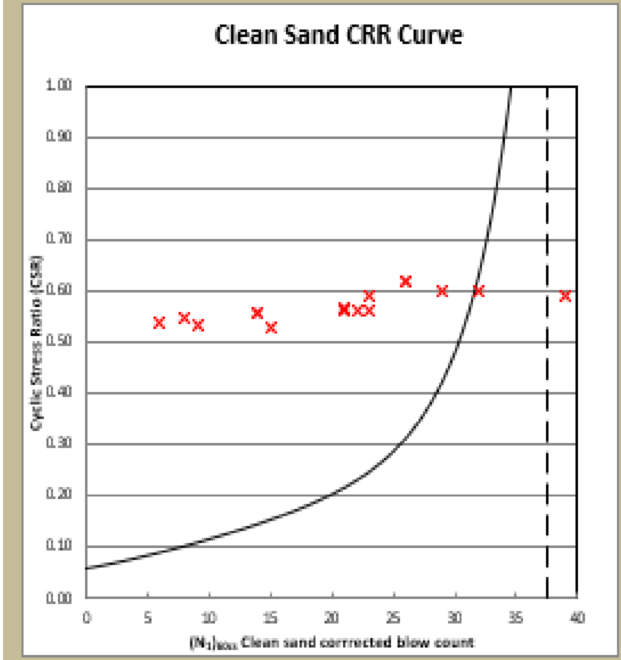
Poinsett County, Arkansas

Job No. 23-031

Plate

Boring Elevation


Factor of Safety Idriss and Boulanger (2008)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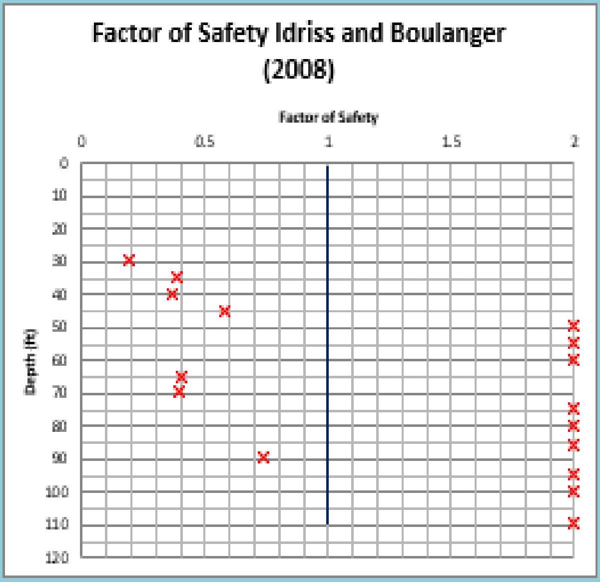
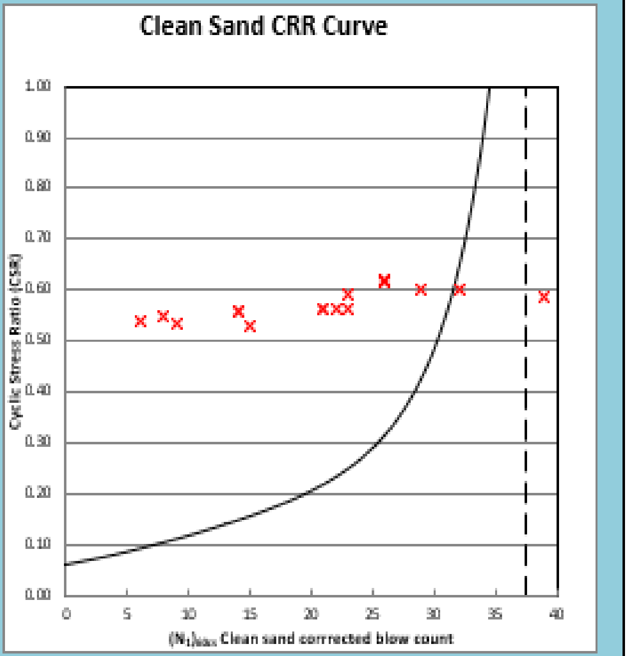
Grade Elevation


Factor of Safety Idriss and Boulanger (2008)


Clean Sand CRR Curve

Boring E9

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LIQUEFACTION ANALYSIS RESULTS

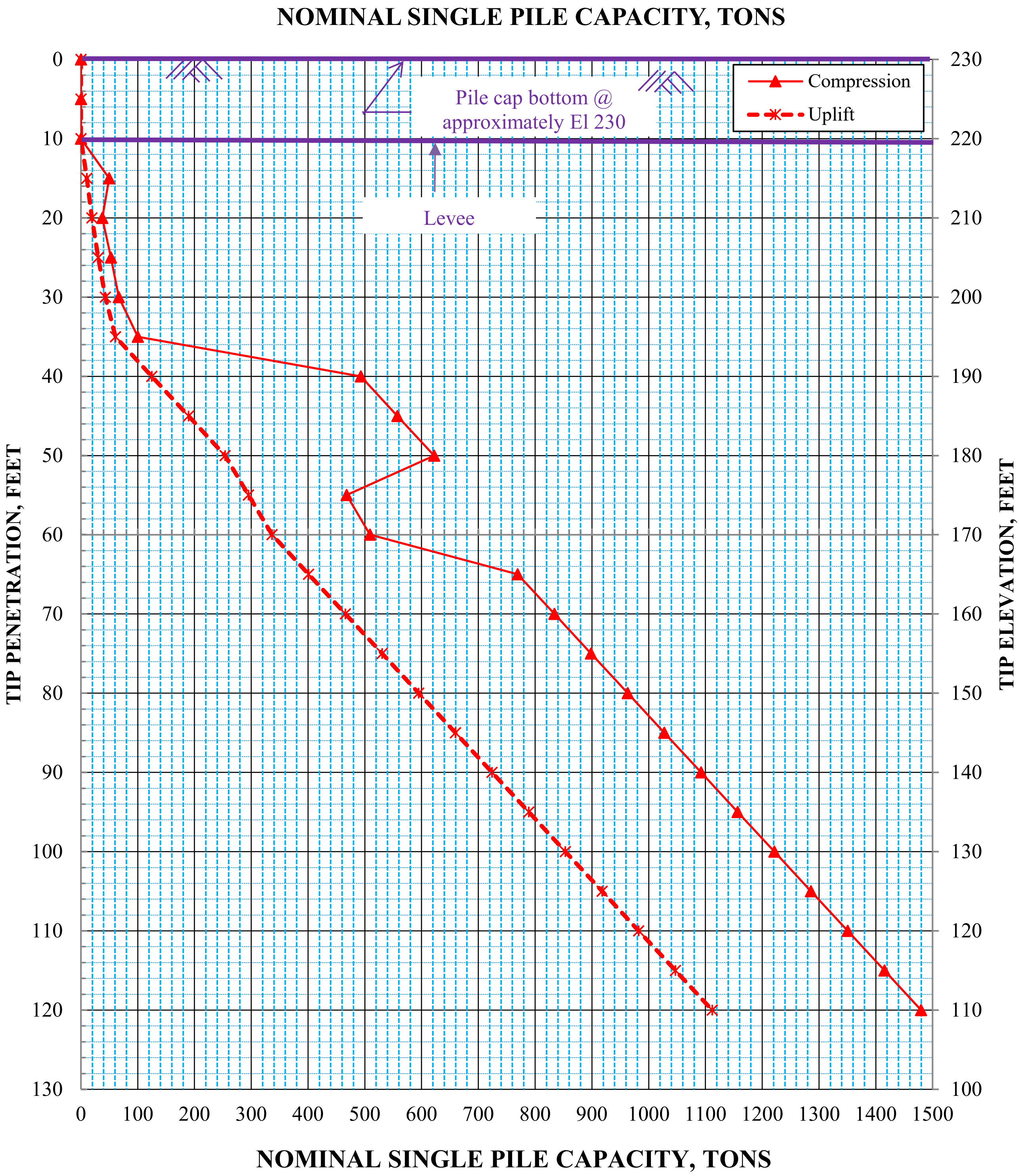
101124 Hwy. 135 over Right Hand Chute of Little River

Poinsett County, Arkansas

Job No. 23-031

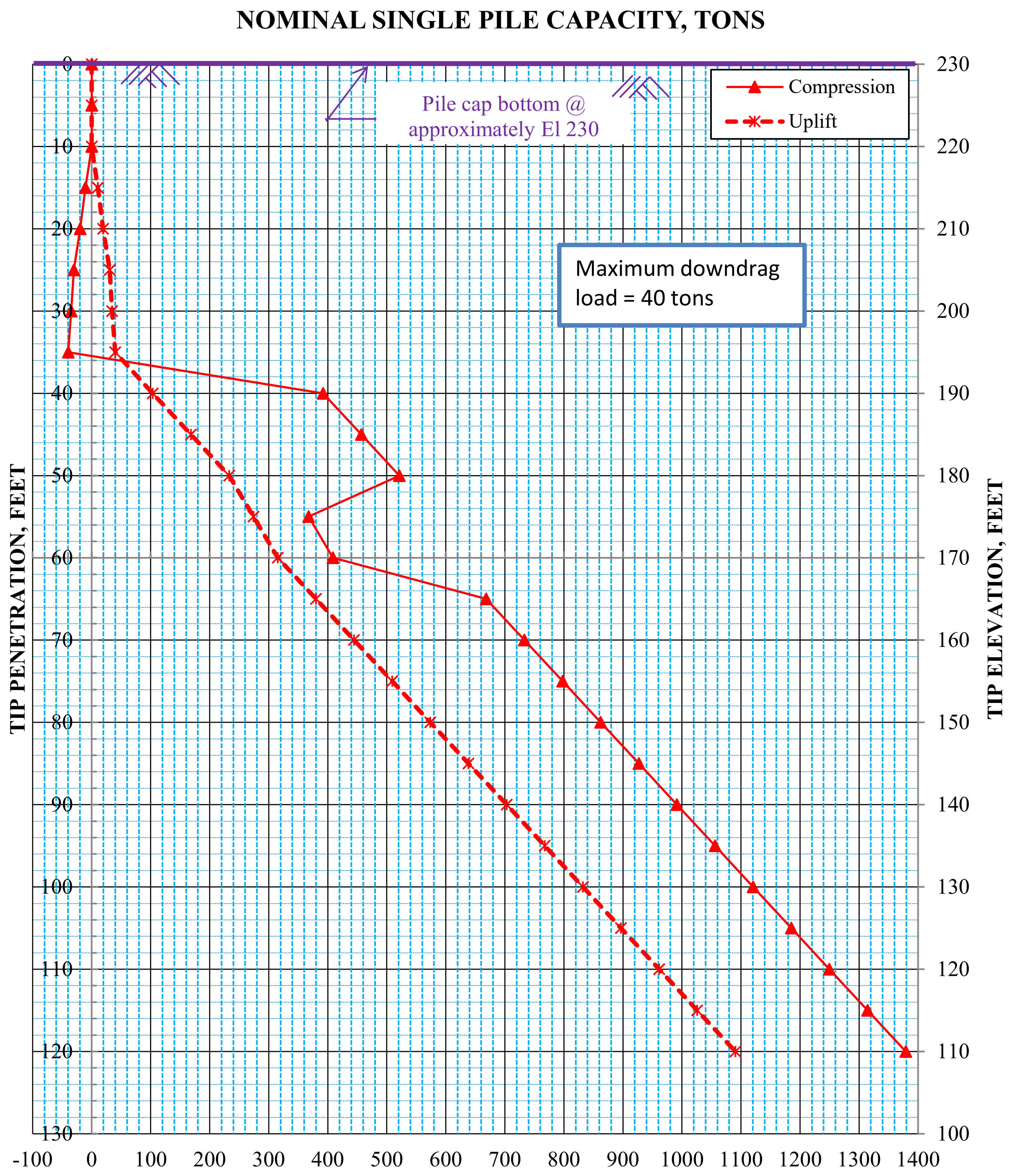
Plate

APPENDIX F



Bent 1 (South Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 195

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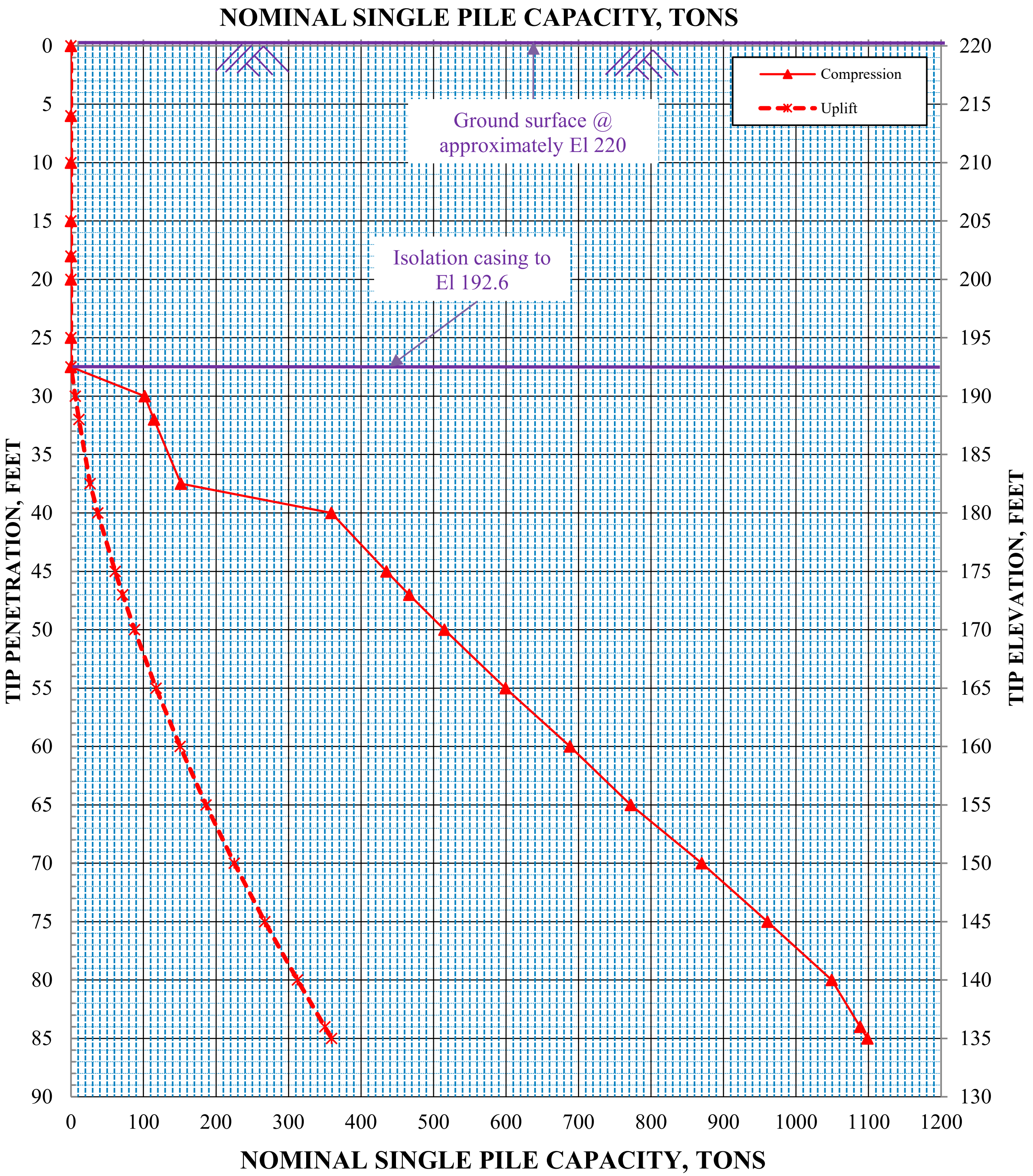
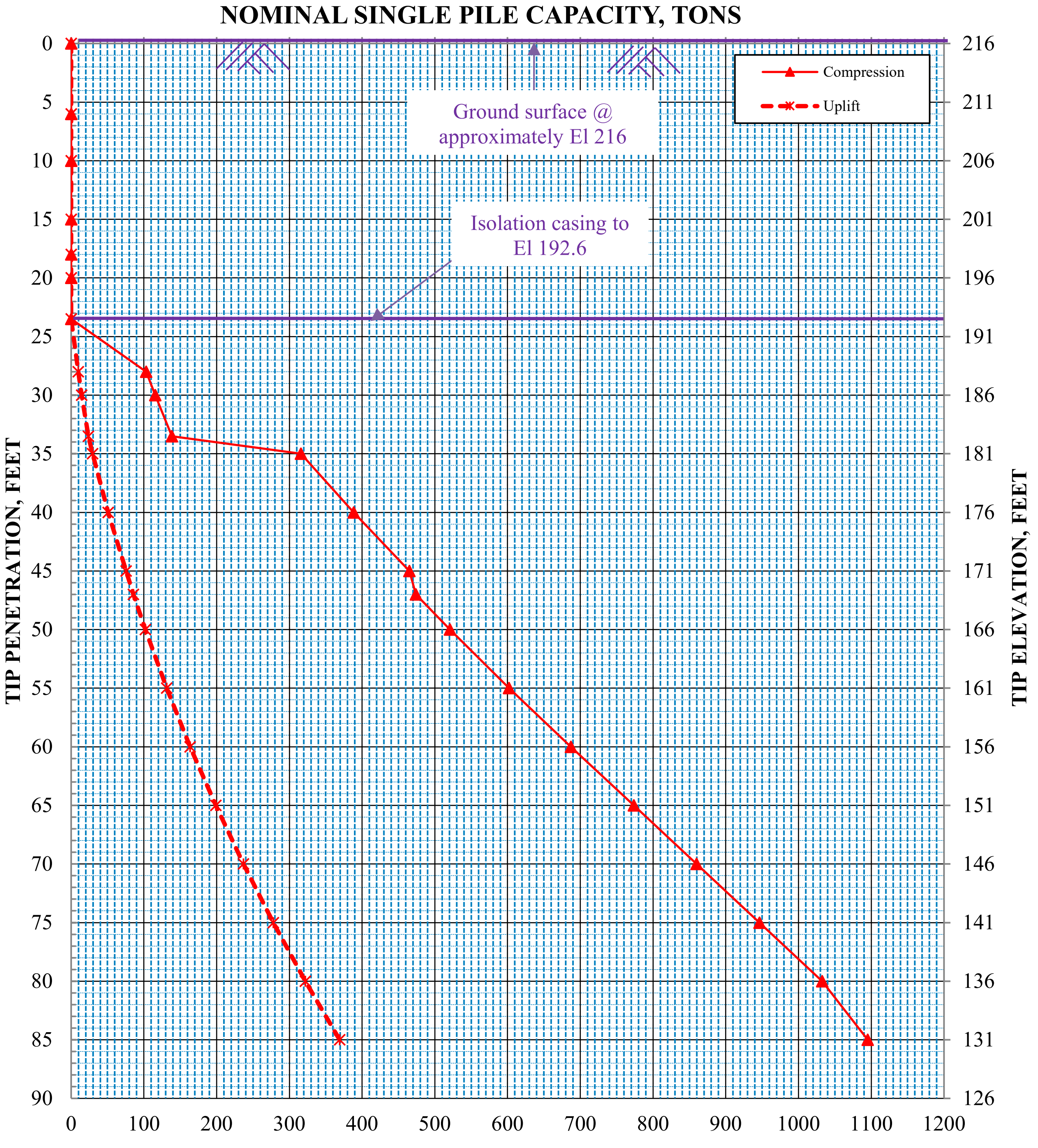


Bent 2 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

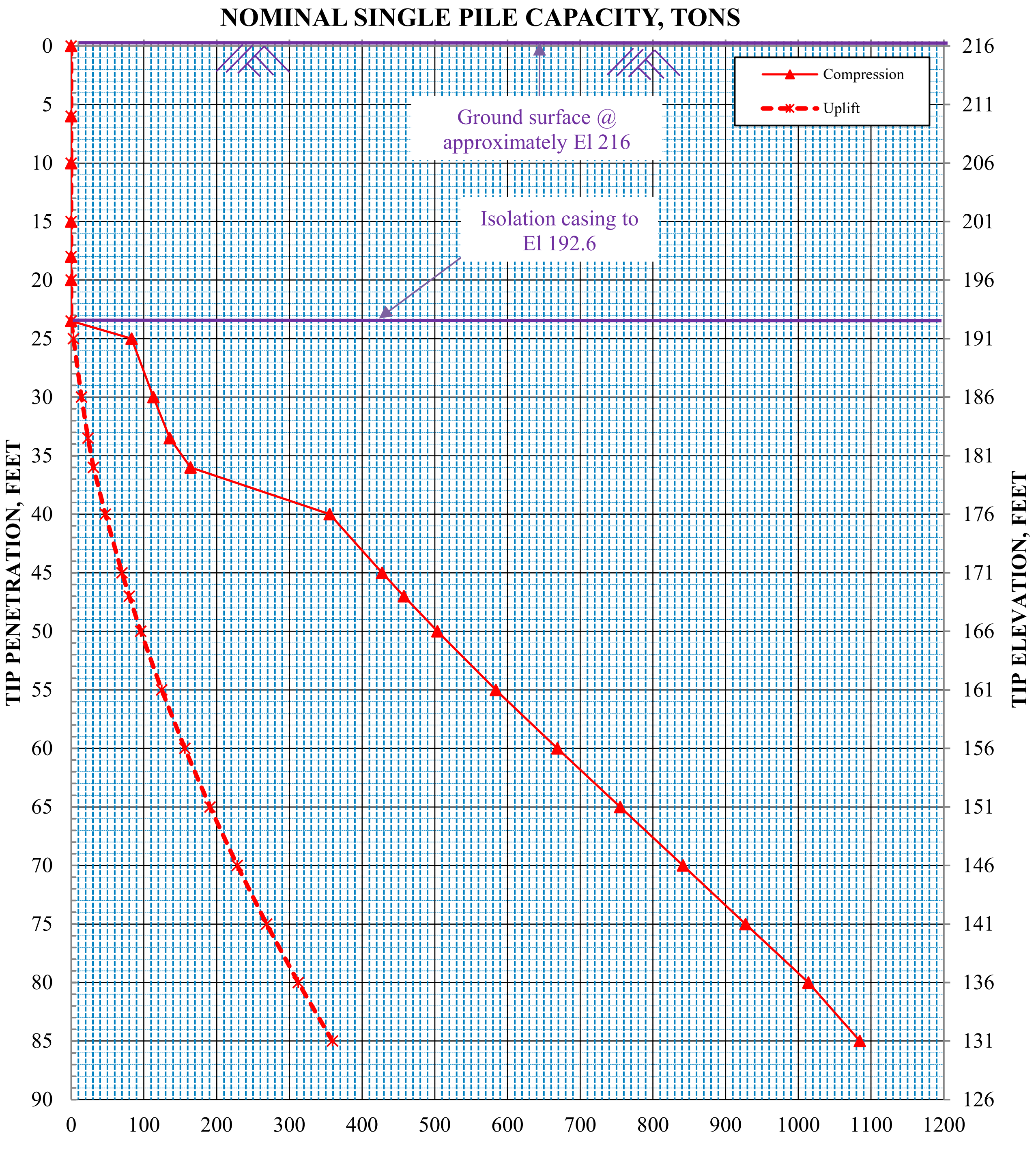
Notes: 1. Driven from channel bottom location
2. No downdrag - isolation casing extends below depth of liquefaction



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 3 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. No downdrag - isolation casing extends below depth of liquefaction



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location

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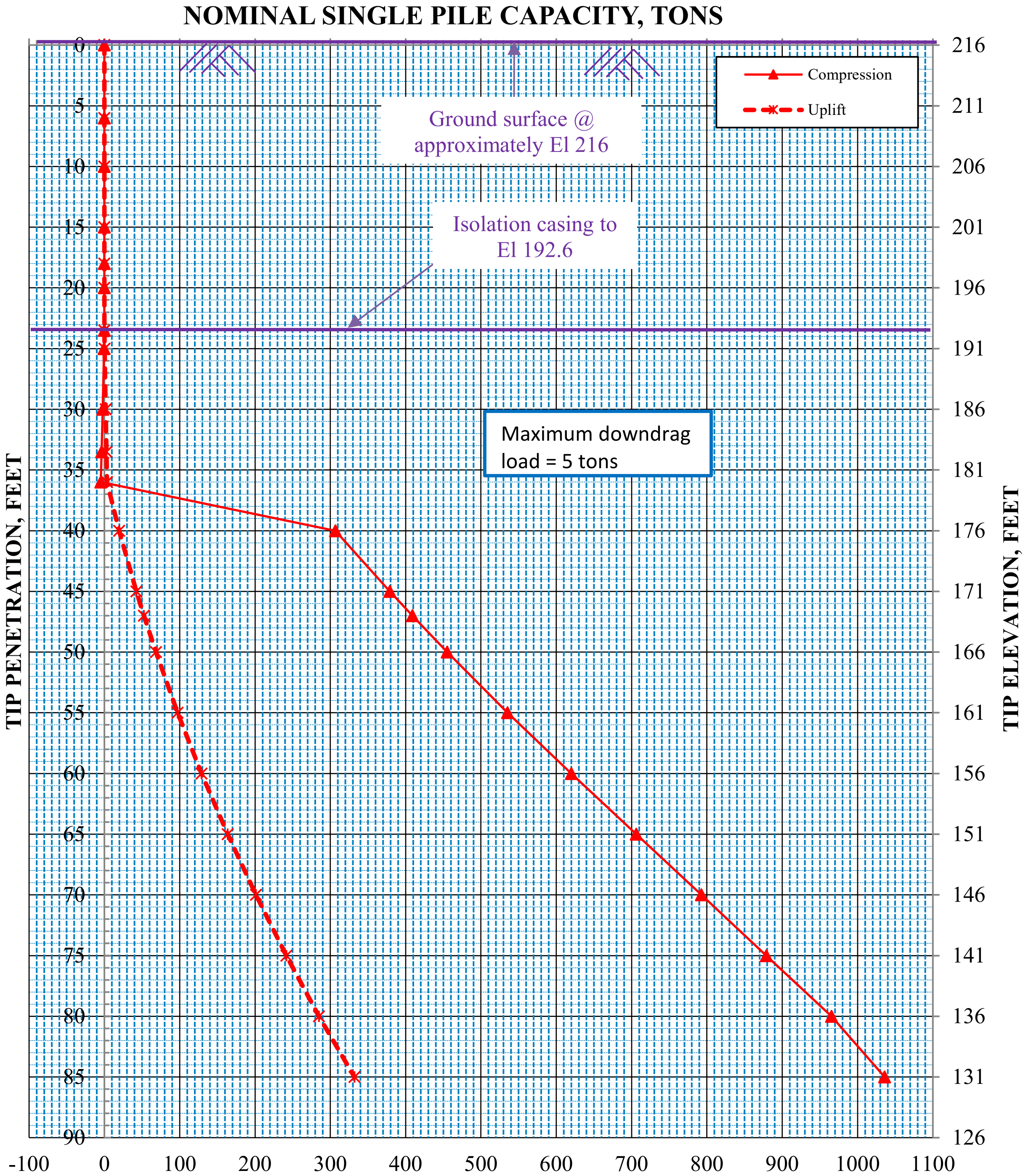

NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 180

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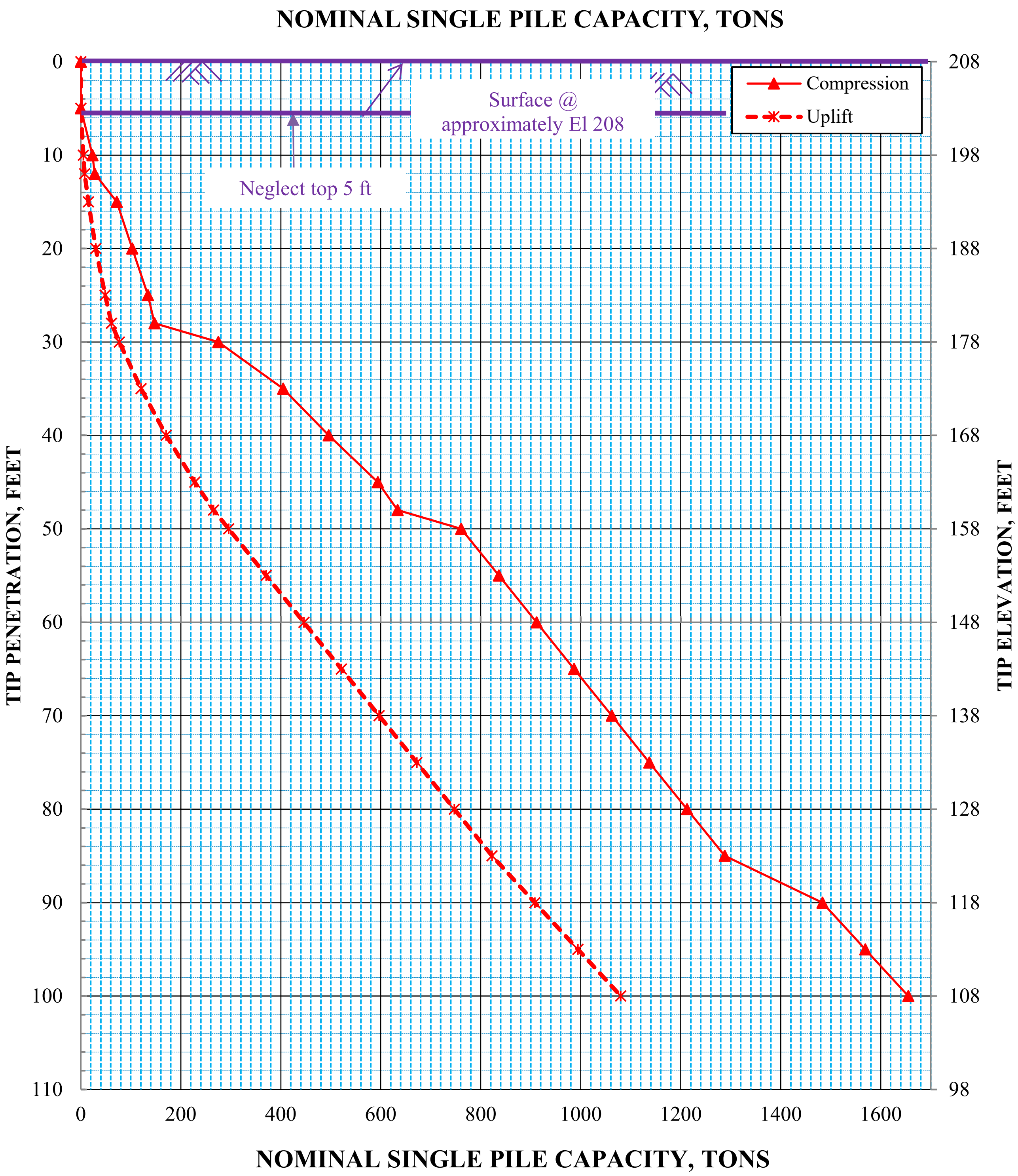

Bent 5 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. No downdrag

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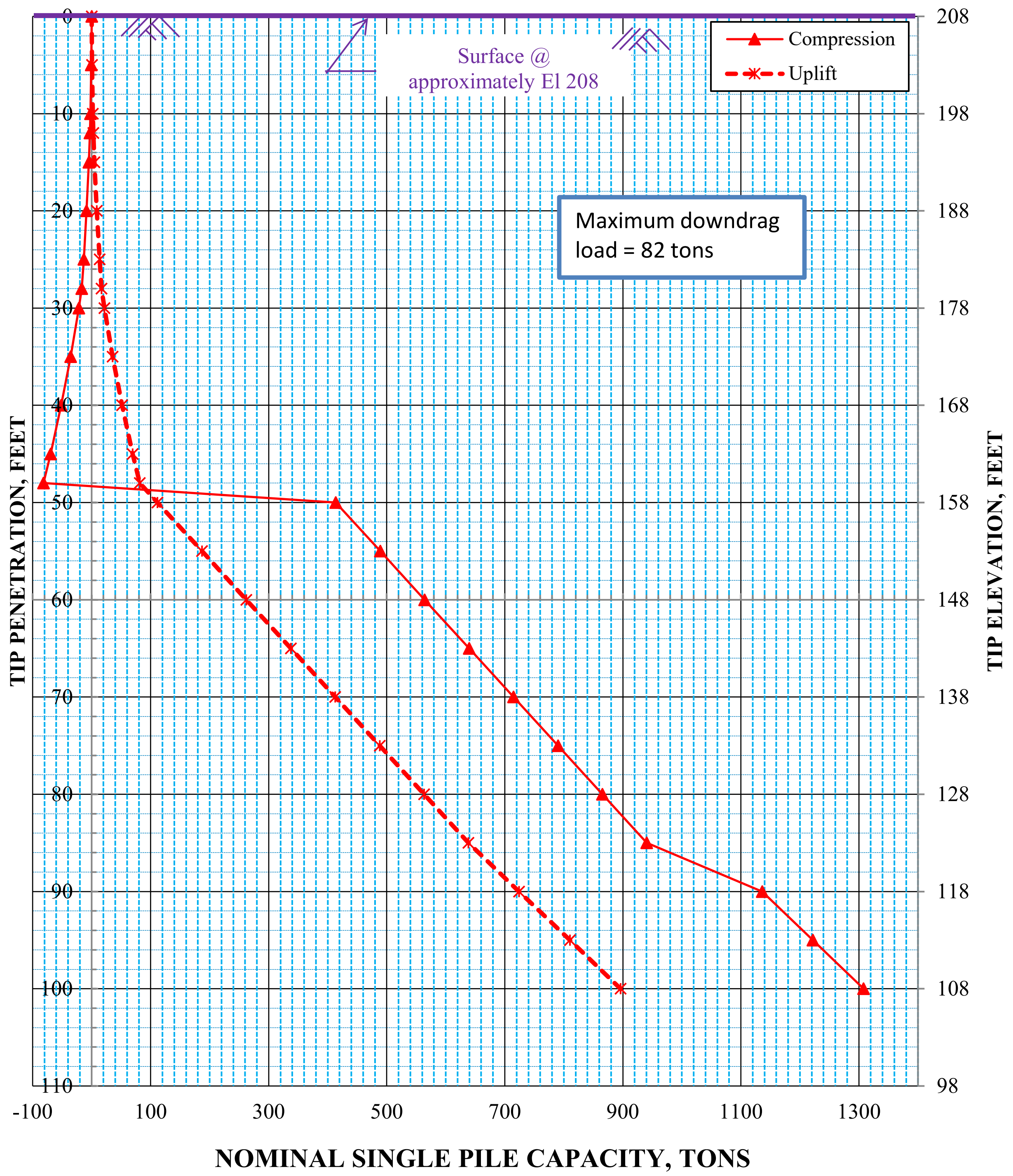
NOMINAL SINGLE PILE CAPACITY, TONS



Bent 5 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 160

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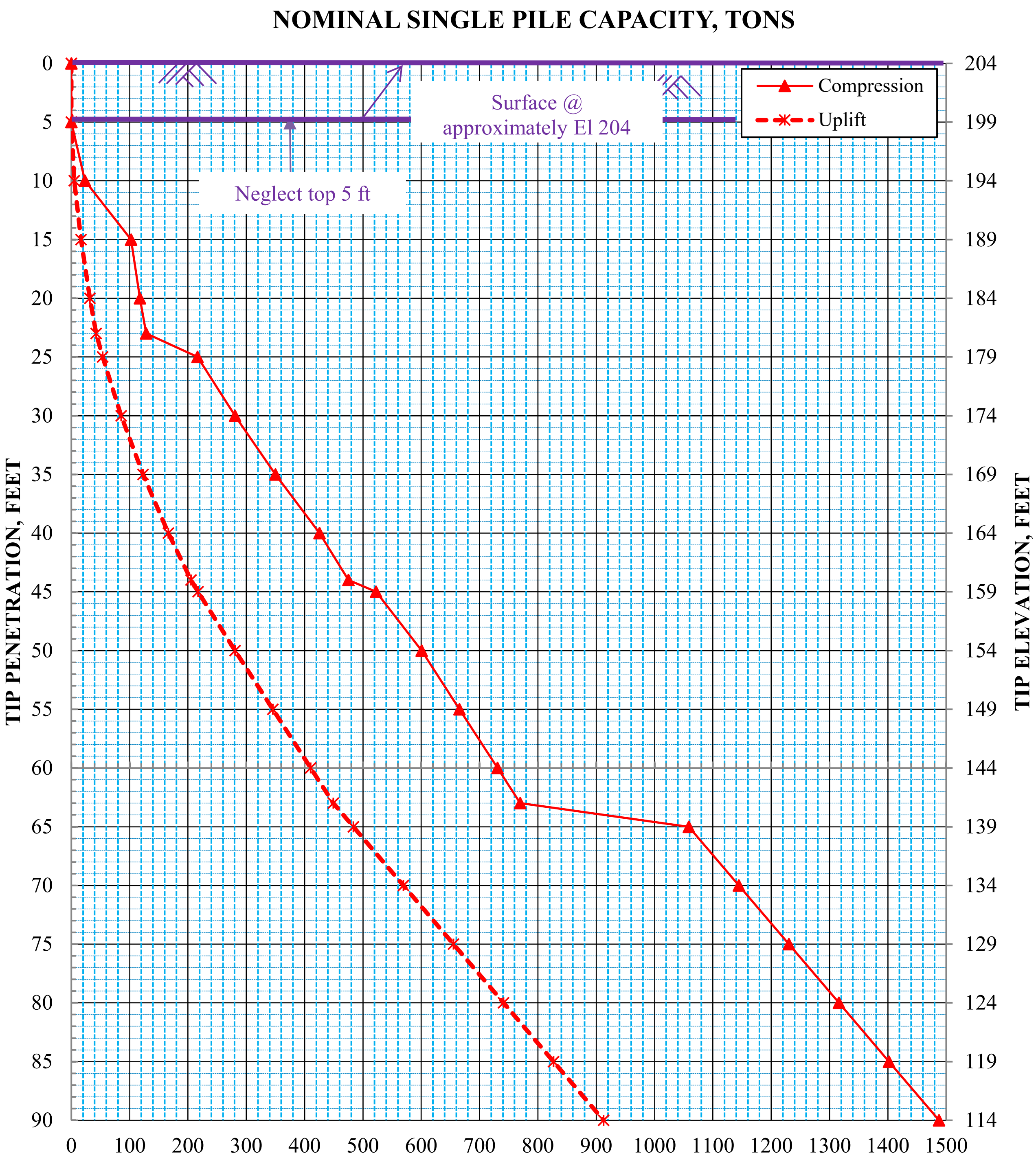
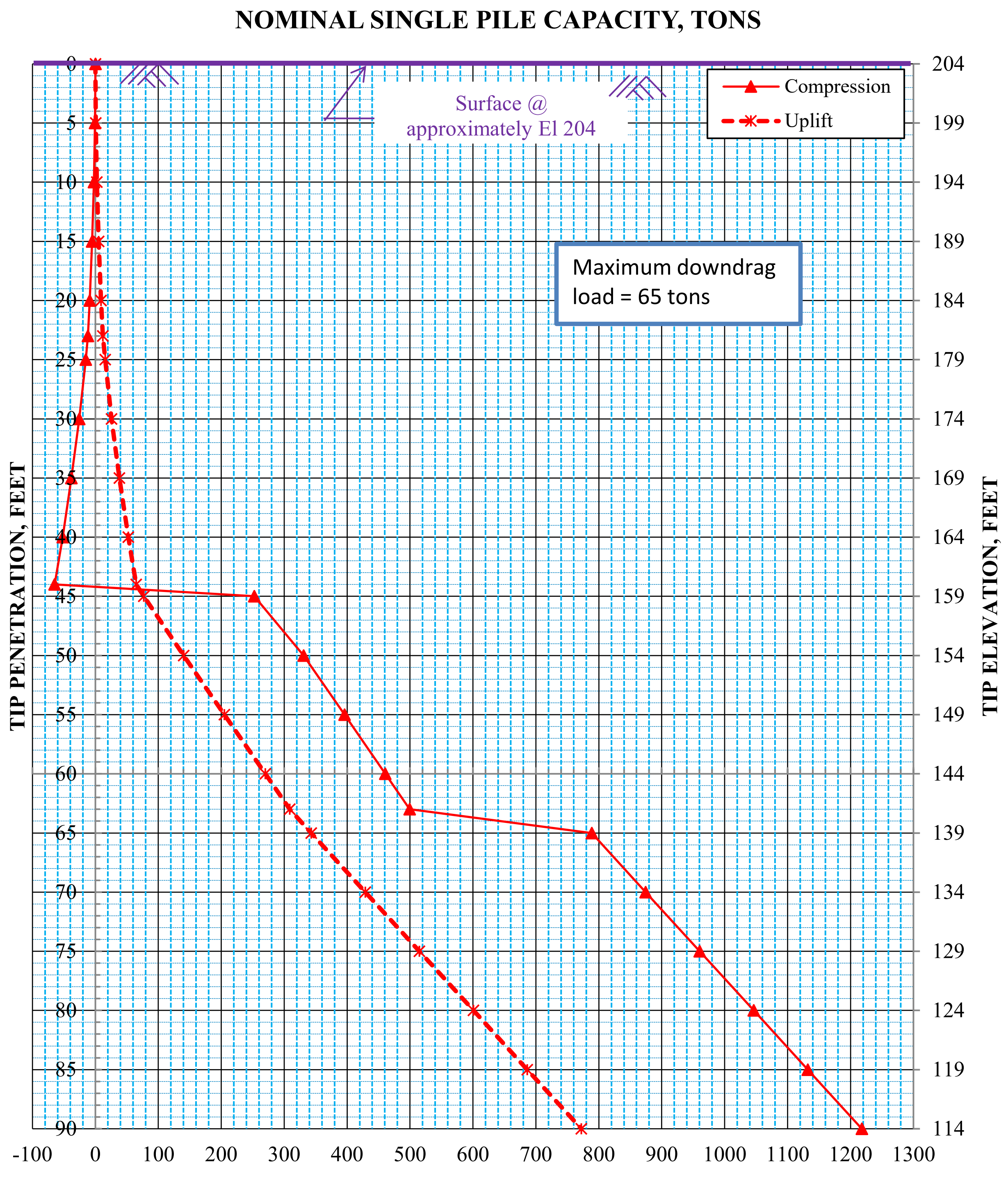


NOMINAL SINGLE PILE CAPACITY, TONS

Bent 6 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

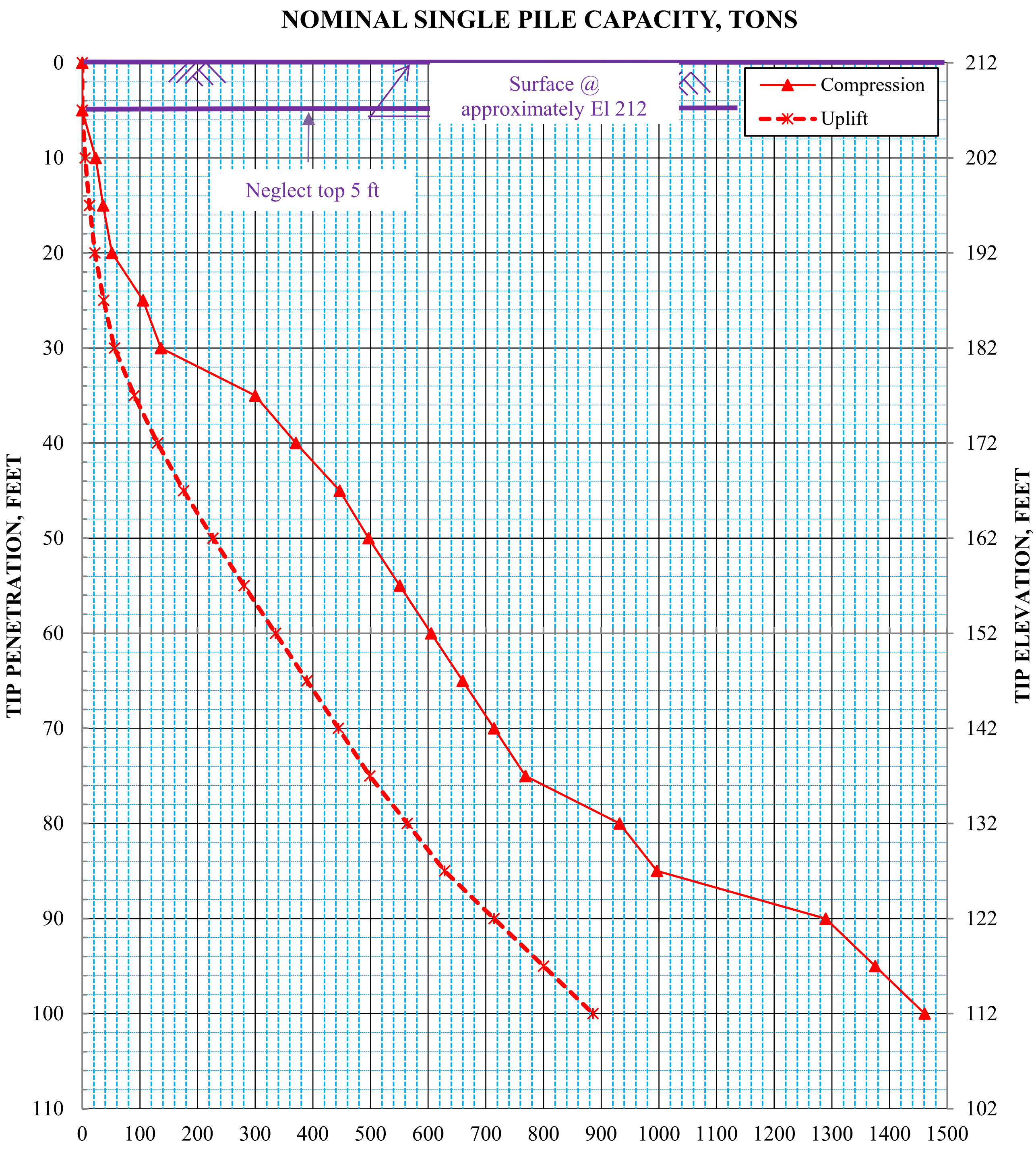
Notes: 1. Driven from channel bottom location
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 6 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 160



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 7 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. No downdrag

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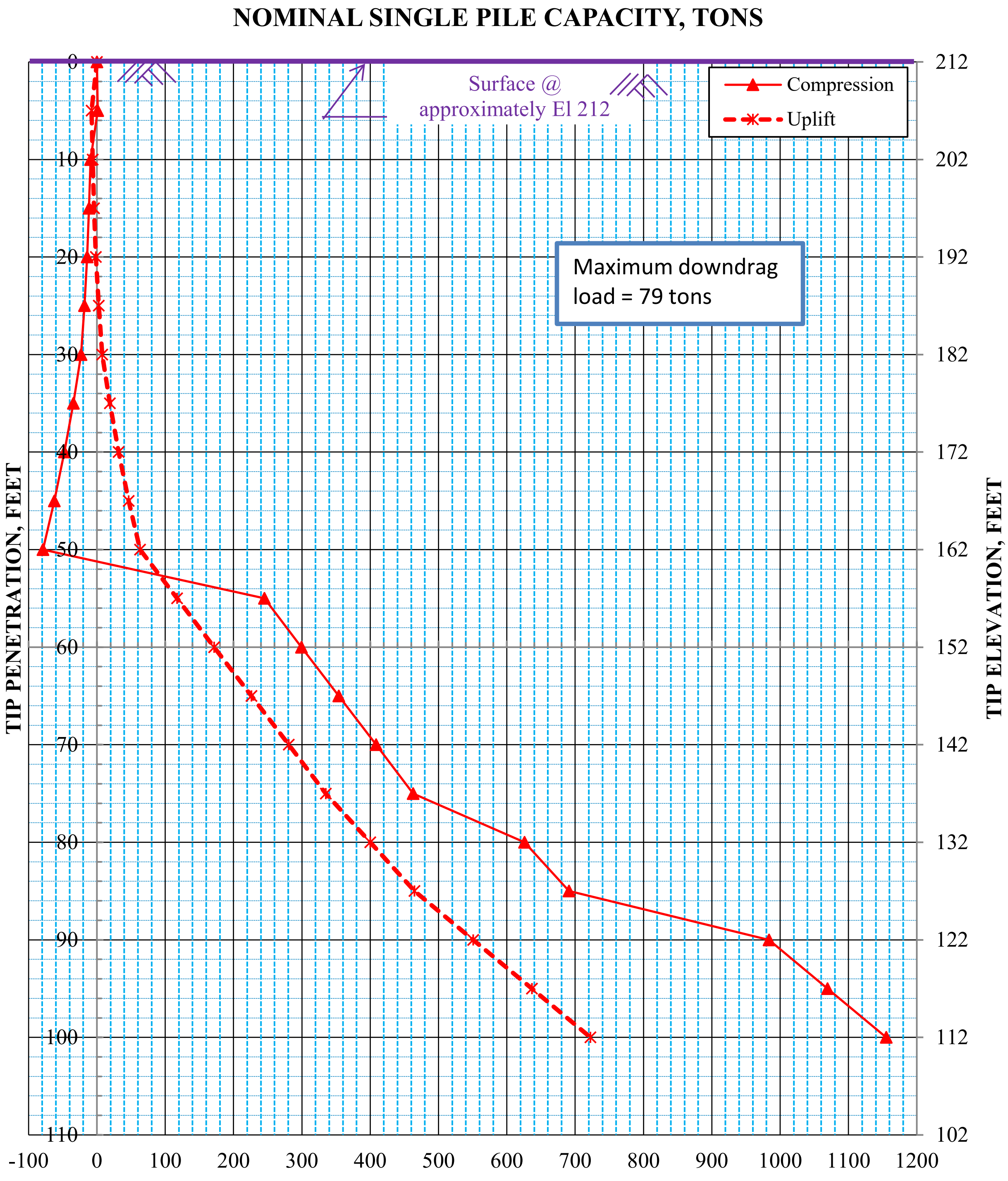


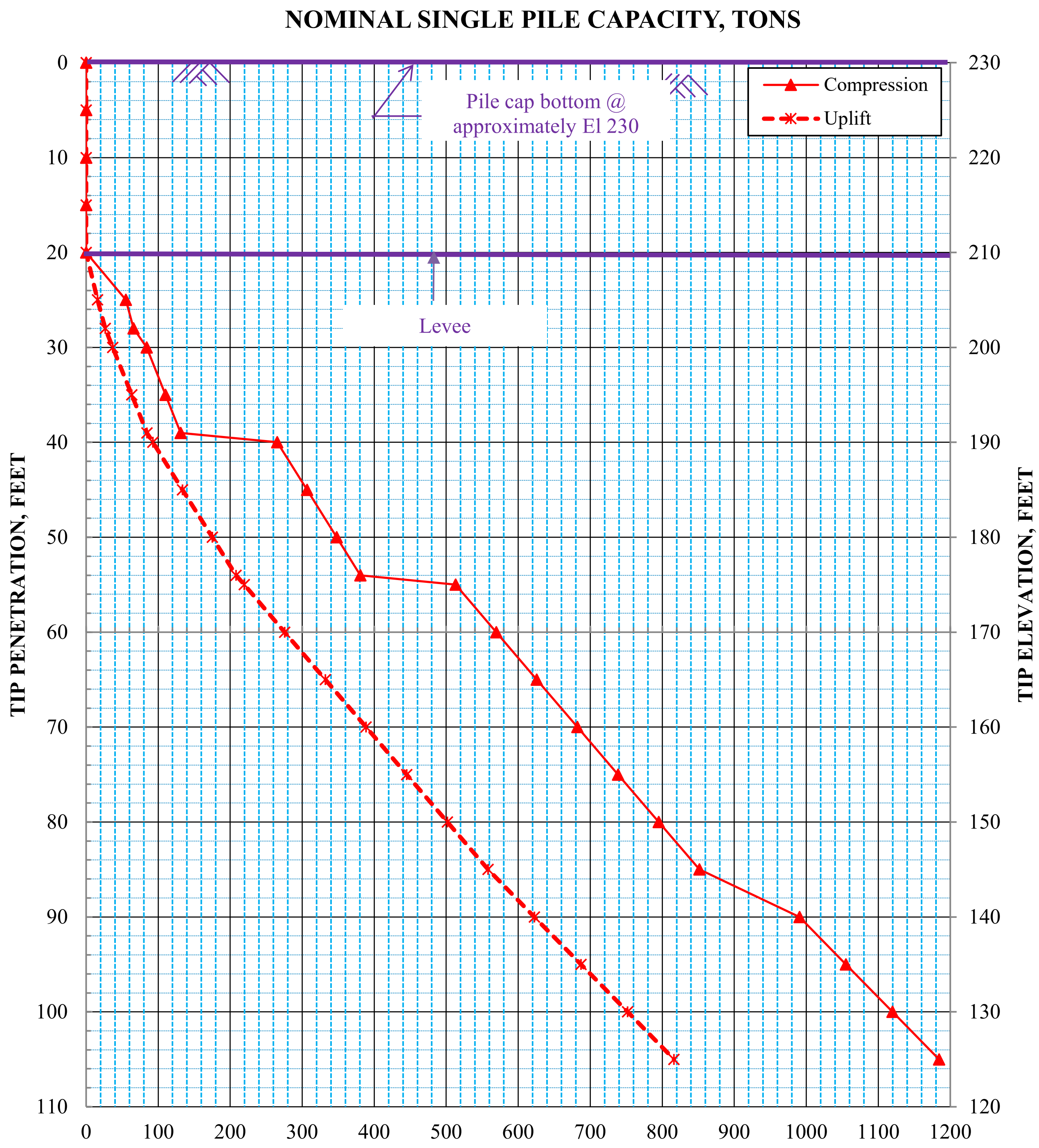
Surface @
approximately El 212

Maximum downdrag load = 79 tons

NOMINAL SINGLE PILE CAPACITY, TONS

Bent 7 (Intermediate Bent)
30-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 162



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 8 (North Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag

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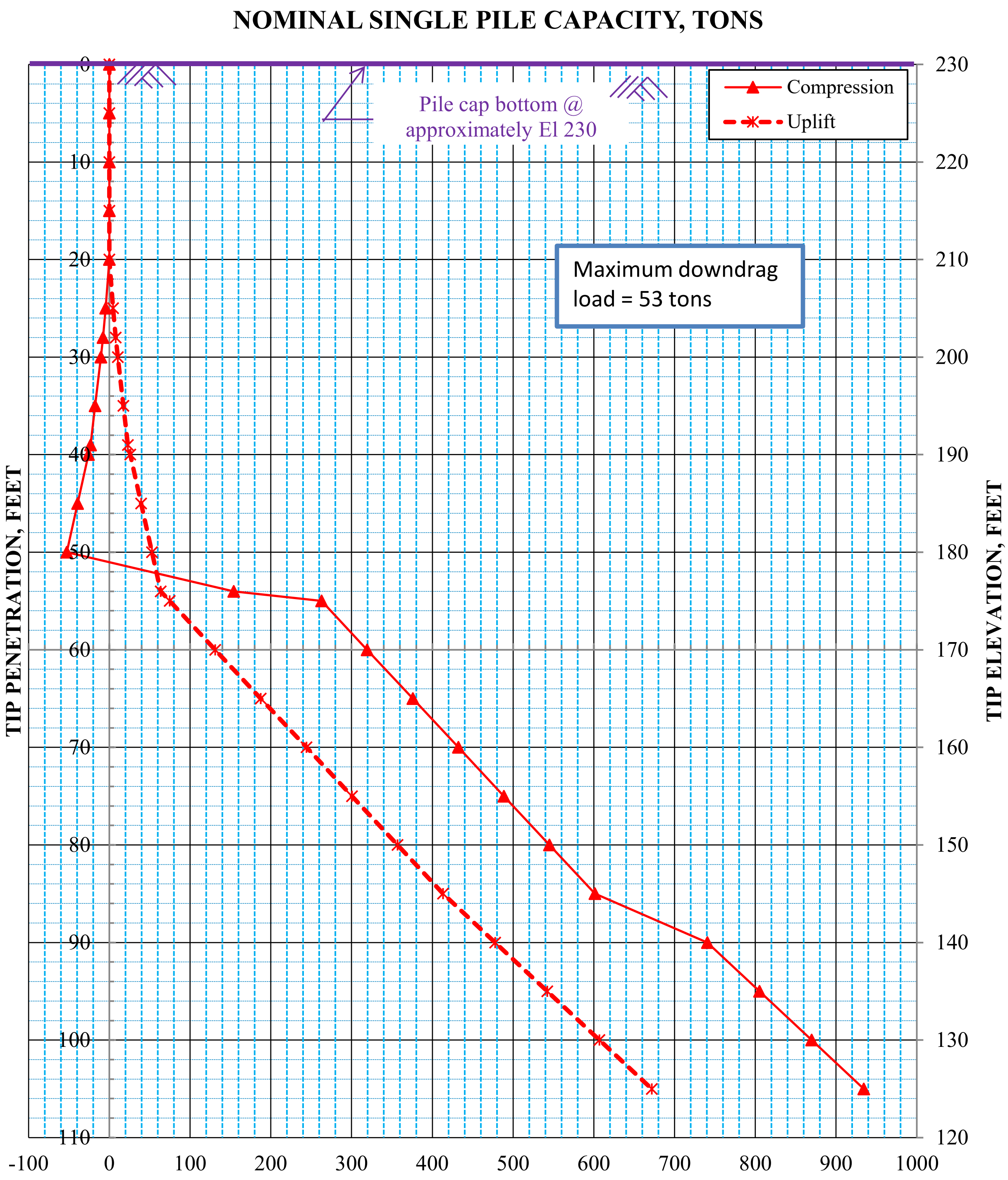

NOMINAL SINGLE PILE CAPACITY, TONS

Bent 8 (North Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 180

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A UES Company

APPENDIX G

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Levee - Neglect**	**Loose to medium dense silty fine SAND**	**Very loose clayey fine SAND**	**Loose clayey fine SAND**	**Dense fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-15	15-30	30-35	35-50	50-60	60 and deeper
Approximate El, ft	230-220	220-215	215-200	200-195	195-180	180-170	below 170
Recommend soil type	NA	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	115	90	48	68	60	68
Cohesion (c), lbs per sq ft	NA	0	0	0	0	0	0
Angle of internal friction (φ), °	NA	30	25	28	38	35	38
Subgrade modulus (k), lbs per cu in.	NA	45	20	20	125	80	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA	NA	NA

Note: Pile cap at ±El 230

Seismic Loading with Liquefaction
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Levee - Neglect**	**Loose to medium dense silty fine SAND**	**Very loose clayey fine sand (liquefiable)**	**Loose clayey fine sand (liquefiable)**	**Dense fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-15	15-30	30-35	35-50	50-60	60 and deeper
Approximate El, ft	230-220	220-215	215-200	200-195	195-180	180-170	below 170
Recommend soil type	NA	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	115	90	48	68	60	68
Cohesion (c), lbs per sq ft	NA	0	0	0	0	0	0
Angle of internal friction (φ), °	NA	30	8	8	38	35	38
Subgrade modulus (k), lbs per cu in.	NA	45	20	20	125	80	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA	NA	NA

Note: Pile cap at ±El 230

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Isolation casing**	**Medium dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-27.4	27.4-37.4	37.4 and deeper
Approximate El, ft	220-192.6	192.6-182.6	below 182.6
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	65	57	68
Cohesion (c), lbs per sq ft	0	0	0
Angle of internal friction (φ), °	0	30	38
Subgrade modulus (k), lbs per cu in.	0	35	125
Strain at 50% (EE50)	NA	NA	NA

Note: 1. Ground surface at ±El 220
2. No liquefaction - isolation casing extends below depth of liquefaction

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Isolation casing**	**Medium dense fine SAND**	**Dense to very dense fine SAND**
Depth below pile cap bottom, ft	0-23.4	23.4-33.4	33.4 and deeper
Approximate El, ft	216-192.6	192.6-182.6	below 182.6
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	65	57	68
Cohesion (c), lbs per sq ft	0	0	0
Angle of internal friction (φ), °	0	30	40
Subgrade modulus (k), lbs per cu in.	0	35	125
Strain at 50% (EE50)	NA	NA	NA

Note: 1. Ground surface at ±El 216
2. No liquefaction - isolation casing extends below depth of liquefaction

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SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Isolation casing**	**Medium dense fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-23.4	23.4-33.4	33.4-36	36 and deeper
Approximate El, ft	216-192.6	192.6-182.6	182.6-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	65	56	56	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	0	30	32	40
Subgrade modulus (k), lbs per cu in.	0	35	50	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: 1. Ground surface at ±El 216

Seismic Loading with Liquefaction

Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Isolation casing**	**Medium dense fine SAND (liquefiable)**	**Medium dense fine SAND (liquefiable)**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-23.4	23.4-33.4	33.4-36	36 and deeper
Approximate El, ft	216-192.6	192.6-182.6	182.6-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	65	56	56	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	0	8	8	40
Subgrade modulus (k), lbs per cu in.	0	20	20	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: 1. Ground surface at ±El 216

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 5: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND**	**Medium dense silty fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-12	12-28	28-48	48 and deeper
Approximate El, ft	208-196	196-180	180-160	below 160
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	56	63	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	28	32	36	37
Subgrade modulus (k), lbs per cu in.	20	50	105	115
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 208

Seismic Loading with Liquefaction

Bent 5: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND (liquefiable)**	**Medium dense silty fine SAND (liquefiable)**	**Medium dense fine SAND (liquefiable)**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-12	12-28	28-48	48 and deeper
Approximate El, ft	208-196	196-180	180-160	below 160
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	56	63	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	8	8	11	37
Subgrade modulus (k), lbs per cu in.	20	20	20	115
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 208

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 6: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND**	**Medium dense silty fine SAND**	**Medium dense fine SAND**	**Dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-23	23-44	44-63	63 and deeper
Approximate El, ft	204-194	194-181	181-160	160-141	below 141
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	56	60	63	68
Cohesion (c), lbs per sq ft	0	0	0	0	0
Angle of internal friction (φ), °	28	32	35	36	38
Subgrade modulus (k), lbs per cu in.	20	50	80	105	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

Note: Ground surface at ±El 204

Seismic Loading with Liquefaction

Bent 6: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND (liquefiable)**	**Medium dense silty fine SAND (liquefiable)**	**Medium dense fine SAND (liquefiable)**	**Dense fine SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-23	23-44	44-63	63 and deeper
Approximate El, ft	204-194	194-181	181-160	160-141	below 141
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	56	60	63	68
Cohesion (c), lbs per sq ft	0	0	0	0	0
Angle of internal friction (φ), °	8	8	11	36	38
Subgrade modulus (k), lbs per cu in.	20	20	20	105	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

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PLATE

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 7: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Very loose to loose silty fine sand**	**Medium dense fine SAND**	**Medium dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-30	30-85	85 and deeper
Approximate El, ft	212-192	192-182	182-127	below 127
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	54	60	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	28	31	35	38
Subgrade modulus (k), lbs per cu in.	20	40	80	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 212

Seismic Loading with Liquefaction
Bent 7: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Very loose to loose silty fine sand (liquefiable)**	**Medium dense fine SAND (liquefiable)**	**Medium dense fine to medium SAND (liquefiable)**	**Medium dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-30	30-50	50-85	85 and deeper
Approximate El, ft	212-192	192-182	182-162	162-127	below 127
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	48	54	60	60	68
Cohesion (c), lbs per sq ft	0	0	0	0	0
Angle of internal friction (φ), °	8	8	11	35	38
Subgrade modulus (k), lbs per cu in.	20	20	20	80	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

Note: Ground surface at ±El 212

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PLATE

SUMMARY OF LATERAL LOAD PARAMETERS

ARDOT 101124 Hwy. 135 over Right Hand Chute of Little River

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 8: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Levee - Neglect**	**Loose silty fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-28	28-39	39-54	54 and deeper
Approximate El, ft	230-210	210-202	202-191	191-176	below 176
Recommend soil type	NA	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	115	58	63	68
Cohesion (c), lbs per sq ft	NA	0	0	0	0
Angle of internal friction (φ), °	NA	28	31	35	38
Subgrade modulus (k), lbs per cu in.	NA	25	40	80	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

Note: Pile cap at ±El 230

Seismic Loading with Liquefaction

Bent 8: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Levee - Neglect**	**Loose silty fine SAND**	**Medium dense fine SAND (liquefiable)**	**Dense fine to medium SAND (liquefiable)**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-28	28-39	39-54	54 and deeper
Approximate El, ft	230-210	210-202	202-191	191-176	below 176
Recommend soil type	NA	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	115	58	63	68
Cohesion (c), lbs per sq ft	NA	0	0	0	0
Angle of internal friction (φ), °	NA	28	8	11	38
Subgrade modulus (k), lbs per cu in.	NA	25	20	20	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

Note: Pile cap at ±El 230

APPENDIX H

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Right Hand Chute of Little River
GHBW Job No. 23-031
Poinsett County, Arkansas

	Design Loading Condition	**Calculated Minimum Factor of Safety**
South End Slope (Bent 1) (2H:1V)	End of Construction	4.22
	Long Term	2.27
	Rapid Drawdown from El 231 to El 214	1.61
	Seismic ($k_h = A_S/2 = 0.432$)	1.06
South Side Slope (Bent 1) (2H:1V)	End of Construction	6.25
	Long Term	1.80
	Rapid Drawdown from El 231 to Existing Grade	1.74
	Seismic ($k_h = A_S/2 = 0.432$)	1.78
North End Slope (Bent 8) – with ground improvement (2H:1V)	End of Construction	3.41
	Long Term	2.24
	Rapid Drawdown from El 231 to El 214	2.45
	Seismic ($k_h = A_S/2 = 0.432$)	1.05
North Side Slope (Bent 8) (2H:1V)	End of Construction	3.37
	Long Term	1.97
	Rapid Drawdown from El 231 to Existing Grade	1.48
	Seismic ($k_h = A_S/2 = 0.432$)	1.10

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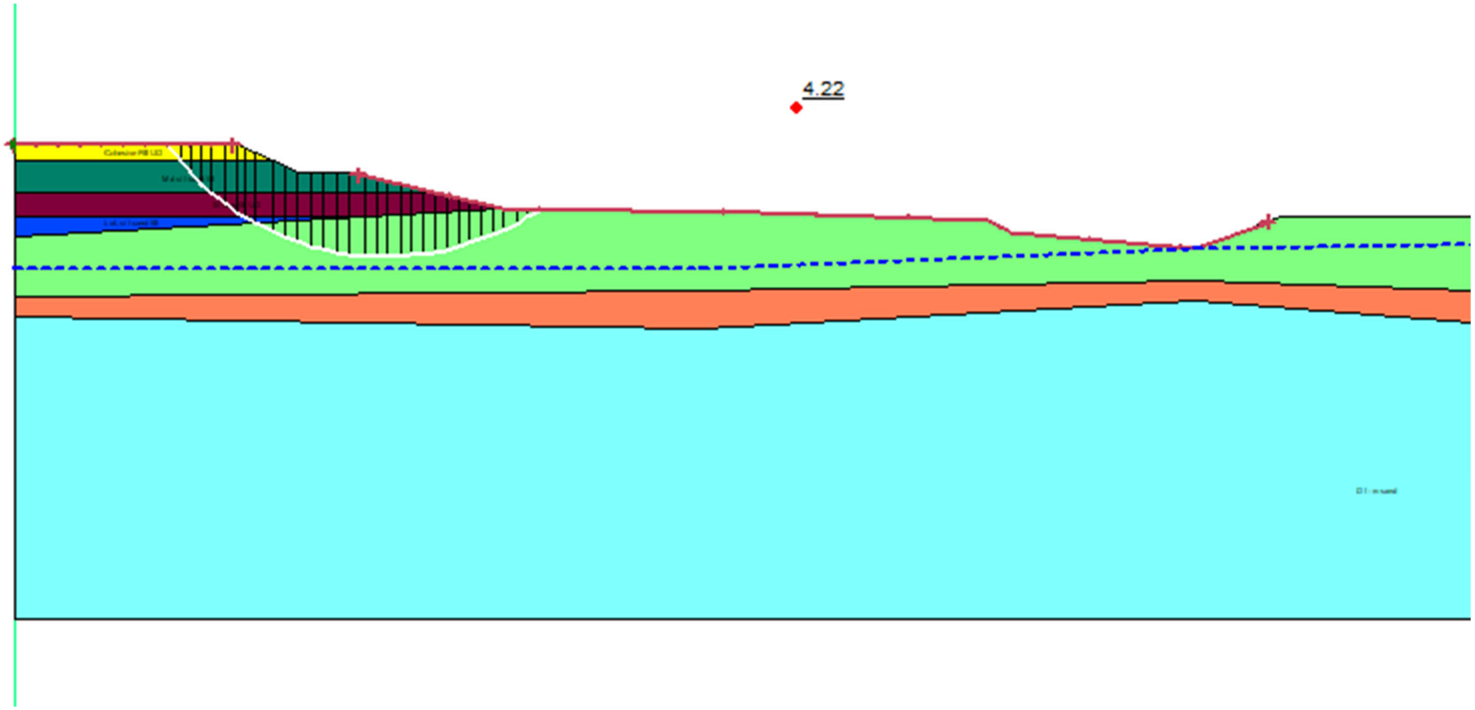
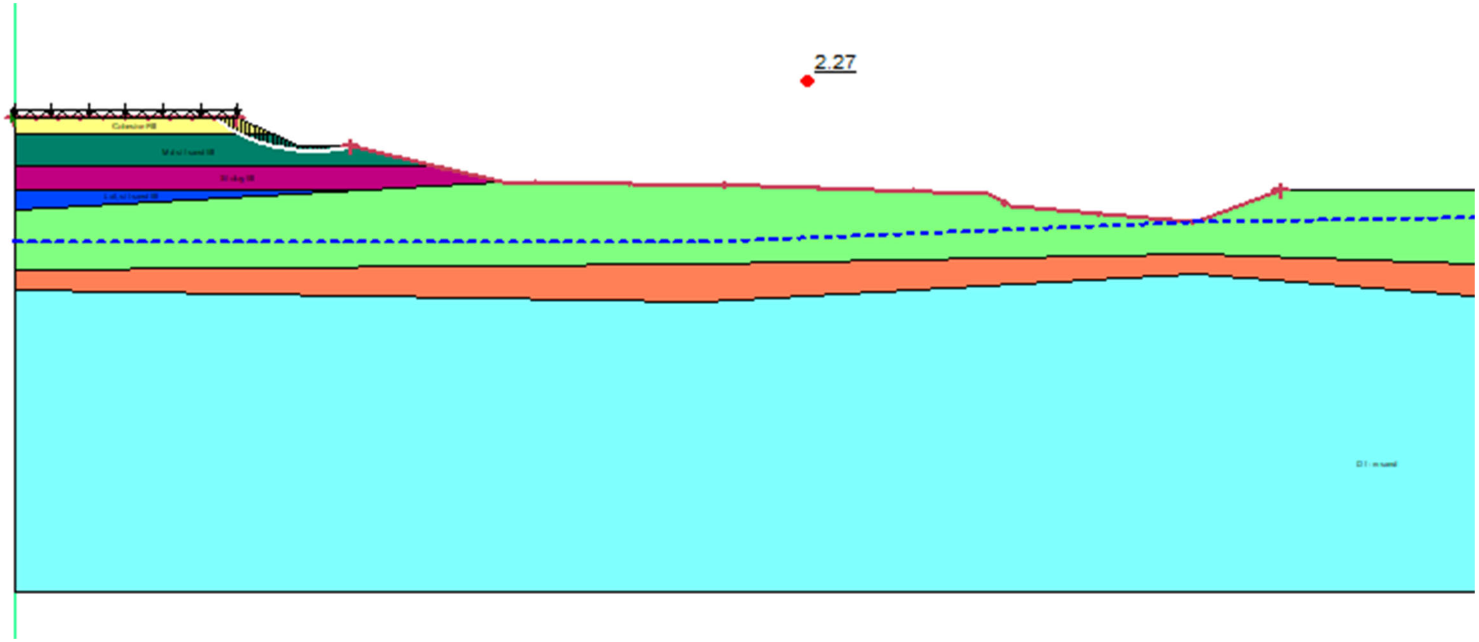


Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=16 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=16 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

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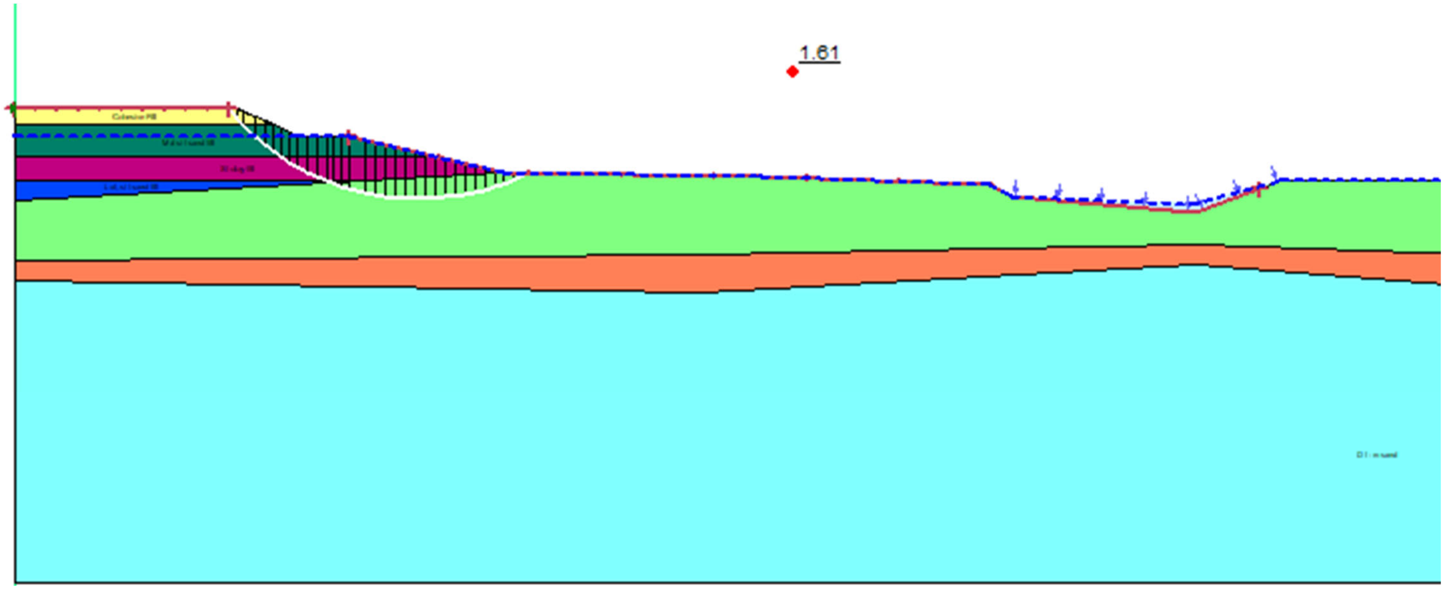
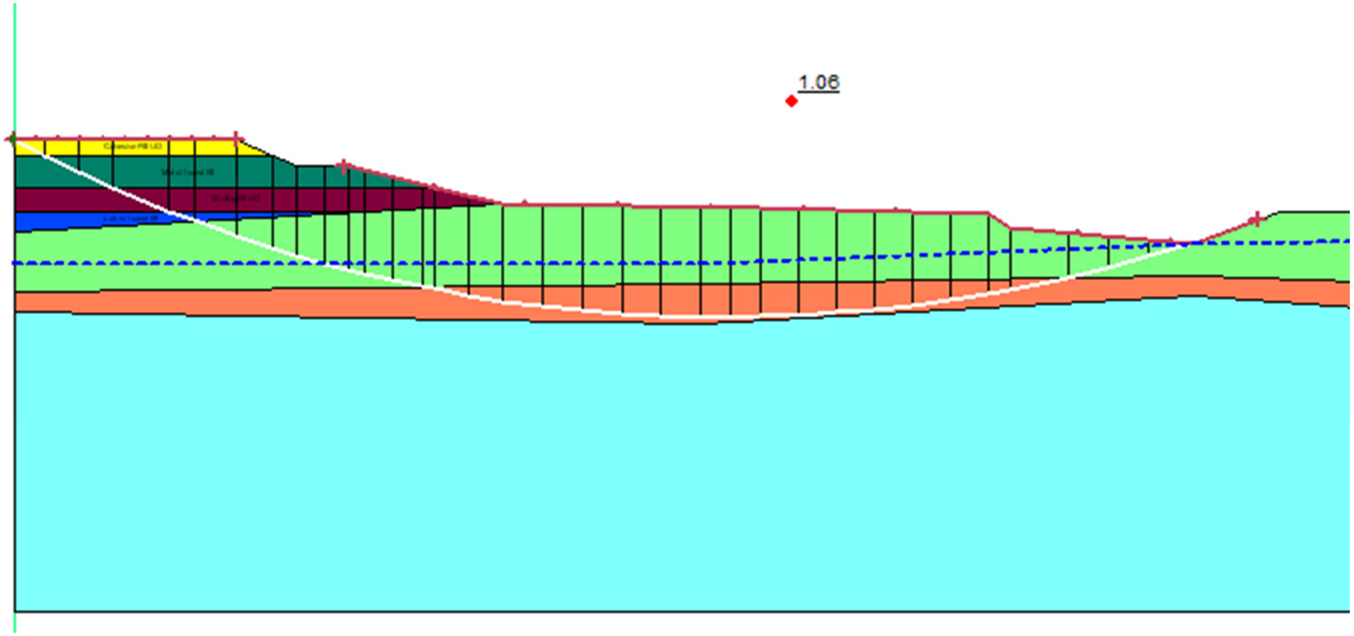
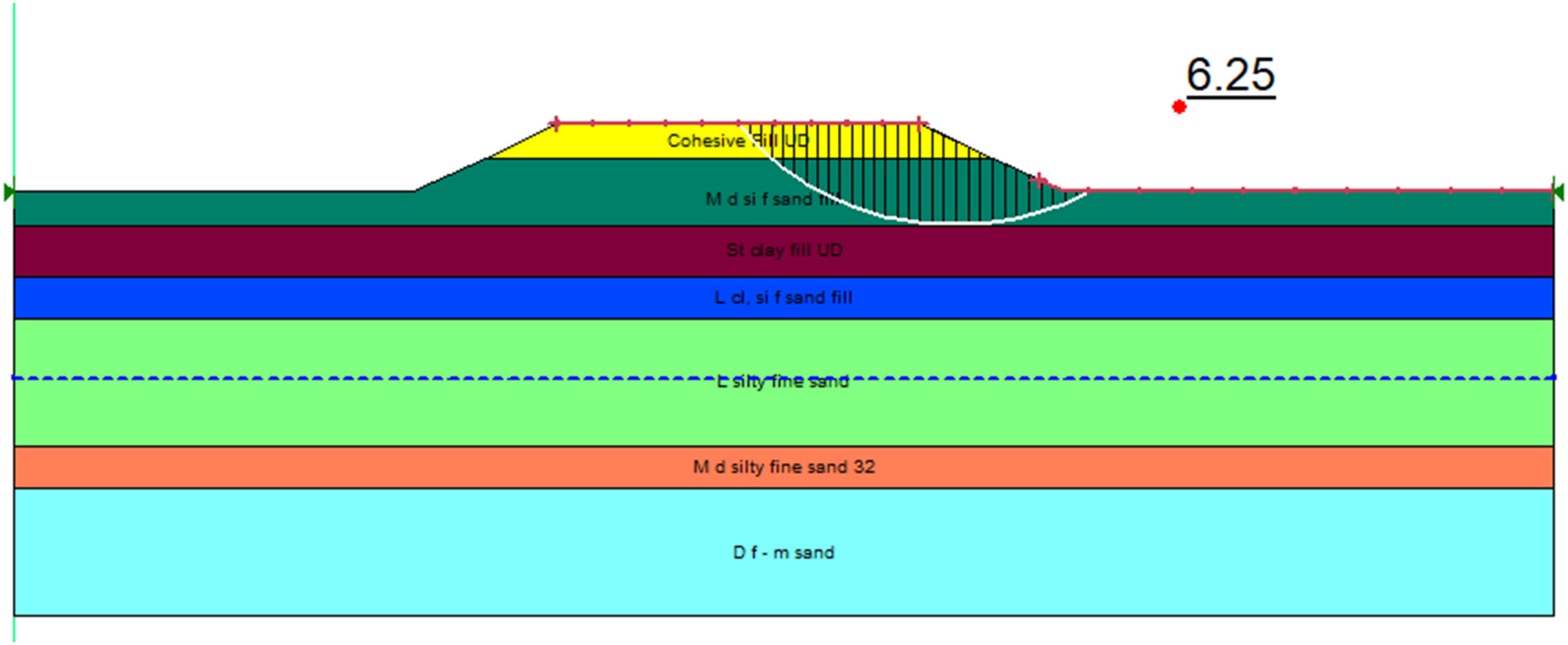


Results of Stability Analyses – Rapid Drawdown Condition from El 231 to El 214
Bent 1 End Slope
2H:1V Slope, H=16 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2 = 0.432$)
Bent 1 End Slope
2H:1V Slope, H=16 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – End of Construction
Bent 1 Side Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

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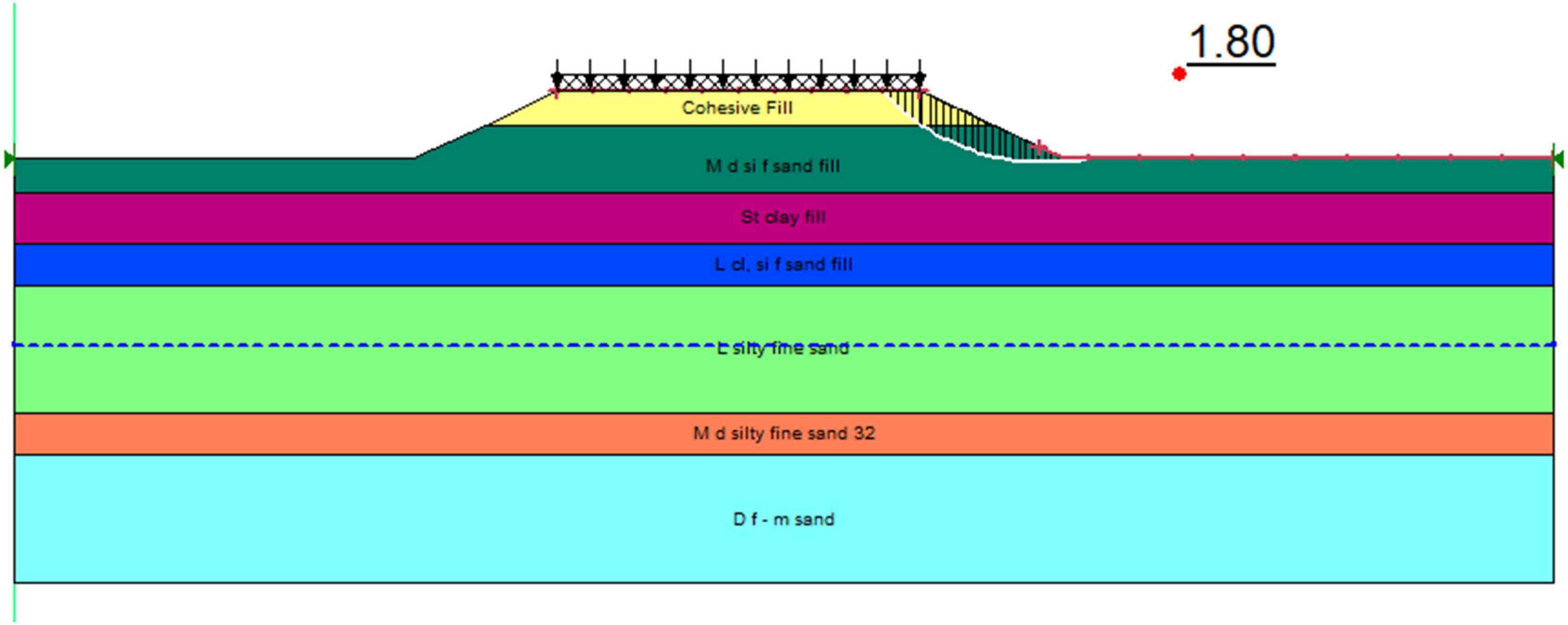

Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
2H:1V Slope, H=8 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

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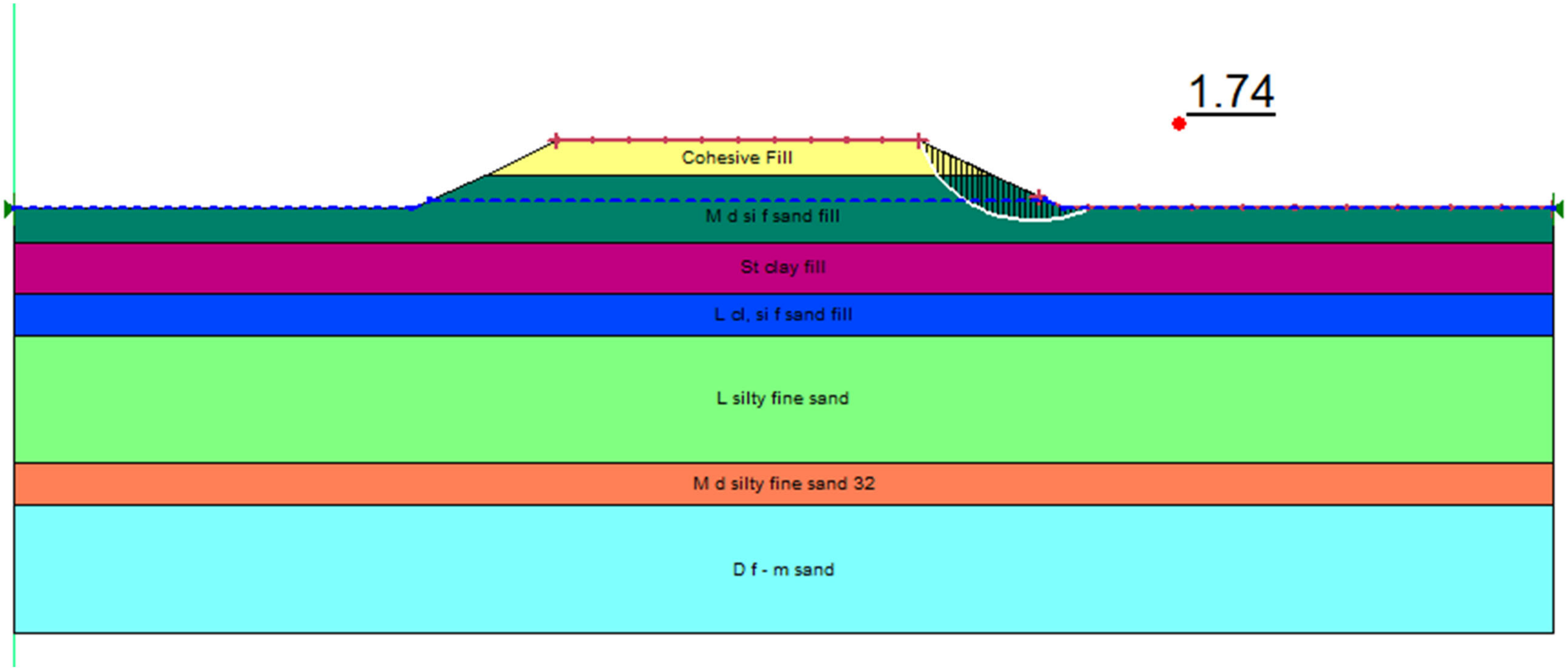

Results of Stability Analyses – Rapid Drawdown Condition from El 231 to Existing Grade
Bent 1 Side Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

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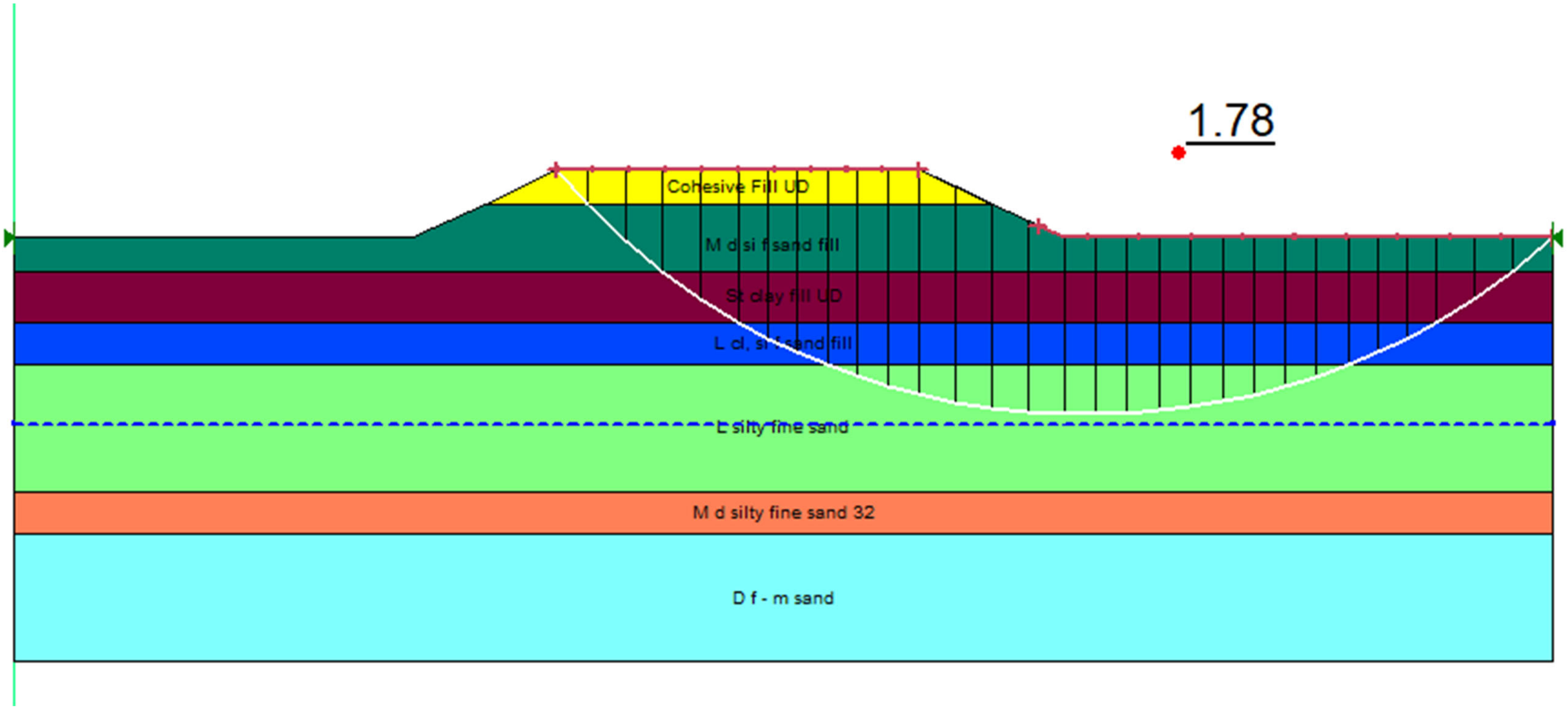
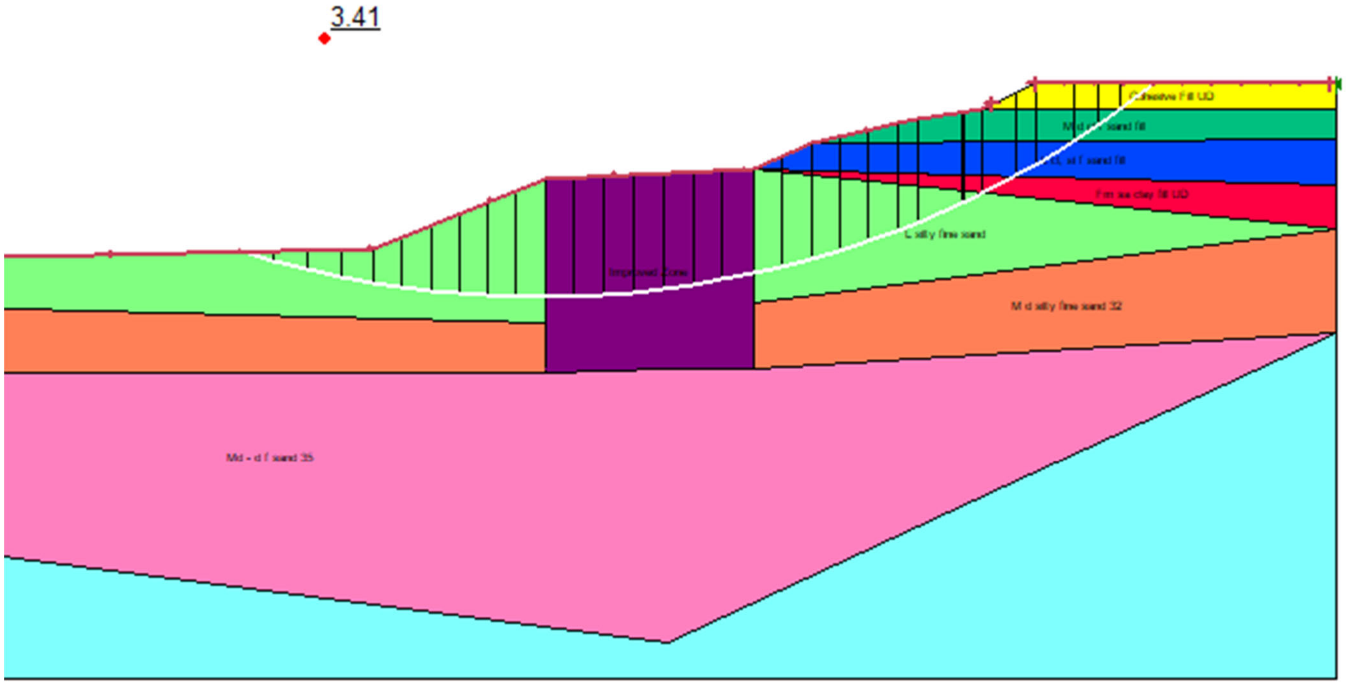
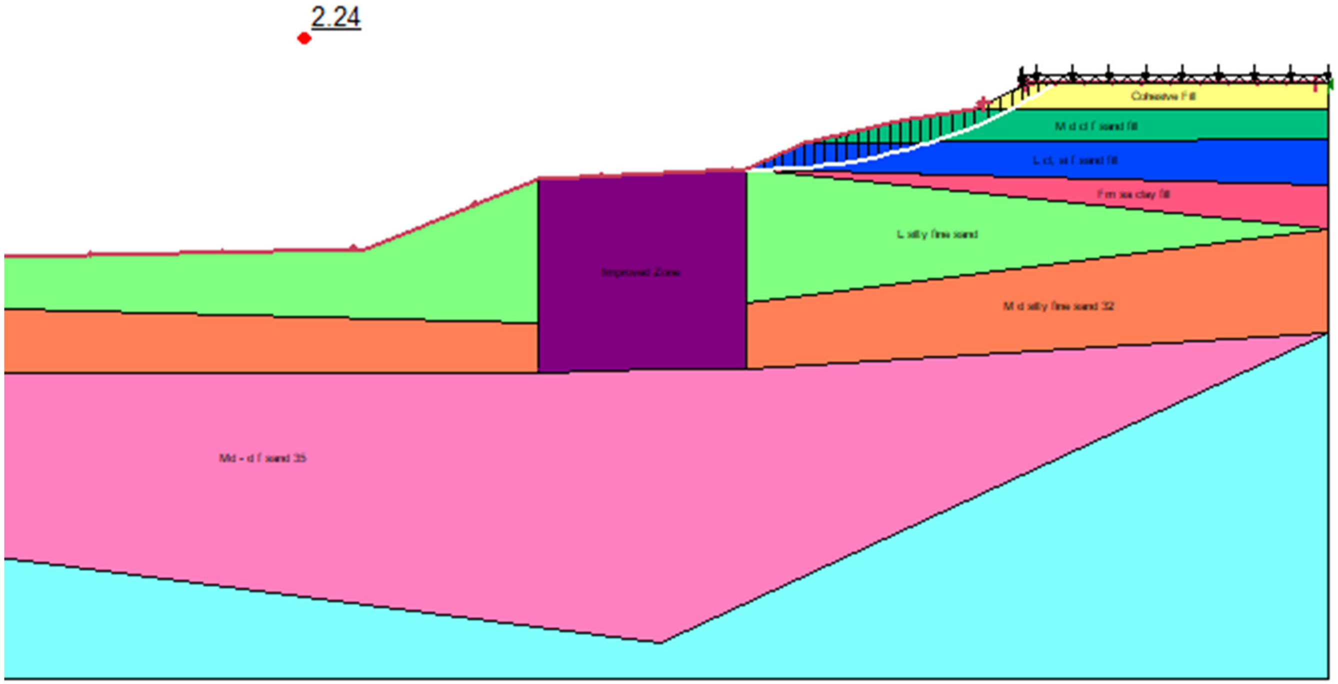


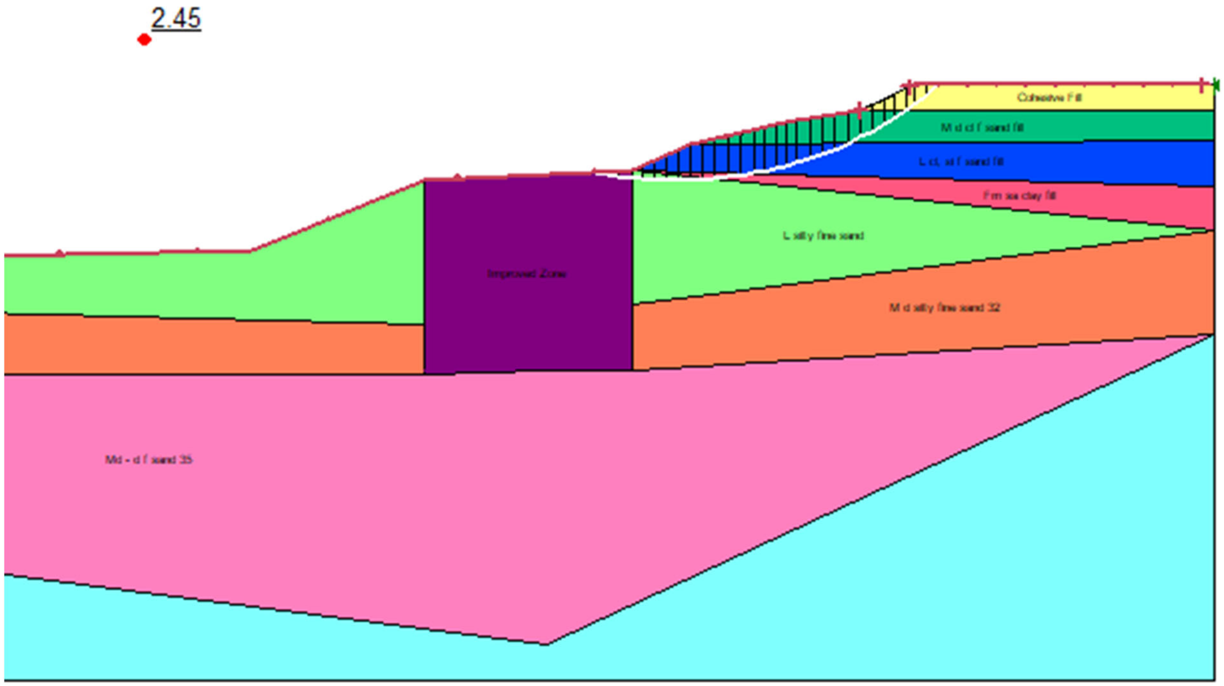
Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.432$)
Bent 1 Side Slope
2H:1V Slope, H=8 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – End of Construction
Bent 8 End Slope
2H:1V Slope, H=33 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Long Term Condition
Bent 8 End Slope
2H:1V Slope, H=33 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Rapid Drawdown Condition, El 231 to El 214
Bent 8 End Slope
2H:1V Slope, H=33 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

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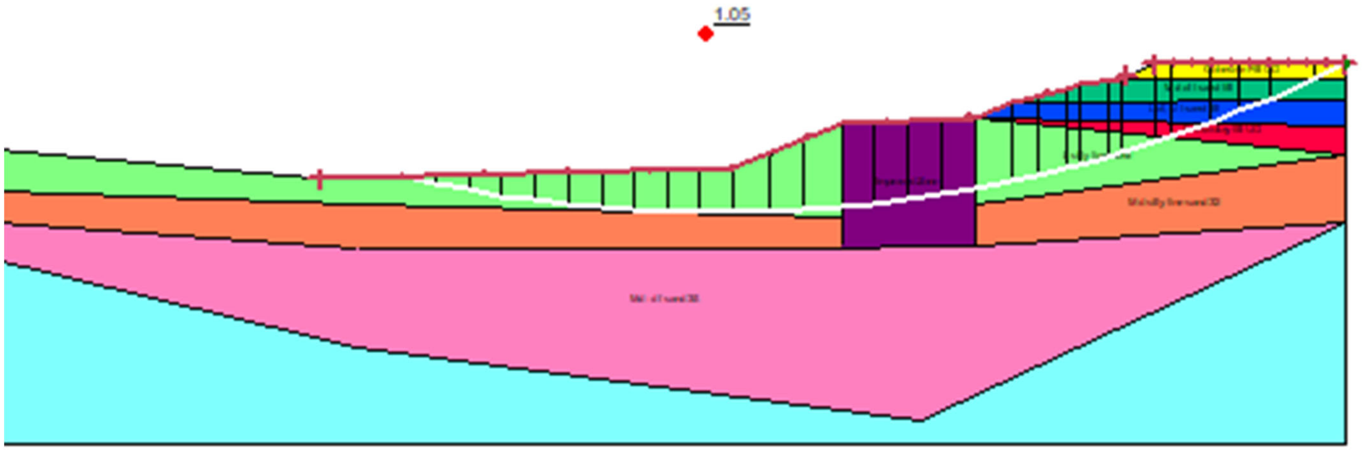
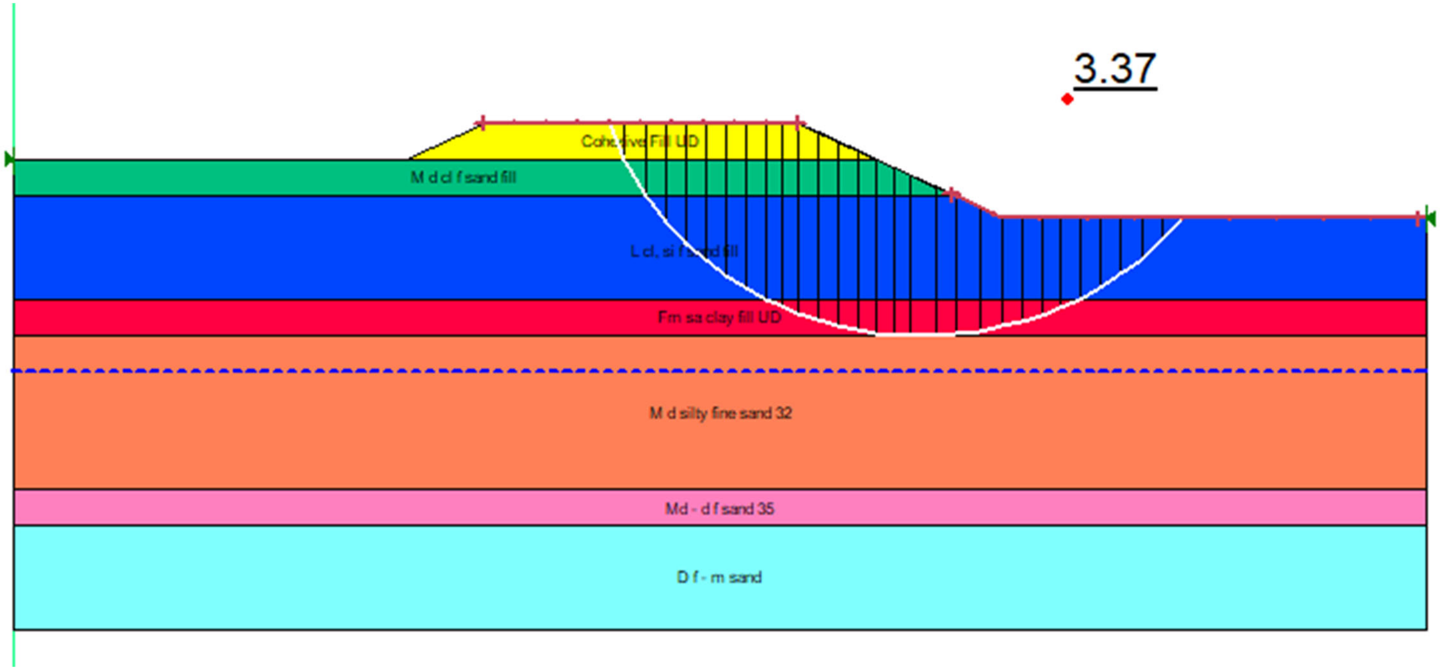
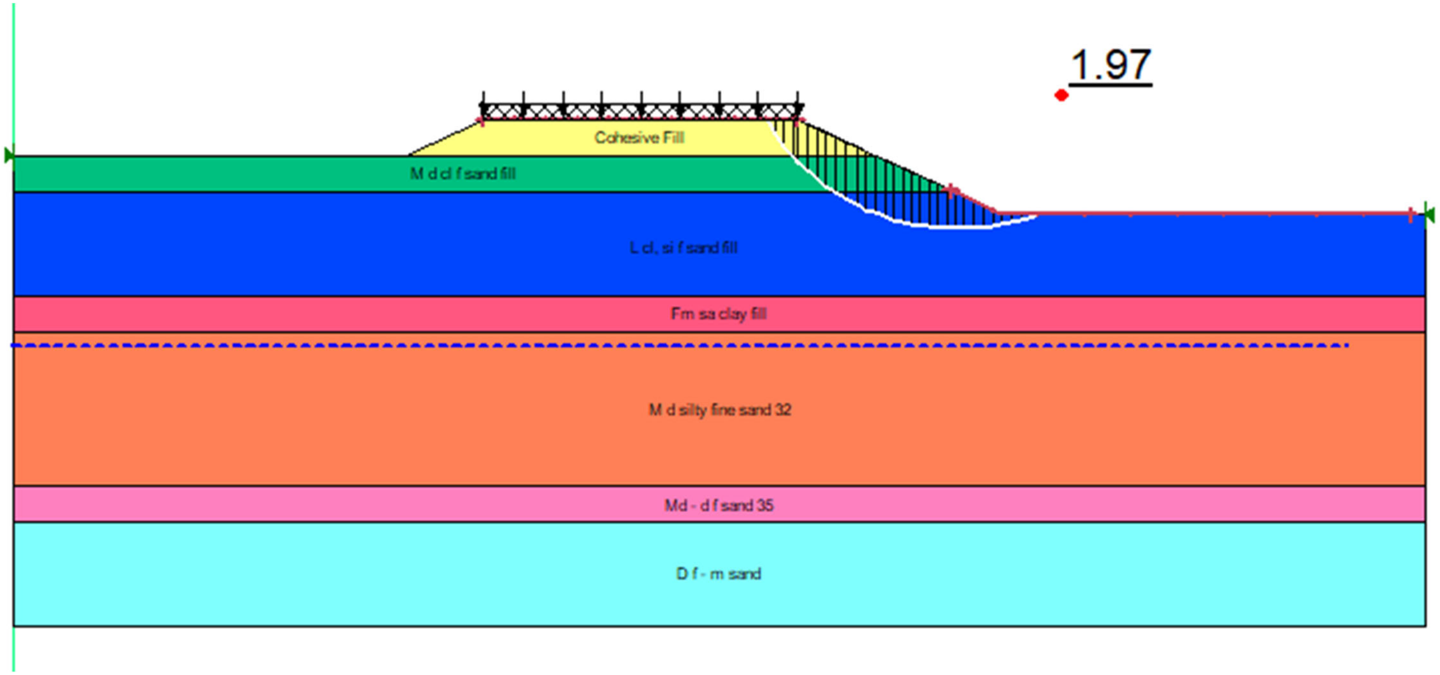


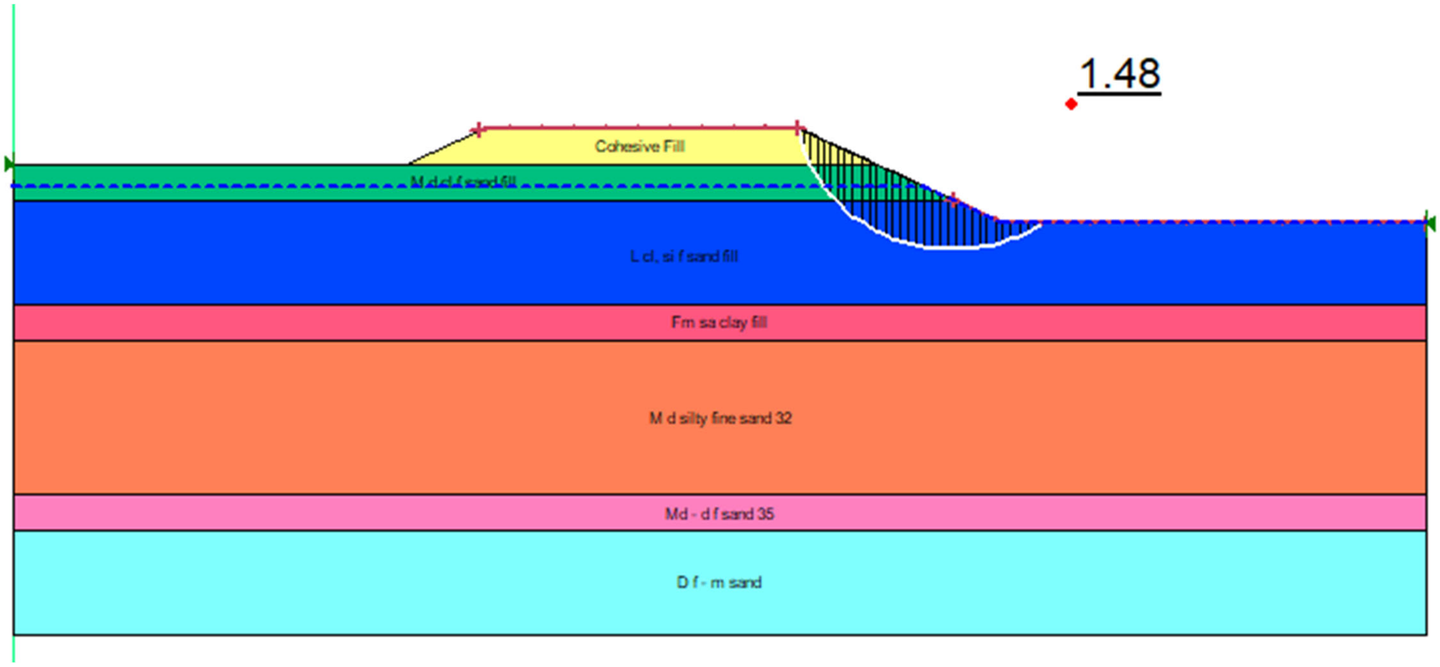
Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.432$)
Bent 8 End Slope
2H:1V Slope, H=33 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



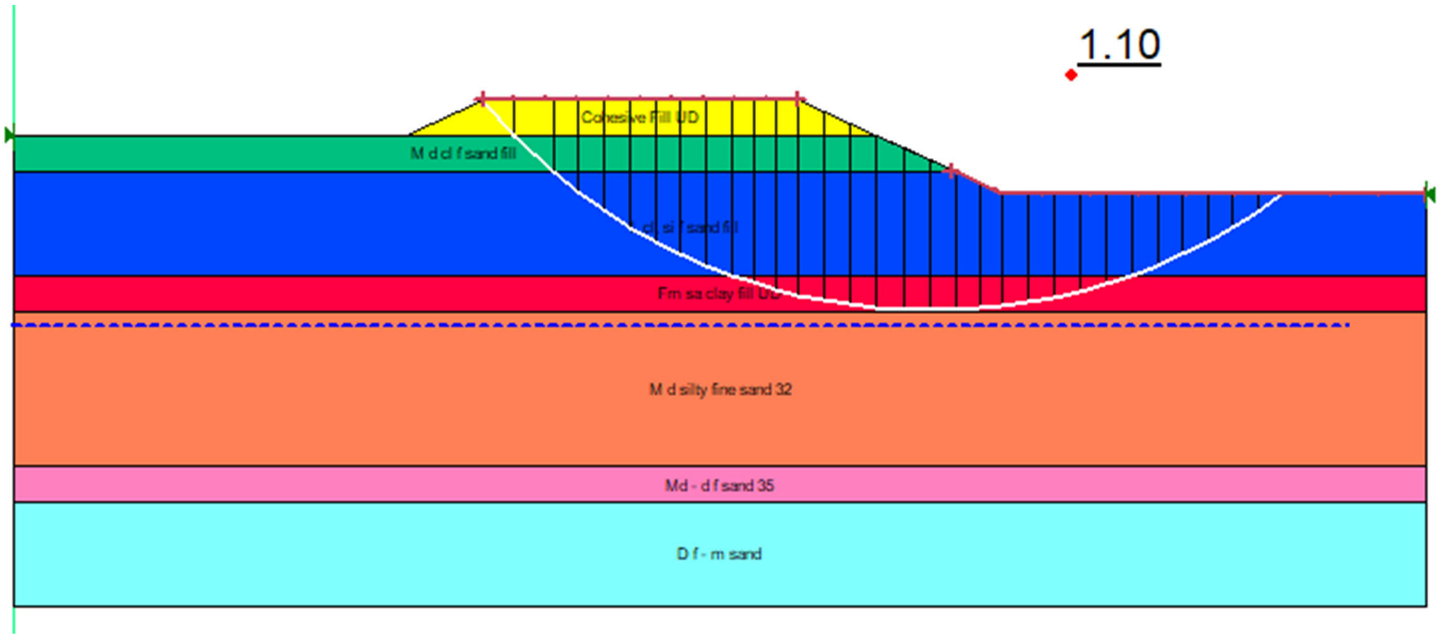
Results of Stability Analyses – End of Construction
Bent 8 Side Slope
2H:1V Slope, H=13 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Long Term Condition
Bent 8 Side Slope
2H:1V Slope, H=13 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River



Results of Stability Analyses – Rapid Drawdown Condition from El 231 to Existing Grade
Bent 8 Side Slope
2H:1V Slope, H=13 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

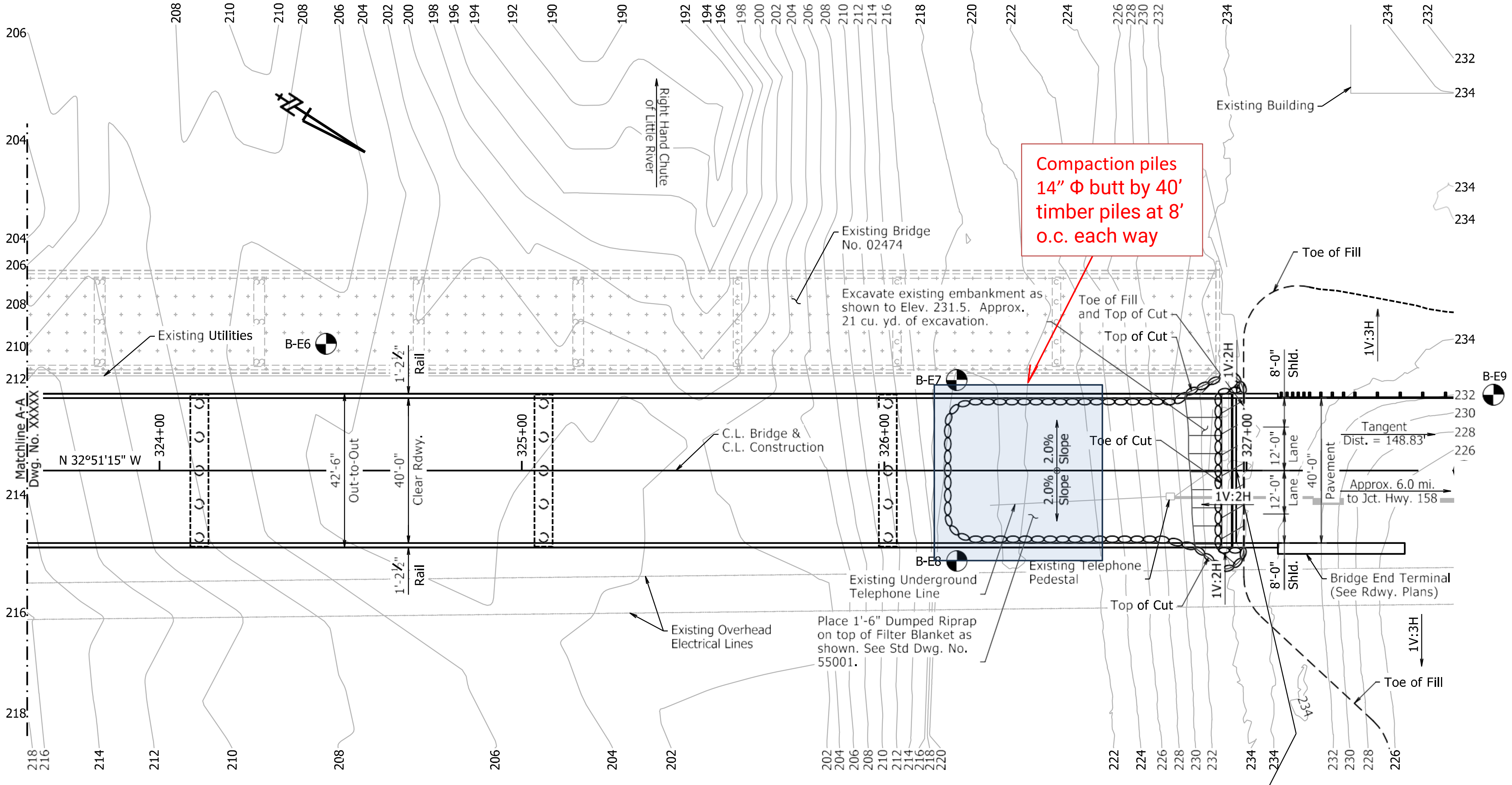


Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.432$)
Bent 8 Side Slope
2H:1V Slope, H=13 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Right Hand Chute of Little River

APPENDIX I

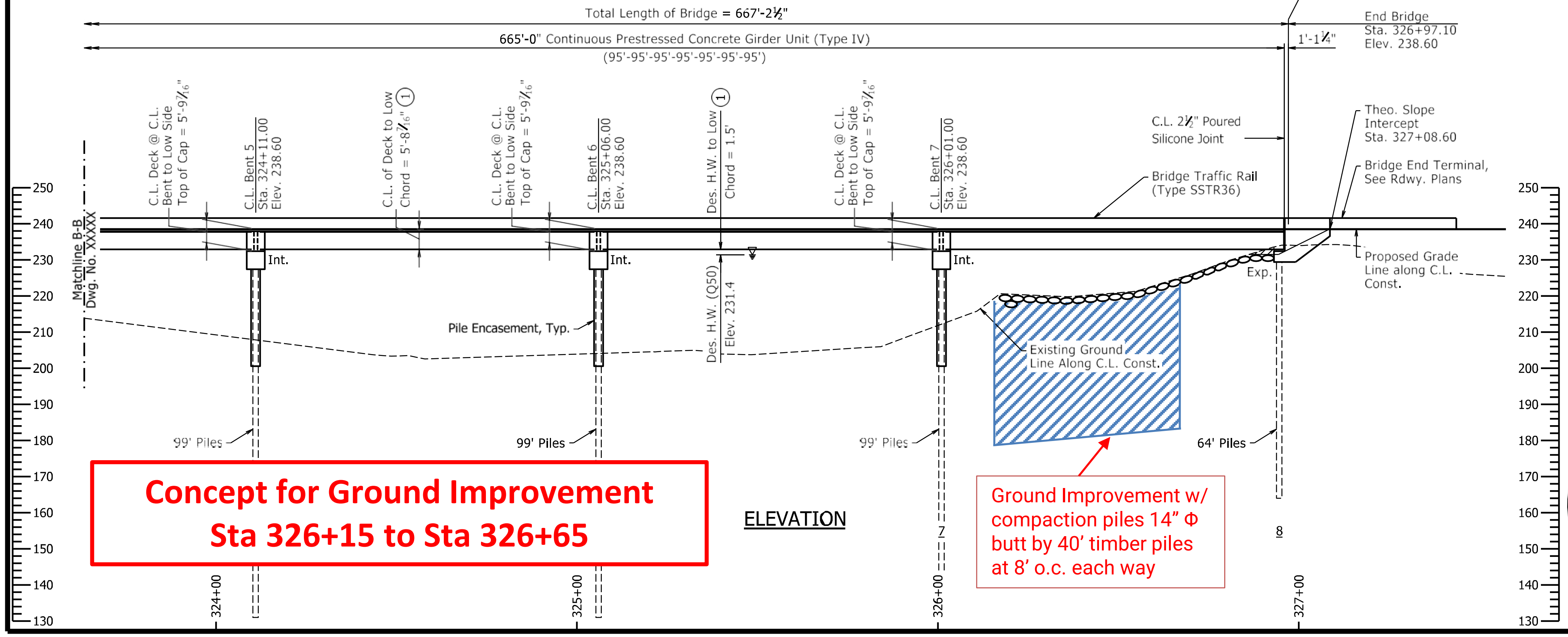
NOTE: GEOTECH REPORT PENDING

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	00000	2	21
		XXXXX		LAYOUT		XXXXX



Compaction piles 14" Φ butt by 40' timber piles at 8' o.c. each way

PLAN



Concept for Ground Improvement Sta 326+15 to Sta 326+65

ELEVATION

Ground Improvement w/ compaction piles 14" Φ butt by 40' timber piles at 8' o.c. each way

GENERAL NOTES (CONT'D.)

PILE ENCASEMENT: Pile encasement for Bents 5 thru 7 shall extend from bottom of cap to 3' below natural or finished ground. See Dwg. Nos. 55021 & XXXXX for additional information. Piles at Bents 2 thru 4 shall be covered by 48" Dia. Isolation Casings in accordance with the Job XXXXX SP "ISOLATION CASING". See Dwg. No. XXXXX for additional details.

BRIDGE DECK: The concrete bridge deck shall be given a tine finish as specified for final finishing in Subsection 802.19 for Class 5 Tined Bridge Roadway Surface Finish.

PROTECTIVE SURFACE TREATMENT: Class 2 Protective Surface Treatment shall be applied to the roadway surface and to the roadway face and top of the Bridge Traffic Rail in accordance with Section 803.

DETAIL DRAWINGS:	DRAWING NO(S).
End Bents	XXXXX-XXXXX
Intermediate Bents	XXXXX-XXXXX
Elastomeric Bearings	XXXXX-XXXXX
665' Prestressed Concrete Girder Unit	XXXXX-XXXXX
Concrete Filled Steel Shell Piling	55021 & XXXXX
Type Special Approach Gutters	XXXXX
Type C2 Approach Slabs	55040C2
Bridge Traffic Rail	55070

EXISTING BRIDGE: Existing Bridge No. 02474 (Log Mile 15.09) is 28.7' wide (24' clear roadway) and 662' long and consists of steel I-beam spans (15 spans total) supported by concrete piles. The existing bridge is located approximately 41' downstream from the proposed new bridge. Plans of the existing structure, if available, may be obtained upon request to the Construction Contract Development Section of the Program Management Division.

REMOVAL AND SALVAGE: After the new bridge is open to traffic, the Contractor shall remove Existing Bridge No. 02474, including existing riprap, in accordance with Section 205. Removal of existing riprap will not be paid for directly but shall be considered subsidiary to the item "Removal of Existing Bridge Structure (Site No._)". All material from the existing bridge shall become the property of the Contractor except the following:

1.The existing USGS stream gage shall remain the property of the USGS. The Contractor shall remove and store the stream gage on site in a manner approved by the Engineer. The Contractor shall notify the USGS 7 business days in advance of removing the existing stream gage. Contact information is as follows:

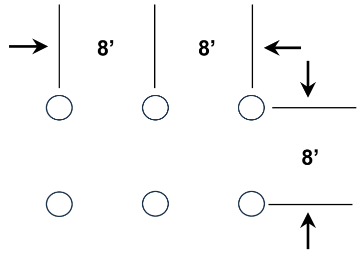
XXXXX
USGS
XXX-XXX-XXXX

2. The existing utilities attached to the bridge shall remain the property of the XXXXX.

This work shall be considered incidental to the item "Removal of Existing Bridge Structure (Site No._)".

MAINTENANCE OF TRAFFIC: See Roadway Plans.

Typical Compaction Pile Layout
(1" = 1')



For R/W Data, See Roadway Plans.

For Soil Boring Information, see Dwg. Nos. XXXXX & XXXXX.

For "VERTICAL ALIGNMENT DATA", see Dwg. No. XXXXX.

① See "HYDRAULIC DATA" table on Dwg. No. XXXXX.

Note:
Use Type Special Approach Gutters and Type C2 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. XXXXX & 55040C2, respectively. Eliminate or modify Type Special Approach Gutter curb section to fit bridge end terminal. No additional payment will be made for this work.

Note:
Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "ROUNDING DETAIL" on Dwg. No. XXXXX.

PRELIMINARY SUBJECT TO REVISION

SHEET 2 OF 4
LAYOUT OF BRIDGE
HWY. 135 OVER RIGHT HAND CHUTE LITTLE RIVER
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: JRF DATE: 10-11-2022 FILENAME: b101124x5_l2.dgn
CHECKED BY: CAW DATE: 11-07-2022 SCALE: 1" = 20'
DESIGNED BY: JRF DATE: 10-04-2022
BRIDGE NO. XXXXX DRAWING NO. XXXXX

USER: JF5222
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x5_l2.dgn
PLOTTED: 8/11/2023 12:23
SCALE: 1:40

APPENDIX J

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer's installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX K

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, "The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project." This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX L

WEAP ANALYSES - STEEL SHELL PILES

Project: 101124 - Hwy 135
Poinsett County, Arkansas
GHBW Project No: 23-031

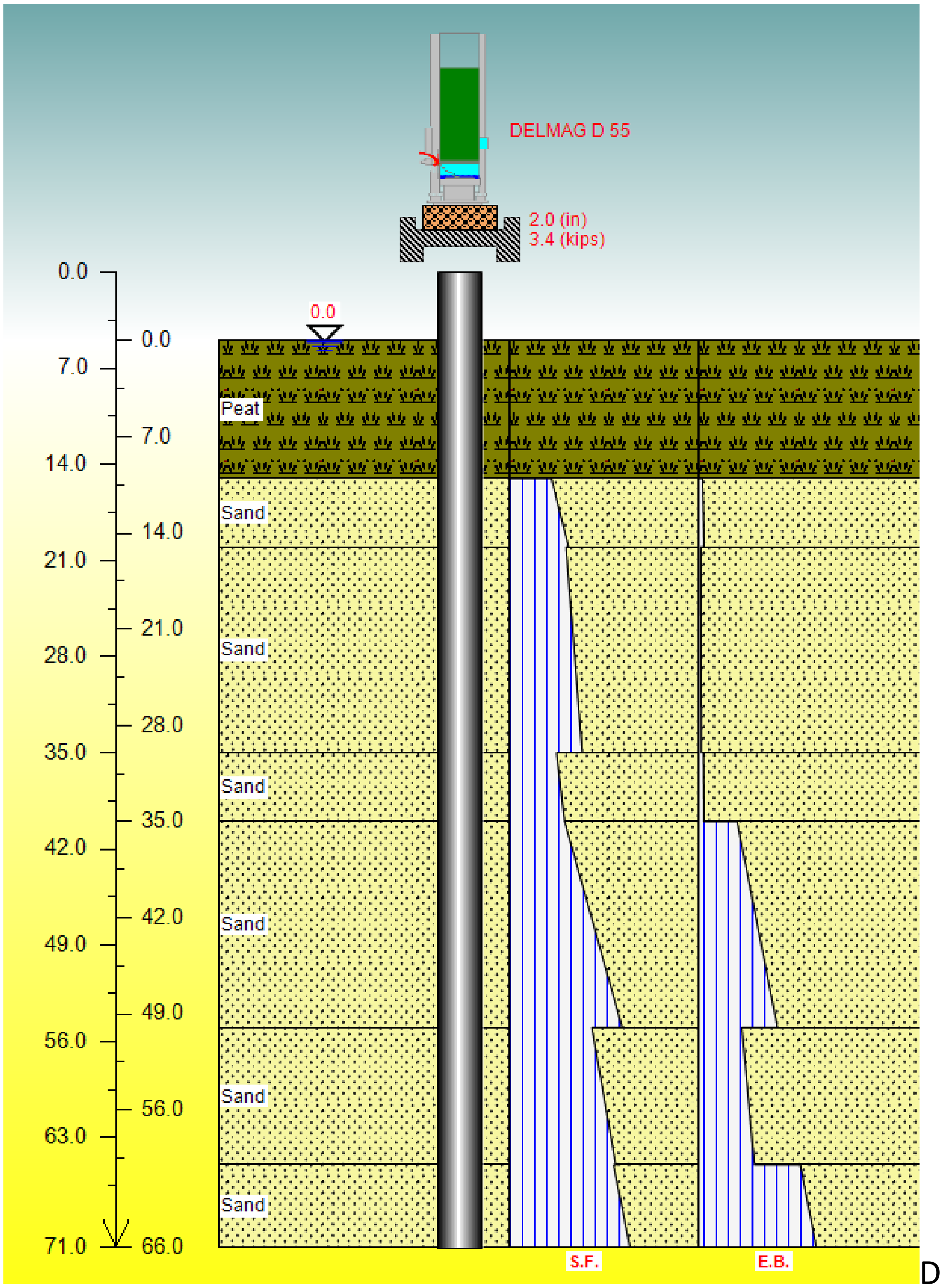
Bridge	Bent	Pile Diameter (in)	Wall Thickness (in)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El.	Min Tip El.	Pile Length (ft)	Min Hammer Energy (ft-kip)	Max Comp Stress, ksi
5 - Right Hand Chute of Little River	1	24	0.75	428	230	164	66	125	37.1
	2	30	0.75	856	220	136	84	248	40.5
	3	30	0.75	856	216	136	80	212	40.4
	4	30	0.75	856	216	136	80	212	40.4
	5	30	0.75	863	208	131	77	212	39.3
	6	30	0.75	863	204	131	73	212	40.0
	7	30	0.75	863	212	131	81	212	39.0
	8	24	0.75	428	230	164	66	125	35.4

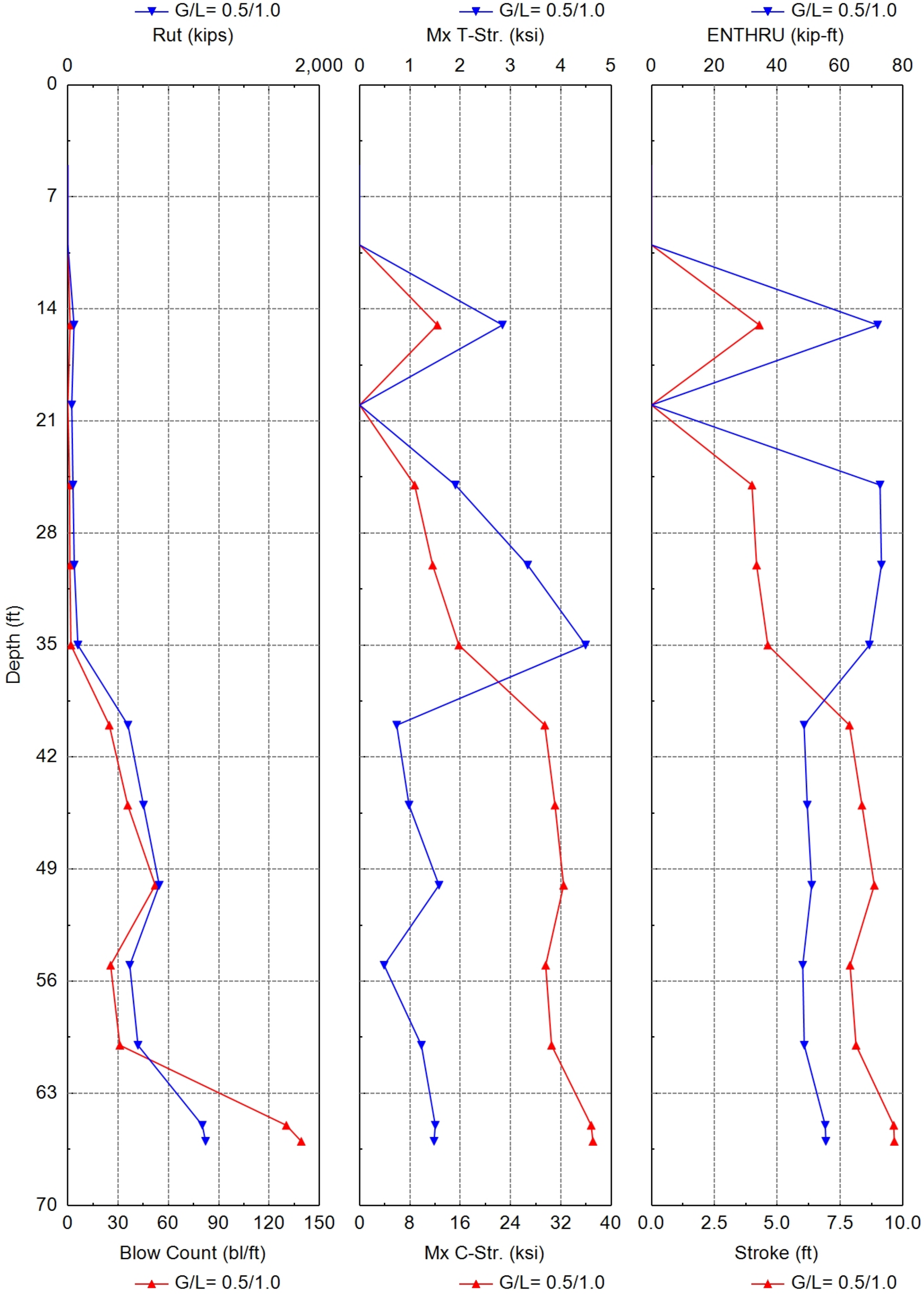
ArDOT 101124 Hwy 135 over RHC of Little River

Bent 1

24-in-diameter Steel Shell Pile

Delmag D55




G/L= 0.5/1.0
Rut (kips)
0
2,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0
1
2
3
4
5
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
7
14
21
28
35
42
49
56
63
70
0
30
60
90
120
150
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
10.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
15.0	48.8	6.3	42.4	1.3	12.349	2.839	4.29	71.9	D 55
20.0	30.8	13.8	17.0	0.0	0.000	0.000	0.00	0.0	D 55
25.0	40.4	21.9	18.5	1.0	8.742	1.903	4.00	72.7	D 55
30.0	50.7	30.7	20.0	1.2	11.603	3.340	4.18	73.2	D 55
35.0	79.9	37.1	42.8	1.8	15.767	4.490	4.62	69.3	D 55
40.0	478.1	45.2	432.8	24.7	29.438	0.736	7.87	48.5	D 55
45.0	601.2	55.8	545.4	35.7	31.069	0.982	8.37	49.6	D 55
50.0	726.8	68.8	658.0	52.1	32.434	1.577	8.86	51.0	D 55
55.0	492.3	80.0	412.4	25.5	29.582	0.486	7.90	48.1	D 55
60.0	557.5	92.6	464.9	31.0	30.519	1.227	8.13	48.6	D 55
65.0	1069.3	106.6	962.7	130.3	36.818	1.503	9.63	55.3	D 55
66.0	1094.9	109.6	985.3	139.2	37.102	1.478	9.65	55.5	D 55

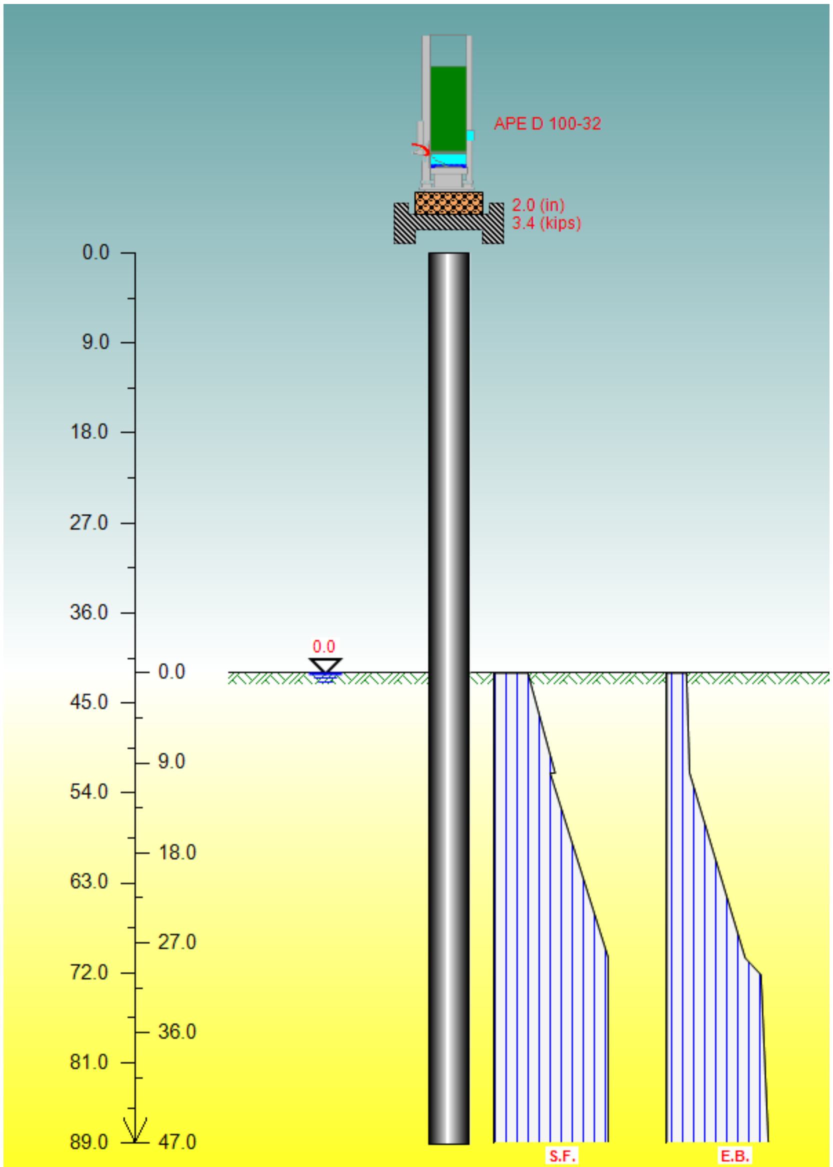
Total driving time: 33 minutes; Total Number of Blows: 1334 (starting at penetration 5.0 ft)

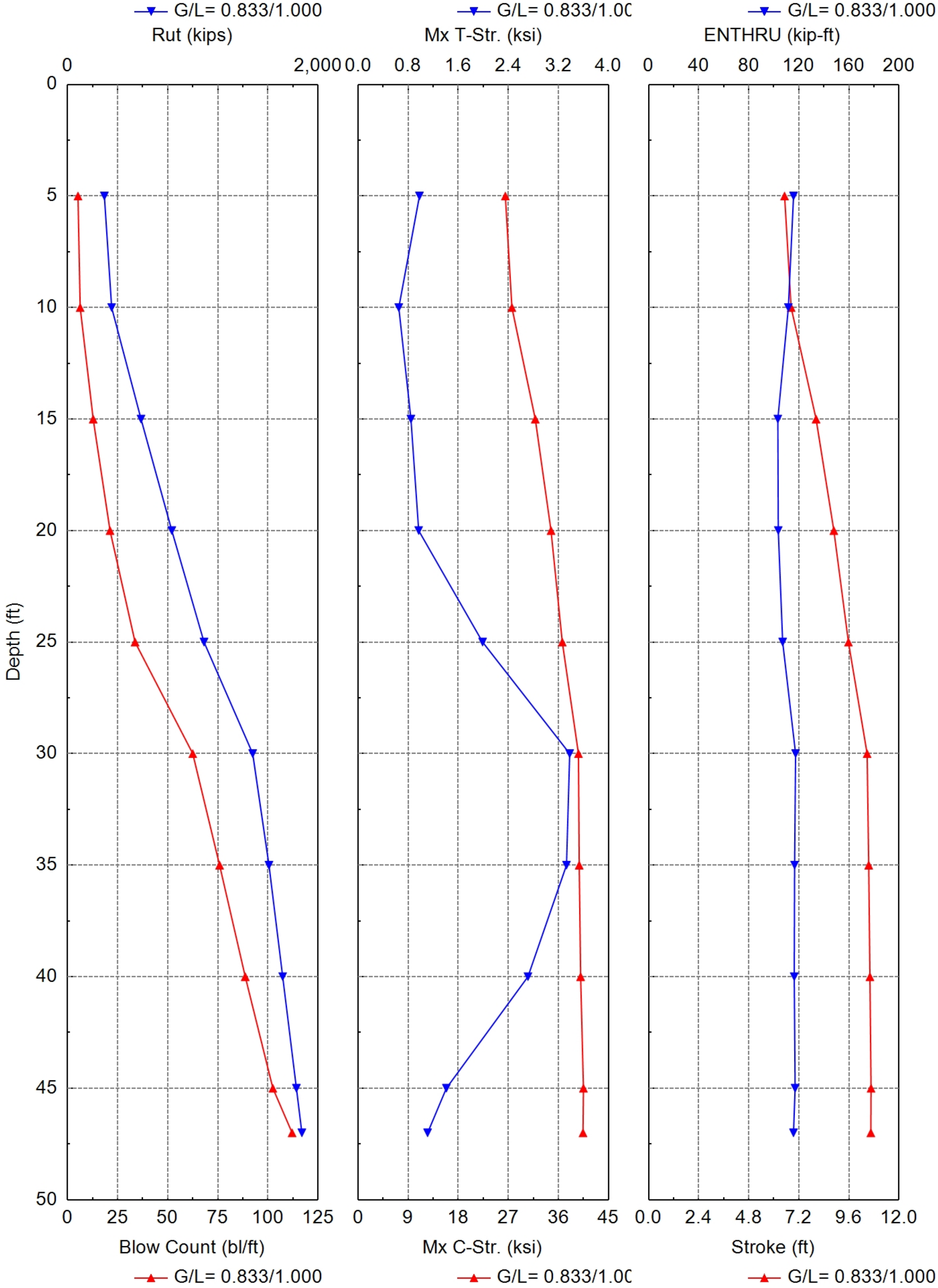
ArDOT 101124 Hwy 135 over RHC of Little River

Bent 2

30-in-diameter Steel Shell Pile

APE D100-32




G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.833/1.000
0
9
18
27
36
45
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	293.9	28.8	265.1	5.2	26.466	0.981	6.52	115.8	D 100-32
10.0	352.2	67.5	284.7	6.3	27.653	0.650	6.83	111.7	D 100-32
15.0	586.8	113.1	473.8	12.8	31.849	0.845	8.03	103.3	D 100-32
20.0	832.8	170.0	662.8	21.2	34.676	0.968	8.88	103.6	D 100-32
25.0	1090.1	238.2	851.9	33.7	36.705	1.990	9.58	107.1	D 100-32
30.0	1478.7	317.2	1161.5	62.5	39.608	3.381	10.47	117.4	D 100-32
35.0	1608.8	399.0	1209.7	75.9	39.746	3.330	10.55	116.6	D 100-32
40.0	1718.3	480.9	1237.5	88.7	40.000	2.714	10.61	116.3	D 100-32
45.0	1827.9	562.7	1265.2	102.5	40.502	1.411	10.66	117.0	D 100-32
47.0	1871.7	595.4	1276.3	112.1	40.430	1.111	10.65	115.8	D 100-32

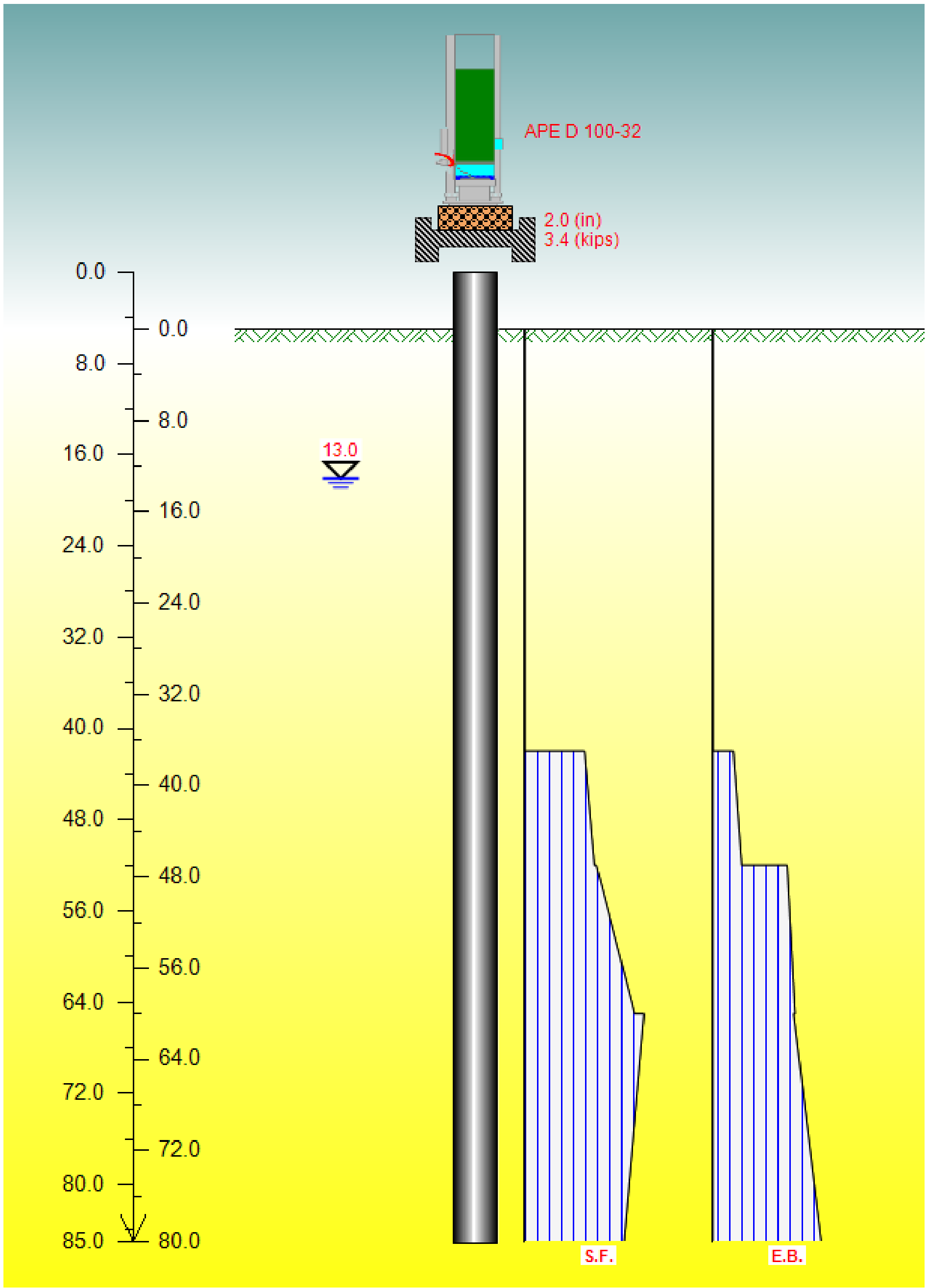
Total driving time: 54 minutes; Total Number of Blows: 1989 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over RHC of Little River

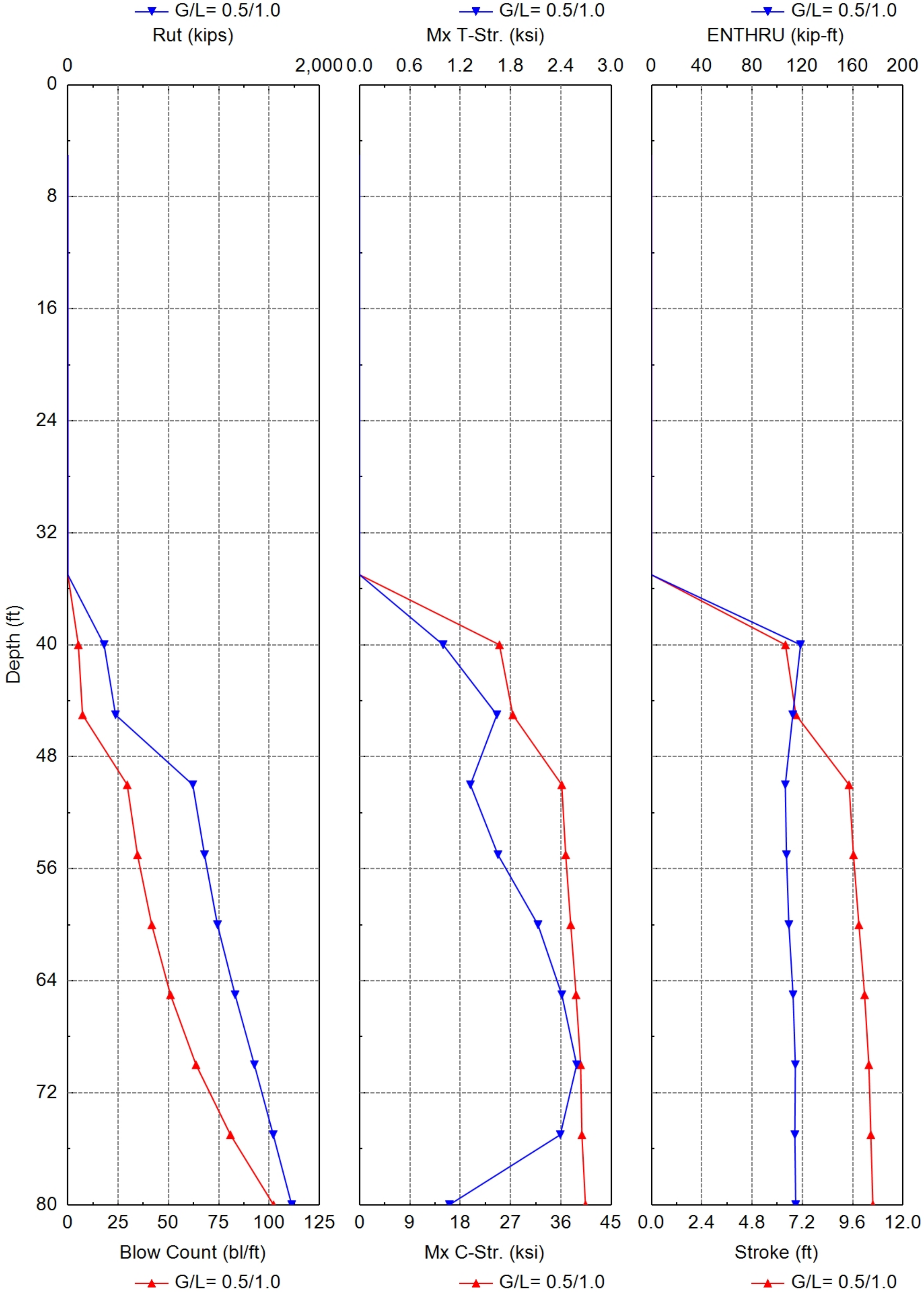
Bent 3

30-in-diameter Steel Shell Pile

APE D100-32



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company


G/L= 0.5/1.0
Rut (kips)
0
2,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
9
18
27
36
45
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
15.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
20.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
25.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
30.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
35.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
40.0	288.3	24.2	264.2	5.2	25.009	0.995	6.39	118.5	D 100-32
45.0	378.9	67.0	311.8	7.3	27.398	1.637	6.89	112.4	D 100-32
50.0	993.5	115.1	878.4	29.5	36.156	1.321	9.42	106.4	D 100-32
55.0	1086.7	172.9	913.9	34.6	36.862	1.650	9.64	107.4	D 100-32
60.0	1189.6	240.4	949.3	41.7	37.766	2.127	9.90	109.2	D 100-32
65.0	1329.7	317.3	1012.4	51.0	38.717	2.411	10.17	112.6	D 100-32
70.0	1483.0	390.8	1092.2	63.7	39.538	2.588	10.37	114.4	D 100-32
75.0	1633.1	461.1	1172.0	80.8	39.759	2.393	10.47	114.0	D 100-32
80.0	1779.8	528.1	1251.7	102.2	40.395	1.071	10.56	114.7	D 100-32

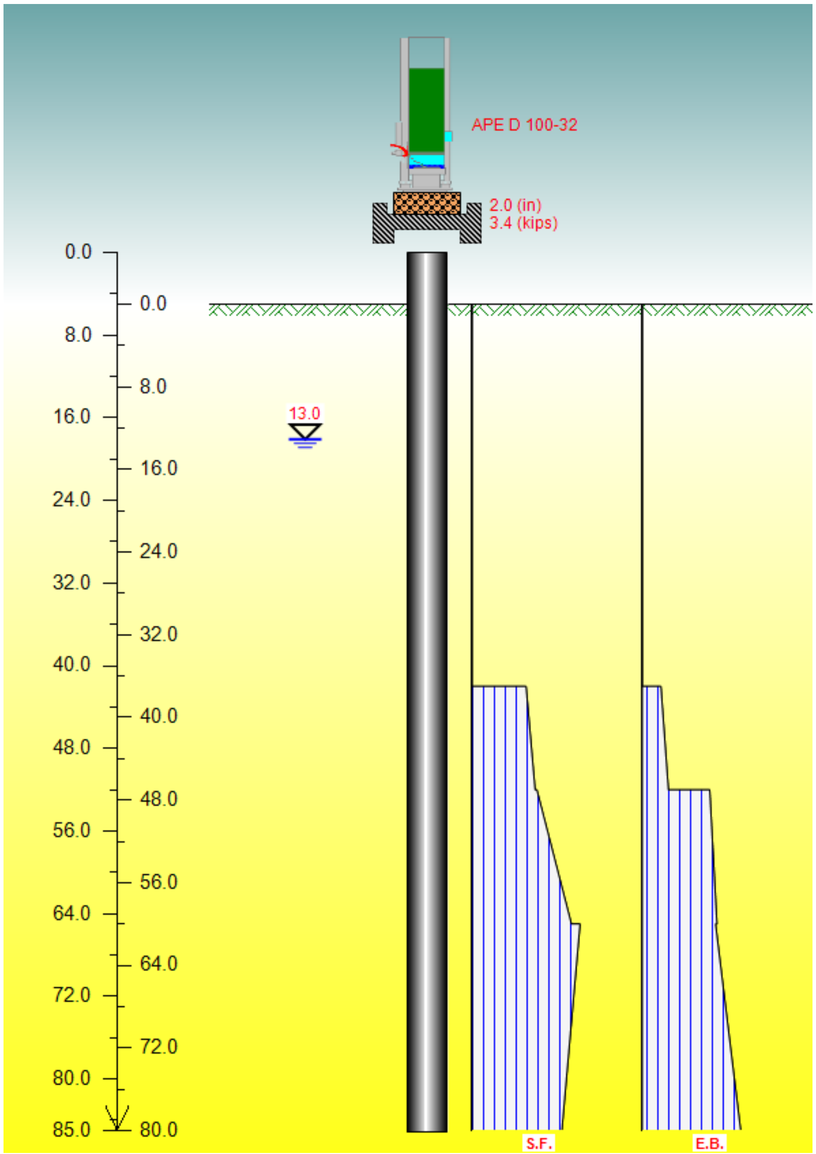
Total driving time: 49 minutes; Total Number of Blows: 1834 (starting at penetration 5.0 ft)

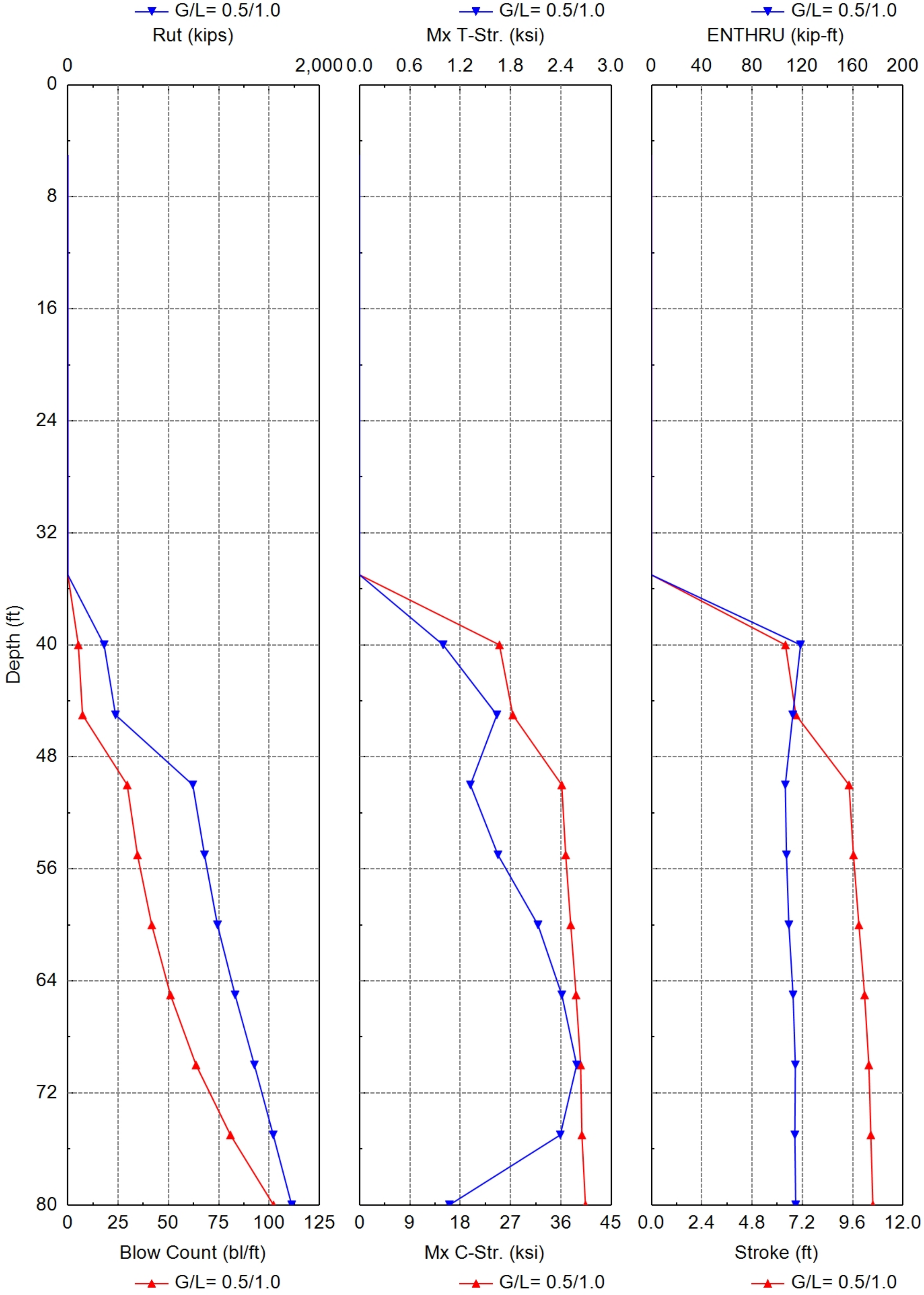
ArDOT 101124 Hwy 135 over RHC of Little River

Bent 4

30-in-diameter Steel Shell Pile

APE D100-32



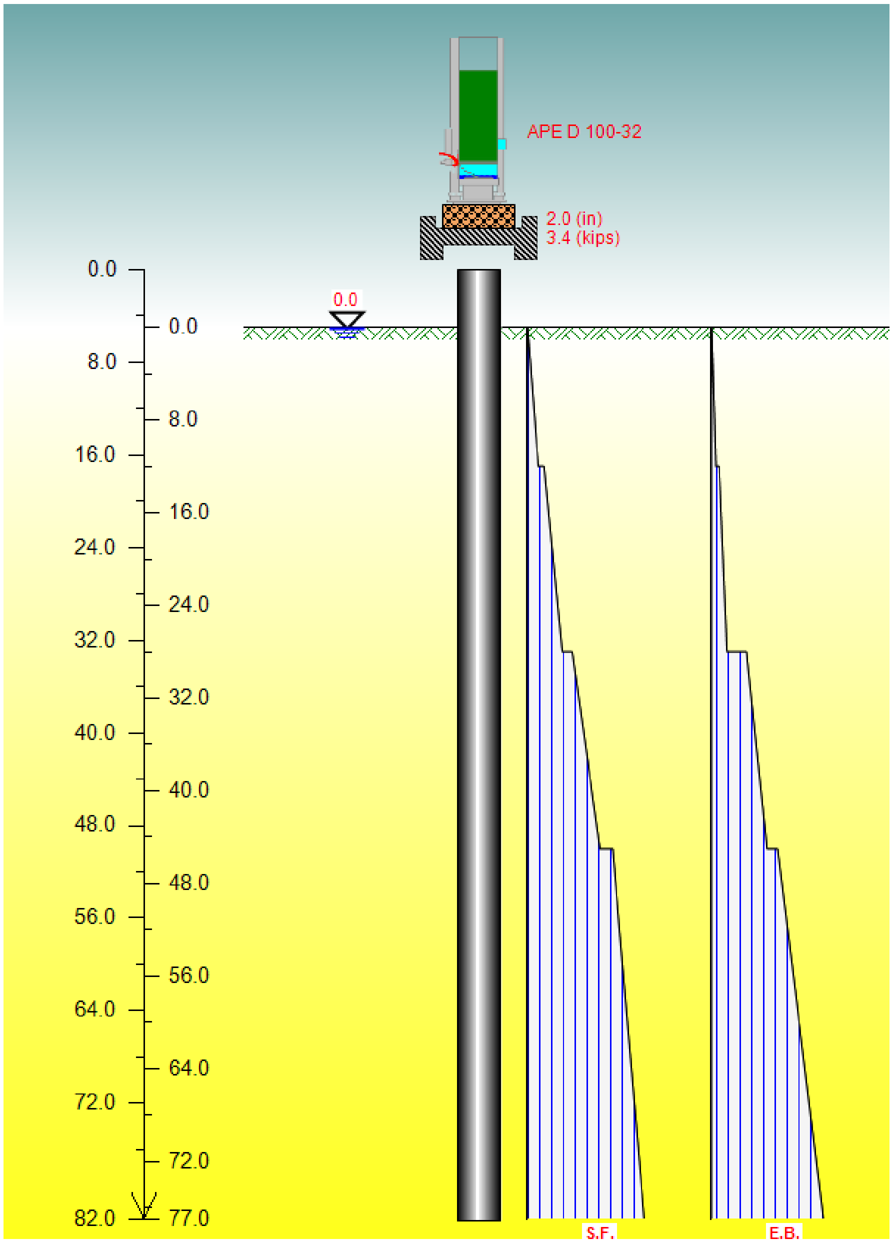

G/L= 0.5/1.0
Rut (kips)
0
2,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
9
18
27
36
45
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

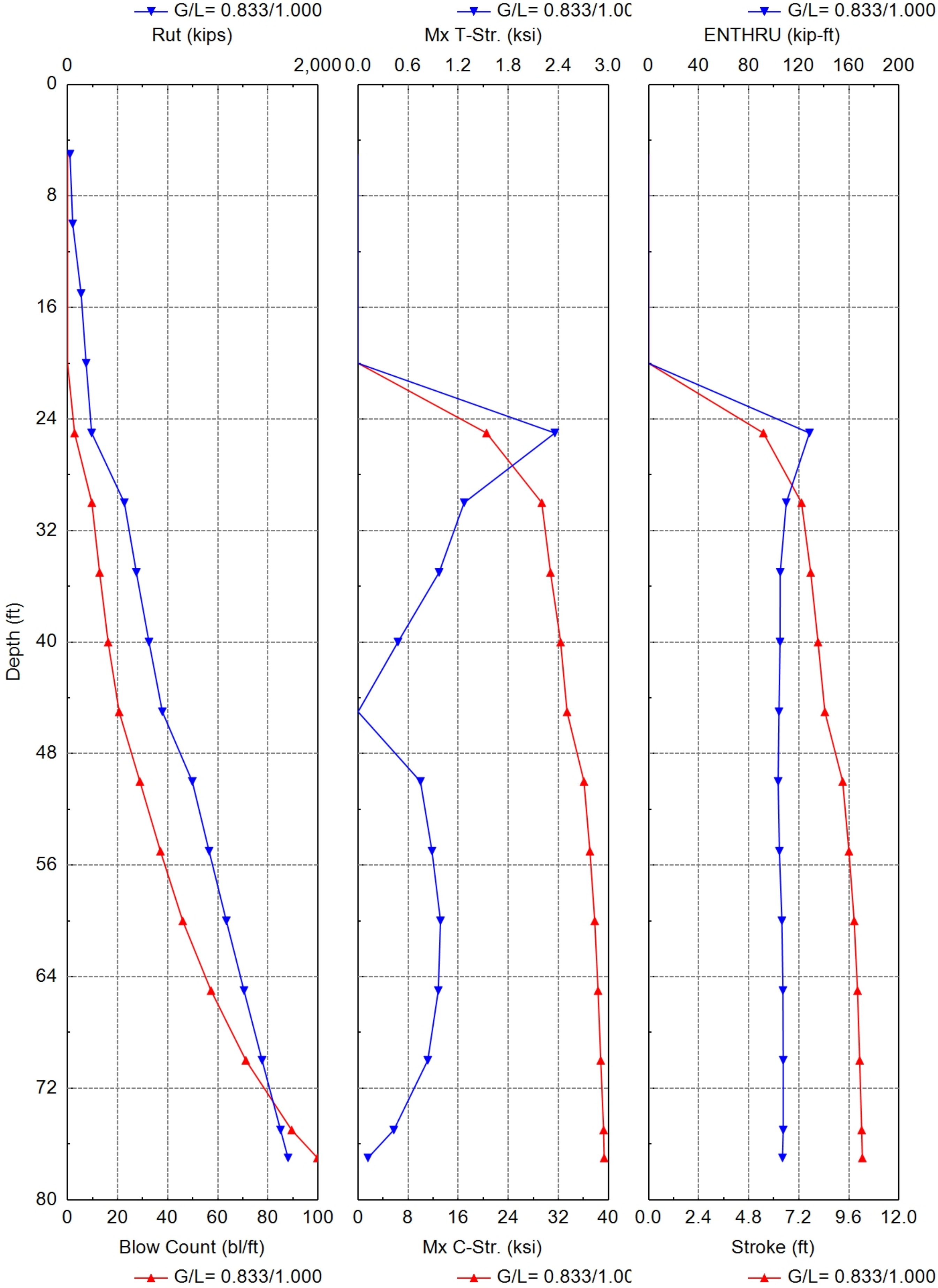
Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
15.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
20.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
25.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
30.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
35.0	0.0	0.0	0.0	0.3	0.000	0.000	11.25	0.0	D 100-32
40.0	288.3	24.2	264.2	5.2	25.009	0.995	6.39	118.5	D 100-32
45.0	378.9	67.0	311.8	7.3	27.398	1.637	6.89	112.4	D 100-32
50.0	993.5	115.1	878.4	29.5	36.156	1.321	9.42	106.4	D 100-32
55.0	1086.7	172.9	913.9	34.6	36.862	1.650	9.64	107.4	D 100-32
60.0	1189.6	240.4	949.3	41.7	37.766	2.127	9.90	109.2	D 100-32
65.0	1329.7	317.3	1012.4	51.0	38.717	2.411	10.17	112.6	D 100-32
70.0	1483.0	390.8	1092.2	63.7	39.538	2.588	10.37	114.4	D 100-32
75.0	1633.1	461.1	1172.0	80.8	39.759	2.393	10.47	114.0	D 100-32
80.0	1779.8	528.1	1251.7	102.2	40.395	1.071	10.56	114.7	D 100-32

Total driving time: 49 minutes; Total Number of Blows: 1834 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over RHC of Little River
Bent 5
30-in-diameter Steel Shell Pile
APE D100-32



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

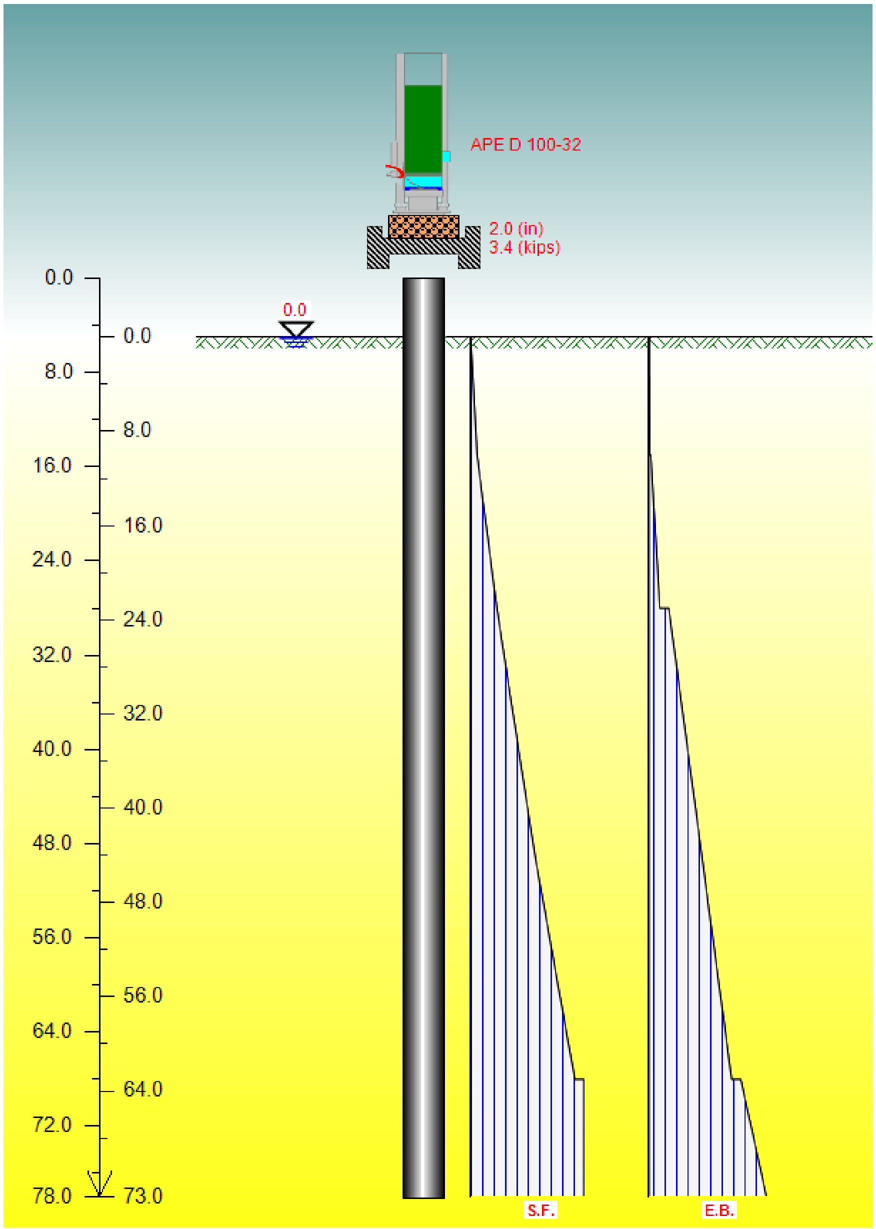

G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
20
40
60
80
100
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

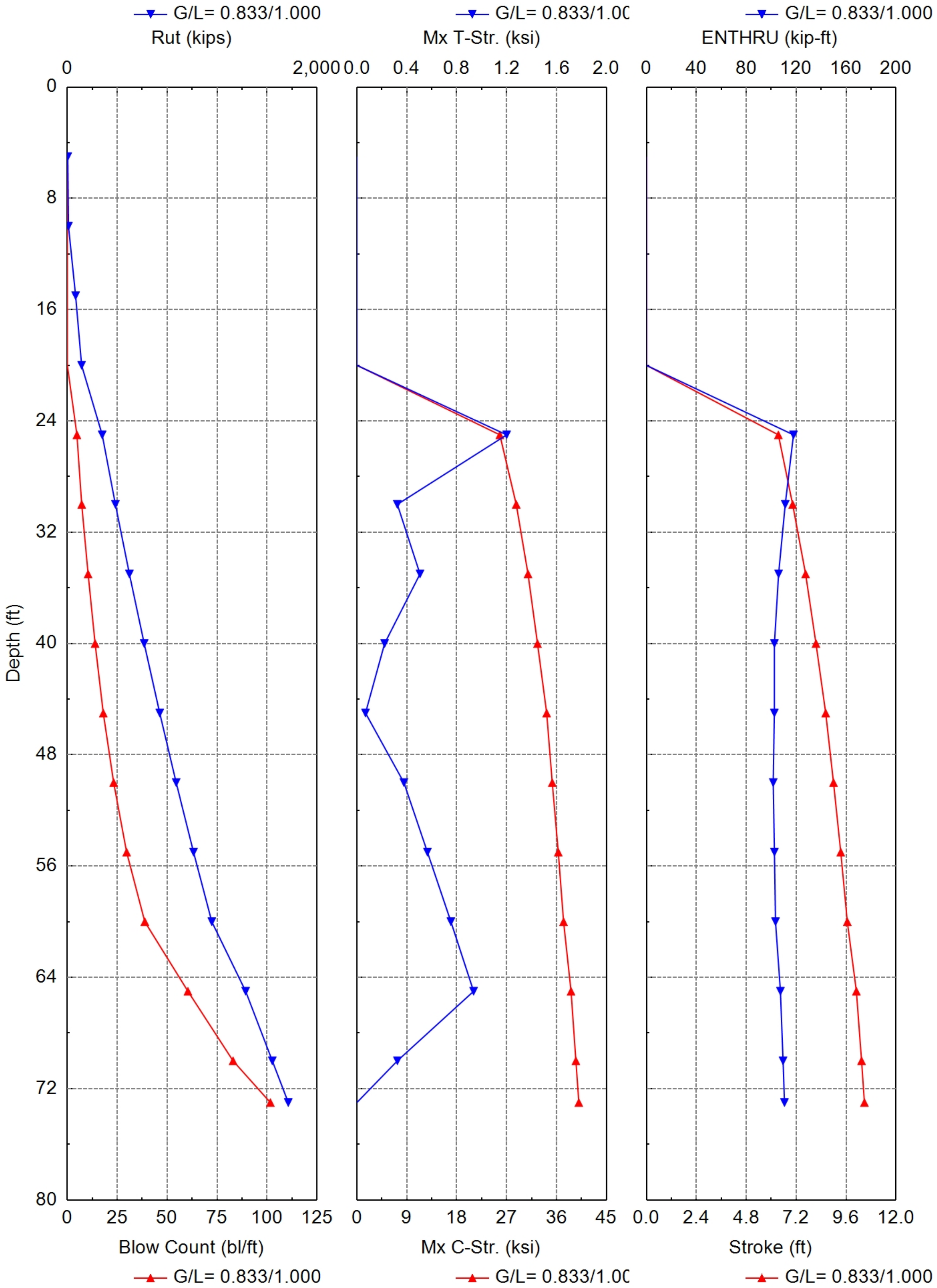
Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	19.0	1.5	17.5	0.3	0.000	0.000	11.25	0.0	D 100-32
10.0	41.0	6.0	35.0	0.0	0.000	0.000	0.00	0.0	D 100-32
15.0	108.1	16.0	92.1	0.0	0.000	0.000	0.00	0.0	D 100-32
20.0	148.4	31.3	117.1	0.0	0.000	0.000	0.00	0.0	D 100-32
25.0	192.4	50.4	142.1	2.8	20.521	2.358	5.50	128.5	D 100-32
30.0	453.8	76.1	377.7	9.7	29.349	1.271	7.33	110.0	D 100-32
35.0	549.5	111.1	438.3	12.8	30.739	0.971	7.77	105.2	D 100-32
40.0	650.6	151.7	499.0	16.2	32.361	0.478	8.12	105.0	D 100-32
45.0	757.3	197.7	559.6	20.6	33.387	0.000	8.45	104.2	D 100-32
50.0	996.3	256.6	739.7	28.9	36.091	0.748	9.30	103.5	D 100-32
55.0	1130.7	318.9	811.8	37.1	37.043	0.886	9.60	104.5	D 100-32
60.0	1268.3	384.5	883.9	46.0	37.811	0.988	9.86	106.5	D 100-32
65.0	1409.3	453.4	956.0	57.3	38.341	0.962	10.01	107.3	D 100-32
70.0	1553.7	525.6	1028.1	71.2	38.767	0.835	10.12	107.6	D 100-32
75.0	1701.3	601.1	1100.2	89.5	39.230	0.428	10.21	107.6	D 100-32
77.0	1761.3	632.3	1129.0	99.8	39.317	0.119	10.25	107.0	D 100-32

Total driving time: 51 minutes; Total Number of Blows: 1927 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over RHC of Little River
Bent 6
30-in-diameter Steel Shell Pile
APE D100-32




G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.833/1.000
0
9
18
27
36
45
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	3.4	1.1	2.3	0.3	0.000	0.000	11.25	0.0	D 100-32
10.0	8.9	4.3	4.7	0.3	0.000	0.000	11.25	0.0	D 100-32
15.0	66.2	11.2	55.1	0.0	0.000	0.000	0.00	0.0	D 100-32
20.0	114.3	23.3	90.9	0.0	0.000	0.000	0.00	0.0	D 100-32
25.0	279.4	40.9	238.5	4.8	25.776	1.200	6.34	117.9	D 100-32
30.0	385.2	64.6	320.6	7.2	28.734	0.324	7.03	111.2	D 100-32
35.0	497.3	94.6	402.8	10.4	30.858	0.506	7.65	105.9	D 100-32
40.0	615.7	130.9	484.9	13.9	32.565	0.222	8.15	102.6	D 100-32
45.0	740.5	173.5	567.0	18.0	34.220	0.069	8.62	102.5	D 100-32
50.0	872.3	223.1	649.1	23.2	35.226	0.377	9.00	101.6	D 100-32
55.0	1011.3	280.1	731.3	29.7	36.310	0.565	9.34	102.6	D 100-32
60.0	1157.7	344.3	813.4	38.8	37.266	0.753	9.66	103.4	D 100-32
65.0	1429.0	417.8	1011.2	60.4	38.616	0.935	10.09	107.4	D 100-32
70.0	1642.6	496.4	1146.2	83.0	39.485	0.324	10.34	109.5	D 100-32
73.0	1770.7	543.5	1227.2	101.7	40.025	0.000	10.48	110.6	D 100-32

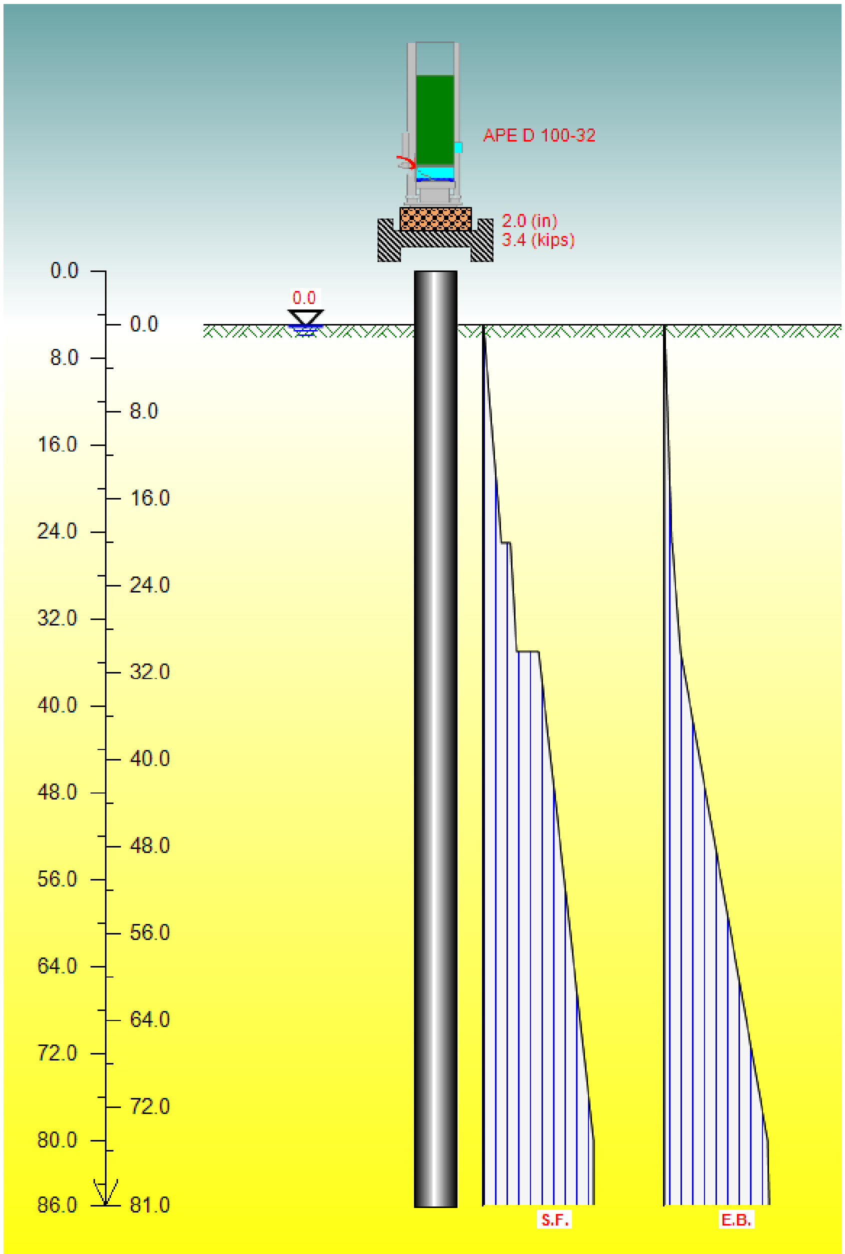
Total driving time: 40 minutes; Total Number of Blows: 1519 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over RHC of Little River

Bent 7

30-in-diameter Steel Shell Pile

APE D100-32



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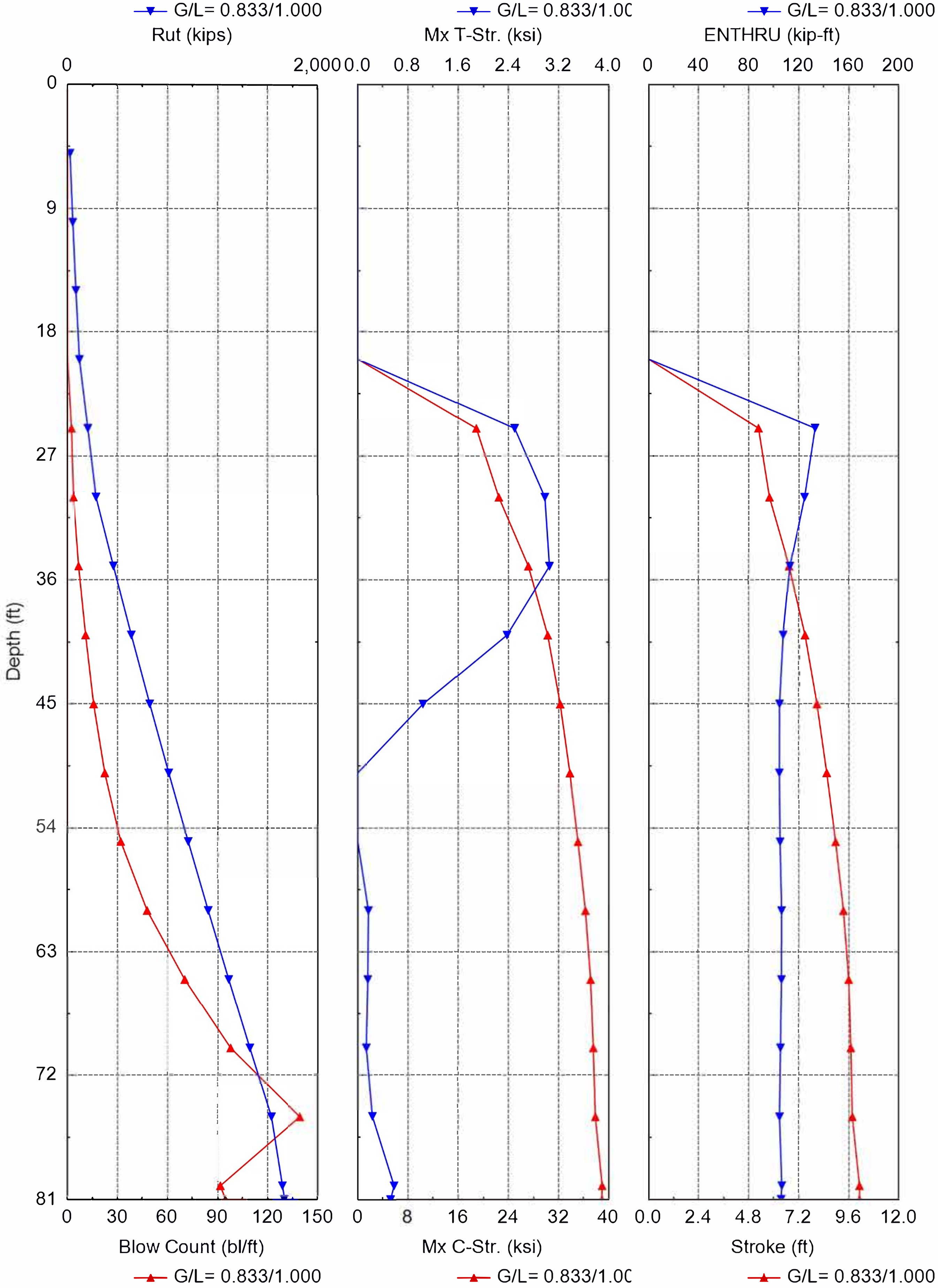
CONSULTING ENGINEERS
A UES Company


G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
40
80
120
160
200
Depth (ft)
0
9
18
27
36
45
54
63
72
81
0
30
60
90
120
150
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	19.0	1.5	17.5	0.3	0.000	0.000	11.25	0.0	D 100-32
10.0	41.1	6.1	35.0	0.0	0.000	0.000	0.00	0.0	D 100-32
15.0	66.1	13.6	52.5	0.0	0.000	0.000	0.00	0.0	D 100-32
20.0	94.2	24.2	70.0	0.0	0.000	0.000	0.00	0.0	D 100-32
25.0	161.4	43.6	117.8	2.3	18.842	2.497	5.26	132.8	D 100-32
30.0	227.1	65.1	162.0	3.5	22.430	2.978	5.79	124.5	D 100-32
35.0	366.1	104.8	261.3	6.6	27.144	3.051	6.73	112.9	D 100-32
40.0	509.3	148.8	360.5	10.8	30.236	2.374	7.50	107.5	D 100-32
45.0	656.6	196.8	459.8	15.6	32.233	1.042	8.07	104.6	D 100-32
50.0	808.2	249.1	559.1	22.3	33.776	0.000	8.54	104.5	D 100-32
55.0	963.9	305.6	658.3	31.9	35.103	0.000	8.97	105.1	D 100-32
60.0	1123.8	366.2	757.6	47.7	36.294	0.169	9.35	106.3	D 100-32
65.0	1287.9	431.0	856.8	70.2	37.120	0.159	9.60	106.2	D 100-32
70.0	1456.1	500.0	956.1	97.7	37.548	0.136	9.70	105.4	D 100-32
75.0	1628.6	573.2	1055.4	139.2	37.875	0.233	9.77	104.6	D 100-32
80.0	1716.1	648.4	1067.7	91.6	38.966	0.579	10.11	106.5	D 100-32
81.0	1733.6	663.5	1070.1	94.7	38.930	0.523	10.11	105.9	D 100-32

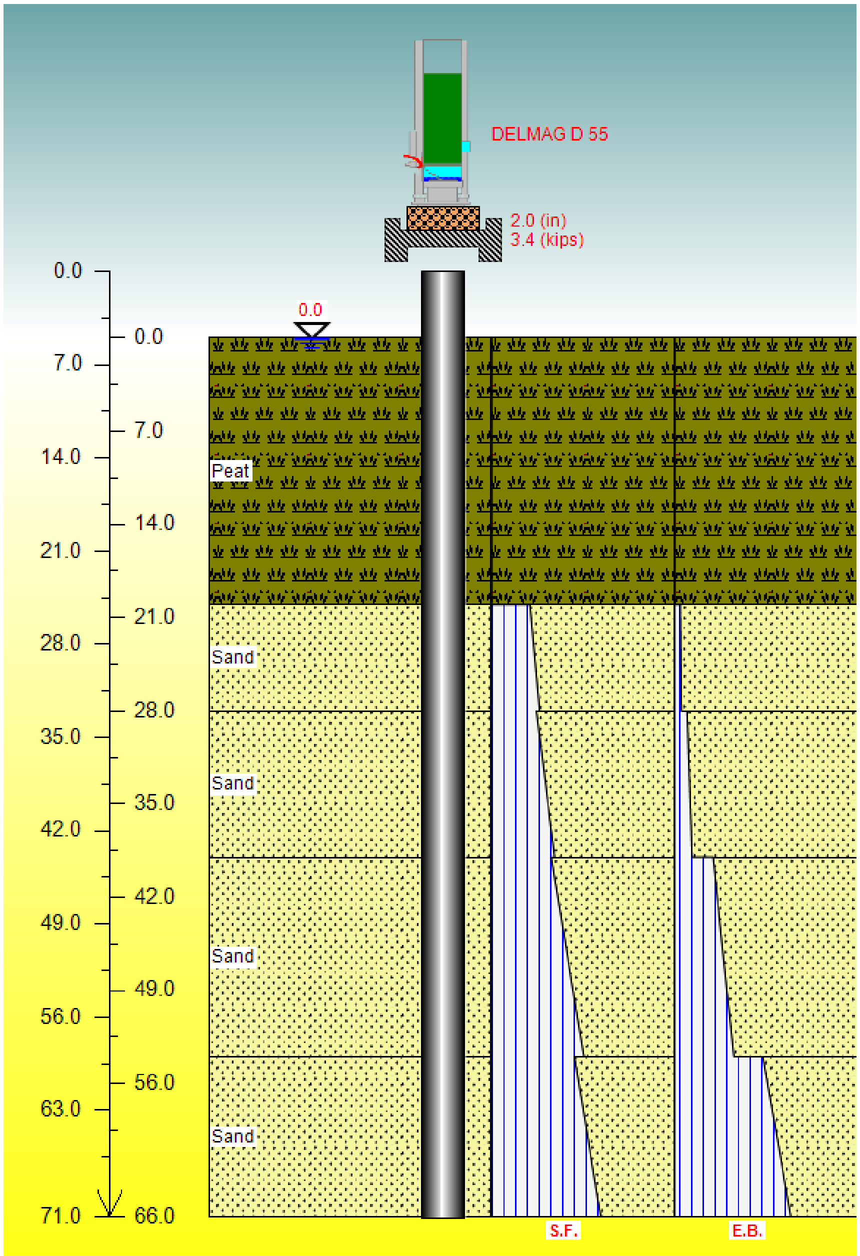
Total driving time: 67 minutes; Total Number of Blows: 2561 (starting at penetration 5.0 ft)

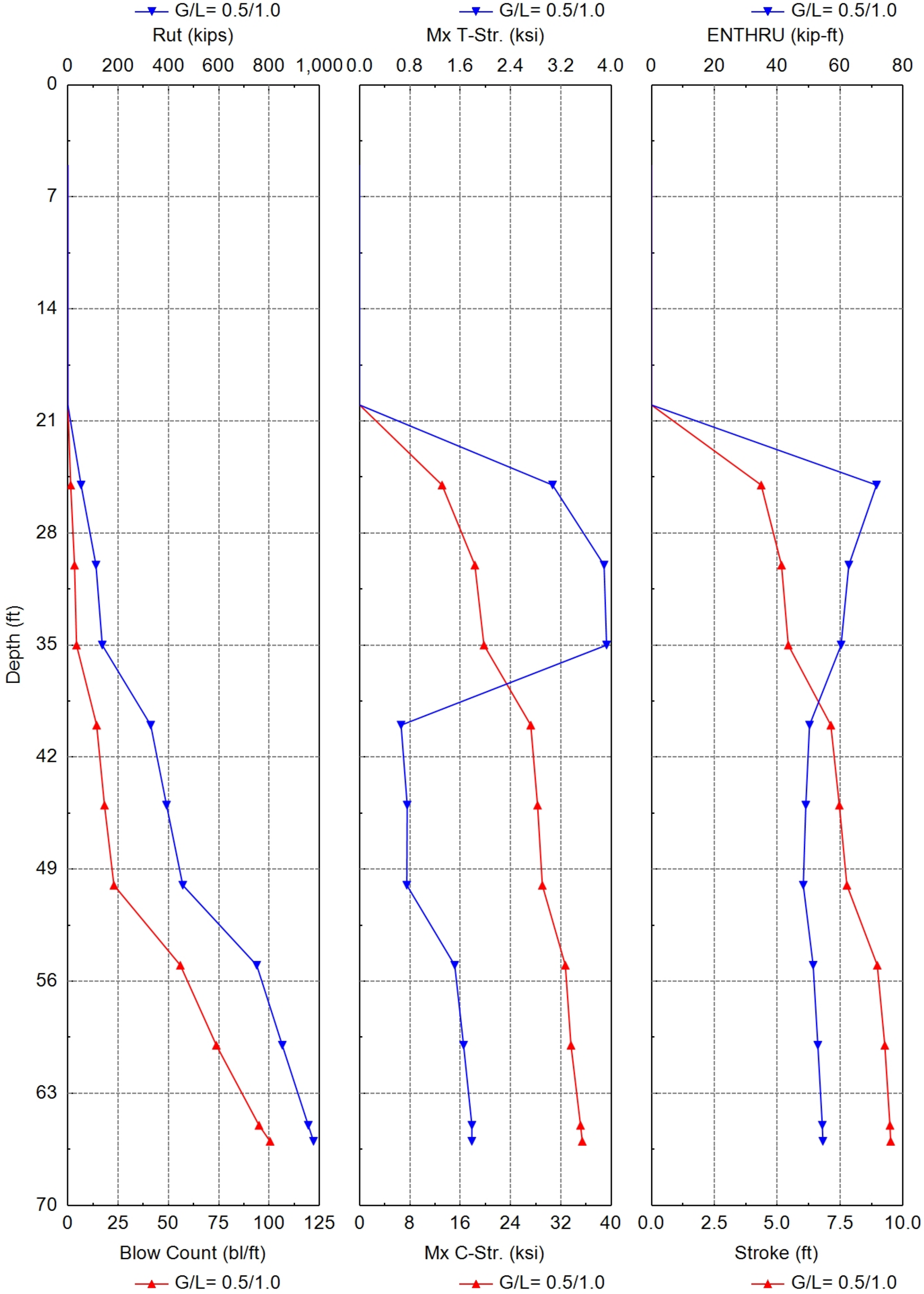
ArDOT 101124 Hwy 135 over RHC of Little River

Bent 8

24-in-diameter Steel Shell Pile

Delmag D55




G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
7
14
21
28
35
42
49
56
63
70
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
10.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
15.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
20.0	0.0	0.0	0.0	0.3	0.000	0.000	10.54	0.0	D 55
25.0	52.6	6.3	46.3	1.4	13.101	3.068	4.36	71.6	D 55
30.0	111.7	13.4	98.3	3.3	18.322	3.887	5.17	62.8	D 55
35.0	136.7	21.4	115.4	4.3	19.766	3.927	5.44	60.3	D 55
40.0	329.5	30.5	299.0	14.3	27.209	0.658	7.13	50.3	D 55
45.0	392.4	40.8	351.5	18.2	28.290	0.756	7.47	49.1	D 55
50.0	456.9	52.8	404.1	22.9	29.054	0.750	7.77	48.3	D 55
55.0	750.9	66.1	684.7	55.9	32.706	1.512	8.98	51.4	D 55
60.0	852.6	80.0	772.5	73.8	33.612	1.652	9.28	52.9	D 55
65.0	955.9	95.6	860.4	95.0	35.125	1.785	9.48	54.3	D 55
66.0	976.8	98.9	877.9	100.5	35.410	1.784	9.51	54.5	D 55

Total driving time: 33 minutes; Total Number of Blows: 1310 (starting at penetration 5.0 ft)

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 15, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER BUFFALO CREEK (SITE 6)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
POINSETT COUNTY, ARKANSAS

INTRODUCTION

Presented herein are the final results of the geotechnical investigation performed for the Hwy. 135 over Buffalo Creek replacement bridge in Poinsett County, Arkansas. This bridge is Site 6 of the ARDOT 110124 Hwy. 135 Strs. & Apprs. (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on May 26, 2023.

We understand the replacement bridge will be a prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 213 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed east of the existing bridge. Site grading will include about 12 ft of fill for the new embankments. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- ♦ Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- ♦ Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- ♦ Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, and construction considerations.

The relationship of these factors to design and construction of the Site 6 replacement bridge has been considered in developing the recommendations and considerations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface conditions in the Site 6 replacement bridge alignment were explored by drilling four (4) sample borings to 100- to 120-ft below existing grades. The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Site 6 Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
F1	347+60	5 ft Lt	35.67805573	-90.34020542	226.9	110
F2	348+65	35 ft Lt	35.67813758	-90.34055788	207±	100
F3	349+45	15 ft Rt	35.67838635	-90.34070662	214.5	110
F4	349+70	20 ft Lt	35.6783424	-90.34083024	226.0	120

The boring logs, presenting descriptions of the soil strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 14. The centerline station and offset of the boring locations and ground surface elevation, as surveyed, is also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 15.

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.

The borings were drilled with a truck-mounted CME-55 HTX rotary-drilling rig, a truck-mounted SIMCO 2800 rotary-drilling rig, and a track-mounted Diedrich D-50 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the energy conversion factor is shown on each boring log.

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

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

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 34 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 2 liquid and plastic (Atterberg) limit determinations and 28 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The Site 6 location is on Hwy. 77, approximately 250 feet southeast of the intersection of Hwy.77 and Rivervale Lane in Poinsett County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. The channel at this location is moderate with well-defined banks. The creek banks are fairly steep and are covered with thick underbrush and numerous trees. The project locale is primarily agricultural land consisting of large, flat fields. Several houses are located south of the bridge along Rivervale Lane. The existing pavements are in poor condition with numerous cracks and some full depth repairs. Surface drainage along the roadway is poor and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent Alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends through exposures of Quaternary Terrace Deposits. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

Seismic Conditions

In light of the results of the borings and the surface geology, a Seismic Site Class D (stiff soil profile) is considered applicable to the bridge location at Site 6 with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Eighth Edition 2017[2]. Given the location and AASHTO code-based values, recommended seismic parameters are summarized below.

- Seismic Site Class D
- 1.0-sec period spectral acceleration coefficient (S_1) = 0.539
- Site amplification factor at 1.0 second (F_v) = 1.5
- 1.0-sec period spectral acceleration coefficient (S_{D1}) = 0.809
- Acceleration for a short (0.2 sec) period (S_S) = 1.876
- Site amplification factor for short period (F_a) = 1.0
- Peak ground acceleration (PGA) = 1.047
- Site amplification factor at PGA (F_{PGA}) = 1.0
- A_s = 1.047

Utilizing these parameters, AASHTO LRFD Seismic Bridge Design Specifications indicate that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Site 6 location of the Hwy. 135 bridge over Buffalo Creek.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 1.047 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Appendix D as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the analyses results are shown on the generalized subsurface profile also provided in Appendix D. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix D.

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[2] AASHTO LRFD Bridge Design Specifications, 8th Edition; AASHTO; 2017.

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

Subsurface Conditions

Based on the results of the borings, the surface and near-surface soils to 2- to 18-ft are comprised of interbedded brown, reddish brown, grayish brown, dark gray, and reddish tan very loose to loose silty and clayey fine sand and very soft to firm clay, silty clay, and fine sandy clay. These soils exhibit low relative density or shear strength and high compressibility. These soils typically classify as A-3, A-4, and A-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to fair subgrade support for pavement structures.

The weak surficial soil units are underlain below 2 to 18 ft by medium dense to dense brown, gray, dark gray, tan, grayish tan, and brownish gray silty fine sand and fine to medium sand units. Some coarse sand and fine gravel are present at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was encountered in the borings at 7.7- to 18.7-ft depth in May and June 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the creek and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 6 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 18-in.-diameter steel shell piles are planned for bridge ends and 28-in.-diameter steel shell piles are planned for the interior bents. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix E. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for "static" conditions prior to a seismic event, with no liquefaction, and full soil shear strength was mobilized for the foundation soils. For the case where liquefaction occurs, the "end of earthquake" condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects. The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix F.

End Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 4) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 23 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a 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 0.5235. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 222 to El 212 was assumed.

Stability analyses results are summarized and presented graphically in Appendix G. As shown in the results, the analyses of the seismic stability of the plan 2H:1V Bent 4 end slope indicates a calculated minimum factor of safety significantly less than 1.05. However, a simplified Newmark block analysis indicates that a maximum permanent displacement of 4.3 inches is expected for the north embankment. We understand that a Newmark displacement of less than 6 inches is typically acceptable for bridges designated as “Other.”

The results of slope stability analyses utilizing residual strengths in soil zones susceptible to liquefaction triggering indicate a calculated minimum factor of safety against sliding in excess of 1.0. Consequently, the potential for flow slide instability is considered low. Given the results of the stability analyses and Newmark block analysis, the stabilities of the slope configurations are considered acceptable.

Subgrade Support

It is understood that pavement sections for the approach roads will be developed by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-3, A-4, and A-6. These classifications correlate

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[4] Slope/W 2020; GEO-SLOPE International; 2020.

with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, undercuts or improvement depths on the order of 2 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. for cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, undercutting is expected to be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 13 to 18 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix H. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

General 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow within about 100 ft of the bridge ends. An example special provision for cohesive embankment fill is provided in Appendix I.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until embankments and bridge work are completed.

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

Groundwater was encountered between 7 and 19 ft in May and June 2023. Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered. Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[5]. In the driveability analyses, the steel shell piles were assumed to be driven

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[5] GRLWEAP 2014; Pile Dynamics, Inc.

from the plan cap bottom elevation or existing grade. Graphical and tabulated results of these analyses are provided in Appendix J.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 74 ft-kips per blow for driving the steel shell piles at the end bents and at least 186 ft-kips per blow for the intermediate bents. 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 installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. 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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment 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 Map
Plate 2	Plan of Borings
Plates 3 through 14	Boring Logs
Plate 15	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Liquefaction Analysis Results
Appendix E	Nominal Pile Capacity Curves

Appendix F	Lateral Load Parameters
Appendix G	Results of Stability Analyses
Appendix H	Example SP – Woven Geotextile
Appendix I	Example SP – Cohesive Embankment Fill Special Provision
Appendix J	Driveability Analysis 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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

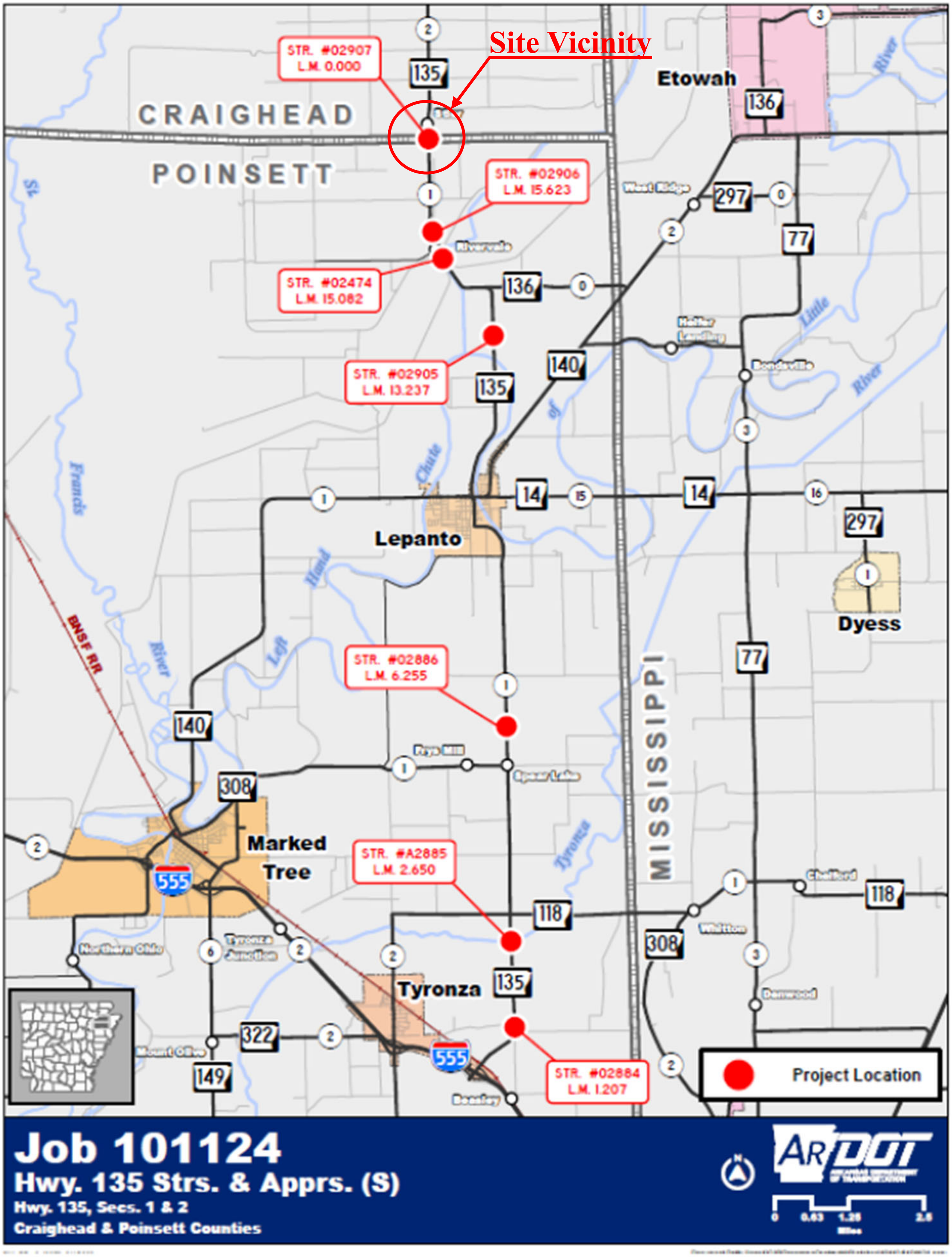
Mark E. Wyatt, P.E.
President

No. 7791
MARK E. WYATT

VMS/MEW:jw

Copies submitted: Arkansas Department of Transportation
Attn: Ms. Jessica Jackson, P.E. (1-email)
Attn: Mr. Paul Tierney (1-email)
Attn: Mr. Yongsheng Zhao, Ph.D., P.E. (1-email)

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

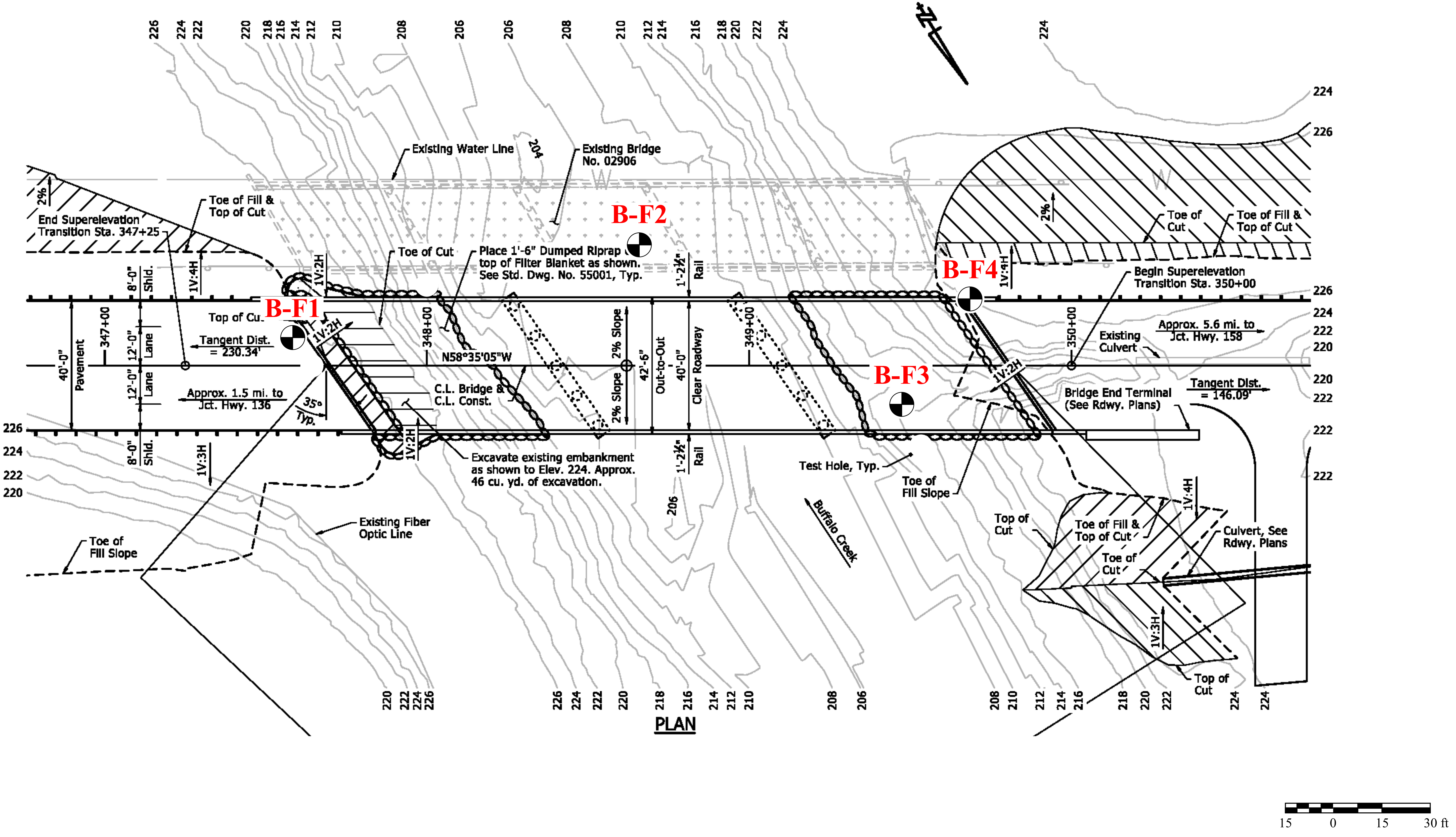


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Buffalo Creek
(Site 6/Bridge F)
Poinsett County, Arkansas

Job No. 23-031

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

PLAN of BORINGS
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Scale: As shown	Job No. 23-031	PLATE 2

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LOG OF BORING NO. F1

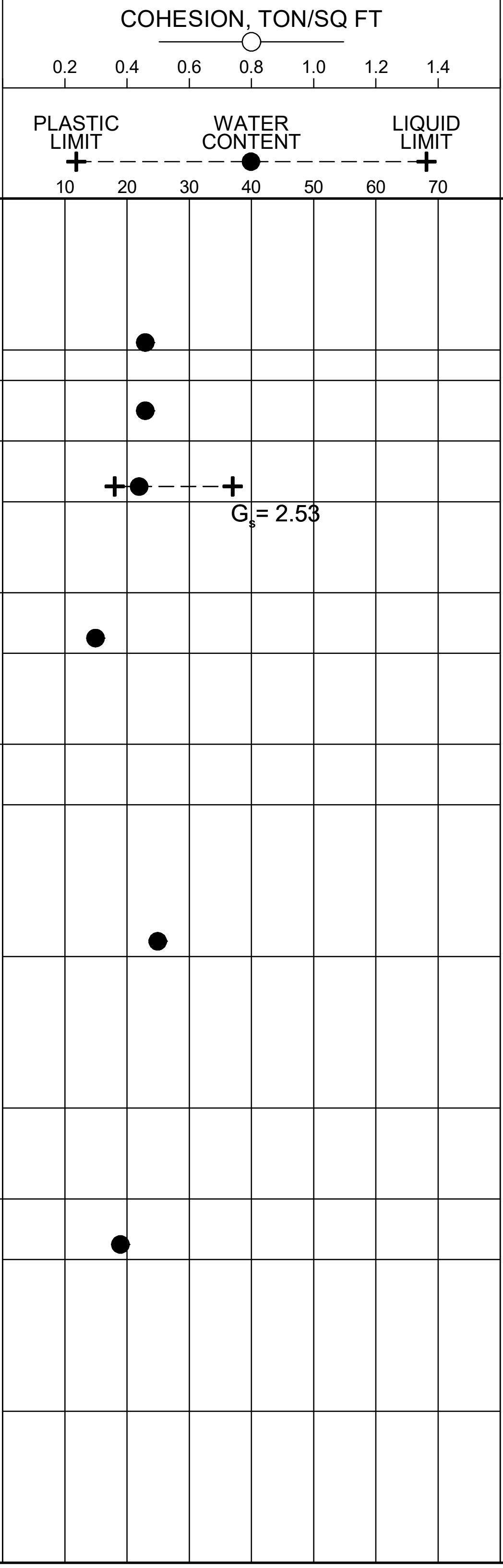
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 347+60, 5 ft Lt

SURF. EL: 226.9

DEPTH, FT	DESCRIPTION OF MATERIAL	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Loose brown silty fine sand (SM) w/clayey fine sand pockets	8		
	- medium dense from 2 to 4 ft	20		
5	- very loose below 4 ft	0/WOH		44
	Very soft grayish brown clay (CH) w/fine sand pockets	0/WOH		
10	Firm yellowish red and gray fine sandy clay (CL)	10		71
15	Medium dense tan and brown fine sand, slightly silty (SM-SP)	13		10
20	Medium dense brownish gray fine sand (SP)	23		
25		14		4
30		17		
35	Medium dense brownish gray fine to medium sand (SP)	27		2
40	- dense below 38 ft	45		
		37		



COMPLETION DEPTH: 110.0 ft
DATE: 5-17-23

DEPTH TO WATER
IN BORING: Dry to 10 ft
DATE: 5/16/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

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Consulting Engineers

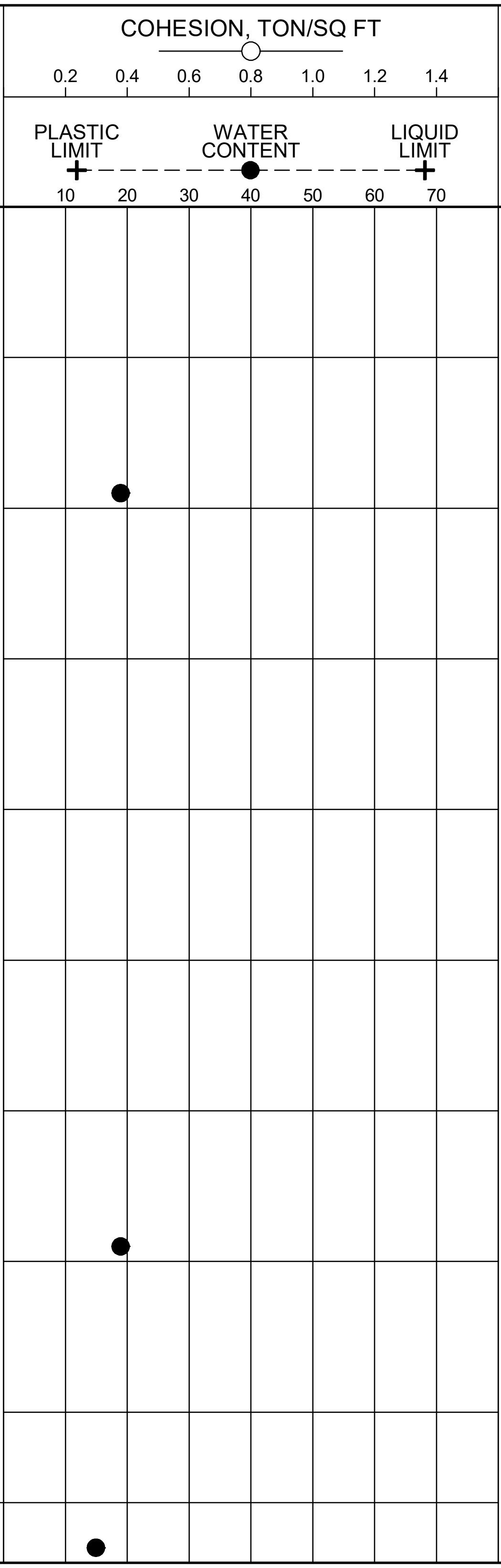
LOG OF BORING NO. F1

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 347+60, 5 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		39		
55	- slightly silty (SM-SP) with trace coarse sand and a little fine gravel below 53 ft	40		8
60		45		
65		45		
70		48		
75		42		
80		45		5
85		48		
	Dense dark brownish gray fine to coarse sand (SW) w/some fine	49		3



COMPLETION DEPTH: 110.0 ft
DATE: 5-17-23

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 5/16/2023

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

LOG OF BORING NO. F1

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 347+60, 5 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
			gravel				
95		☒		49			
100		☒		58			
			Dense brownish gray fine to medium sand (SP)				
105							
110		☒		61			
			NOTE: Drilled with SIMCO 2800 ECF= 1.19				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 5-17-23

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 5/16/2023

23-031

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L O G O F B O R I N G N O. F2

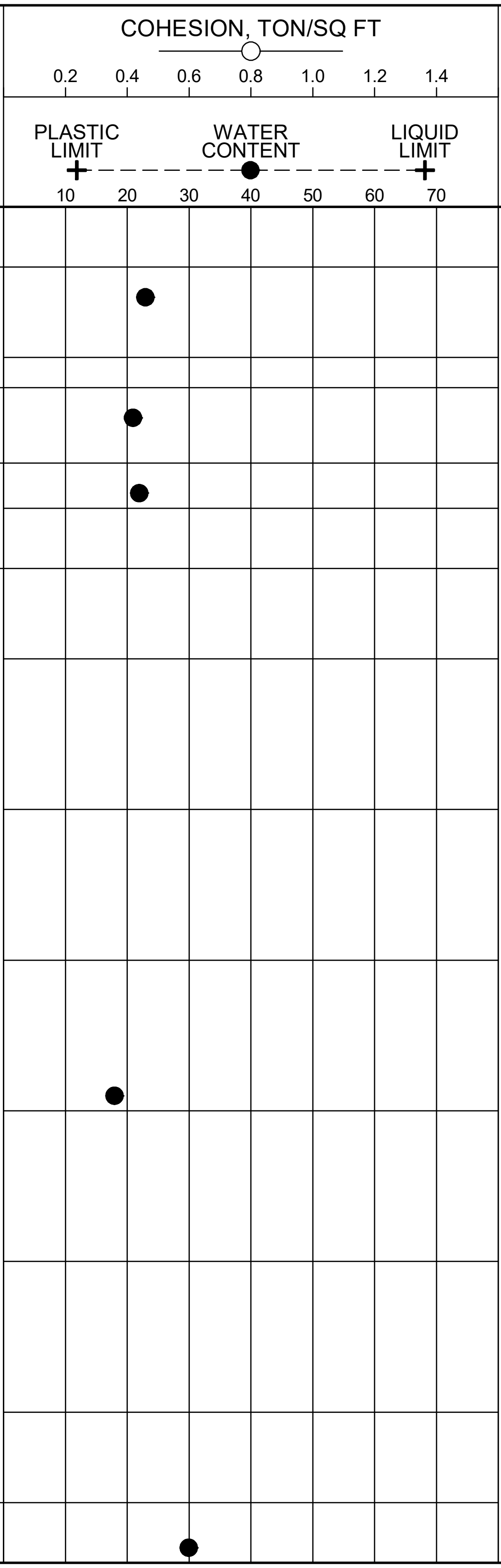
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Wash

LOCATION: Approx Sta 348+65, 35 ft Lt

SURF. EL: 207±

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Very loose dark gray clayey fine sand (SC)	3		
	Dense brown and gray fine sand (SP)	61		3
5		73		
	Dense gray and brown fine to medium sand, slightly silty (SM-SP)	64		5
10	Medium dense dark brown and gray fine to medium sand, slightly silty (SP-SM) w/organic inclusions	17		7
15	Dense brownish gray fine to medium sand (SP) w/trace coarse sand	61		
20		59		
25	- medium dense at 24 to 29 ft	36		
30	- dense below 29 ft	74		4
35		65		
40		87		
	Medium dense gray fine sand, slightly silty (SM-SP)	22		10



COMPLETION DEPTH: 100.0 ft
DATE: 6-7-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/7/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

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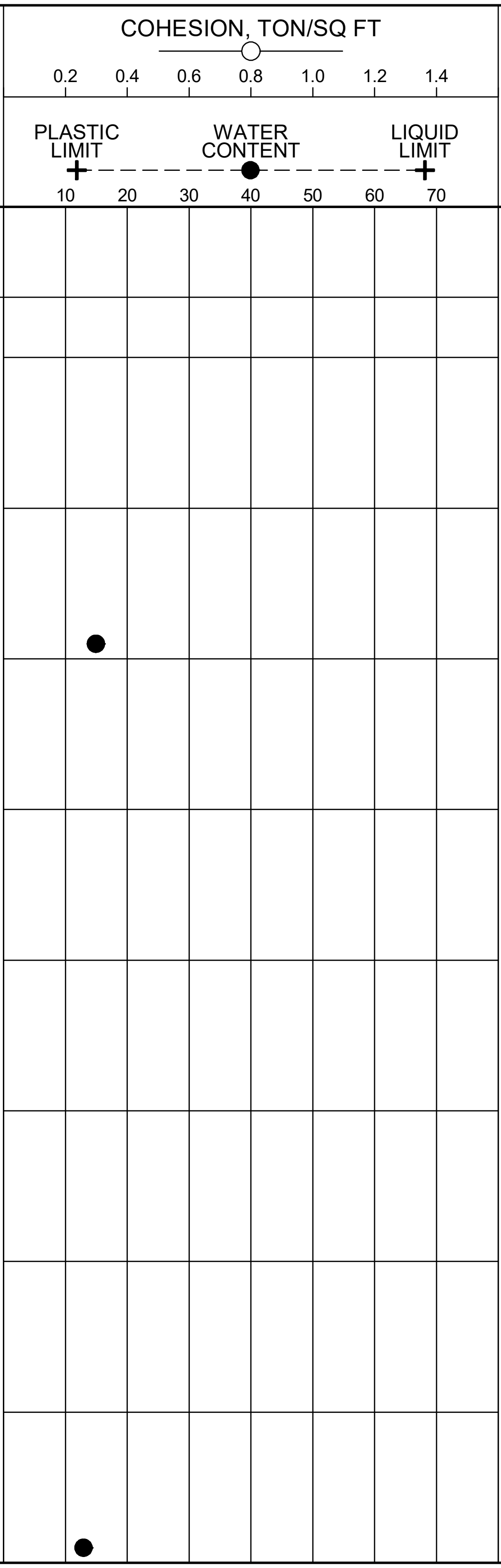
LOG OF BORING NO. F2

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Wash
LOCATION: Approx Sta 348+65, 35 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	Dense gray fine to medium sand, slightly silty (SM-SP) w/trace coarse sand	72		
	- medium dense at 53 to 58 ft			
55		35		
	- dense at 58 to 63 ft			
60	- dark gray and brown with trace fine gravel below 59 ft	47		6
	- medium dense at 63 to 78 ft - slightly clayey at 63 to 68 ft			
65		37		
70		27		
75	- with occasional fine to coarse gravel below 74 ft	33		
	- dense below 78 ft			
80		51		
85		58		
		72		6



COMPLETION DEPTH: 100.0 ft
DATE: 6-7-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/7/2023

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

LOG OF BORING NO. F2

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Wash

LOCATION: Approx Sta 348+65, 35 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2 0.4 0.6 0.8 1.0 1.2 1.4) / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT (10 20 30 40 50 60 70)	- No. 200 %
95	▨	☒		40			
100	▨	☒		90			
105							
110							
115							
120							
125							
130							

NOTE 1: Drilled from bridge deck.
NOTE 2: Deck to water: 18.1 ft
NOTE 3: Deck to mudline: 21.6 ft
NOTE 4: Set 45 ft HDX Casing.
NOTE 5: Drilled with CME-55 HTX
ECF= 1.28

COMPLETION DEPTH: 100.0 ft
DATE: 6-7-23

DEPTH TO WATER
IN BORING: NA

DATE: 6/7/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

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

LOG OF BORING NO. F3

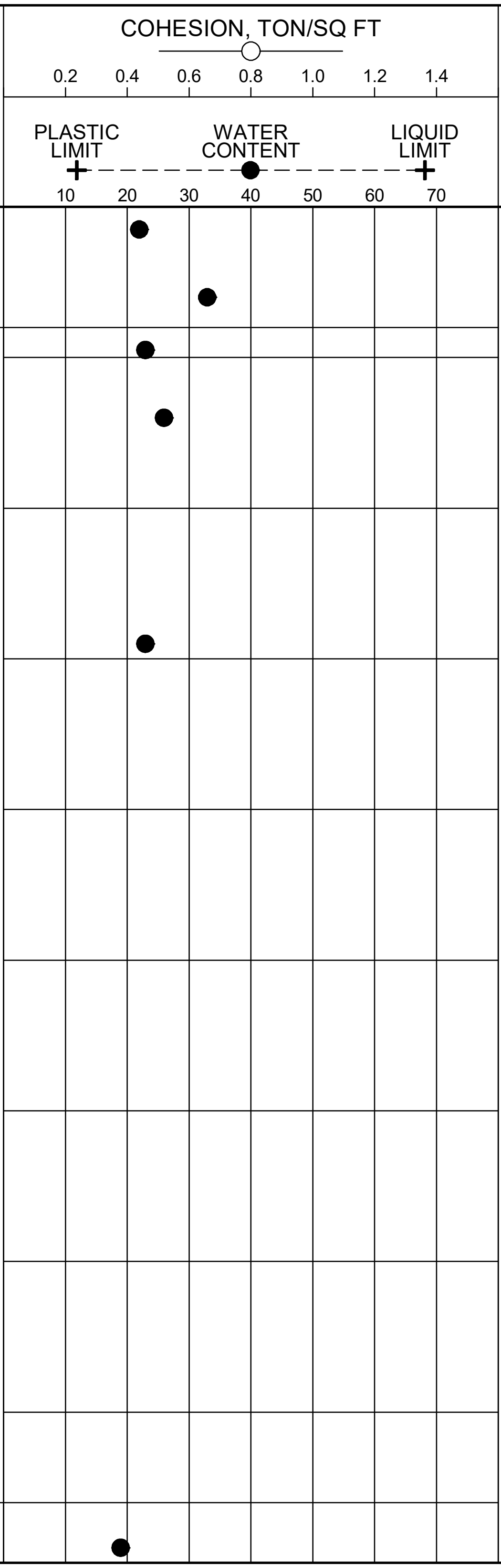
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 349+45, 15 ft Rt

SURF. EL: 214.5

DEPTH, FT	DESCRIPTION OF MATERIAL	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown silty clay (CL) w/silty fine sand seams	7		
		7		
5	Very loose to loose grayish brown fine sand, slightly silty (SM-SP)	6		
	- loose at 6 to 8 ft	11		7
	- medium dense, grayish tan below 8 ft	16		
15		17		9
20		29		
25		33		
30	- greenish gray and tan with occasional dark gray nodules at 28 to 38 ft	30		
35		29		
40	- grayish tan below 38 ft	30		
	Medium dense grayish tan fine to medium sand (SP)	31		4



COMPLETION DEPTH: 110.0 ft
DATE: 6-21-23

DEPTH TO WATER
IN BORING: 7.7 ft

DATE: 6/21/2023

23-031

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

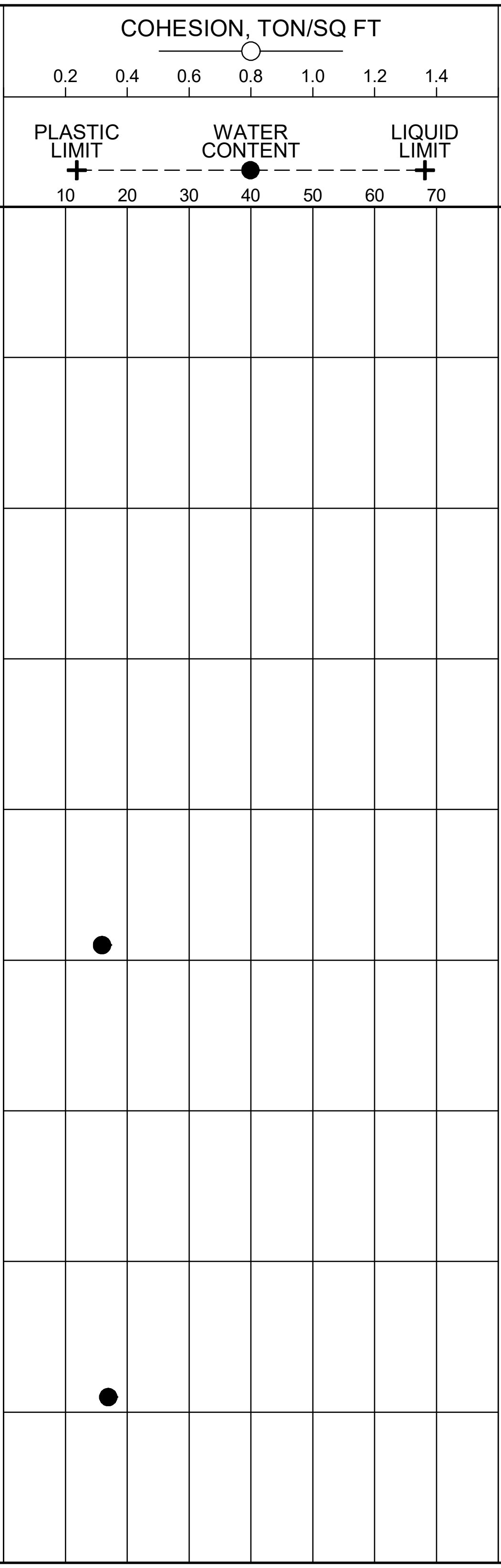
L O G O F B O R I N G N O. F3

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 349+45, 15 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N₆₀, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒		33		
55		☒	- slightly silty (SM-SP) below 53 ft	36		
60		☒		33		
65		☒		34		
70		☒	- dense below 68 ft	43		6
75		☒		54		
80		☒		57		
85		☒	- with more medium sand (SM-SW) below 83 ft	63		6
		☒		63		



COMPLETION DEPTH: 110.0 ft
DATE: 6-21-23

DEPTH TO WATER
IN BORING: 7.7 ft

DATE: 6/21/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

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

LOG OF BORING NO. F3

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Approx Sta 349+45, 15 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
95		☒	- with trace coarse sand at 93 to 98 ft	66			
100		☒		66			
105							
110		☒		69			
			NOTE: Drilled with Diedrich D-50 ECF= 1.43				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-21-23

DEPTH TO WATER
IN BORING: 7.7 ft

DATE: 6/21/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

23-031

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

L O G O F B O R I N G N O. F4

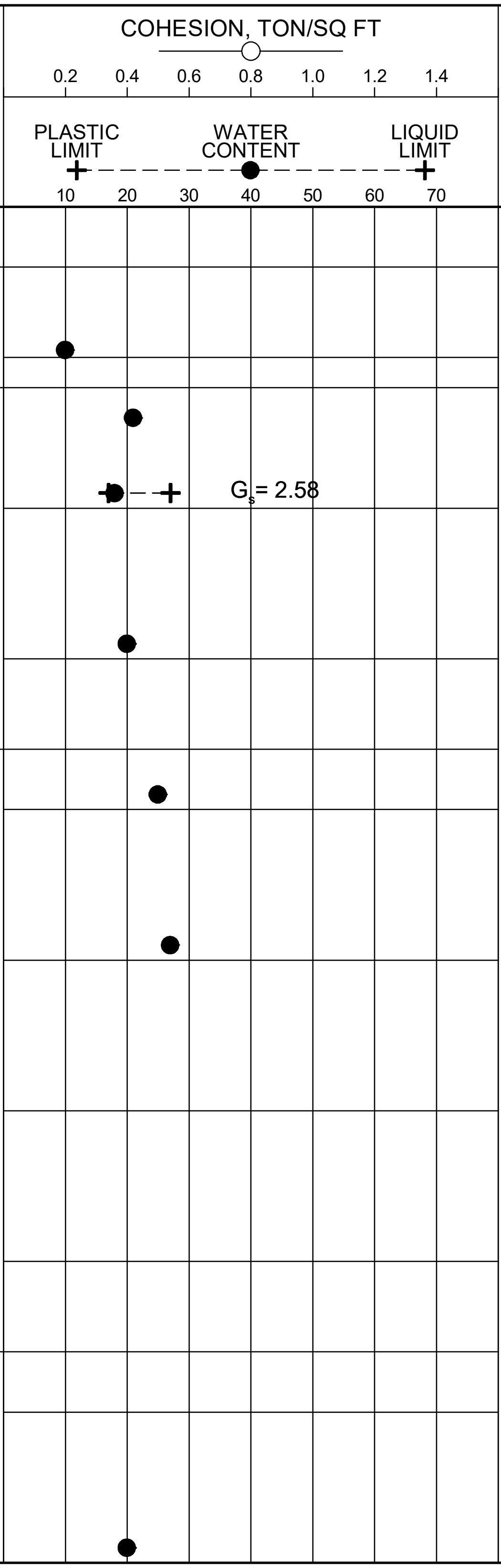
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 349+70, 20 ft Lt

SURF. EL: 226.0

DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Loose brown silty fine sand (SM) w/organics (possible fill)	7		
Loose reddish brown silty fine sand (SM) w/clay pockets	7		
	7		17
Very loose brown clayey fine sand (SC)	4		
- loose below 8 ft	6		47
- reddish tan and light brownish gray below 13 ft	11		
Medium dense brown fine sand, slightly silty (SM-SP)	15		5
- brownish gray below 23 ft	17		8
	24		
	30		
Dense brownish gray fine to medium sand (SM-SP)	37		
	49		4



DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40

COMPLETION DEPTH: 120.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 16.7 ft

DATE: 5/18/2023

LGBNEW 23-031 BRIDGE F.GPJ 7-28-23

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

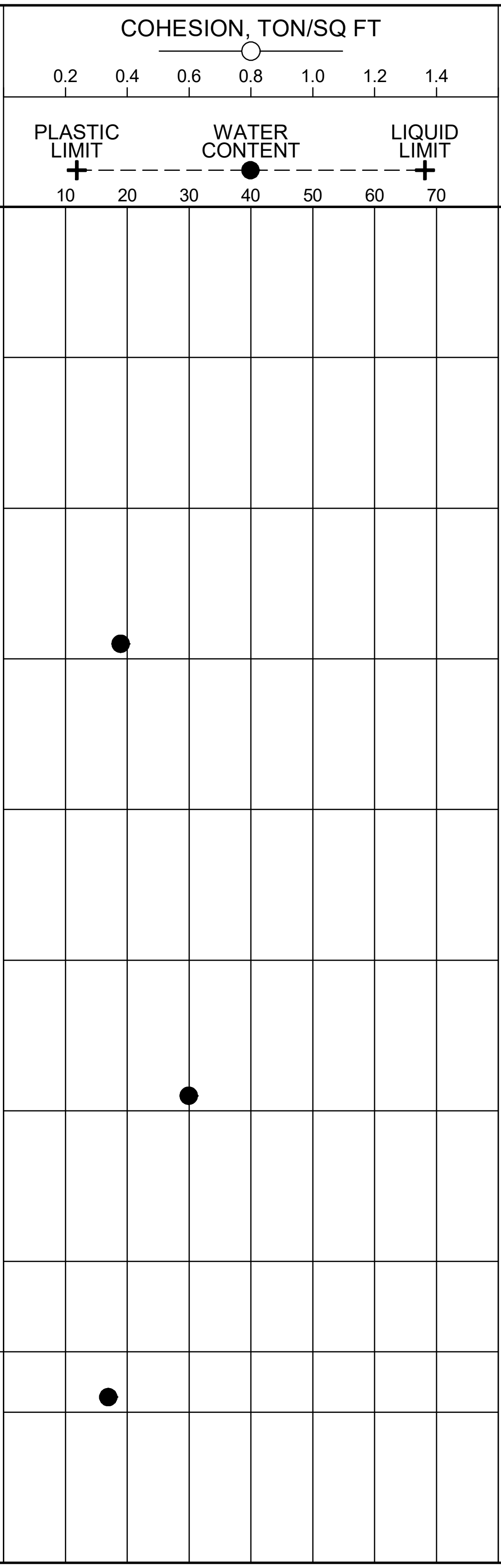
LOG OF BORING NO. F4

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 349+70, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒		48		
55		☒		52		
60		☒	- with more medium sand below 58 ft	55		2
65		☒		71		
70		☒		65		
75		☒	- with occasional organic inclusions below 73 ft	55		4
80		☒	- dark brownish gray below 78 ft	57		
85		☒	Dense gray fine to coarse sand (SW) w/trace fine gravel	57		2
		☒		58		



COMPLETION DEPTH: 120.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 16.7 ft

DATE: 5/18/2023

L O G O F B O R I N G N O. F4

101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 349+70, 20 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
95		☒		55			
100		☒		56			
105							
110		☒		57			
115							
120		☒		56			
125			NOTE: Drilled with SIMCO 2800 ECF: 1.19				
130							

COMPLETION DEPTH: 120.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 16.7 ft

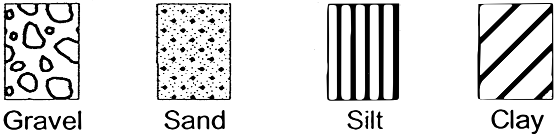
DATE: 5/18/2023

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

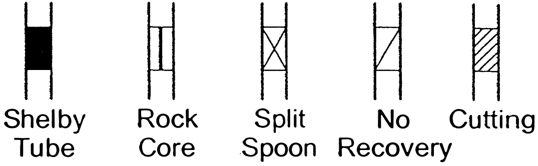
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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 9-26-02

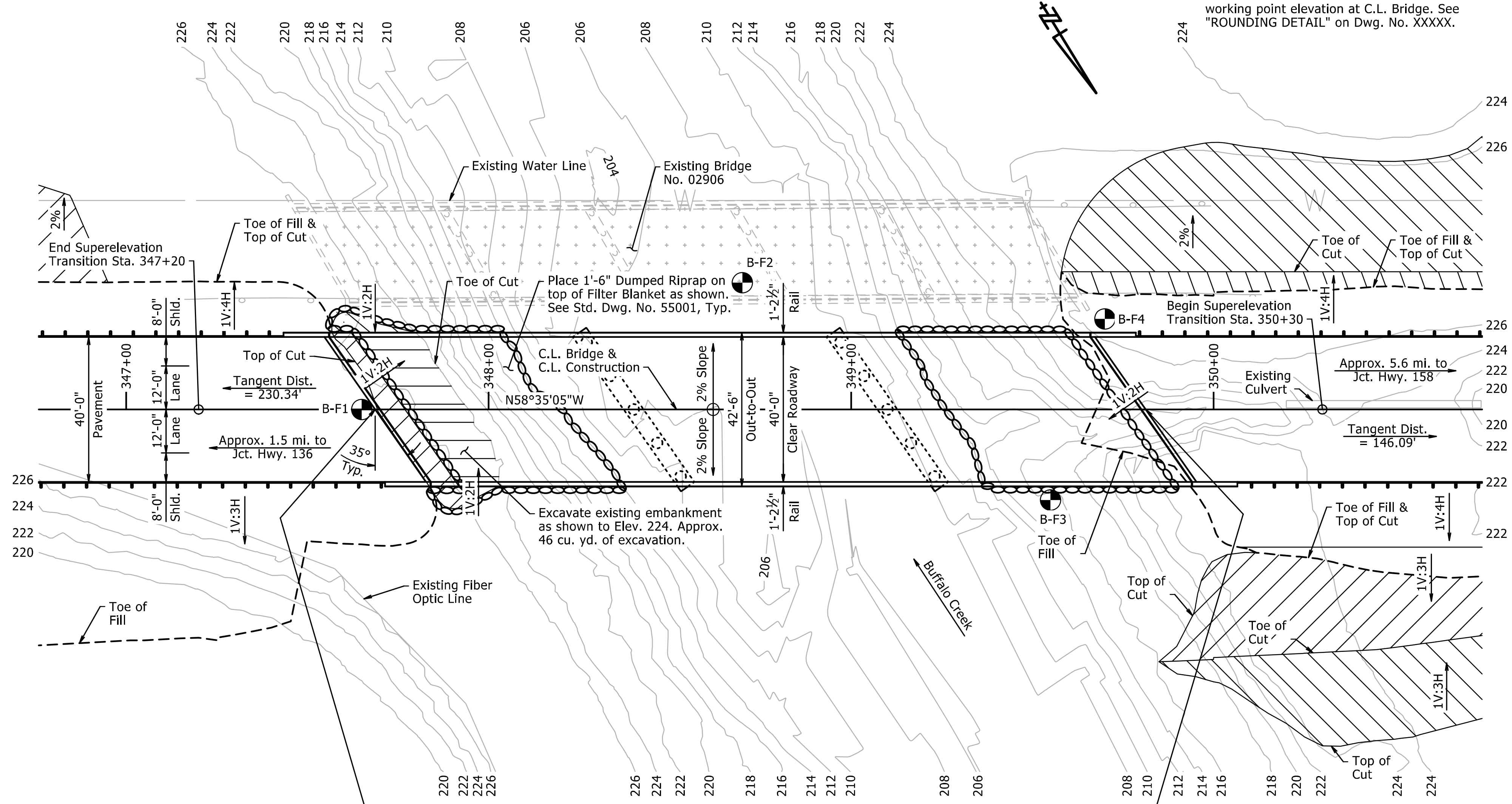
APPENDIX A

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	230	356
		07652	LAYOUT			66676

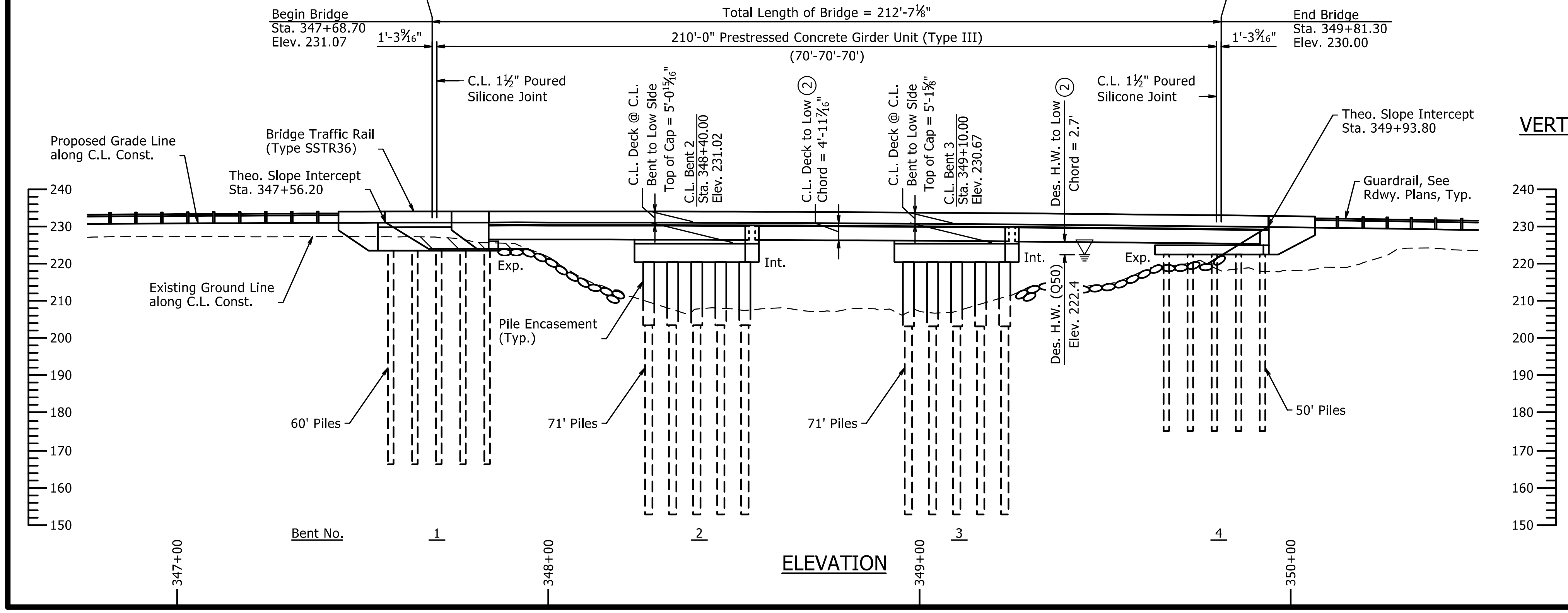
For R/W Data, See Roadway Plans.

For Soil Boring information, see Dwg. No. XXXXX.

Note: Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "ROUNDING DETAIL" on Dwg. No. XXXXX.



PLAN



ELEVATION

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

SEISMIC ZONE: 4 S_{D1} = 0.809 SITE CLASS: D

SEISMIC OPERATIONAL CLASS: OTHER

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 6,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

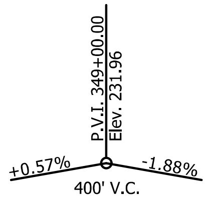
BORING LOGS: Boring logs may be obtained from the Construction Contract Development Section of the Program Management Division.

STEEL SHELL PILING: Piling in Bents 1 and 4 shall be 18" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of 397 tons per pile. Piling in Bents 2 and 3 shall be 28" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of 688 and 950 tons per pile, respectively. All piling shall be driven with an approved air, steam, or diesel hammer to a minimum tip elevation of 164 and 173 or lower at Bents 1 and 4, respectively, and to a minimum tip elevation of 153 or lower at Bents 2 and 3. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (__" Dia.)".

For additional General Notes, see Dwg. No. XXXXX.

Note: Use Type 5 Special Approach Gutters and Type C2 Approach Slabs (width = 24'-0") at both ends of bridge. See Dwg. Nos. XXXXX & 55040C2, respectively. Eliminate or modify Type Special Approach Gutter curb section to fit bridge end terminal. No additional payment will be made for this work.



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along C.L. Construction

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	① NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	8,370	222.4	222.4
BASE	100	9,220	222.6	222.6
EXTREME	500	11,780	223.3	223.3
OVERTOPPING	>500	---	---	---

① Unconstricted water surface elevation without structure or roadway approaches.

② Proposed Low Bridge Chord Elev. = 225.06 feet at Station 349+80.00

100 yr. backwater elevation for existing structure = 222.7 feet
Drainage Area = 297.0 sq. miles
Historical H.W. Elev. = 221.9 feet

PRELIMINARY SUBJECT TO REVISION

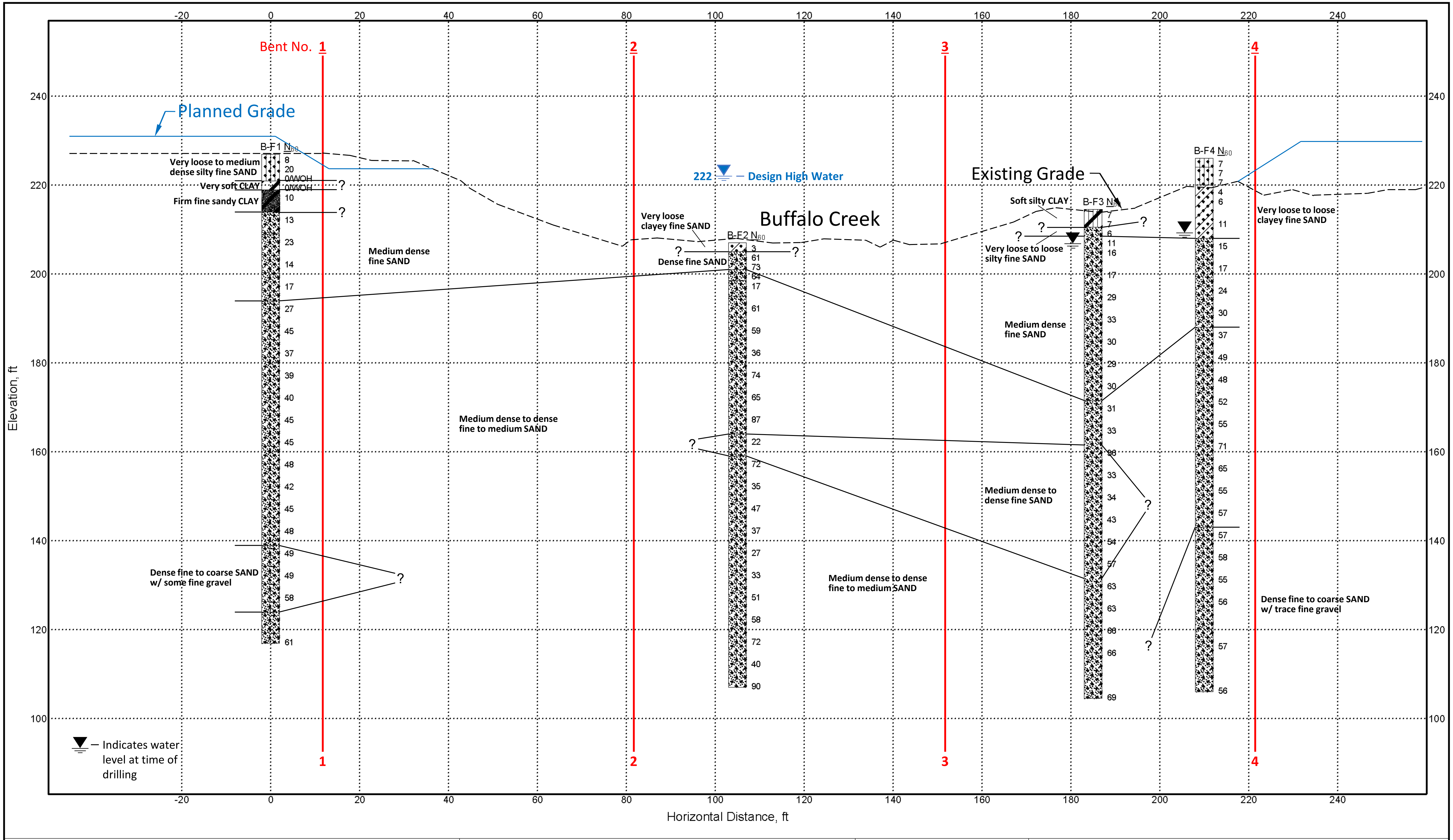
SHEET 1 OF 2
LAYOUT OF BRIDGE
HWY. 135 OVER BUFFALO CREEK
HWY. 135 STRS. & APPRS. (S)
POINSETT COUNTY
ROUTE 135 SEC. 1
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: LDG DATE: 10-06-2022 FILENAME: b101124x6_l1.dgn
CHECKED BY: CAW DATE: 11-16-2022 SCALE: 1" = 20'
DESIGNED BY: LDG DATE: 10-05-2022
BRIDGE NO. 07652 DRAWING NO. 66676

USER: CTAUSER
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x6_l1.dgn
PLOTTED: 8/24/2023 4:35:28 PM SCALE: 40.0000 ' / In.

G:\22110001_101124\TRANSP\dgn\bridge\b101124x6_l1.dgn 8/24/2023 4:35:28 PM

APPENDIX B



Grubbs, Hoskyn, Barton & Wyatt, LLC

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

SCALE:
1'' = 20' Horizontal
1'' = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Buffalo Creek (Site 6)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
F1	4.5-5.5	22	---	---	---	---	---	---	---	100	---	---	44	SM	A-4
F1	9-10	22	37	18	19	100	100	100	100	100	100	100	71	CL	A-6
F1	14-15	15	---	---	---	---	---	---	---	100	---	---	10	SM-SP	A-3
F1	24-25	25	---	---	---	100	100	100	100	100	100	85	4	SP	A-3
F1	34-35	19	---	---	---	100	100	100	100	100	100	62	2	SP	A-3
F1	54-55	19	---	---	---	100	100	100	84	73	66	28	8	SM-SW	A-1-b
F1	79-80	18	---	---	---	100	100	94	88	80	73	24	5	SM-SW	A-1-b
F1	89-90	15	---	---	---	100	100	100	98	77	54	23	3	SW	A-1-b
F2	2.5-3.5	23	---	---	---	---	---	---	---	100	---	---	3	SP	A-3
F2	6.5-7.5	21	---	---	---	100	100	100	100	100	100	62	5	SM-SP	A-3
F2	9-10	22	---	---	---	---	---	---	---	99	---	---	7	SM-SP	A-3
F2	29-30	18	---	---	---	100	100	100	100	100	100	40	4	SP	A-1-b
F2	44-45	30	---	---	---	100	100	100	100	100	100	94	10	SM-SP	A-3
F2	59-60	15	---	---	---	100	100	100	99	96	90	39	6	SM-SP	A-1-b
F2	89-90	13	---	---	---	100	100	100	91	88	82	30	6	SM-SW	A-1-b
F3	4.5-5.5	23	---	---	---	---	---	---	---	100	---	---	7	SM-SP	A-3
F3	14-15	23	---	---	---	100	100	100	100	100	100	84	9	SM-SP	A-3
F3	44-45	19	---	---	---	100	100	100	100	100	99	43	4	SP	A-1-b
F3	69-70	16	---	---	---	100	100	100	100	99	97	58	6	SM-SP	A-3
F3	84-85	17	---	---	---	100	100	100	100	99	97	24	6	SM-SW	A-1-b
F4	4.5-5.5	10	---	---	---	---	---	---	---	100	---	---	17	SM	A-2-4
F4	9-10	10	27	17	10	100	100	100	100	100	100	99	47	SC	A-4

SUMMARY of CLASSIFICATION TEST RESULTS

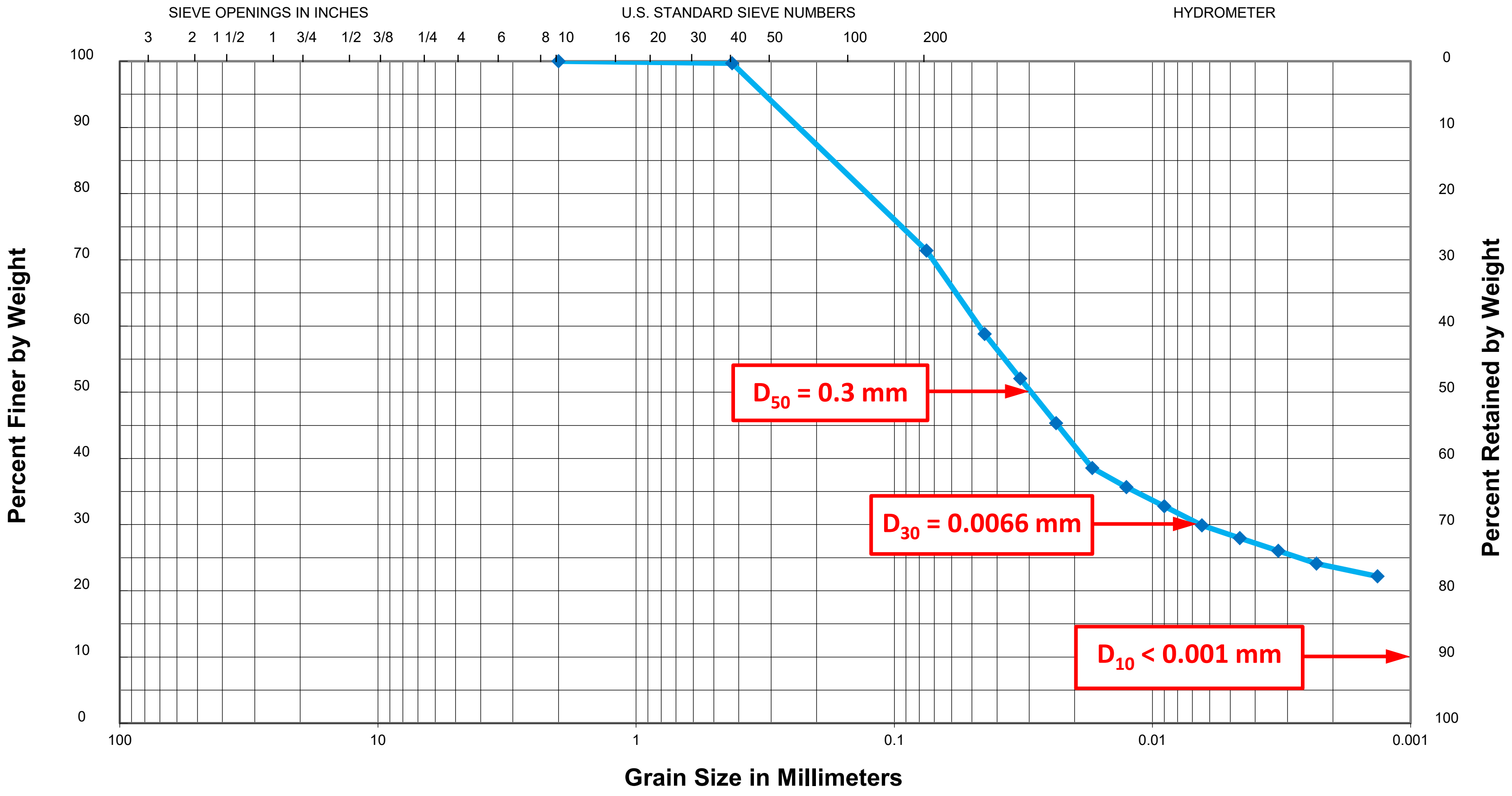
PROJECT: 101124 Hwy. 135 over Buffalo Creek (Site 6)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
F4	19-20	25	---	---	---	---	---	---	---	100	---	---	6	SM-SP	A-3
F4	24-25	27	---	---	---	---	---	---	---	99	---	---	8	SM-SP	A-3
F4	44-45	20	---	---	---	100	100	100	99	99	98	76	5	SM-SP	A-3
F4	59-60	19	---	---	---	100	100	100	100	100	100	22	2	SP	A-1-b
F4	74-75	21	---	---	---	100	100	100	100	100	99	36	4	SP	A-1-b
F4	84-85	17	---	---	---	100	100	100	96	90	78	15	2	SW	A-1-b

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.3 mm

D_{30} = 0.0066 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

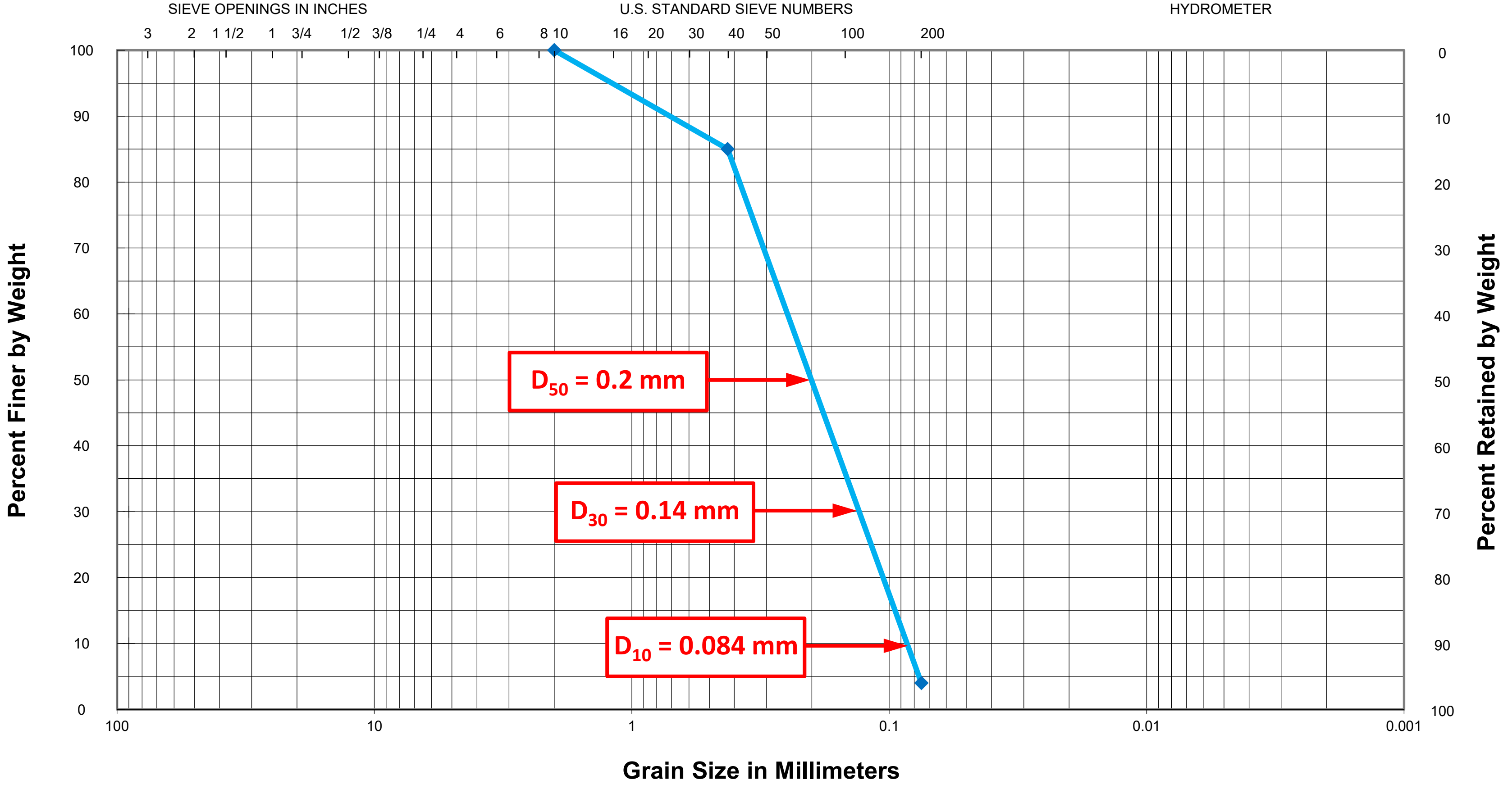
Sample: Boring F1, 9-10 ft; LL= 37, PL=18, PI=19
Description: Yellowish red and gray fine sandy CLAY

USCS Classification = CL
AASHTO Classification = A-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



<table>
<tr><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

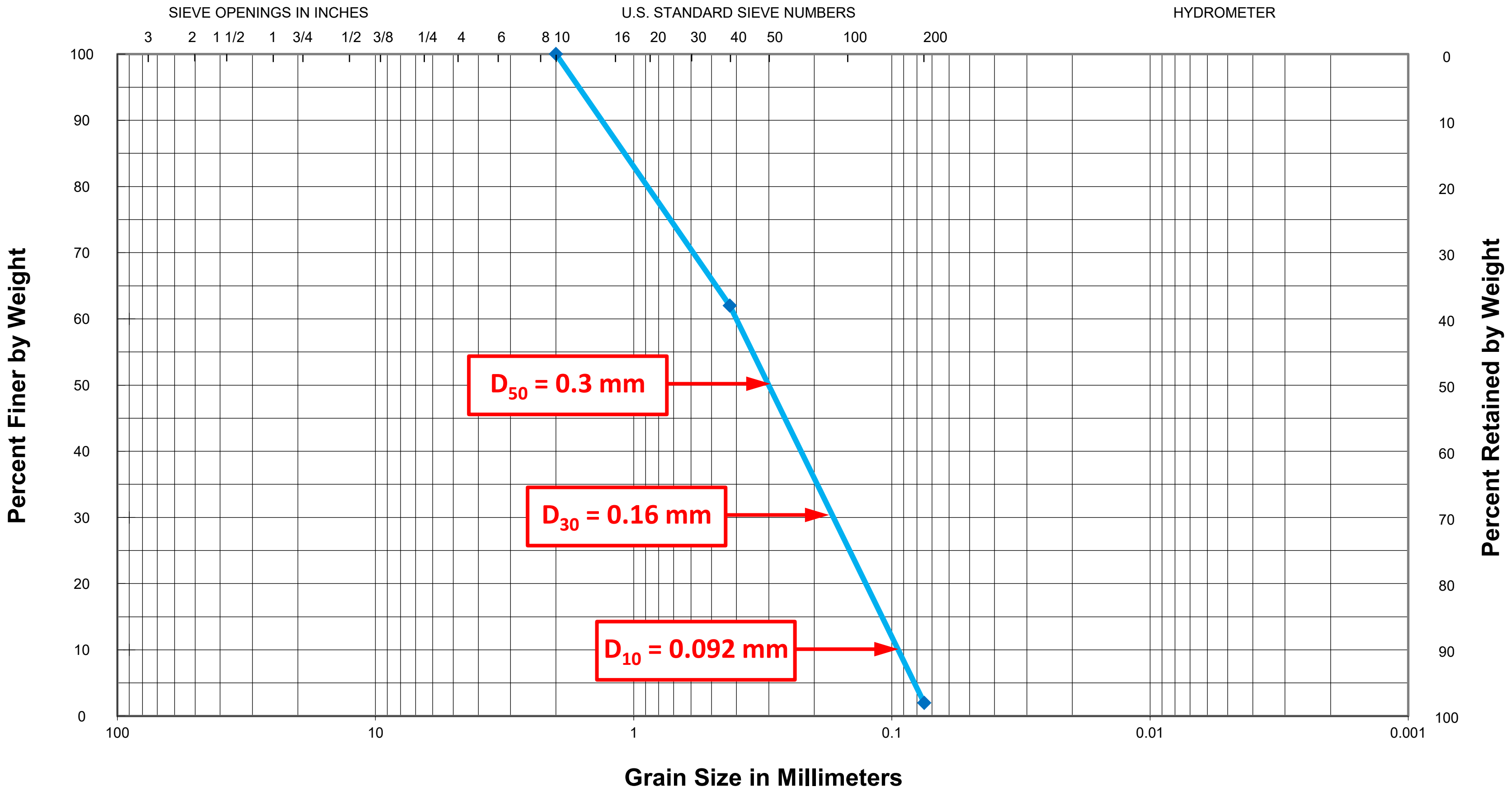
Sample: Boring F1, 24-25 ft
Description: Brownish gray fine SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

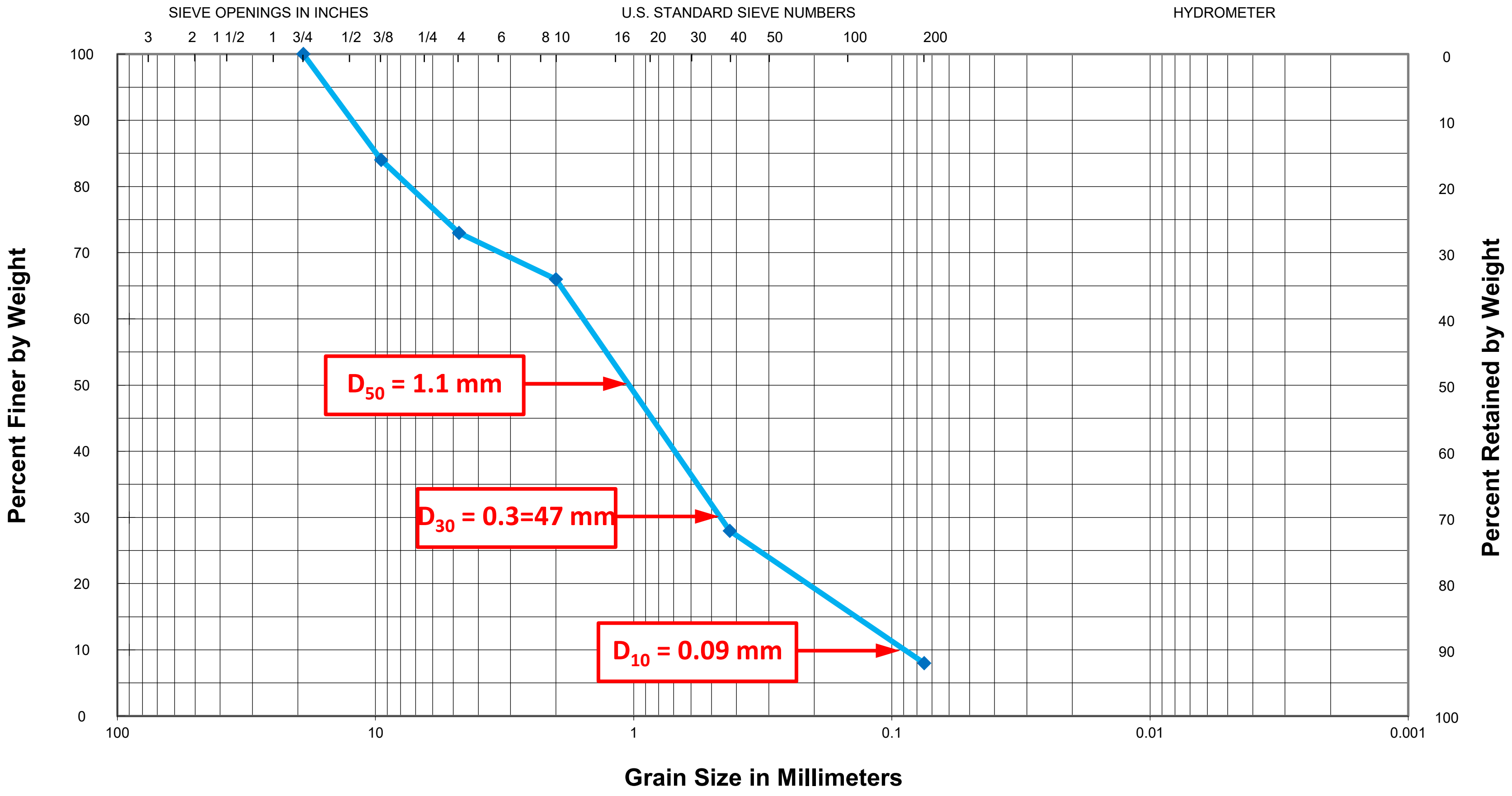
Sample: Boring F1, 34-35 ft
Description: Brownish gray fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

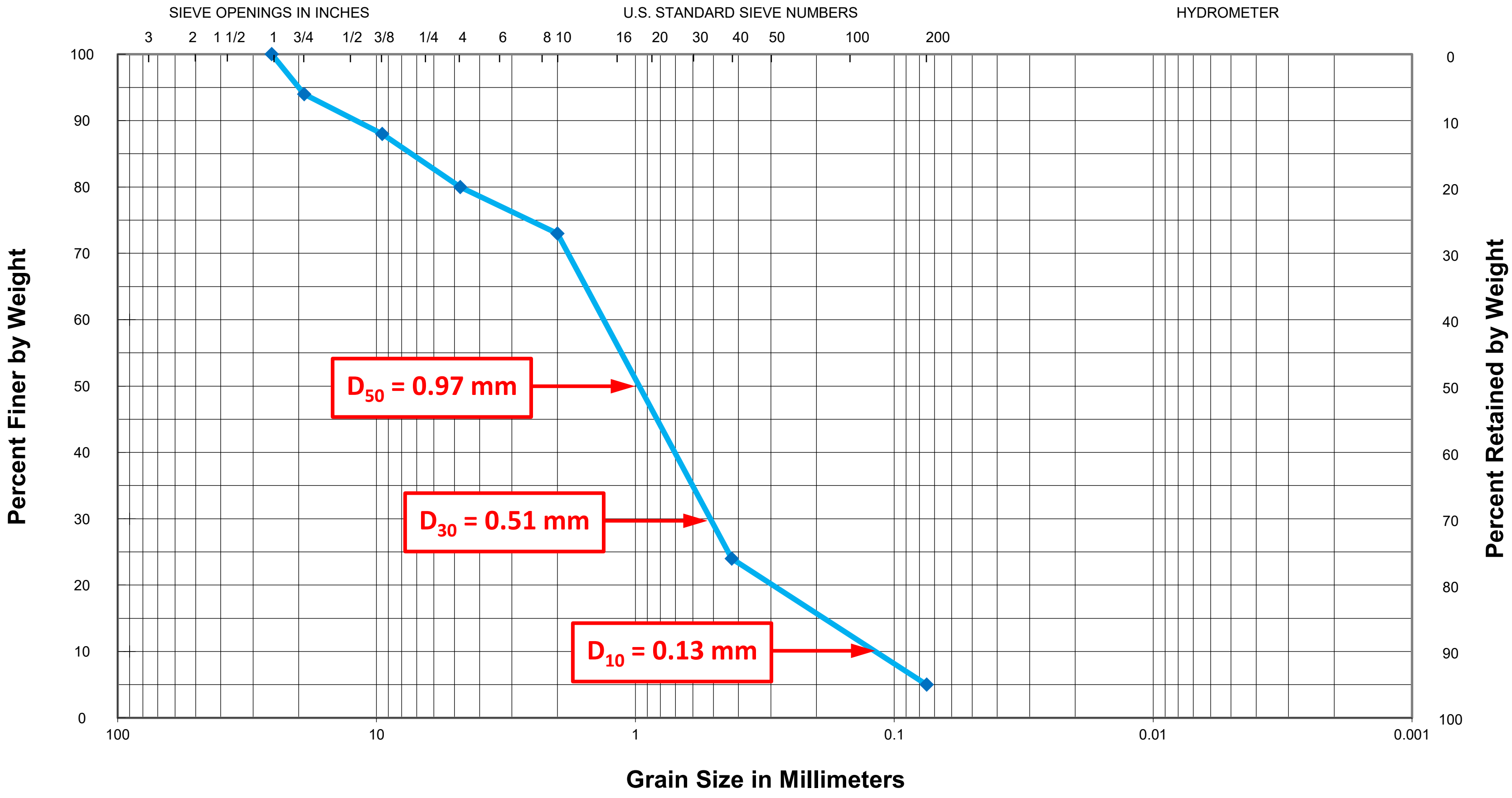
Sample: Boring F1, 54-55 ft
Description: Brownish gray fine to medium SAND, slightly silty w/ trace coarse sand and a little fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.97 mm

D_{30} = 0.51 mm

D_{10} = 0.13 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

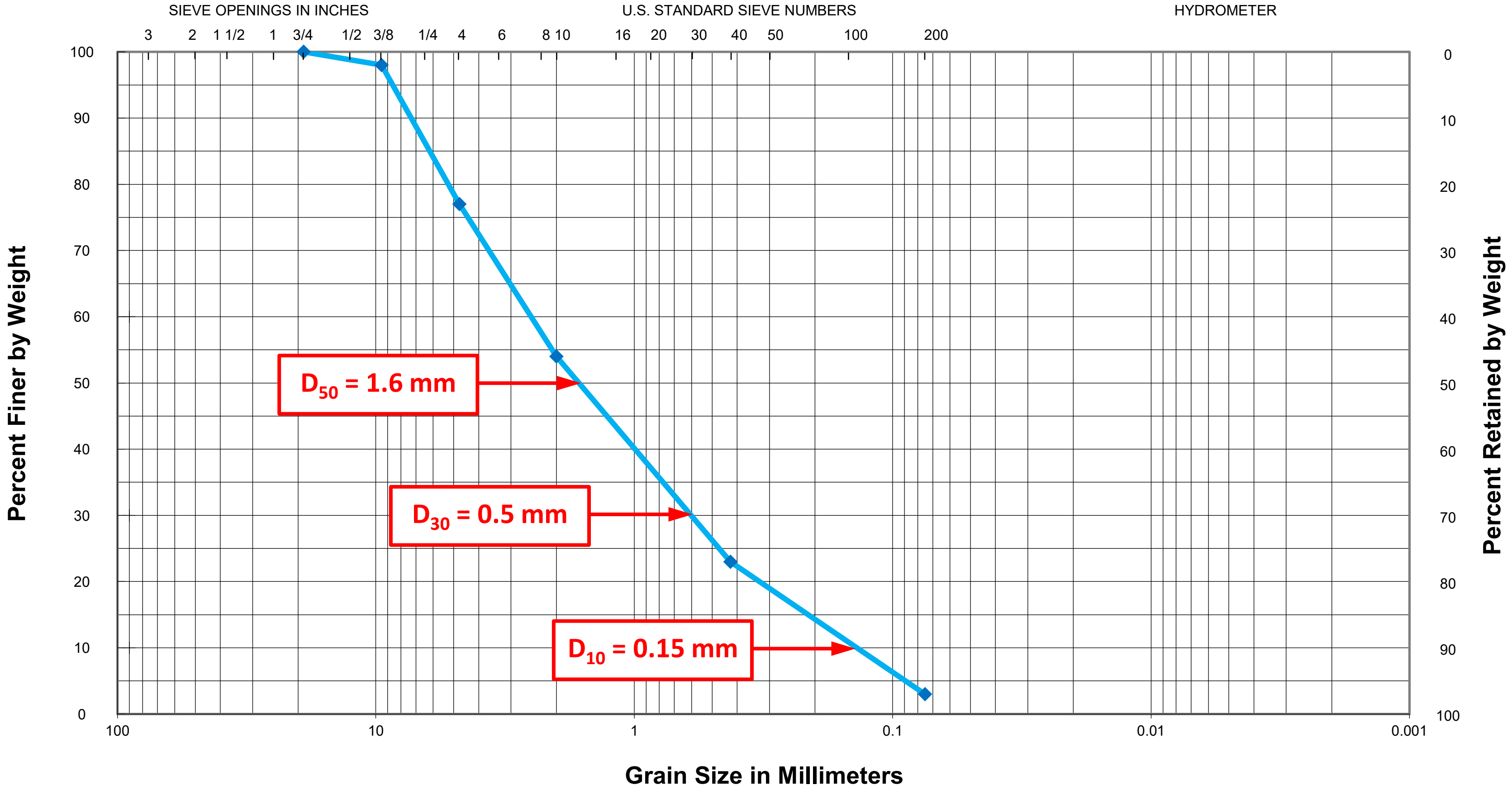
Sample: Boring F1, 79-80 ft
Description: Brownish gray fine to medium SAND w/ trace coarse sand and a little fine to coarse gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 1.6 mm

D_{30} = 0.5 mm

D_{10} = 0.15 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

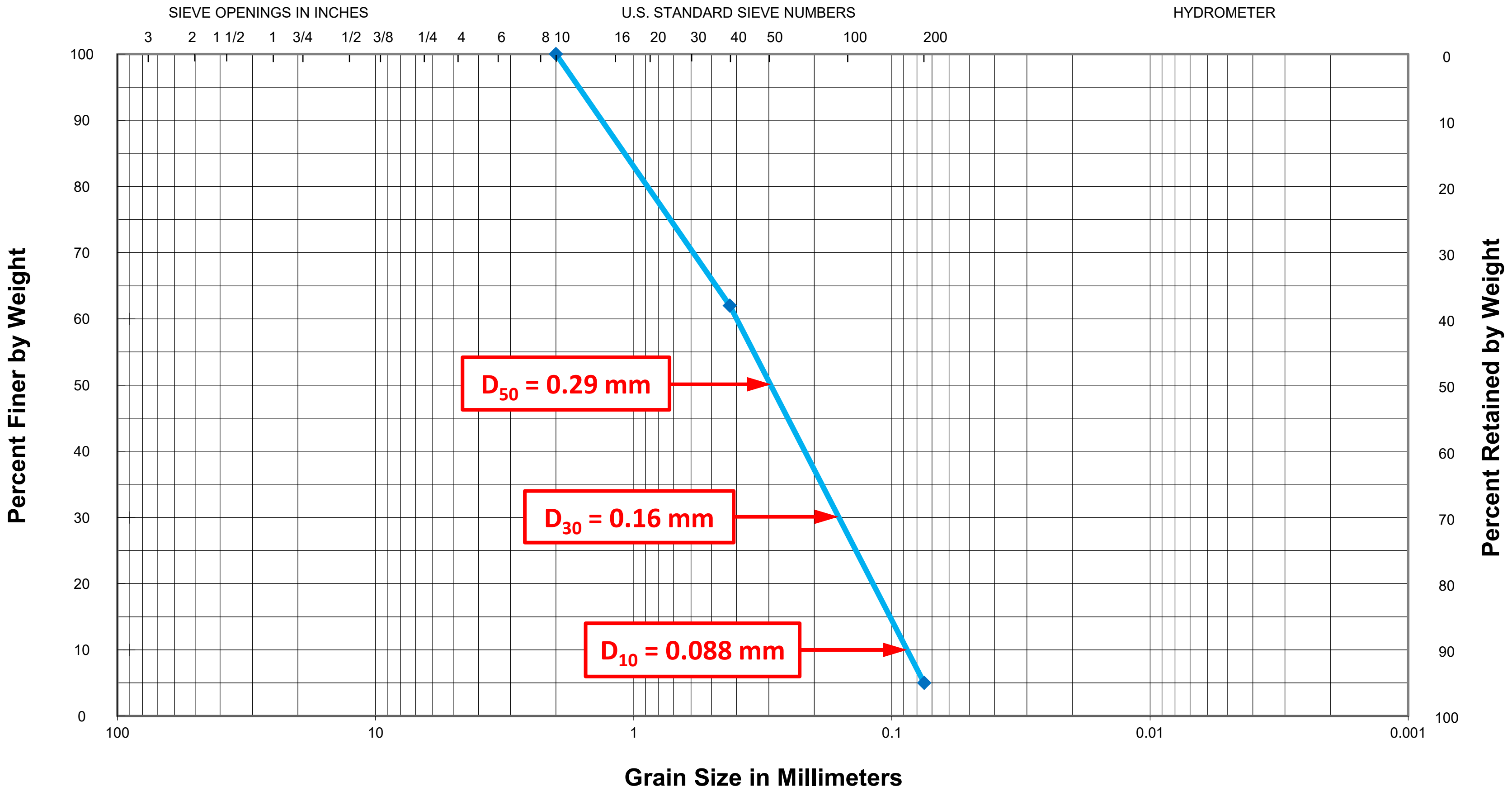
Sample: Boring F1, 89-90 ft
Description: Dark brownish gray fine to coarse SAND w/ some fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

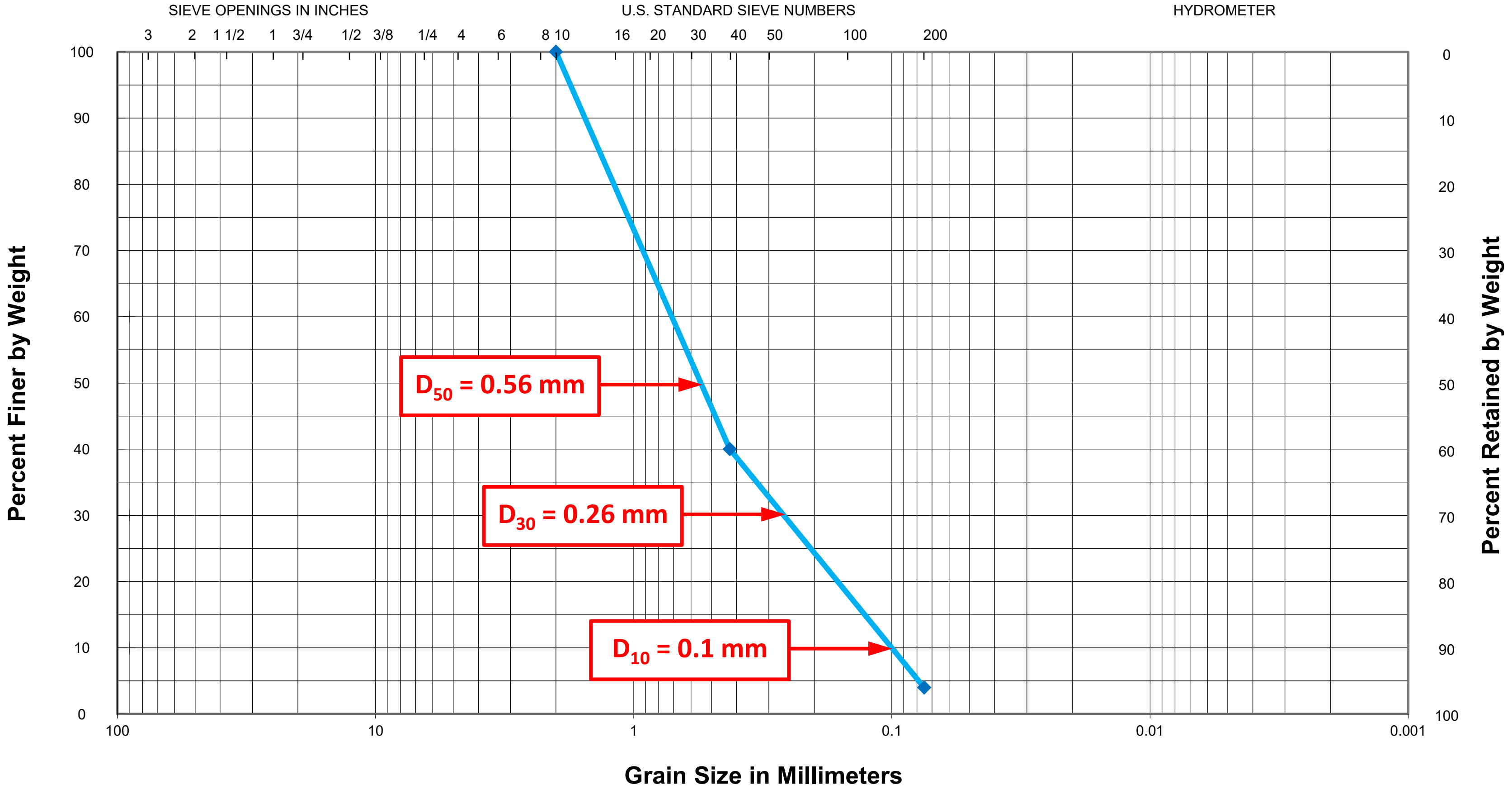
Sample: Boring F2, 6.5-7.5 ft
Description: Gray and brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

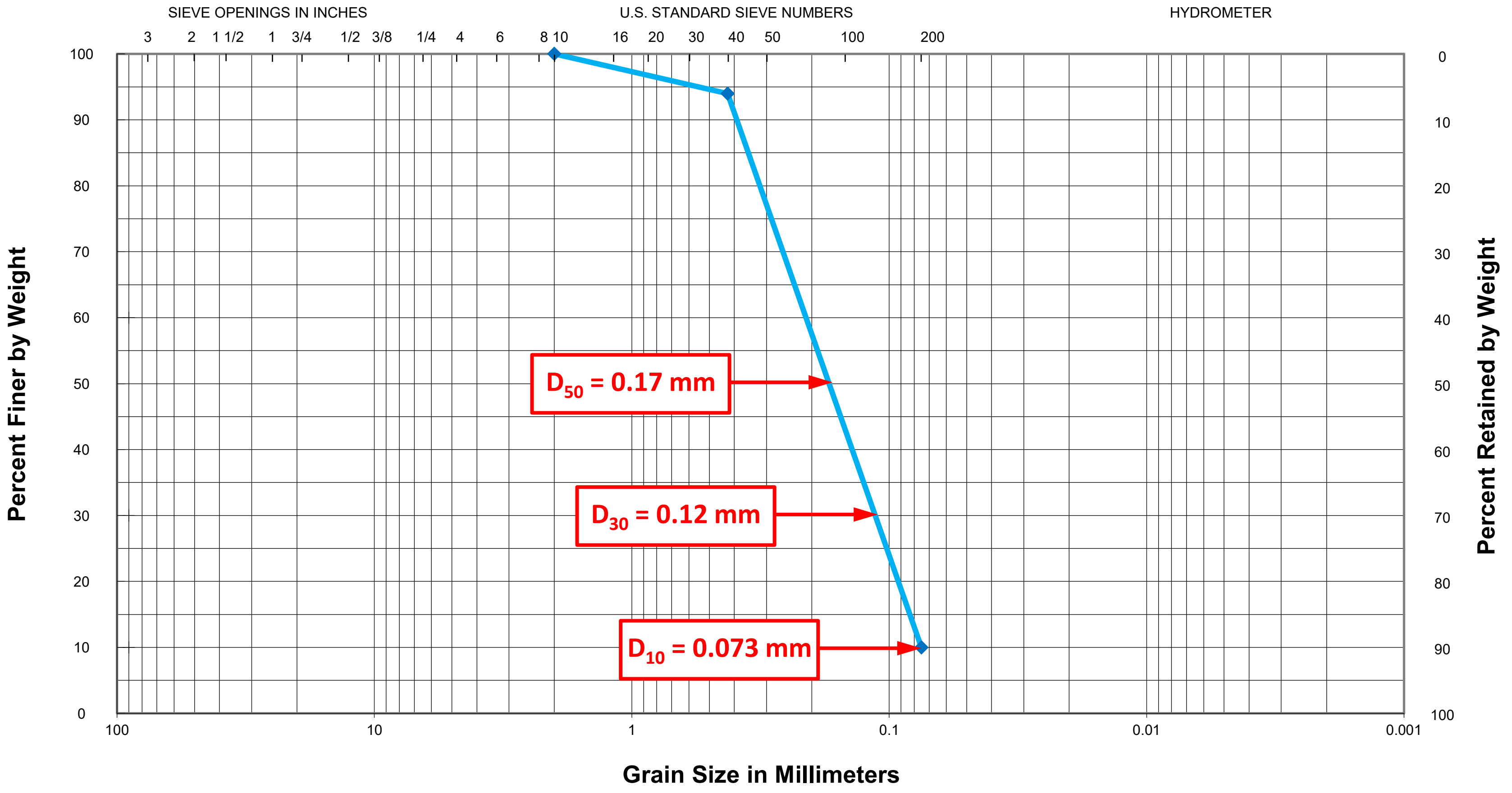
Sample: Boring F2, 29-30 ft
Description: Brownish gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.17 mm

D_{30} = 0.12 mm

D_{10} = 0.073 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

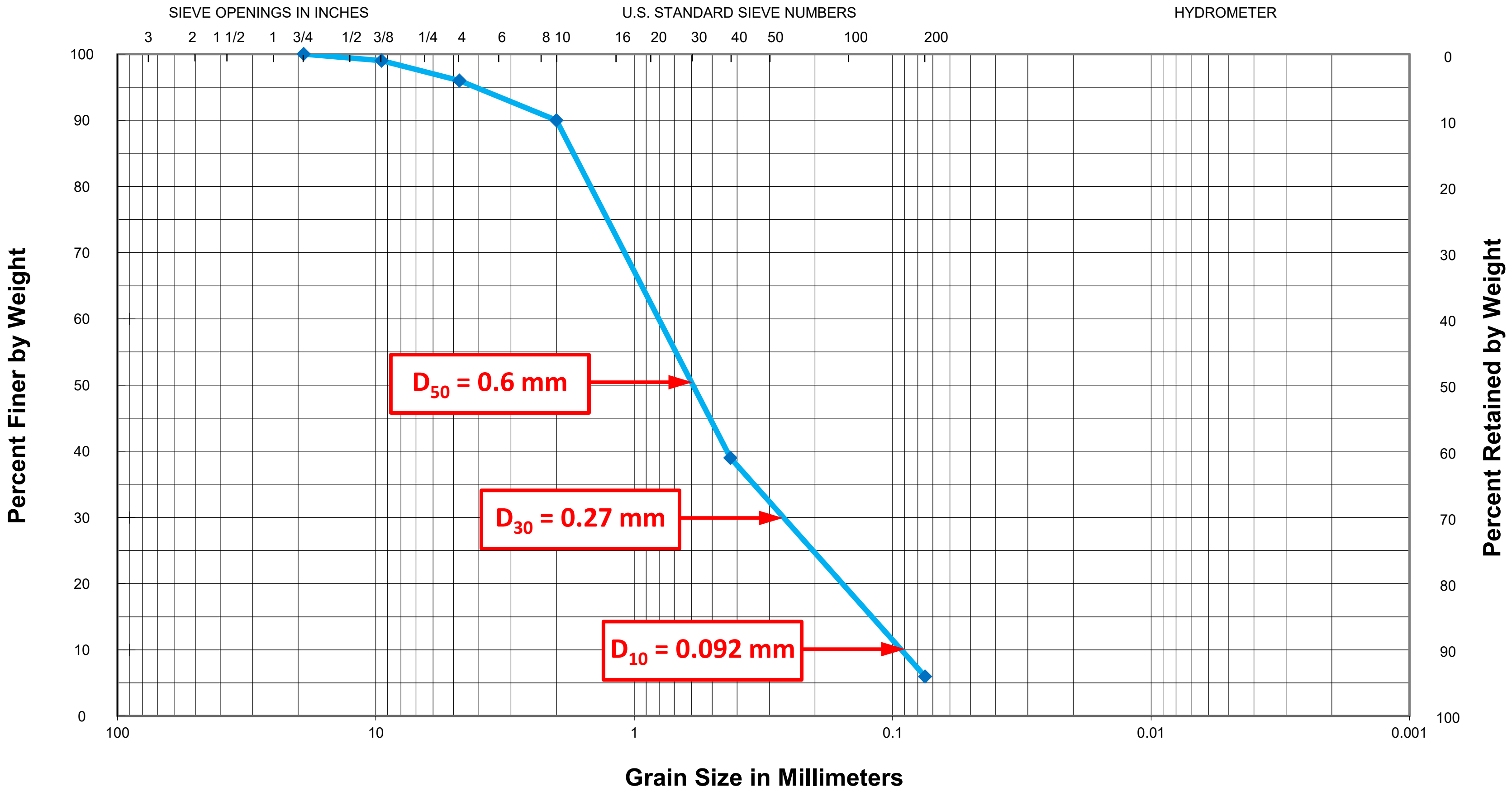
Sample: Boring F2, 44-45 ft
Description: Gray fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

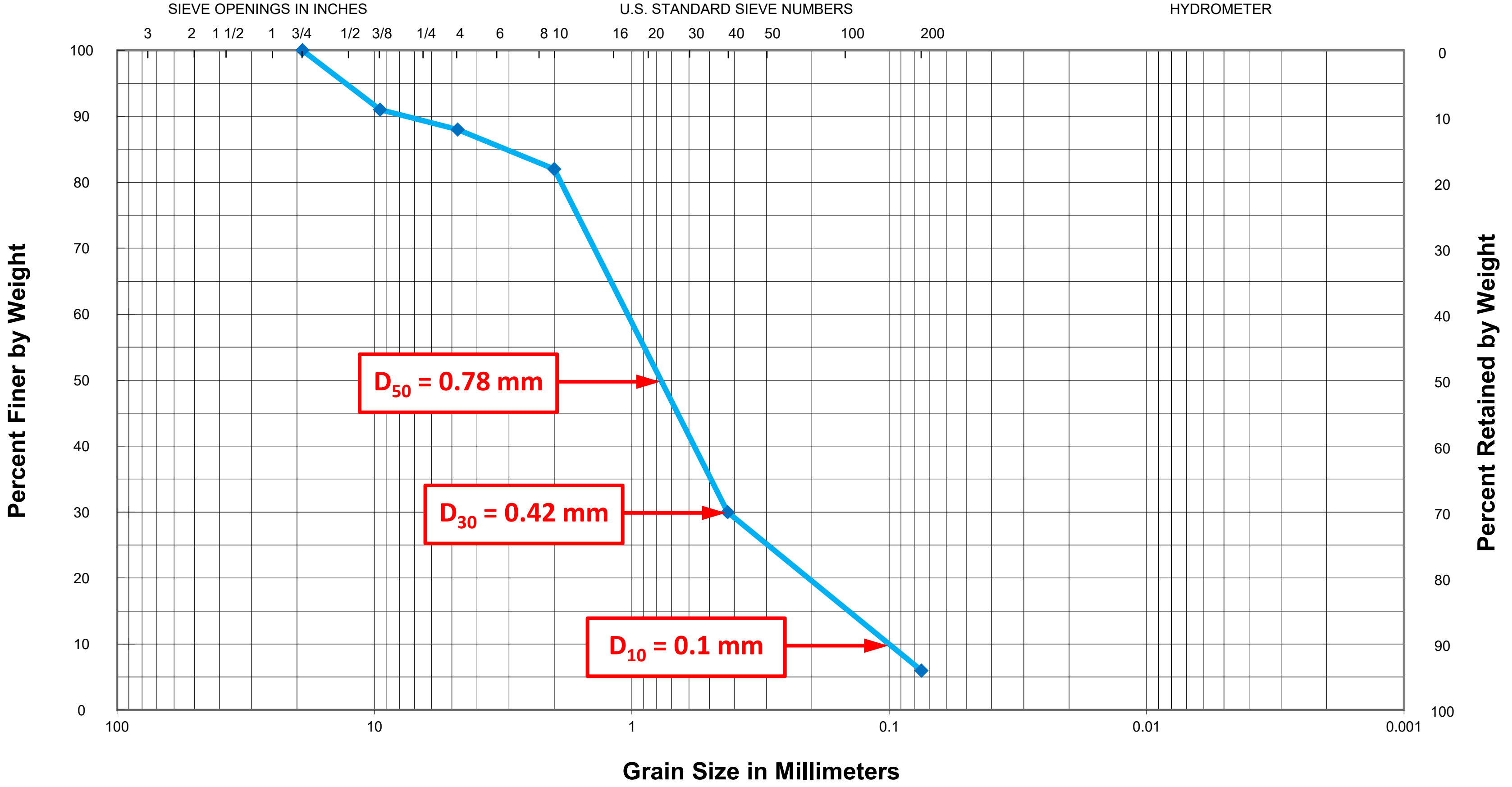
Sample: Boring F2, 59-60 ft
Description: Dark gray and brown fine to medium SAND, slightly silty w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

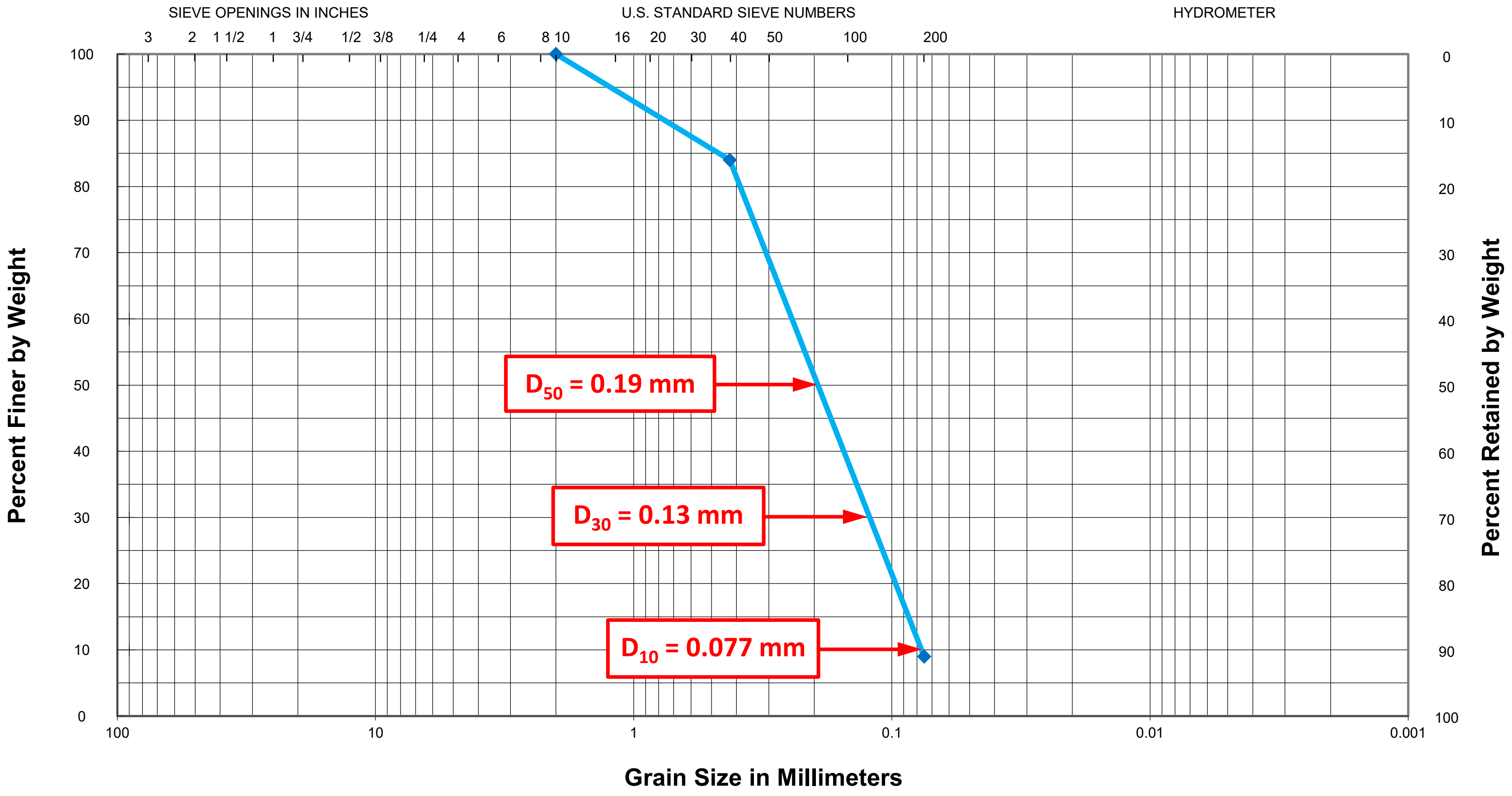
Sample: Boring F2, 89-90 ft
Description: Gray and brown fine to medium SAND, slightly silty w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

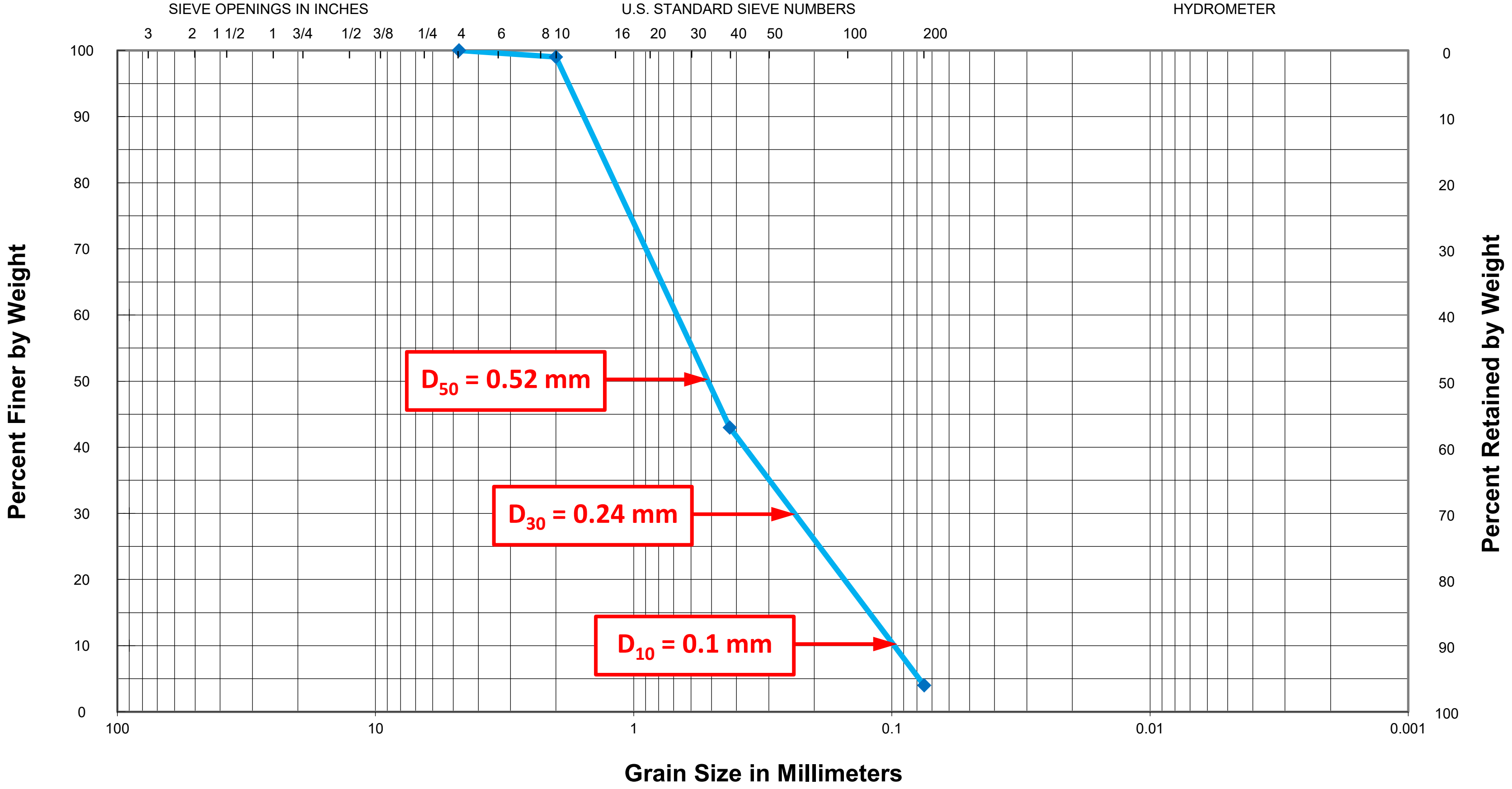
Sample: Boring F3, 14-15 ft
Description: Grayish tan fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

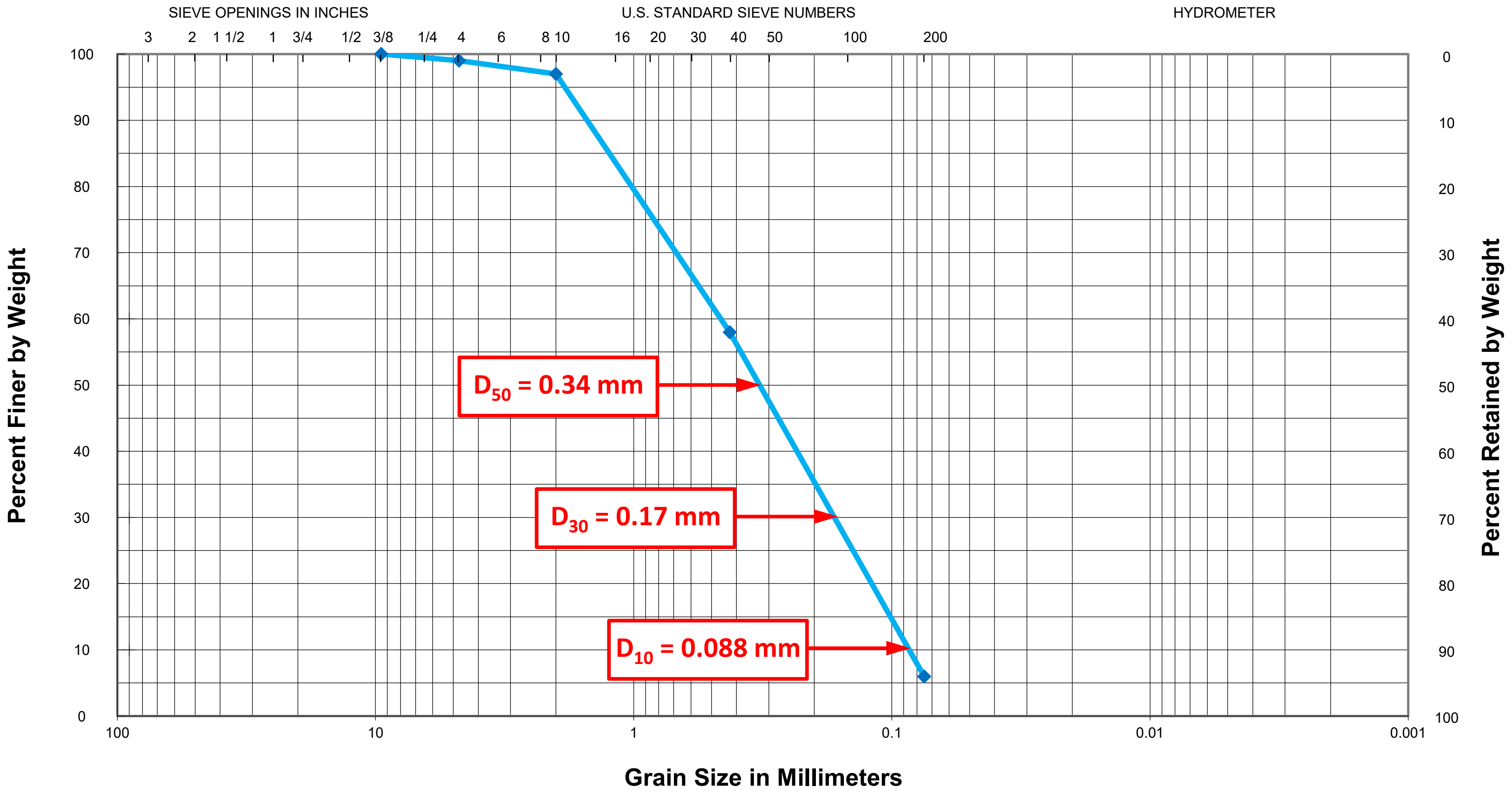
Sample: Boring F3, 44-45 ft
Description: Grayish tan fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

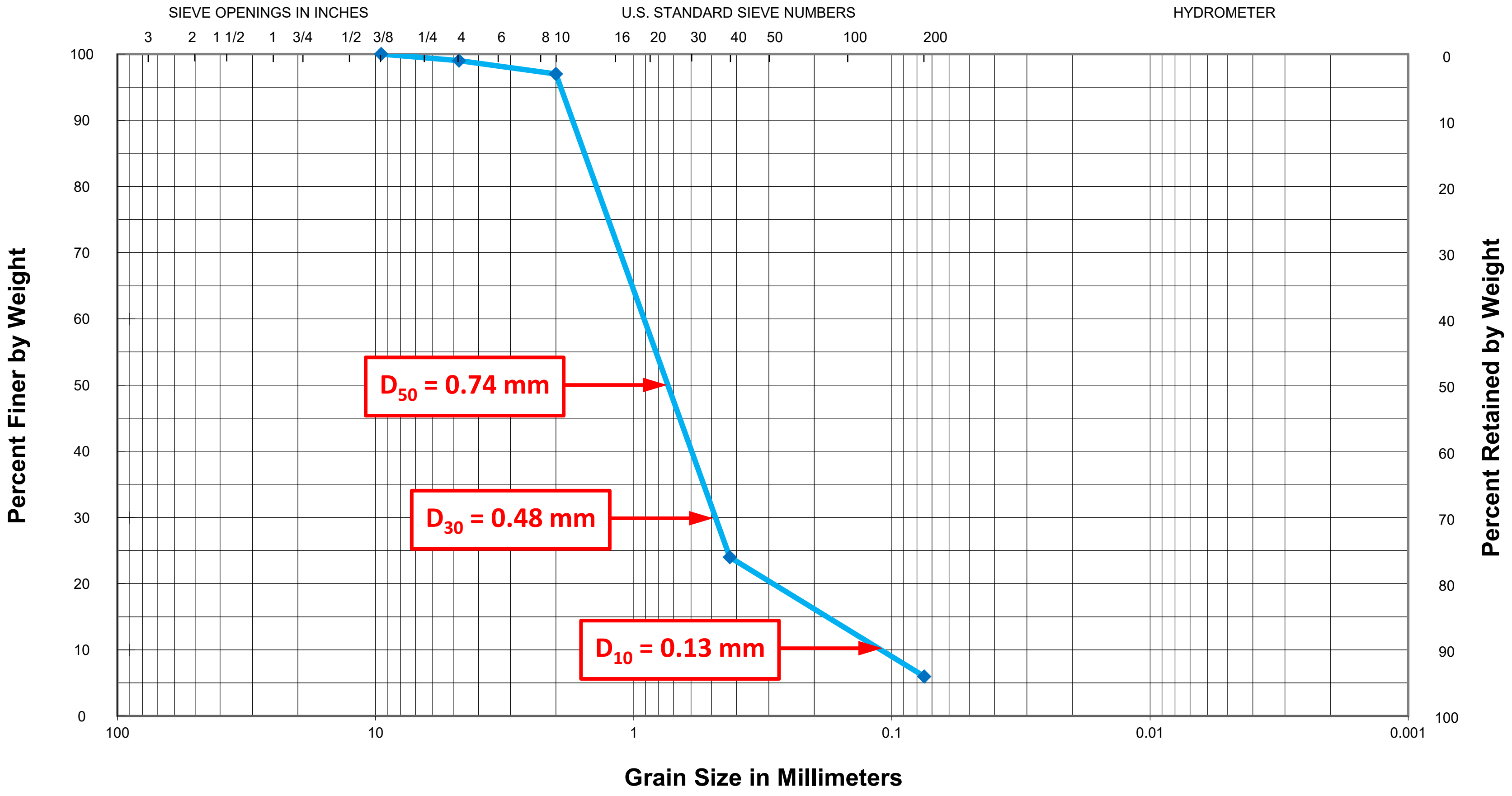
Sample: Boring F3, 69-70 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

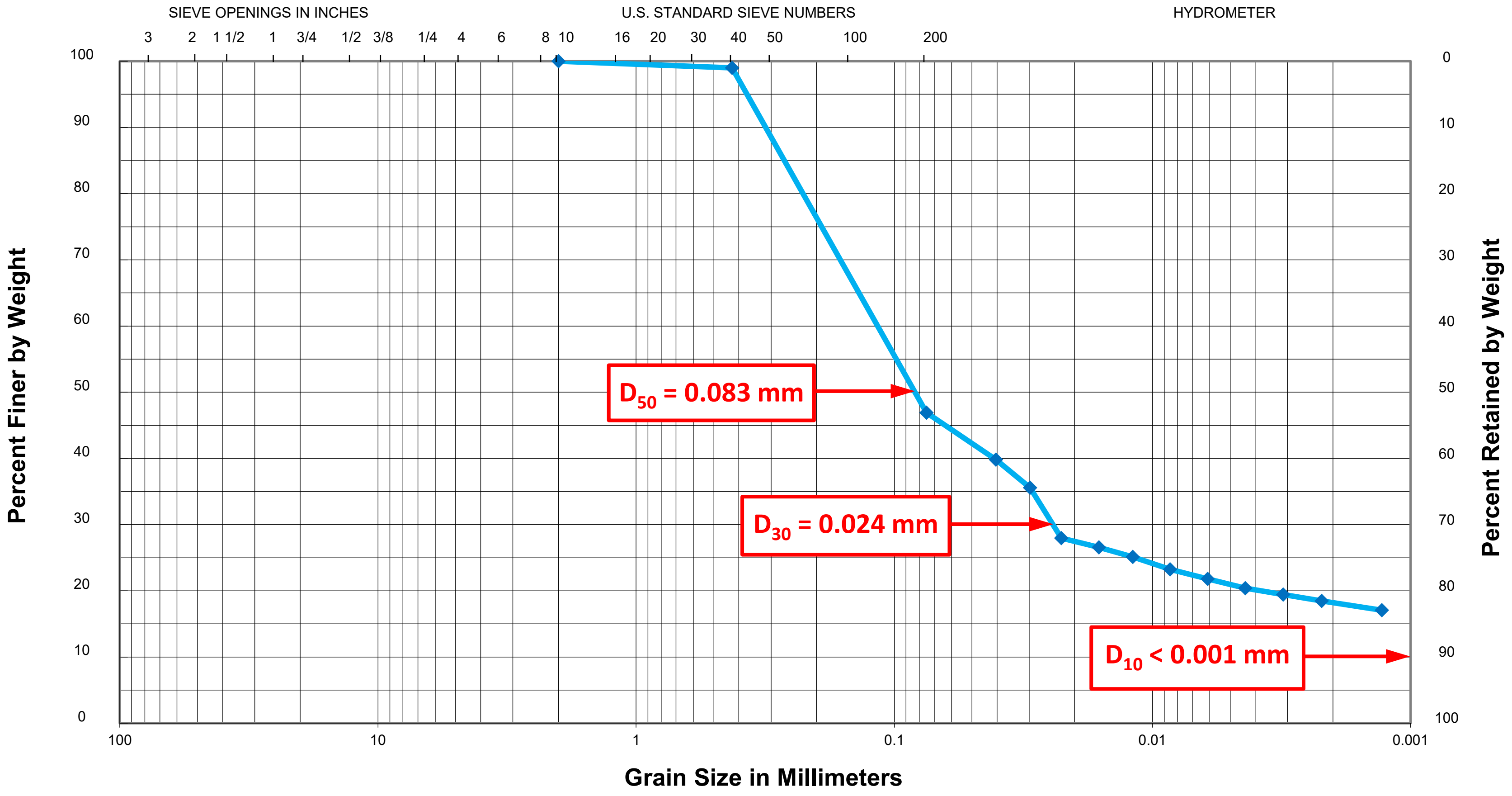
Sample: Boring F3, 84-85 ft
Description: Grayish tan fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

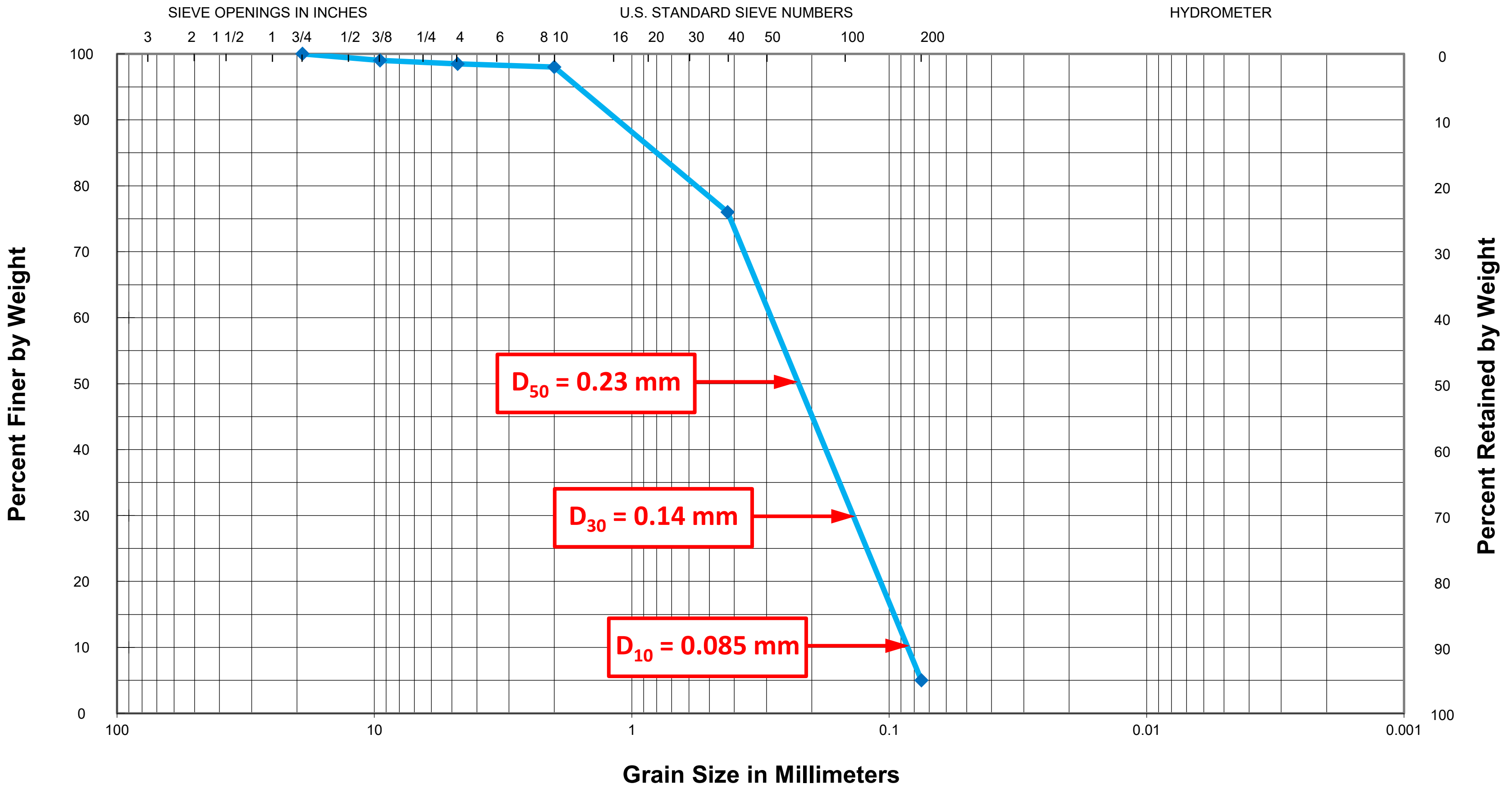
Sample: Boring F4, 9-10 ft
Description: Brown clayey fine SAND

USCS Classification = SC
AASHTO Classification = A-4

23-031

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

GRAIN SIZE CURVE



D_{50} = 0.23 mm

D_{30} = 0.14 mm

D_{10} = 0.085 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

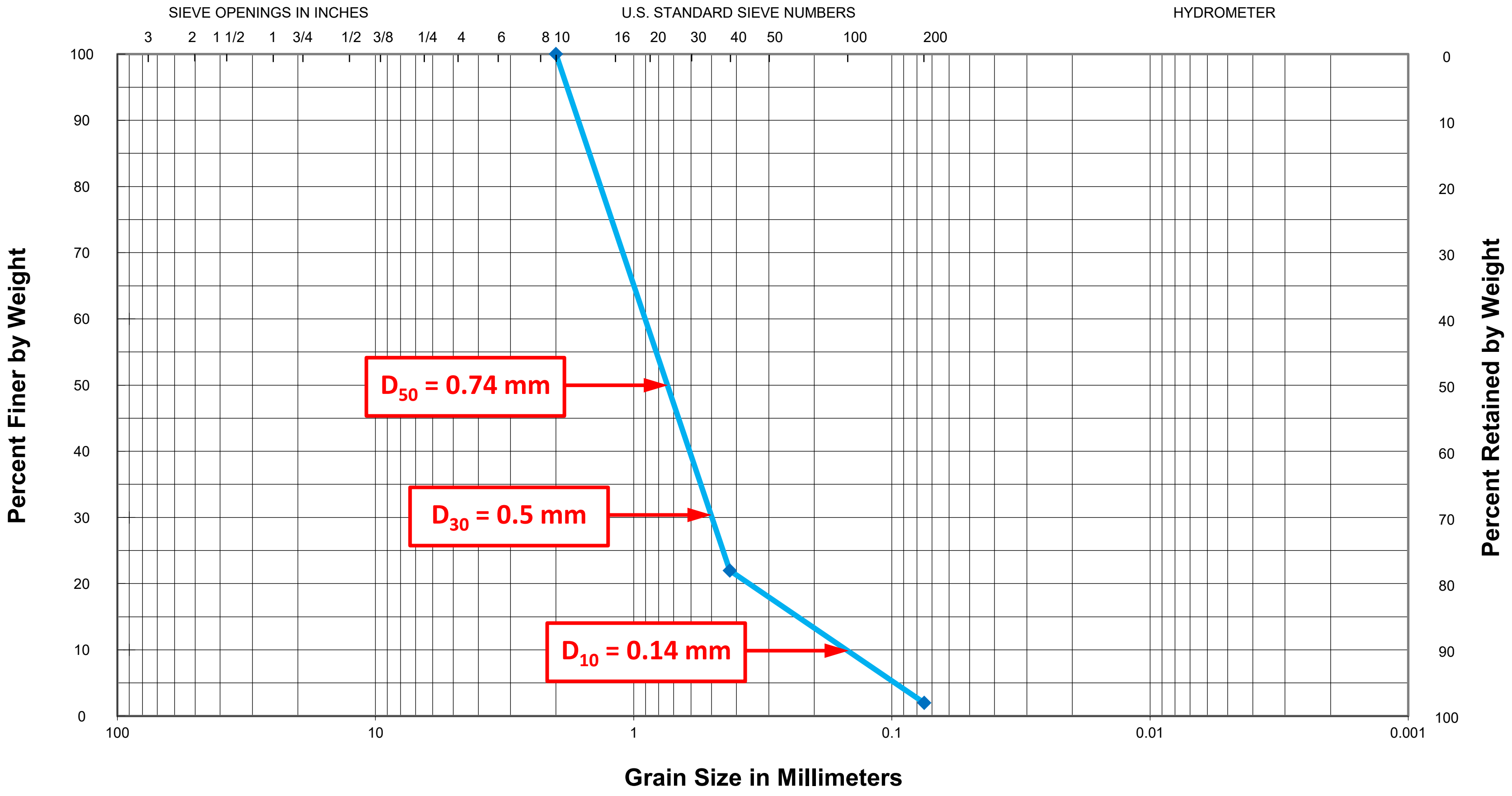
Sample: Boring F4, 44-45 ft
Description: Brownish gray fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

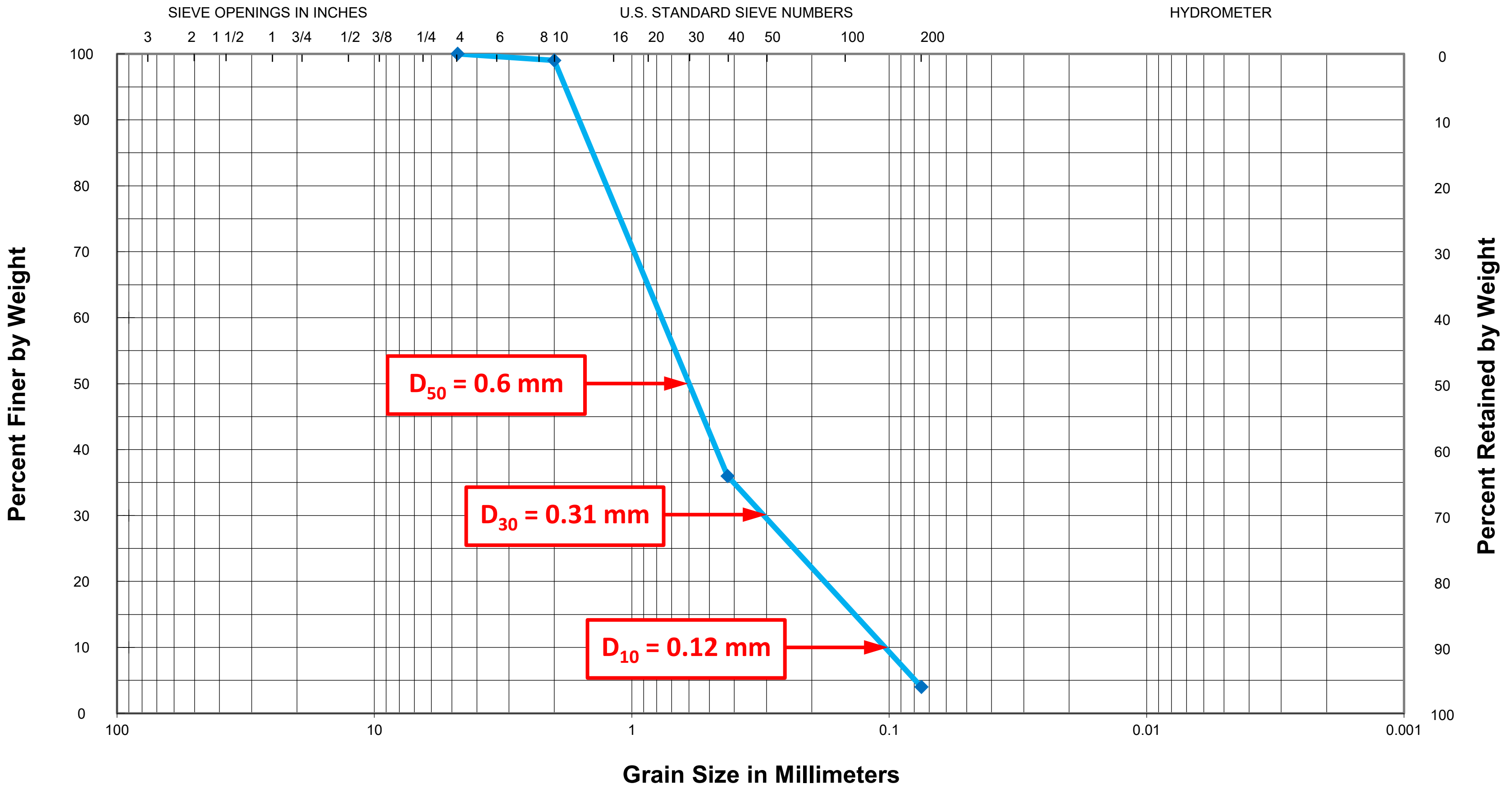
Sample: Boring F4, 59-60 ft
Description: Grayish brown fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

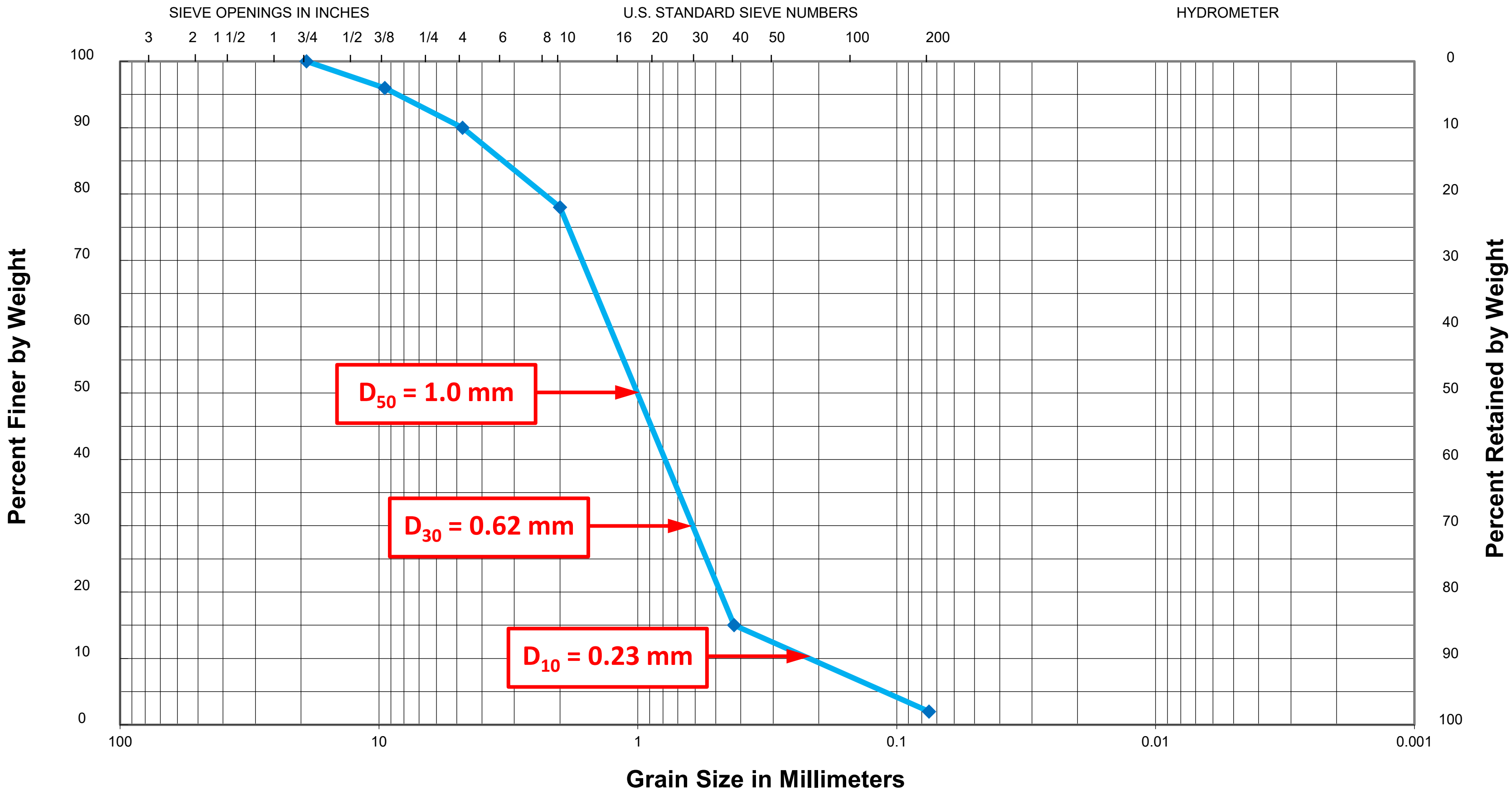
Sample: Boring F4, 74-75 ft
Description: Brownish gray fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

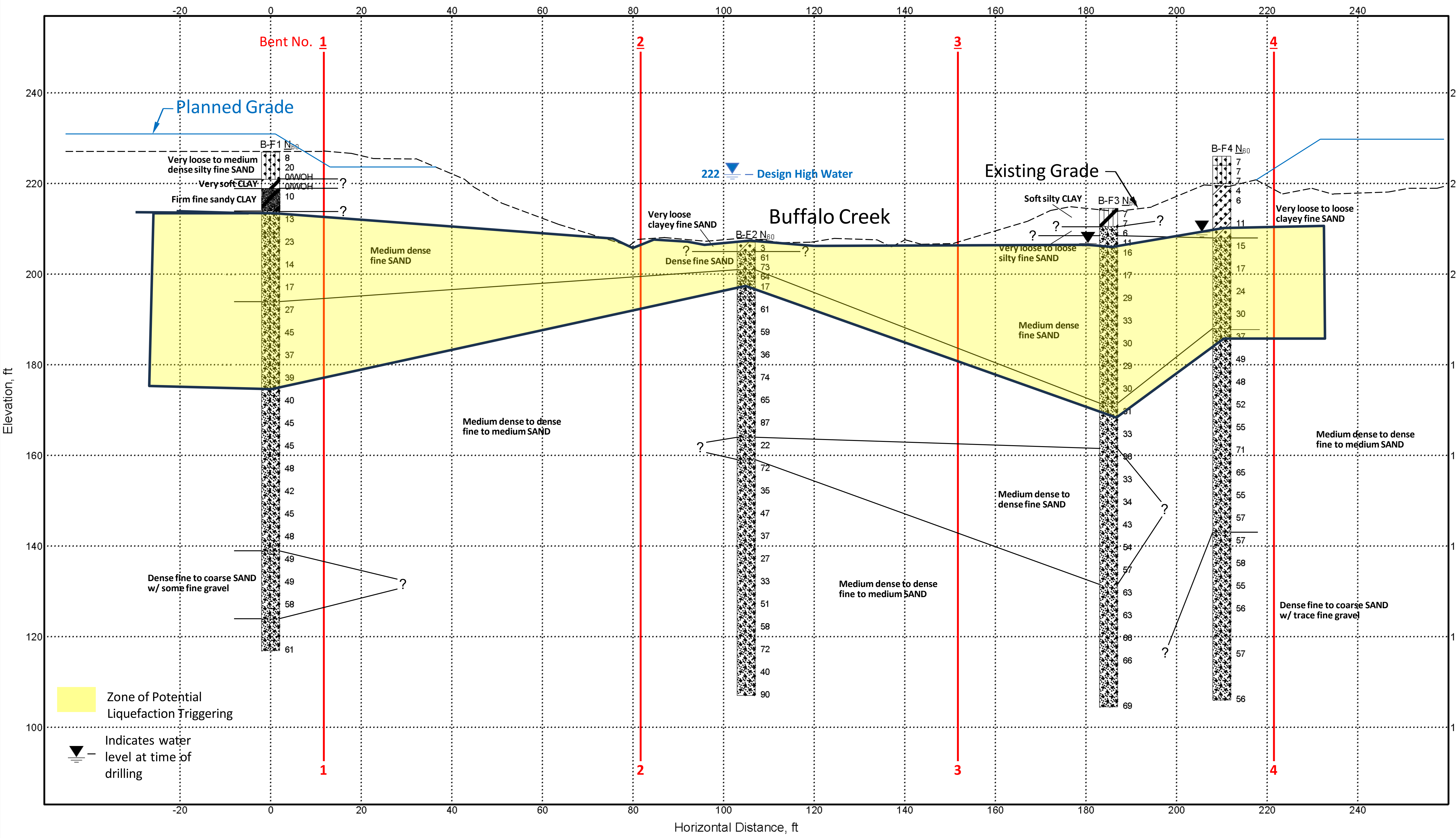


GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring F4, 84-85 ft
Description: Gray fine to coarse SAND w/ trace fine gravel

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

APPENDIX D



Grubbs, Hoskyn,
Barton & Wyatt, LLC

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

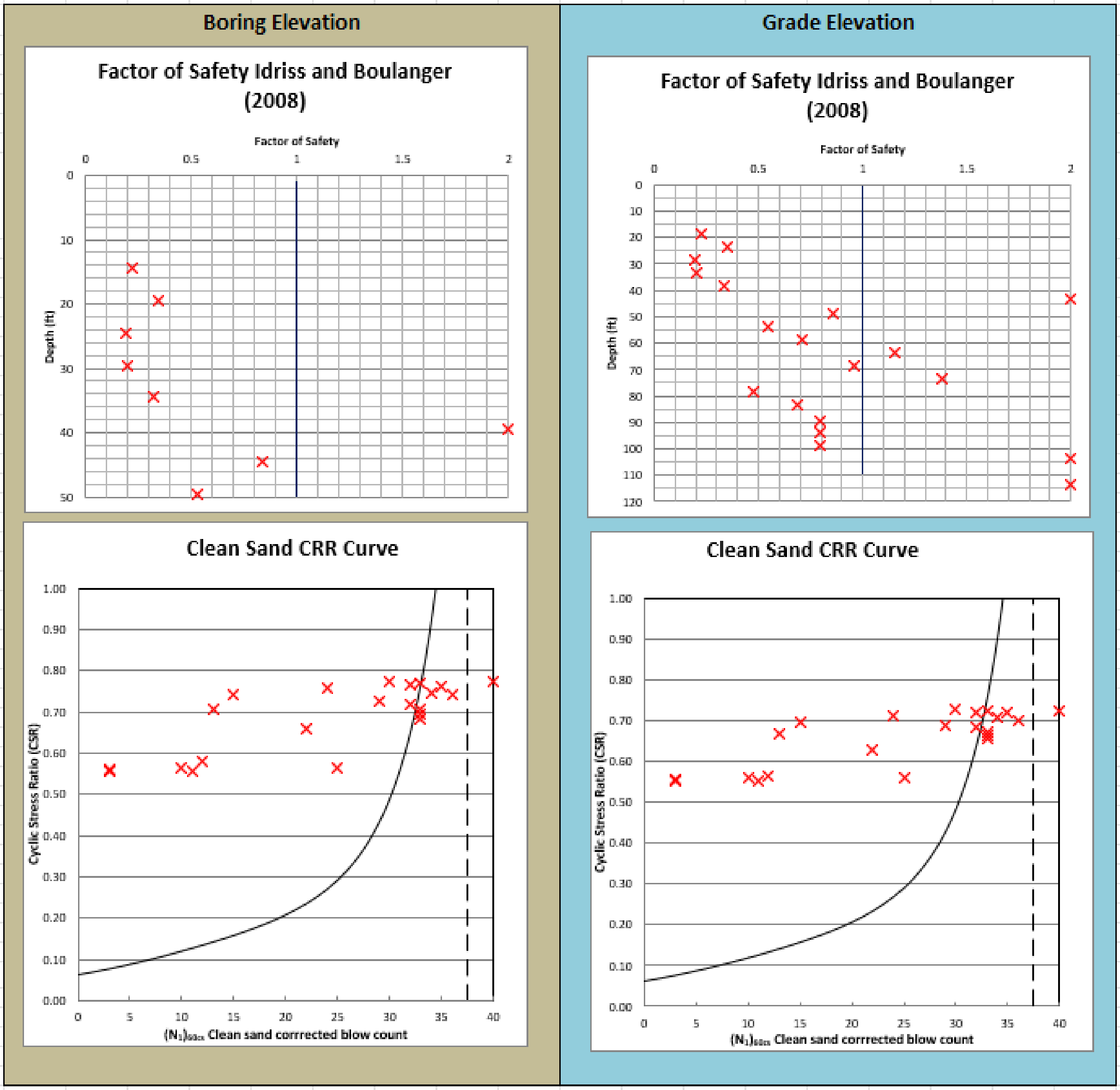
SCALE:
1" = 20' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Project Number: 23-031

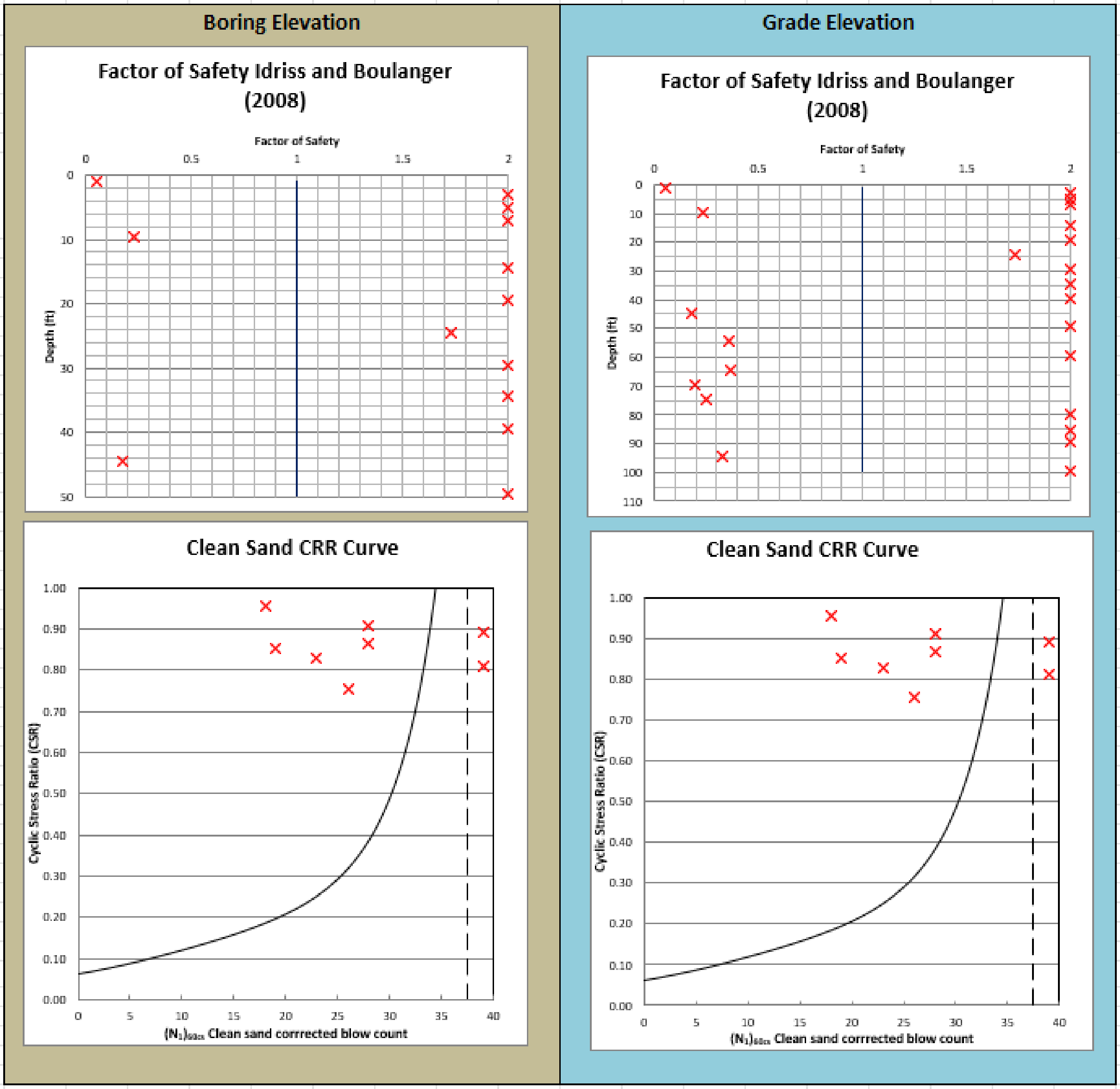
Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Buffalo Creek
Bent 1 / Boring F1
GHBW Job No. 23-031
Poinsett County, Arkansas



Liquefaction Analysis Results

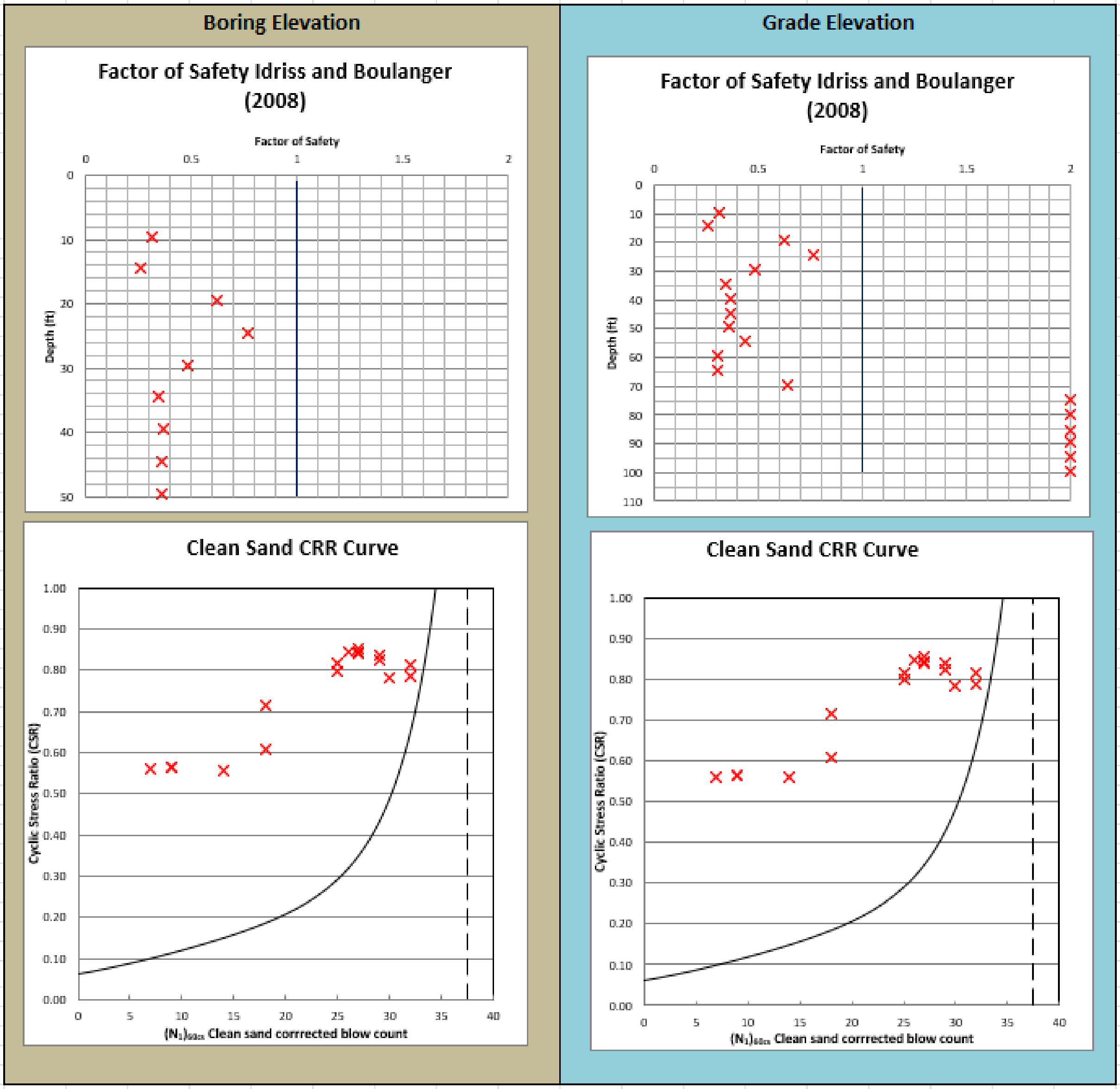
ARDOT 101124 Hwy 135 over Buffalo Creek
Bent 2 / Boring F2
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Liquefaction Analysis Results

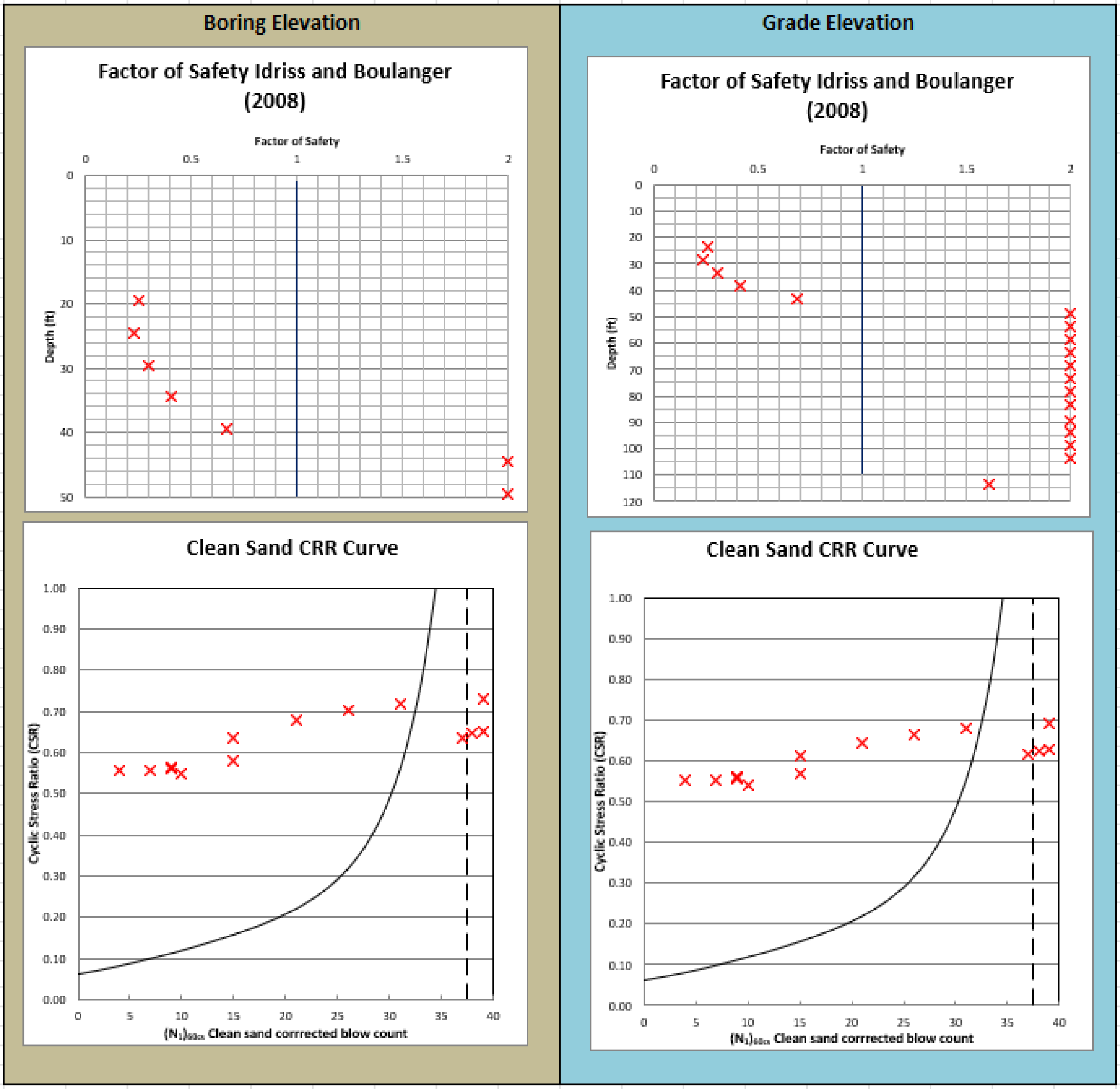
ARDOT 101124 Hwy 135 over Buffalo Creek
Bent 3 / Boring F3
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

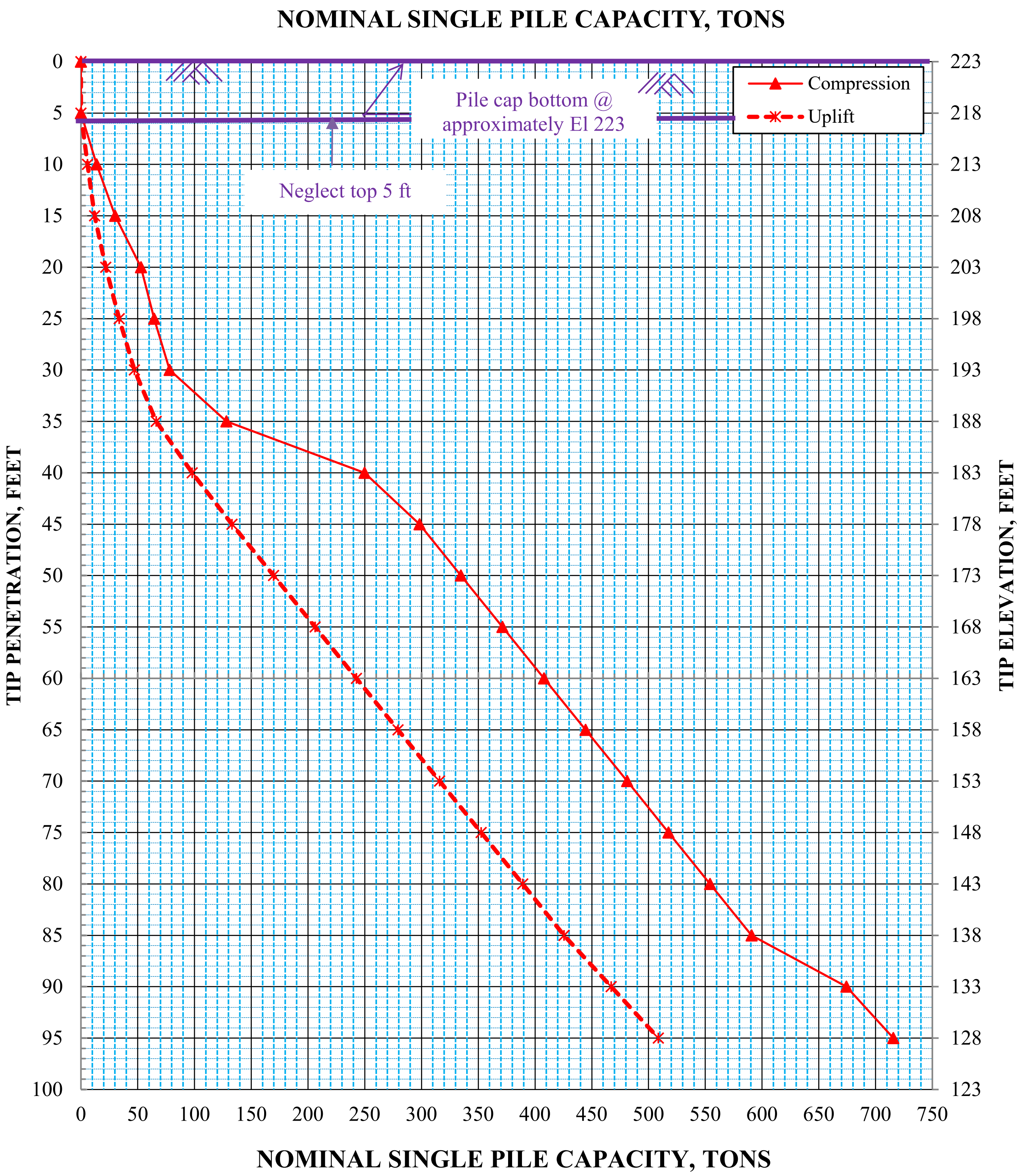
Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Buffalo Creek
Bent 4 / Boring F4
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

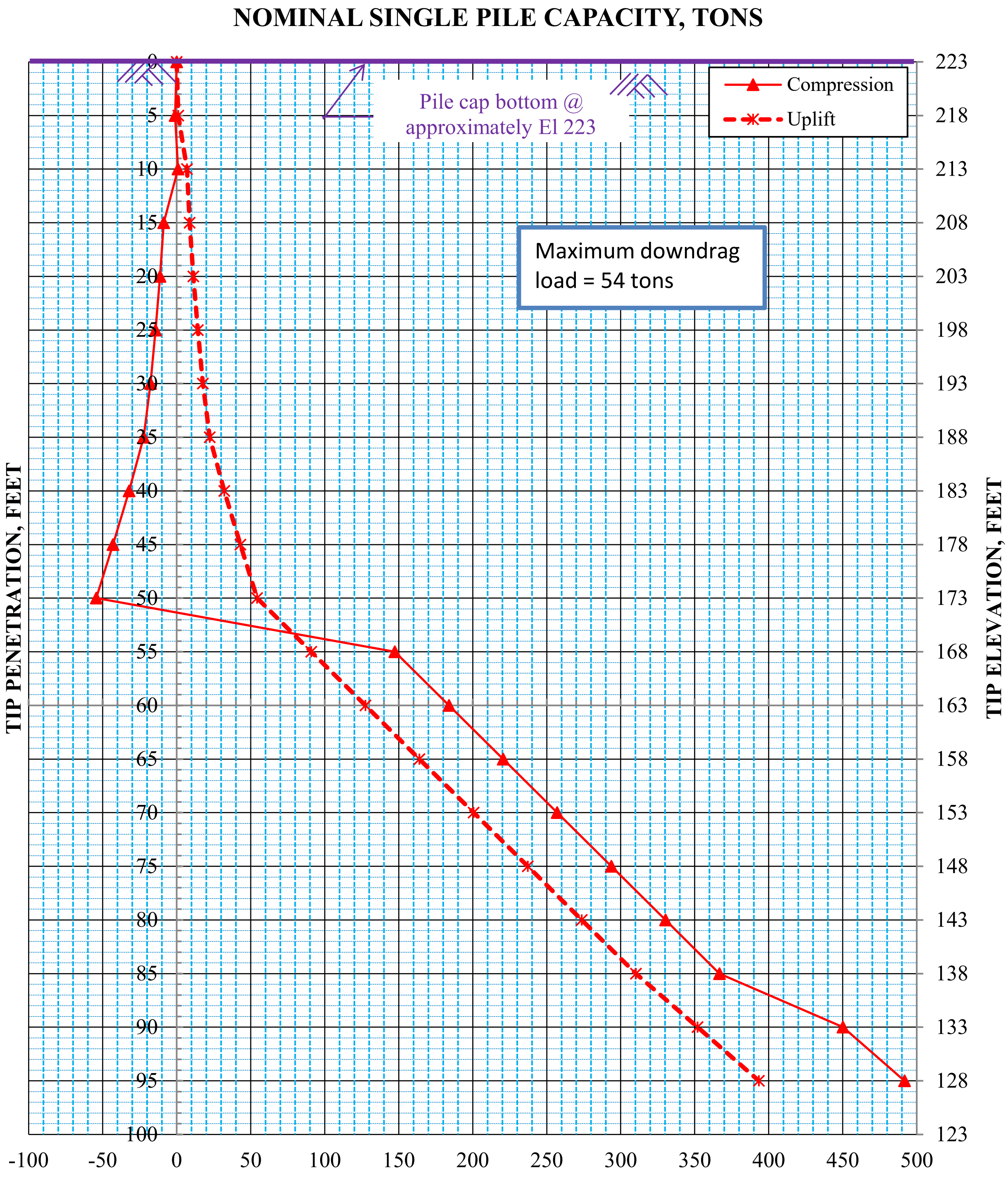
APPENDIX E



Bent 1 (South Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

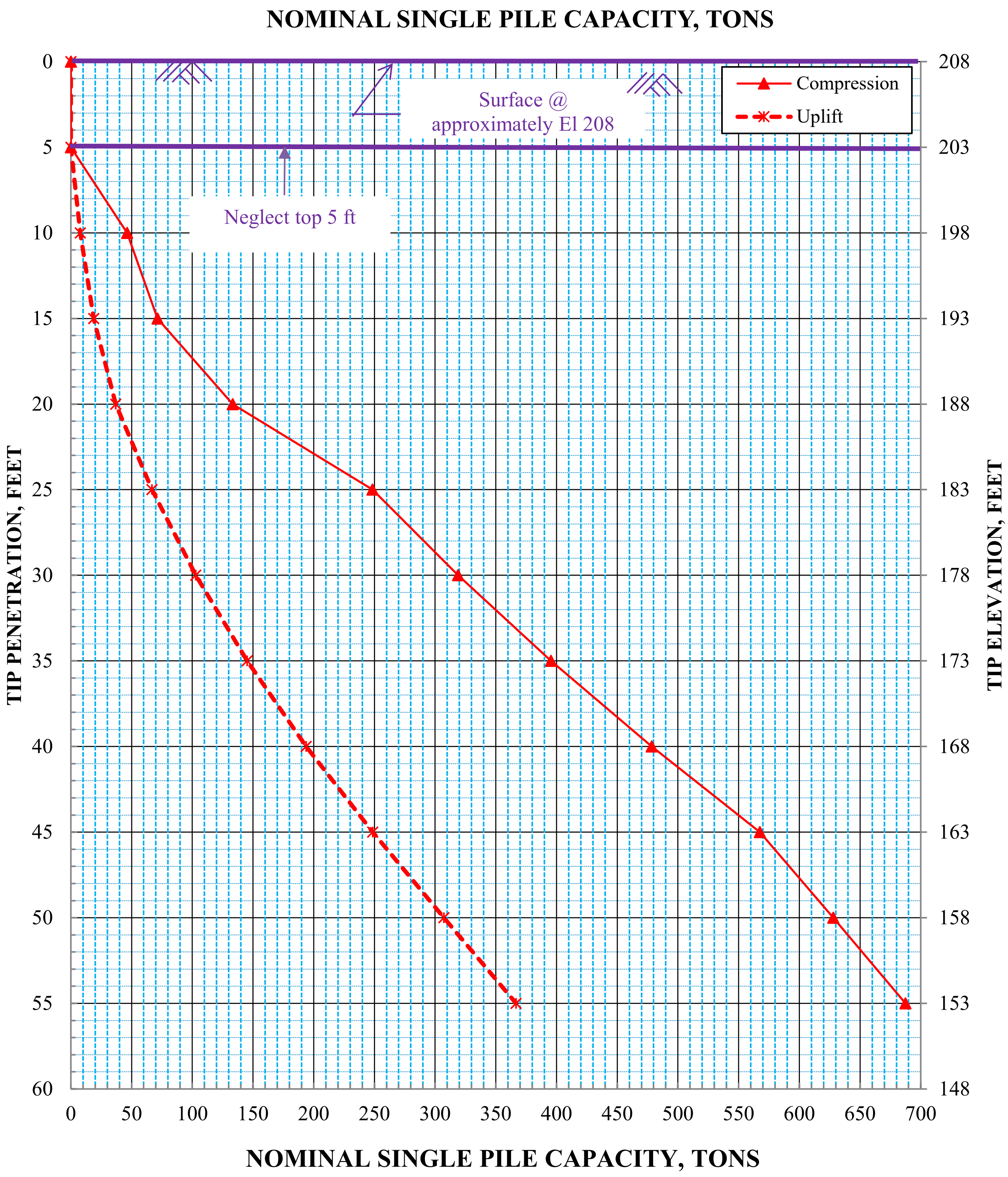
Notes: 1. Driven from cap bottom elevation
2. No downdrag

Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 1 (South Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

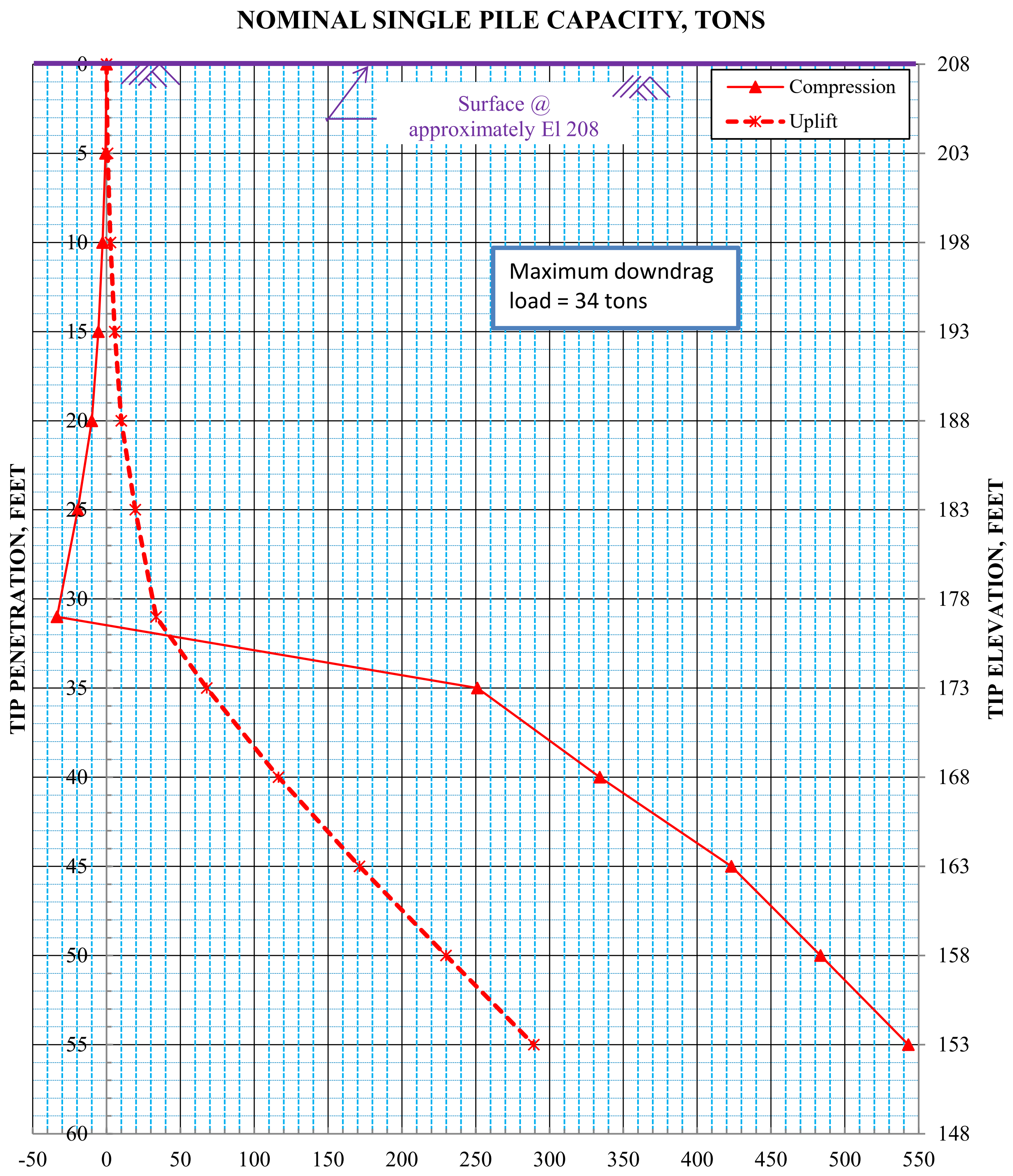
Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 173



Bent 2 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. No downdrag

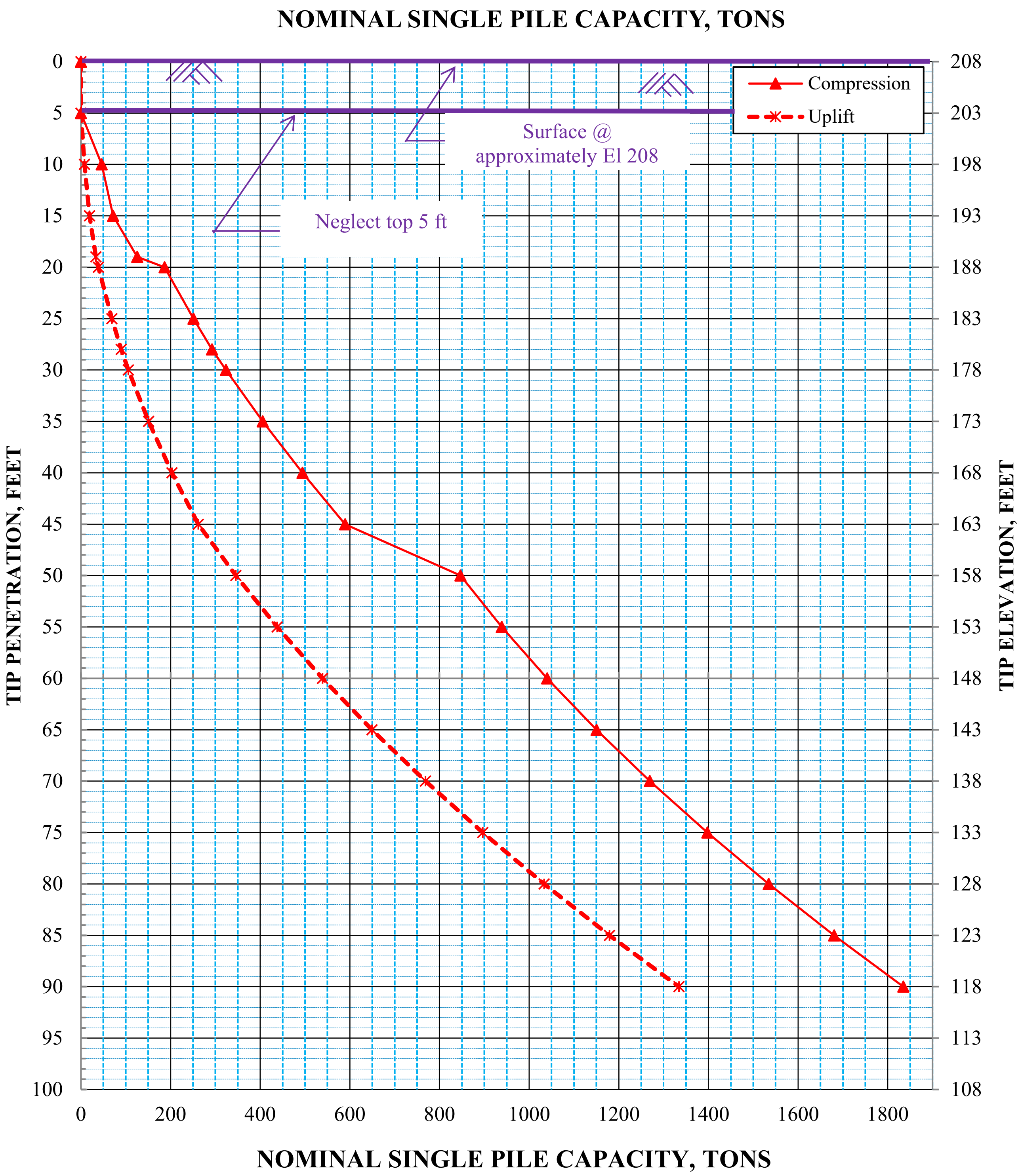
Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers



NOMINAL SINGLE PILE CAPACITY, TONS

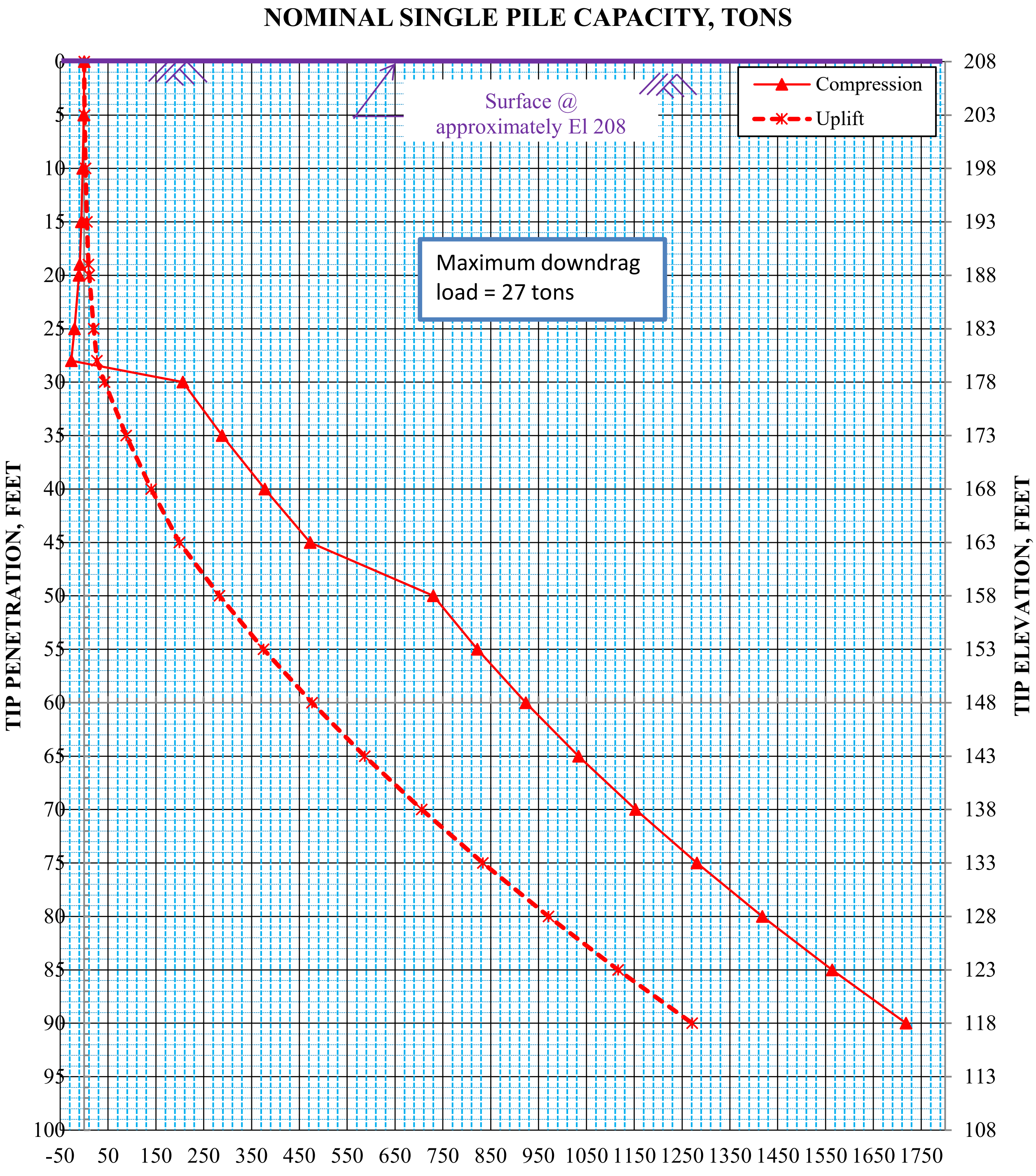
Bent 2 (Intermediate Bent)
28-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 177



Bent 3 (Intermediate Bent)
28-in.-Diameter Steel Shell
ArDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

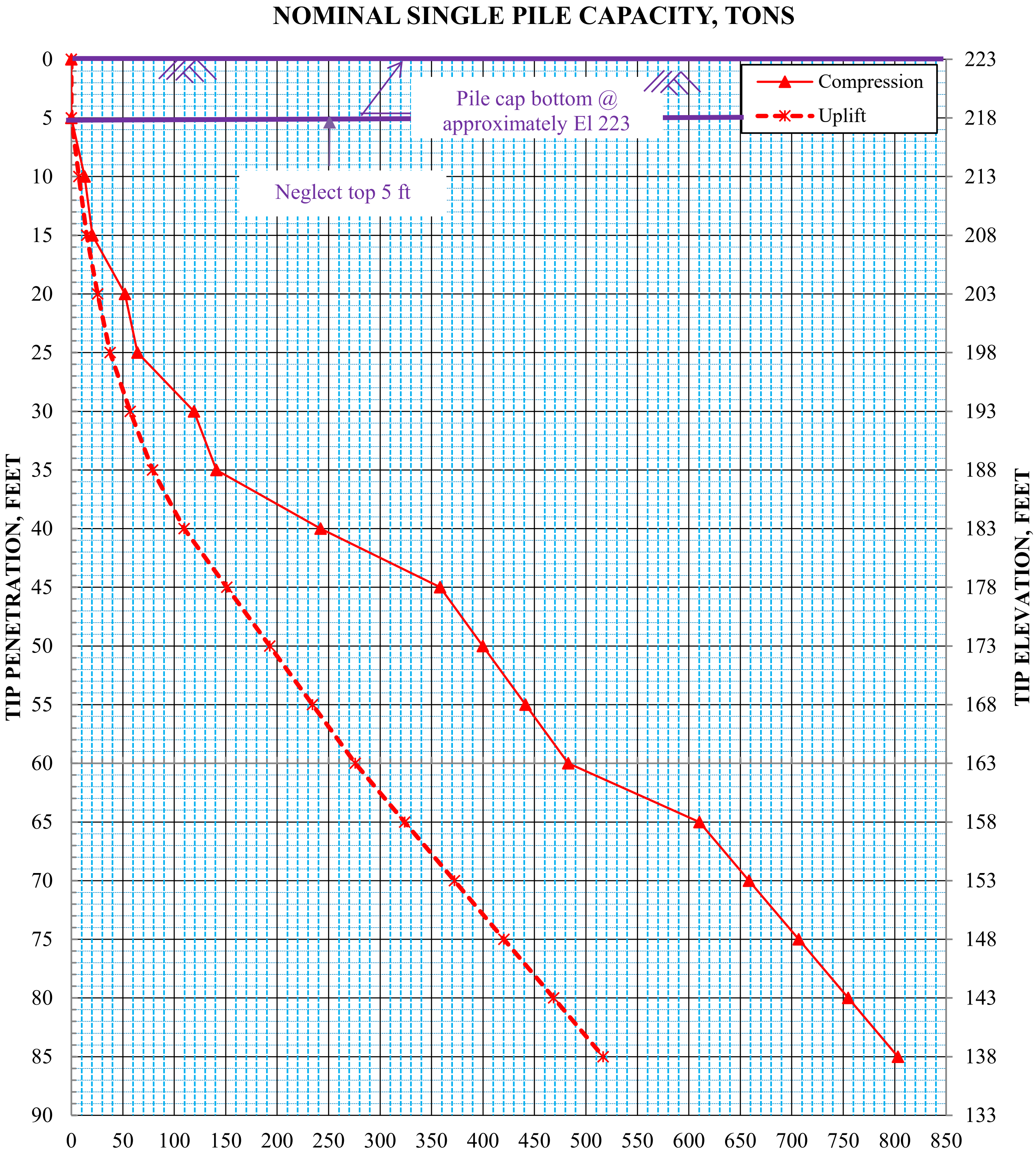
Notes: 1. Driven from channel bottom location
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

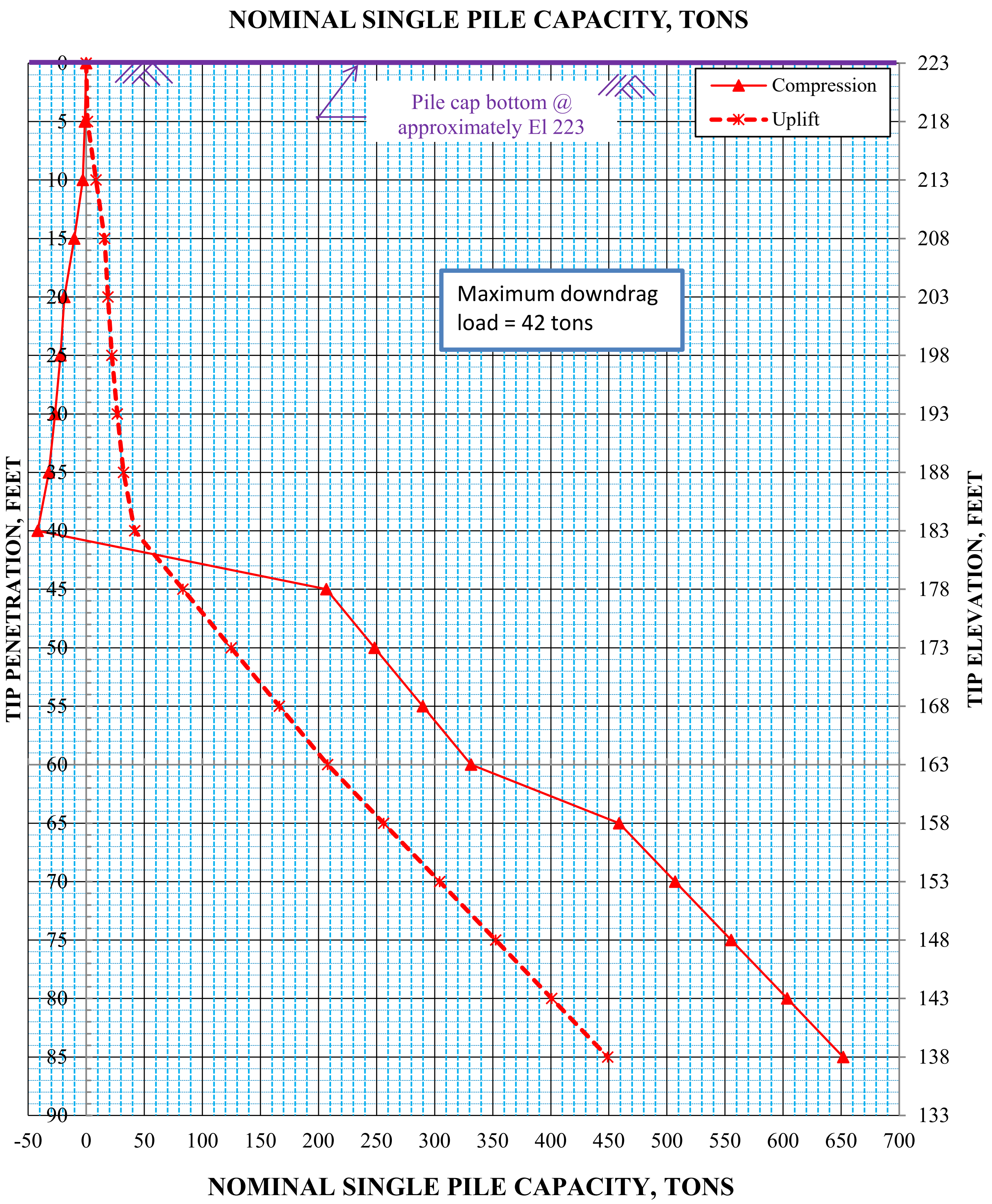
Bent 3 (Intermediate Bent)
28-in.-Diameter Steel Shell
ArDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom location
2. Downdrag to ±El 180



NOMINAL SINGLE PILE CAPACITY, TONS
Bent 4 (North Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag



Bent 4 (North Bridge End)
18-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Buffalo Creek
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 183

Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers

APPENDIX F

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Buffalo Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft to firm sandy CLAY**	**Loose to medium dense silty fine SAND**	**Medium dense silty fine SAND**	**Medium dense fine to medium SAND**	**Dense fine to medium SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-15	15-30	30-35	35-85	85 and deeper
Approximate El, ft	223-213	213-208	108-193	193-188	188-138	below 138
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	53	56	58	63	63
Cohesion (c), lbs per sq ft	500	0	0	0	0	0
Angle of internal friction (φ), °	0	30	32	34	37	38
Subgrade modulus (k), lbs per cu in.	30	35	50	60	115	125
Strain at 50% (EE50)	0.02	NA	NA	NA	NA	NA

Note: Pile cap bottom at ±El 223

Seismic Loading with Liquefaction
Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Soft to firm sandy CLAY**	**Loose to medium dense silty fine SAND (liquefiable)**	**Medium dense silty fine SAND (liquefiable)**	**Medium dense to dense fine to medium SAND (liquefiable)**	**Dense fine to medium SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-10	10-15	15-30	30-50	50-85	85 and deeper
Approximate El, ft	223-213	213-208	108-193	193-173	173-138	below 138
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	53	56	58	63	63
Cohesion (c), lbs per sq ft	500	0	0	0	0	0
Angle of internal friction (φ), °	0	8	8	11	37	38
Subgrade modulus (k), lbs per cu in.	30	20	20	20	115	125
Strain at 50% (EE50)	0.02	NA	NA	NA	NA	NA

Note: Pile cap bottom at ±El 223

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Buffalo Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Medium dense silty fine SAND**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-55	55 and deeper
Approximate El, ft	208-188	188-153	below 153
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	56	63	68
Cohesion (c), lbs per sq ft	0	0	0
Angle of internal friction (φ), °	32	36	38
Subgrade modulus (k), lbs per cu in.	50	105	125
Strain at 50% (EE50)	NA	NA	NA

Note: Ground surface at ±El 208

Seismic Loading with Liquefaction

Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Medium dense silty fine SAND (liquefiable)**	**Dense fine to medium SAND (liquefiable)**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-20	20-31	31-55	55 and deeper
Approximate El, ft	208-188	188-177	177-153	below 153
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	56	63	63	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	8	11	36	38
Subgrade modulus (k), lbs per cu in.	20	20	105	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 208

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Buffalo Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Medium dense silty fine SAND**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-19	19-28	28 and deeper
Approximate El, ft	208-189	189-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	56	63	68
Cohesion (c), lbs per sq ft	0	0	0
Angle of internal friction (φ), °	33	36	38
Subgrade modulus (k), lbs per cu in.	55	105	125
Strain at 50% (EE50)	NA	NA	NA

Note: Ground surface at ±El 208

Seismic Loading with Liquefaction

Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Medium dense silty fine SAND (liquefiable)**	**Dense fine to medium SAND (liquefiable)**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-19	19-28	28 and deeper
Approximate El, ft	208-189	189-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	56	63	68
Cohesion (c), lbs per sq ft	0	0	0
Angle of internal friction (φ), °	8	11	38
Subgrade modulus (k), lbs per cu in.	20	20	125
Strain at 50% (EE50)	NA	NA	NA

Note: Ground surface at ±El 208

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Buffalo Creek

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Soft to firm fine sandy CLAY	Medium dense silty fine SAND	Medium dense silty fine SAND	Dense fine to medium SAND	Dense to very dense fine to coarse SAND
Depth below pile cap bottom, ft	0-15	15-25	25-35	35-60	60 and deeper
Approximate El, ft	223-208	208-198	198-188	188-163	below 163
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	55	58	65	68
Cohesion (c), lbs per sq ft	700	0	0	0	0
Angle of internal friction (φ), °	0	31	34	38	40
Subgrade modulus (k), lbs per cu in.	100	40	60	125	130
Strain at 50% (EE50)	0.01	NA	NA	NA	NA

Note: Pile cap bottom at ±El 223

Seismic Loading with Liquefaction
Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

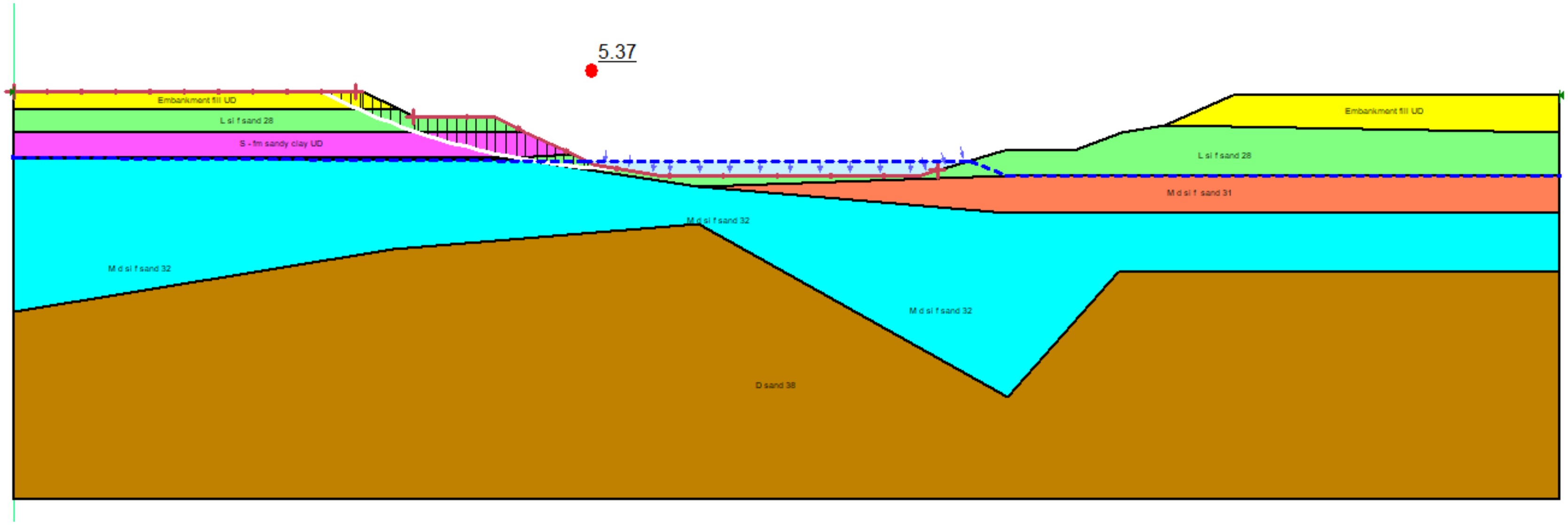
Generalized Stratigraphy	Soft to firm fine sandy CLAY	Medium dense silty fine SAND (liquefiable)	Medium dense to dense silty fine SAND (liquefiable)	Dense fine to medium SAND	Dense to very dense fine to coarse SAND
Depth below pile cap bottom, ft	0-15	15-25	25-40	40-60	60 and deeper
Approximate El, ft	223-208	208-198	198-183	183-163	below 163
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	115	55	58	65	68
Cohesion (c), lbs per sq ft	700	0	0	0	0
Angle of internal friction (φ), °	0	8	11	38	40
Subgrade modulus (k), lbs per cu in.	100	20	20	125	130
Strain at 50% (EE50)	0.01	NA	NA	NA	NA

Note: Pile cap bottom at ±El 223

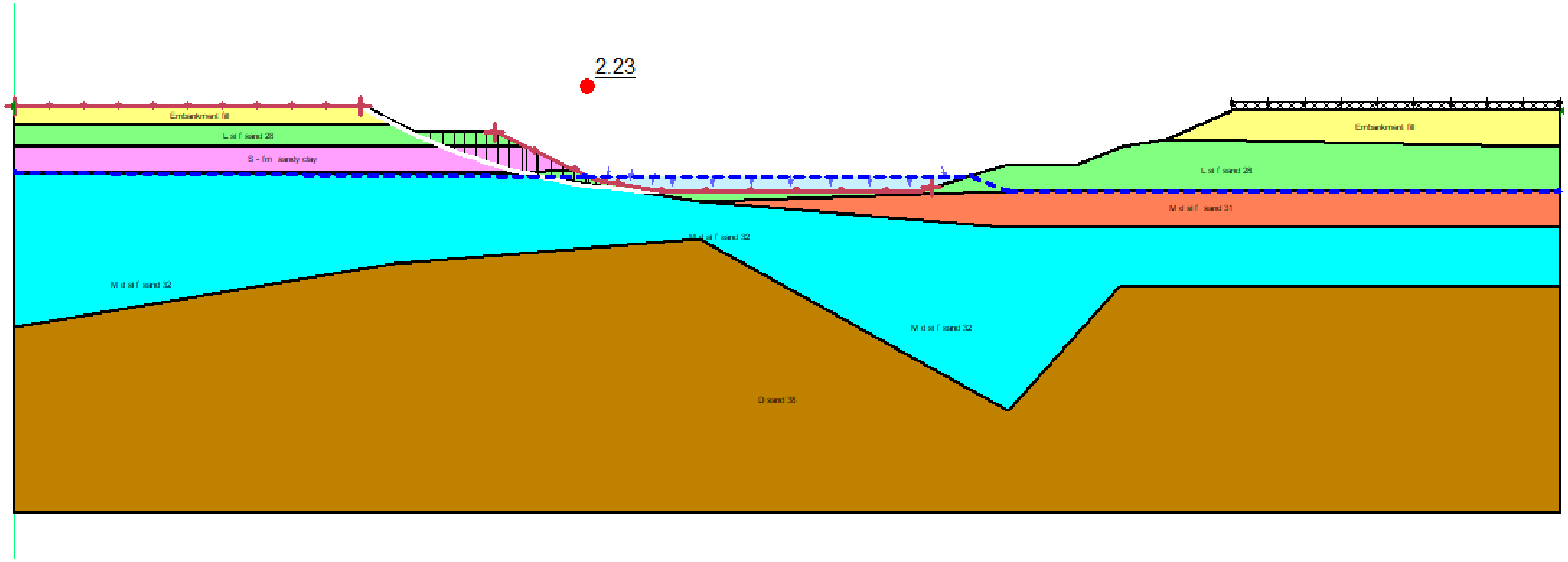
APPENDIX G

Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Buffalo Creek
GHBW Job No. 23-031
Poinsett County, Arkansas

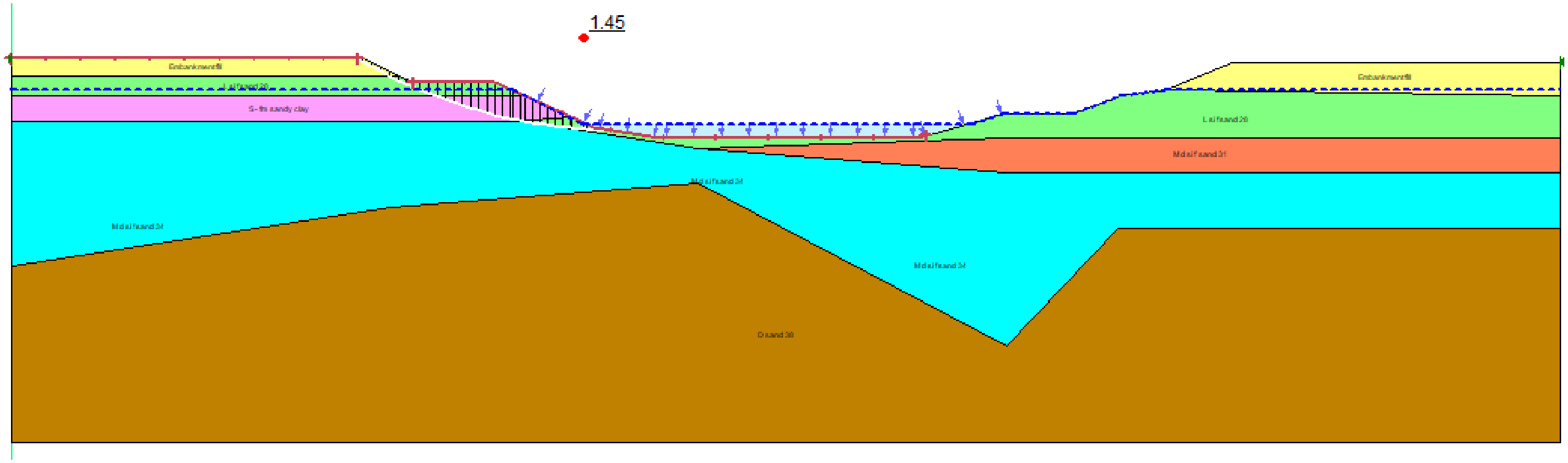
	Design Loading Condition	Calculated Minimum Factor of Safety
South End Slope (Bent 1) (2H:1V)	End of Construction	5.37
	Long Term	2.23
	Rapid Drawdown from El 222 to El 212	1.45
	Seismic (k_h =A_S/2 = 0.5235)	1.15
South Side Slope (Bent 1) (3H:1V)	End of Construction	5.35
	Long Term	2.35
	Rapid Drawdown from El 222 to El 212	2.00
	Seismic (k_h =A_S/2 = 0.5235)	1.07
North End Slope (Bent 4) (2H:1V)	End of Construction	3.27
	Long Term	2.00
	Rapid Drawdown from El 222 to El 212	1.26
	Seismic (k_h =A_S/2 = 0.5235)	0.79
	Lateral Spread	1.12
North Side Slope (Bent 4) (4H:1V)	End of Construction	5.25
	Long Term	2.51
	Rapid Drawdown from El 222 to El 212	1.86
	Seismic (k_h =A_S/2 = 0.5235)	1.26



Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Rapid Drawdown Condition from El 222 to El 212
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
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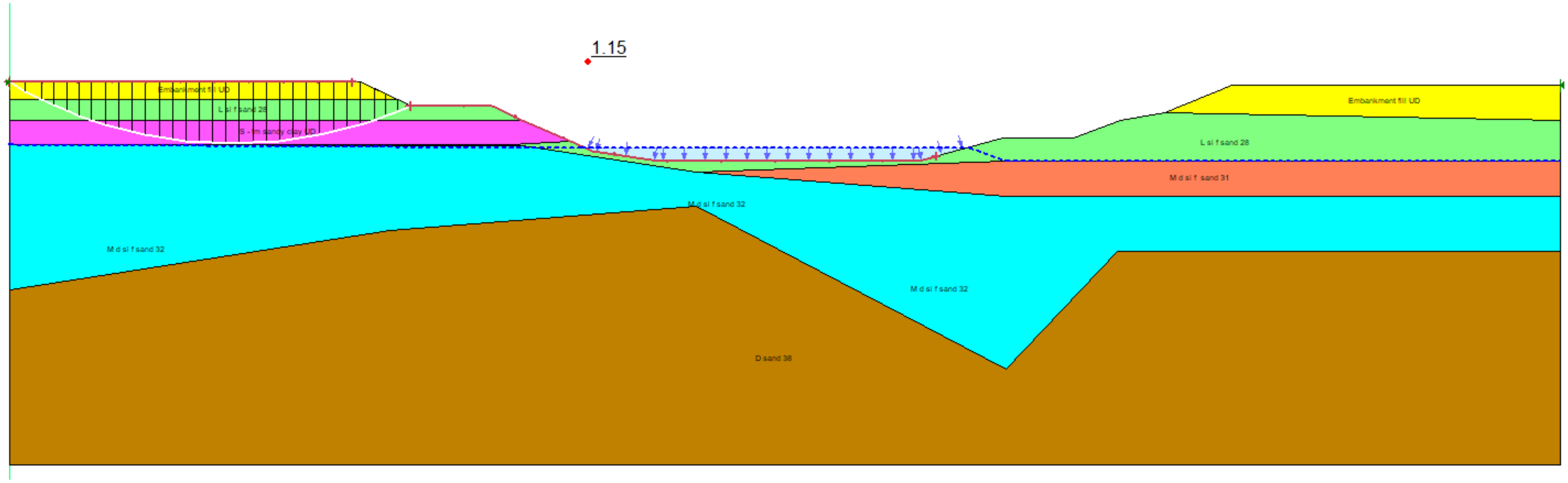
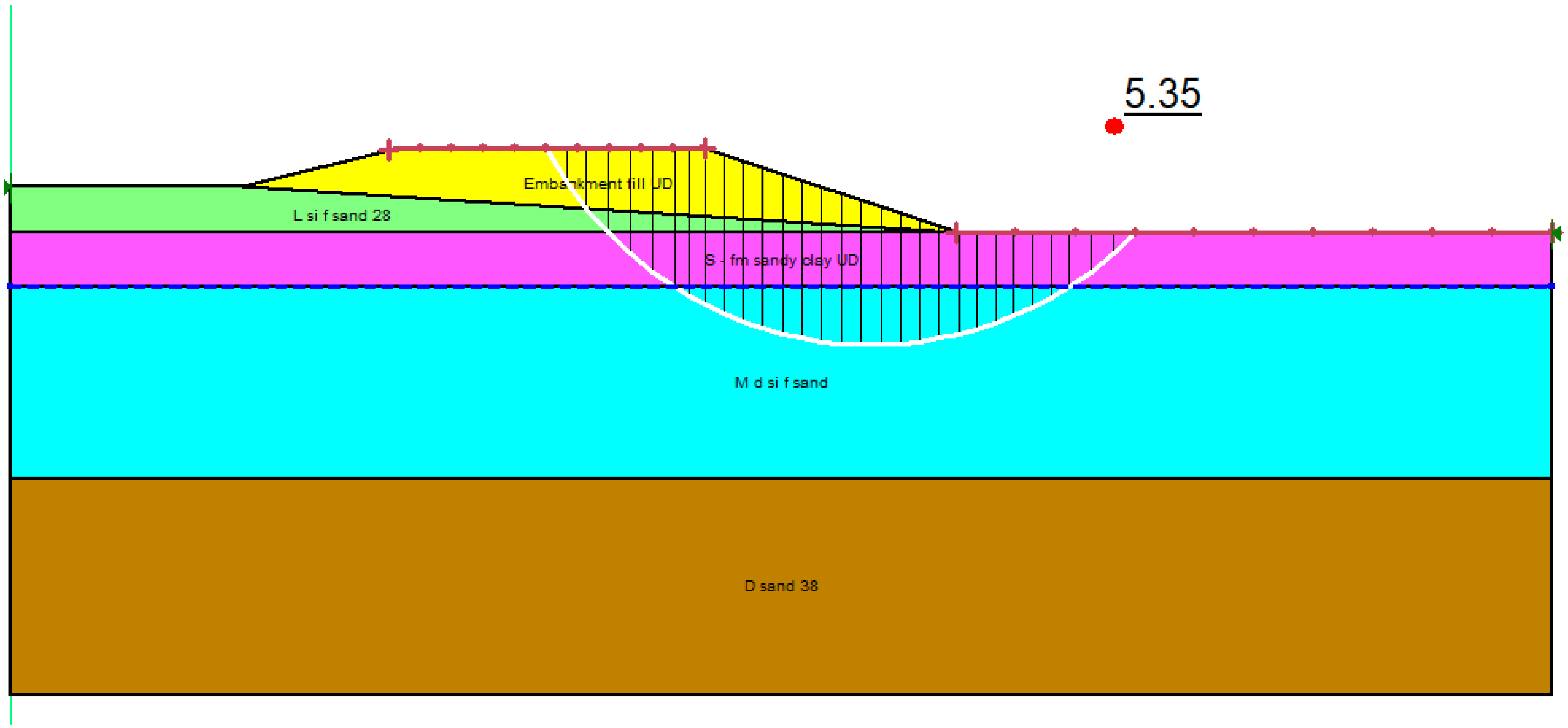
A UES Company

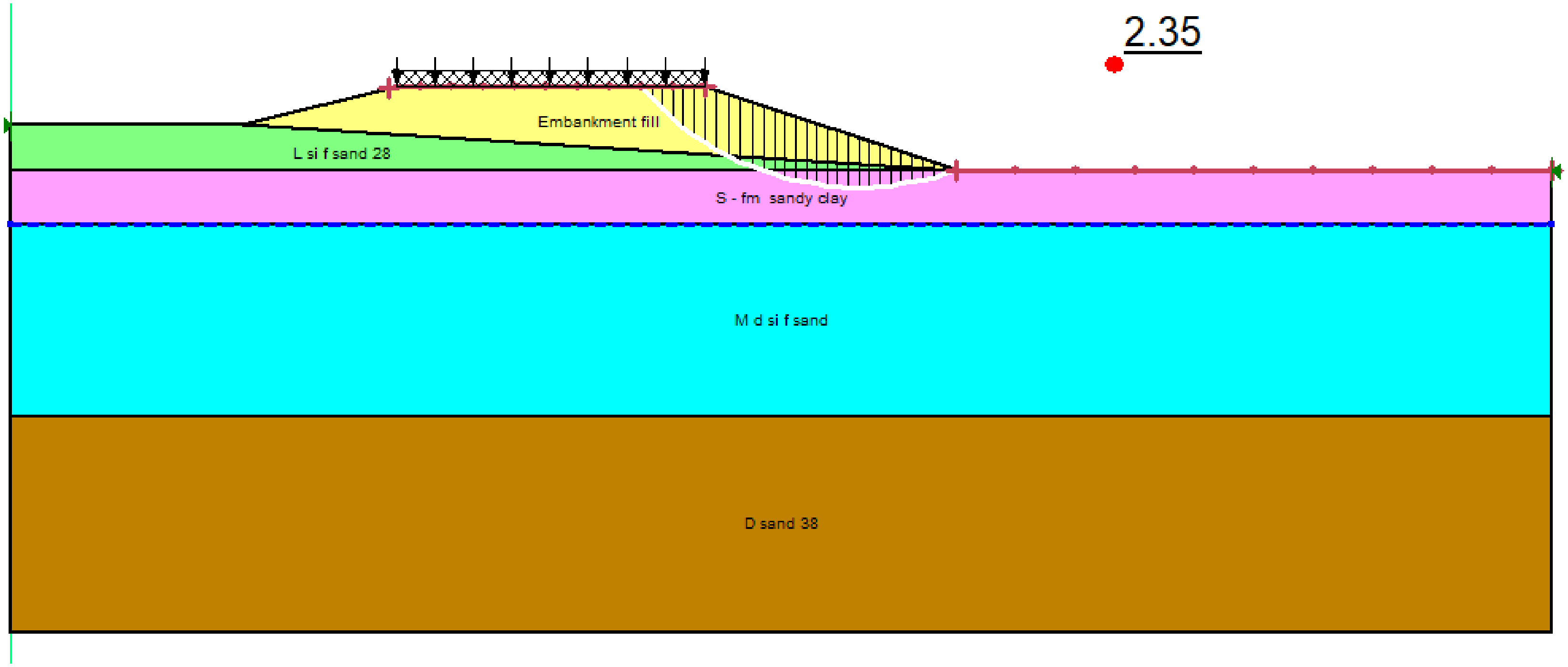


Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.5235$)
Bent 1 End Slope
2H:1V Slope, H=23 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

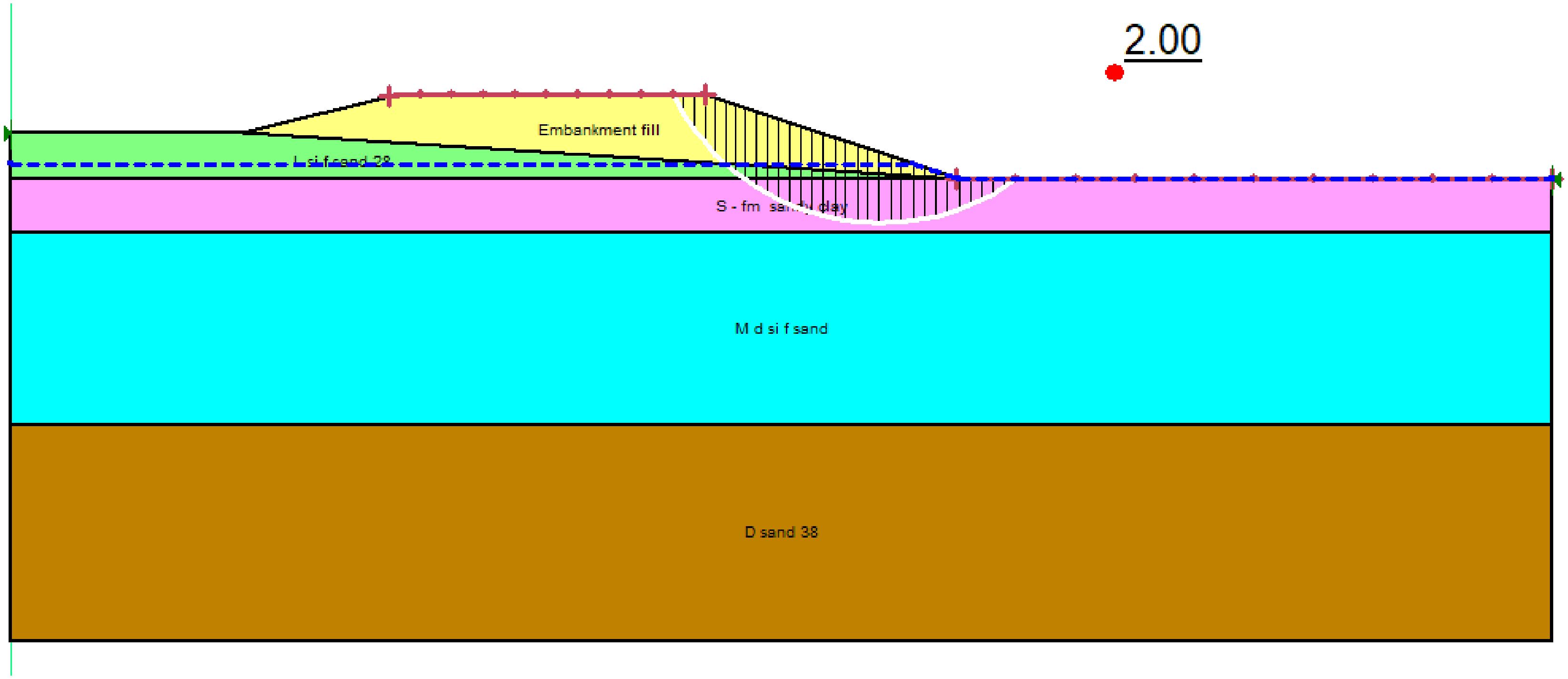


Results of Stability Analyses – End of Construction
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Rapid Drawdown Condition from El 222 to Existing Grade
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

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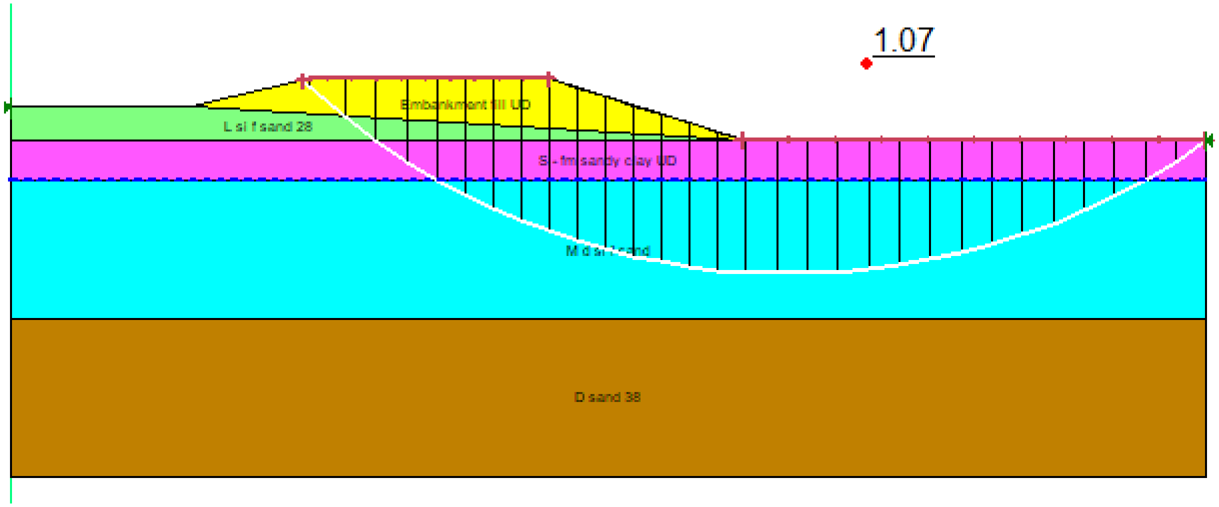
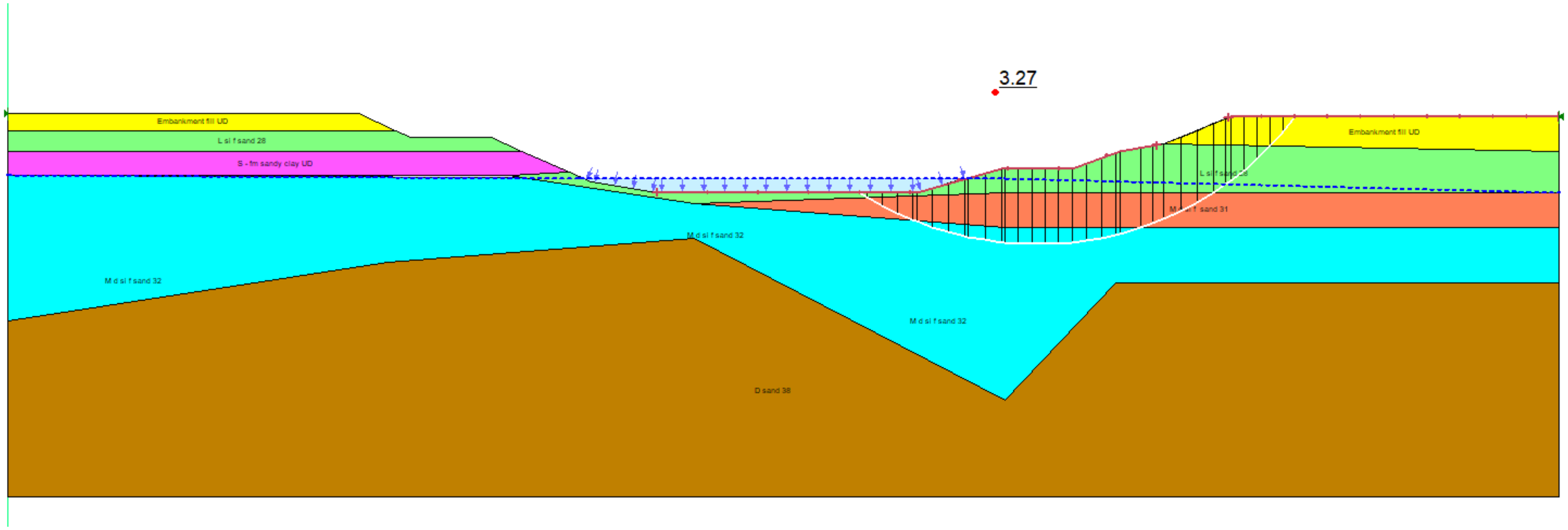
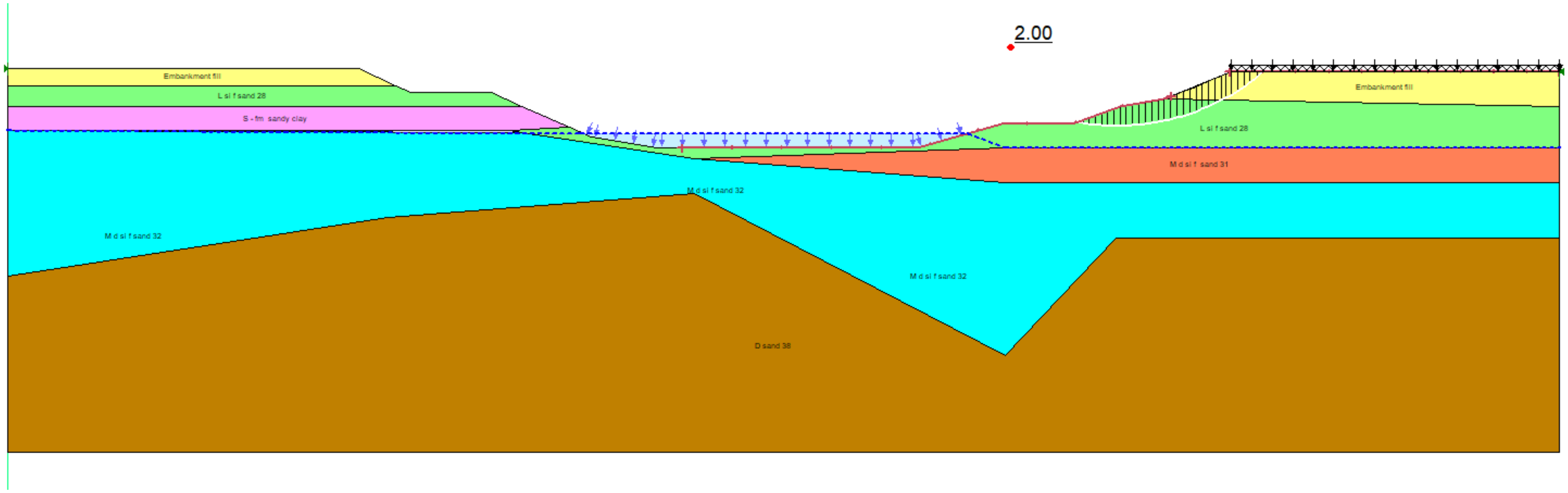
A UES Company



Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.5235)
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=22 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=22 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

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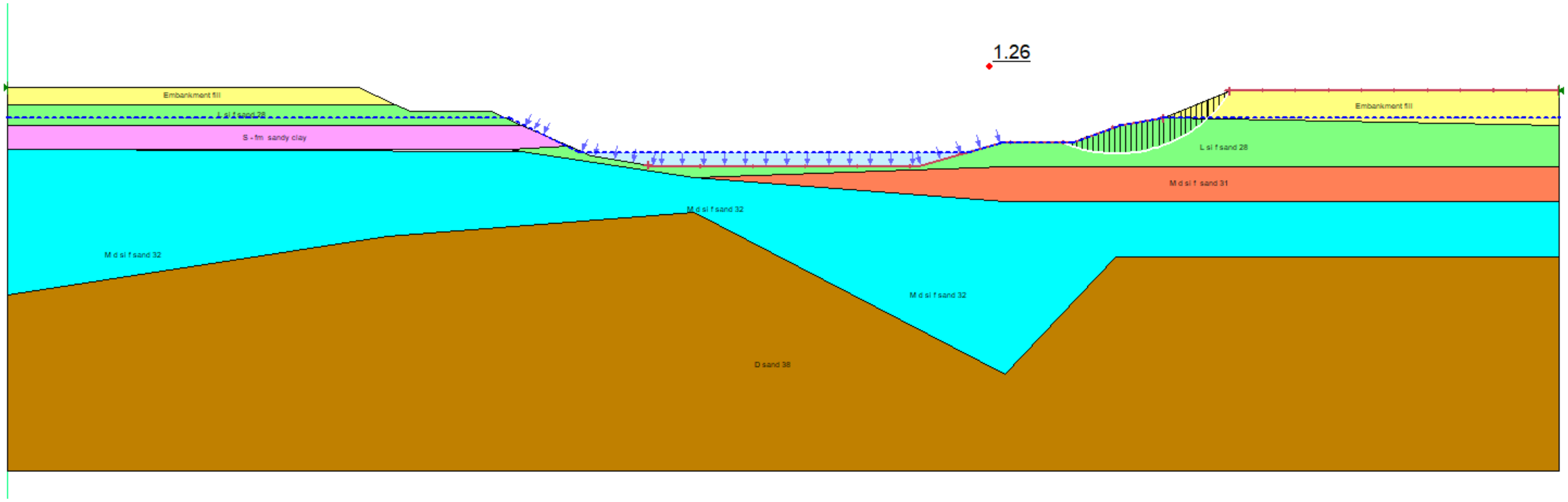
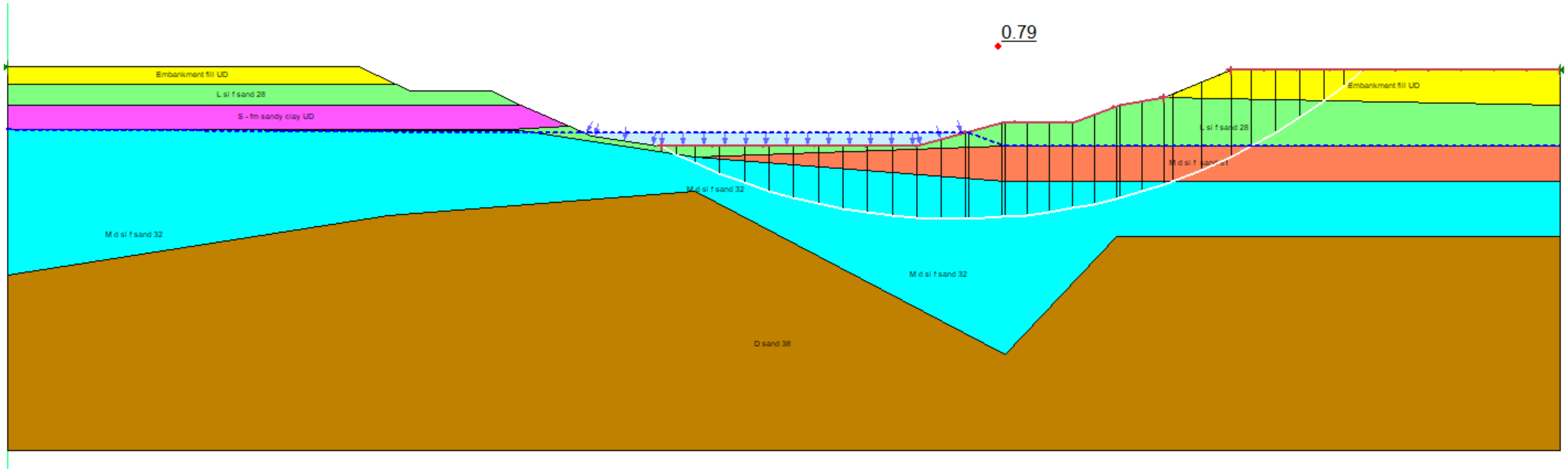
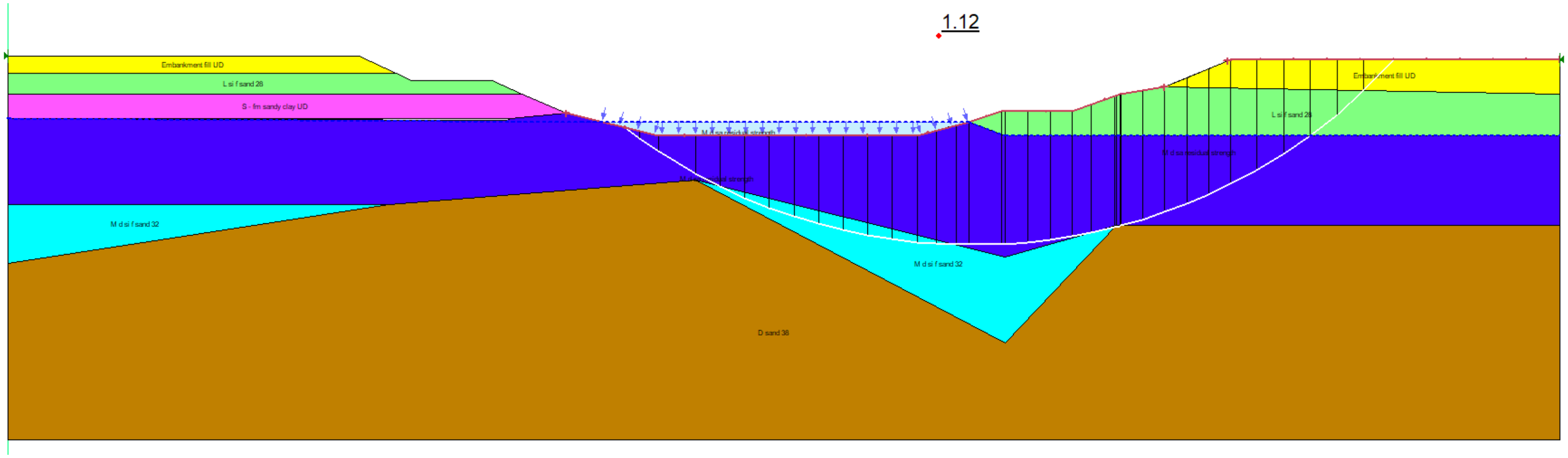


Results of Stability Analyses – Rapid Drawdown Condition, El 222 to El 212
Bent 4 End Slope
2H:1V Slope, H=22 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.5235)
Bent 4 End Slope
2H:1V Slope, H=22 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Lateral Flow
Bent 4 End Slope
2H:1V Slope, H=22 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
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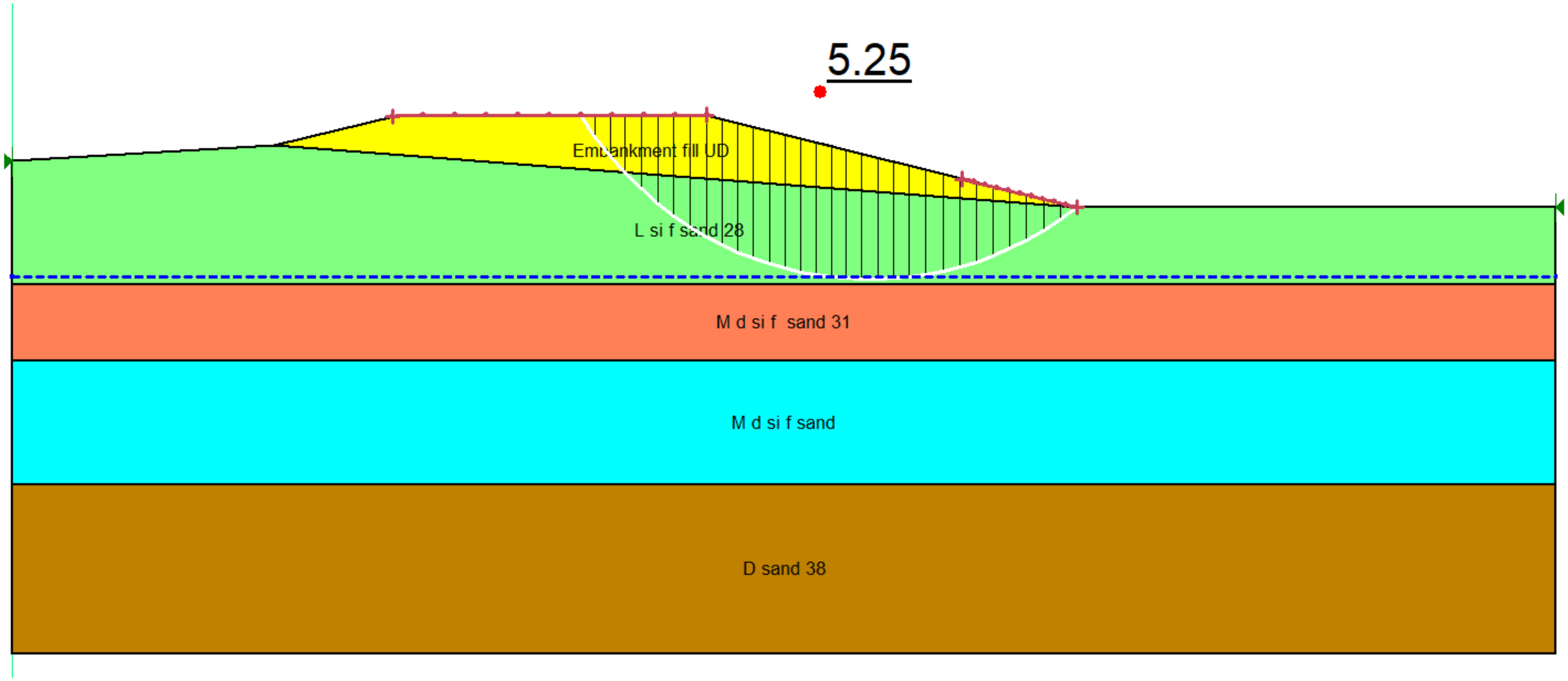
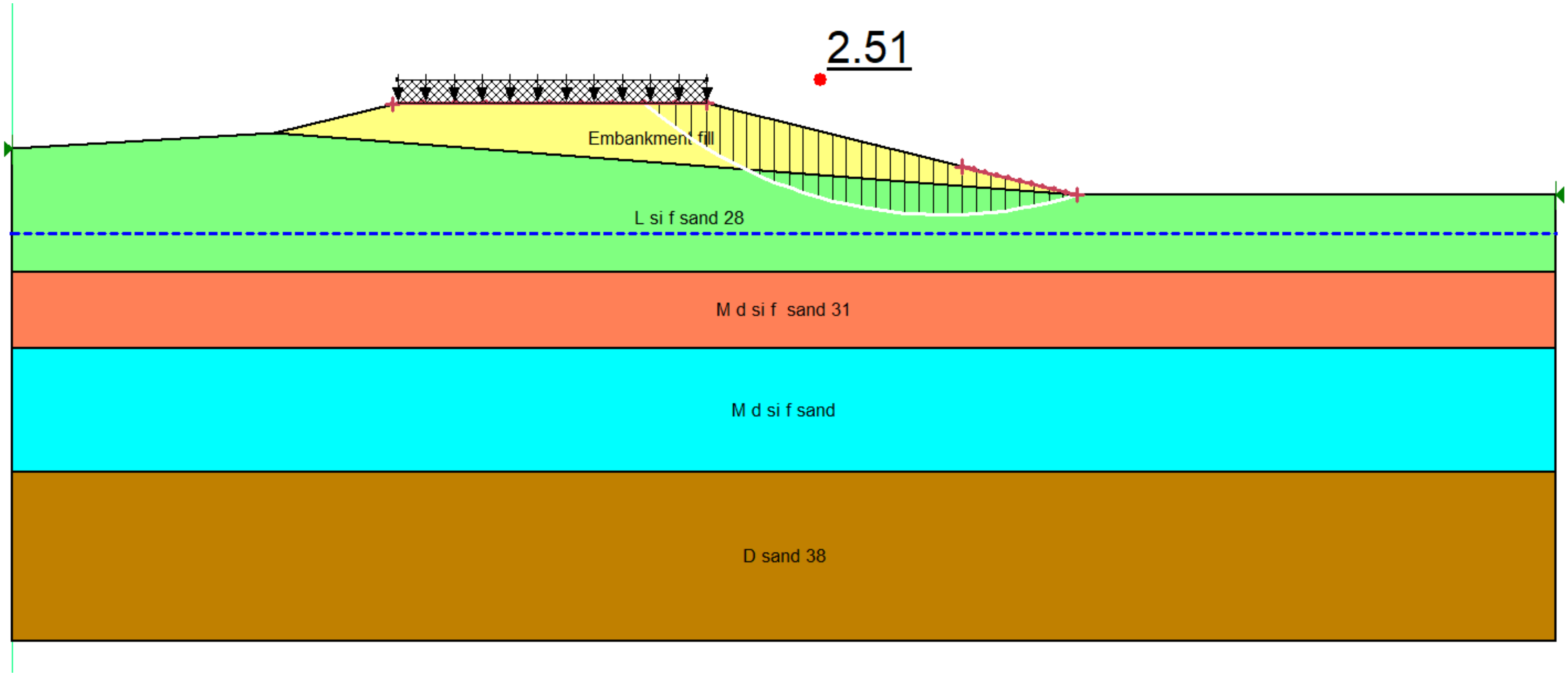
A UES Company



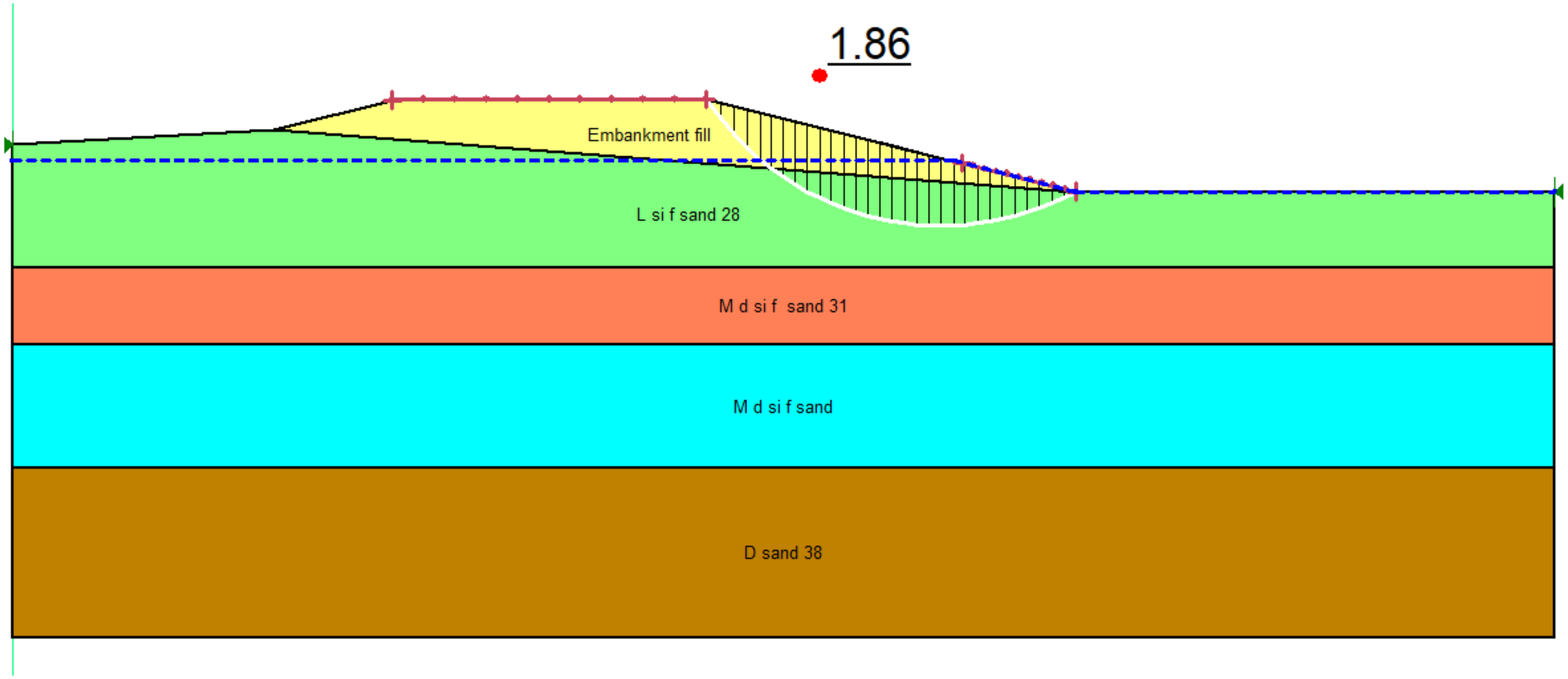
Results of Stability Analyses – End of Construction
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

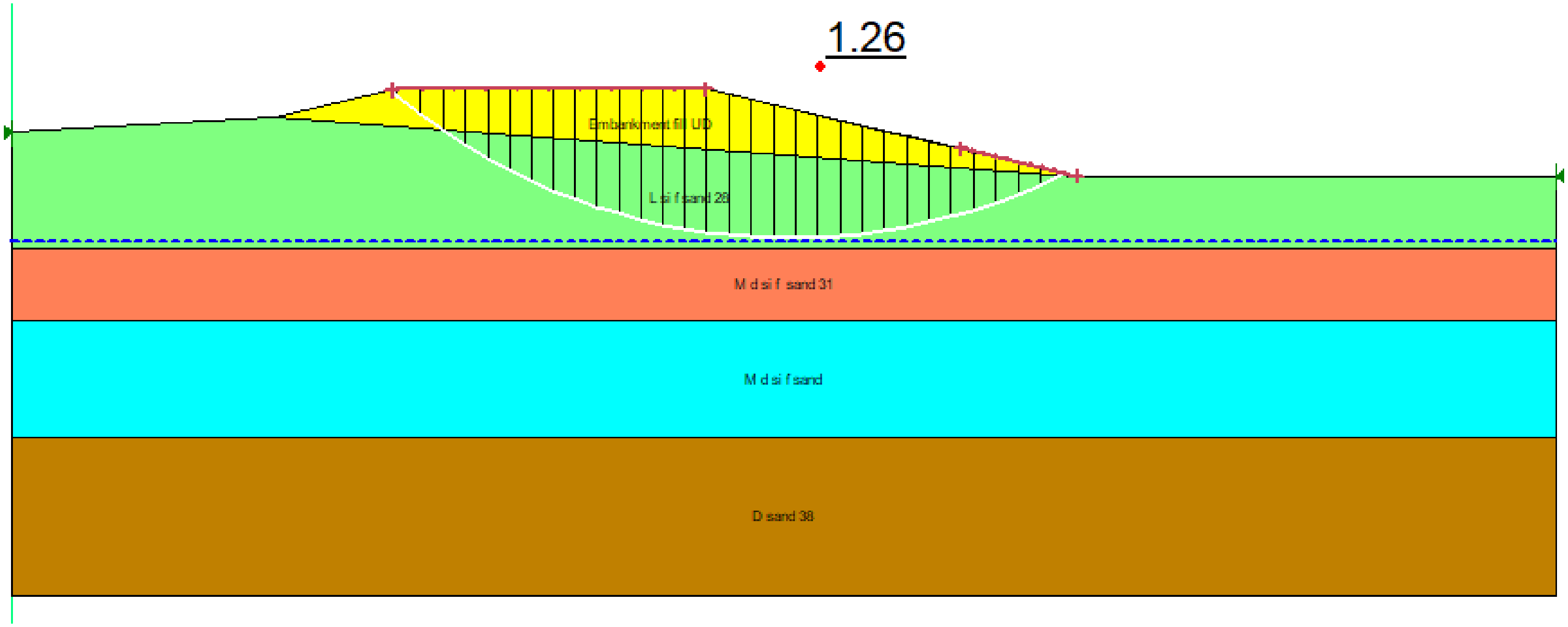


Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Rapid Drawdown Condition from El 222 to Existing Grade
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.5235$)
Bent 4 Side Slope
3H:1V Slope, H=12 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Buffalo Creek

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

APPENDIX H

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer’s installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX I

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, “The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project.” This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX J

WEAP ANALYSES - STEEL SHELL PILES

Project: 101124 - Hwy 135
Poinsett County, Arkansas
GHBW Project No: 23-031

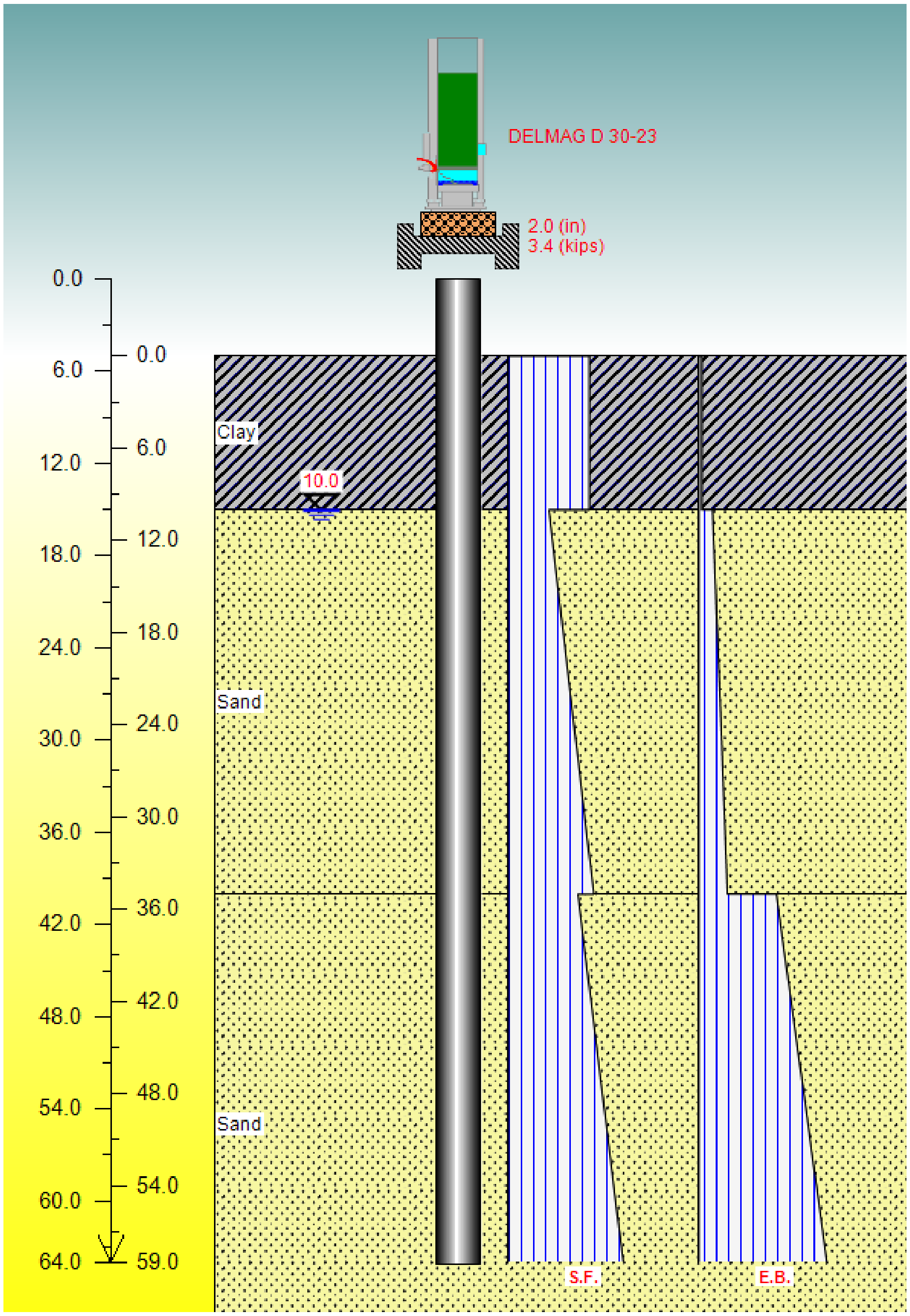
Bridge	Bent	Pile Diameter (in)	Wall Thickness (in)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El.	Min Tip El.	Pile Length (ft)	Min Hammer Energy (ft-kip)	Max Comp Stress, ksi	Notes
6 - Buffalo Creek	1	18	0.75	356	223	164	59	74	30.6	
	2	28	0.75	611	208	128	80	186	36.3	Tip at El 128
	3	28	0.75	611	208	143	65	186	37.3	Tip at El 143
	4	18	0.75	356	223	173	50	74	34.8	

ArDOT 101124 Hwy 135 over Buffalo Creek

Bent 1

18-in-diameter Steel Shell Pile

Delmag D30-23



Grubbs, Hoskyn, Barton & Wyatt, LLC
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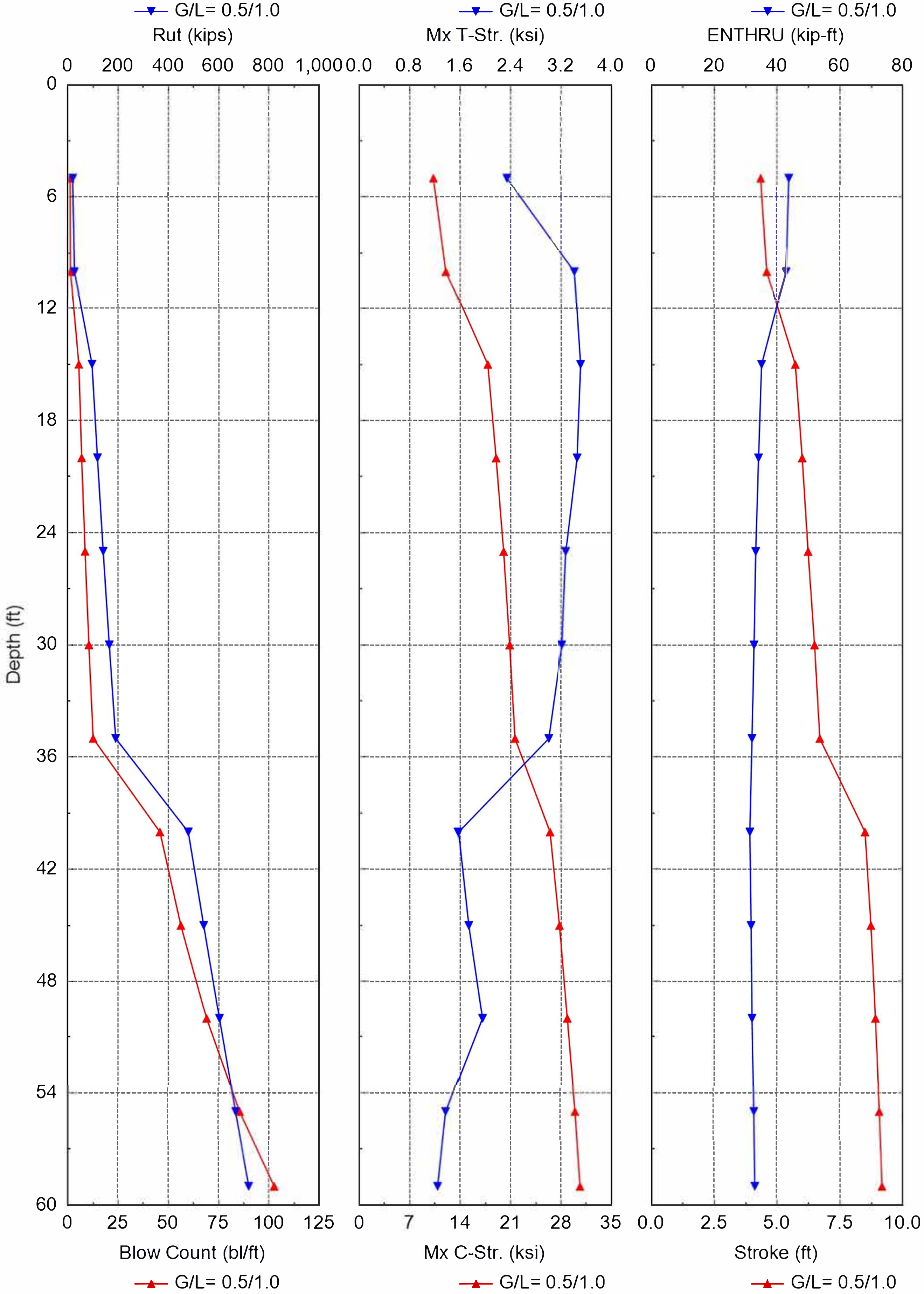
A UES Company


G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.7	6.8	11.9	1.1	10.302	2.338	4.34	43.7	D 30-23
10.0	25.8	13.8	11.9	1.3	12.040	3.411	4.58	42.9	D 30-23
15.0	96.4	20.2	76.1	5.5	17.787	3.507	5.72	35.1	D 30-23
20.0	118.0	27.9	90.1	6.9	18.935	3.452	6.00	34.1	D 30-23
25.0	140.9	36.9	104.0	8.6	19.988	3.273	6.23	33.2	D 30-23
30.0	165.0	47.1	117.9	10.5	20.827	3.212	6.49	32.7	D 30-23
35.0	190.5	58.6	131.9	12.6	21.567	3.003	6.70	32.0	D 30-23
40.0	479.1	69.2	409.9	45.8	26.443	1.569	8.49	31.3	D 30-23
45.0	540.5	81.1	459.3	56.2	27.755	1.730	8.73	31.7	D 30-23
50.0	603.1	94.4	508.7	69.0	28.839	1.949	8.91	32.0	D 30-23
55.0	667.2	109.0	558.2	85.4	29.930	1.371	9.06	32.5	D 30-23
59.0	719.4	121.7	597.7	102.6	30.630	1.245	9.17	32.9	D 30-23

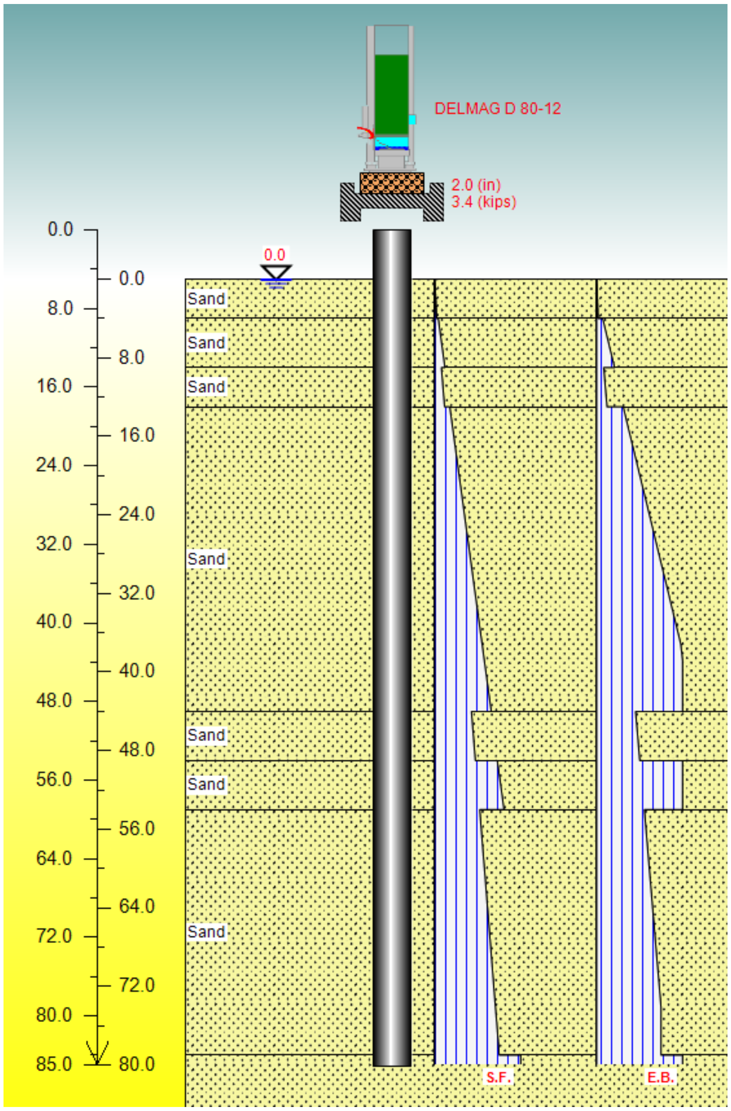
Total driving time: 41 minutes; Total Number of Blows: 1674 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Buffalo Creek

Bent 2

28-in-diameter Steel Shell Pile

Delmag D80-12



Grubbs, Hoskyn, Barton & Wyatt, LLC
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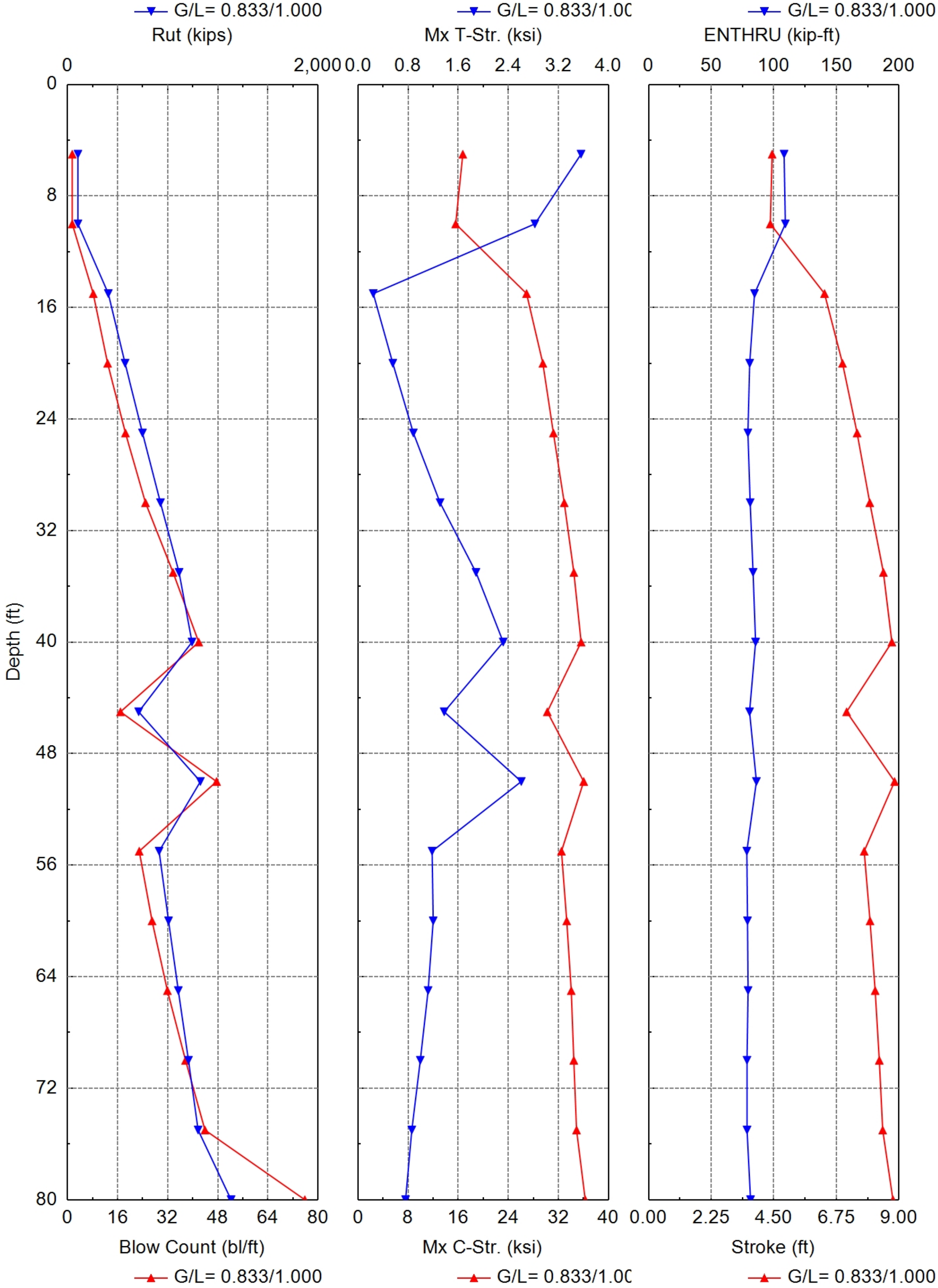
A UES Company


G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
8
16
24
32
40
48
56
64
72
80
0
16
32
48
64
80
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.00
2.25
4.50
6.75
9.00
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	82.8	1.0	81.8	1.5	16.735	3.562	4.44	108.2	D 80-12
10.0	82.9	6.2	76.7	1.5	15.592	2.825	4.38	109.2	D 80-12
15.0	325.4	14.4	311.0	8.2	26.918	0.243	6.33	84.7	D 80-12
20.0	459.5	29.2	430.3	12.8	29.517	0.555	6.97	80.8	D 80-12
25.0	598.4	48.9	549.5	18.5	31.205	0.883	7.50	79.4	D 80-12
30.0	742.2	73.3	668.8	24.9	32.936	1.310	7.95	81.1	D 80-12
35.0	890.7	102.6	788.1	33.7	34.472	1.884	8.44	83.5	D 80-12
40.0	993.9	136.6	857.4	41.9	35.637	2.319	8.75	85.5	D 80-12
45.0	567.9	172.5	395.4	16.9	30.229	1.376	7.12	80.7	D 80-12
50.0	1061.0	203.7	857.4	47.6	36.043	2.607	8.84	86.1	D 80-12
55.0	731.1	248.1	483.0	22.9	32.494	1.183	7.75	78.5	D 80-12
60.0	806.5	282.0	524.5	27.0	33.336	1.200	7.96	79.0	D 80-12
65.0	884.8	318.7	566.1	31.9	34.058	1.119	8.14	79.5	D 80-12
70.0	965.8	358.2	607.6	37.7	34.464	0.994	8.29	78.6	D 80-12
75.0	1042.7	400.5	642.1	43.9	34.903	0.859	8.42	78.7	D 80-12
80.0	1306.0	448.6	857.4	75.8	36.290	0.759	8.78	81.5	D 80-12

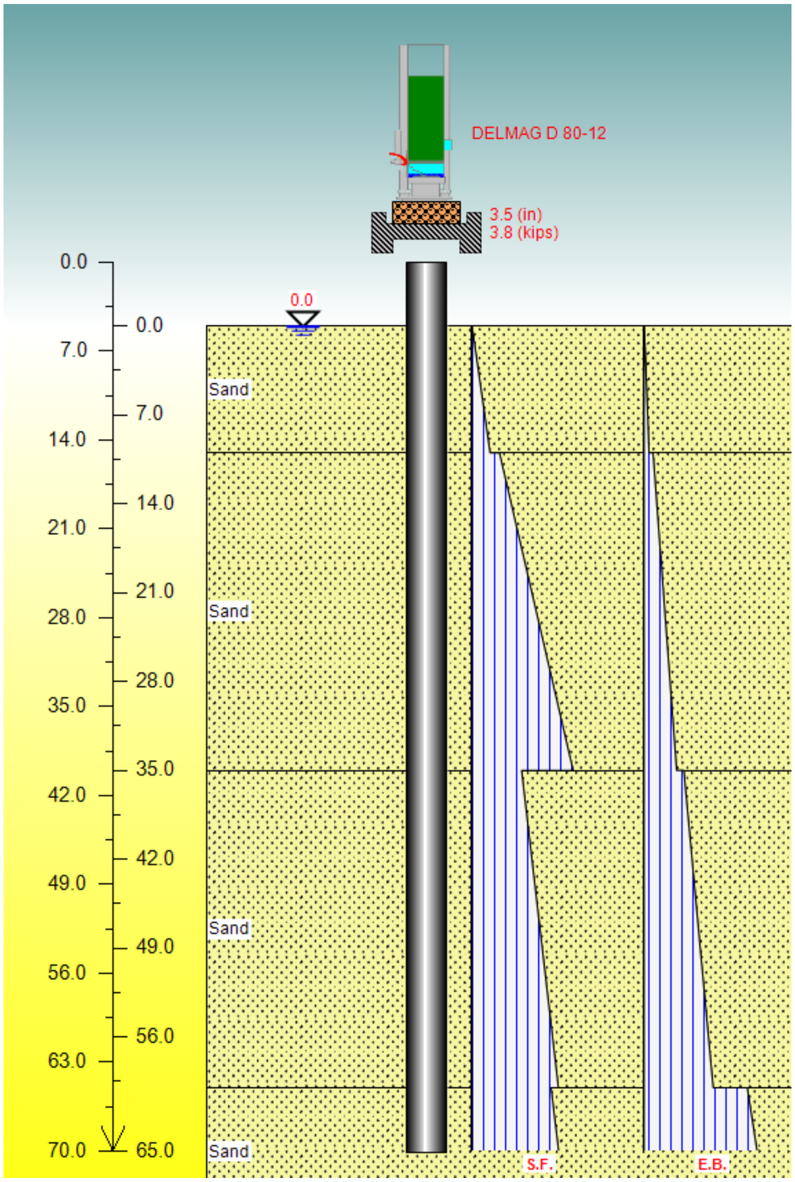
Total driving time: 49 minutes; Total Number of Blows: 2040 (starting at penetration 5.0 ft)

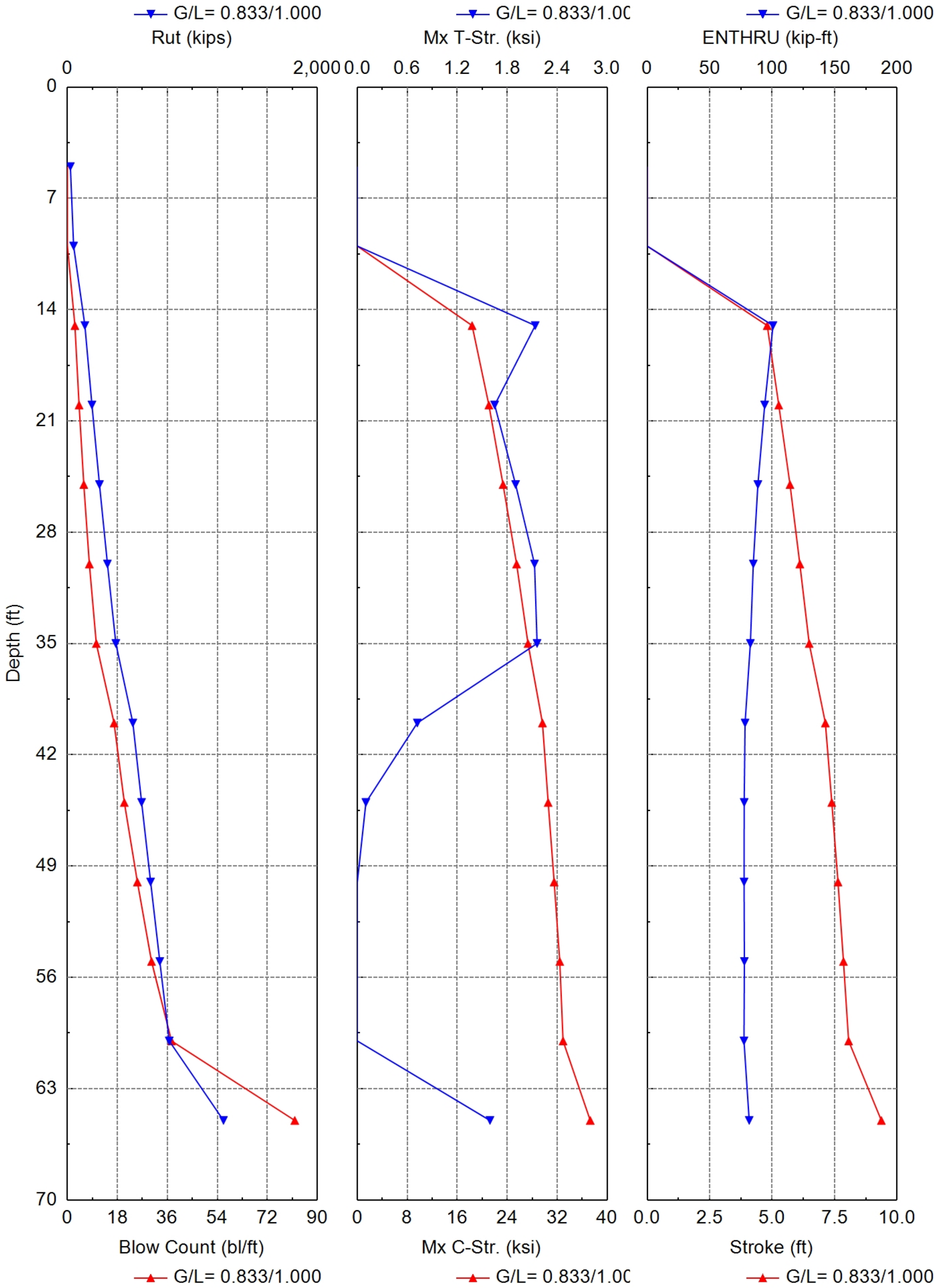
ArDOT 101124 Hwy 135 over Buffalo Creek

Bent 3

28-in-diameter Steel Shell Pile

Delmag D80-12





G/L= 0.833/1.000
Rut (kips)
0
2,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
50
100
150
200
Depth (ft)
0
7
14
21
28
35
42
49
56
63
70
0
18
36
54
72
90
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	23.6	1.1	22.5	0.3	0.000	0.000	10.57	0.0	D 80-12
10.0	49.5	4.4	45.1	0.0	0.000	0.000	0.00	0.0	D 80-12
15.0	139.8	13.0	126.8	2.7	18.381	2.136	4.80	100.7	D 80-12
20.0	196.3	25.2	171.1	4.2	21.064	1.650	5.27	94.1	D 80-12
25.0	256.5	41.0	215.5	5.9	23.333	1.901	5.72	88.6	D 80-12
30.0	320.3	60.5	259.8	7.9	25.504	2.128	6.11	84.9	D 80-12
35.0	387.7	83.6	304.2	10.4	27.326	2.159	6.48	82.6	D 80-12
40.0	523.8	96.7	427.2	16.8	29.642	0.721	7.13	78.4	D 80-12
45.0	594.0	111.6	482.3	20.5	30.558	0.103	7.39	77.6	D 80-12
50.0	665.9	128.3	537.5	25.2	31.517	0.000	7.64	77.5	D 80-12
55.0	739.6	146.9	592.7	30.3	32.415	0.000	7.86	77.8	D 80-12
60.0	815.2	167.3	647.9	37.5	32.941	0.000	8.06	77.5	D 80-12
65.0	1249.3	187.6	1061.6	81.9	37.295	1.593	9.37	81.7	D 80-12

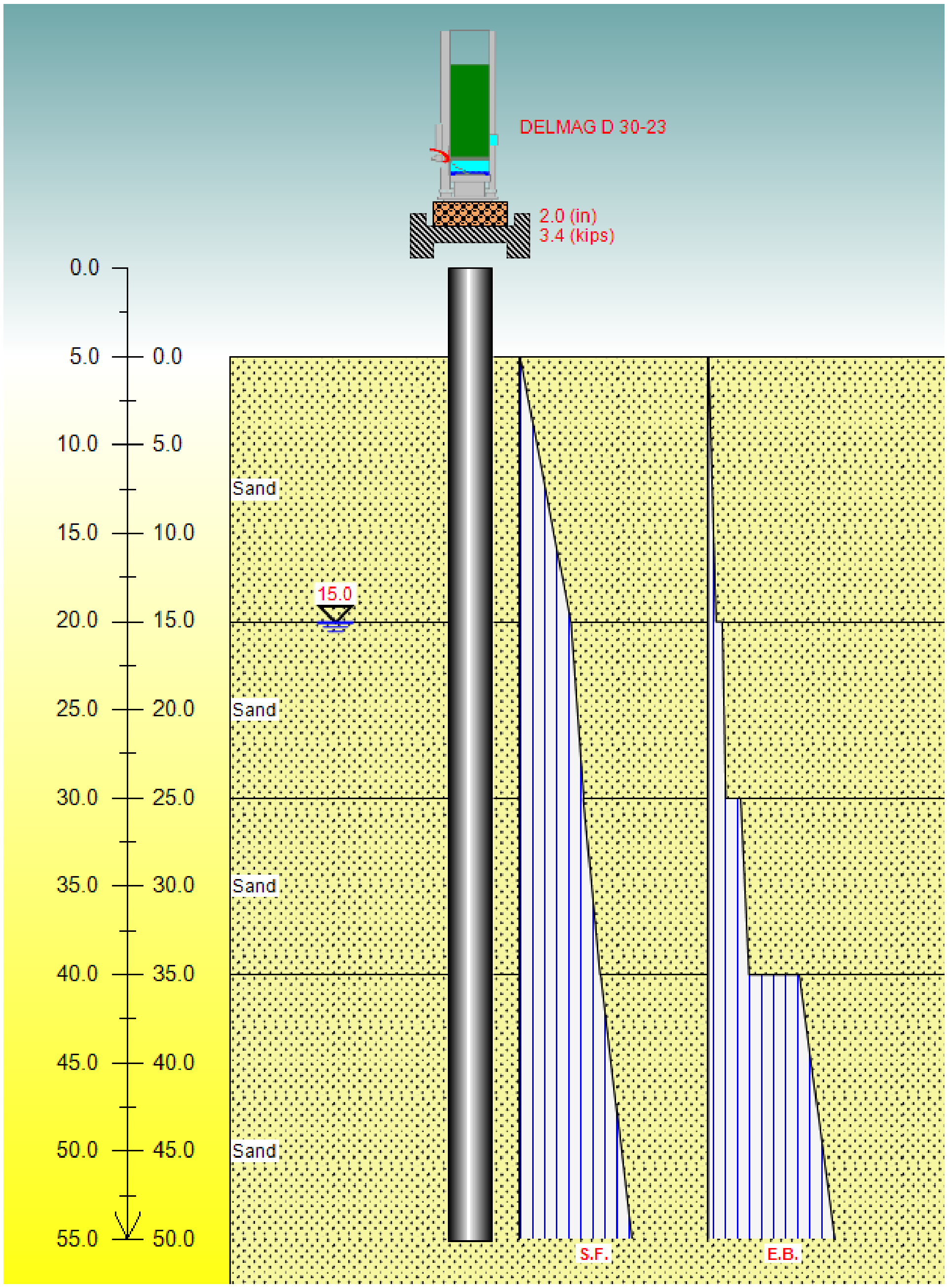
Total driving time: 24 minutes; Total Number of Blows: 1013 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Buffalo Creek

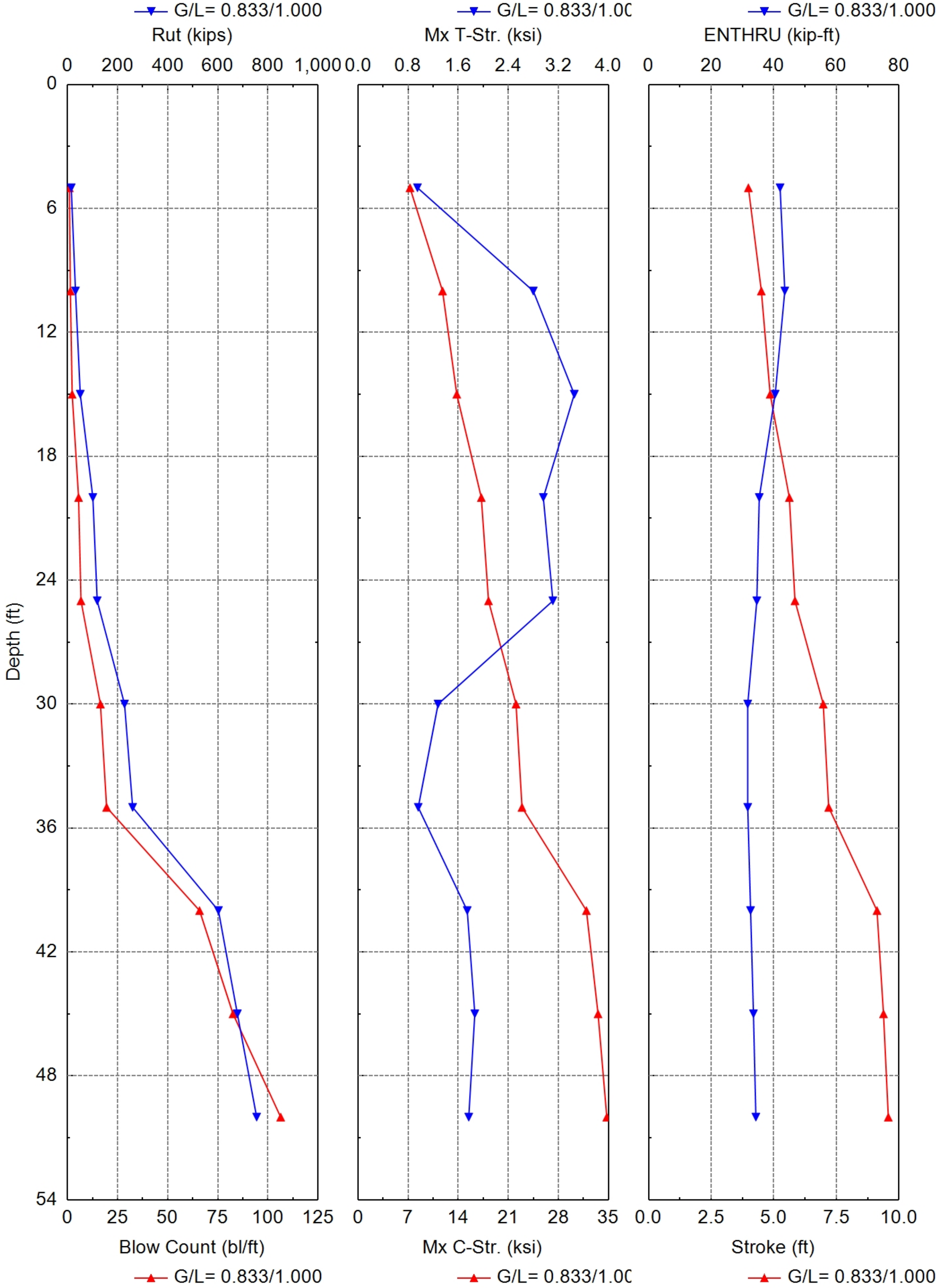
Bent 4

18-in-diameter Steel Shell Pile

Delmag D30-23



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company


G/L= 0.833/1.000
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
20
40
60
80
Depth (ft)
0
6
12
18
24
30
36
42
48
54
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.833/1.000
0
7
14
21
28
35
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	14.7	1.1	13.6	0.9	7.257	0.949	3.99	42.0	D 30-23
10.0	31.6	4.3	27.3	1.4	11.808	2.799	4.50	43.5	D 30-23
15.0	50.6	9.8	40.9	2.3	13.798	3.454	4.86	40.5	D 30-23
20.0	101.1	16.7	84.4	5.5	17.226	2.960	5.62	35.4	D 30-23
25.0	118.4	24.5	94.0	6.7	18.232	3.112	5.85	34.6	D 30-23
30.0	227.5	33.1	194.4	16.4	22.082	1.279	6.98	31.7	D 30-23
35.0	259.9	42.8	217.1	19.5	22.915	0.961	7.19	31.7	D 30-23
40.0	602.6	53.8	548.7	65.9	31.920	1.744	9.12	32.6	D 30-23
45.0	678.2	66.2	612.0	82.6	33.548	1.865	9.38	33.5	D 30-23
50.0	755.3	79.9	675.4	106.4	34.771	1.768	9.57	34.3	D 30-23

Total driving time: 31 minutes; Total Number of Blows: 1269 (starting at penetration 5.0 ft)

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

Materials Testing
Geotechnical Engineering
Environmental
Building Sciences & Safety
Inspections & Code Compliance
Virtual Design Consulting

September 15, 2023
Job No. 23-031

Arkansas Department of Transportation
10324 Interstate 30
Little Rock, Arkansas 72209

Attn: Ms. Jessica Jackson, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
HWY. 135 OVER UNNAMED DITCH (SITE 7)
ARDOT 101124 HWY. 135 STR. & APPRS. (S)
CRAIGHEAD COUNTY, ARKANSAS

INTRODUCTION

This report provides the final results of the geotechnical investigation performed for the Hwy. 135 over Unnamed Ditch replacement bridge in Craighead County, Arkansas. This bridge is Site 7 of the ARDOT 110124 Hwy. 135 Strs & Apprs (S) project. The ARDOT Job 110124 geotechnical investigation was authorized by the Arkansas Department of Transportation Task Order No. G001 on March 31, 2023. Notice to proceed with the field studies was received on April 1, 2023. Preliminary results and design recommendations have been provided throughout the course of this study. An interim report for this project site was submitted on May 26, 2023. This revised report supersedes the previous submittal of September 10, 2023.

We understand the replacement bridge over Unnamed Ditch will be a prestressed concrete girder unit with four (4) bents, three (3) spans, and a total length of approximately 151 feet. We also understand that a foundation system consisting of steel shell piles is planned at the bridge ends and intermediate bents. Foundation loads of the new bridge are anticipated to be moderate. Simple slopes will be utilized at the bridge ends with end slopes at approximate 2-horizontal to 1-vertical (2H:1V) configurations and side slopes at 3-horizontal to 1-vertical (3H:1V) configurations. The replacement bridge will be constructed west of the existing bridge. Site grading will include about 14 ft of fill. A preliminary bridge layout is provided in Appendix A.

The purposes of this geotechnical study were to explore subsurface conditions in the alignment of the replacement bridge and the approach embankments. The data developed through the field and laboratory studies were utilized to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been accomplished by a multi-phased study that included the following.

- Drilling sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing.
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations and conclusions for seismic site class, seismic design category/seismic performance zone, liquefaction potential, ground improvement, foundation design, embankment configurations, 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 in the Site 7 replacement bridge alignment were explored by drilling four (4) sample borings to 110- to 120-ft depth (Borings G1 to G4). The boring locations were selected by the Designer (Crafton Tull) and adjusted as required for site access. The site vicinity is shown on Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2.

The subsurface exploration program is summarized in the table below.

Table 1: Summary of Exploration Program

Boring No.	Approx Sta	Approx Offset, ft	GPS Coordinates (degrees)		Approx Surf El, ft	Completion Depth, ft
			Latitude	Longitude		
G1	427+90	10 ft Rt	35.700572	-90.341287	225.8	110
G2	428+25	25 ft Lt	35.700673	-90.341405	214.1	110
G3	428+75	CL	35.700815	-90.341327	213.0	120
G4	429+40	25 ft Lt	35.700985	-90.341399	223.2	120

The boring logs, presenting descriptions of the soil strata encountered in the borings and the results of field and laboratory tests, are included as Plates 3 through 14. The centerline station and offset of the boring locations and approximate ground surface elevation, surveyed, are also shown on the logs. A key to the terms and symbols used on the logs is presented as Plate 15.

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.

The borings were drilled with a truck-mounted SIMCO 2800 rotary-drilling rig and a track-mounted Diedrich D-50 rotary-drilling rig. The bridge borings were advanced using a combination of dry-auger and rotary-wash drilling methods. 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 portion thereof, is defined as the Standard Penetration Number (N). SPT N_{60}-values are shown on the boring logs in the "Blows Per Ft" column. The drilling rig utilized for each particular boring and the appropriate energy conversion factor is shown on each boring log.

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

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

LABORATORY TESTING

Laboratory testing was performed to evaluate subgrade and foundation soil plasticity and to confirm visual classification. The testing program included natural water content determinations (AASHTO T 265), liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90), and sieve analyses (AASHTO T 88). Soil shear strength or relative density was estimated in the field using SPT results.

Laboratory test results are shown on the logs at the appropriate depth. A total of 11 natural water content determinations were performed to develop data on in-situ soil water content for each boring. The results of these tests are plotted on the logs as solid circles, in accordance with the scale and symbols shown in the legend located in the upper-right corner.

To verify field classification and to evaluate soil plasticity, 3 liquid and plastic (Atterberg) limit determinations and 31 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.

A summary of classification test results and classification by the Unified Soil Classification System and AASHTO Classification System is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The location of 101124 Site 7 is on Hwy. 135 where the Unnamed Ditch channel crosses the highway alignment immediately north of CR 890 in Craighead County. The existing bridge is a two-lane structure with a concrete deck, steel girders, and a concrete pile foundation system. The channel at this location is narrow with well-defined banks. The banks are steep and lined with grass, variable sparse to thick underbrush, and occasional small trees. Drainage features are present in both the southern and northern bents of the proposed bridge. Riprap has been locally placed over the drainage channels, but erosion is apparently still on-going. The project locale is primarily agricultural land consisting of open flat fields. An abandoned barn is located northeast of the existing bridge. The existing two-lane roadway is on embankment and the existing pavements are in poor condition. Surface drainage along the roadway is poor to fair and standing water is common after rain events.

Site Geology

The project alignment is located in the Gulf Coastal Plain Physiographic Province. The geology of this area is typified by Recent Alluvium and variable Tertiary sediments. The Geologic Map of Arkansas[1] indicates the alignment extends through exposures of Quaternary Terrace

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[1] Geologic Map of Arkansas; US Geological Survey and Arkansas Geological Commission; 1993

Deposits. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The depth of bedrock (Paleozoic rocks) in this area is reported to exceed 2200 feet.

Seismic Conditions

In light of the results of the borings and the surface geology, a Seismic Site Class D (stiff soil profile) is considered applicable to the bridge location at Site 7 with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[2]. Given the location and AASHTO code-based values, preliminarily recommended seismic parameters are summarized below.

- Seismic Site Class D
- 1.0-sec period spectral acceleration coefficient (S_1) = 0.539
- Site amplification factor at 1.0 second (F_v) = 1.5
- 1.0-sec period spectral acceleration coefficient (S_{D1}) = 0.809
- Acceleration for a short (0.2 sec) period (S_S) = 1.876
- Site amplification factor for short period (F_a) = 1.0
- Peak ground acceleration (PGA) = 1.047
- Site amplification factor at PGA (F_{PGA}) = 1.0
- A_s = 1.047

Utilizing these parameters, AASHTO LRFD Seismic Bridge Design Specifications indicate that a Seismic Performance Zone 4 and a Seismic Design Category (SDC) D are fitting for the Site 7 location of the Hwy. 135 bridge over Unnamed Ditch.

Liquefaction Analyses

Liquefaction analyses were performed to evaluate the liquefaction potential of the subsurface soils. The analyses were performed utilizing the results of the borings and the methodology and procedures proposed by Idriss and Boulanger[3] in 2008. A design PGA value of 1.047 and an earthquake Moment Magnitude (M_w) of 7.7 were utilized in the liquefaction analyses.

The results of the liquefaction analyses are provided in Appendix D as plots of calculated factors of safety against liquefaction potential. The potentially liquefiable zones indicated by the analyses results are shown on the generalized subsurface profile also provided in Appendix D. Isolated zones of calculated liquefaction triggering in excess of about 50-ft depth which are separated from shallower zones of liquefaction triggering by relatively thick zones of non-

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

[3] "Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, MNO-12, Idriss and Boulanger, 2008.

triggering soils, are considered to pose a low risk of liquefaction. These deeper zones have not been considered liquefiable in development of the plot shown in Appendix D.

Subsurface Conditions

Based on the results of the borings, the surface soils to 2- to 4-ft are locally comprised of on-site fill consisting of very loose to medium dense tan, brown, and dark brown silty fine sand (SM) with occasional fine to coarse gravel. The results of the borings indicate that the fill compaction is poor to good, with variable compressibility and relative density. These soils typically classify as A-2 to A-4 by the AASHTO classification system (AASHTO M 145), which correlates with fair subgrade support for pavement structures.

Below the fill or at the ground surface to 6- to 22-ft is brown, gray, grayish brown, tan, and reddish tan loose to medium dense silty fine sand and fine sandy silt (SM, SP-SM, and ML) and soft to firm silty clay (CL). The silty fine sand contains silt seams and layers and occasional organic inclusions. The silty sand/sandy silt and silty clay exhibit low to moderate relative density or shear strength and moderate to high compressibility. These typically classify as A-2-4, A-3, A-4, and A-7-6 by the AASHTO classification system (AASHTO M 145), which correlates with poor to fair subgrade support for pavement structures. Relative density is generally medium below about 13 ft depth and compressibility decreases.

The basal unit encountered in the borings is medium dense to dense brown, gray, and brownish gray fine to medium sand strata (SP and SP-SM). Some coarse sand, organic inclusions, and fine gravel are present at depth. These granular units exhibit medium to high relative density and low compressibility. Relative density typically increases with depth.

Groundwater Conditions

Groundwater was encountered in the borings at 4- to 17-ft depth in May and June 2023. Groundwater levels will vary, depending upon seasonal precipitation, surface runoff and infiltration, and stream levels in the ditch and other surface water features.

ANALYSES and RECOMMENDATIONS

Foundation Design

Foundations for the Site 7 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 and liquefaction of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

Based on the results of the borings and the anticipated moderate bridge foundation loads, we recommend a deep foundation system comprised of piling be utilized to support the foundation loads at the abutments and interior bents of the new bridge. Steel shell piles are considered suitable foundations for this site. Given the likelihood of liquefaction triggering in strong seismic events, there is the potential for significant downdrag on piles due to liquefaction settlement. Recommendations for piling are discussed in the following report sections.

Piling

We recommend the bridge foundation loads be supported on a deep foundation system comprised of steel shell piles. We understand that 16-in.-diameter steel shell piles are planned for bridge ends and 24-in.-diameter steel shell piles are planned for the interior bents. All steel shell piles will be filled with concrete after initial driving. Shear rings, shear studs, or other equivalents may be considered on the inside walls of the steel shells to enhance bonding between the concrete and the steel shells.

Nominal single pile capacity curves are provided in Appendix E. Nominal axial pile capacities have been developed using static pile capacity formulae, the results of the borings, and the plan pile cap bottom elevations shown on the preliminary bridge layout drawings.

Pile capacity was evaluated for "static" conditions prior to a seismic event, with no liquefaction, and full soil shear strength is mobilized for the foundation soils. For the case where liquefaction occurs, the "end of earthquake" condition was evaluated as the condition immediately after occurrence of the design earthquake. In this case, the foundation soils are liquefied and full excess pore water pressure is generated. Consequently, residual shear strength of full liquefaction is utilized for the liquefied foundation soils. Downdrag is assumed to be mobilized on the piles by the liquefied soils and soils above the liquefied zone as a result of liquefaction settlement.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.25 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading and collision, resistance factors of 1.0 for compression and 0.8 for uplift.

The recommended nominal axial capacities are based on single, isolated foundations. Piles spaced closer than three (3) pile diameters may develop lower individual capacity due to group effects.

The potential for group capacity reductions should be evaluated for pile spacing closer than three (3) diameters.

Battered piles can be utilized to resist lateral loads. The axial capacity of battered piles may be taken as equivalent to that of a vertical pile with the same tip elevation and embedment. Special driving equipment is typically required where pile batter exceeds about 1-horizontal to 4-vertical.

We understand that a detailed lateral load analysis will be performed by others. Recommended parameters for use in lateral load analyses are summarized in Appendix F.

End Slope Stability

The replacement bridge will include new end slope configurations on the south (Bent 1) and north (Bent 4) ends. Plan bridge end embankment configurations are 2-horizontal to 1-vertical (2H:1V) with 3-horizontal to 1-vertical (3H:1V) side slope configurations. The bridge end embankments will have maximum heights of about 23 feet.

To evaluate suitability of the end slope plan configurations, slope stability analyses have been performed. A 250 lbs per sq ft uniform surcharge from vehicles was included for the purposes of stability analyses. Stability analyses were performed using the computer program SLOPE/W 2020[4] and a 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 0.5235. This $A_s/2$ value was developed as one-half of the peak ground acceleration (PGA) value from the site-specific seismic hazard analysis. For evaluation of the rapid drawdown condition, a water surface elevation drop from El 225 to El 212 was assumed.

Stability analyses results are summarized and presented graphically in Appendix G. As shown in the results, the analyses of the seismic stability of the plan 2H:1V end slopes indicates a calculated minimum factor of safety less than 1.05. However, a simplified Newmark block analysis indicates that the maximum permanent displacement is between 2.5 and 2.7 inches for the north and south embankments, respectively. We understand that a Newmark displacement of less than 6 inches is considered acceptable for bridges designated as "Other."

The results of slope stability analyses utilizing residual strengths in soil zones susceptible to liquefaction triggering indicate a calculated minimum factor of safety against sliding in excess

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[4] Slope/W 2020; GEO-SLOPE International; 2020.

of 1.0. Consequently, the potential for flow slide instability is considered low. Given the results of the stability analyses and Newmark block analysis, the stabilities of the slope configurations are considered acceptable. In addition, a suitable factor of safety against lateral flow was calculated for all cases.

Subgrade Support

It is understood that "standard" pavement sections will be utilized by the Department. Based on the results of the borings and laboratory tests, the on-site subgrade soils are expected to be comprised primarily of embankment fill. The on-site soils are anticipated to predominantly classify by AASHTO M 145 as A-3 and A-4. These classifications correlate with fair to poor subgrade support for pavements. Locally-available borrow, which is likely to be used as unclassified embankment fill, is expected to have similar classification.

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

- Resilient Modulus (M_R): 2400 lbs per sq inch
- R value: 4

The approach road pavement subgrade should be evaluated by the Engineer or Department at the time of construction. Areas of unstable or otherwise unsuitable subgrade should be improved by undercut and replacement or treatment with additives as approved by the Engineer. Depending on seasonal site conditions and final grading plans, localized undercuts or improvement depths on the order of 2 to 3 ft below existing grades, more or less, could be warranted to develop a stable subgrade.

We recommend that any soils classifying as AASHTO A-7-5 or A-7-6 and soils and with a plasticity index (PI) in excess of 18 be excluded from use as subgrade within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18.

Site Grading and Subgrade Preparation

Site grading and site preparation in the bridge alignment should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. The depth of stripping will be variable, with deeper stripping depths in wooded areas, and less stripping required in open areas. In general, the stripping depth is estimated to be about 6 to 9 in. in cleared areas but may be 18 to 24 in. or more in areas with thick underbrush and/or trees. 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 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. Any abandoned piling should be cut off at least 3 ft below final grade.

Following required pavement demolition, clearing and grubbing, and stripping, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and/or unsuitable soils should be determined. Thorough proof-rolling should be performed to verify subgrade stability. 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, or other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, localized undercutting could be required to develop subgrade stability. The zone of weak soils which could potentially be unstable subgrade typically extends to depths of 4 to 13 ft below existing grades. Consequently, the maximum undercut depth for subgrade improvement has been estimated to be about 3 ft based on the anticipated use of stone backfill (ARDOT Standard Specifications Section 207). Where embankment heights exceed 4 ft after light stripping, the stone backfill may be placed on the subgrade and grades raised above the stone. Where grades are raised over soft subgrade by placing stone backfill, we recommend that the stone backfill be placed on a heavy subgrade support geotextile. An example special provision for this geotextile is provided in Appendix H. Where embankment heights are less than about 4 ft, undercutting will be required to keep the stone backfill below the embankment face. The undercut depth should be sufficient to provide at least 1 ft of earthen embankment fill over the top of the stone backfill.

Stone backfill should not be utilized in areas where structural piles will be driven. Where there will be potential conflicts with driven piles, subgrade improvement should be achieved by use of sand fill over heavy subgrade support geotextile. Depending on sand properties, a lift thickness of 2 to 3 ft or more could be required to achieve a stable working platform for additional fill compaction. Where the heavy subgrade support geotextile is used, at least 2 ft of fill over the geotextile will be required to contain the geotextile during pile driving. Use of stabilization additives can be considered as an alternate to stone backfill to stabilize the subgrade in areas where piles will be driven.

In lieu of undercutting and replacing unsuitable or unstable soils, consideration may be given to using additives to improve soil workability and 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 or Department. 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. The specific stabilization method for each site should be approved by the Engineer.

In the event that the subgrade is stable at the time of construction and required undercut depths are less than about 3 ft, undercut backfill may consist of embankment fill as approved by the Engineer. Subgrade conditions should be field verified by the Engineer based on specific observations during subgrade preparation.

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.

General 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. Given the high seismic zone, we recommend that new embankment fill consist of cohesive borrow within about 100 ft of the bridge ends. An example special provision for cohesive embankment fill is provided in Appendix I.

Subgrade preparation should comply with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 212. Embankments should be constructed in accordance with ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 210. Fill and backfill should be placed in nominal 6- to 10-in.-thick loose lifts. All fill and backfill must be placed in horizontal lifts. Where fill is placed against existing slopes, short vertical cuts should be "notched" in the existing slope face to facilitate bonding of horizontal fill lifts. The in-place density and water content should be determined for each fill 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. Density and water content of all earthwork should be maintained until embankments and bridge work are completed.

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

Groundwater was encountered between 4- to 17-ft in May and June 2023. Shallow perched groundwater could be encountered in the near-surface soils. The volume of groundwater produced can be highly variable depending on the condition of the soil in the immediate vicinity of the excavation. In addition, seasonal surface seeps or springs could develop.

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), stone backfill (ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 207), or clean aggregate (ARDOT 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Subsection 625.02, Type 2 and vented to positive discharge. 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 ARDOT Standard Specifications for Highway Construction, 2014 Edition, Section 805. Pre-boring or pre-excavation for pile installation is not generally anticipated but could be warranted where obstructions, riprap, or debris are encountered.

Any abandoned piling from the prior bridge should be cut off at least 3 ft below final or the grade of pile cap bottoms.

To evaluate required hammer energy for driving equipment, driveability analyses were performed. For these analyses, wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2014[5]. In the driveability analyses, the steel shell piles were assumed to be driven from the plan cap bottom elevation or existing grade. Graphical and tabulated results of these analyses are provided in Appendix J.

Based on the results of the driveability analyses, we recommend a hammer system capable of delivering at least 74 ft-kips per blow for driving the steel shell piles at the end bents and at interior Bent 2. For intermediate Bent 3, we recommend a hammer system capable of delivering at least 91 ft-kips per blow for driving the steel shell piles. 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 installation.

The density of the granular foundation soils increases with depth. As a result, difficult driving could be experienced at depth. Use of a higher energy hammer could be warranted.

Safe bearing capacity of production piles should be determined by Standard Specifications for Highway Construction, 2014 Edition, Section 805.09, Method B. 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 shell 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 shell piles should be limited to about 20 blows per inch. We recommend that practical pile refusal be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work, ground improvements, and all foundation and embankment 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.

---

[5] GRLWEAP 2014; Pile Dynamics, Inc.

The following illustrations are attached and complete this submittal.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 14	Boring Logs
Plate 15	Key to Terms and Symbols
Appendix A	Preliminary Bridge Layout
Appendix B	Generalized Subsurface Profile
Appendix C	Laboratory Test Results
Appendix D	Liquefaction Analysis Results
Appendix E	Nominal Pile Capacity Curves
Appendix F	Lateral Load Parameters
Appendix G	Results of Stability Analyses
Appendix H	Example SP – Woven Geotextile
Appendix I	Example SP – Cohesive Embankment Fill Special Provision
Appendix J	Driveability Analysis 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 during final design or construction, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, LLC

Velleta M. Scott

Velleta M. Scott, P.E.
Sr. Project Engineer

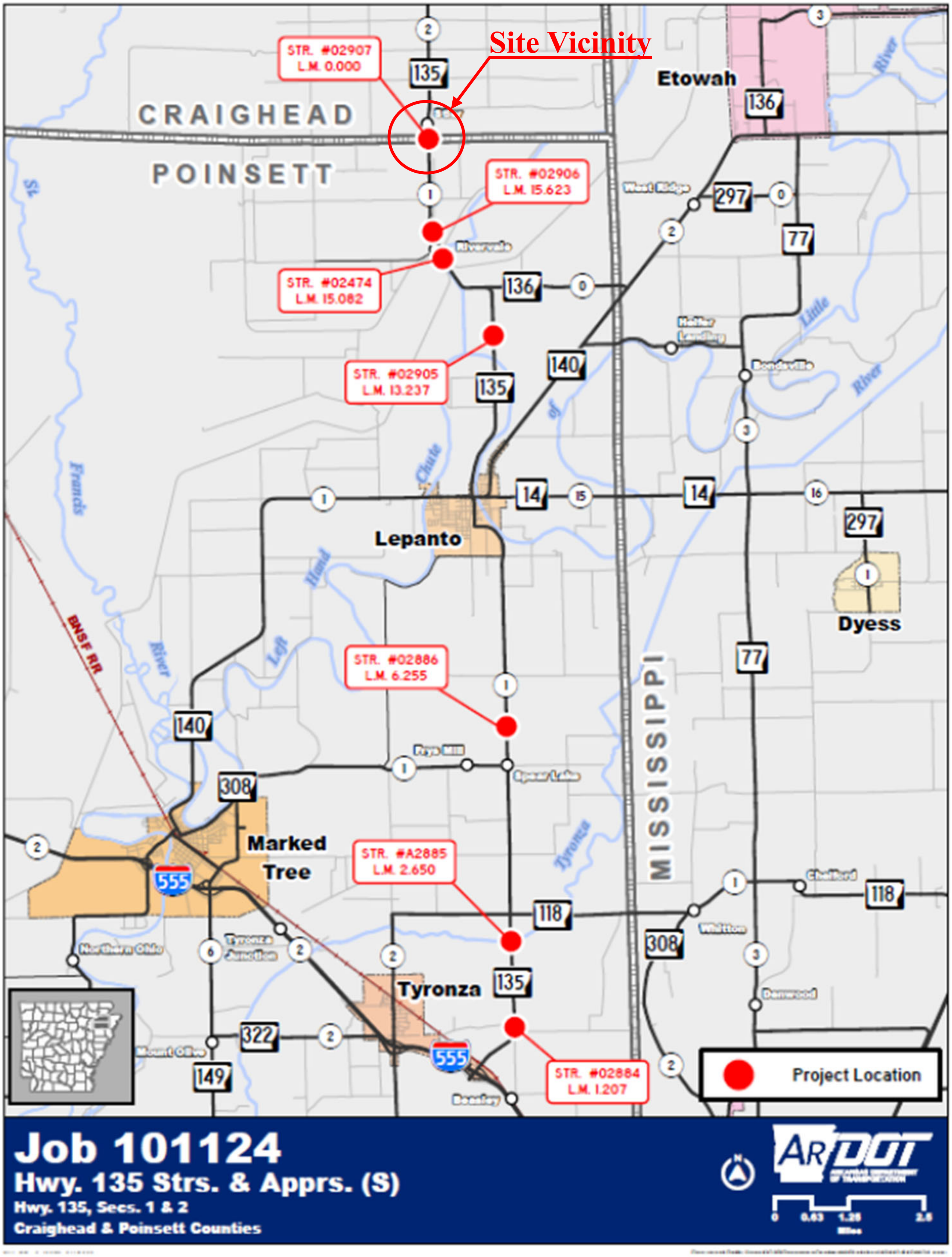
No. 7791
MARK E. WYATT

Mark E. Wyatt, P.E.
President

VMS/MEW:jw

Copies submitted: Arkansas Department of Transportation
Attn: Ms. Jessica Jackson, P.E. (1-email)
Attn: Mr. Paul Tierney (1-email)
Attn: Mr. Yongsheng Zhao, Ph.D., P.E. (1-email)

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

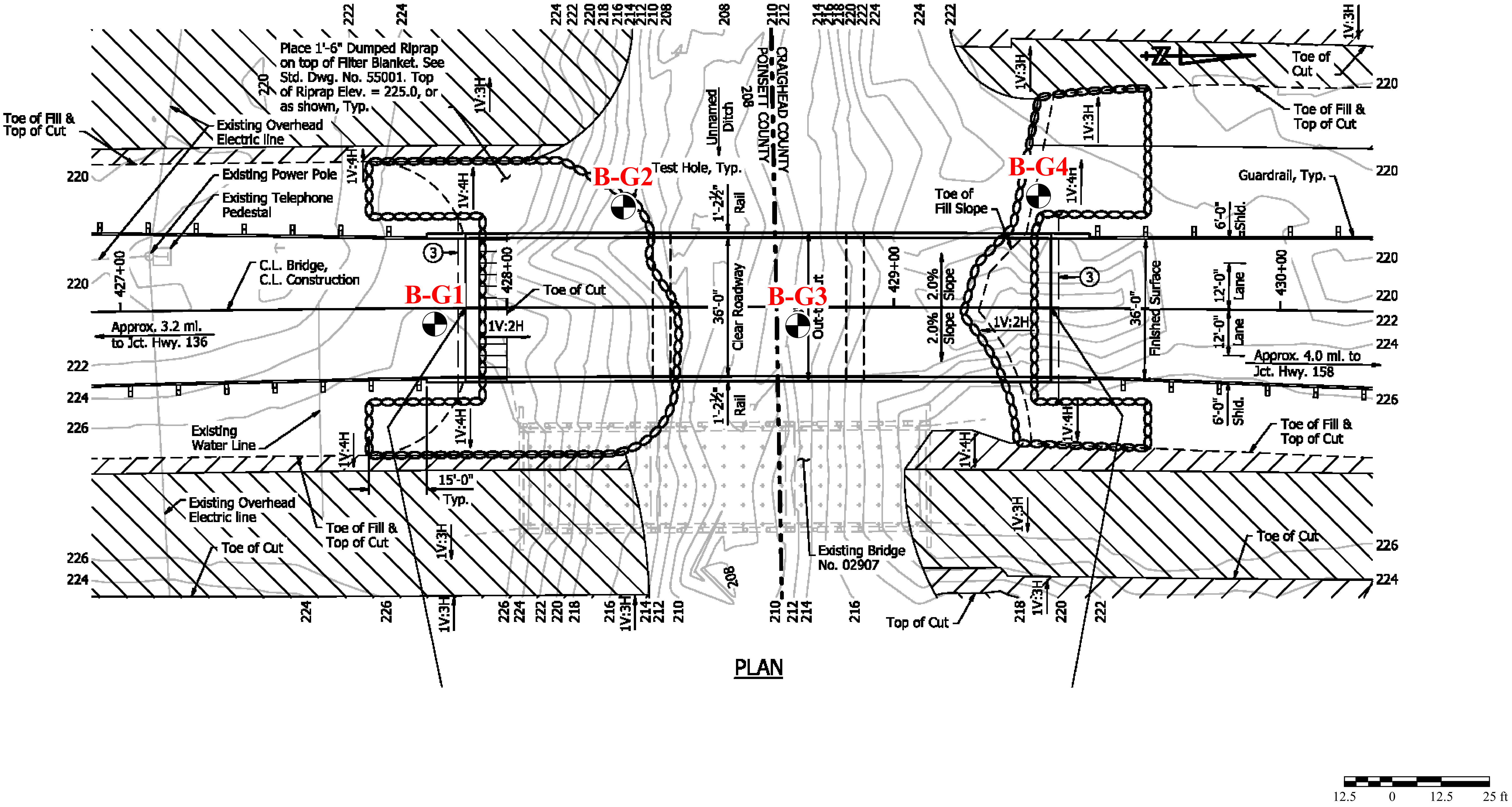


Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

SITE VICINITY MAP
101124 Hwy. 135 over Unnamed Ditch
(Site 7/Bridge G)
Poinsett & Craighead County, Arkansas

Job No. 23-031

Plate 1



PLAN

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A UES Company

PLAN of BORINGS
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

Scale: As shown

Job No. 23-031

PLATE 2

23-031

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L O G O F B O R I N G N O. G1

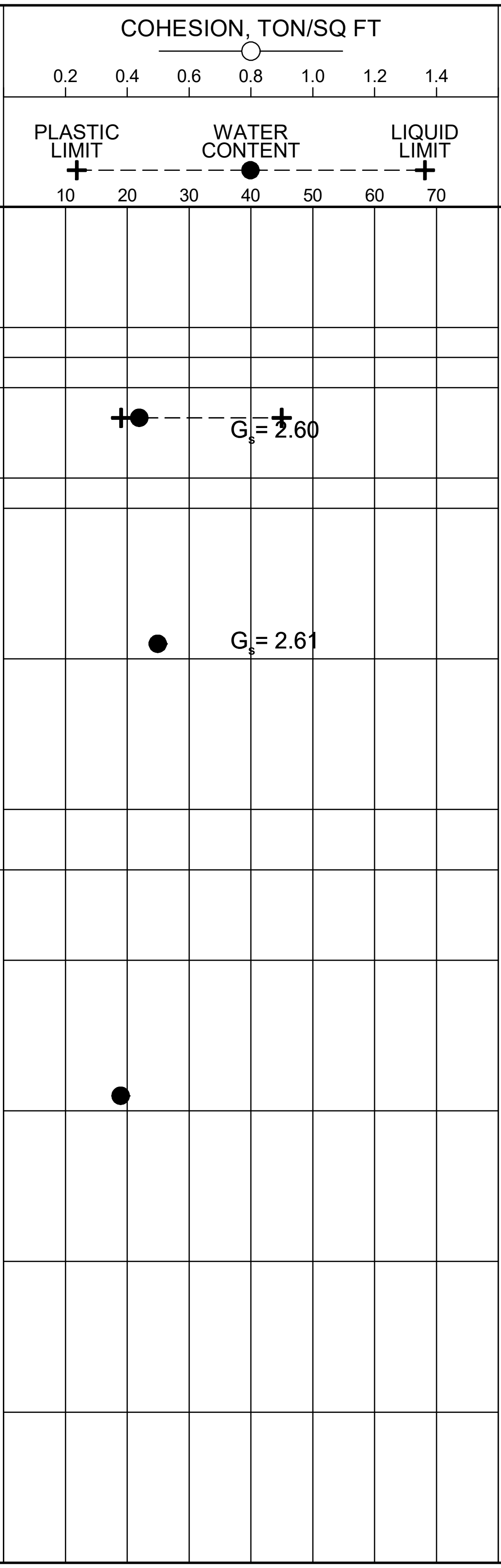
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 427+90, 10 ft Rt

SURF. EL: 225.8

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Medium dense tan and dark brown silty fine sand (SM) w/a little fine to coarse gravel (fill)	21		
		19		
5	Medium dense grayish brown silty fine sand (SM) w/clay pockets	19		
	Soft gray, tan, and brown silty clay, slightly sandy (CL)	9		90
		9		
10	Medium dense gray and brown silty fine sand (SM) w/silt seams and layers, wet	14		
15		20		42
20		26		
	Dense brown fine to medium sand (SP)			
25		54		
	- medium dense from 28 to 43 ft			
30		34		3
35		26		
40		32		
	- dense below 43 ft	66		



COMPLETION DEPTH: 110.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 17.4 ft

DATE: 5/18/2023

LGBNEW 23-031 BRIDGE G.GPJ 7-26-23

23-031

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

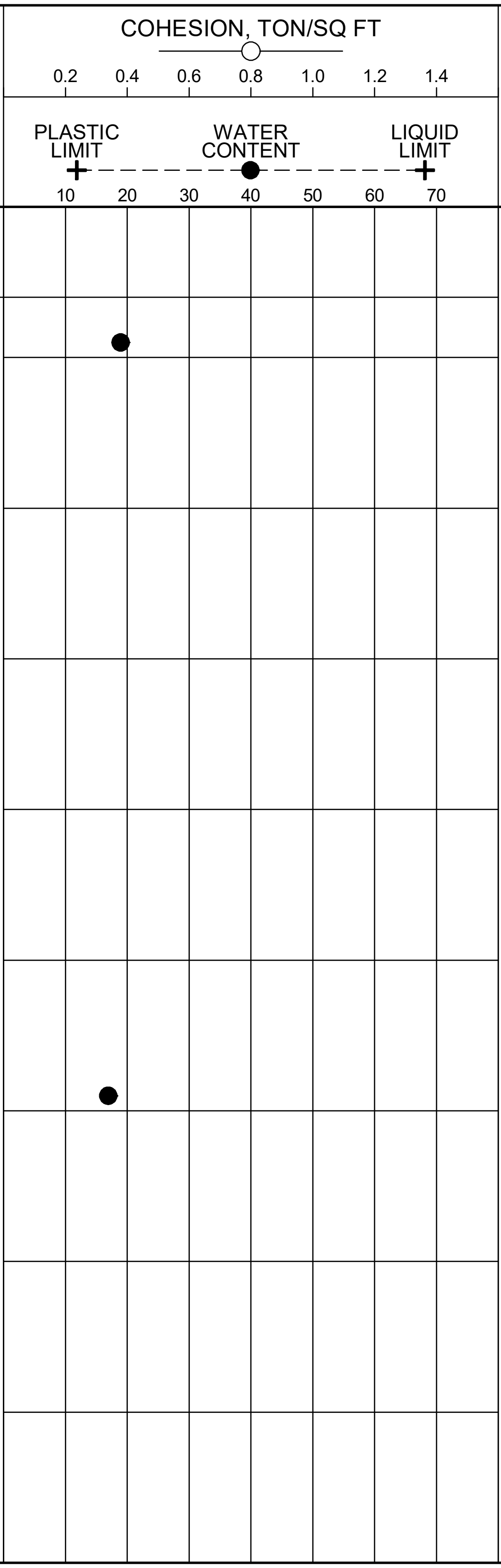
LOG OF BORING NO. G1

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 427+90, 10 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	Dense grayish brown fine to medium sand (SP)	43		4
55		69		
60		50		
65		54		
70		75		
75	- gray and grayish brown, slightly silty (SP-SM) below 73 ft	72		5
80		54		
85		85		
		54		



COMPLETION DEPTH: 110.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 17.4 ft

DATE: 5/18/2023

LGBNEW 23-031 BRIDGE G.GPJ 7-26-23

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

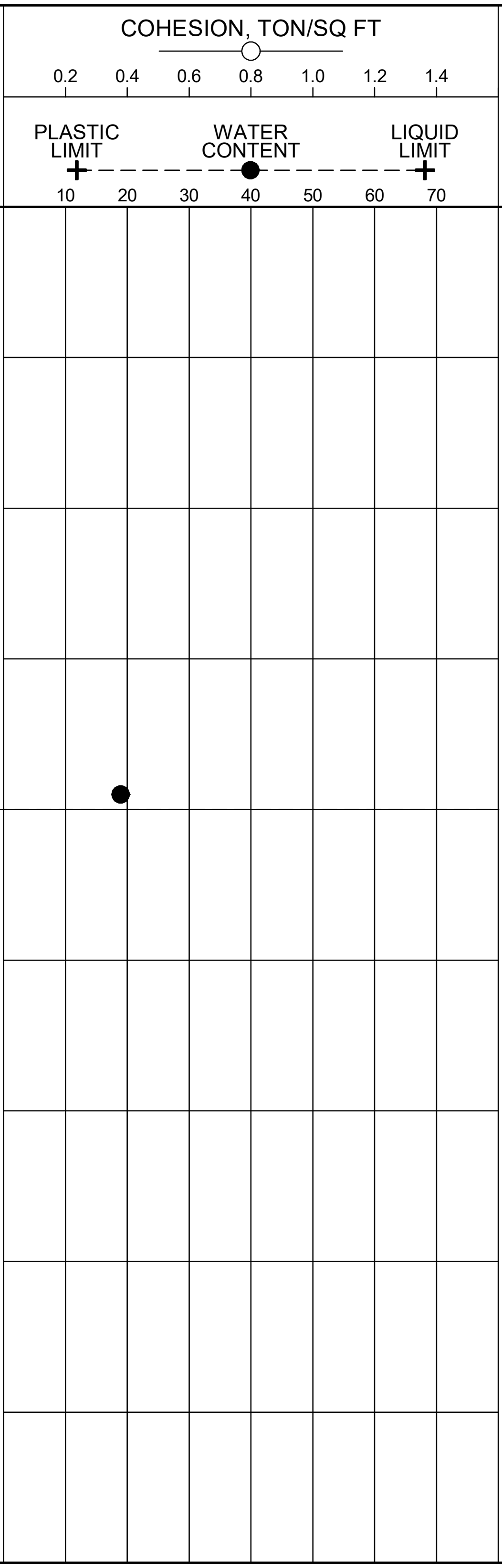
LOG OF BORING NO. G1

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Approx Sta 427+90, 10 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
95	▨	☒		56		
100	▨	☒	- with trace coarse sand and fine gravel below 98 ft	46		
105	▨					
110	▨	☒		123		5
115			Note: Drilled with Diedrich D-50 ECF=1.43			
120						
125						
130						



COMPLETION DEPTH: 110.0 ft
DATE: 5-19-23

DEPTH TO WATER
IN BORING: 17.4 ft

DATE: 5/18/2023

23-031

Grubbs, Hoskyn, Barton & Wyatt, Inc.
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LOG OF BORING NO. G2

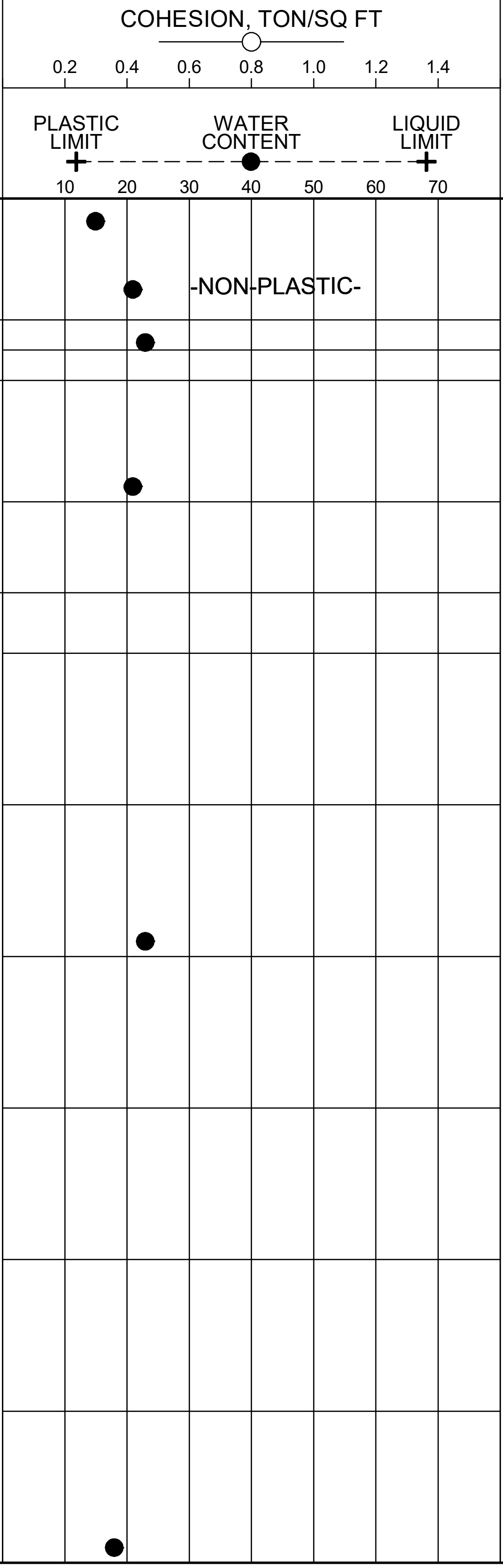
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 428+25, 25 ft Lt

SURF. EL: 214.1

DEPTH, FT	DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
	Very loose to loose brown silty fine sand (SM)	6		
	- very loose below 2 ft	4		46
5	Firm gray and reddish tan silty clay (CL) w/ferrous stains	13		
	Loose tan and gray silty fine sand (SM) w/ferrous stains	13		
	- brown, moist below 8 ft			
10		9		23
15	Loose to medium dense grayish brown fine to medium sand (SP)	14		
	- medium dense from 18 to 33 ft			
20		32		
25		42		3
30		39		
	- dense from 33 to 53 ft			
35		49		
40		49		
	- slightly silty (SP-SM) below 43 ft	57		7



COMPLETION DEPTH: 110.0 ft
DATE: 6-20-23

DEPTH TO WATER
IN BORING: 11.2 ft

DATE: 6/20/2023

LGBNEW 23-031 BRIDGE G.GPJ 7-26-23

23-031

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

L O G O F B O R I N G N O. G2

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 428+25, 25 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
			- with dark gray nodules below 48 ft				
50		☒		53			
			- medium dense from 53 to 63 ft				
55		☒		40			
60		☒		42			
			- dense below 63 ft				
65		☒		54			
70		☒		57			
75		☒		66		●	6
80		☒		66			
85		☒		69			
		☒		59			

COMPLETION DEPTH: 110.0 ft
DATE: 6-20-23

DEPTH TO WATER
IN BORING: 11.2 ft

DATE: 6/20/2023

LGBNEW 23-031 BRIDGE G.GPJ 7-26-23

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

L O G O F B O R I N G N O. G2

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 428+25, 25 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
95		☒	- less silty (SP) with trace coarse sand and fine gravel below 93 ft	47		●	4
100		☒		70			
105							
110		☒		60			
			NOTE: Drilled with Diedrich D-50 ECF= 1.43.				
115							
120							
125							
130							

COMPLETION DEPTH: 110.0 ft
DATE: 6-20-23

DEPTH TO WATER
IN BORING: 11.2 ft

DATE: 6/20/2023

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Consulting Engineers

LOG OF BORING NO. G3

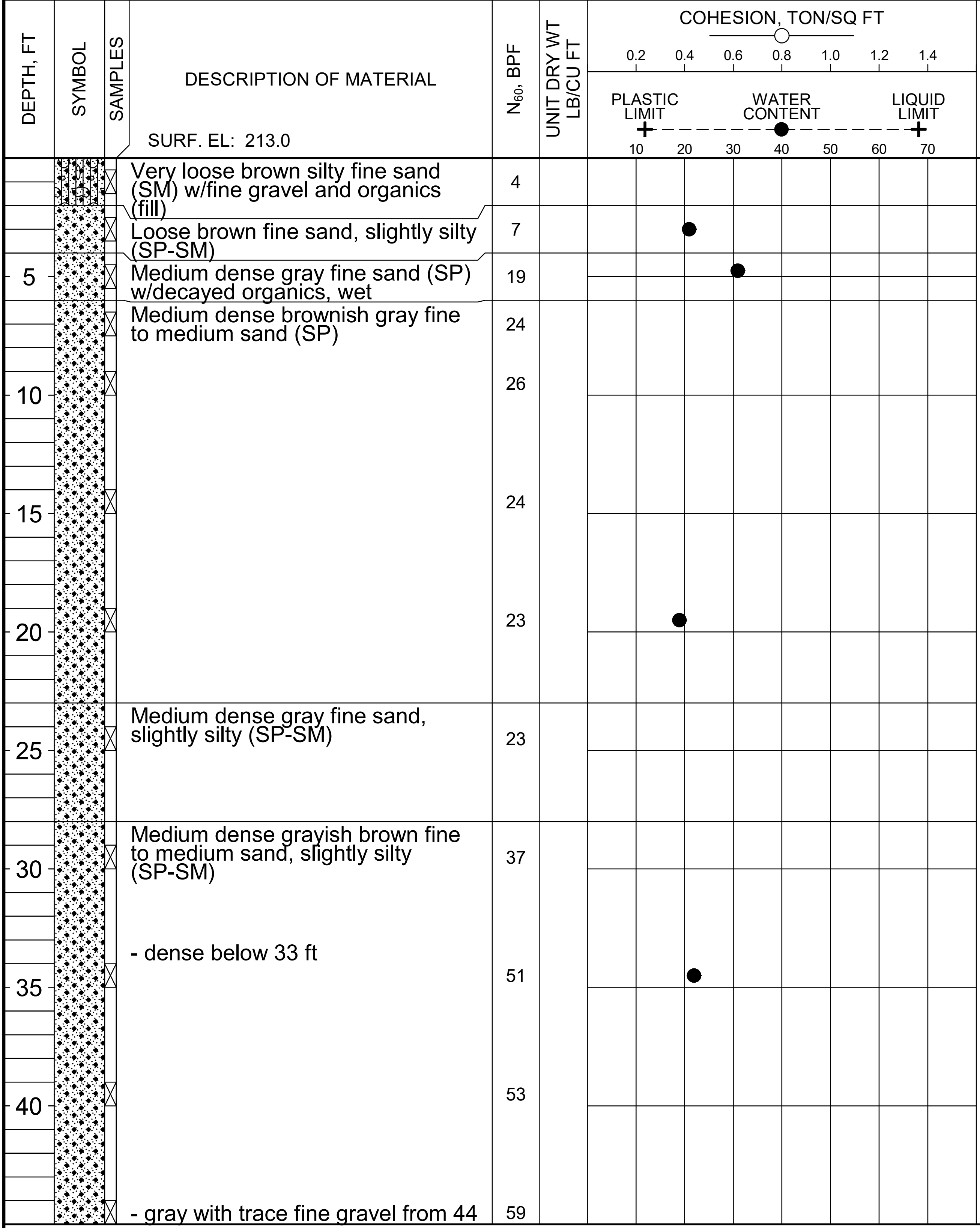
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: HSA to 10 ft /Wash

LOCATION: Approx Sta 428+75, CL

SURF. EL: 213.0

DESCRIPTION OF MATERIAL	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Very loose brown silty fine sand (SM) w/fine gravel and organics (fill)	4		
Loose brown fine sand, slightly silty (SP-SM)	7		7
Medium dense gray fine sand (SP) w/decayed organics, wet	19		3
Medium dense brownish gray fine to medium sand (SP)	24		
	26		
	24		
	23		3
Medium dense gray fine sand, slightly silty (SP-SM)	23		
Medium dense grayish brown fine to medium sand, slightly silty (SP-SM)	37		
- dense below 33 ft	51		5
	53		
- gray with trace fine gravel from 44	59		



COMPLETION DEPTH: 120.0 ft
DATE: 6-15-23

DEPTH TO WATER
IN BORING: 4.3 ft

DATE: 6/14/2023

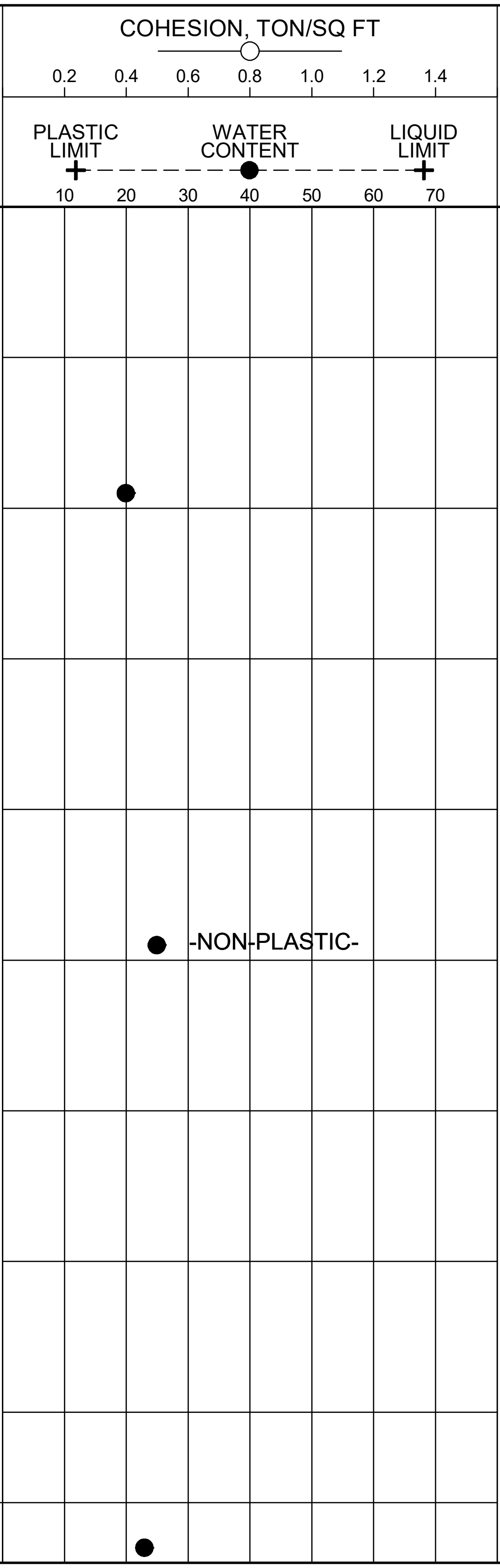
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. G3

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: HSA to 10 ft /Wash
LOCATION: Approx Sta 428+75, CL

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
48		to 49 ft			
50	☒	- with trace coarse sand from 49 to 54 ft	53		
55	☒		51		5
60	☒		60		
65	☒		61		
70	☒		59		9
75	☒	- with some coarse sand below 74 ft	53		
80	☒	- with trace fine gravel at 79 to 84 ft	60		
85	☒		56		
88		Dense brownish gray fine sand, slightly silty (SP-SM) w/decayed			
90	☒		46		11



COMPLETION DEPTH: 120.0 ft
DATE: 6-15-23

DEPTH TO WATER
IN BORING: 4.3 ft
DATE: 6/14/2023

23-031

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

LOG OF BORING NO. G3

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: HSA to 10 ft /Wash

LOCATION: Approx Sta 428+75, CL

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N₆₀, BPF	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + — WATER CONTENT ● — LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
		organics				
95	☒	Dense grayish brown fine to medium sand (SP) w/trace coarse sand and trace fine gravel	50			
100	☒		64			
105						
110	☒	Dense grayish brown fine to coarse sand (SP) w/a little fine gravel	47		●	4
115						
120	☒		61			
125		NOTE: Drilled with Diedrich D-50 ECF= 1.43.				
130						

COMPLETION DEPTH: 120.0 ft
DATE: 6-15-23

DEPTH TO WATER
IN BORING: 4.3 ft

DATE: 6/14/2023

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

LOG OF BORING NO. G4

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

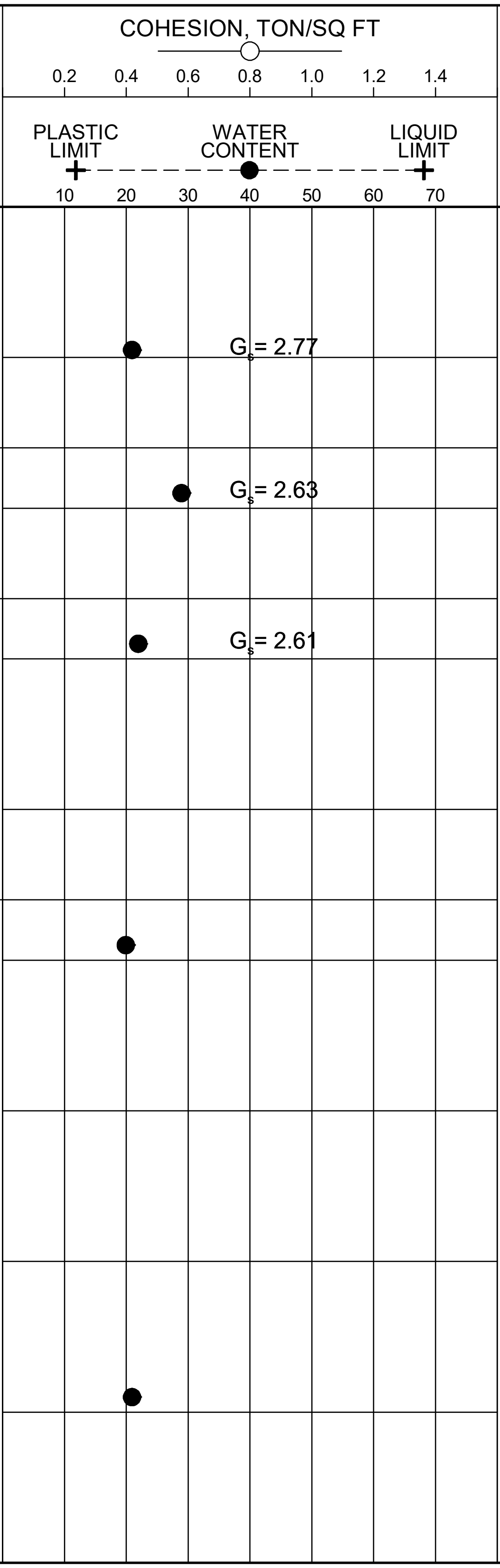
TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 429+40, 25 ft Lt

SURF. EL: 223.2

DESCRIPTION OF MATERIAL	N60, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
Soft light brownish gray and tan silty clay, slightly sandy (CL)	5 5 5 7		82
Loose brown and brownish gray fine sandy silt (ML)	7		77
Medium dense brown fine sand (SP) w/occasional organic inclusions	32 30		3
Medium dense brown fine to medium sand, slightly silty (SP-SM) - dense below 33 ft - grayish brown with occasional organic inclusion below 43 ft	31 30 37 42 70		5 8

Depth scale, ft: 5, 10, 15, 20, 25, 30, 35, 40



COMPLETION DEPTH: 120.0 ft
DATE: 5-18-23

DEPTH TO WATER
IN BORING: 10.8 ft

DATE: 5/16/2023

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

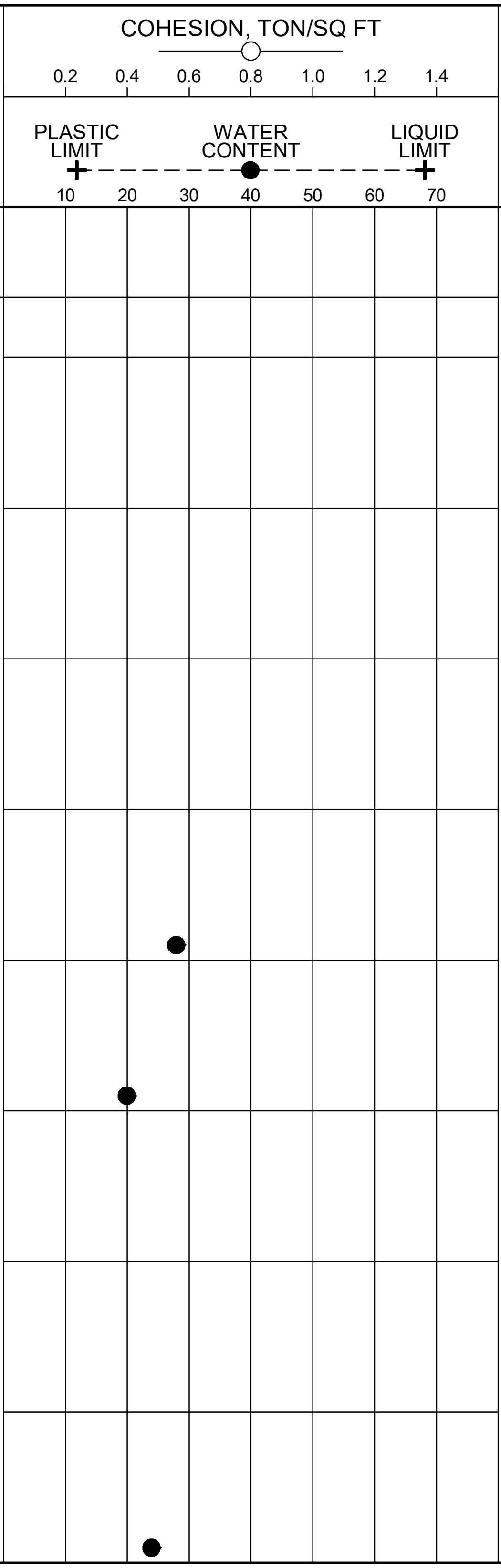
LOG OF BORING NO. G4

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 429+40, 25 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
50	Dense grayish brown and brown fine sand, slightly silty (SP-SM)	43		
55		51		
60	- medium dense from 58 to 63 ft	33		
65	- dense below 63 ft	42		
70		57		9
75		48		6
80		44		
85		42		
	- medium dense below 88 ft	32		6



COMPLETION DEPTH: 120.0 ft
DATE: 5-18-23

DEPTH TO WATER
IN BORING: 10.8 ft

DATE: 5/16/2023

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

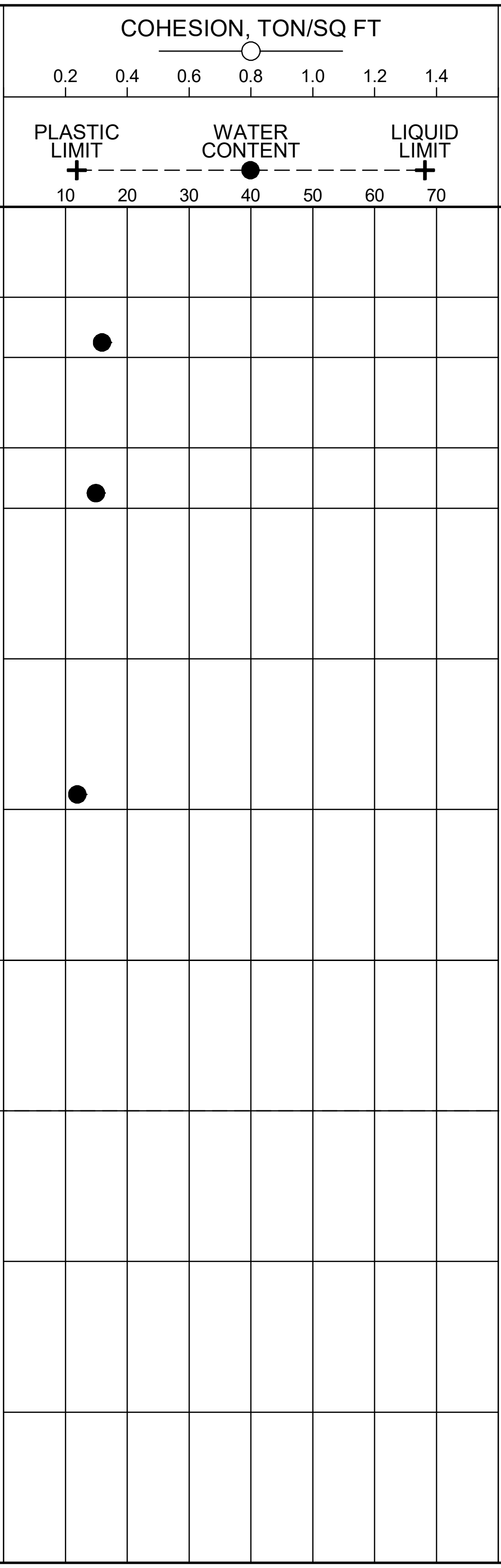
LOG OF BORING NO. G4

101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Approx Sta 429+40, 25 ft Lt

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL (continued)	N_{60}, BPF	UNIT DRY WT LB/CU FT	- No. 200 %
95	☒	Medium dense brown fine to medium sand (SP) w/trace fine gravel	31		4
100	☒	Dense gray and tan fine to coarse sand (SW) w/trace fine gravel	43		4
105		- medium dense with more fine gravel and trace coarse gravel below 105 ft			
110	☒		26		3
115		Medium dense gray fine sand, slightly silty (SP-SM)			
120	☒		35		
125		NOTE: Drilled with SIMCO 2800 ECF= 1.19			
130					



COMPLETION DEPTH: 120.0 ft
DATE: 5-18-23

DEPTH TO WATER
IN BORING: 10.8 ft

DATE: 5/16/2023

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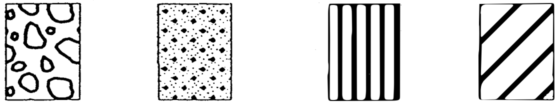
Barton & Wyatt, Inc.
Consulting Engineers

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

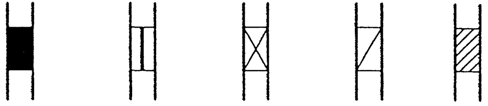
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



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 9-26-02

APPENDIX A

DATE REVISED	DATE REVISED	FED.RD. DIST.NO.	STATE	JOB NO.	SHEET NO.	TOTAL SHEETS
		6	ARK.	101124	112	183
		XXXXX		LAYOUT		XXXXX

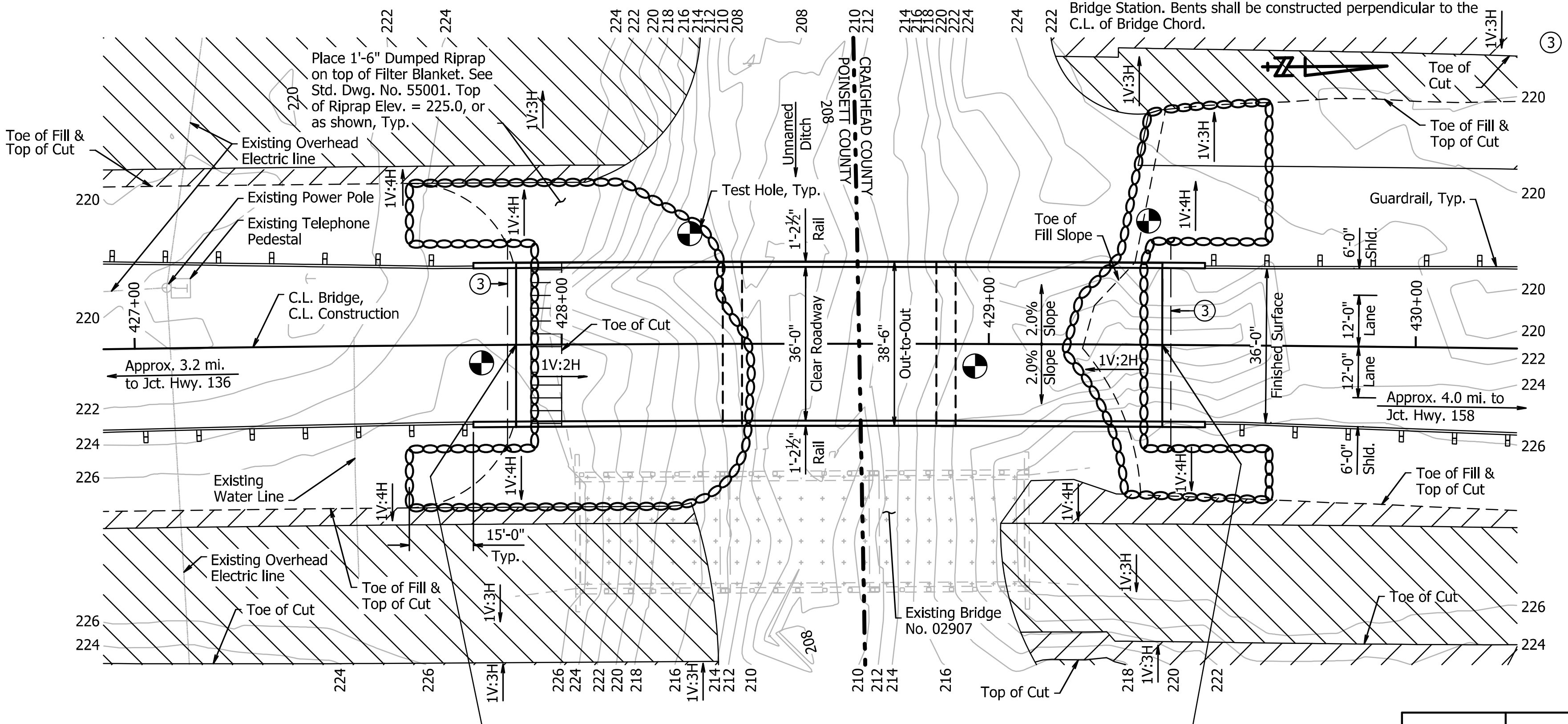
For R/W Data, See Roadway Plans.

NOTE: C.L. Construction is on a 00°30'00" curve right. The longitudinal lines of the deck shall be constructed on curves concentric with C.L. Construction. C.L. Girders are parallel to a Bridge Chord between the Begin Bridge Station and the End Bridge Station. Bents shall be constructed perpendicular to the C.L. of Bridge Chord.

Notes:

Use Type 1 & 2 Special Approach Gutters and Type C1 Approach Slabs at both ends of bridge. See Dwg. Nos. XXXXX, XXXXX & Std. Dwg. No. 55040C1, respectively.

③ 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".



PLAN

HORIZONTAL CURVE DATA

Along C.L. Const.

P.I. = 426+36.02
△ = 7°58'55" RT.
D = 00°30'00"
T = 799.49'
L = 1596.39'
P.C. = 418+36.53
P.T. = 434+32.92

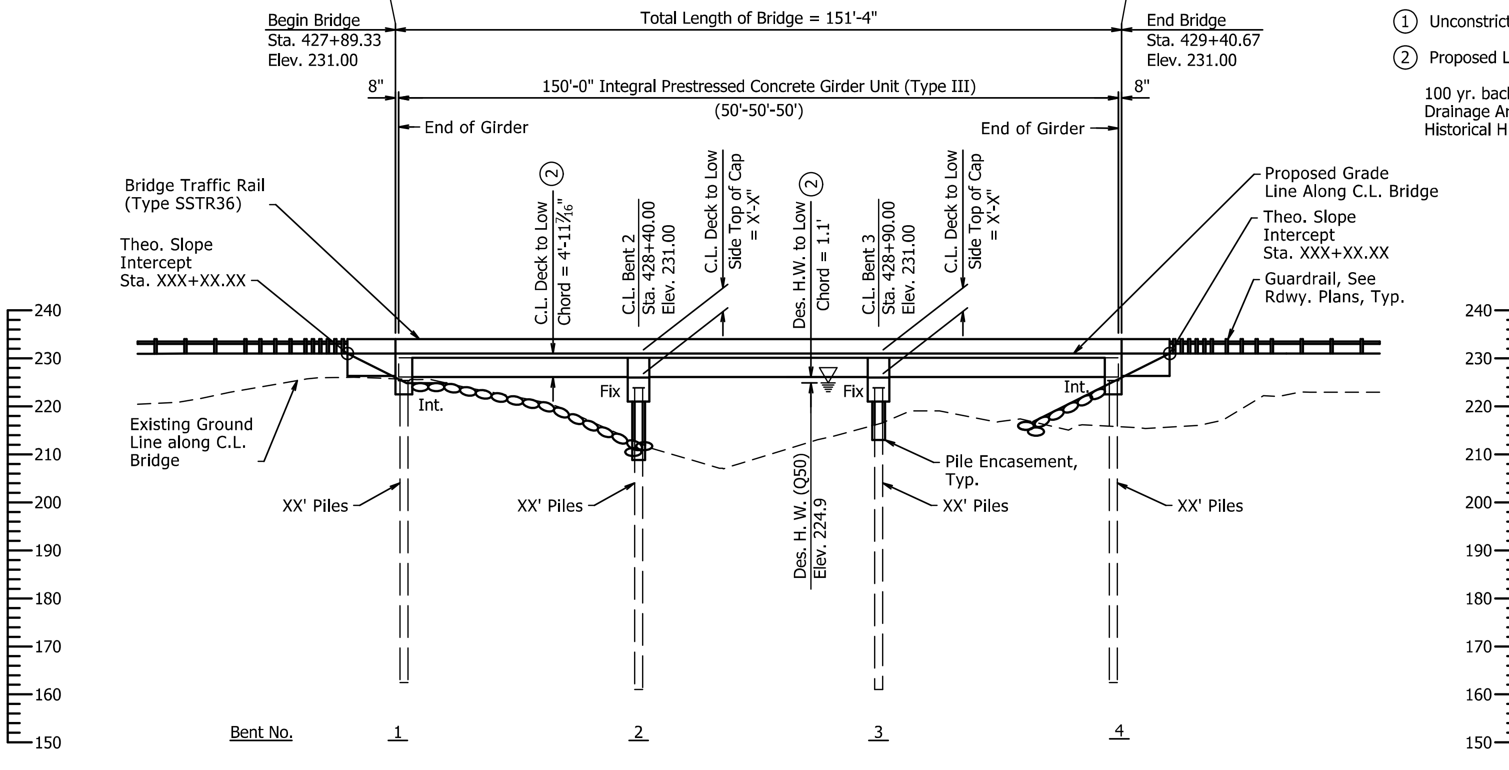
HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	① NATURAL W.S. ELEVATION	W.S. ELEVATION WITH BACKWATER
	YEARS	CFS	FEET	FEET
DESIGN	50	1370	224.9	224.9
BASE	100	1500	225.3	225.3
EXTREME	500	1780	226.2	226.2
OVERTOPPING	5	910	223.1	223.1

① Unconstricted water surface elevation without structure or roadway approaches.

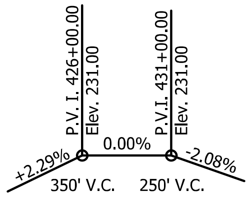
② Proposed Low Bridge Chord Elev. = 226.05 feet at Station 427+92.33

100 yr. backwater elevation for existing structure = 225.4 feet
Drainage Area = 21.8 sq. miles
Historical H.W. Elev. = N/A



ELEVATION

NOTE: For Soil Borings, see Dwg. No. XXXXX.



VERTICAL ALIGNMENT DATA

Theoretical Elev. Along C.L. Construction

NOTE: Stations shown are along C.L. Construction. Elevations shown are theoretical working point elevations at C.L. Bridge. Any vertical dimension referenced to C.L. Deck is based on theoretical working point elevation at C.L. Bridge. See "Rounding Detail" on Std. Dwg. No. 55007 for additional information.

GENERAL NOTES

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

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

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 9th Edition (2020).

LIVE LOADING: HL-93

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

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (prestressed concrete girders)	f'c = 6,000 psi
Prestressing Strands (AASHTO M 203, Gr. 270)	fpu = 270,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M 31 or M 322, Type A)	fy = 60,000 psi
Structural Steel (ASTM A709, Gr. 50)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 50W)	Fy = 50,000 psi
Structural Steel (ASTM A709, Gr. 36)	Fy = 36,000 psi

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

STEEL SHELL PILING: Piling in Bents 1 and 4 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. Piling in Bents 2 & 3 shall be XX" diameter concrete filled steel shell piles and shall be driven to a minimum ultimate bearing capacity of XX tons per pile. All piling shall be driven with an approved air, steam, or diesel hammer to a minimum tip elevation of _____ or lower at Bents 1 and 4 and to a minimum tip elevation of ______ or lower at Bents 2 and 3. Piling in end bents shall be driven after embankment to bottom of cap is in place. Lengths of piling shown are assumed for estimating quantities only. Actual lengths are to be determined in the field. No additional payment will be made for cut-off or build-up. Test piles are not required but may be driven for the Contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods as approved by the Engineer may be required to achieve minimum penetration. This work shall not be paid for directly, but shall be considered incidental to the item "Steel Shell Piling (__" Dia.)".

PREBORING: Preboring is required for all piling at Bents 1 and 4. Prebored holes shall have a diameter 6" greater than the diameter of the pile for a depth of 10' below the bottom of the cap. The void space around the pile after completion of driving shall be backfilled with sand or pea gravel. The Contractor shall be responsible for keeping prebored holes free of debris prior to backfilling which may require the use of temporary casings or other approved methods. Any related cost for backfilling and temporary casing will not be paid for directly, but shall be considered subsidiary to the item "Preboring".

DRIVING SYSTEM: The driving system approval and the ultimate bearing capacity determination for piling shall be based on the requirements of Subsection 805.09(b), "Method B - Wave Equation Analysis (WEAP)". It is estimated that the minimum rated hammer energy required to obtain the ultimate bearing capacity for all piles will be _____foot pounds per blow.

PILE ENCASEMENT: Pile encasement for Bents 2 & 3 shall extend from bottom of cap to 3' below natural or finished ground. See Standard Drawing Number 55021 for additional information.

BRIDGE DECK: The concrete bridge deck shall be given a tine finish as specified for final finishing in Subsection 802.19 for Class 5 Tined Bridge Roadway Surface Finish.

PROTECTIVE SURFACE TREATMENT: Class 2 Protective Surface Treatment shall be applied to the roadway surface and to the roadway face and top of the Bridge Traffic Rail in accordance with Section 803.

DETAIL DRAWINGS:	DRAWING NO(S).
End Bents	______
Intermediate Bents	______
Elastomeric Bearings	______
150'-0" Integral Prestressed Concrete Girder Unit	______
Concrete Filled Steel Shell Piling	55021
Type 1 & 2 Special Approach Gutters	XXXXX & XXXXX
Type C1 Approach Slabs	55040C1
Bridge Traffic Rail	55070

For Additional General Notes, see Dwg. No. XXXXX.

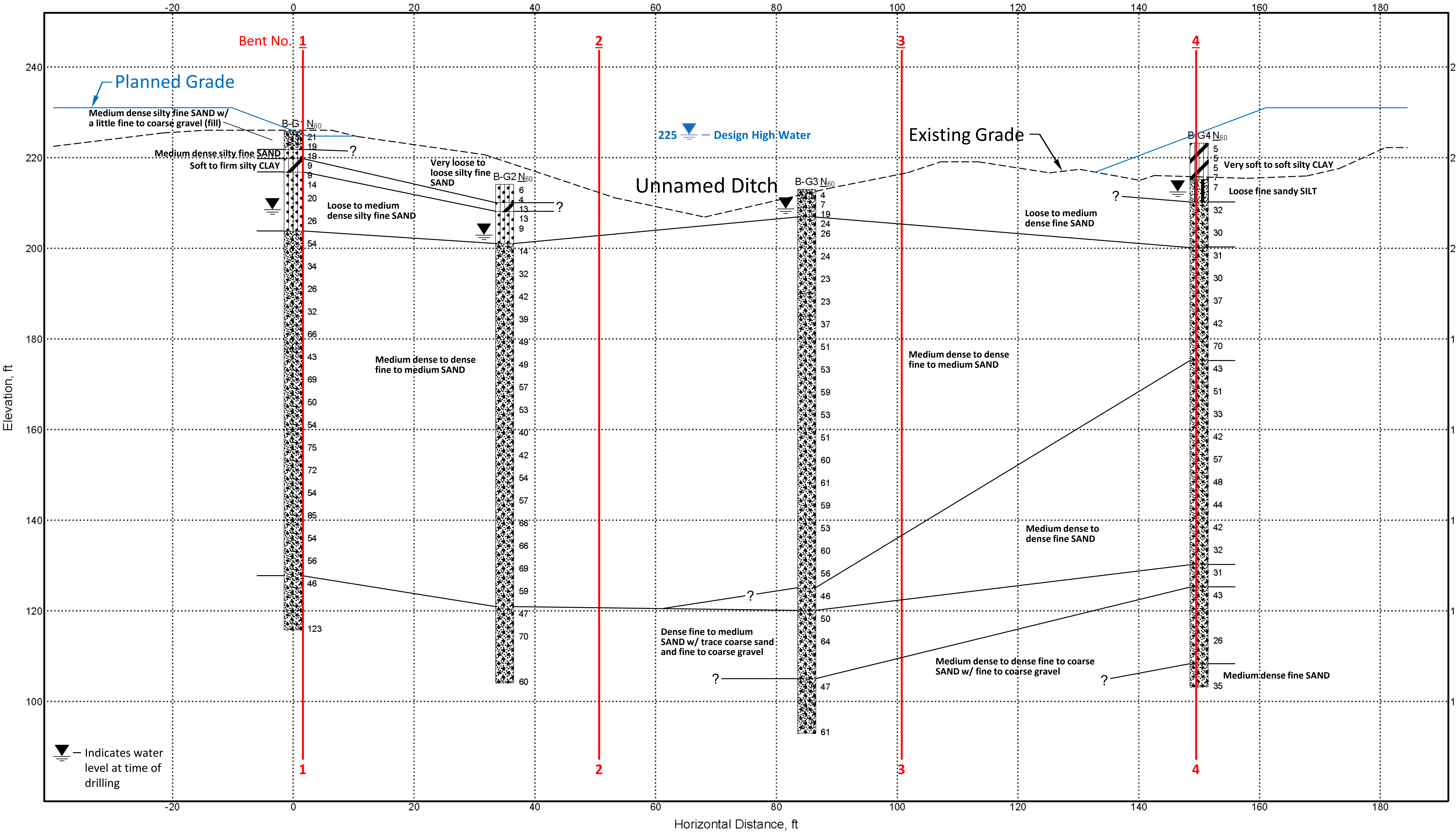
PRELIMINARY
SUBJECT TO
REVISION

SHEET 1 OF 2
LAYOUT OF BRIDGE
HWY. 135 OVER UNNAMED CREEK
HWY. 135 STRS. & APPRS. (S)
CRAIGHEAD & POINSETT COUNTIES
ROUTE 135 SECTIONS 1 & 2
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: BWC DATE: 10-07-22 FILENAME: b101124x7_l1.dgn
CHECKED BY: CAW DATE: 10-27-22 SCALE: 1" = 20'
DESIGNED BY: KRM DATE: 9-30-22
BRIDGE NO. XXXXX DRAWING NO. XXXXX

USER: CTAUSER
DESIGN FILE: G:\22110001_101124\TRANSP\dgn\bridge\b101124x7_l1.dgn
PLOTTED: 1/26/2023 3:35:50 PM SCALE: 40.0000 ' / In.

APPENDIX B



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Barton & Wyatt, LLC

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

SCALE:
1" = 15' Horizontal
1" = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

Project Number: 23-031

APPENDIX C

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: 101124 Hwy. 135 over Unnamed Ditch (Site 7)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
G1	6.5-7.5	22	41	19	22	100	100	100	100	99	98	97	90	CL	A-7-6
G1	14-15	25	---	---	---	100	100	100	100	100	100	88	42	SM	A-4
G1	29-30	19	---	---	---	100	100	100	100	99	99	58	3	SP	A-3
G1	49-50	19	---	---	---	100	100	100	100	100	99	58	4	SP	A-3
G1	74-75	17	---	---	---	100	100	100	99	97	92	31	5	SM-SW	A-1-b
G1	109-110	19	---	---	---	100	100	100	97	93	86	33	5	SM-SW	A-1-b
G2	2.5-3.5	21	NON-PLASTIC			---	---	---	---	99	---	---	46	SM	A-4
G2	9-10	21	---	---	---	---	---	---	---	100	---	---	23	SM	A-2-4
G2	24-25	23	---	---	---	100	100	100	100	100	100	69	3	SP	A-3
G2	44-45	18	---	---	---	100	100	100	100	100	99	51	7	SM-SP	A-3
G2	74-75	20	---	---	---	100	100	100	97	94	91	40	6	SM-SP	A-1-b
G2	94-95	17	---	---	---	100	100	100	98	96	94	45	4	SP	A-1-b
G3	2.5-3.5	21	---	---	---	---	---	---	---	100	---	---	7	SM-SP	A-3
G3	4.5-5.5	31	---	---	---	---	---	---	---	100	---	---	3	SP	A-3
G3	19-20	19	---	---	---	100	100	100	100	100	100	84	3	SP	A-3
G3	34-35	22	---	---	---	100	100	100	100	100	100	83	5	SM-SP	A-3
G3	54-55	20	---	---	---	100	100	100	100	100	100	70	5	SM-SP	A-3
G3	69-70	25	NON-PLASTIC			100	100	100	100	100	100	79	9	SM-SP	A-3
G3	89-90	23	---	---	---	---	---	---	---	100	---	---	11	SM-SP	A-2-4
G3	109-110	13	---	---	---	100	100	100	89	81	72	39	4	SP	A-1-b
G4	4.5-5.5	21	---	---	---	100	100	100	100	100	100	98	82	CL	A-6

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS

PLATE

SUMMARY of CLASSIFICATION TEST RESULTS

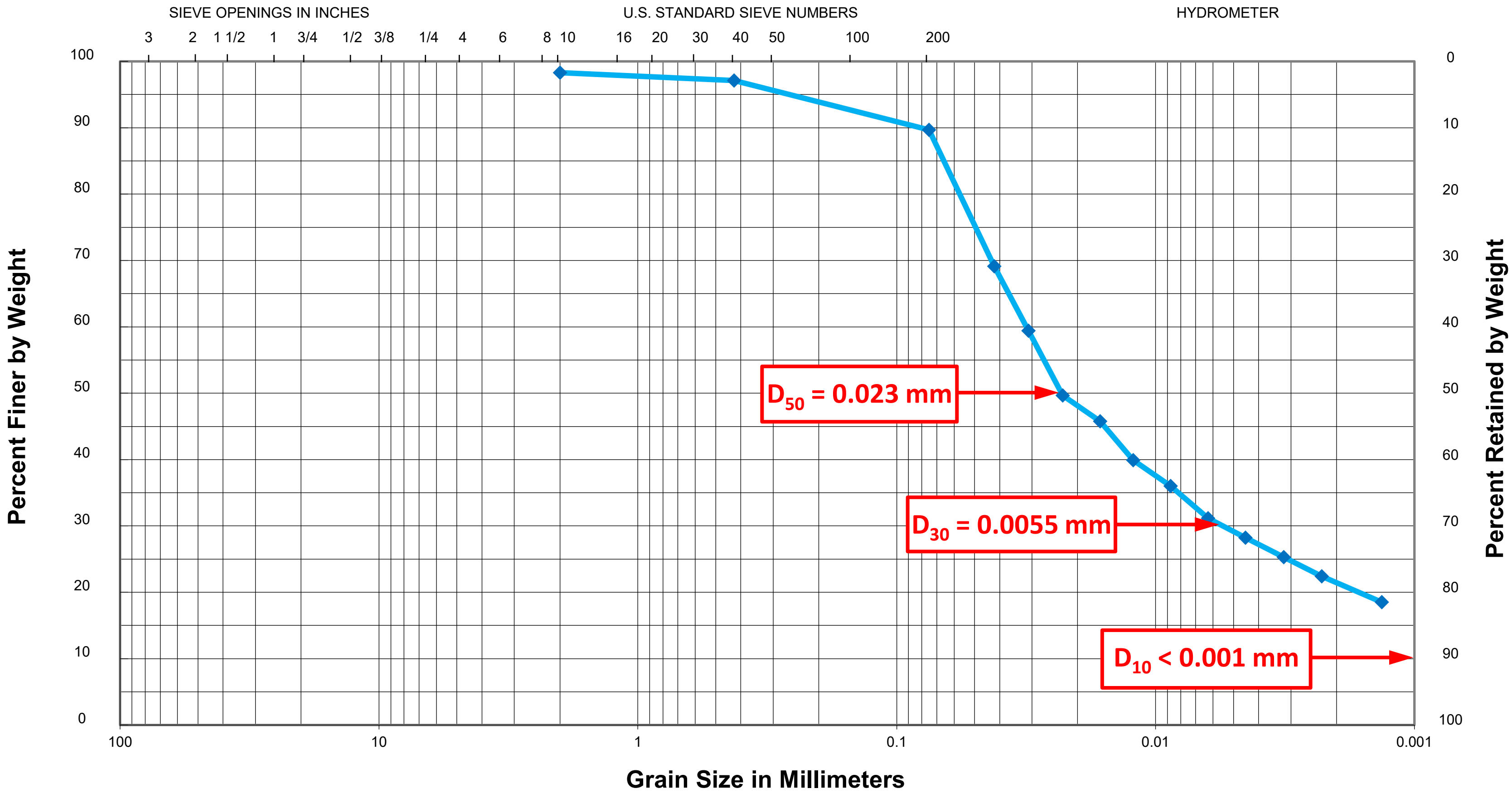
PROJECT: 101124 Hwy. 135 over Unnamed Ditch (Site 7)
LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

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		
G4	9-10	29	---	---	---	100	100	100	100	100	100	99	77	ML	A-4
G4	14-15	22	---	---	---	100	100	100	100	100	100	84	3	SP	A-3
G4	24-25	20	---	---	---	100	100	100	100	100	100	73	5	SM-SP	A-3
G4	39-40	21	---	---	---	100	100	100	100	99	99	75	8	SM-SP	A-3
G4	69-70	25	---	---	---	100	100	100	100	100	100	96	9	SM-SP	A-3
G4	74-75	20	---	---	---	100	100	100	100	100	100	82	6	SM-SP	A-3
G4	89-90	24	---	---	---	100	100	100	100	100	100	96	6	SM-SP	A-3
G4	94-95	16	---	---	---	100	100	100	93	90	88	36	4	SP	A-1-b
G4	99-100	15	---	---	---	100	100	100	97	89	79	32	4	SW	A-1-b
G4	109-110	12	---	---	---	100	100	95	88	63	41	14	3	SW	A-1-a

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



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<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
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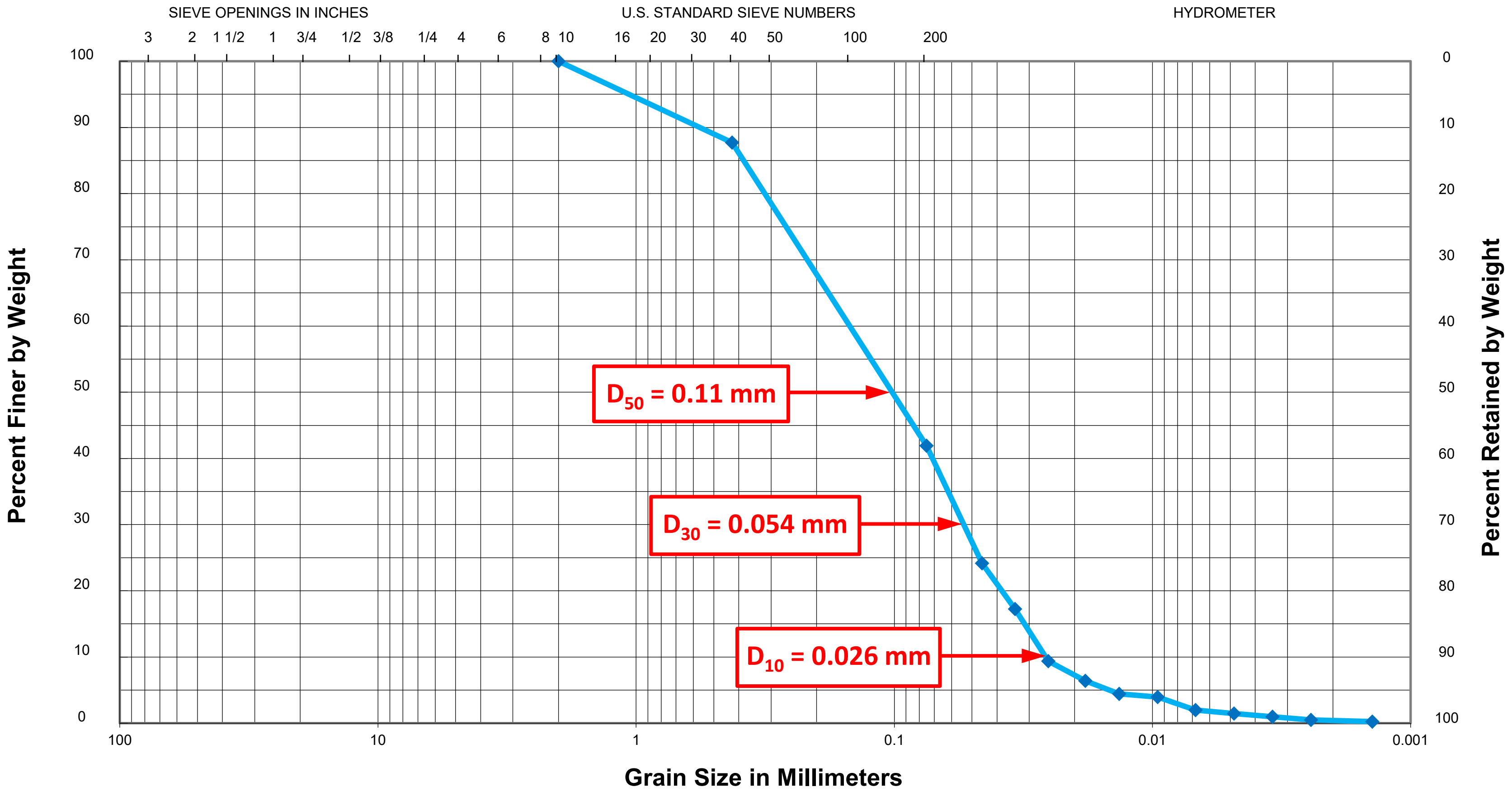
Sample: Boring G1, 6.5-7.5 ft
Description: Gray, tan, and brown silty CLAY, slightly sandy

USCS Classification = CL
AASHTO Classification = A-7-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring G1, 14-15 ft
Description: Gray and brown silty fine SAND w/ silt seams and layers

USCS Classification = SM
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

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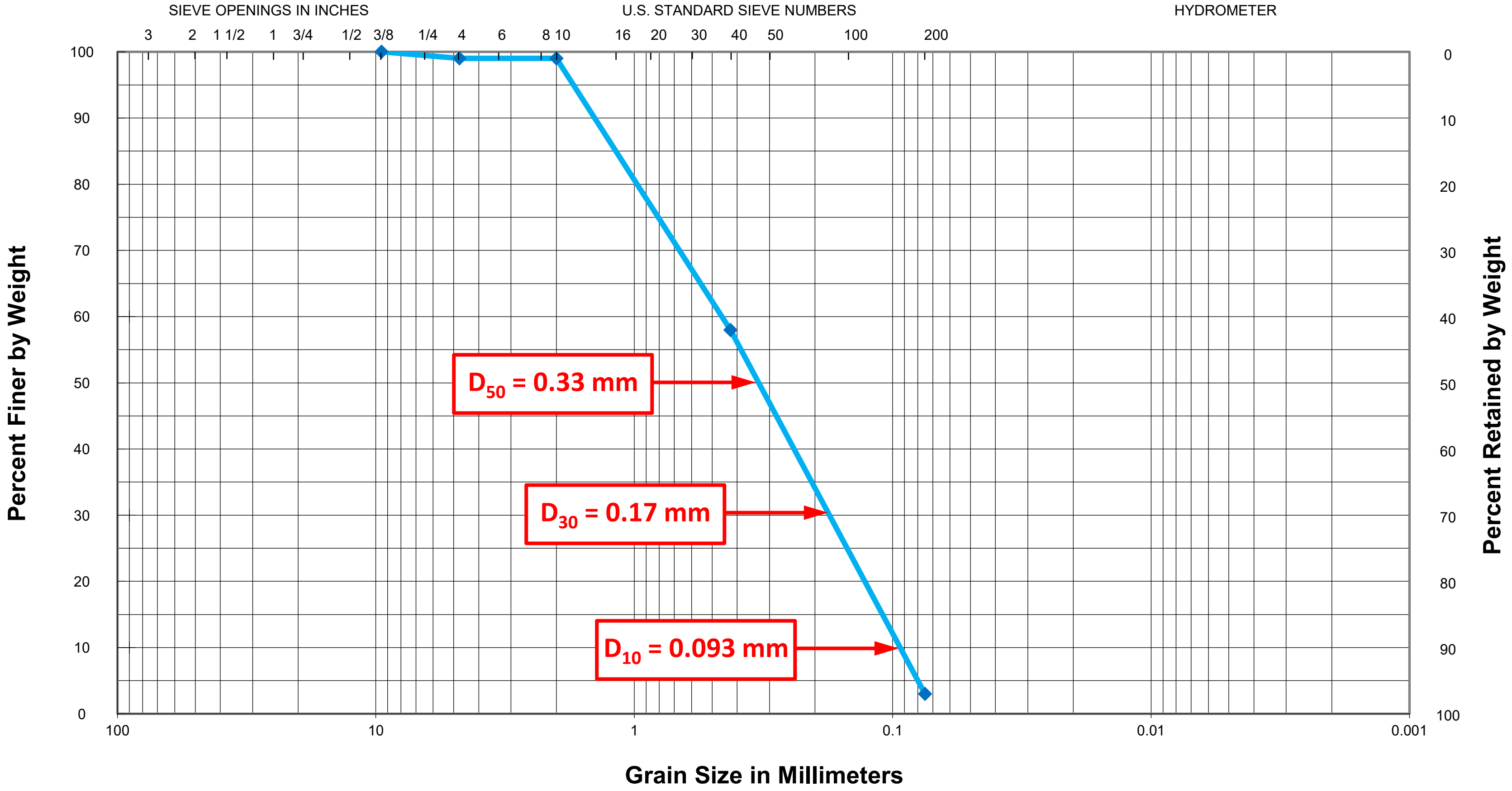

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G1, 29-30 ft
Description: Brown fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

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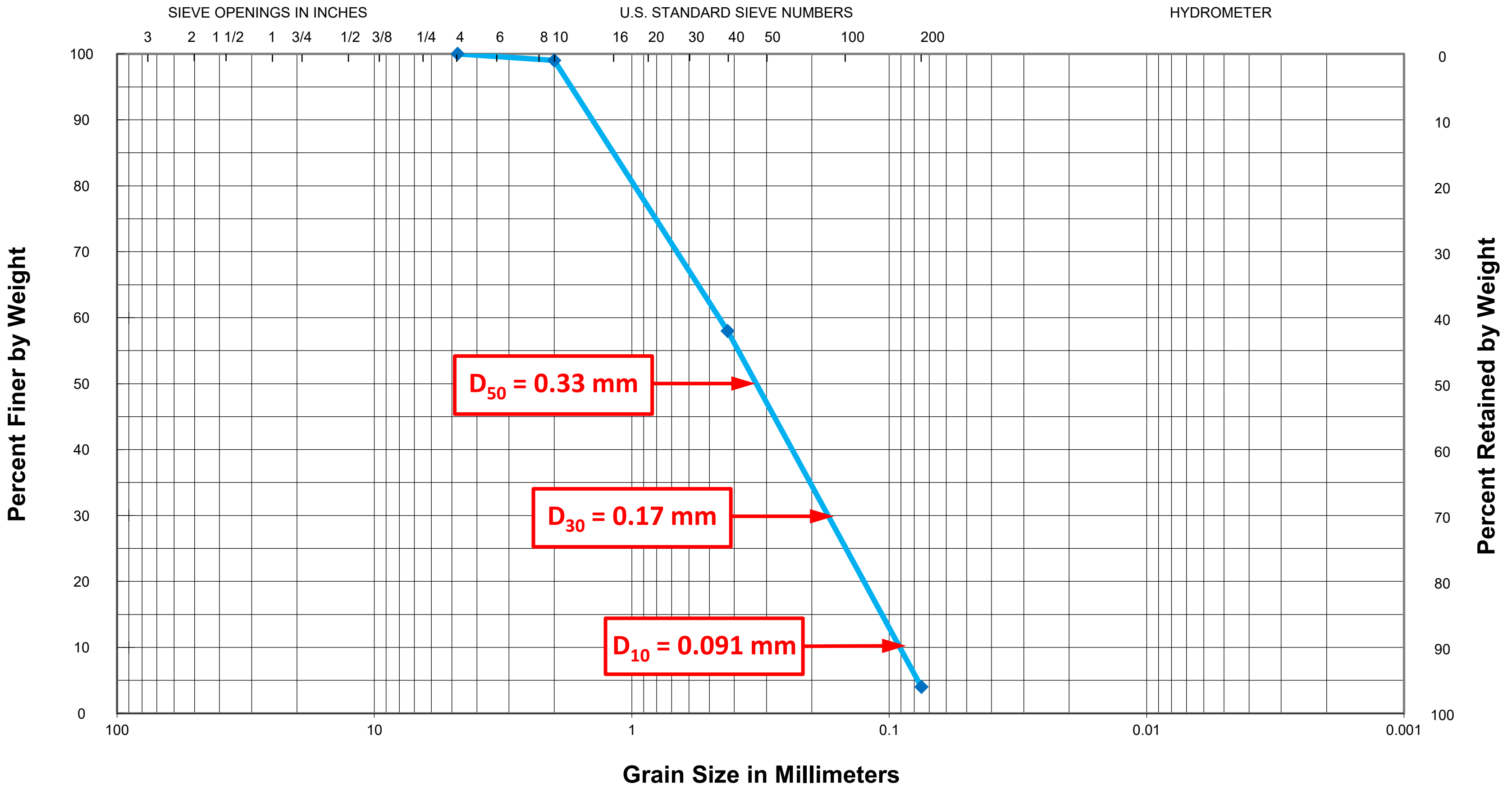
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G1, 49-50 ft
Description: Grayish brown fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

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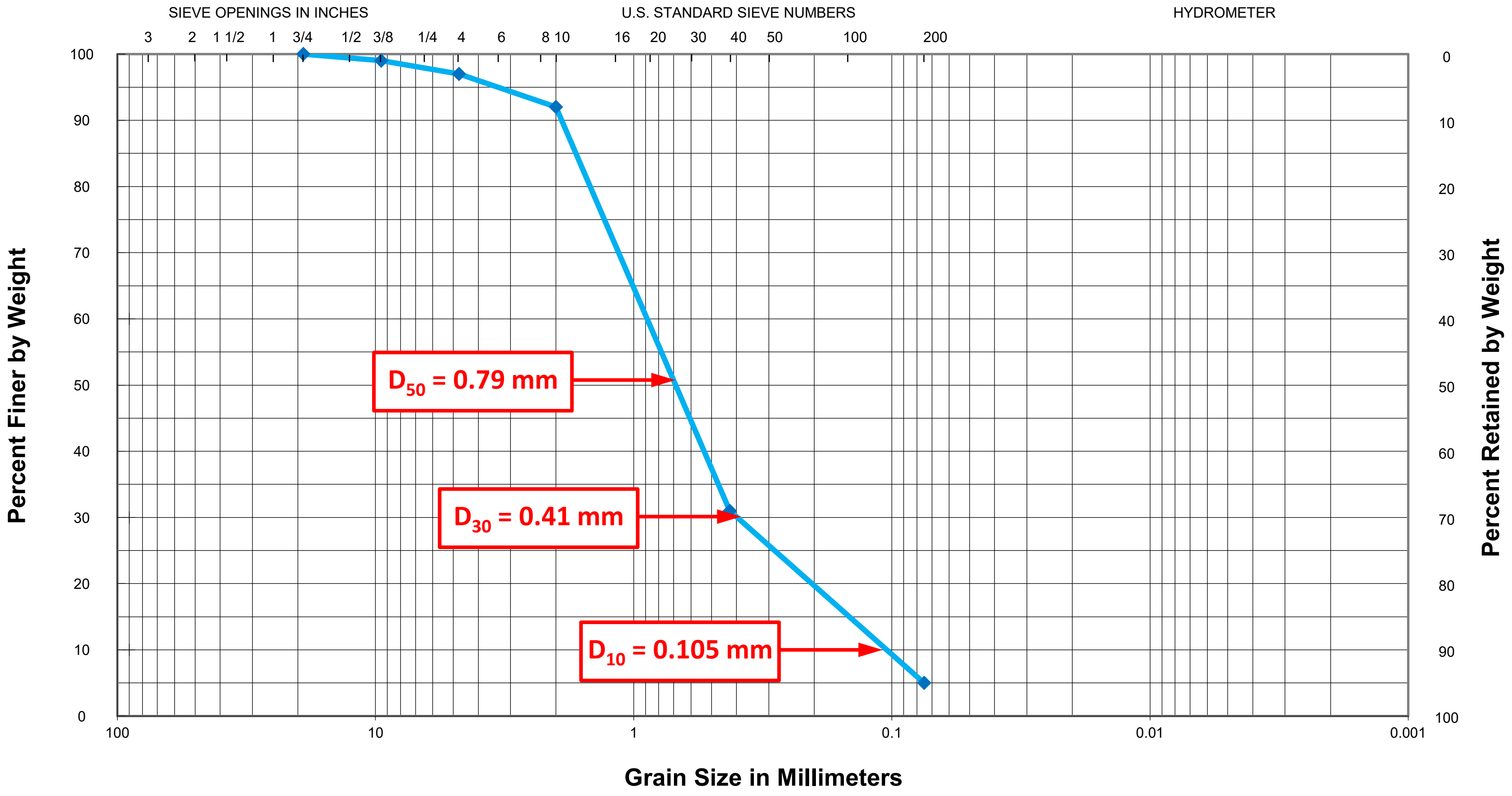

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G1, 74-75 ft
Description: Gray and grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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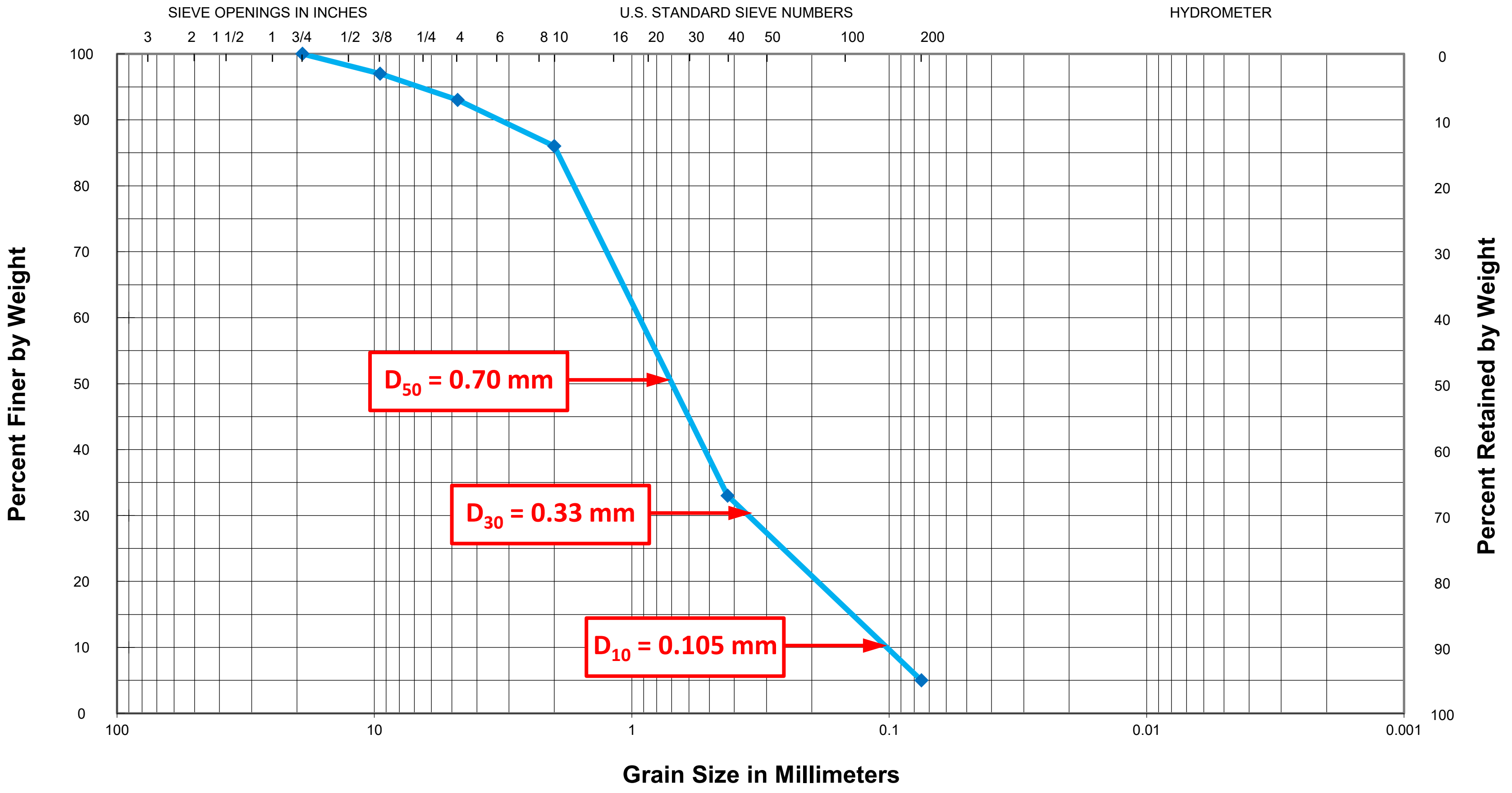


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

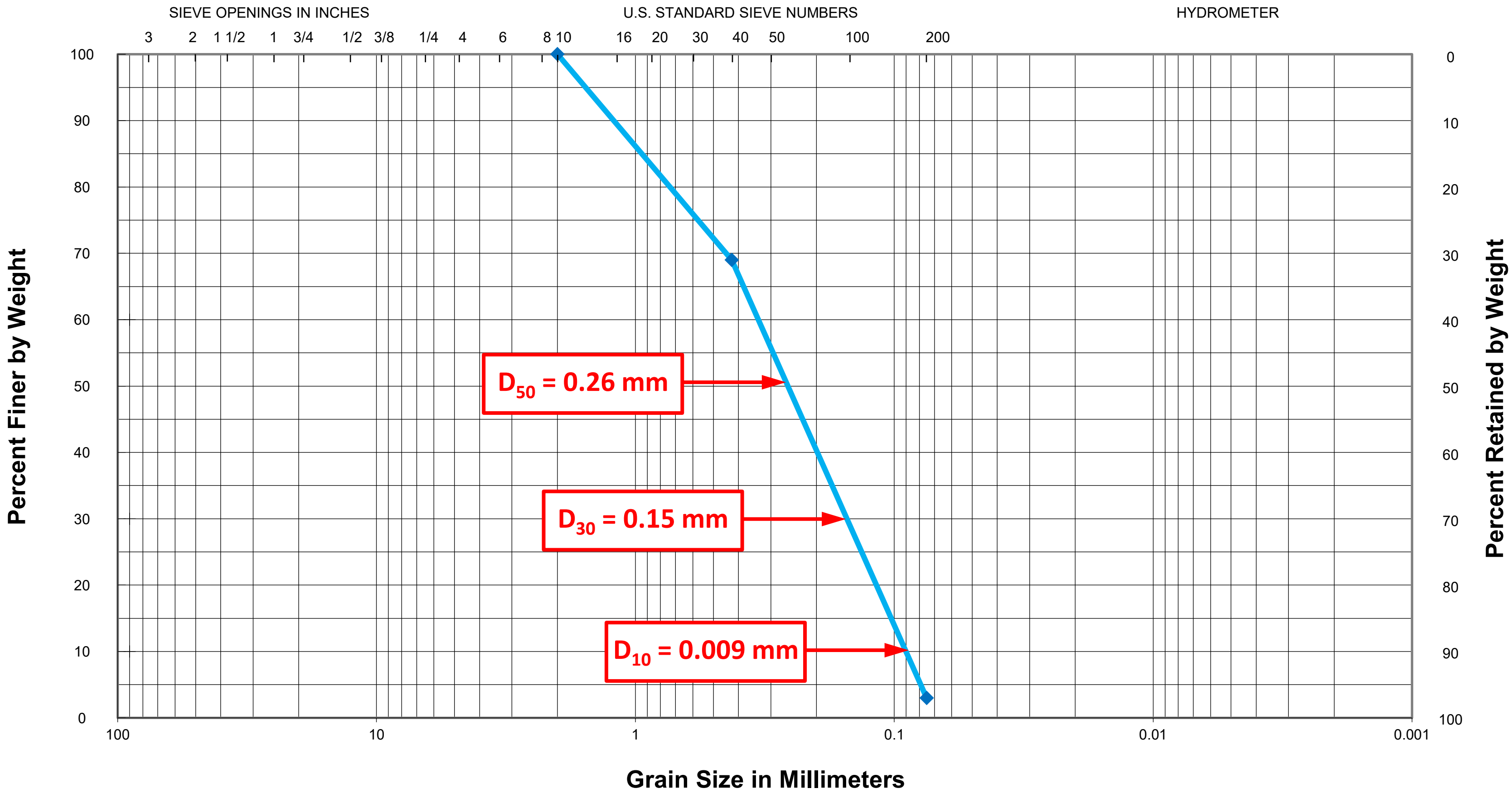
Sample: Boring G1, 109-110 ft
Description: Grayish brown fine to medium SAND, slightly silty w/ trace coarse sand and fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.26 mm

D_{30} = 0.15 mm

D_{10} = 0.009 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G2, 24-25 ft
Description: Grayish brown fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

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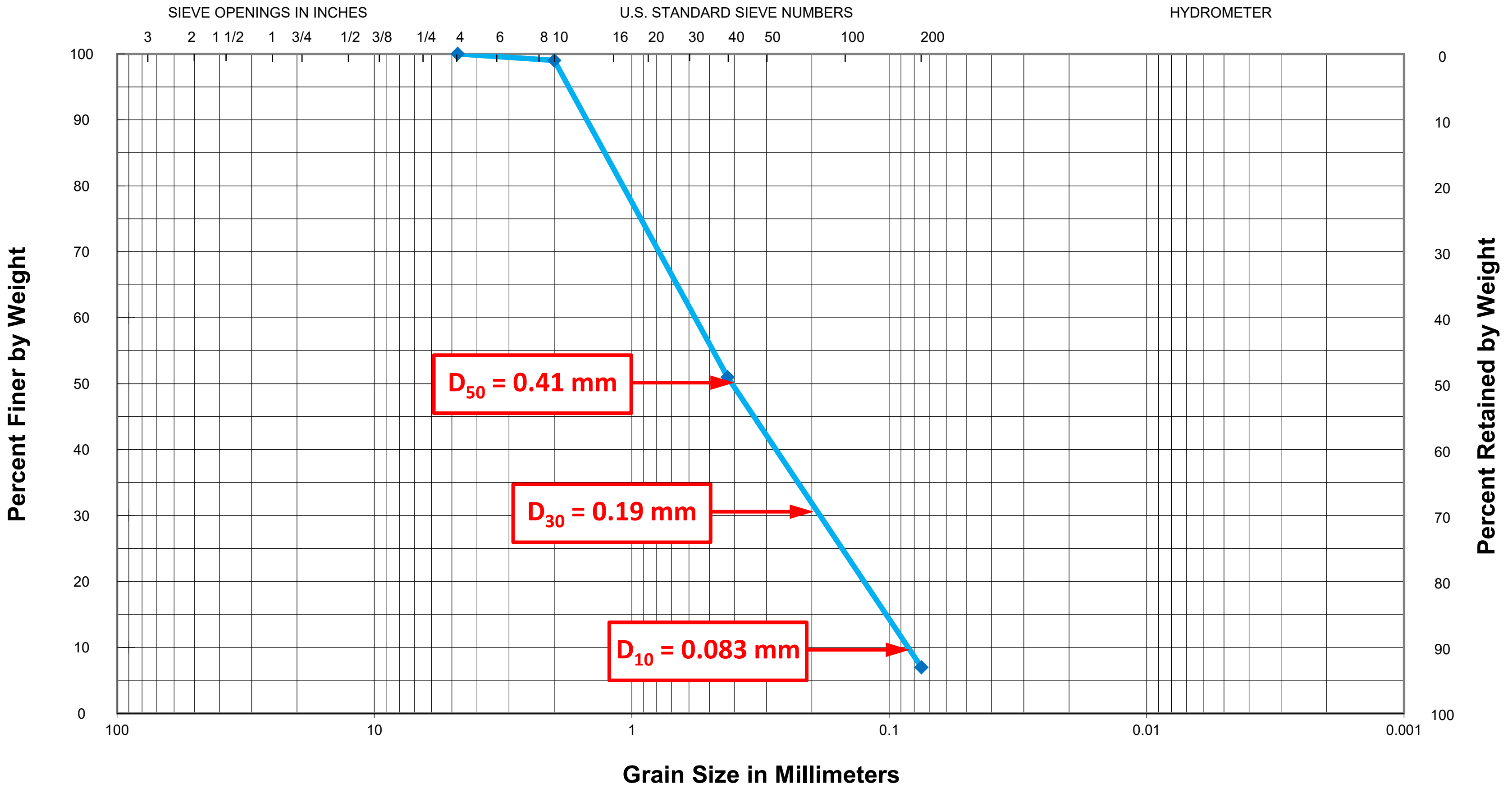

D_{50} = 0.41 mm

D_{30} = 0.19 mm

D_{10} = 0.083 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G2, 44-45 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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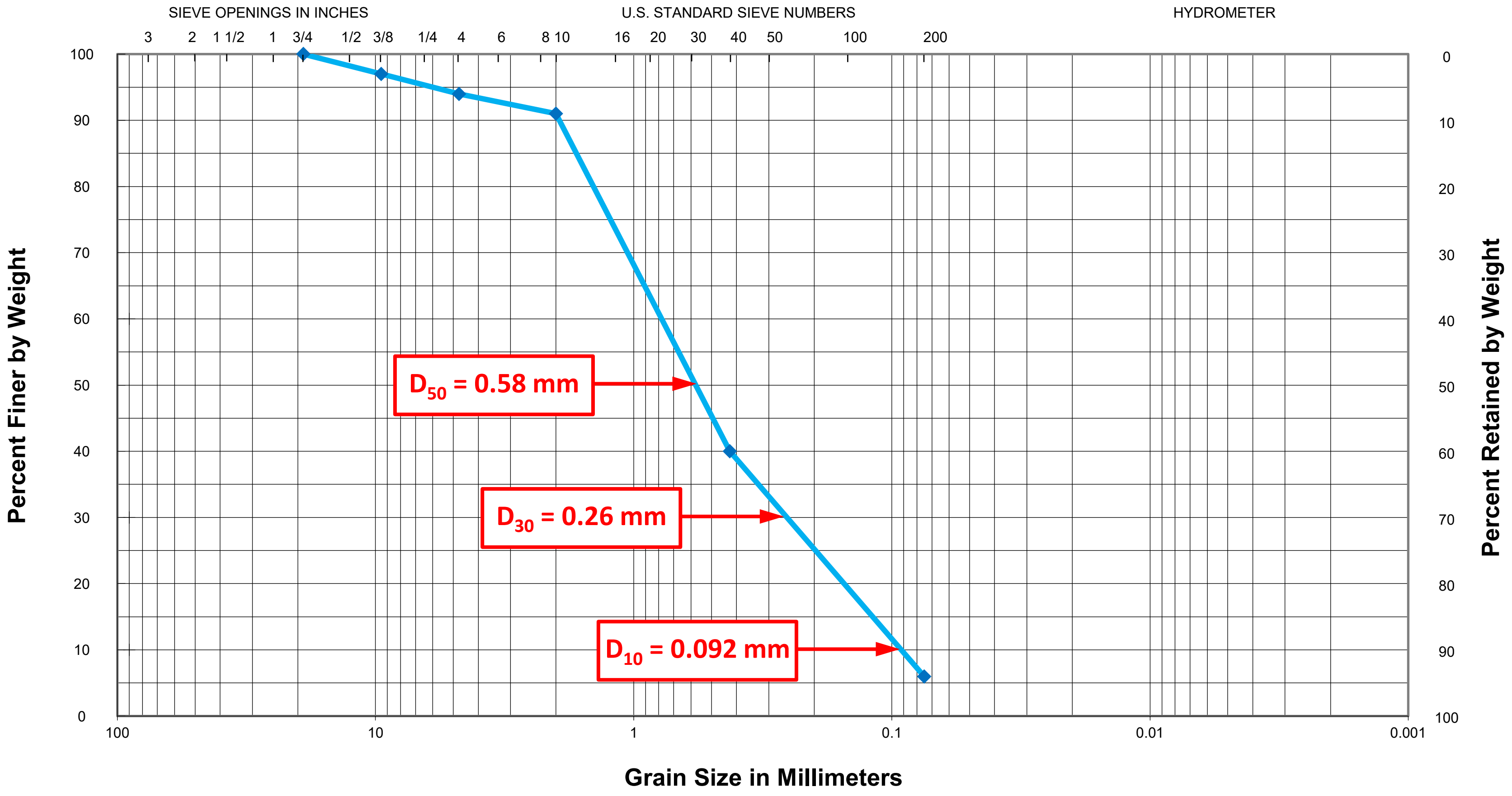
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

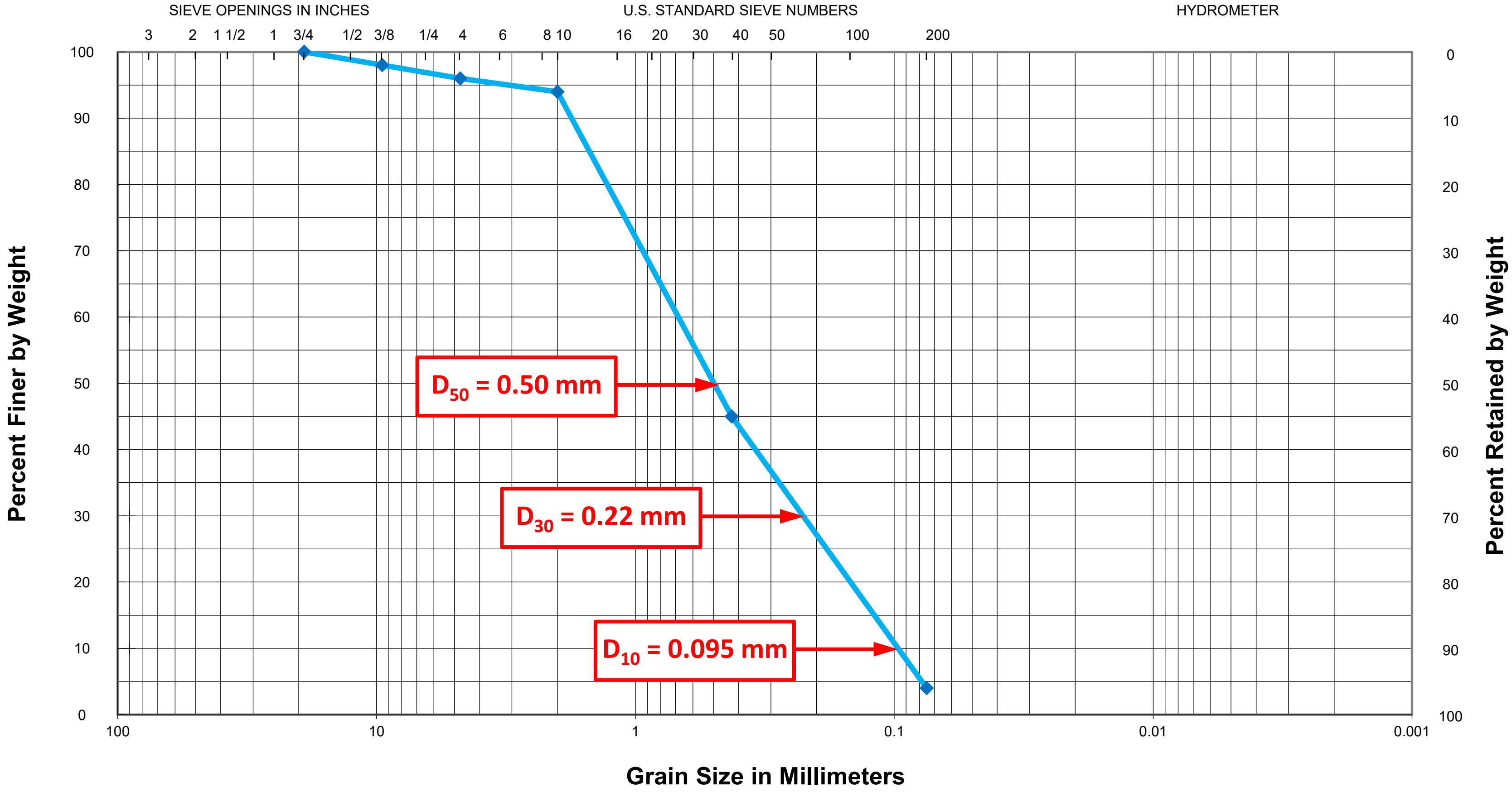
Sample: Boring G2, 74-75 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

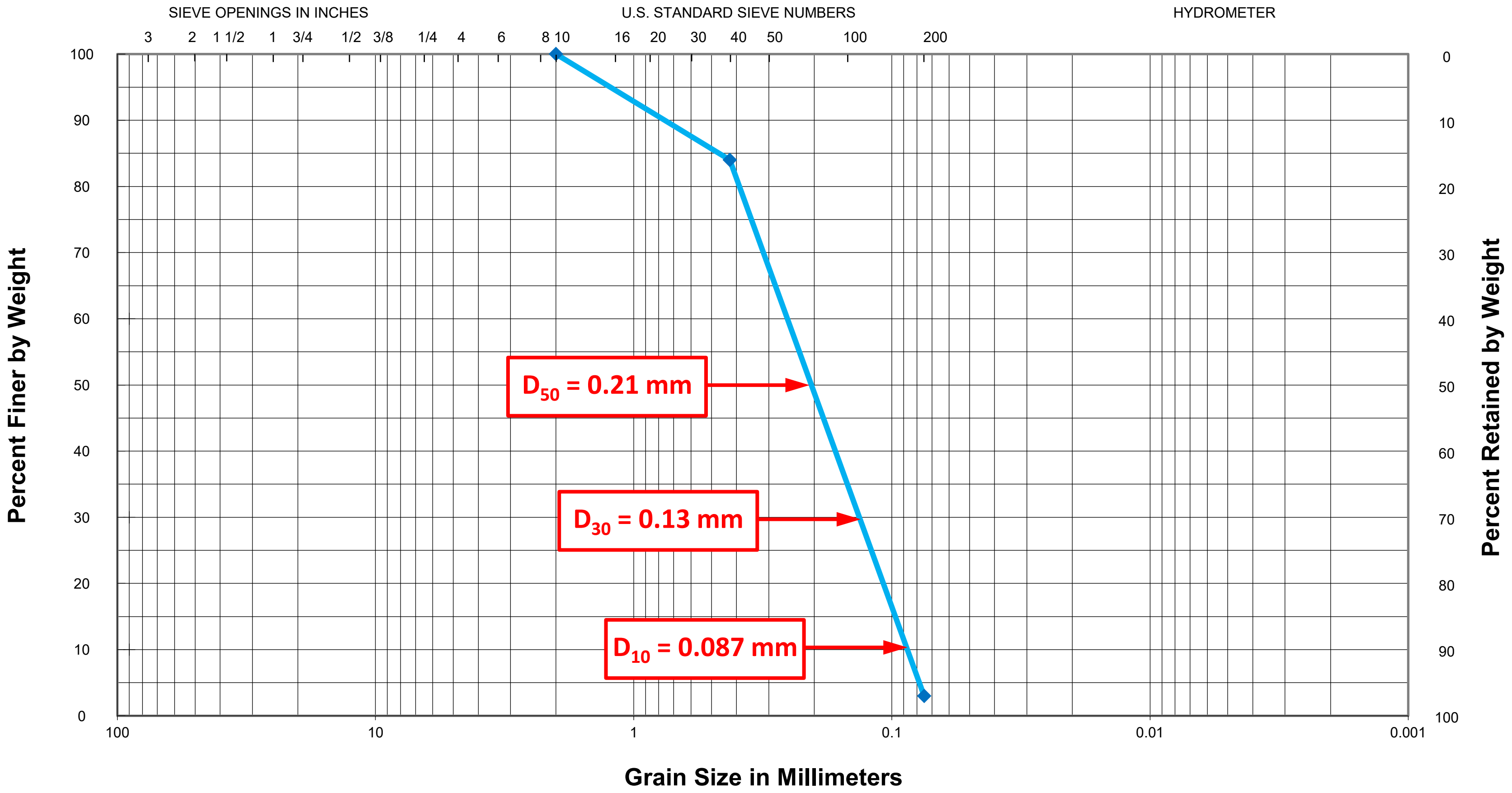
Sample: Boring G2, 94-95 ft
Description: Grayish tan fine to medium SAND

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.21 mm

D_{30} = 0.13 mm

D_{10} = 0.087 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

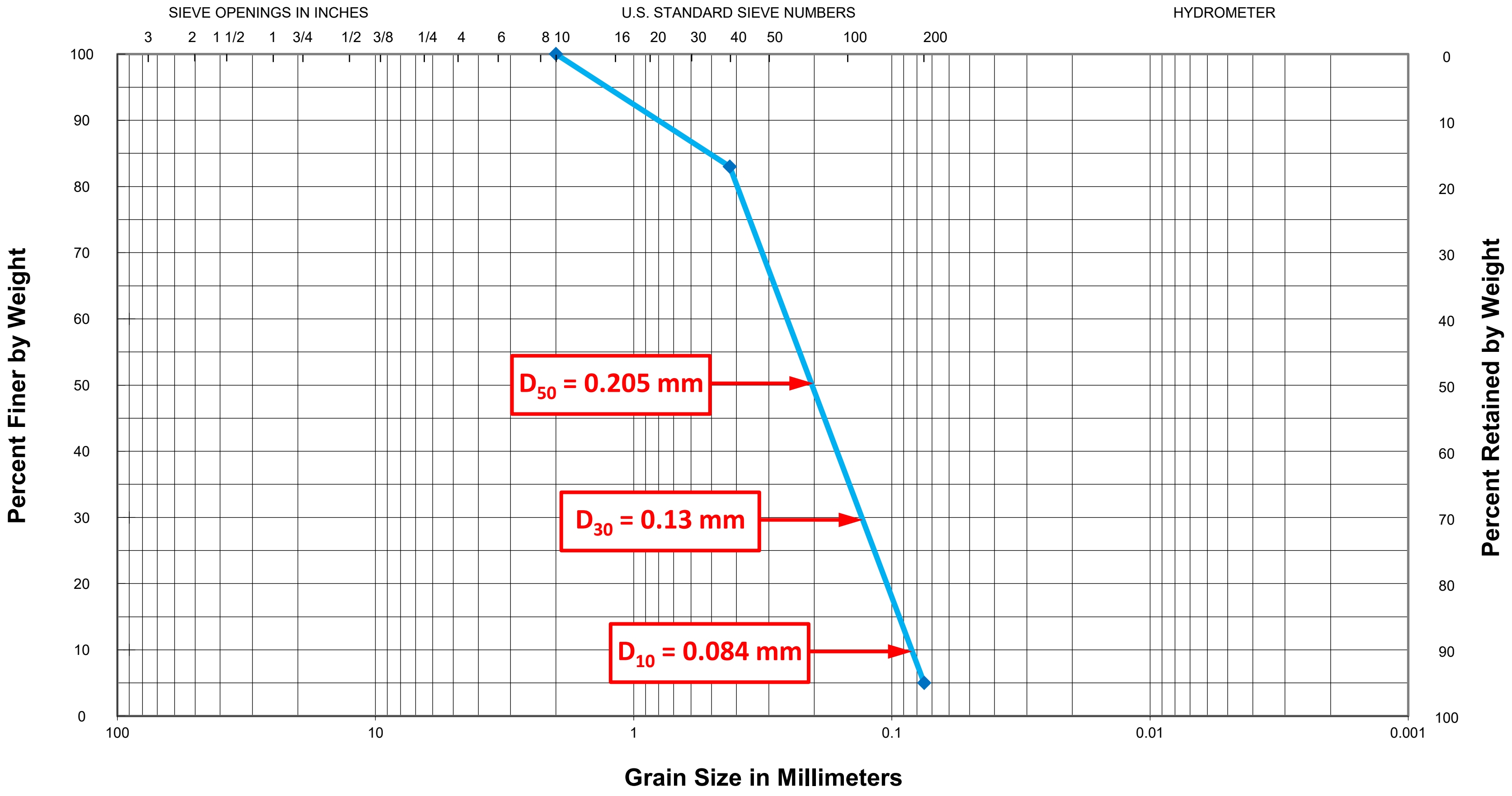
Sample: Boring G3, 19-20 ft
Description: Brownish gray fine to medium SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.205 mm

D_{30} = 0.13 mm

D_{10} = 0.084 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

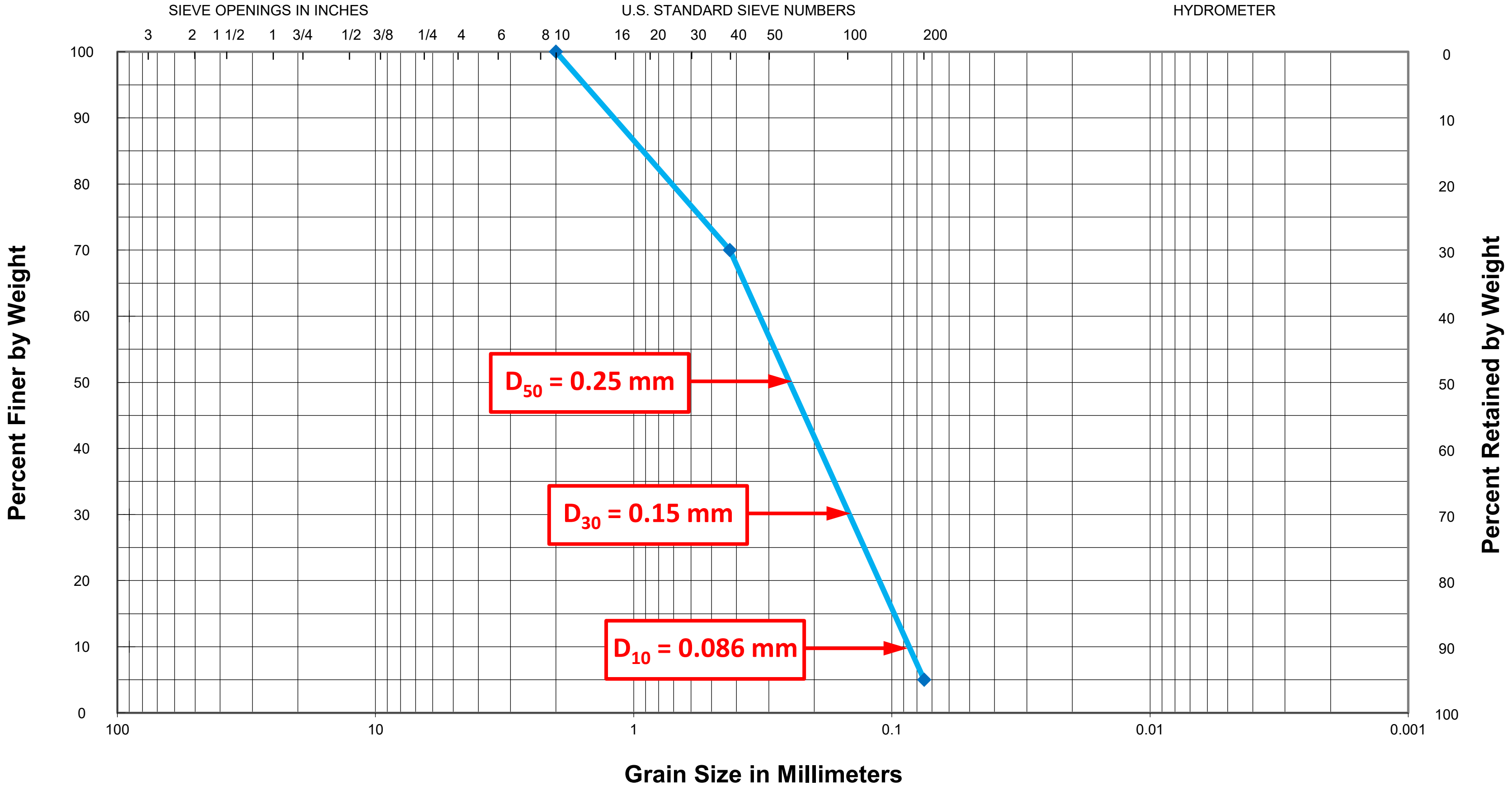
Sample: Boring G3, 34-35 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

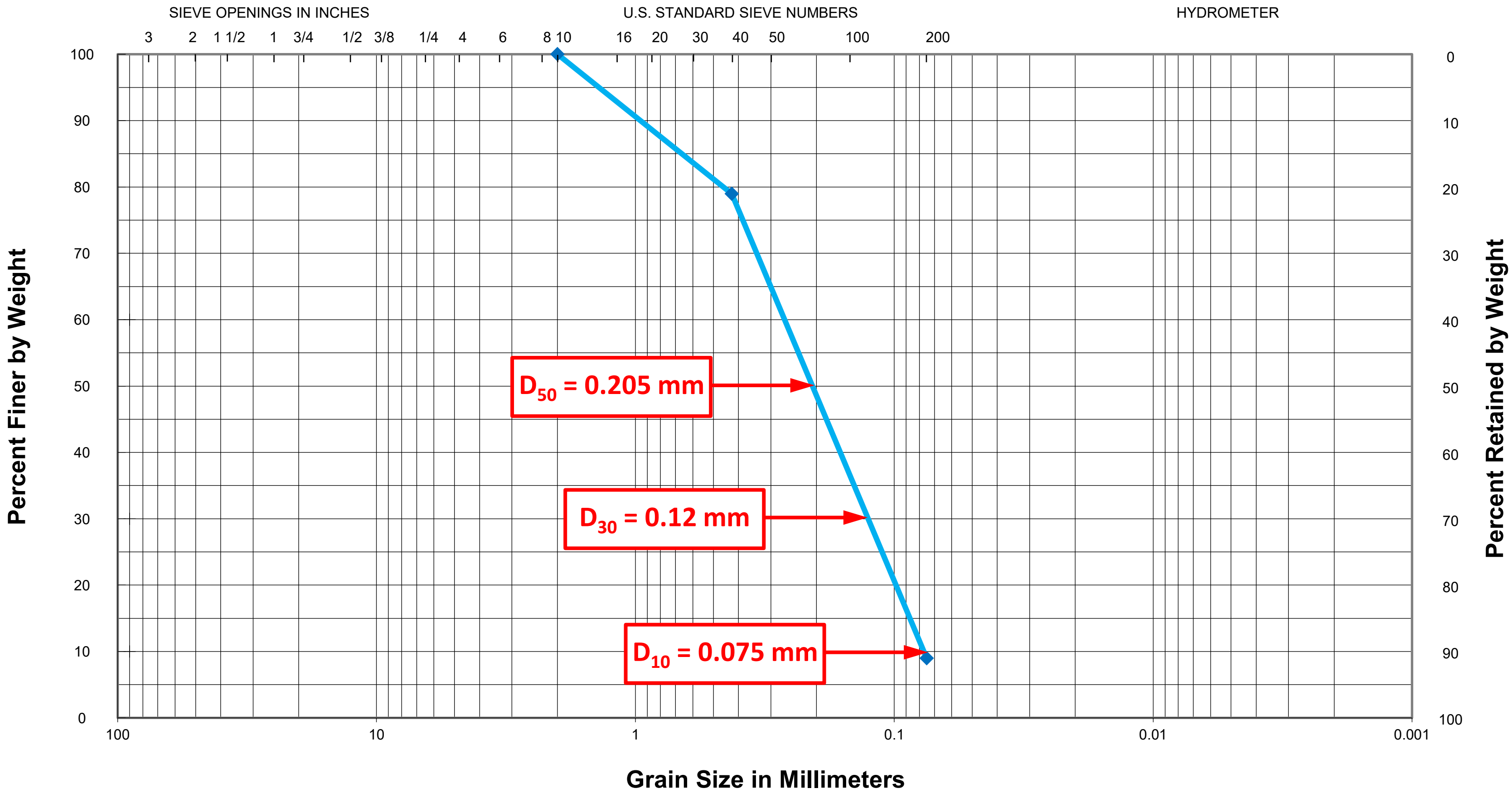
Sample: Boring G3, 54-55 ft
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



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<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

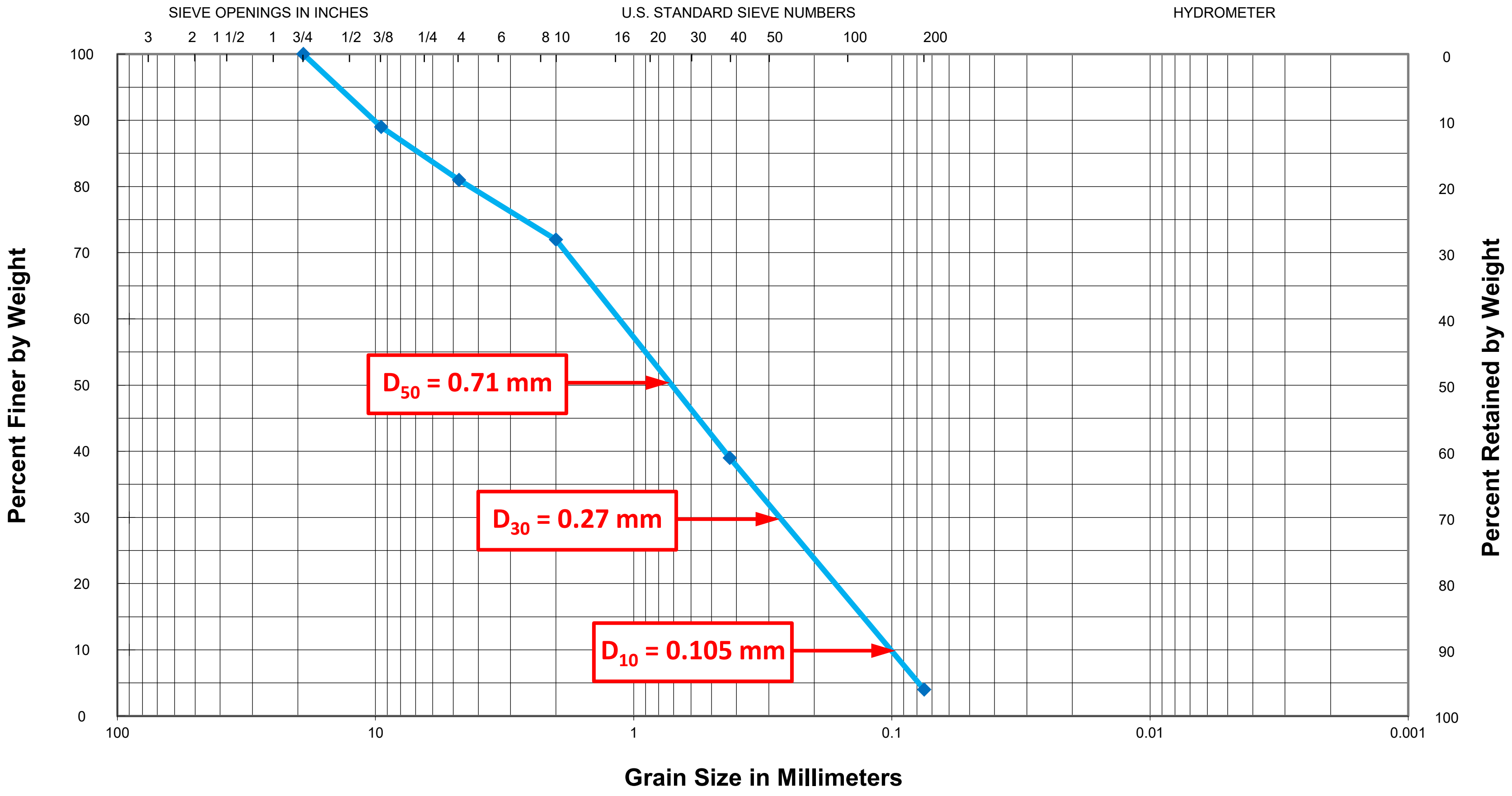
Sample: Boring G3, 69-70 ft; NON-PLASTIC
Description: Grayish brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

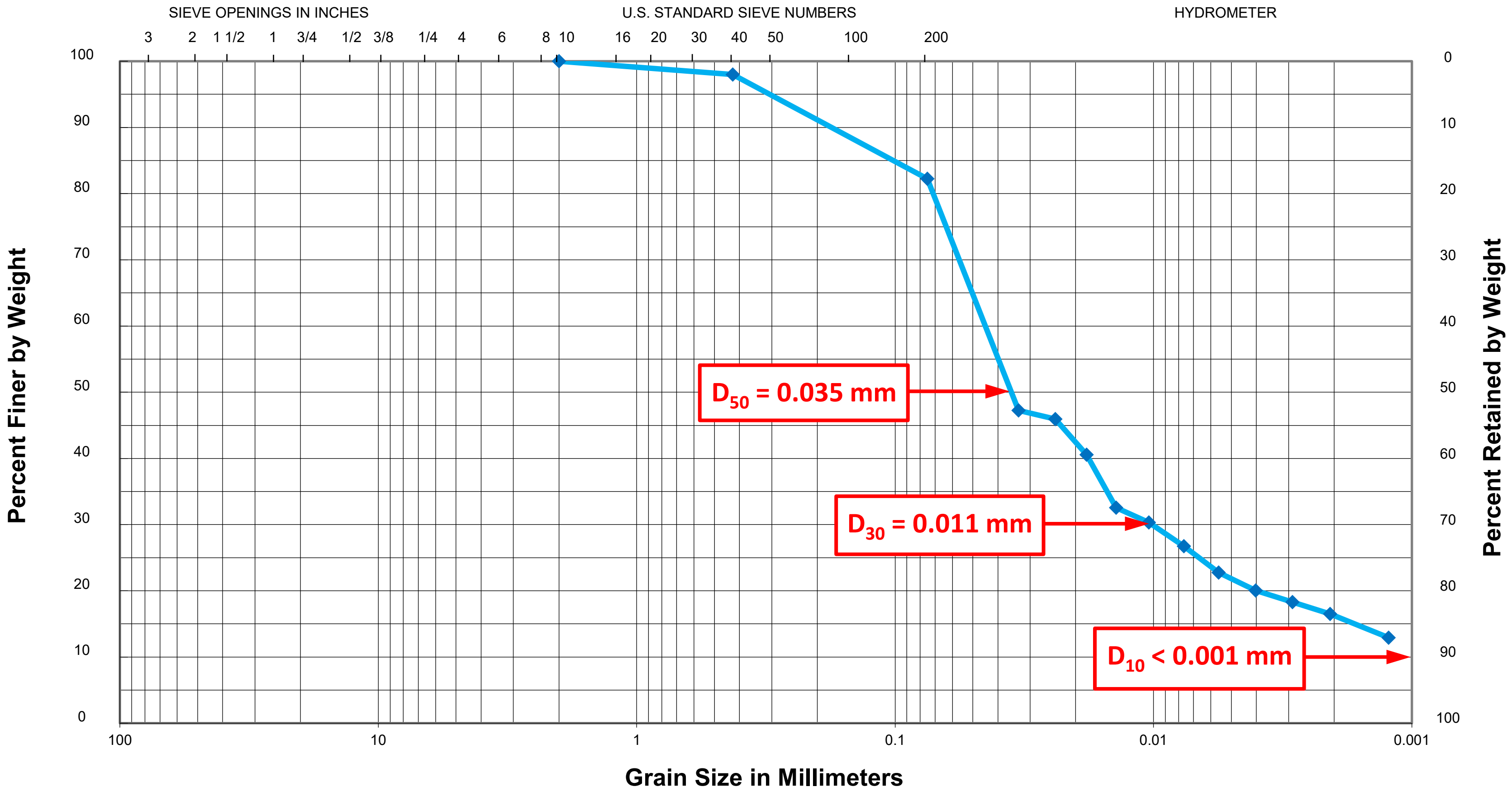


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G3, 109-110 ft
Description: Grayish brown fine to coarse SAND w/ a little fine gravel

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

GRAIN SIZE CURVE



D_{50} = 0.035 mm

D_{30} = 0.011 mm

D_{10} < 0.001 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

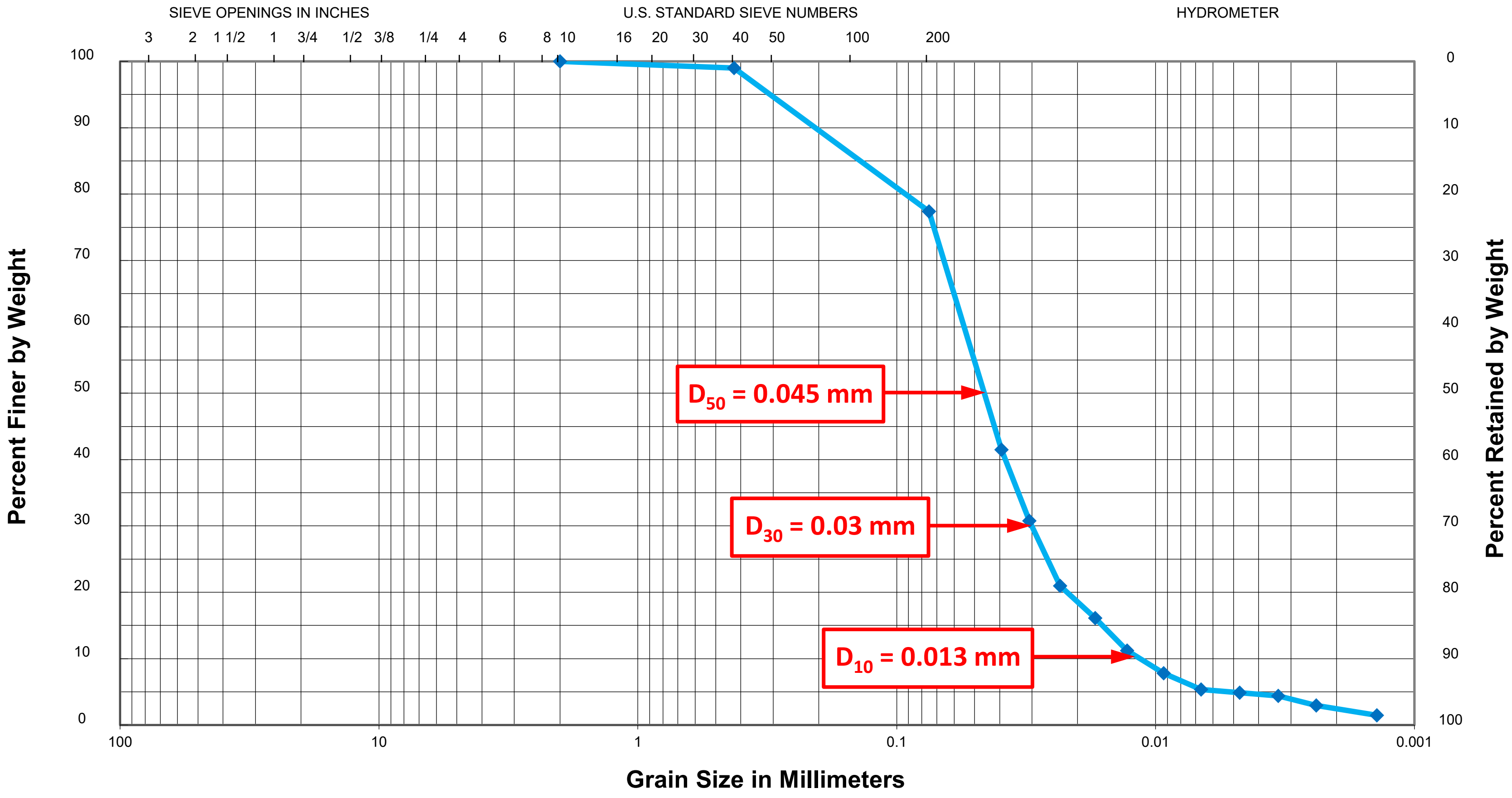
Sample: Boring G4, 4.5-5.5 ft
Description: Light brownish gray and tan silty CLAY, slightly sandy

USCS Classification = CL
AASHTO Classification = A-6

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

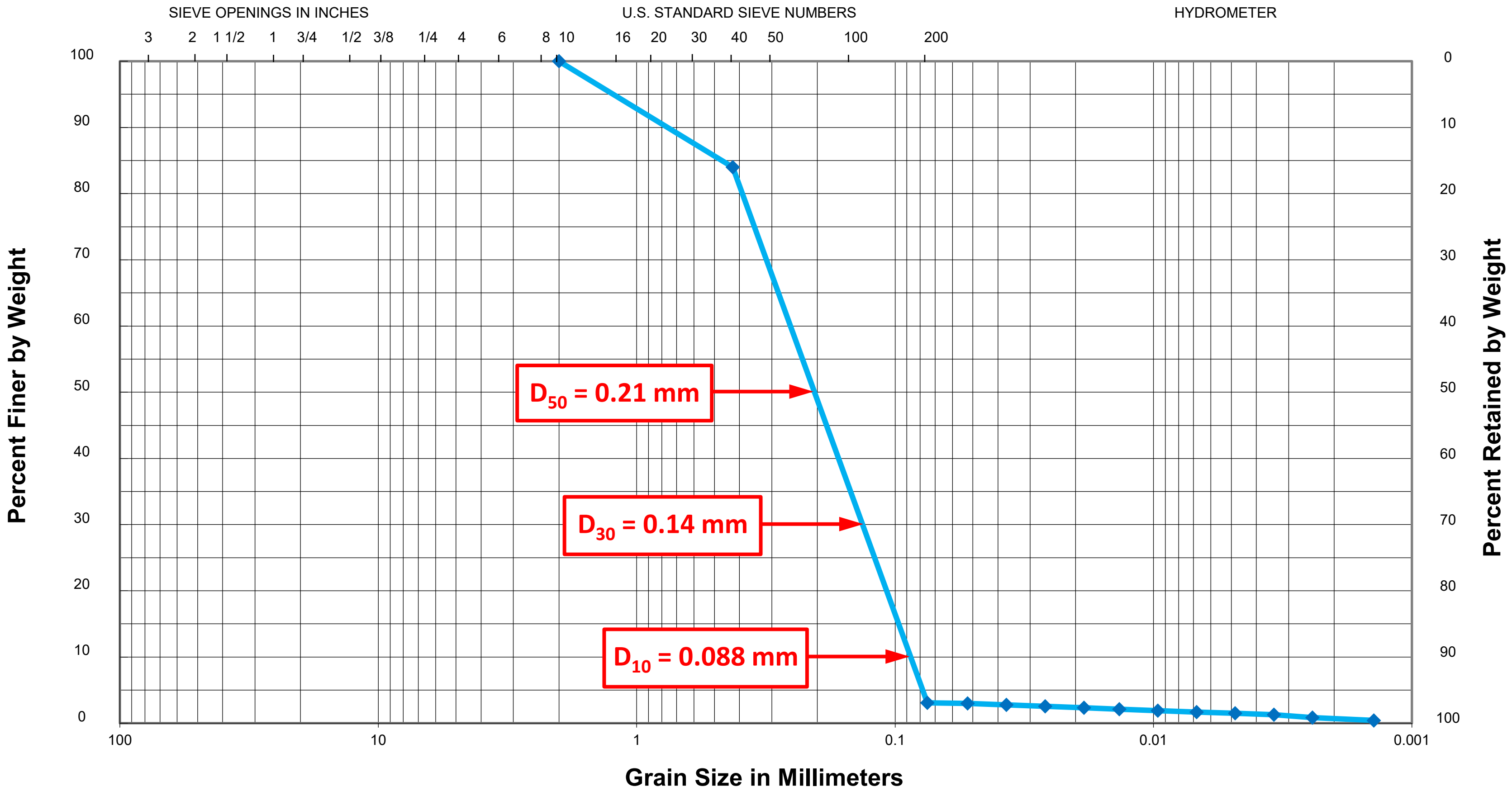
Sample: Boring G4, 9-10 ft
Description: Brown and brownish gray fine sandy SILT

USCS Classification = ML
AASHTO Classification = A-4

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.21 mm

D_{30} = 0.14 mm

D_{10} = 0.088 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

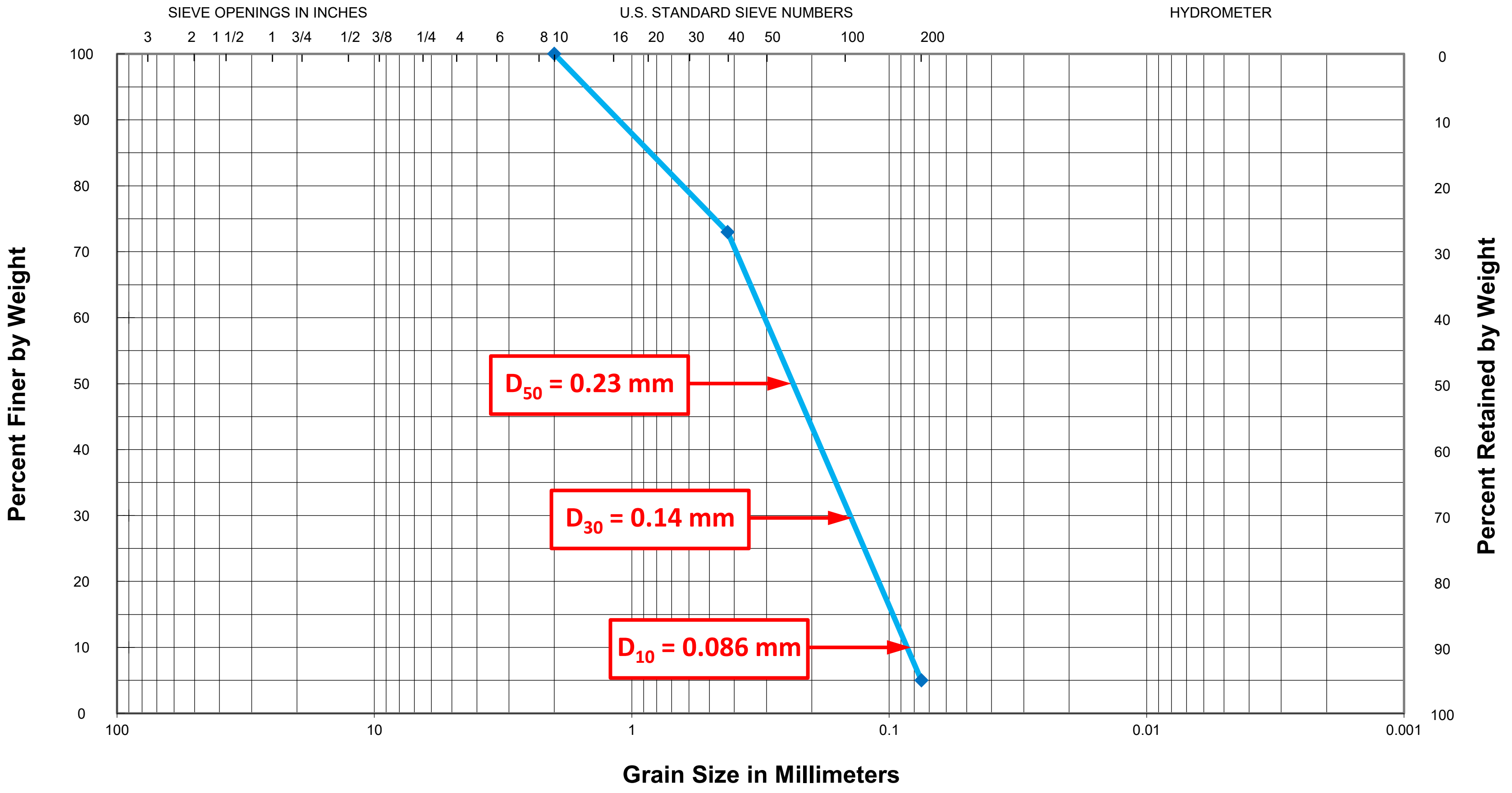
Sample: Boring G4, 14-15 ft
Description: Brown fine SAND

USCS Classification = SP
AASHTO Classification = A-3

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

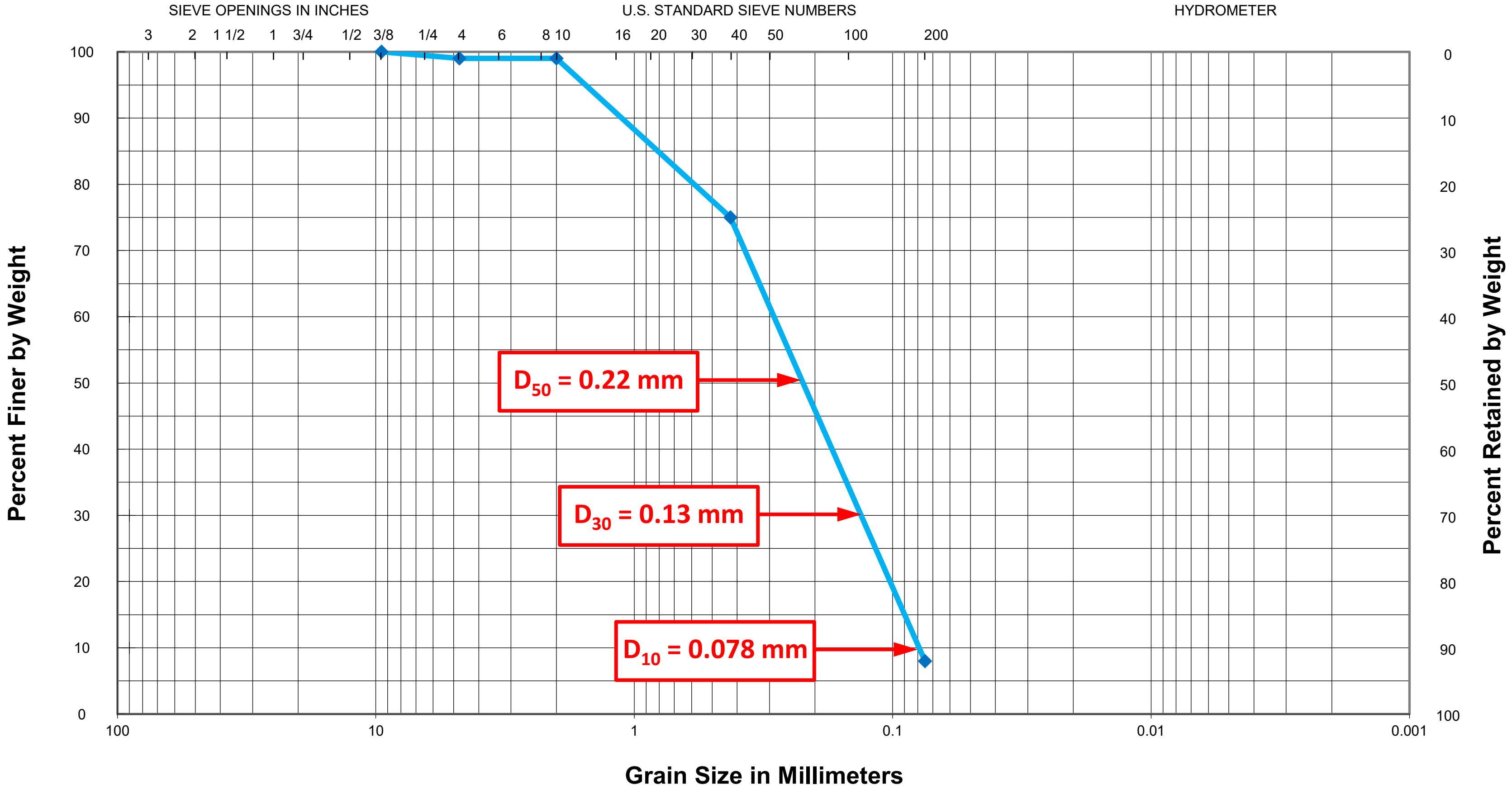
Sample: Boring G4, 24-25 ft
Description: Brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



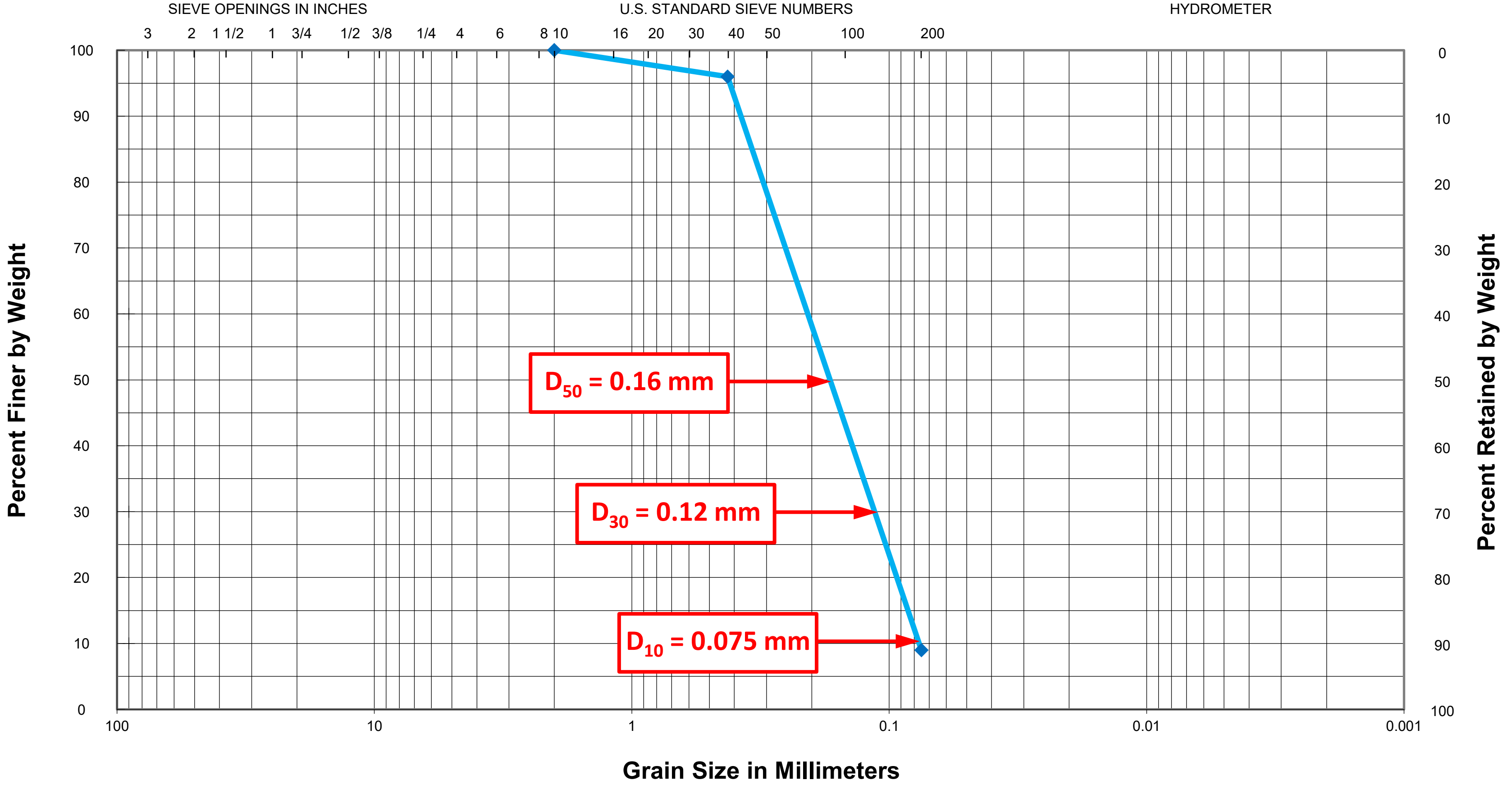
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G4, 39-40 ft
Description: Brown fine to medium SAND, slightly silty

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

23-031

GRAIN SIZE CURVE



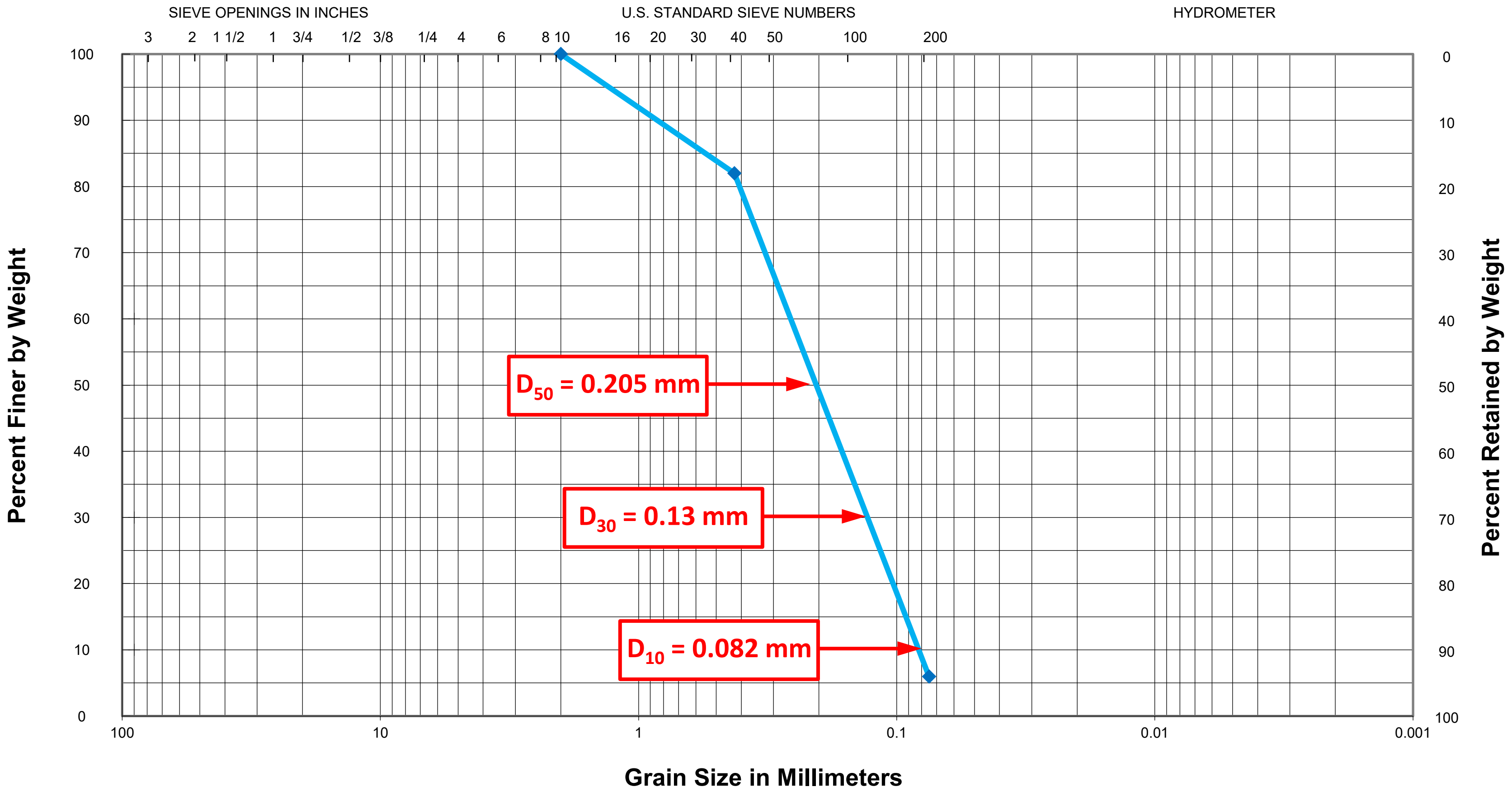
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G4, 69-70 ft
Description: Grayish brown and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE



D_{50} = 0.205 mm

D_{30} = 0.13 mm

D_{10} = 0.082 mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring G4, 74-75 ft
Description: Grayish brown and brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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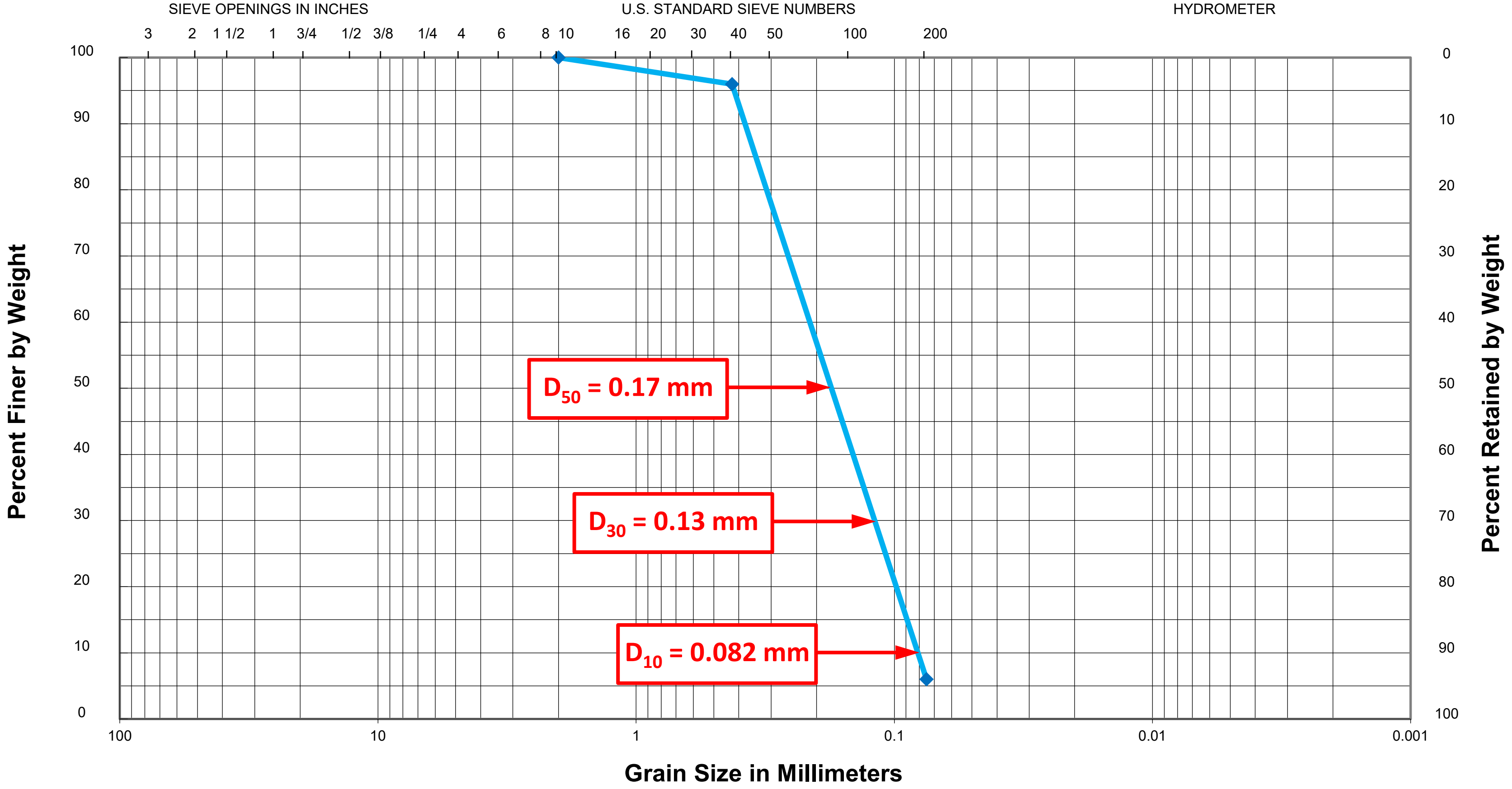
CONSULTING ENGINEERS
A UES Company



D_{50} = 0.17 mm

D_{30} = 0.13 mm

D_{10} = 0.082 mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring G4, 89-90 ft
Description: Graysih brown fine SAND, slightly silty

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

23-031

GRAIN SIZE CURVE

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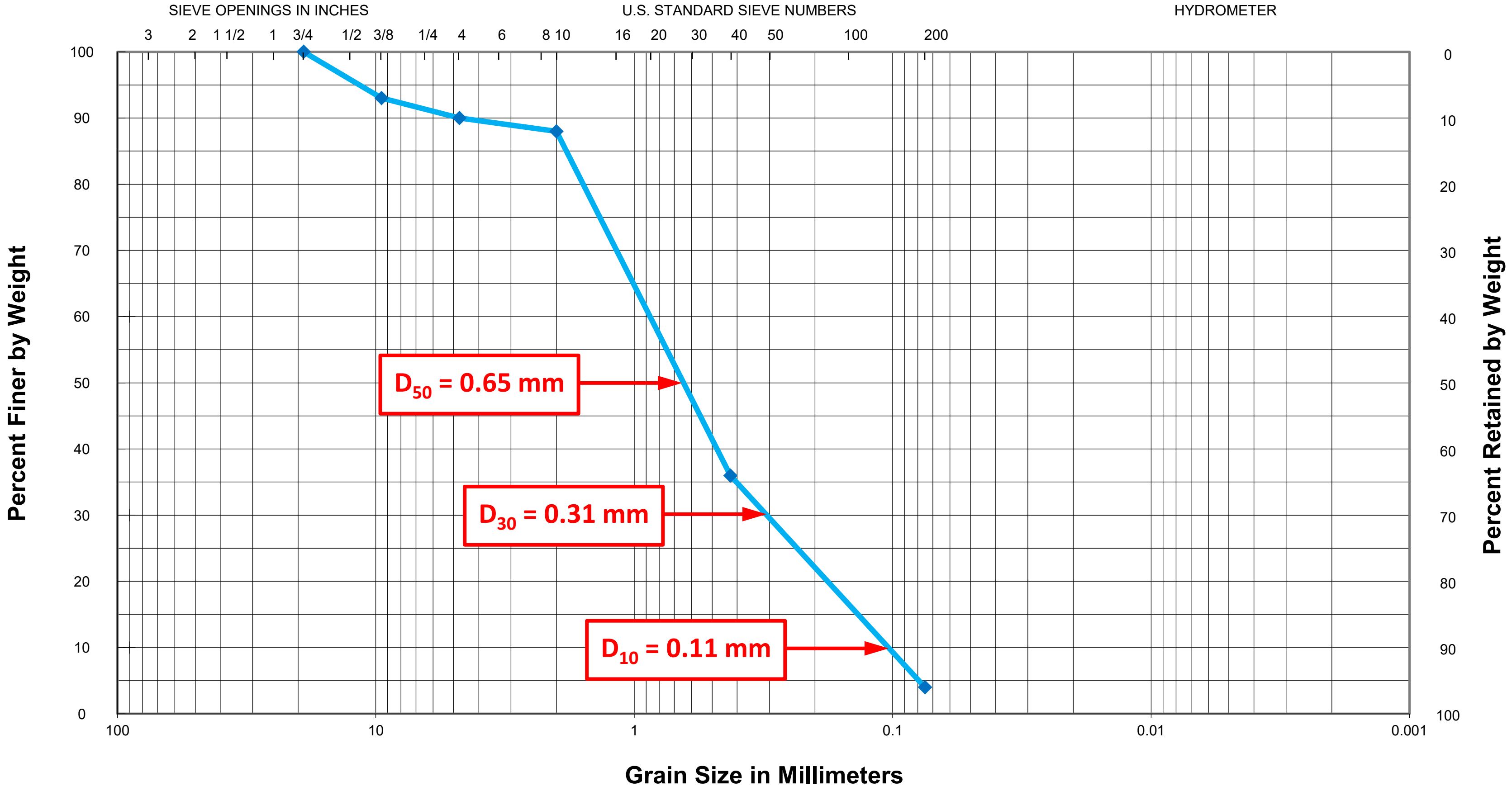
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

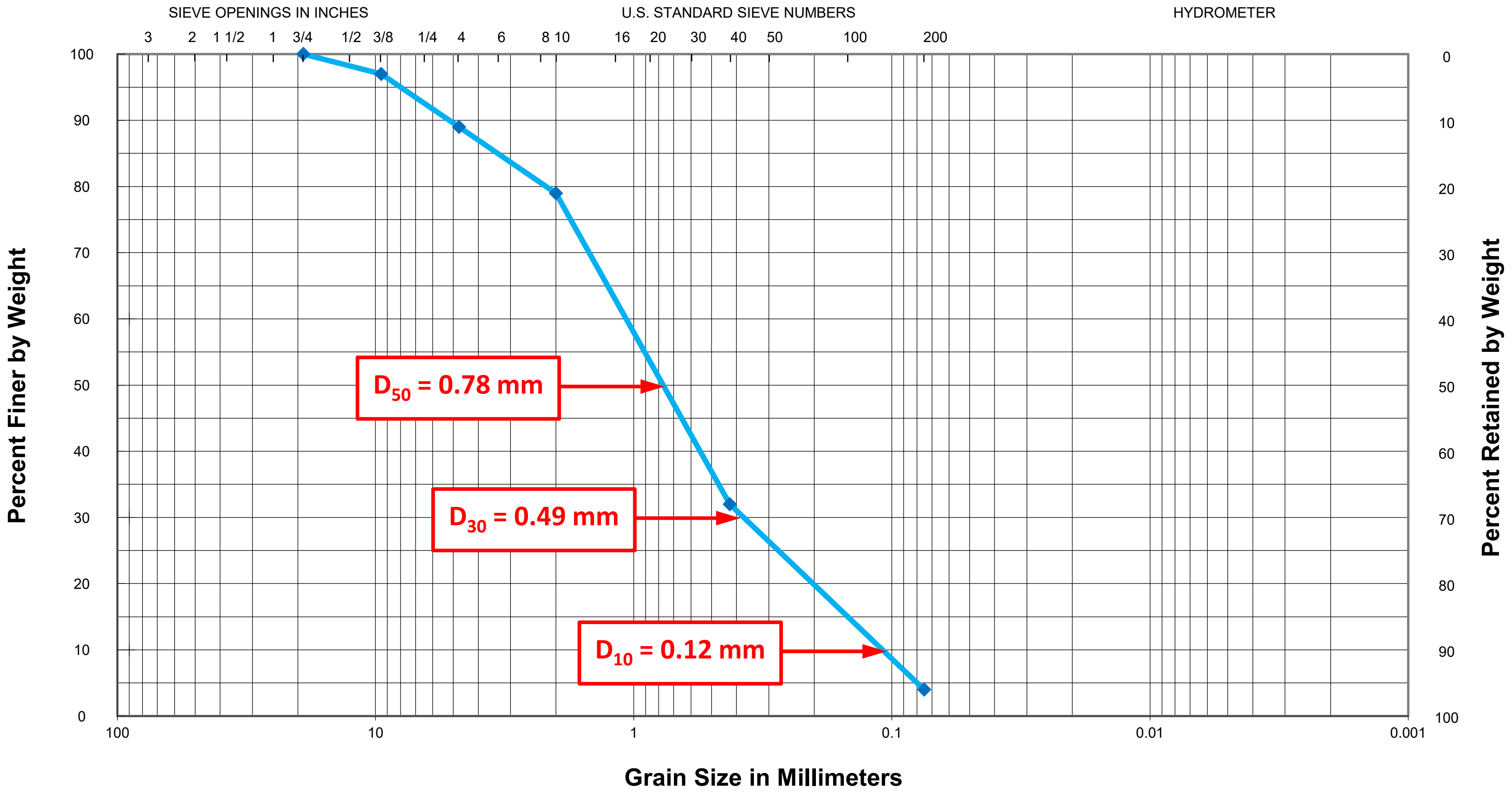
Sample: Boring G4, 94-95 ft
Description: Brown fine to medium SAND w/ trace fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

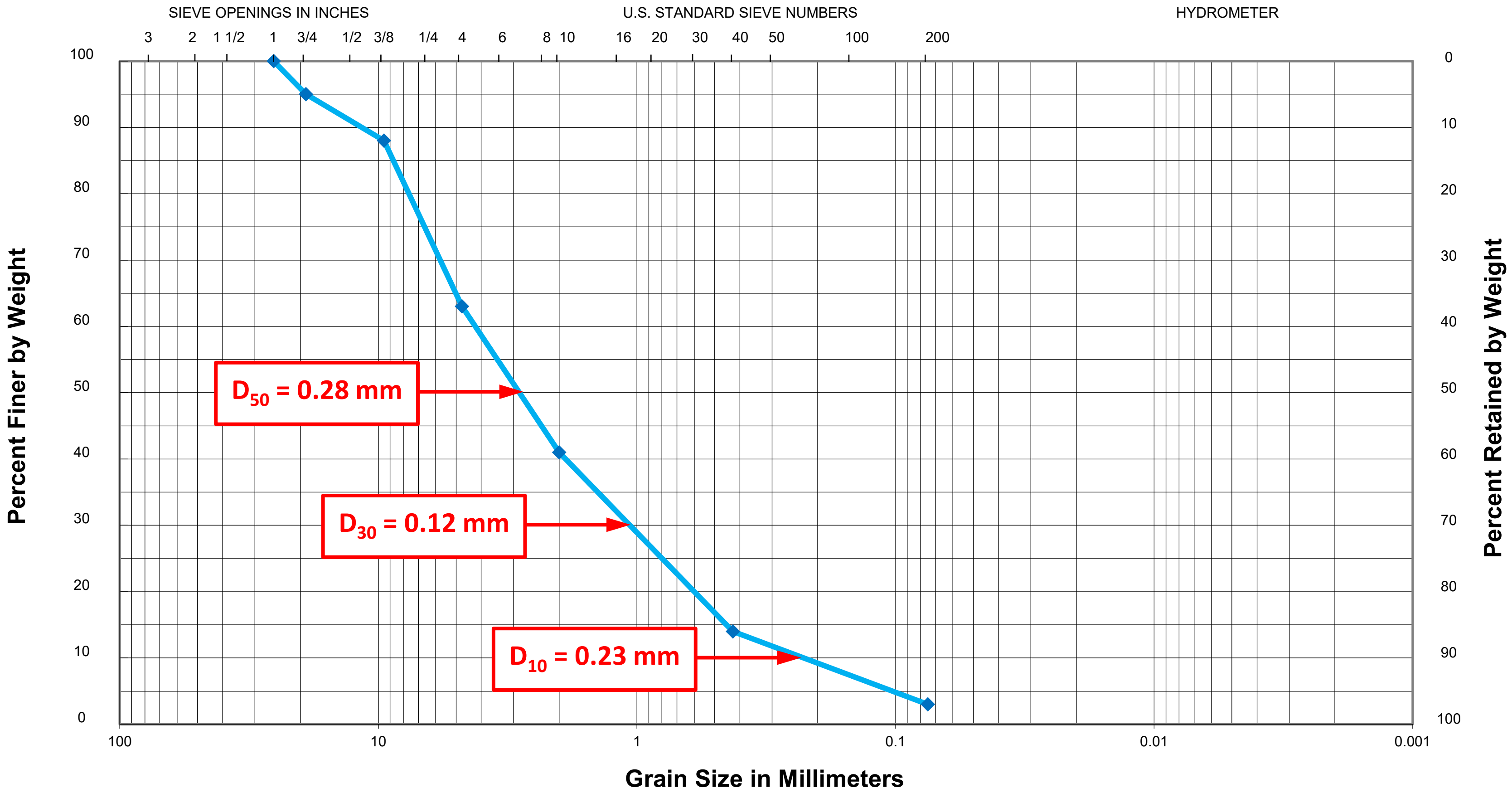
Sample: Boring G4, 99-100 ft
Description: Gray and tan fine to coarse SAND w/ trace fine gravel

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

23-031

GRAIN SIZE CURVE

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

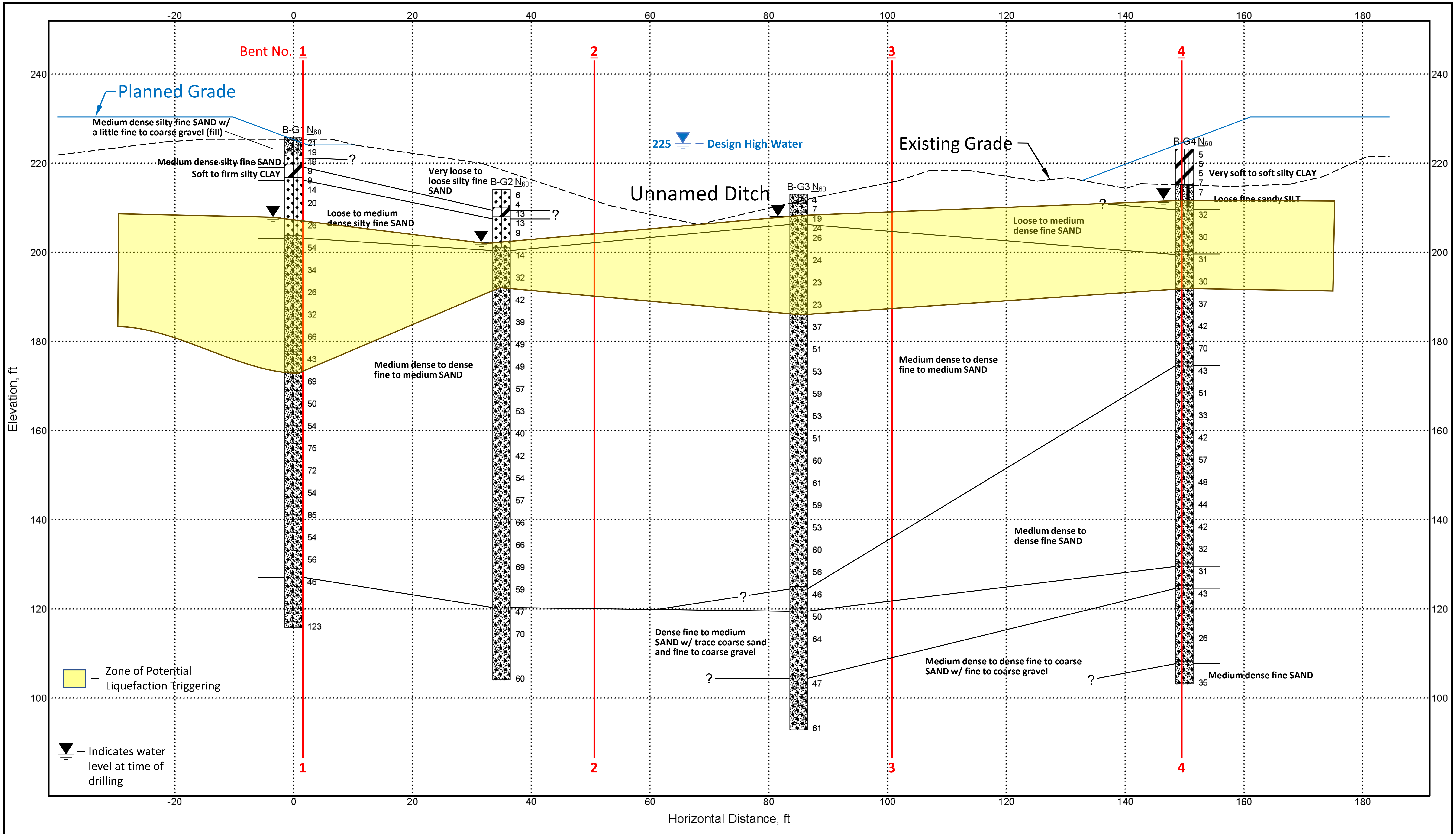


GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring G4, 109-110 ft
Description: Gray and tan fine to coarse SAND w/ some fine to coarse gravel

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

APPENDIX D



Grubbs, Hoskyn,
Barton & Wyatt, LLC

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

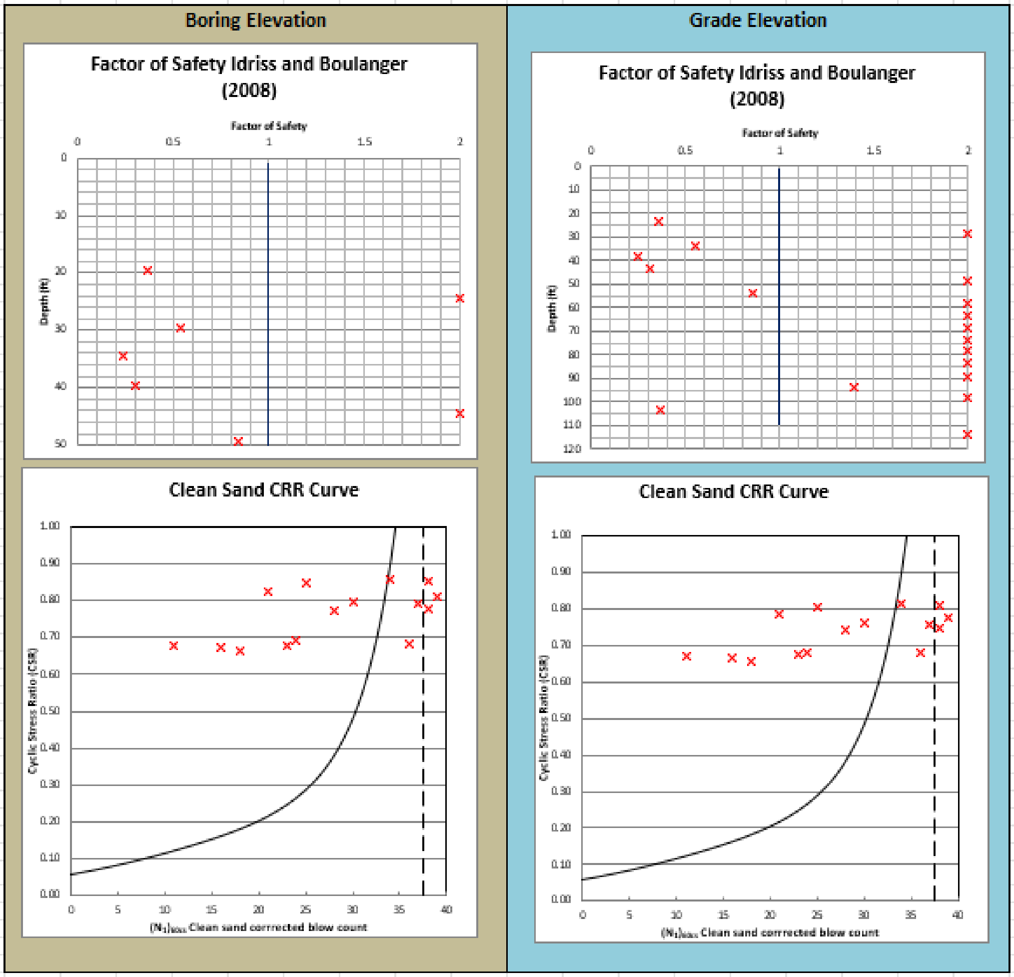
SCALE:
1'' = 15' Horizontal
1'' = 20' Vertical

Generalized Subsurface Profile
101124 Hwy. 135 over Unnamed Ditch
Poinsett & Craighead County, Arkansas

Project Number: 23-031

Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Unnamed Ditch
Bent 1 / Boring G1
GHBW Job No. 23-031
Poinsett County, Arkansas

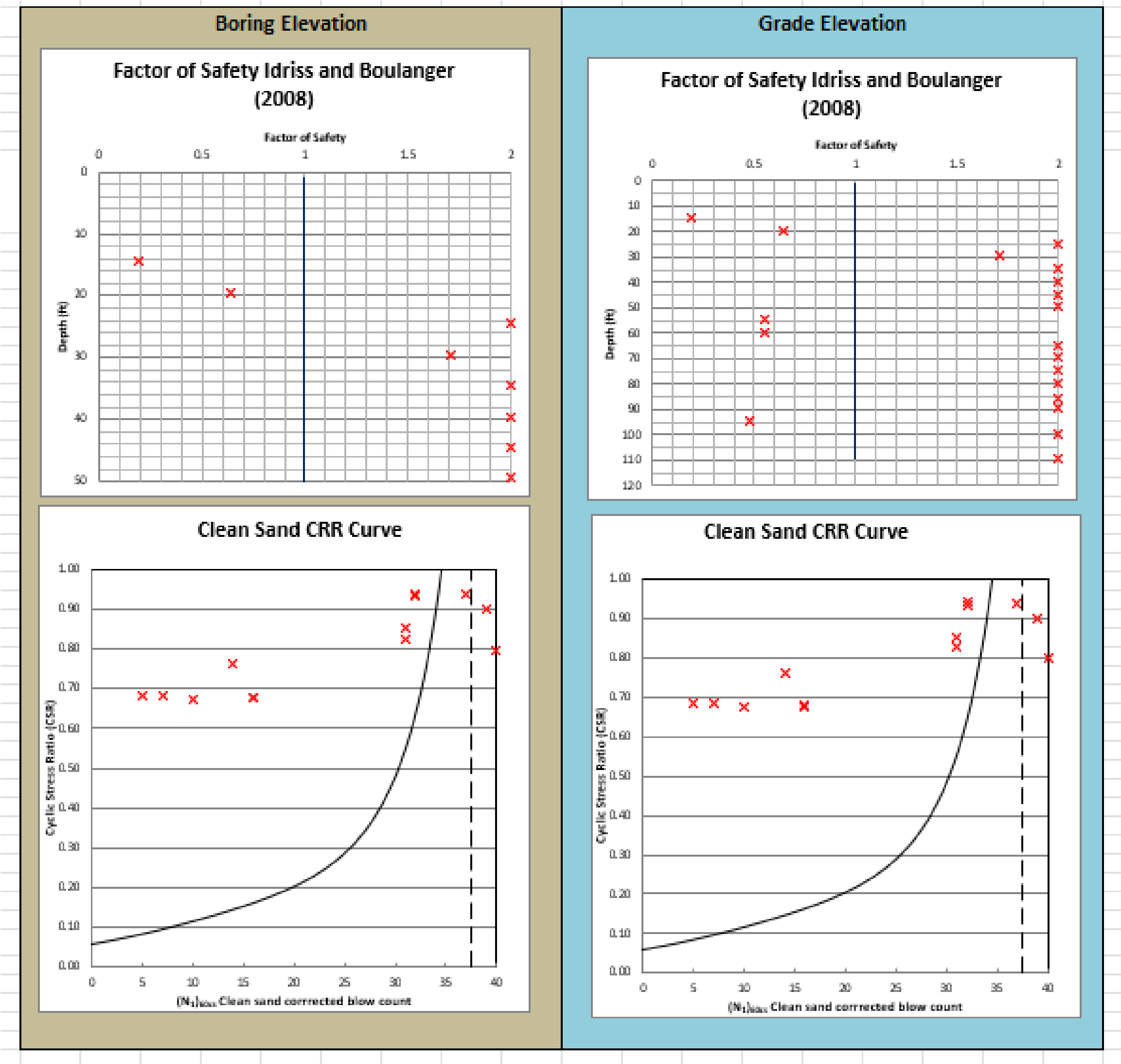


Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS

A UES Company

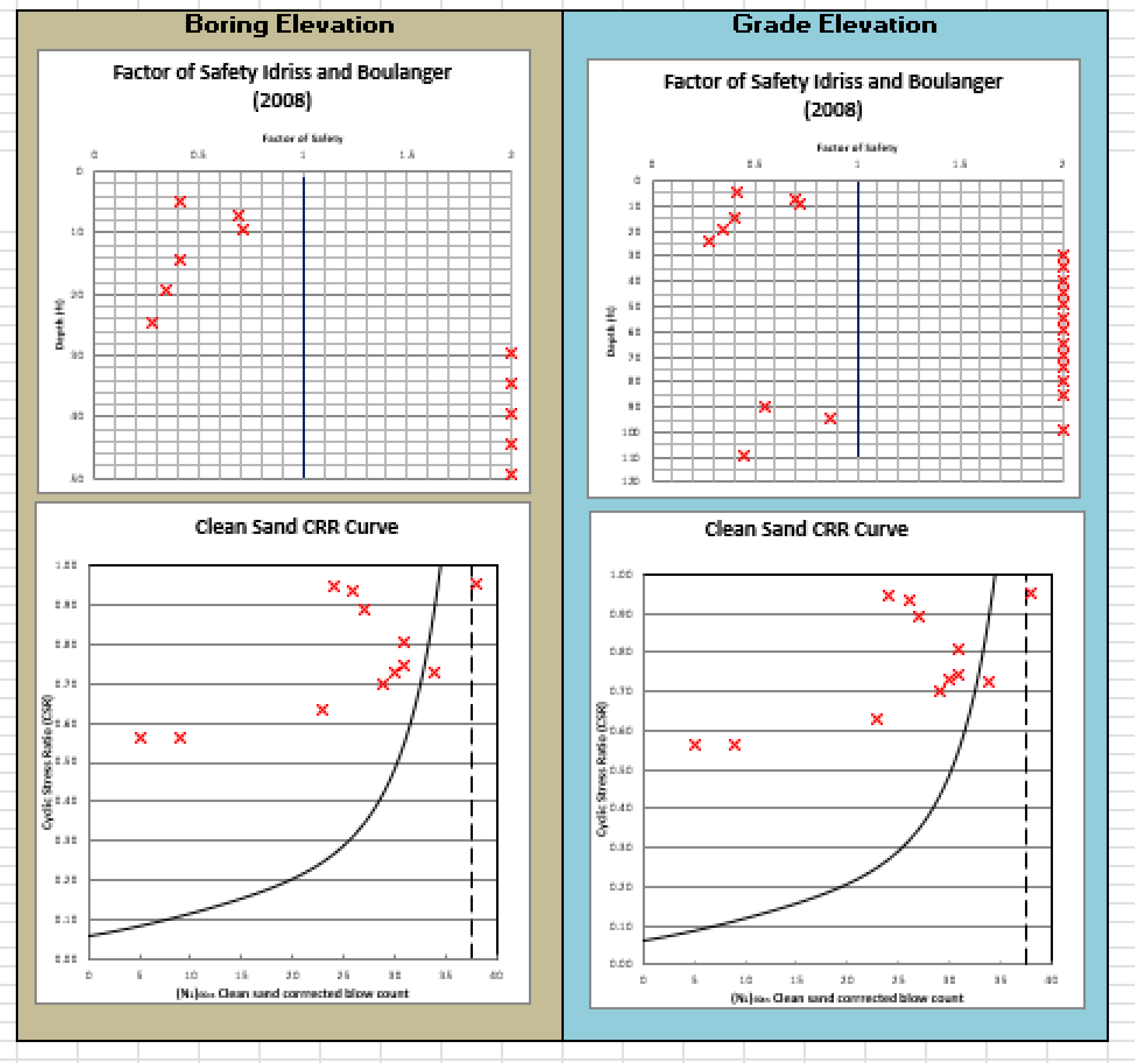
Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Unnamed Ditch
Bent 2 / Boring G2
GHBW Job No. 23-031
Poinsett County, Arkansas



Liquefaction Analysis Results

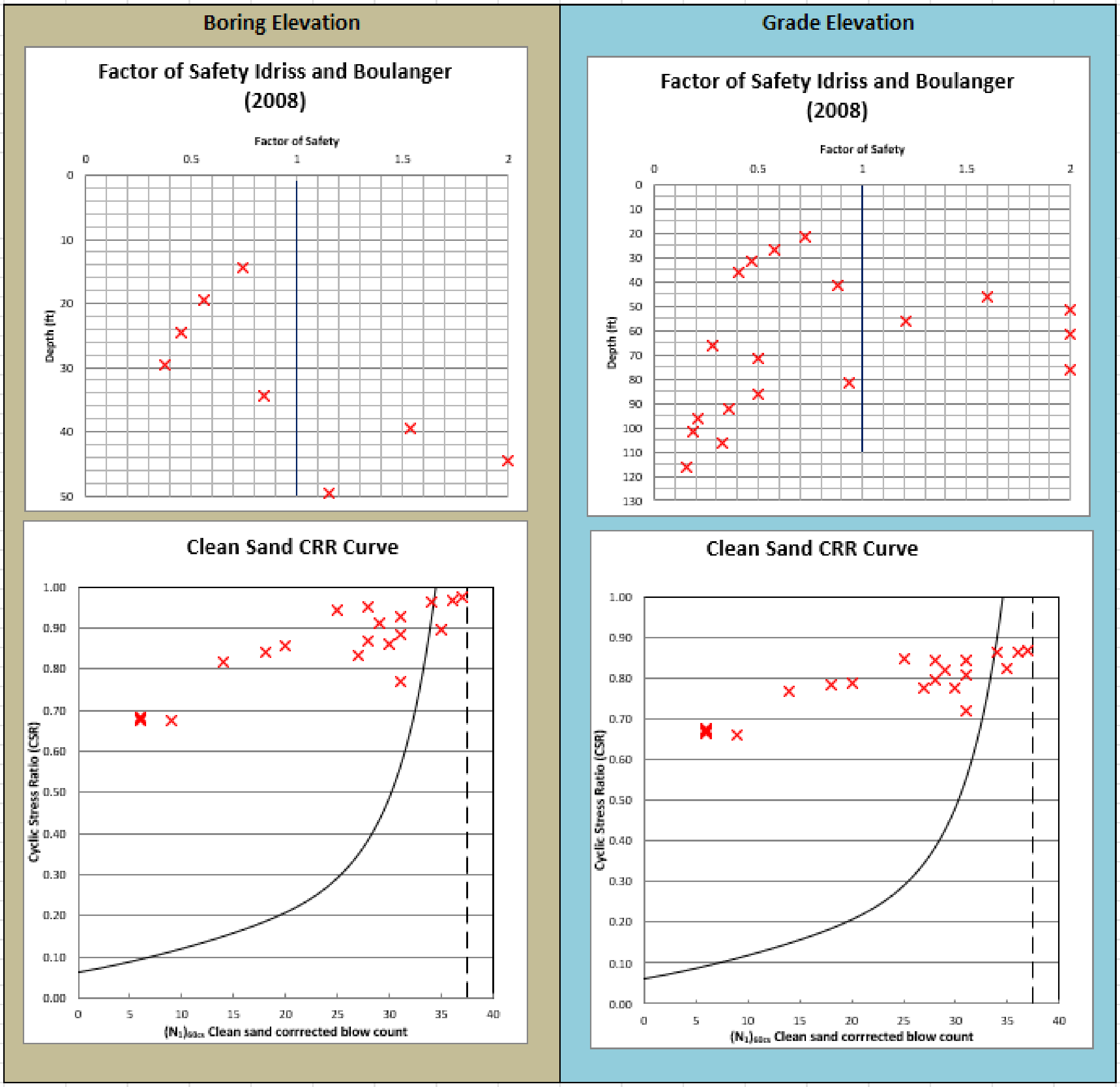
ARDOT 101124 Hwy 135 over Unnamed Ditch
Bent 3 / Boring G3
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

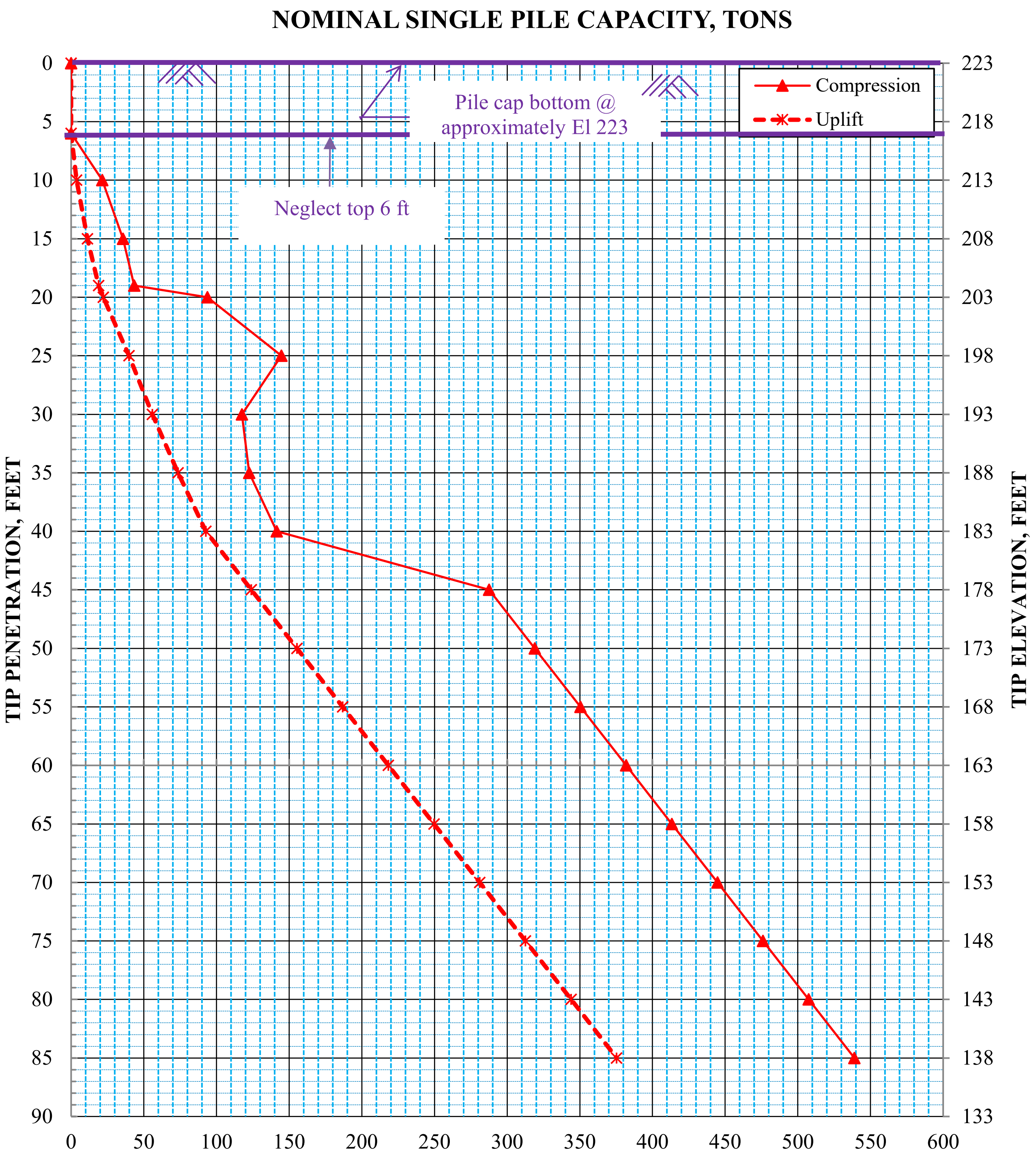
Liquefaction Analysis Results

ARDOT 101124 Hwy 135 over Unnamed Ditch
Bent 4 / Boring G4
GHBW Job No. 23-031
Poinsett County, Arkansas



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

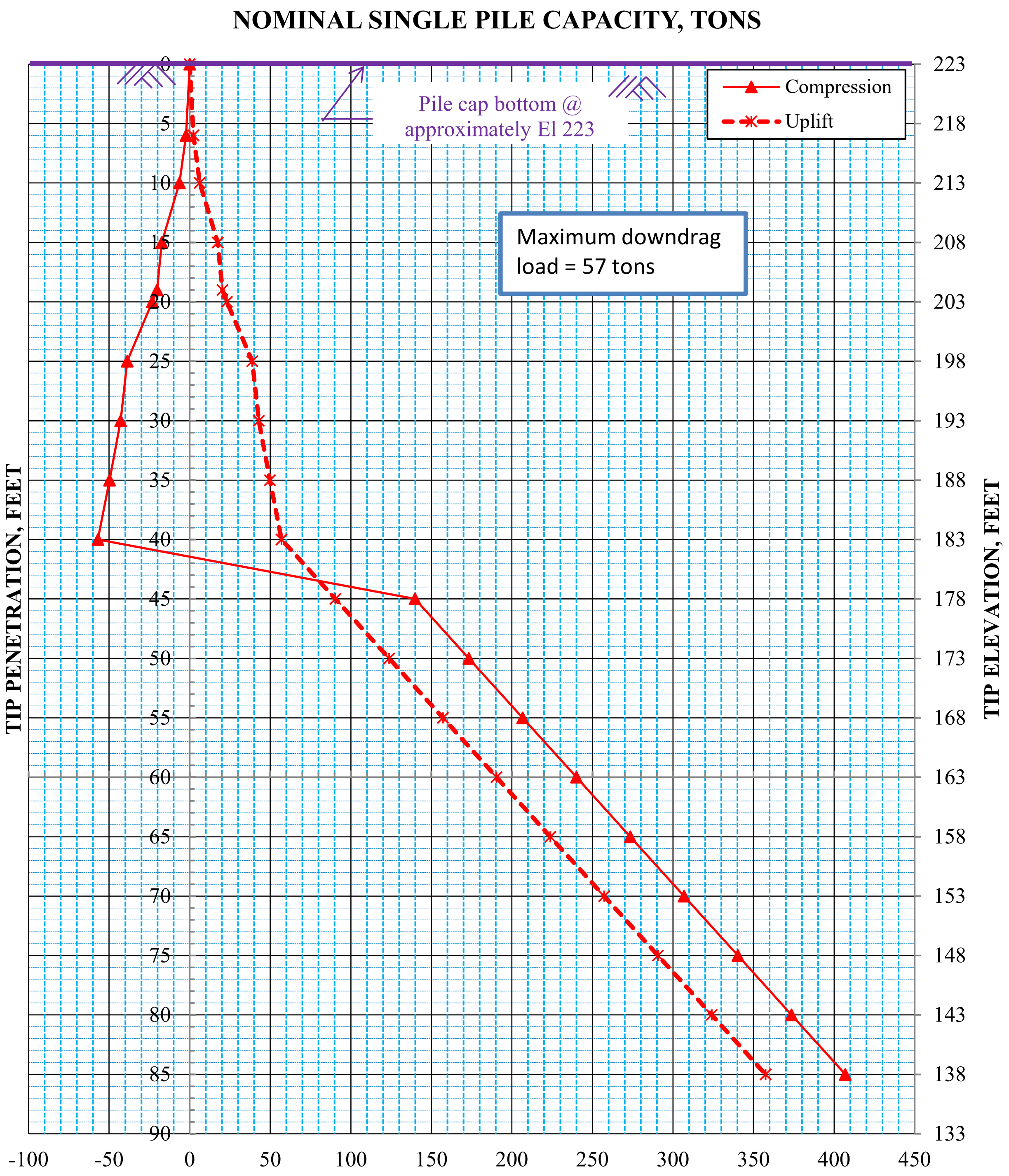
APPENDIX E



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

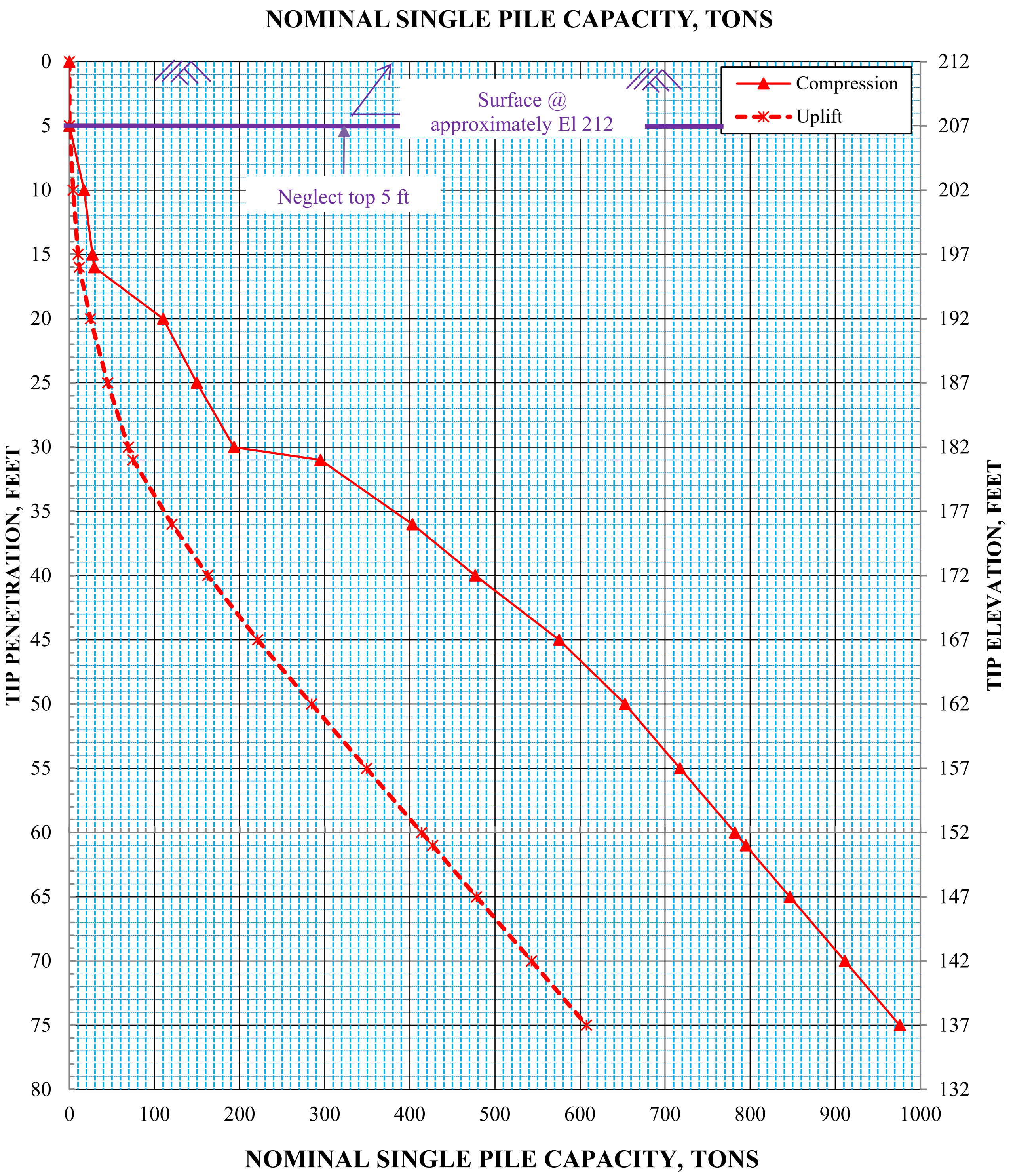
Notes: 1. Driven from cap bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 1 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 183

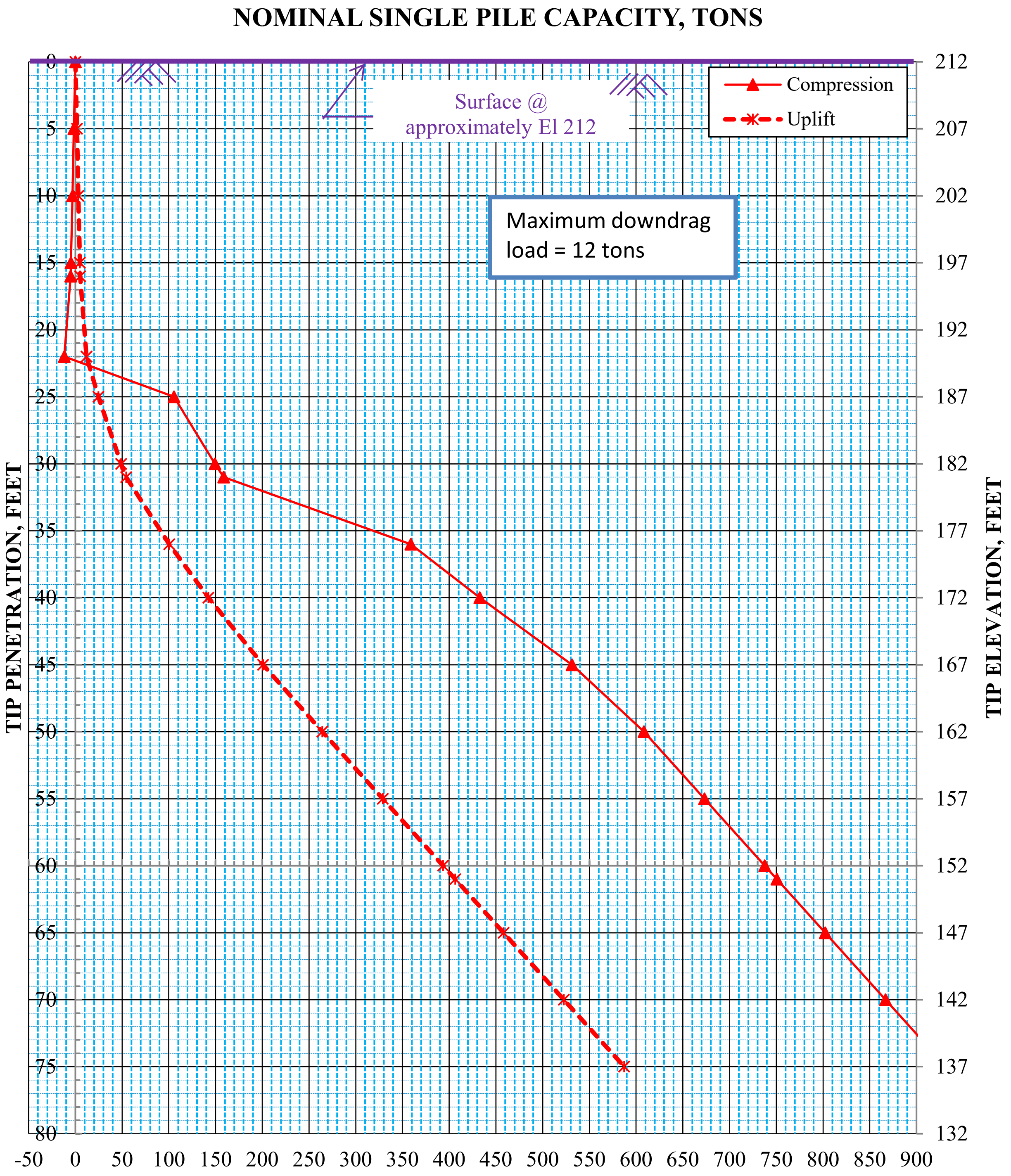


NOMINAL SINGLE PILE CAPACITY, TONS

Bent 2 (Intermediate Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. No downdrag

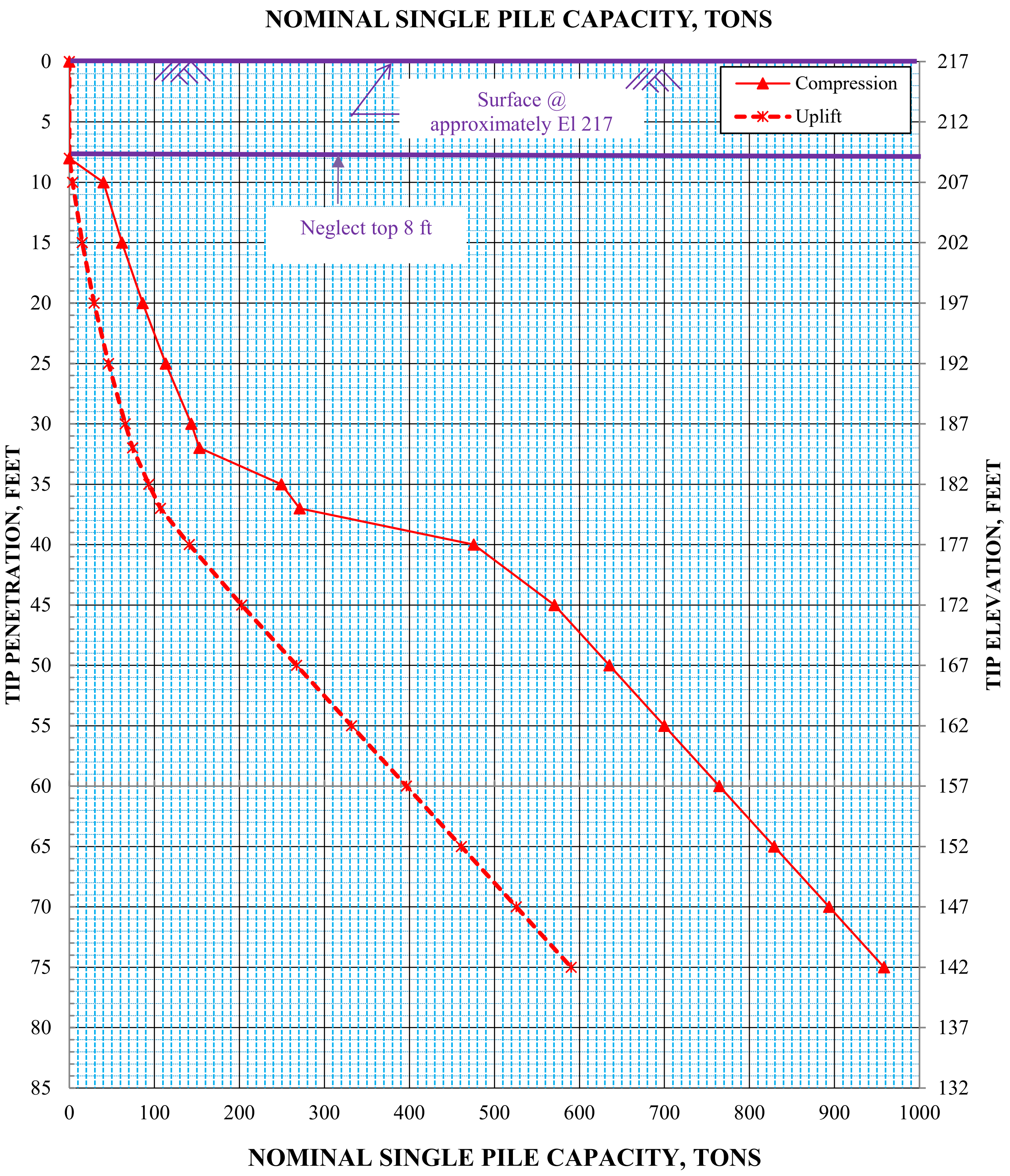
Grubbs, Hoskyn, Barton & Wyatt, LLC
Consulting Engineers



NOMINAL SINGLE PILE CAPACITY, TONS

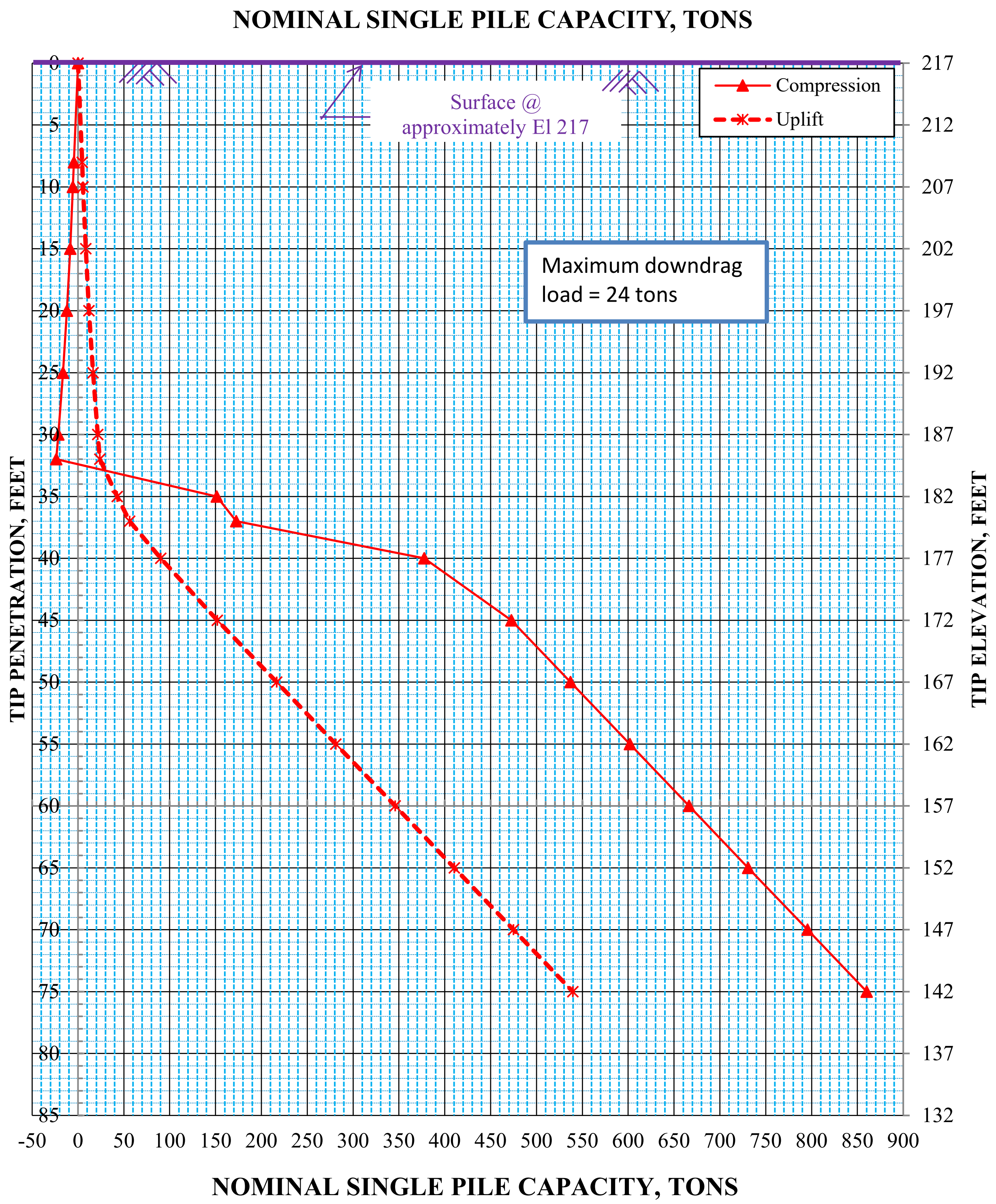
Bent 2 (Interior Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 190



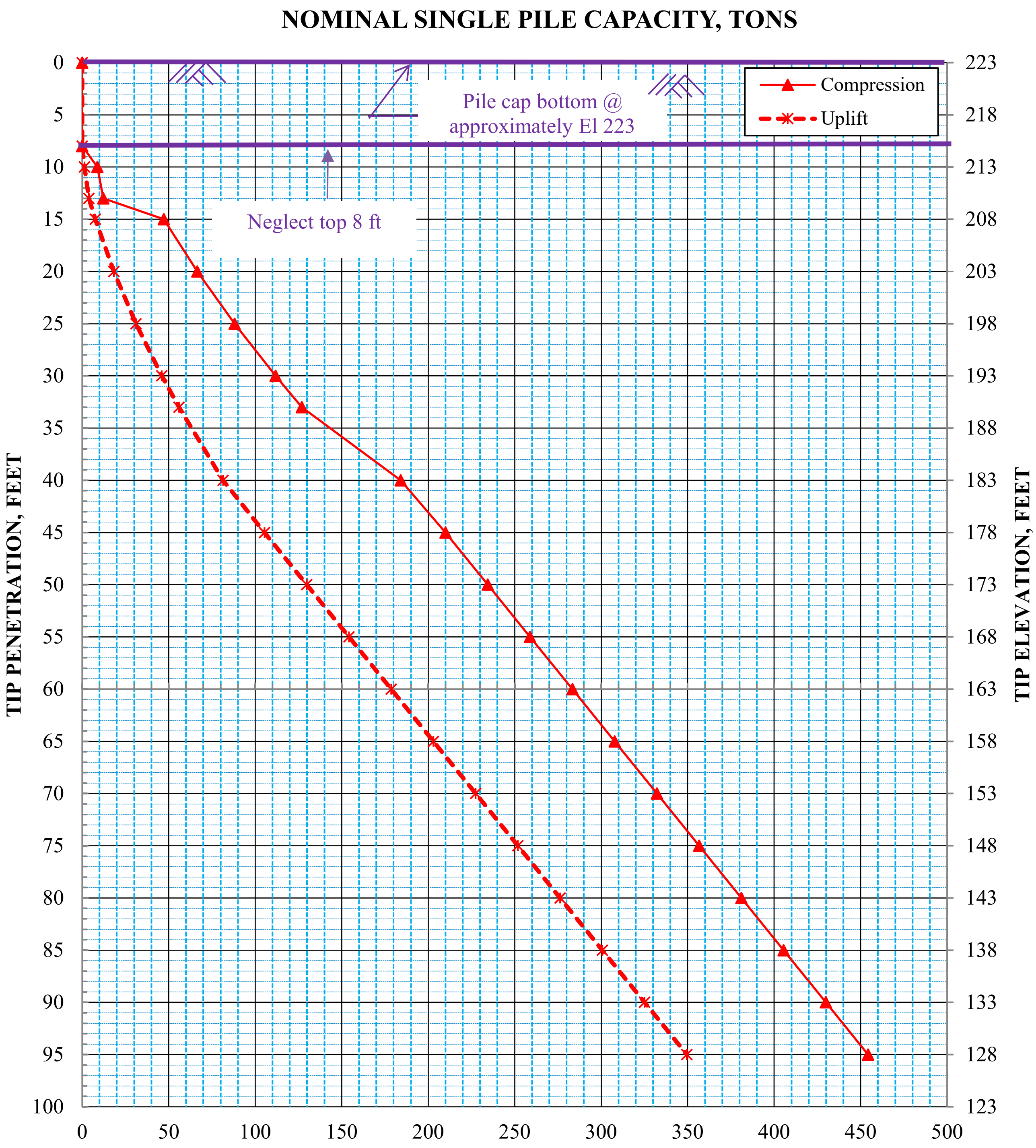
Bent 3 (Interior Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from channel bottom elevation
2. No downdrag



Bent 3 (Interior Bent)
24-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

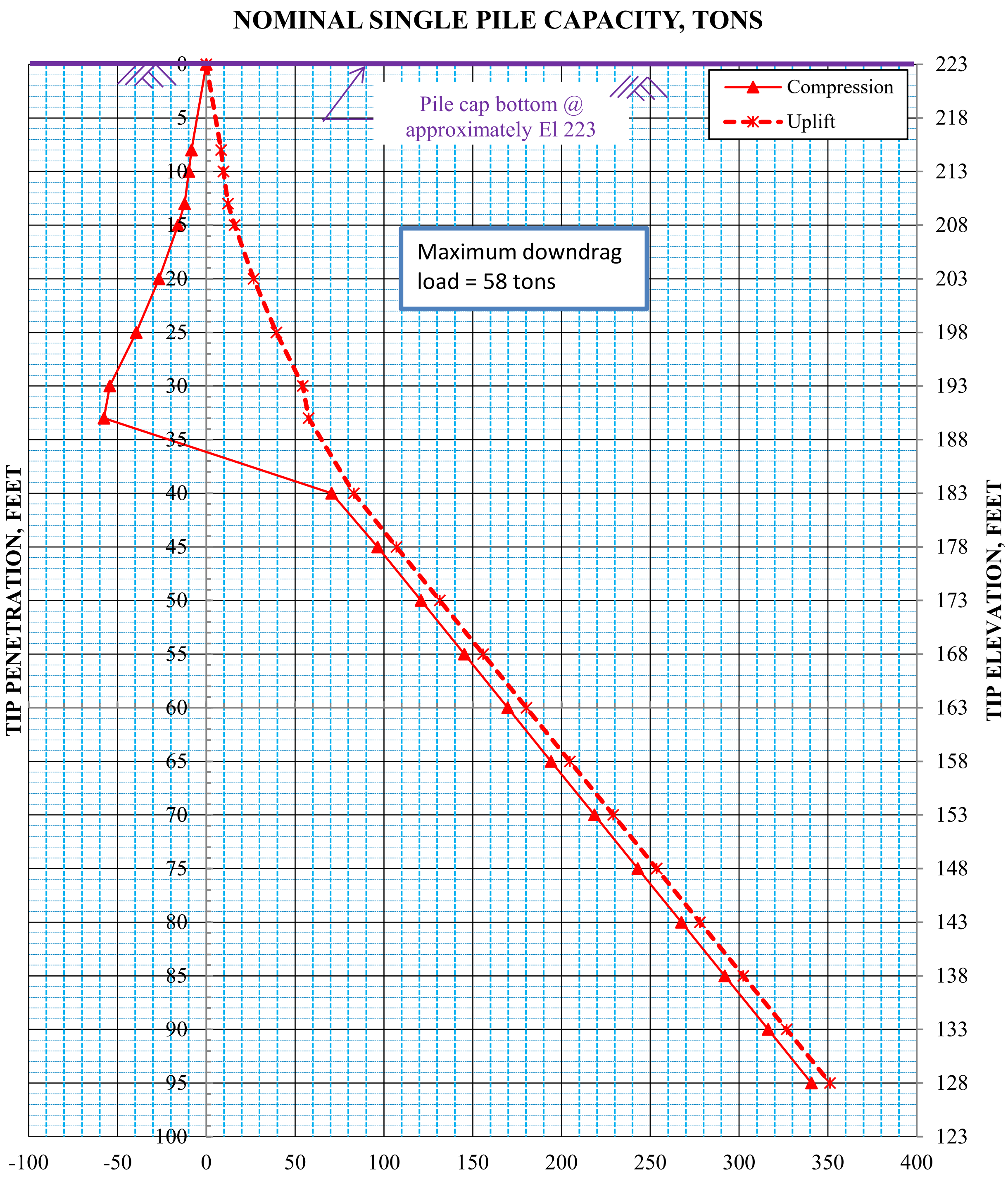
Notes: 1. Driven from channel bottom elevation
2. Downdrag to ±El 185



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (South Bridge End)
16-in.-Diameter Steel Shell
ARDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. No downdrag



NOMINAL SINGLE PILE CAPACITY, TONS

Bent 4 (South Bridge End)
16-in.-Diameter Steel Shell
ArDOT 101124 Hwy. 135 over Unnamed Creek
Poinsett County, Arkansas

Notes: 1. Driven from cap bottom elevation
2. Downdrag to ±El 190

APPENDIX F

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Unnamed Ditch

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading

Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff sandy CLAY fill**	**Firm silty CLAY**	**Medium dense silty fine SAND**	**Medium dense to dense fine SAND**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-3	3-6	6-19	19-40	40 and deeper
Approximate El, ft	223-220	220-217	217-204	204-183	below 183
Recommend soil type	Sand (Reese)	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	118	110	118	58	63
Cohesion (c), lbs per sq ft	0	750	0	0	0
Angle of internal friction (φ), °	32	0	32	35	38
Subgrade modulus (k), lbs per cu in.	25	100	60	90	125
Strain at 50% (EE50)	NA	0.01	NA	NA	NA

Note: Pile cap bottom at ±El 223

Seismic Loading with Liquefaction

Bent 1: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Stiff sandy CLAY fill**	**Firm silty CLAY**	**Medium dense silty fine SAND**	**Medium dense to dense fine SAND (liquefiable)**	**Dense to very dense fine to medium SAND**
Depth below pile cap bottom, ft	0-3	3-6	6-19	19-40	40 and deeper
Approximate El, ft	223-220	220-217	217-204	204-183	below 183
Recommend soil type	Sand (Reese)	Stiff clay without free water	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	118	110	118	58	63
Cohesion (c), lbs per sq ft	0	750	0	0	0
Angle of internal friction (φ), °	32	0	32	11	38
Subgrade modulus (k), lbs per cu in.	25	100	60	20	125
Strain at 50% (EE50)	NA	0.01	NA	NA	NA

Note: Pile cap bottom at ±El 223

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Unnamed Ditch

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND**	**Loose silty fine SAND**	**Medium dense fine SAND**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-9	9-16	16-31	31-60	60 and deeper
Approximate El, ft	212-203	203-196	196-181	196-152	below 152
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	48	60	65	68
Cohesion (c), lbs per sq ft	0	0	0	0	0
Angle of internal friction (φ), °	28	28	35	37	38
Subgrade modulus (k), lbs per cu in.	25	20	80	115	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA

Note: Ground surface at ±El 212

Seismic Loading with Liquefaction
Bent 2: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Loose silty fine SAND**	**Loose silty fine SAND (liquefiable)**	**Medium dense fine SAND (liquefiable)**	**Medium dense fine SAND**	**Dense fine to medium SAND**	**Dense fine to medium SAND**
Depth below pile cap bottom, ft	0-9	9-16	16-22	22-31	31-60	60 and deeper
Approximate El, ft	212-203	203-196	196-190	190-181	196-152	below 152
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	48	60	60	65	68
Cohesion (c), lbs per sq ft	0	0	0	0	0	0
Angle of internal friction (φ), °	28	8	11	35	37	38
Subgrade modulus (k), lbs per cu in.	25	20	20	80	115	125
Strain at 50% (EE50)	NA	NA	NA	NA	NA	NA

Note: Ground surface at ±El 212

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Unnamed Ditch

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Loose silty fine SAND	Medium dense fine SAND	Medium dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-8	8-32	32-37	37 and deeper
Approximate El, ft	217-209	209-185	185-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	56	60	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	28	32	35	38
Subgrade modulus (k), lbs per cu in.	25	50	80	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 217

Seismic Loading with Liquefaction
Bent 3: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	Loose silty fine SAND	Medium dense fine SAND (liquefiable)	Medium dense fine to medium SAND	Dense fine to medium SAND
Depth below pile cap bottom, ft	0-8	8-32	32-37	37 and deeper
Approximate El, ft	217-209	209-185	185-180	below 180
Recommend soil type	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	56	60	68
Cohesion (c), lbs per sq ft	0	0	0	0
Angle of internal friction (φ), °	28	8	35	38
Subgrade modulus (k), lbs per cu in.	25	20	80	125
Strain at 50% (EE50)	NA	NA	NA	NA

Note: Ground surface at ±El 217

SUMMARY OF LATERAL LOAD PARAMETERS
ARDOT 101124 Hwy. 135 over Unnamed Ditch

LOCATION: Poinsett County, Arkansas
GHBW JOB NUMBER: 23-031

Static Loading
Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Firm silty CLAY**	**Loose silty fine SAND**	**Medium dense silty fine SAND**	**Medium dense fine SAND**	**Medium dense to dense fine to medium SAND**
Depth below pile cap bottom, ft	0-8	8-13	13-30	30-33	33 and deeper
Approximate El, ft	223-215	215-210	210-193	193-190	below 190
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	110	60	60	63
Cohesion (c), lbs per sq ft	500	0	0	0	0
Angle of internal friction (φ), °	0	28	35	34	36
Subgrade modulus (k), lbs per cu in.	30	25	60	60	90
Strain at 50% (EE50)	0.02	NA	NA	NA	NA

Note: Pile cap bottom at ±El 223

Seismic Loading with Liquefaction
Bent 4: Recommended Parameters for Lateral Load Analyses Using LPILE©

Generalized Stratigraphy	**Firm silty CLAY**	**Loose silty fine SAND**	**Medium dense silty fine SAND**	**Medium dense fine SAND (liquefiable)**	**Medium dense to dense fine to medium SAND**
Depth below pile cap bottom, ft	0-8	8-13	13-30	30-33	33 and deeper
Approximate El, ft	223-215	215-210	210-193	193-190	below 190
Recommend soil type	Soft clay	Sand (Reese)	Sand (Reese)	Sand (Reese)	Sand (Reese)
Effective unit weight (γ), lbs per cu ft	110	110	60	60	63
Cohesion (c), lbs per sq ft	500	0	0	0	0
Angle of internal friction (φ), °	0	28	35	11	36
Subgrade modulus (k), lbs per cu in.	30	25	60	20	90
Strain at 50% (EE50)	0.02	NA	NA	NA	NA

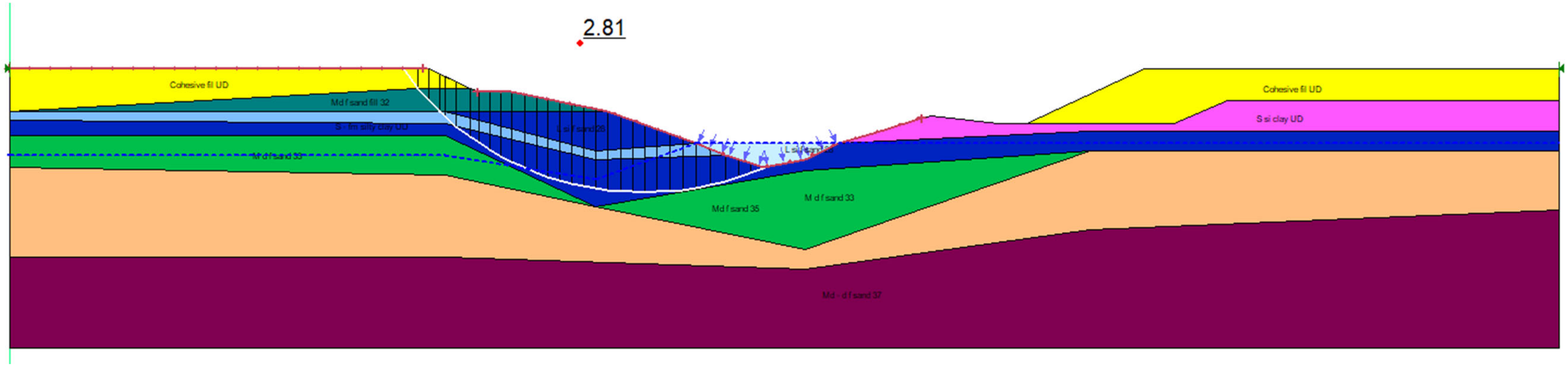
Note: Pile cap bottom at ±El 223

APPENDIX G

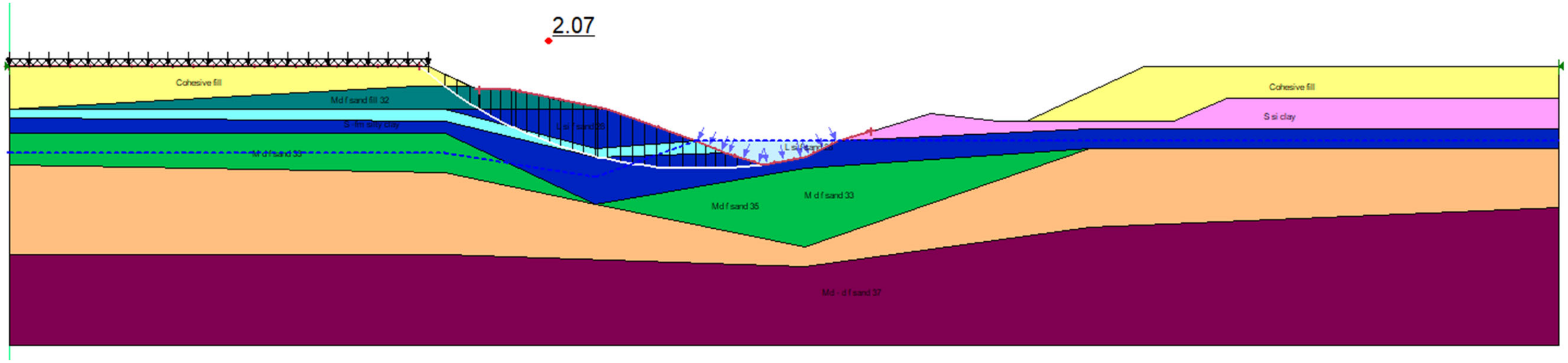
Summary of Stability Analysis Results
ARDOT 101124 Hwy 135 over Unnamed Ditch
GHBW Job No. 23-031
Poinsett County, Arkansas

	Design Loading Condition	**Calculated Minimum Factor of Safety**
South End Slope (Bent 1) (2H:1V)	End of Construction	2.81
	Long Term	2.07
	Rapid Drawdown from El 225 to El 212	1.12
	Seismic (k_h =A_S/2 = 0.5235)	0.87
	Lateral Spread	1.49
South Side Slope (Bent 1) (3H:1V)	End of Construction	4.54
	Long Term	2.32
	Rapid Drawdown from El 225 to Existing Grade	2.51
	Seismic (k_h =A_S/2 = 0. 5235	1.33
North End Slope (Bent 4) (2H:1V)	End of Construction	3.42
	Long Term	2.00
	Rapid Drawdown from El 225 to El 212	1.30
	Seismic (k_h =A_S/2 = 0. 5235)	0.88
	Lateral Spread	1.33
North Side Slope (Bent 4) (3H:1V)	End of Construction	4.10
	Long Term	2.16
	Rapid Drawdown from El 225 to Existing Grade	1.23
	Seismic (k_h =A_S/2 = 0. 5235	1.06

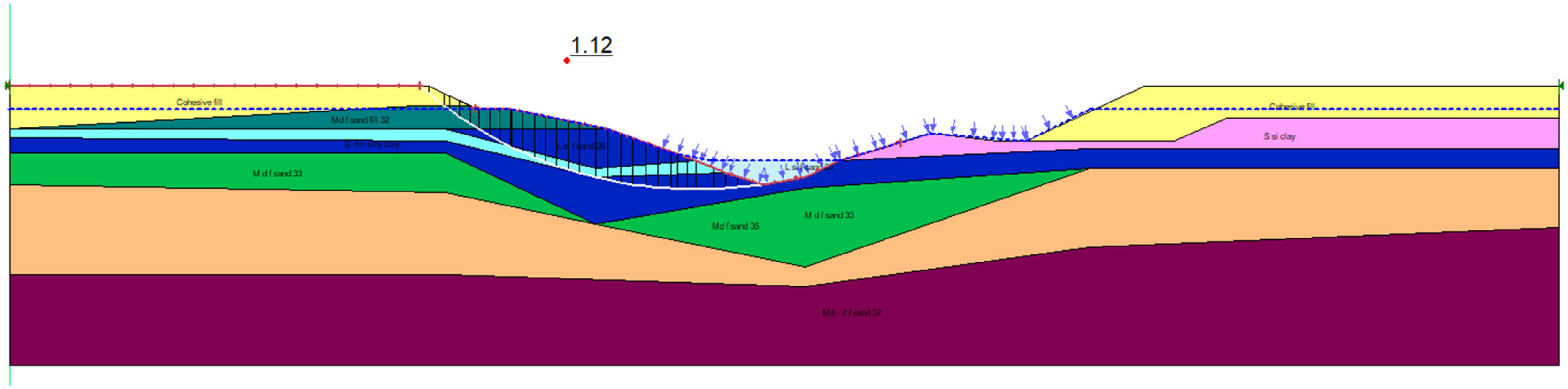
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – End of Construction
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

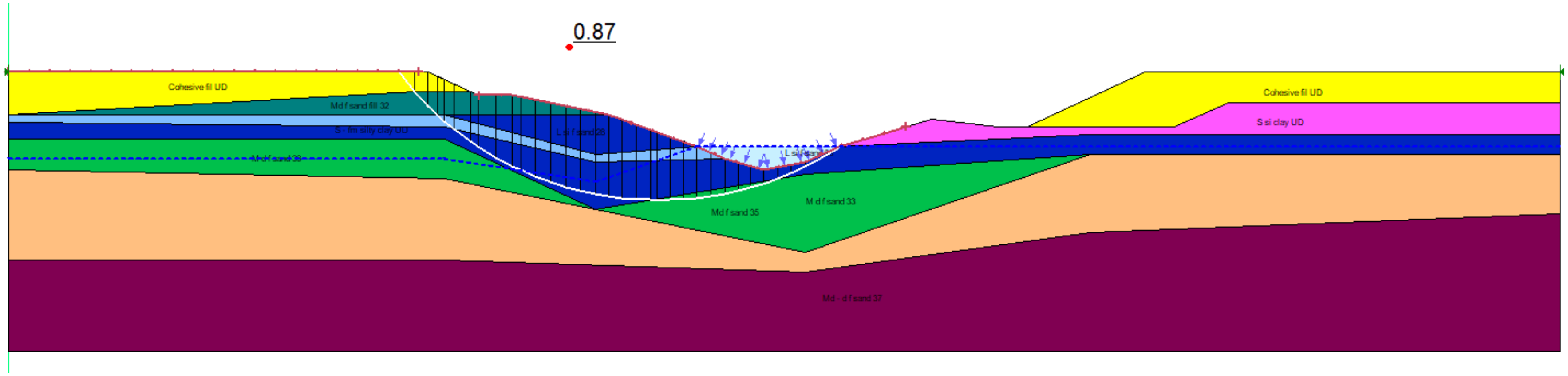


Results of Stability Analyses – Long Term Condition
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



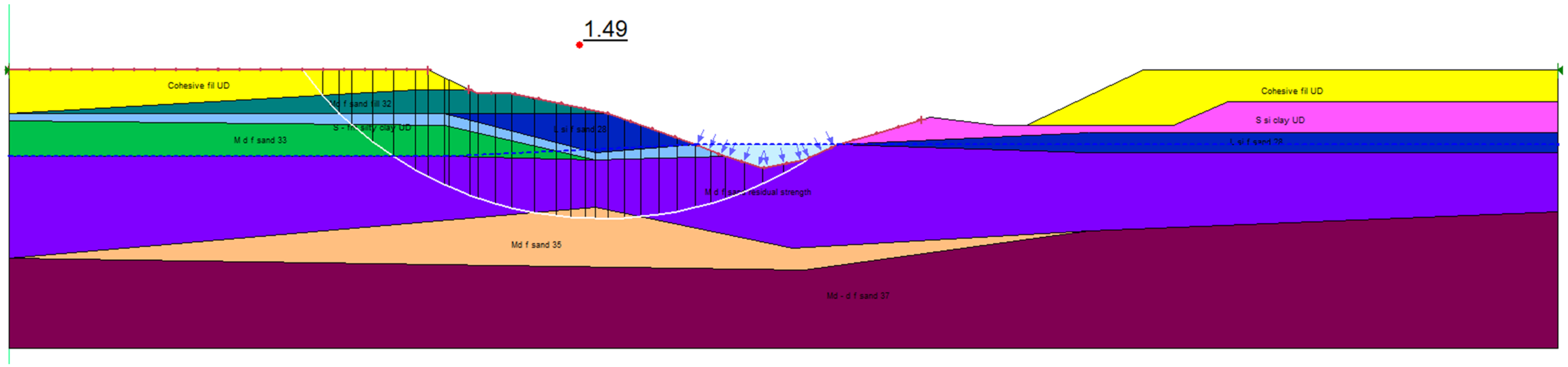
Results of Stability Analyses – Rapid Drawdown Condition from El 225 to El 212
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

Grubbs, Hoskyn,
Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



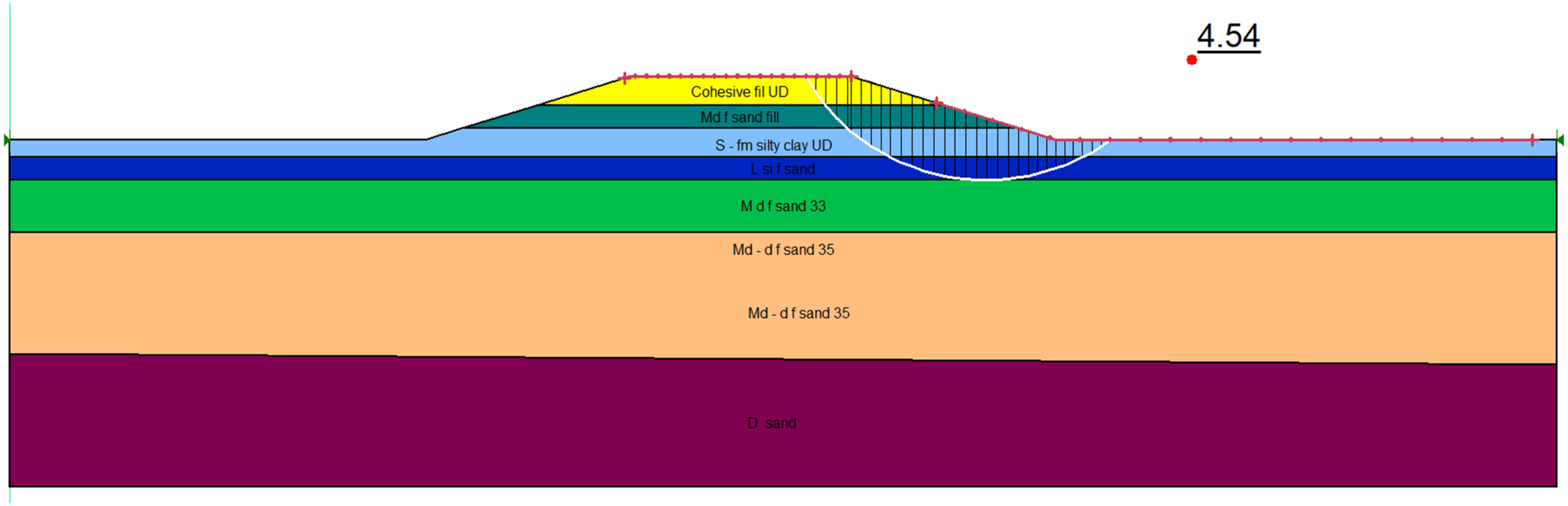
Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.5235$)
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

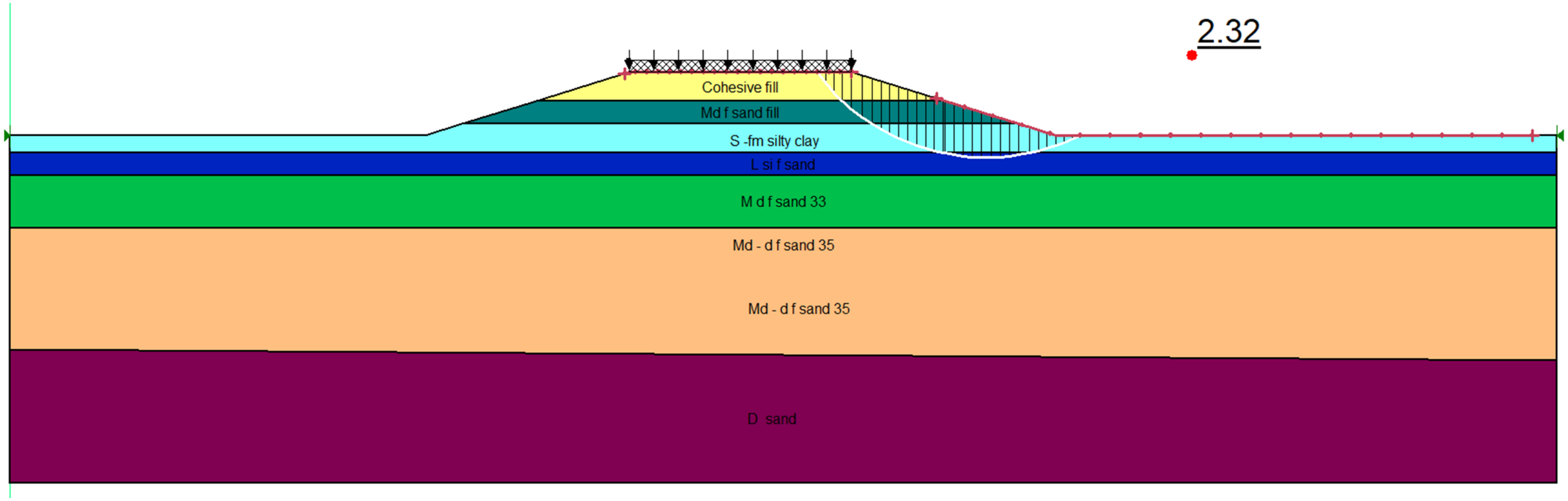


Results of Stability Analyses – Lateral Spread
Bent 1 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

1

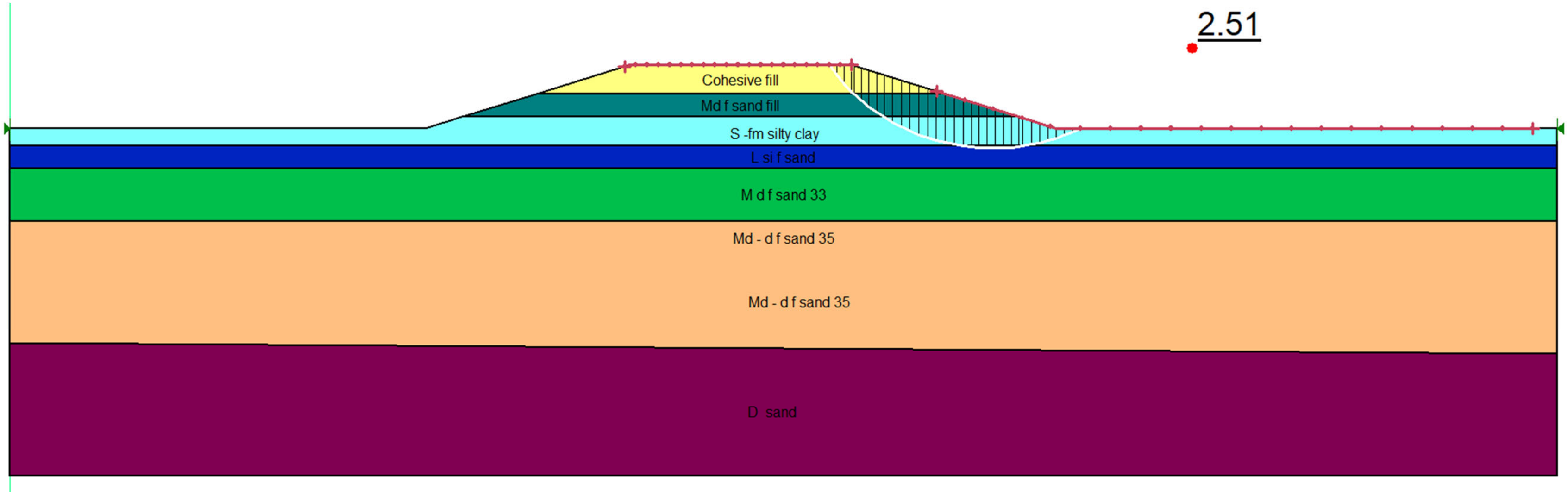


Results of Stability Analyses – End of Construction
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

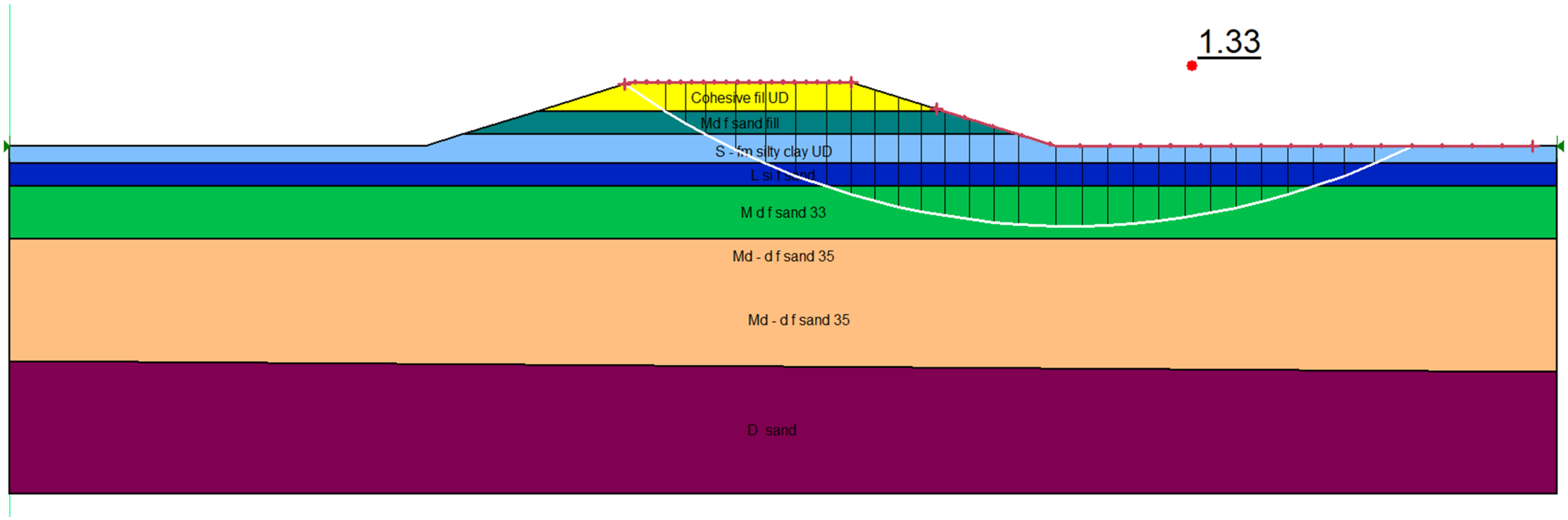


Results of Stability Analyses – Long Term Condition
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

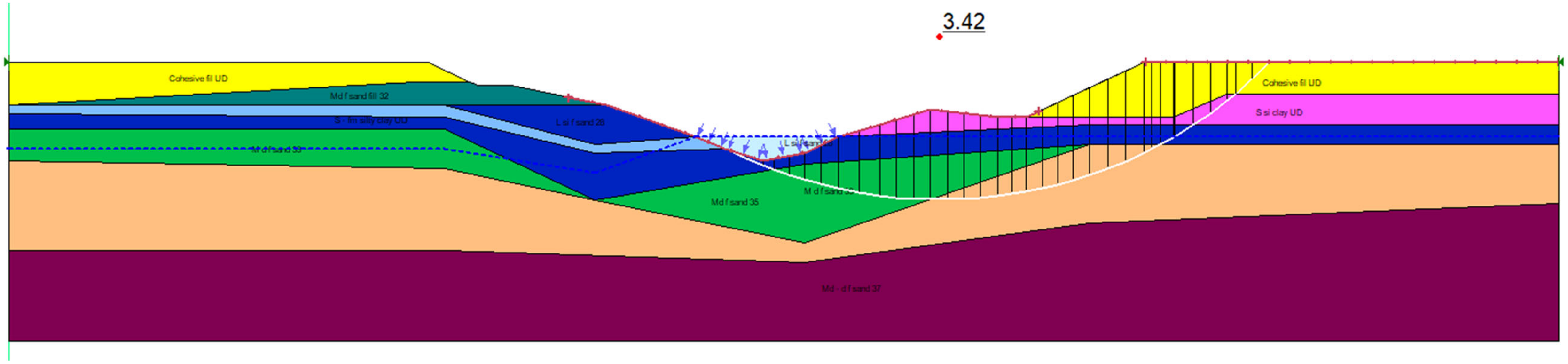
Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



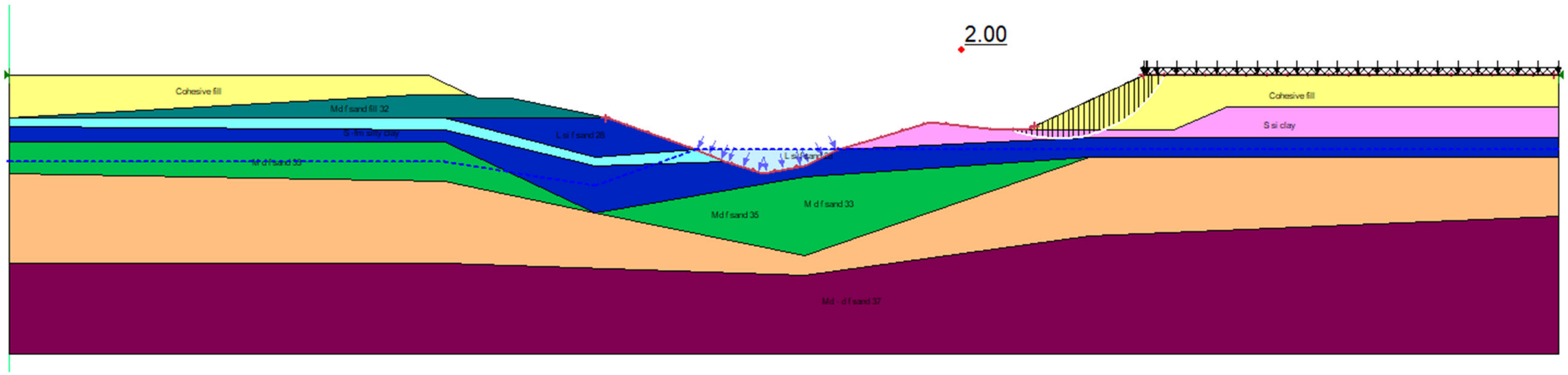
Results of Stability Analyses – Rapid Drawdown El 225 to Existing Grade
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



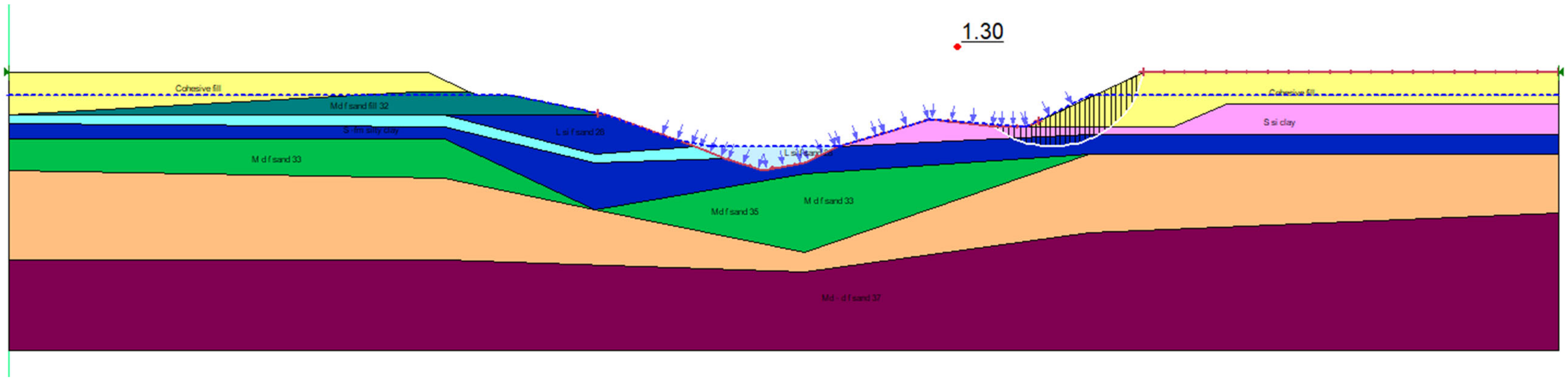
Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.5235)
Bent 1 Side Slope
3H:1V Slope, H=11 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



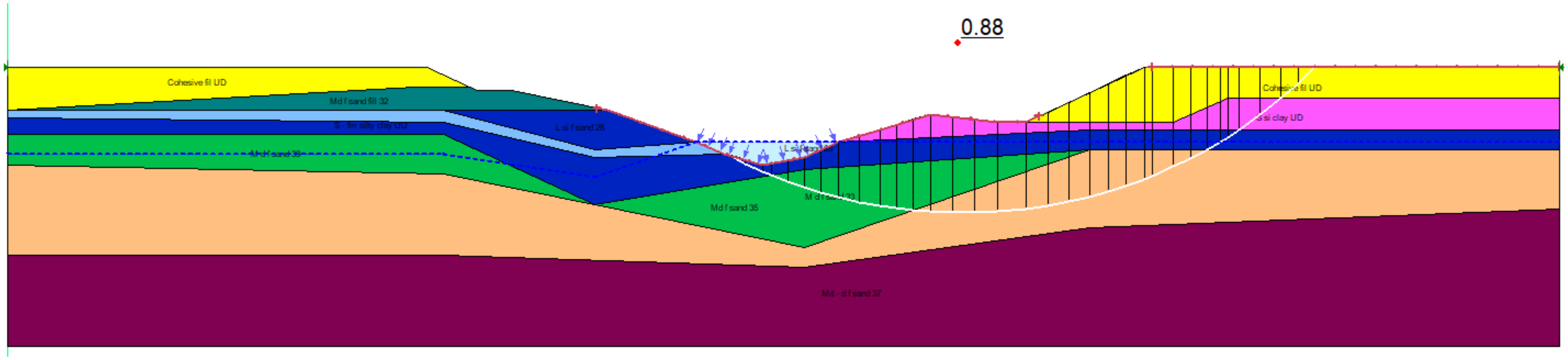
Results of Stability Analyses – End of Construction
Bent 4 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



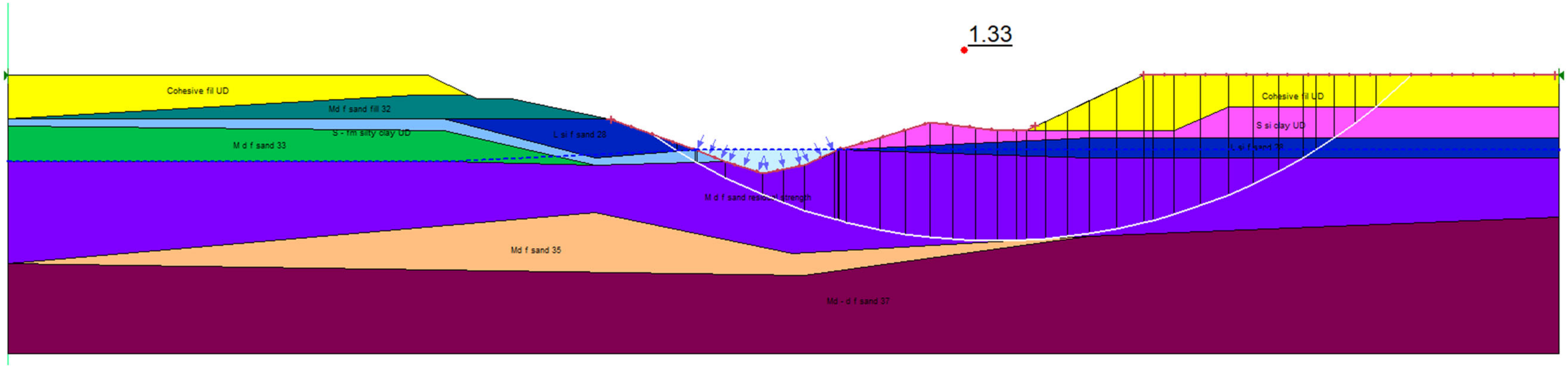
Results of Stability Analyses – Long Term Condition
Bent 4 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



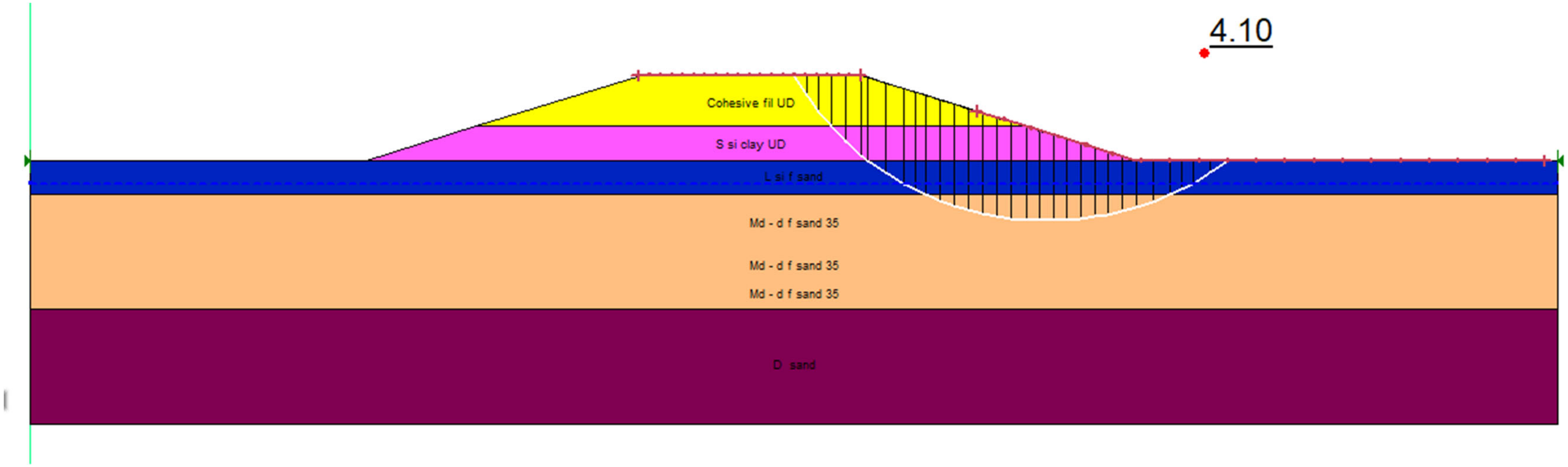
Results of Stability Analyses – Rapid Drawdown Condition, El 225 to El 212
Bent 4 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



Results of Stability Analyses – Seismic Condition (k_h = A_S /2= 0.5235)
Bent 4 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

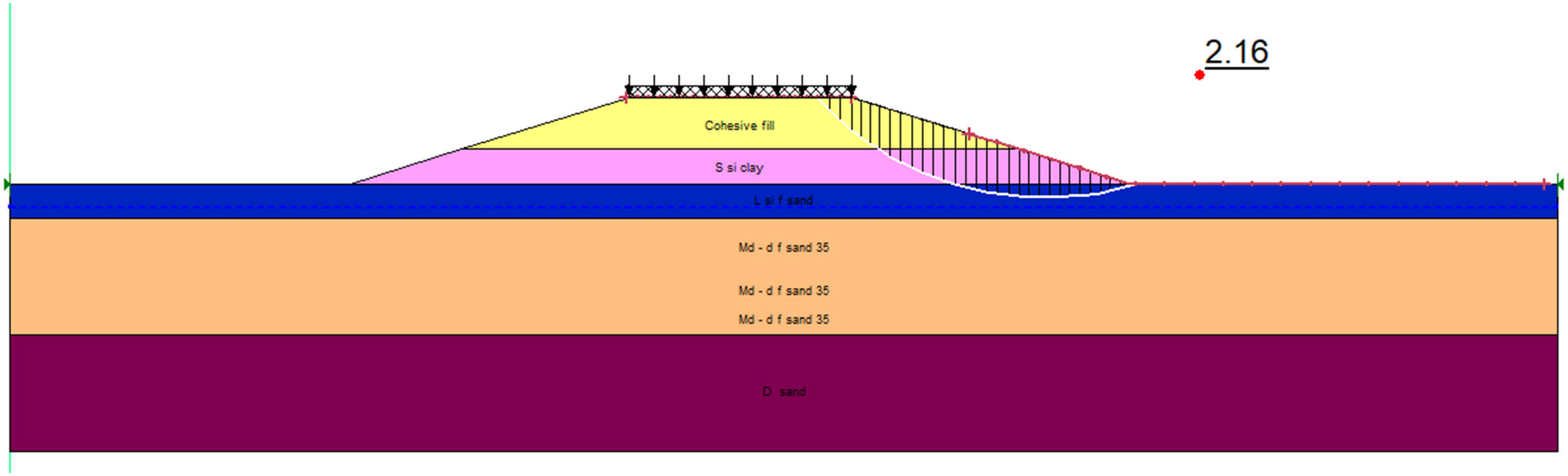


Results of Stability Analyses – Lateral Spread
Bent 4 End Slope
2H:1V Slope, H=25 ft ±
23-031 – ArDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

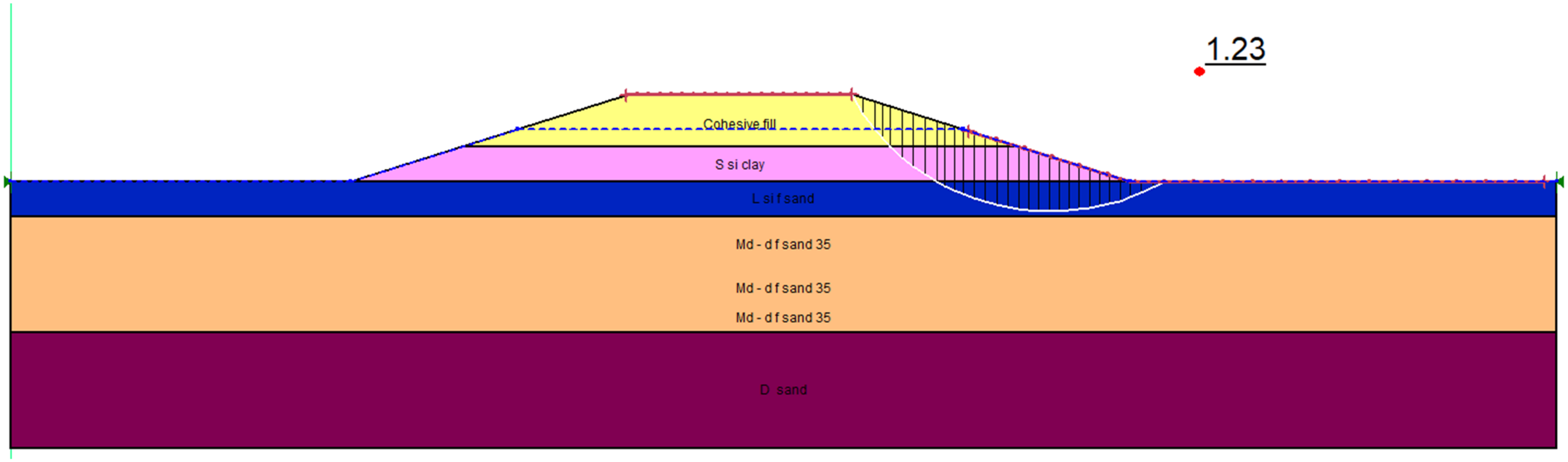


Results of Stability Analyses – End of Construction
Bent 4 Side Slope
3H:1V Slope, H=15 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company

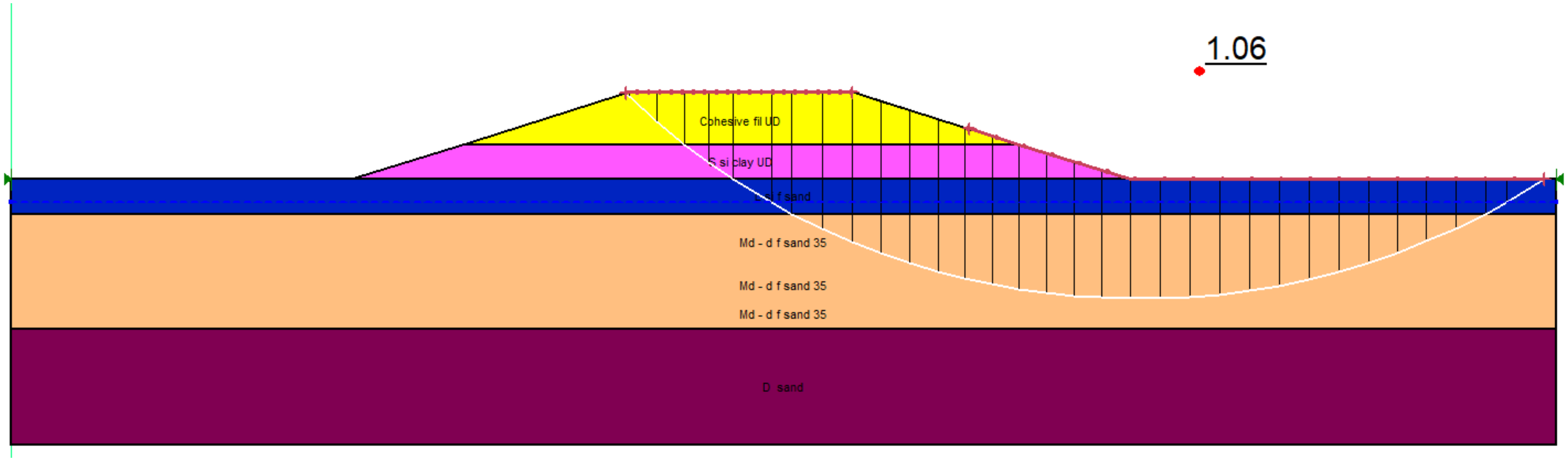


Results of Stability Analyses – Long Term Condition
Bent 4 Side Slope
3H:1V Slope, H=15 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch



Results of Stability Analyses – Rapid Drawdown Condition, El 225 to Existing Grade
Bent 4 Side Slope
3H:1V Slope, H=15 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company



Results of Stability Analyses – Seismic Condition ($k_h = A_S /2= 0.5235$)
Bent 4 Side Slope
3H:1V Slope, H=15 ft ±
23-031 – ARDOT Job No. 101124 – Hwy. 35 over Unnamed Ditch

APPENDIX H

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SPECIAL PROVISION

JOB 100955

WOVEN GEOTEXTILE FABRIC FOR SUBGRADE REINFORCEMENT

Description: This item shall consist of furnishing and installing a woven geotextile for subgrade reinforcement system in close conformity with the lines, grades and dimensions as established by the Engineer.

Materials: Geotextile fabric shall be woven synthetic fiber fabric meeting the following requirements:

The geotextile structure shall remain dimensionally stable under construction stresses and have a high resistance to damage during construction, to ultraviolet degradation and to all forms of chemical and biological degradation encountered in the soil being reinforced.

Provide a woven geotextile with a minimum tensile strength of 1500 lbs/ft in the Cross Machine Direction (CD) at 5 percent strain and minimum tensile strength of 1500 lbs/ft in the Machine Direction (MD) at 5 percent strain when tested in accordance with ASTM D4595. The geotextile fabric shall also meet the requirements of Type 10 geotextile fabric as described in Section 625 of the Standard Specifications for Highway Construction 2014 Edition.

Identify, store and handle geotextile according to ASTM D4873. Limit geotextile fabric exposure to ultraviolet radiation to less than 10 days.

The Contractor shall furnish to the Engineer a production certification that the geotextile supplied meets the respective criteria set forth in these specifications. The certification shall state the name of the Manufacturer, product name, style number, chemical composition of the filaments, ribs, or yarns, and other information to fully describe the fabric. The Manufacturer shall have an on-site GAI-LAP accredited laboratory used for their quality control program. The production lot number must be provided with the supplied material. Quality control test results shall be provided upon request by the Engineer. Independent third party test data used to identify values for creep, durability and installation damage must be included with the production certification.

Construction Methods: The woven geotextile fabric shall be installed at locations shown in the plans or as directed by the Engineer and shall follow Manufacturer’s installation requirements. The woven geotextile fabric shall be oriented such that the roll length is oriented parallel to the centerline. Adjacent rolls shall be overlapped a minimum of 2 feet and shall be tied together using pins or staples, unless otherwise recommended by the Manufacturer. Care shall be taken to ensure that the geotextile fabric sections do not separate at longitudinal or transverse laps during construction. The placement of the geotextile fabric around corners may require cutting and diagonal lapping.

The geotextile fabric shall be pinned at the beginning of the roll but shall be left free elsewhere to relieve wrinkles or folds in the material during the placement of stone backfill or base material. Sections of geotextile fabric which are damaged by construction activity shall be repaired or replaced at the Contractor's expense.

Rubber-tired vehicles shall be driven at speeds less than 10 mph and in straight paths over the fabric. A minimum fill thickness of 6 in. is required prior to operation of tracked construction equipment over the fabric. Tracked construction equipment shall not be operated directly upon fabric.

Method of Measurement: Woven Geotextile Fabric will be measured by the square yard of horizontal surface area covered by the material. No measurement will be made for lapping of the material required by the plans or required by the Manufacturers installation requirements.

Basis of Payment: Work completed and accepted and measured as provided will be paid for at the contract unit price bid per square yard for Woven Geotextile Fabric, which price shall be full compensation for furnishing, storing, and placing materials; for lapping and/or splicing; for necessary repairs; and for all labor, equipment, tools, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	**Pay Unit**
Woven Geotextile Fabric	Square Yard

APPENDIX I

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

Description. This Special Provision shall be supplementary to Section 210, Excavation and Embankment, of the Standard Specifications, Edition of 2014. The following sentence shall be added after the last sentence of the first paragraph in Subsection 210.09 of the Standard Specifications, “The Contractor shall be responsible for maintaining the stability of all embankment materials incorporated into the project.” This special provision shall apply to all compacted embankment within 100 ft of the bridge end slope intercept.

Highly plastic or predominantly silty soils shall not be used in embankments without chemical treatment. All embankment material, including material excavated from cut areas within the project limits, placed by the Contractor shall be evaluated in accordance with Table 1. Chemical treatment required by Table 1 for material placed by the Contractor shall be provided at no additional cost to the Department. Blending of multiple soil materials will not be allowed. Cut material not utilized on the project shall be removed from the project limits at no additional cost to the Department.

Table 1. Treatment requirements for Compacted Embankment

% Passing #200 Sieve	Plasticity Index	Treatment
≤ 50%	No Limitations	4% Portland Cement
>50%	PI ≤ 9	4% Portland Cement
>50%	9 < PI ≤ 25	None Required
>50%	25 < PI ≤ 35	4% Quicklime (dry)
>50%	PI > 35	6% Quicklime (dry)

Soils with ≤ 50 percent passing the #200 sieve shall not be used in the outer 18 in. of embankments without approved cement treatment.

The quantity of chemical treatment required by this Special Provision shall be calculated by multiplying the percent of treatment required in Table 1 by the Maximum Dry Unit Weight of the material being treated and the volume of soil being treated. Layer thickness for this calculation shall be the loose, uncompacted lift thickness.

Example: Maximum Dry Unit Weight = 110 lb/cf
Treatment Required = 4%
Volume of Soil = 12,000 cf

(110 lb/cf × (4/100) × 12,000 cf) / (2000 lb/ton) = 26.4 Tons

Quality Control and Acceptance. The Contractor shall perform quality control and acceptance sampling and testing of all embankment material in accordance with Subsection 210.02 of the Standard Specifications. Additionally, the Contractor shall perform testing for gradation and

ARKANSAS DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION

JOB NO. 101124

COMPACTED COHESIVE EMBANKMENT

plasticity index for all embankment material in accordance with Section 306 of the Standard Specifications except that the size of the standard lot will be 3000 cubic yards. If quicklime is utilized, maximum laboratory density and optimum moisture shall be determined from a field sample obtained after initial mixing. If cement is utilized, maximum laboratory density and optimum moisture shall be determined in accordance with AASHTO T 134-19. Additional testing may be required when deemed necessary by the Engineer based on visual examination of the material.

Construction Requirements. Spreading and mixing of material shall be performed at its final location. The spreading and mixing procedures shall thoroughly and uniformly disperse the lime or cement additive into the soil. Chemical treatment shall be mixed and processed throughout the entire depth of each lift. Mixing shall be accomplished by means of rotary tillers, pulvimixers, or mechanical equipment as approved by the Engineer. Any procedure that results in excessive loss of lime or that does not achieve the desired results shall be immediately discontinued. Acceptance of material shall be in accordance with the Quality Control and Acceptance section of this special provision for in- place material.

Method of Measurement. All embankments constructed as described above will be measured as Compacted Embankment in accordance with Section 210 of the Standard Specifications and shall also include all labor, material, and equipment for furnishing, hauling, placing, and applying lime or cement additive; for pulverizing, watering, mixing, and compacting the additive to modify soil to meet the requirements herein; for performing quality control and acceptance sampling and testing; and for all labor, equipment, tools, and incidentals necessary to complete and maintain the work. Treatment of materials used for construction of embankments will not be paid for separately, but full compensation will be considered included in the contract price bid for Compacted Embankment.

Basis of Payment. The basis of payment shall be in accordance with Subsection 210.13(c) of the Standard Specifications and shall include all cost associated with furnishing, hauling, placing, and processing chemical treatments in soils at locations required by this Special Provision.

Payment will be made under:

Pay Item	**Pay Unit**
Compacted Embankment	Cubic Yard

APPENDIX J

WEAP ANALYSES - STEEL SHELL PILES

Project: 101124 - Hwy 135

Poinsett County, Arkansas

GHBW Project No: 23-031

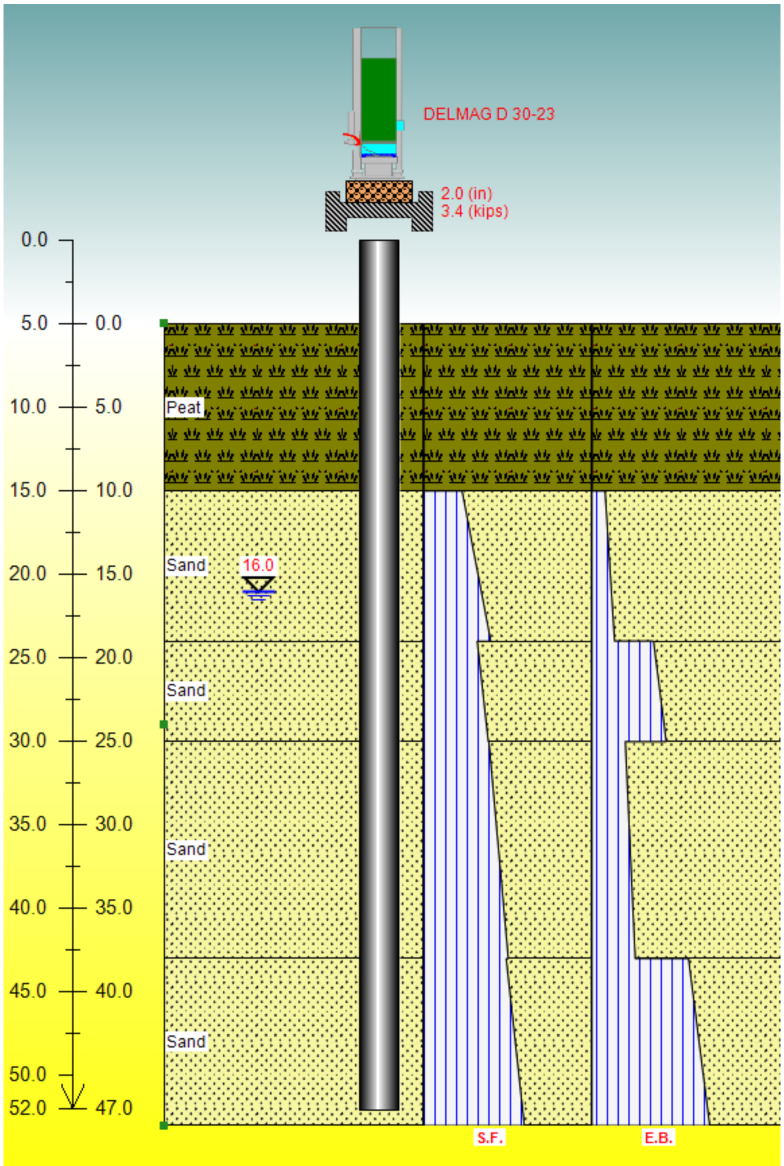
Bridge	Bent	Pile Diameter (in)	Wall Thickness (in)	Min Ult Capacity for Axial Resistance (tons)	Pile Cap El.	Min Tip El.	Pile Length (ft)	Min Hammer Energy (ft-kip)	Max Comp Stress, ksi
7 - Unnamed Ditch	1	16	0.75	266	223	176	47	74	35.1
	2	24	0.50	360	212	176	36	74	34.5
	3	24	0.50	361	217	175	42	91	38.3
	4	16	0.75	250	223	167	56	74	28.1

ArDOT 101124 Hwy 135 over Unnamed Ditch

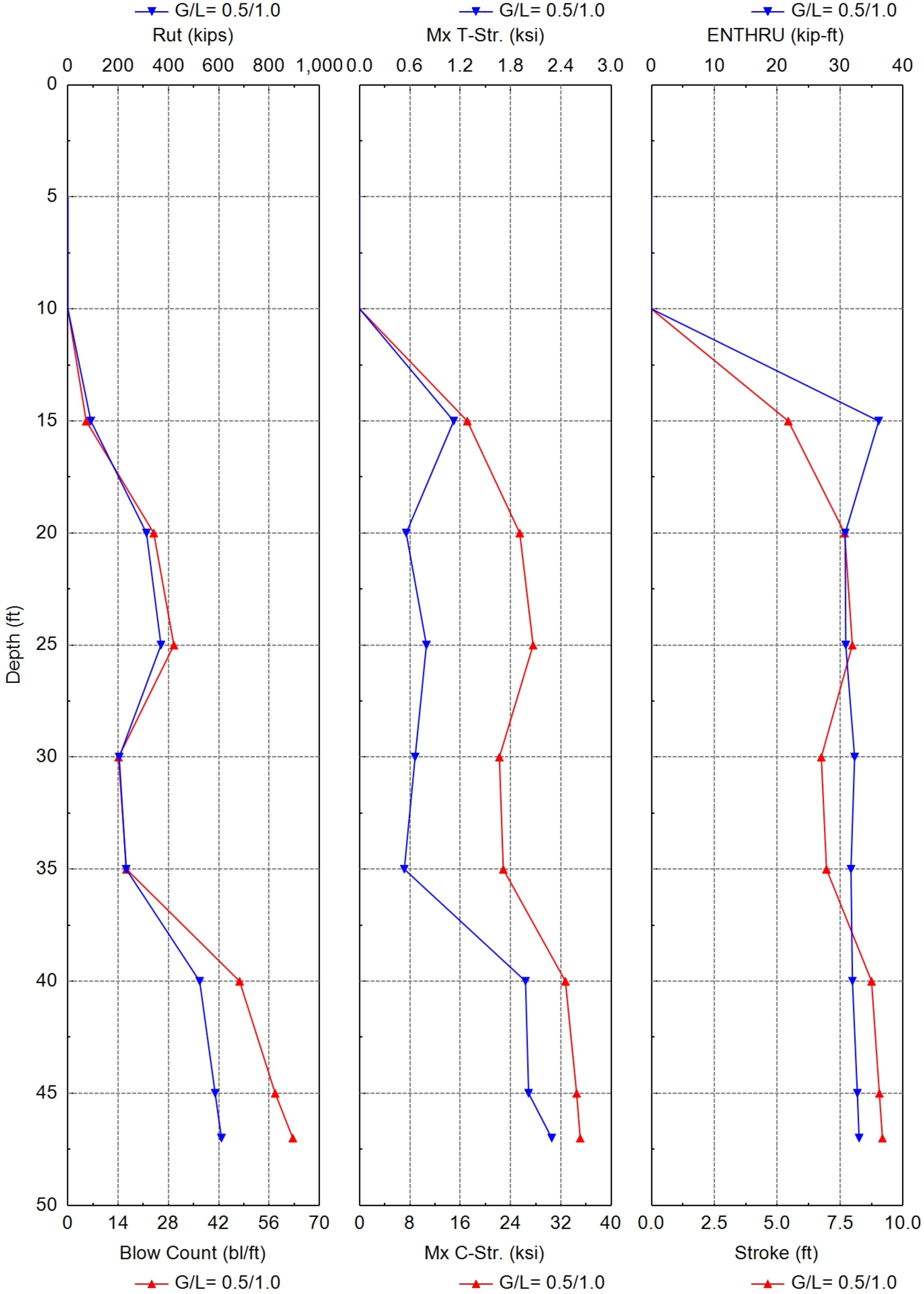
Bent 1

16-in-diameter Steel Shell Pile

Delmag D30-23



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company


G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.6
1.2
1.8
2.4
3.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
10
20
30
40
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
0
14
28
42
56
70
Blow Count (bl/ft)
G/L= 0.5/1.0
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.18	0.0	D 30-23
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.18	0.0	D 30-23
15.0	91.5	5.9	85.6	5.1	17.112	1.122	5.44	36.1	D 30-23
20.0	312.5	13.5	299.0	23.9	25.459	0.555	7.68	30.8	D 30-23
25.0	370.2	21.1	349.0	29.5	27.584	0.796	7.99	30.9	D 30-23
30.0	204.0	29.9	174.1	14.1	22.215	0.661	6.75	32.3	D 30-23
35.0	231.8	39.6	192.1	16.3	22.839	0.533	6.96	31.7	D 30-23
40.0	524.1	50.2	473.8	47.8	32.720	1.978	8.75	32.0	D 30-23
45.0	585.6	61.8	523.9	57.7	34.507	2.015	9.06	32.7	D 30-23
47.0	610.6	66.7	543.9	62.6	35.076	2.291	9.18	33.0	D 30-23

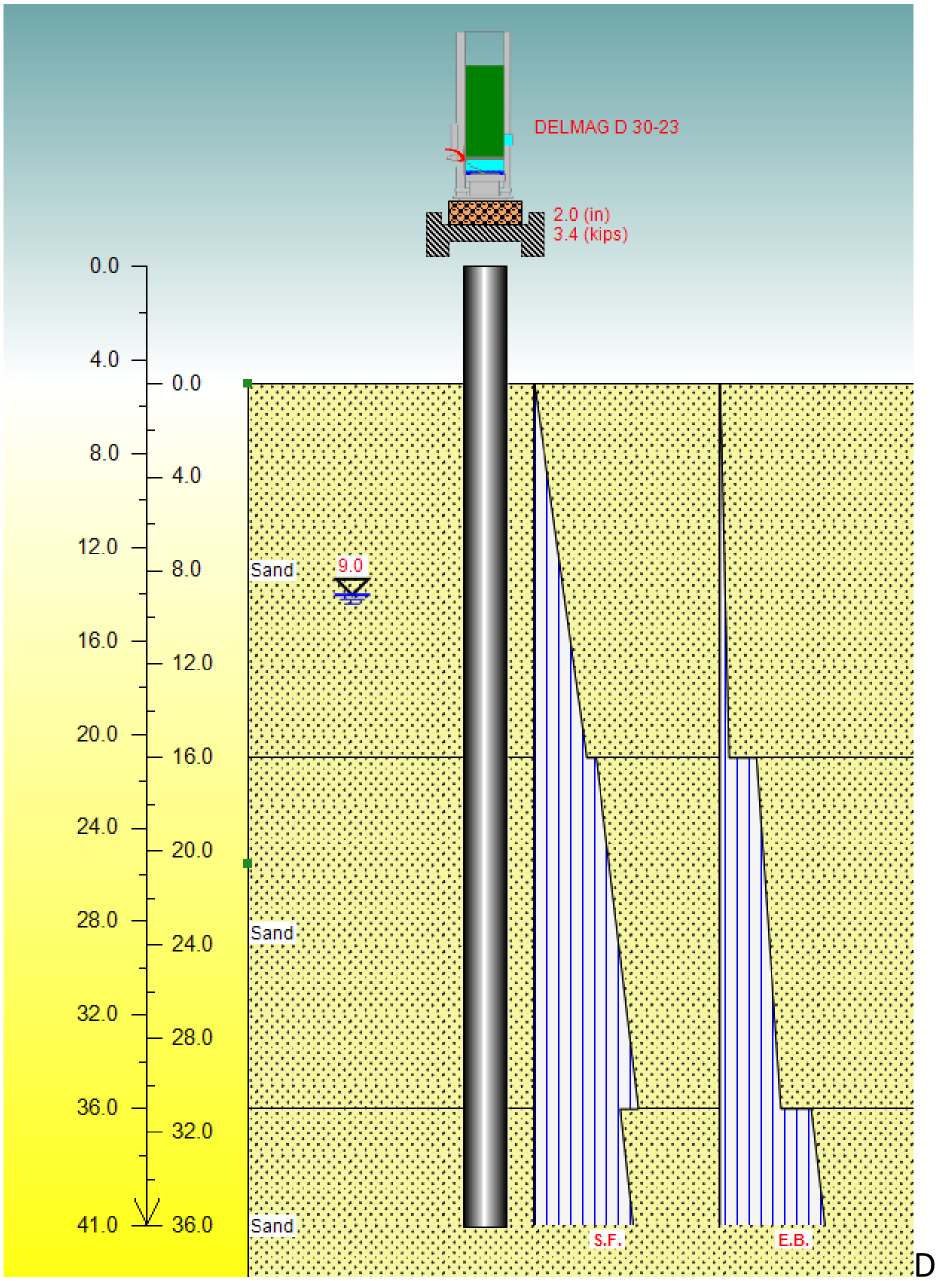
Total driving time: 23 minutes; Total Number of Blows: 950 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Unnamed Ditch

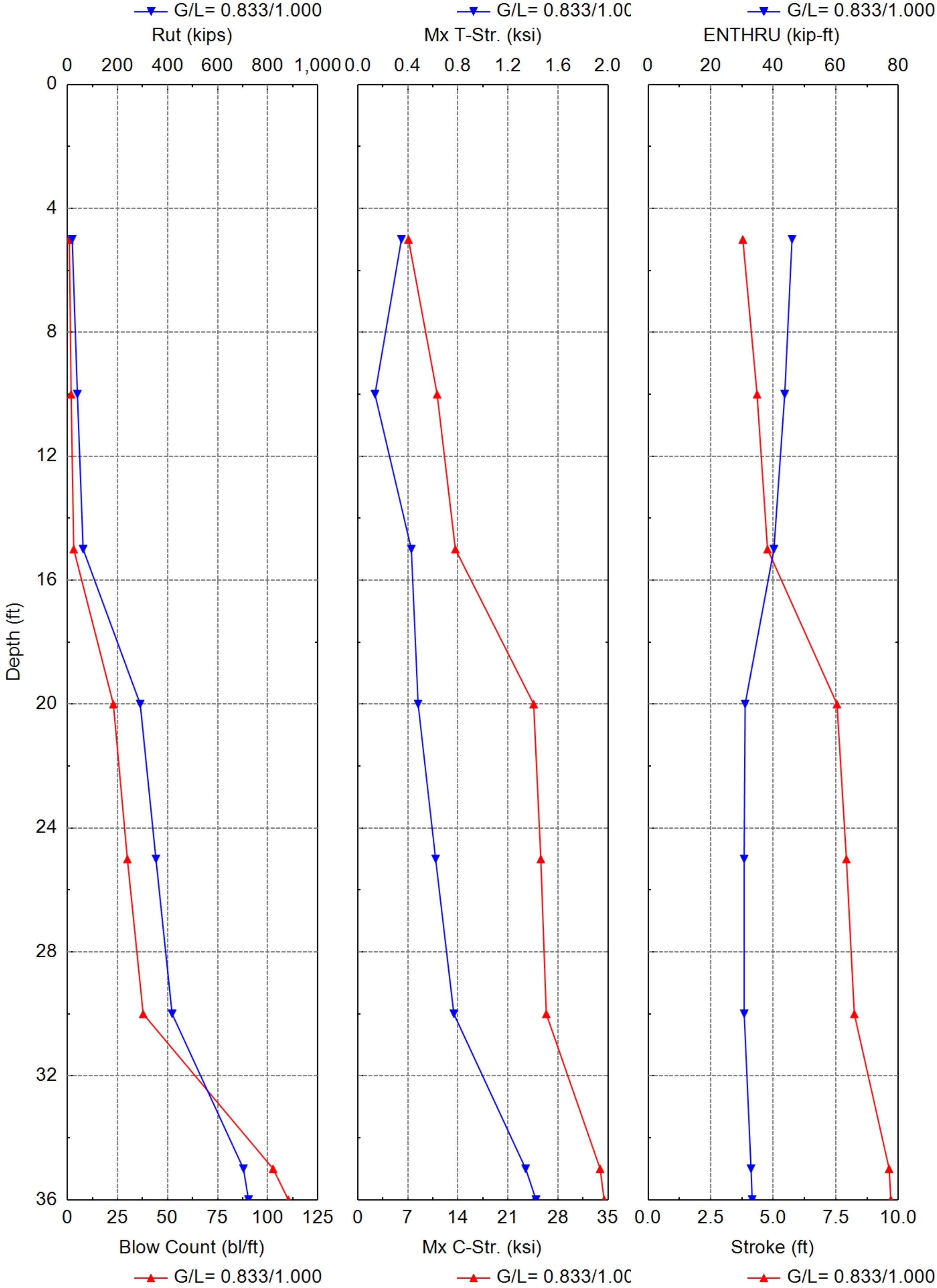
Bent 2

24-in-diameter Steel Shell Pile

Delmag D30-23



Grubbs, Hoskyn, Barton & Wyatt, LLC
CONSULTING ENGINEERS
A UES Company


G/L= 0.833/1.000
Rut (kips)
0 200 400 600 800 1,000
G/L= 0.833/1.00
Mx T-Str. (ksi)
0.0 0.4 0.8 1.2 1.6 2.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0 20 40 60 80
Depth (ft)
0 4 8 12 16 20 24 28 32 36
0 25 50 75 100 125
Blow Count (bl/ft)
G/L= 0.833/1.000
0 7 14 21 28 35
Mx C-Str. (ksi)
G/L= 0.833/1.00
0.0 2.5 5.0 7.5 10.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.7	1.1	17.6	0.9	7.085	0.348	3.79	46.0	D 30-23
10.0	39.5	4.2	35.3	1.8	11.096	0.137	4.35	43.7	D 30-23
15.0	62.4	9.5	52.9	3.1	13.644	0.428	4.77	40.2	D 30-23
20.0	290.5	17.7	272.8	22.9	24.598	0.482	7.55	31.0	D 30-23
25.0	353.3	28.0	325.3	29.9	25.598	0.622	7.92	30.7	D 30-23
30.0	418.0	40.1	377.9	37.8	26.358	0.768	8.24	30.7	D 30-23
35.0	703.2	52.2	651.1	102.7	33.896	1.342	9.63	32.9	D 30-23
36.0	723.3	54.7	668.6	110.3	34.453	1.425	9.70	33.3	D 30-23

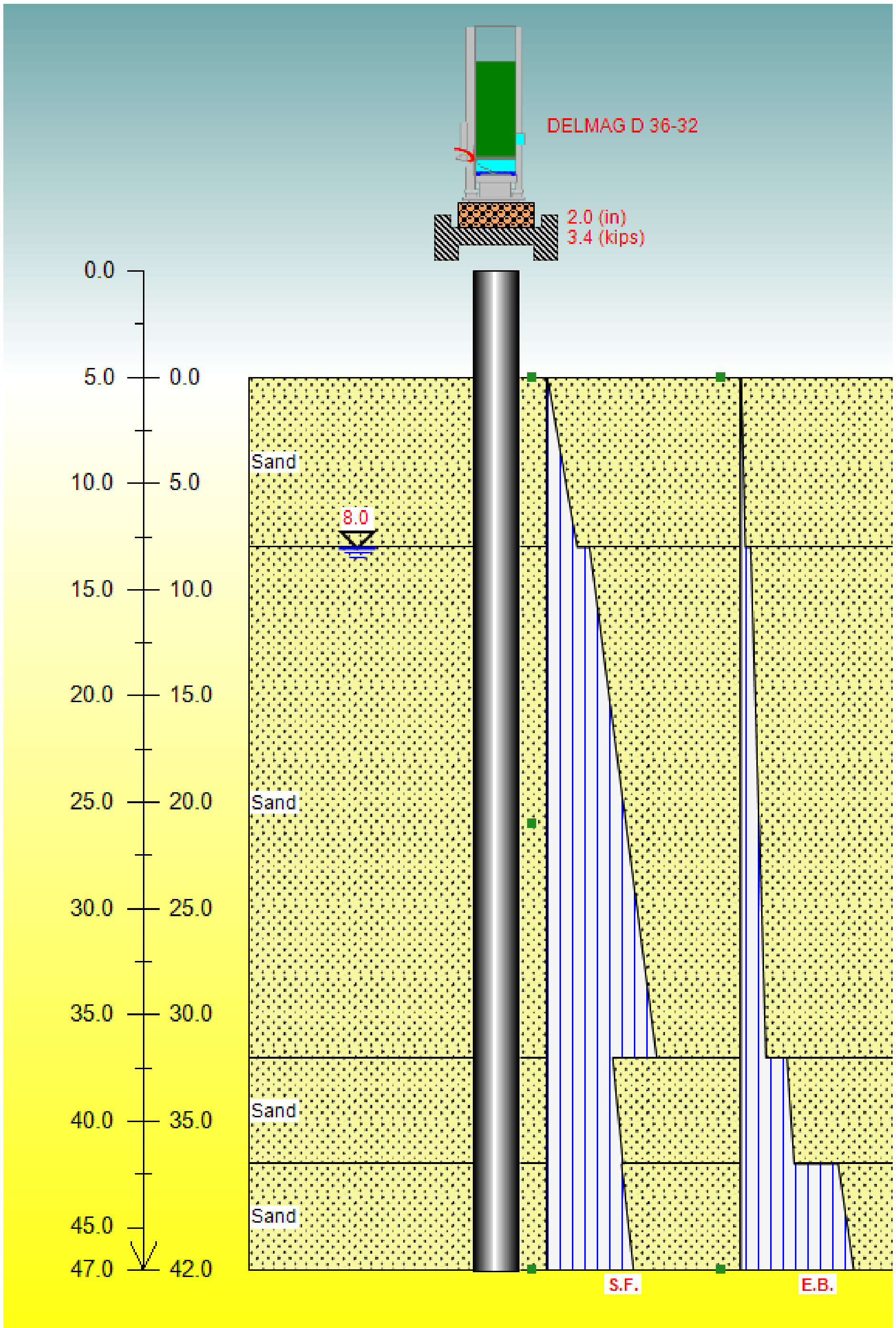
Total driving time: 20 minutes; Total Number of Blows: 842 (starting at penetration 5.0 ft)

ArDOT 101124 Hwy 135 over Ditch No. 1

Bent 3

24-in-diameter Steel Shell Pile

Delmag D36-32



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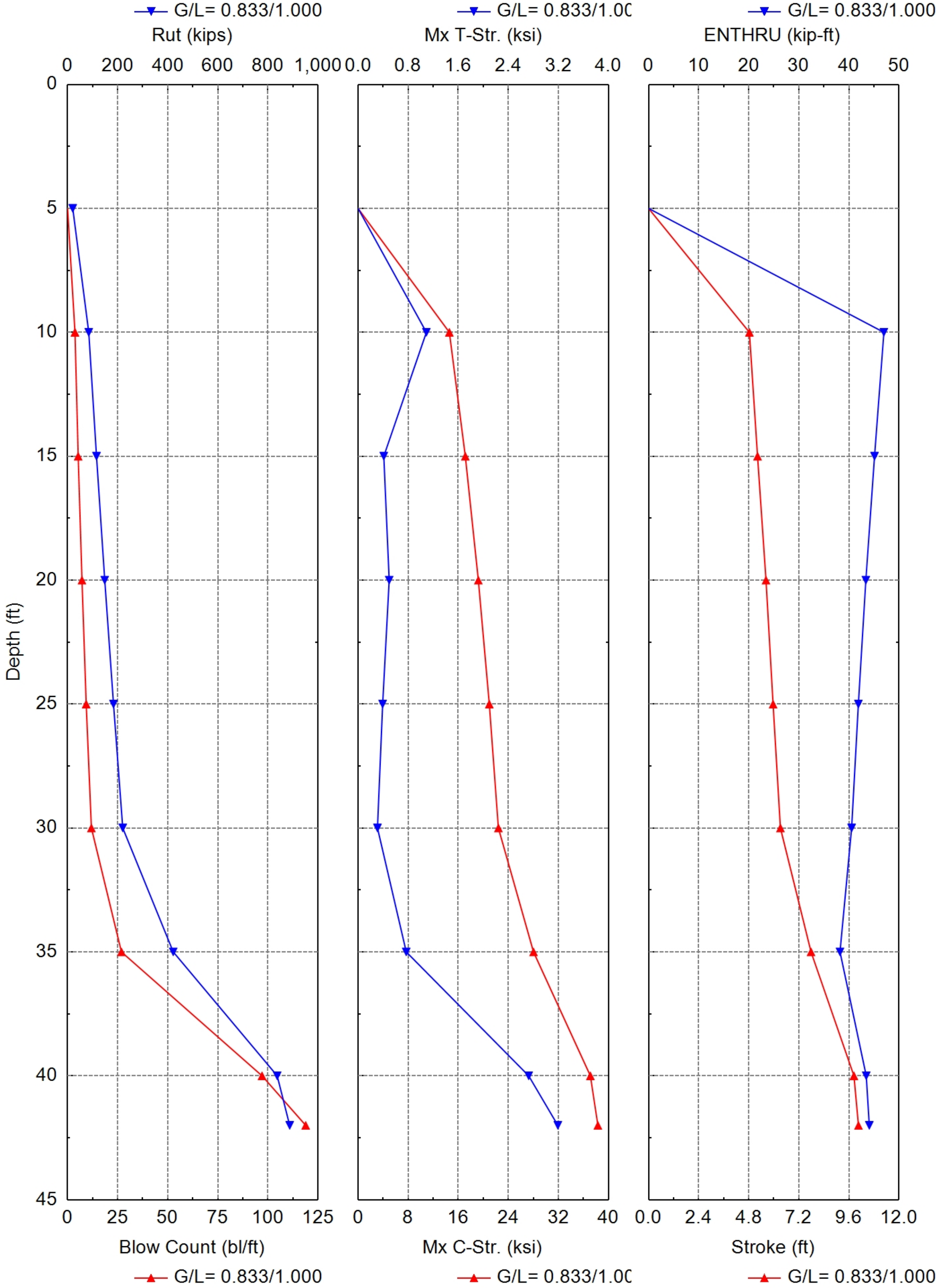

G/L= 0.833/1.000
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.833/1.0C
Mx T-Str. (ksi)
0.0
0.8
1.6
2.4
3.2
4.0
G/L= 0.833/1.000
ENTHRU (kip-ft)
0
10
20
30
40
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.833/1.000
0
8
16
24
32
40
Mx C-Str. (ksi)
G/L= 0.833/1.0C
0.0
2.4
4.8
7.2
9.6
12.0
Stroke (ft)
G/L= 0.833/1.000

Gain/Loss Factor at Shaft/Toe = 0.833/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow Ct bl/ft	Mx C-Str ksi	Mx T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	20.7	1.4	19.4	0.0	0.000	0.000	0.00	0.0	D 36-32
10.0	84.6	6.1	78.5	3.7	14.582	1.091	4.83	47.0	D 36-32
15.0	115.5	14.0	101.5	5.3	17.135	0.411	5.22	45.1	D 36-32
20.0	148.5	24.0	124.5	7.2	19.211	0.497	5.63	43.4	D 36-32
25.0	183.4	36.0	147.4	9.3	20.965	0.393	5.97	41.9	D 36-32
30.0	220.4	50.0	170.4	11.9	22.415	0.310	6.32	40.6	D 36-32
35.0	422.0	62.1	359.9	26.9	28.027	0.770	7.79	38.2	D 36-32
40.0	837.3	73.2	764.1	97.1	37.112	2.726	9.84	43.5	D 36-32
42.0	887.2	78.0	809.1	118.9	38.315	3.192	10.06	44.1	D 36-32

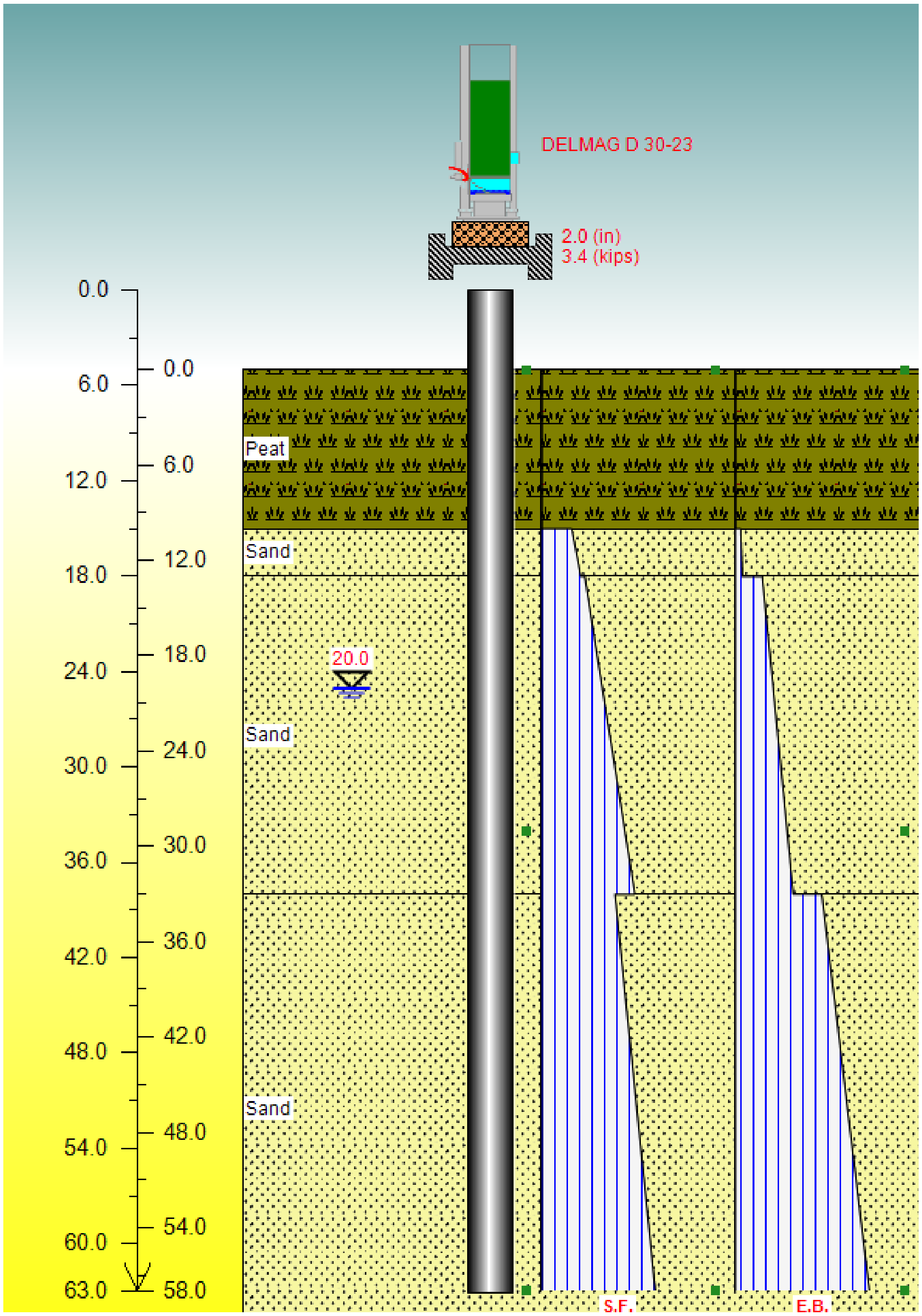
Total driving time: 19 minutes; Total Number of Blows: 779 (starting at penetration 5.0 ft)

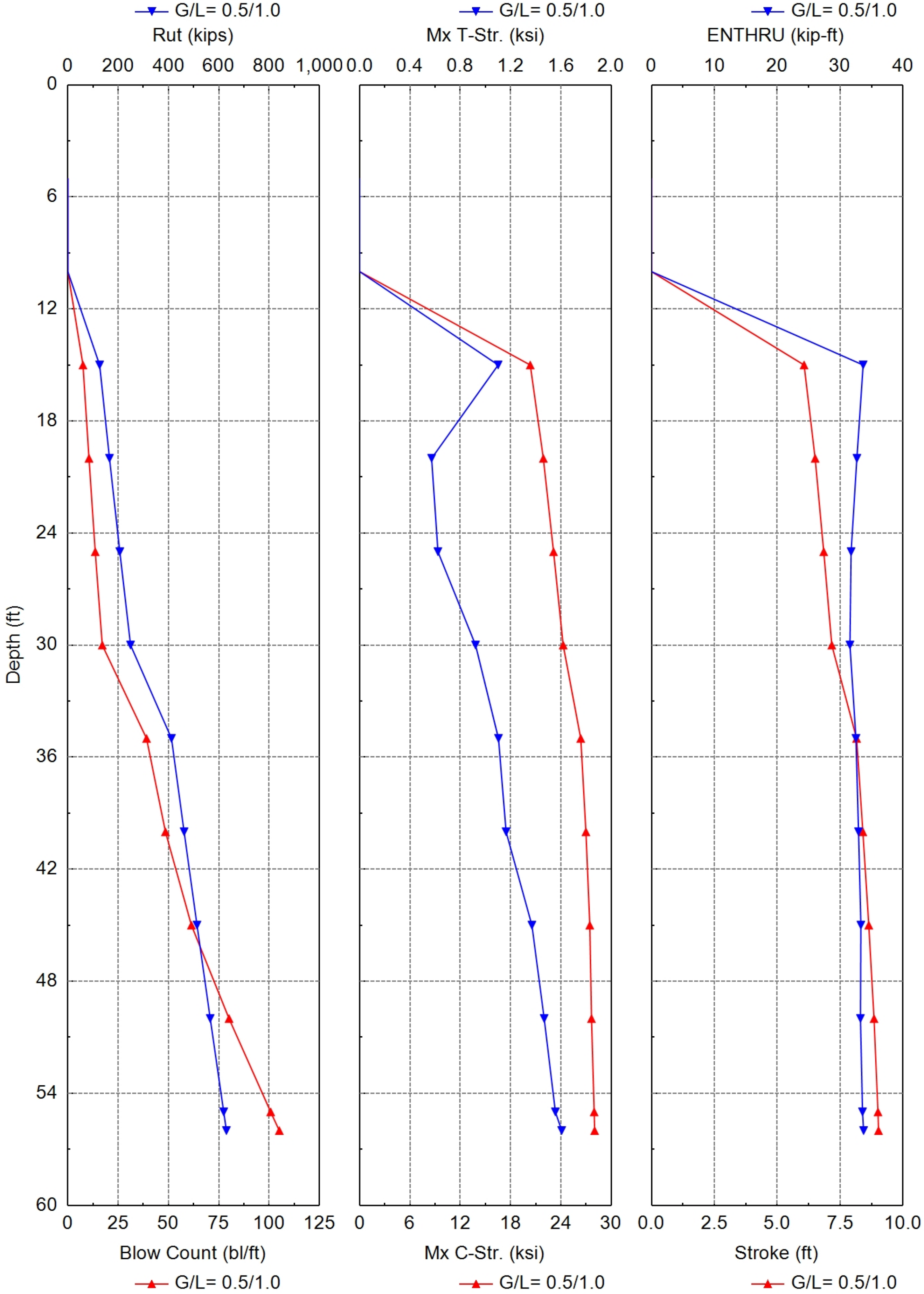
ArDOT 101124 Hwy 135 over Unnamed Ditch

Bent 4

16-in-diameter Steel Shell Pile

Delmag D 30-23




G/L= 0.5/1.0
Rut (kips)
0
200
400
600
800
1,000
G/L= 0.5/1.0
Mx T-Str. (ksi)
0.0
0.4
0.8
1.2
1.6
2.0
G/L= 0.5/1.0
ENTHRU (kip-ft)
0
10
20
30
40
Depth (ft)
0
6
12
18
24
30
36
42
48
54
60
0
25
50
75
100
125
Blow Count (bl/ft)
G/L= 0.5/1.0
0
6
12
18
24
30
Mx C-Str. (ksi)
G/L= 0.5/1.0
0.0
2.5
5.0
7.5
10.0
Stroke (ft)
G/L= 0.5/1.0

Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth	Rut	Rshaft	Rtoe	Blow Ct	Mx C-Str	Mx T-Str.	Stroke	ENTHRU	Hammer
ft	kips	kips	kips	bl/ft	ksi	ksi	ft	kip-ft	-
5.0	0.0	0.0	0.0	0.3	0.000	0.000	11.18	0.0	D 30-23
10.0	0.0	0.0	0.0	0.3	0.000	0.000	11.18	0.0	D 30-23
15.0	126.2	5.4	120.8	7.5	20.346	1.101	6.07	33.7	D 30-23
20.0	165.5	12.9	152.6	10.5	21.908	0.572	6.51	32.7	D 30-23
25.0	206.5	22.2	184.3	13.6	23.124	0.622	6.85	31.7	D 30-23
30.0	249.3	33.2	216.0	17.1	24.274	0.921	7.17	31.6	D 30-23
35.0	412.0	44.8	367.1	39.1	26.370	1.104	8.15	32.5	D 30-23
40.0	462.2	56.0	406.2	48.5	26.987	1.164	8.41	32.9	D 30-23
45.0	513.5	68.3	445.2	61.4	27.450	1.370	8.64	33.3	D 30-23
50.0	566.0	81.7	484.2	80.1	27.653	1.467	8.85	33.2	D 30-23
55.0	619.6	96.3	523.3	100.8	27.973	1.556	9.01	33.6	D 30-23
56.0	630.4	99.3	531.1	105.1	28.029	1.607	9.03	33.7	D 30-23

Total driving time: 43 minutes; Total Number of Blows: 1746 (starting at penetration 5.0 ft)

Site-Specific Seismic Ground Motion
Response Analysis (SSGMRA)
ArDOT Job No. 101124
Hwy. 135, Sections 1 & 2
Craighead & Poinsett Counties, Arkansas

Prepared for:

ARKANSAS DEPARTMENT OF TRANSPORTATION (ARDOT)
LITTLE ROCK, ARKANSAS

Prepared by:

GEOTECHNOLOGY, LLC
MEMPHIS, TENNESSEE

Date:

AUGUST 8, 2023

Geotechnology Project No.:

J043013.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY
A UES Company

August 8, 2023

Mr. Paul Tinsley, P.E.
Geotechnical Engineering Manager
Arkansas Department of Transportation (ARDOT)
PO Box 2261
Little Rock, Arkansas 72203

Re: Site-Specific Seismic Ground Motion Response Analysis (SSGMRA)
ArDOT Job No. 101124
Craighead & Poinsett Counties, Arkansas
Hwy. 135, Sections 1 & 2
Geotechnology Project No. J043013.01

Dear Mr. Tinsley:

Presented in this report are the results of site-specific seismic ground motion response analyses completed for the referenced project based on the provided geotechnical data, measured shear-wave velocity data, and provisions of the AASHTO LRFD Bridge Design Specifications. Our services were performed in general accordance with the scope of work under Task Order No G005. Our services were authorized under the existing on-call contract with ArDOT.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions regarding this report, or if we can be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY, LLC.

Duncan Adrian, P.E.
Geotechnical Manager

No. 19074
DUNCAN BLAKE ADRIAN
8-8-23

SAS/DBA/ASE:dba

Copies submitted: Client (email)

TABLE OF CONTENTS

LIST OF TABLES

EXECUTIVE SUMMARY

The following executive summary is provided solely for the purpose of overview. A party who relies on this report should read each section.

- The project includes the construction of several new bridges at crossings along Hwy 135 in Craighead and Poinsett Counties, Arkansas. Our scope of services included shear wave velocity testing at four (4) locations using SCPT methods and performing site-specific seismic ground motion response analyses to develop seismic design accelerations for the bridges.
- Based on the measured shear wave velocity, the calculated weighted average shear wave velocity per AASHTO (2020) ranged from 701 to 712 ft/sec at the four locations, which indicates Site Class D for each site.
- The site-specific seismic ground motion response analysis includes interpretation of the soil conditions based on the CPT sounding data. We consolidated the analyses to two site response analyses based on the soil conditions.
- Presented in Table 1 below is a summary of the results of code-based acceleration parameters for each site. Presented in
- Table 2 is a summary of the site-specific response results.

Table 1. Summary of Seismic Parameters Based on AASHTO Mapped Values

Parameter	**Site 2 - Tyronza River**	**Site 5 – Righthand Chute Little River**	
		South Side	North Side
Average Vs100 (ft/s)	701	709	701
AASHTO Site Class (Sec. 3.10.3.1 of AASHTO)	D	D	
As (g) (Site-adjusted PGA)	0.978	1.047	
SDS (g) (0.2 sec)	1.726	1.883	
SD1 (g) (1 Sec)	0.703	0.822	
Seismic Performance Zone	ZONE 4	ZONE 4	

Table 2. Summary of Site-Specific Response Results

Period	**Site 2-Tyronza River**	**Site 5 – Righthand Chute Little River**
A_s (g) (Site-adjusted PGA)	0.769	0.864
S_{DS} (g) (0.2 sec)	1.565	1.673
S_{D1} (g) (1 Sec)	1.197	1.247
Seismic Performance Zone	ZONE 4	ZONE 4

SITE-SPECIFIC SEISMIC GROUND MOTION RESPONSE ANALYSIS (SSGMRA)
HWY. 135, SECTIONS 1 & 2
CRAIGHEAD & POINSETT COUNTIES, ARKANSAS

August 8, 2023 | Geotechnology Project No. J043013.01

1.0 INTRODUCTION AND SCOPE OF WORK

Geotechnology, LLC prepared this site-specific ground motion response analysis (SSGMRA) for the Arkansas Department of Transportation (ARDOT) for Hwy. 135, Sections 1 & 2, located in Craighead & Poinsett Counties, Arkansas. The project includes the construction of several new bridges at crossings along Hwy 135.

In general, the purpose of our services was to perform a site-specific seismic ground motion response analysis (SSGMRA) by developing shear wave velocity profiles at each site, interpreting the soil conditions based on the CPT sounding data, developing a target response spectrum using probabilistic seismic hazard analysis methods, selecting ground motions for use in a site response model of the site, and performing a one-dimensional ground motion analysis to determine the seismic response at the ground surface.

It is our understanding the project will be designed in accordance with the AASHTO LRFD Bridge Design Specifications, herein referred to as AASHTO. The analysis was based on the CPT sounding data, and our experience with the current state of practice for site-specific ground motion response analyses..

A copy of "Important Information about This Geotechnical-Engineering Report," published by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association (GBA), is included in Appendix A for your review. The publication discusses report limitations and ways to manage risk associated with subsurface conditions.

2.0 PROJECT AND SITE DESCRIPTION

The project sites are located at existing bridges along Hwy 135 in Craghead and Poinsett Counties in Northeast Arkansas. The locations of the sites are shown in Figures 1, 3 and 5 in Appendix B. The project includes bridge replacements at each site. Site-specific seismic accelerations were requested for design of the new structures.

3.0 GEOTECHNICAL INFORMATION

3.1 General Geology

The site is located in Poinsett and Craighead counties in northeast Arkansas on the Gulf Coastal Plain. Geologically this area of the state is part of the Mississippi Embayment, a trough-like depression plunging southward along an axis approximating the present course of the Mississippi River and more locally part of the Arkansas River Valley. Based on general geologic maps of the area, the geology consists of Quaternary aged alluvial deposits of unconsolidated gravels, sand, silt, and clay unconformably overlying Tertiary aged deposits of sands, clays, marls and lignites.

3.2 Geotechnical Exploration

Four seismic cone penetration testing (CPT) soundings were performed for this project designated as SCPT-2, SCPT-5-North, SCPT-5-South, and SCPT-7. The following table shows the site location and approximate latitude and longitude of the CPT soundings. The approximate locations of the CPT soundings are also shown on Figures 2, 4, and 6 in Appendix B.

Table 3. Site locations

Location	CPT Designation	Lat. / Long.
Site 2	SCPT-2	35.505714°/-90.322910°
Site 5	SCPT-5-South	35.671814°/-90.338193°
	SCPT-5-North	35.672887°/-90.339106°
Site 7	SCPT-7	35.700568°/-90.341323°

The CPT soundings were advanced using a 20-ton, track-mounted Vertek direct-push rig on May 15 and 16, 2023 to depths of approximately 100 feet. The data was collected using a Vertek 15 square-centimeter end area, seismic piezometric cone with a u_2 pore pressure location (behind the cone) following the procedures outlined in ASTM D3441 and D5778. A plot of the CPT measurements are presented in Appendix C along with interpreted soil behavior types.

3.3 Subsurface and Groundwater Information

Based on the CPT sounding data, the general soil profile at the project site consisted of interbedded alluvial deposits of clay, silty clay, silty sand, and sandy silt to a depth of between 10 and 20 feet underlain by sand. However, in SCPT-2, clay extended from the ground surface to a depth of approximately 32 feet and was underlain by medium dense to dense sand. The CPT sounding logs are presented in Appendix C.

Groundwater was interpreted at depths ranging from 8 to 22 feet in the soundings. Groundwater levels will vary over time because of seasonal variations in precipitation, influence of adjacent streams and rivers, and other factors not evident at the time of exploration.

3.4 Shear-Wave Velocity Profile

Our field services included downhole, seismic-cone testing to measure the shear wave velocity of the soil profile at each CPT location. The following table includes the weighted average shear wave velocities (V_{S100}) within the upper 100 feet at each CPT location. The results of the shear wave velocity measurements indicate each site is Site Class D, “stiff soil” profile. Presented in Appendix B are the measured shear wave velocity profiles and average shear wave velocity in the upper 100 feet.

Table 4. Average Shear Wave Velocity and AASHTO Site Classification

CPT Designation	Average Shear Wave Velocity	AASHTO Site Class
SCPT-2	701	D
SCPT-5-South	709	D
SCPT-5-North	701	D
SCPT-7	712	D

4.0 SITE-SPECIFIC ANALYSIS AND RESULTS

Based on our review of the soil conditions, the conditions and shear wave velocity at SCPT-5-South, SCPT-5-North and SCPT-7 were similar. However, SCPT-2 at the Tyronza River site varied from the other SCPTs, consisting of a relatively thick clay layer in the upper approximately 32 feet. Therefore, we performed two site-specific response analyses for the project. A site response was performed for the Righthand Chute of the Little River, which was based on the soil conditions and shear wave velocity profile of SCPT-5-South and SCPT-5-North. An additional site response analysis was performed for the Tyronza River Site, which was based on the soil conditions and shear wave velocity profile of SCPT-2.

The AASHTO LRFD Bridge Design Specifications, 9th Edition (2020) was used as the reference procedure for the site response analysis. The details and results of the analyses are included in the attached reports found in Appendix D. A summary of the site-specific results is provided in Table 5.

Table 5. Summary of Site-Specific Response Results

Period	Site 2-Tyronza River	Site 5 – Righthand Chute Little River
A_S (g) (Site-adjusted PGA)	0.769	0.864
S_{DS} (g) (0.2 sec)	1.565	1.673
S_{D1} (g) (1 Sec)	1.197	1.247
Seismic Performance Zone	ZONE 4	ZONE 4

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

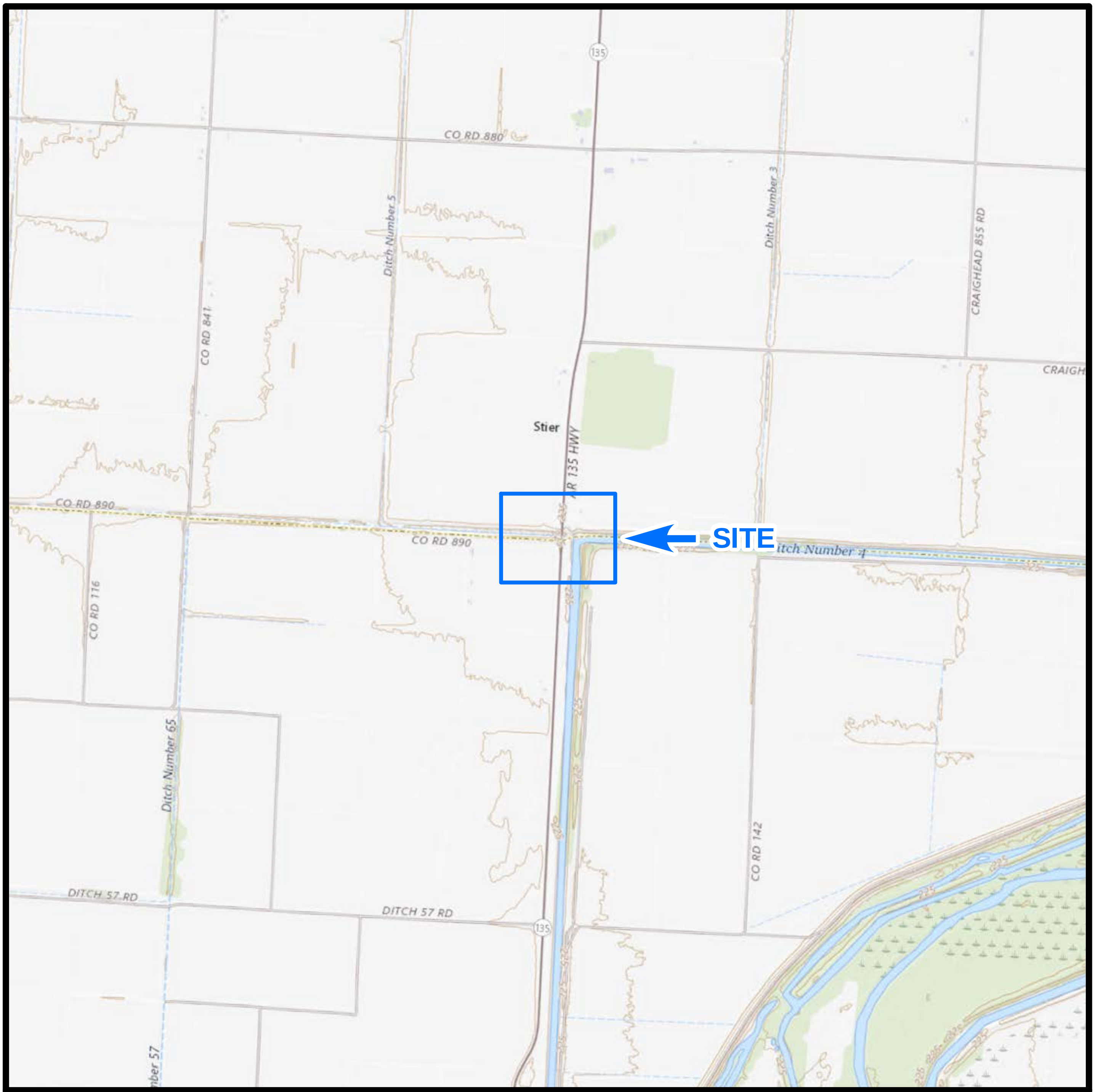
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – FIGURES



NOTES
1. Plan adapted from 7.5 minute U.S.G.S. maps for Riverale & Lepanto quadrangles, last revised in 2017.

0 1,000 2,000 4,000 Feet

N

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/31/2023	Date:	Date:

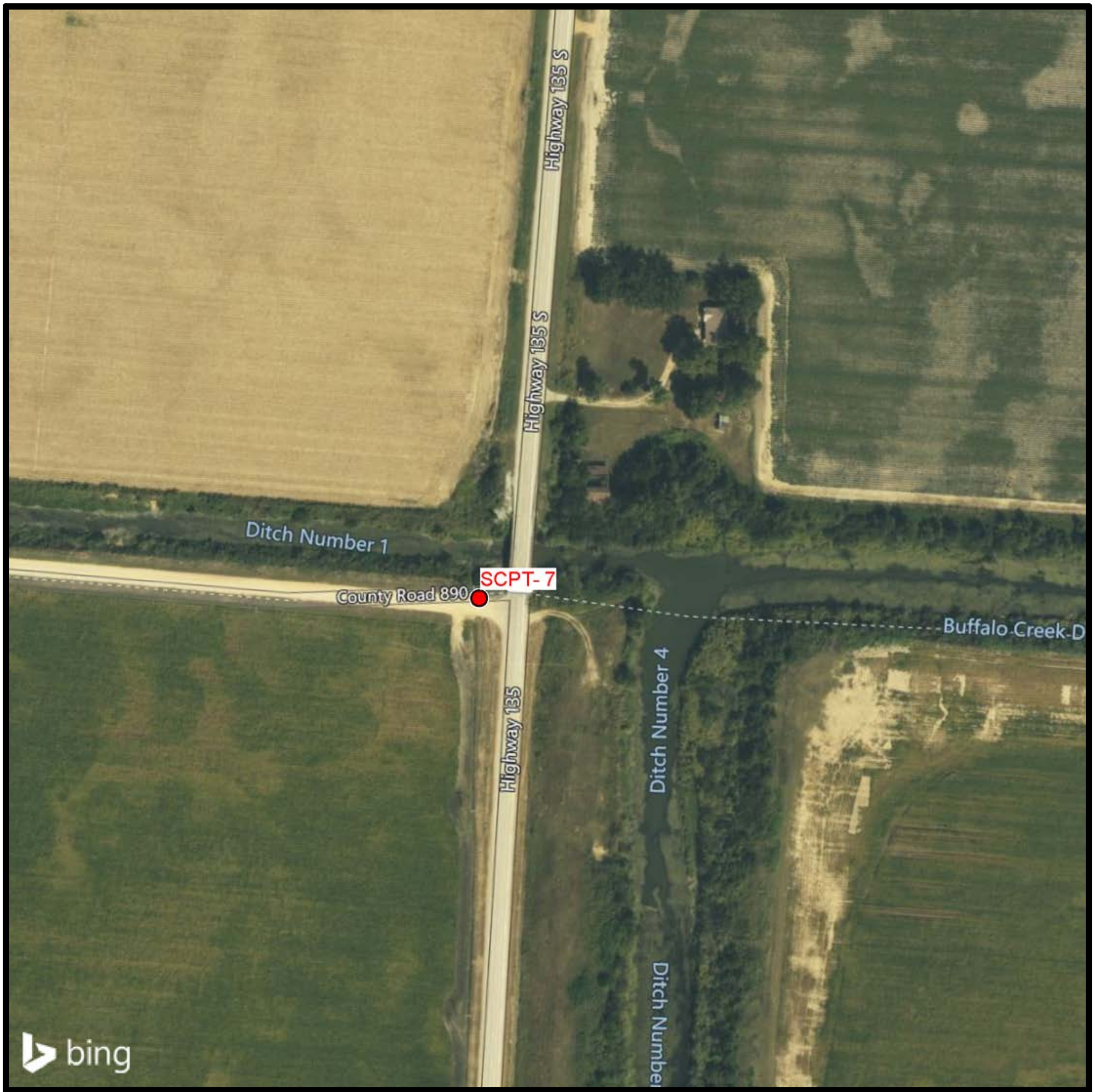
GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

SITE LOCATION
AND TOPOGRAPHY

Project Number
J043013.01

FIGURE 1



0 100 200 400 Feet

NOTES:
1. Borings were located in the field with reference to site features and are shown approximate only.

Legend

● Boring Locations

N

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/30/2023	Date:	Date:

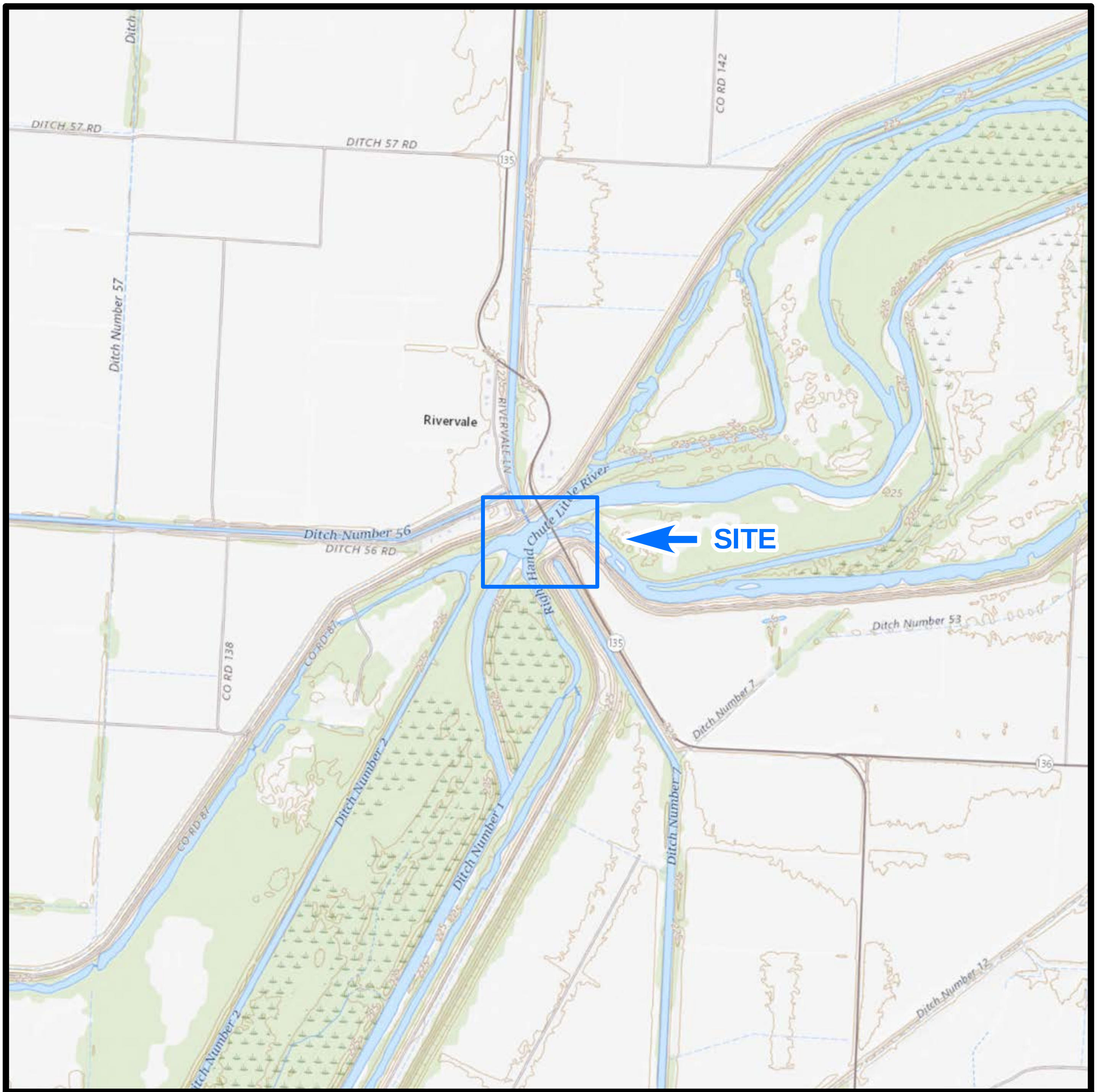
GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

AERIAL PHOTOGRAPH OF
SUBJECT PROPERTY

Project Number
J043013.01

FIGURE 2



NOTES
1. Plan adapted from 7.5 minute U.S.G.S. maps for Riverale & Lepanto quadrangles, last revised in 2017.

0 1,000 2,000 4,000
Feet

N

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/31/2023	Date:	Date:

GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

SITE LOCATION
AND TOPOGRAPHY

Project Number J043013.01	FIGURE 3

NOTES:
1. Borings were located in the field with reference to site features and are shown approximate only.

Legend

● Boring Locations

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/30/2023	Date:	Date:

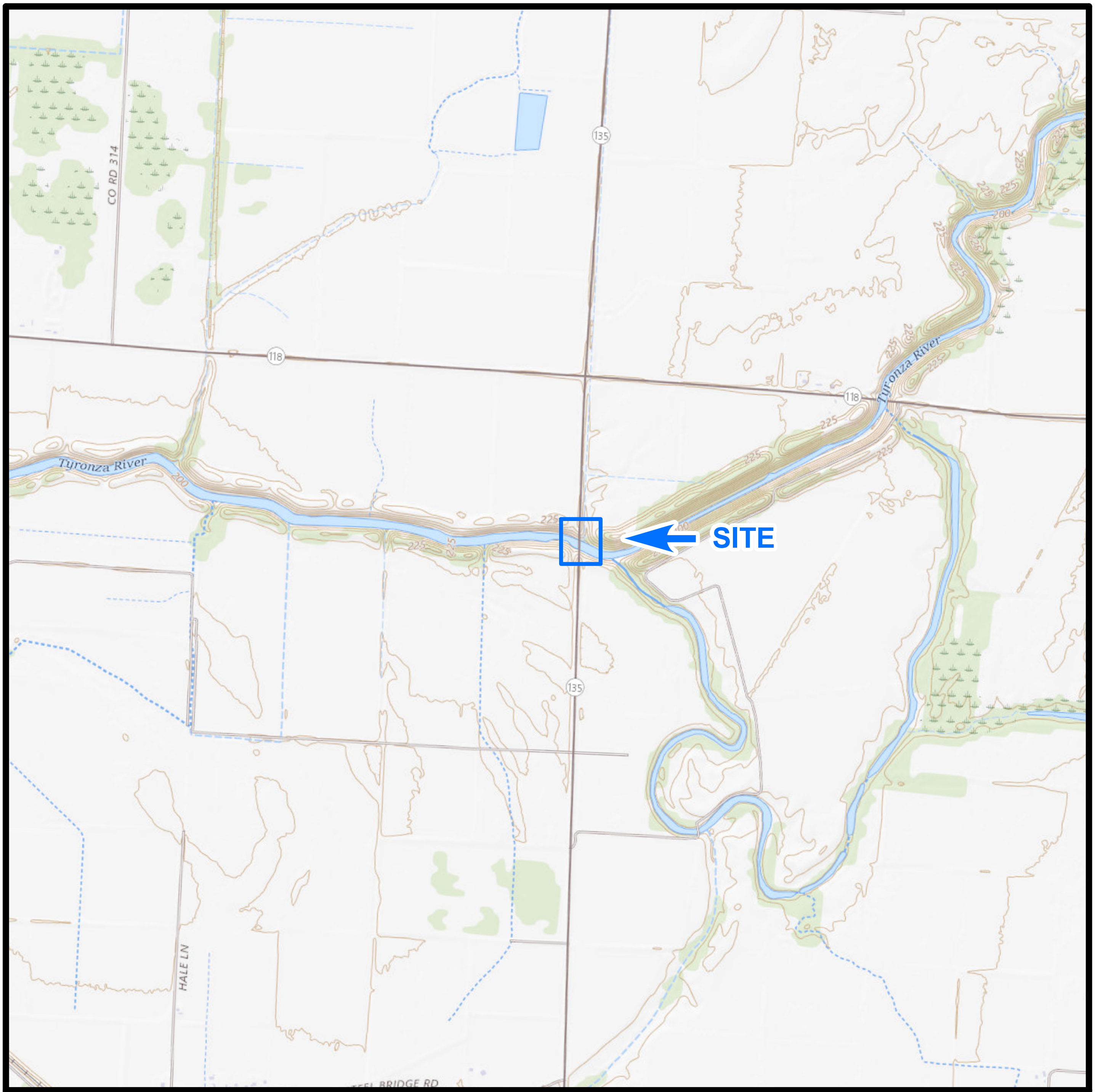
GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

AERIAL PHOTOGRAPH OF
SUBJECT PROPERTY

Project Number
J043013.01

FIGURE 4



NOTES
1. Plan adapted from 7.5 minute U.S.G.S. maps
for Riverale & Lepanto quadrangles,
last revised in 2017.

0 1,000 2,000 4,000 Feet

N

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/31/2023	Date:	Date:

GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

SITE LOCATION
AND TOPOGRAPHY

Project Number
J043013.01

FIGURE 5

NOTES:
1. Borings were located in the field with reference to site features and are shown approximate only.

Legend

● Boring Locations

N

Drawn By: PCM	Ck'd By:	App'vd By:
Date: 07/30/2023	Date:	Date:

GEOTECHNOLOGY
A UES Company

ARDOT Job 101124 - Seismic Analysis
Craighead & Poinsett Counties, Arkansas

AERIAL PHOTOGRAPH OF
SUBJECT PROPERTY

Project Number
J043013.01

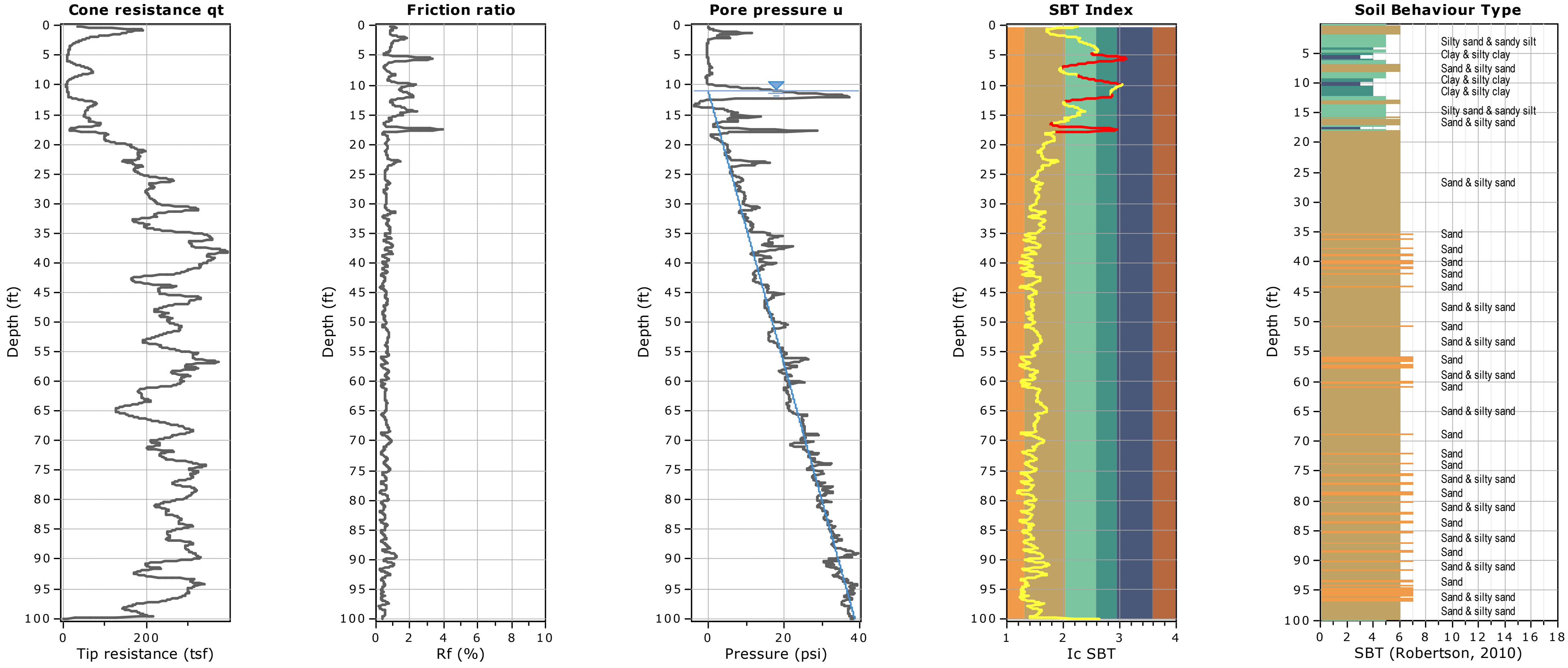
FIGURE 6

APPENDIX C – CPT INTERPRETATION PLOTS

SCPT-7

Total depth: 100.09 ft, Date: 5/15/2023
Coords: lat 35.700528° lon -90.341281°
Cone Operator: Jett

Project: ArDOT Job 101124
Location: Poinsett County, Arkansas



SBT legend

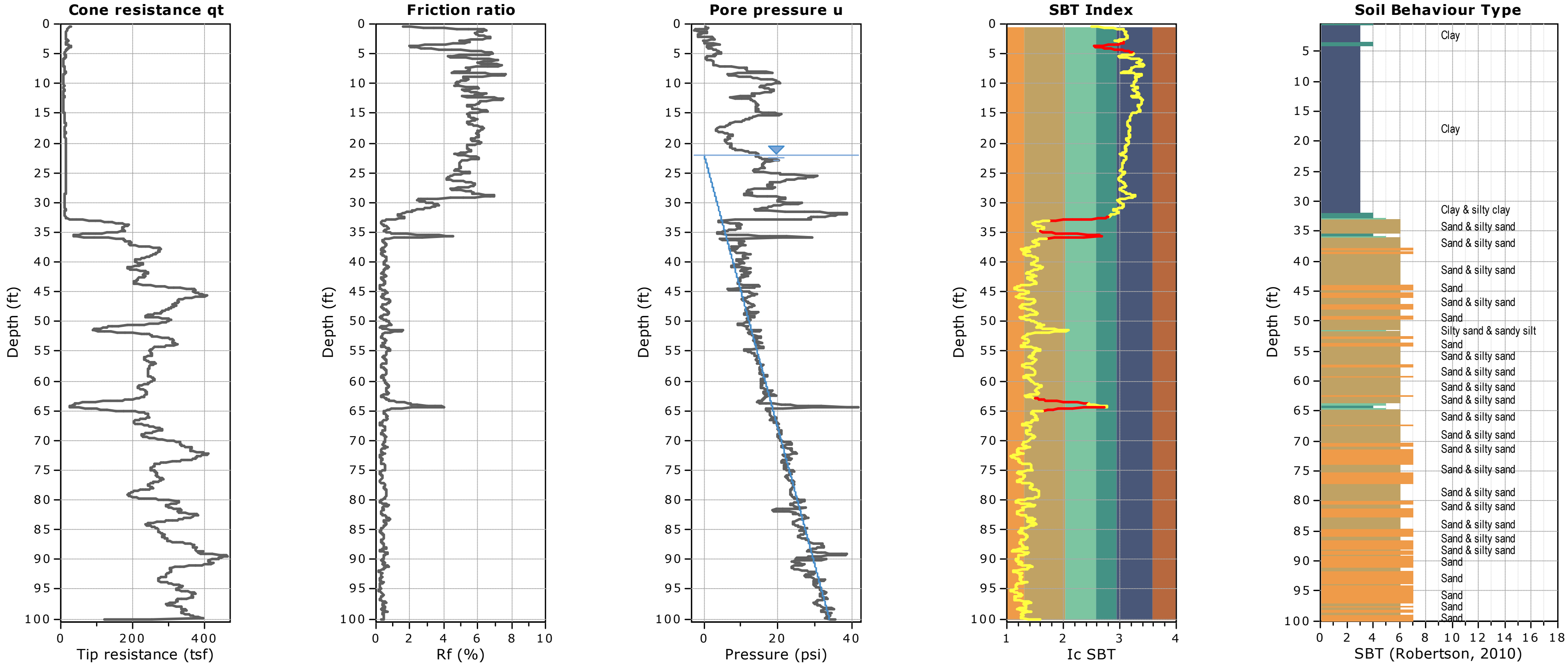
- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

GEOTECHNOLOGY

SCPT-2

Total depth: 100.09 ft, Date: 5/16/2023
Coords: lat 35.505736° lon -90.32289°
Cone Operator: Jett

Project: **ArDOT Job 101124**
Location: **Poinsett County, Arkansas**



Cone resistance qt

Friction ratio

Pore pressure u

SBT Index

Soil Behaviour Type

SBT legend

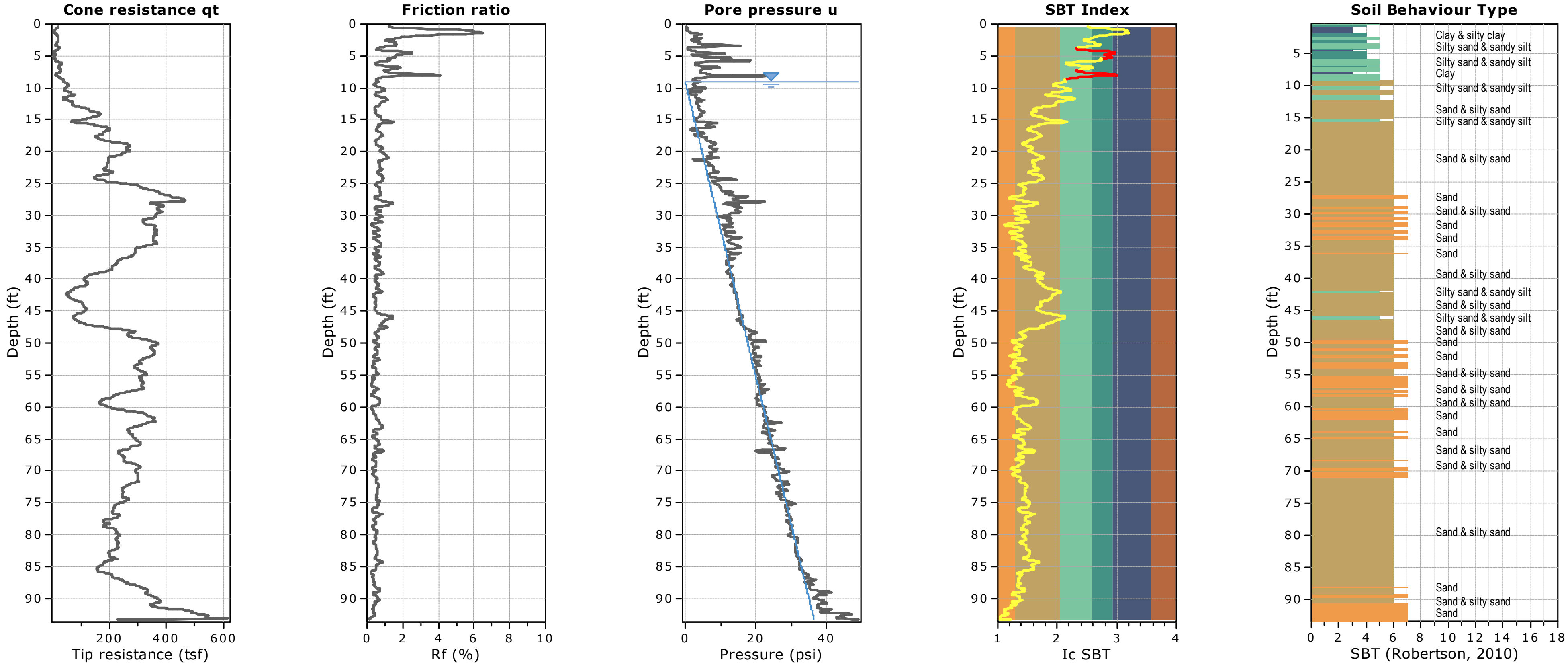
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

GEOTECHNOLOGY

SCPT-5 SOUTH

Total depth: 93.31 ft, Date: 5/16/2023
Coords: lat 35.671799° lon -90.338139°
Cone Operator: Jett

Project: ArDOT Job 101124
Location: Poinsett County, Arkansas



SBT legend

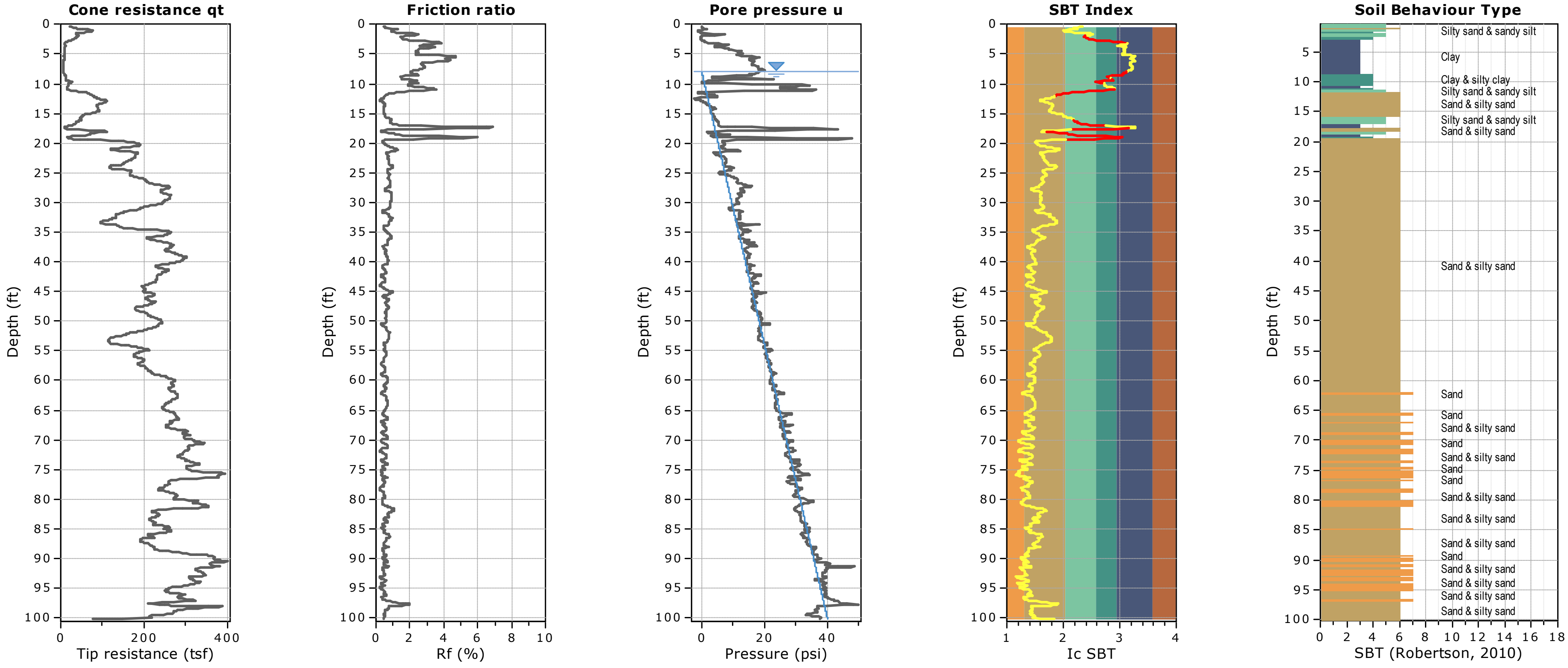
🟥 1. Sensitive fine grained	🟦 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

GEOTECHNOLOGY

SCPT-5 NORTH

Total depth: 100.17 ft, Date: 5/16/2023
Coords: lat 0° lon 0°
Cone Operator: Jett

Project: ArDOT Job 101124
Location: Poinsett County, Arkansas



SBT legend

🟥 1. Sensitive fine grained	🟦 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

APPENDIX D – SITE-SPECIFIC SEISMIC STUDY REPORTS

Site-Specific Seismic Study
Chute Little River Site
Poinsett County, Arkansas

By

Shahram Pezeshk, Ph.D., P.E.
Email: s.pezeshk@aol.com
901-606-6934

June 17, 2023

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Chute Little River Site
Poinsett County, Arkansas

1.0. EXECUTIVE SUMMARY

The executive summary provides an overview of my understanding of the project and recommendations. Information and recommendations presented in the executive summary should not be used without reviewing the entire Report.

- The location of the study site is 35.67288°N and 90.3391°W (See Appendix A).
- Based on the recommendations of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, A_S (zero-period), S_{DS} (short period), and S_{D1} (long period) are provided in Table 3.
- Site-specific recommendations following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions are provided in Table 5 and Table 6.

2.0. SCOPE OF WORK

The purpose of our study is to estimate the design spectra following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. The structural design of new buildings allows two procedures for determining design ground motions:

1. *General Procedure*. In this method, the response spectrum is determined using the following steps: (1) develop the rock spectrum using seismic design maps for values of Peak Ground Acceleration (PGA) and spectral acceleration at periods of 0.2 and 1.0 seconds; (2) determine the Site Class using the shear-wave velocity (V_s) measurements from the upper 100 feet of the soil profile, and (3) adjust the rock spectrum for site class to develop the general response spectrum.

2. *Site-Specific Procedure*. In this method, the response spectrum is determined using a combination of probabilistic seismic hazard and site response analyses. The site-specific response spectrum may not be less than 2/3 of the general response spectrum.

Briefly, the scope of our services for the site-specific investigation included the following steps:

1. Perform probabilistic seismic hazard analysis (PSHA) to estimate ground motions in the rock underlying the site;
2. Determine Uniform Hazard Response Spectrum (UHRS) at the rock level;
3. Determine probabilistic consistent magnitude and distances from deaggregation;
4. Select ground motions consistent with magnitude and distances obtained in step 3;
5. Perform spectral matching to match the selected ground motions to the UHRS of Step 2;
6. Perform one-dimensional equivalent linear site-specific ground response analysis using the site-specific earthquake time histories by using the computer program SHAKE91 (Idriss and Sun, 1992) and considering the uncertainties associated with the shear-wave velocity and layer thicknesses for the soil profile; and
7. Develop site-specific response spectra for the existing subsurface conditions using the procedure outlined in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2022 Interim Revisions, based on 7 percent probability of exceedance in 75 years and 5 percent damping for a single degree of freedom (SDOF) structure.

3.0. SUBSURFACE CONDITIONS

This study is based on the available information on the soil stratigraphy provided by Geotechnology and the shear-wave velocity profile obtained using Seismic Cone Penetration Testing (SCPT).

4.0. SHEAR-WAVE VELOCITY PROFILE

Seismic Cone Penetration Testing (SCPT) was performed by Geotechnology (a UES Company). Table 1 provides the shear-wave velocity obtained from SCPT.

Table 1. Shear-Wave Velocities Measured.

SCPT North			**SCPT South**		
Depth1 (ft)	**Depth2 (ft)**	**V_s (ft/sec)**	**Depth1 (ft)**	**Depth2 (ft)**	**V_s (ft/sec)**
4.43	7.74	390.22	1.80	5.05	658.85
7.74	11.02	597.94	5.05	8.20	495.94
11.02	14.30	581.64	8.20	11.51	404.33
14.30	17.61	743.18	11.51	14.79	904.79
17.61	20.89	467.43	14.79	18.07	543.66
20.89	24.21	703.63	18.07	21.32	574.59
24.21	27.49	635.76	21.32	24.60	656.92
27.49	30.86	722.78	24.60	27.88	602.60
30.86	33.98	687.85	27.88	31.26	800.48
33.98	37.20	745.54	31.26	34.54	699.23
37.20	40.64	808.59	34.54	37.79	815.05
40.64	43.92	743.38	37.79	41.03	630.51
43.92	47.10	696.67	41.03	44.31	633.17
47.10	50.38	700.58	44.31	47.56	723.99
50.38	53.60	895.80	47.56	50.84	602.08
53.60	56.97	716.84	50.84	54.05	781.76
56.97	60.22	775.72	54.05	57.30	767.82
60.22	63.47	815.41	57.30	60.55	840.43
63.47	66.72	877.92	60.55	63.80	858.84
66.72	70.03	698.64	63.80	67.17	1063.80
70.03	73.28	929.22	67.17	70.45	658.30
73.28	76.52	851.26	70.45	73.70	878.74
76.52	79.80	770.87	73.70	76.98	897.70
79.80	83.05	940.90	76.98	80.23	785.43
83.05	86.53	1354.08	80.23	83.44	900.03
86.53	89.74	598.04	83.44	86.69	742.56
89.74	92.92	1072.10	86.69	89.94	861.23
92.92	96.37	894.72			
96.37	100.01	856.77			

To construct a base-case profile, the velocity profile was extemded to a deeper geologic unit that represents the reference site condition. To extend the shallower portion of the velocity profile to the deeper portion, the 3D velocity model developed for Central United States (CUS) was used. The CUS 3D velocity model has been developed by Ramirez-Guzman *et al.* (2012) and is a result of several efforts in previous years including Allen and Wald (2007), Chung and Rogers (2010), Cramer *et al.* (2004), Ginzburg *et al.* (1983), Gomberg *et al.* (2003), Mooney *et al.* (1983), Prodehl *et al.* (1984), and Stewart (1968).

5.0 GENERAL INFORMATION

For this project, we have been requested to perform a site-specific seismic study to produce the ground surface response spectrum and a set of time series based on the seismic parameters used in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, which include: seismic hazards related to 7 percent probability of exceedance in 75 years and 5 percent damping for SDOF structure.

6.0. REGIONAL SEISMICITY

Petersen et al. (2019) used fault models from the 2014 NSHM to model large earthquakes and apply gridded, smoothed seismicity models from an earthquake catalog to account for smaller earthquakes on and off the faults. They developed new seismicity catalogs for the CEUS and WUS, including earthquakes from 2013 through 2017 that occurred since the last model was constructed. Between 2013, when the catalog was last updated, and 2018, strongly felt earthquakes (magnitude 4+) occurred in almost half of the states in the United States. Figure 1 shows the USGS 2018 declustered catalog for CEUS.

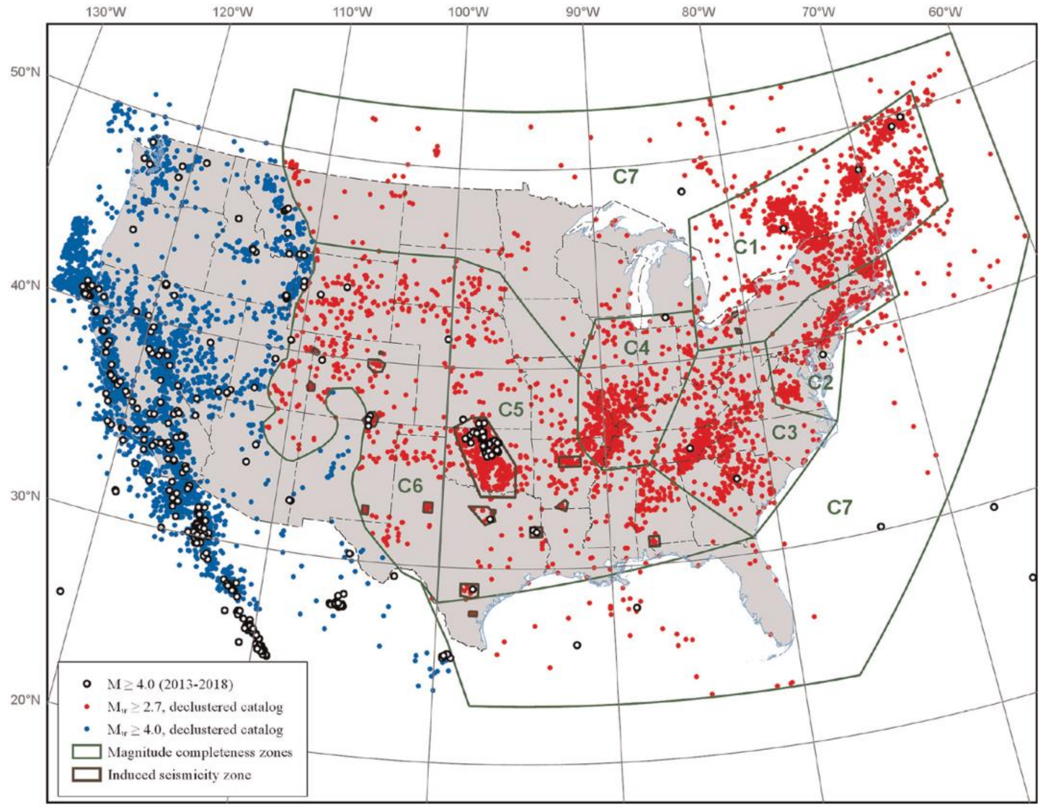


Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

7.0. SEISMIC HAZARD ANALYSIS

A PSHA was performed to estimate the seismic ground motions for a rock site condition. The analytical model used for the PSHA is based on models developed initially by Cornell (1968). These models' underlying assumption is that earthquakes occur in space and time within a particular seismic zone is entirely random (i.e., a Poisson process). This type of probabilistic model is commonly used for seismic hazard analyses of essential facilities throughout the world.

The two primary components of the probabilistic model are:

1. The seismic source models specify the spatial, temporal, and magnitude distribution of earthquake occurrences expected in each of the seismic sources, and

2. The ground-motion attenuation models which determine the distribution of ground motions expected at the site for a potential earthquake occurrence (characterized by magnitude and location, and usually by other factors) on a seismic source.

The above two components comprise the inputs to the PSHA. In the PSHA, probability-of-exceedance rates (hazard curves) are computed for a range of horizontal ground motions. These ground motions are expressed in terms of peak ground acceleration (PGA) and 5 percent-damped pseudo absolute spectral accelerations (S_a) at various single-degree-of-freedom oscillator periods. From the probability-of-exceedance rates, the Uniform Hazard Response Spectrum (UHRS) corresponding to average return periods of 7% probability of exceedance in 75 years is computed.

7.1. SEISMIC SOURCE MODELS

The USGS seismic source models have been used for this project. The USGS addressed the causes of earthquakes in the Central and Eastern United States in two ways: (1) earthquake fault; and (2) background or smoothed seismicity models, which forecast the occurrence rates and magnitudes of potential seismic events.

7.2. GROUND MOTION MODELS

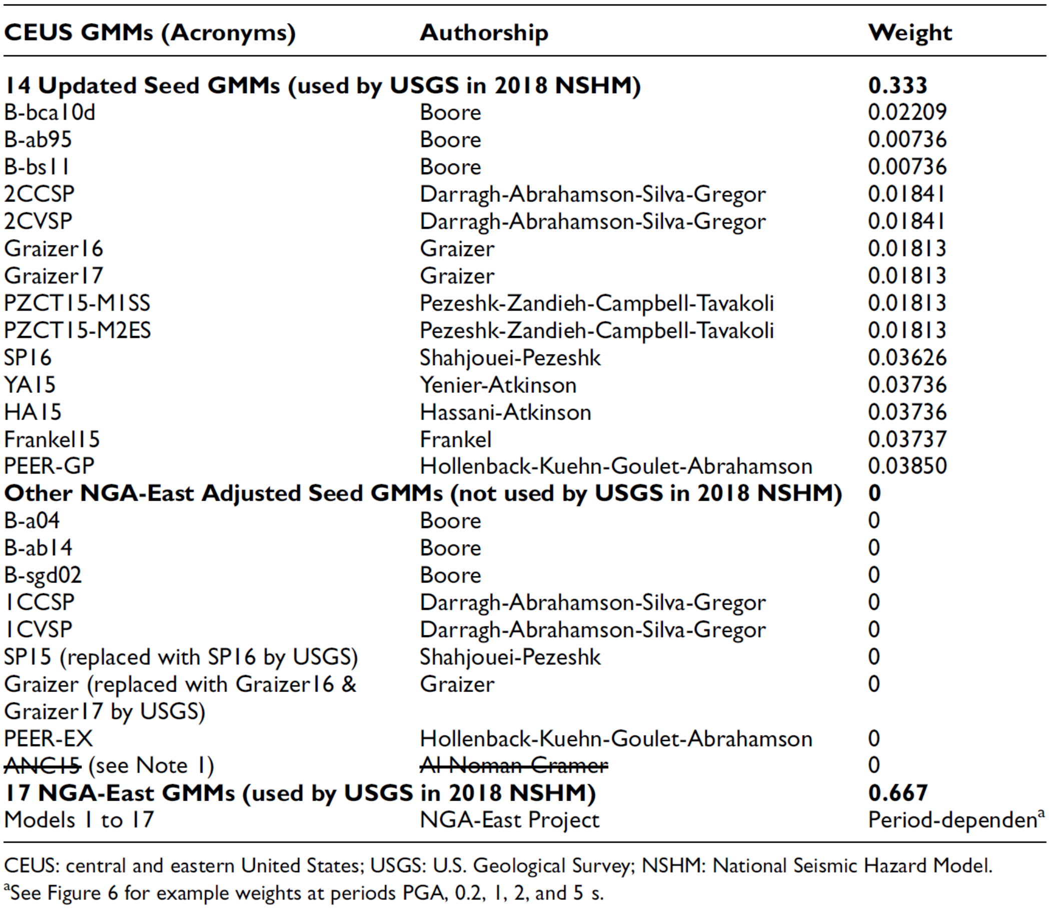
In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), also known as attenuation relationships. Various attenuation relationships have developed for specific regions using a database of appropriate ground motion records.

Petersen et al. (2020a) presented only a summary of the CEUS GMM updates, which included comparisons of the 2018 weighted median GMMs to the 2014 National Seismic Hazard Model (NSHM) and an overview of the aleatory variability (GMM standard deviation) and site-effect models. Rezaeian et al. (2021) discuss the CEUS GMM updates and implementation in the 2018 NSHM in detail. These updates consist of (1) 31 new GMMs, including the state-of-the-art Next Generation Attenuation relationships for central and eastern North America (NGA-East) (Goulet et al., 2018, 2017, 2021; Pacific Earthquake Engineering Research Center (PEER), 2015a), (2) an associated model of aleatory variability (based on Al Atik, 2015; Goulet et al., 2017; Stewart et al., 2019), and (3) a new site-effect model (for amplification or deamplification) specific to the CEUS (Hashash et al., 2020; Stewart et al., 2020). In the following, we discuss the individual GMMs in terms of their medians, assigned weights, weighted averages, attenuations with distance, and epistemic uncertainty.

According to Rezaeian et al. (2021), NSHM 2018 was updated to generate national seismic hazard maps for the Central and Eastern United States. The logic tree weights are based on the distance and the geometric spreading term used by each model. The models with a faster geometric spreading term are given more weight. The New Madrid seismic zone is the most likely seismic source that could affect the considered site. NSHM removed the attenuation relationships not applicable beyond 500 km, and weights were renormalized.

Table 2 lists the selected GMMs from the NSHM 2018 models with their associated weights. Three of the models were developed by Pezeshk and his colleagues [Pezeshk et al. 2015; 2018 (PZCT15-M1SS, PZCT15-M2ES), Shajouei and Pezeshk (2016) (SP16)].

Table 2. Ground Motion Models (GMMs).



CEUS GMMs (Acronyms)	**Authorship**	**Weight**
14 Updated Seed GMMs (used by USGS in 2018 NSHM)		**0.333**
B-bca10d	Boore	0.02209
B-ab95	Boore	0.00736
B-bs11	Boore	0.00736
2CCSP	Darragh-Abrahamson-Silva-Gregor	0.01841
2CVSP	Darragh-Abrahamson-Silva-Gregor	0.01841
Graizer16	Graizer	0.01813
Graizer17	Graizer	0.01813
PZCT15-M1SS	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
PZCT15-M2ES	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
SP16	Shahjouei-Pezeshk	0.03626
YA15	Yenier-Atkinson	0.03736
HA15	Hassani-Atkinson	0.03736
Frankel15	Frankel	0.03737
PEER-GP	Hollenback-Kuehn-Goulet-Abrahamson	0.03850
Other NGA-East Adjusted Seed GMMs (not used by USGS in 2018 NSHM)		**0**
B-a04	Boore	0
B-ab14	Boore	0
B-sgd02	Boore	0
1CCSP	Darragh-Abrahamson-Silva-Gregor	0
1CVSP	Darragh-Abrahamson-Silva-Gregor	0
SP15 (replaced with SP16 by USGS)	Shahjouei-Pezeshk	0
Graizer (replaced with Graizer16 & Graizer17 by USGS)	Graizer	0
PEER-EX	Hollenback-Kuehn-Goulet-Abrahamson	0
~~ANC15~~ (see Note 1)	~~Al Noman-Cramer~~	0
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

CEUS: central and eastern United States; USGS: U.S. Geological Survey; NSHM: National Seismic Hazard Model.

[a] See Figure 6 for example weights at periods PGA, 0.2, 1, 2, and 5 s.

7.3. TREATMENT OF UNCERTAINTIES

Seismic-hazard studies distinguish between two types of uncertainty, namely epistemic and aleatory. Aleatory uncertainty is probabilistic variability that results from a natural physical process. For example, the size, location, and time of the next earthquake on a fault and the details of the ground motion are considered aleatory uncertainties. In advanced seismic hazard studies, integration is performed over aleatory uncertainties to get a single hazard curve—the epistemic uncertainty results from a lack of knowledge about earthquakes and their effects. In principle, epistemic uncertainties are addressed by multiple models and parameters. The most well-known epistemic uncertainties associated with the input parameters in seismic hazard analysis include the uncertainties in seismic source models (i.e., tectonic stresses, geological features, geometries, etc.), seismicity (i.e., activity rate, slip rate, etc.), and attenuation relationships (source, path, and site effects). The USGS 2014 procedure (Petersen *et al.*, 2014) is followed in this project to address the uncertainty in seismic-source characterization, which is quantified by considering alternative geometries, multiple magnitude-recurrence parameters, and multiple maximum magnitudes.

8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Time-averaged shear-wave velocity in the top 100 ft (30 m) is defined as V_{S30}. The V_{S30} for the study site is determined to be about 705 ft/sec, which according to the Guide Specifications, the study site is determined to be a Site Class "D" (Table 3.4.2.1-1, Site Class Definitions). Site coefficients F_{pga}, F_a, and F_v for the study site following Tables 3.4.2.3-1 and 3.4.2.302 mapped spectral acceleration are summarized in Table 3.

8.1. Dynamic Soil Properties

Low-strain soil shear modulus and damping are the required dynamic soil properties for seismic ground response analysis. A brief discussion of these properties is given below.

8.1.1. Low Strain Soil Shear Modulus

A key parameter necessary to evaluate the dynamic response of soils is the dynamic shear modulus, $G_{s,}$ or shear wave velocity, which is also related to the dynamic shear modulus. Values of shear wave velocity or shear modulus can be determined either by measuring in the laboratory on undisturbed soil samples or by performing seismic field tests. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than 10^{-4} percent), referred to as G_{max}, is the desired design parameter.

Laboratory measurement of shear wave velocity or low-strain soil shear modulus was beyond the scope of our services. Various correlations and typical values are available in the literature to estimate the approximate value of shear-wave velocity and G_{max}.

8.1.2. Damping

The inelastic behavior of soil (discussed later) also gives rise to the energy absorption characteristics of soil, known as material damping. Damping is generally expressed as a percentage of critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the study based on the strain levels in the soil, as explained in subsequent sections of this Report.

8.1.3. Effect of Strain on Dynamic Soil Properties

It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus is not a constant value but degrades nonlinearly with increasing strain in the soil. Dynamic analyses considering the true nonlinear behavior of soil are complicated and are an active and current research area. Accordingly, an equivalent linear analysis is typically used in practice. Equivalent linear analyses consist of performing a series of linear analyses in an iterative process, using, for each analysis, soil properties consistent with the strains resulting from the previous one. An equivalent linear site response analysis is used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain.

9.0. CODE-BASED DESIGN APPROACH

9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Using the United States Geological Survey (USGS) Hazard Maps and the project location, the mapped 0.2-second spectral response acceleration (S_s) and the mapped 1.0-second spectral response acceleration (S_1) are provided in Table 3. Based on the average shear-wave velocities of the top 100 ft of soil, the site class has been determined to be site class “D.” Based on the mapped spectral acceleration and site class D, the site coefficients F_{PGA}, $F_{a,}$ and F_v are provided in Table 3. provides a summary of these parameters.

Table 3. Mapped Provisional Design Response Spectrum Parameters at 5% Damping.

Parameter	Value
F_a	1.000
F_v	1.500
F_{PGA}	1.000
S_S	1.883
S_1	0.548
S_{DS}	1.883
S_{D1}	0.822
PGA	1.047
A_s	1.047

10.0. SITE-SPECIFIC PROCEDURE

The probabilistic seismic hazard analysis (PSHA) considers all potential earthquake sources that will contribute to hazards at a specific site. The PSHA factors in contributions from all magnitudes, distances, and probability of occurrence for all sources. This study used PSHA to estimate PGA and spectral acceleration at various periods for a B/C NEHRP site condition for a 7% probability of exceedance in 75 years.

The PSHA was performed to obtain a uniform hazard response spectrum (UHRS). The PSHA and de-aggregation results were used to select earthquakes for the site response analyses. Eleven horizontal components (total of 11) of previously recorded earthquakes within the range of de-aggregation magnitudes and distances were selected.

Table *4* provides the mean and the modal deaggregation magnitude and distances for various periods. The UHRS was selected as the target spectrum, and the chosen time histories were matched with the target spectrum. As an example, acceleration, velocity, and displacement time histories for a typically selected earthquake are illustrated in Figure 2. The same process was repeated for all eleven earthquakes for both components.

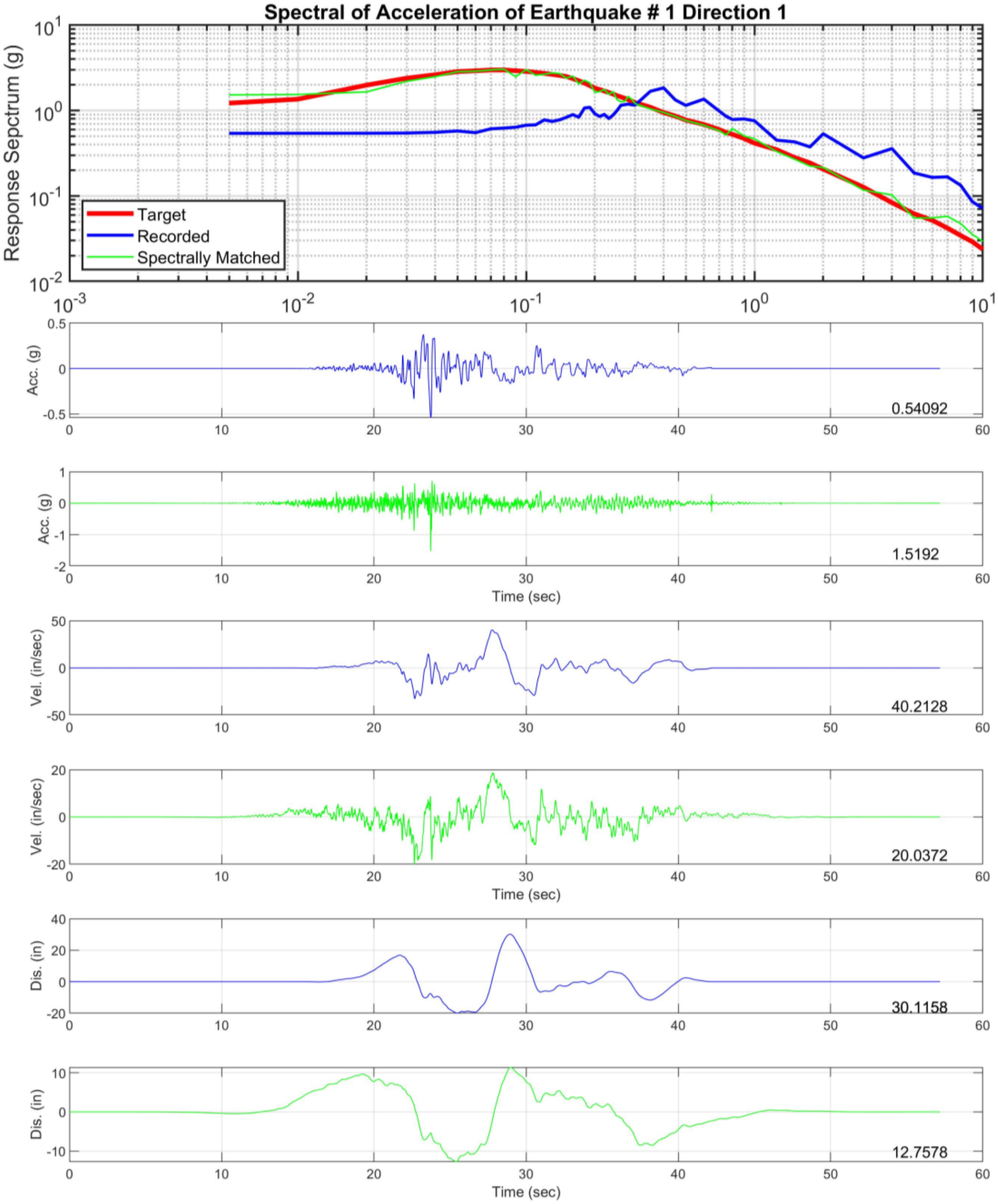


Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			**Mode**		
Period	**M**	**R (km)**	**Period**	**M**	**R (km)**
PGA	7.35	14.10	PGA	7.55	11.63
0.01	7.36	14.23	0.01	7.54	11.73
0.02	7.34	14.49	0.02	7.54	11.53
0.03	7.35	14.57	0.03	7.54	11.53
0.05	7.36	15.48	0.05	7.55	11.77
0.075	7.37	15.70	0.075	7.55	11.73
0.10	7.39	16.49	0.10	7.55	11.74
0.20	7.43	17.83	0.20	7.54	11.64
0.50	7.48	19.98	0.50	7.55	11.79
0.75	7.50	21.51	0.75	7.54	11.72
1.00	7.51	23.00	1.00	7.54	11.70
2.00	7.55	25.79	2.00	7.55	11.51
3.00	7.58	27.19	3.00	7.54	11.40
4.00	7.60	27.87	4.00	7.54	11.74
5.00	7.61	28.41	5.00	7.54	11.81
7.50	7.63	28.85	7.50	7.54	11.66
10.00	7.64	29.57	10.00	7.55	12.00

10.1. Seismic Hazard Analysis

The uniform hazard response spectrum (UHRS) and the magnitude and distance deaggregation for a 7 percent probability of exceedance in 75 years (equivalent to a return period of about 1033 years) are calculated from the PSHA. The seismic hazard is calculated for the uniform firm site condition with 760 m/s shear-wave velocity in the upper 30 m (V_{s30}), representing the boundary between NEHRP site classes B and C.

10.2. Variability in Soil's Shear-Wave and Thickness Profile

A probabilistic characterization of the soil shear-wave velocity profile was used to simulate shear-wave profiles. Two separate components; one for the thickness of each layer called the layering model that captures the variability in the thickness of soil layers, and one for the shear-wave velocity associated with each layer called the velocity model to account for the variability in the shear-wave velocity of each layer are used. A non-homogeneous Poisson model is used with a depth-dependent rate to account for the fact that the soil thickness of layers increases with depth.

In this project, the variability in the shear-wave velocity are considered. The model used statistically captures the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth. A total of 60 cases were generated. These 60 soil profiles are used to capture the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

10.3. Site-Specific Results

Following the procedure outlined above, the site-specific response spectra were obtained, analyzing sixty profiles for each matched ground motion with the UHRS.

The site-specific results were obtained by performing PSHA using all seismic sources and faults and appropriate and recent ground motion prediction equations for Central and Eastern United States following the provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. All uncertainties associated with each aspect of the site-specific analysis were carefully considered. Figure 3 shows the design response spectra, Guide Specifications, and 2/3 of Guide Specifications design spectra. In this figure, the site-specific spectrum is not limited to 2/3 of the Guide Specifications response spectrum for illustration.

Site-specific seismic design recommendations following the Guide Specifications provisions are provided in Table 5 and Table 6. The recommendation is to use the design Sa values provided in Table 5. Figure 4 shows the design response spectra, Guide Specifications, 2/3 of Guide Specifications design spectra, and the site-specific design spectrum constructed based on three periods of PGA, 0.2 sec and 1 sec. In Figure 4, the site-specific response spectrum is adjusted not to be less than 2/3 of the Guide Specifications design response spectrum.

11.0. DESIGN RESPONSE SPECTRAL PARAMETERS

The design spectral response acceleration parameters listed in Table 5 were developed following Guide Specifications.

Table 5. Site-Specific Spectral Acceleration Considering 5% Damping following the Guide Specifications.

Period	Site-Specific Response Spectra
(s)	(g)
0.010	0.864
0.030	0.867
0.040	0.890
0.050	0.953
0.070	1.081
0.100	1.319
0.150	1.593
0.200	1.673
0.250	1.769
0.300	1.727
0.400	1.476
0.500	1.572
0.750	1.458
1.000	1.247
1.500	0.831
2.000	0.551
3.000	0.237
4.000	0.180
5.000	0.141
7.500	0.095
10.000	0.087

Table 6. Site-Specific Response Accelerations Considering 5% Damping.

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	1.673
S_{D1}	1.247
S_{MS}	1.673
S_{M1}	1.247
MCE_G	0.864

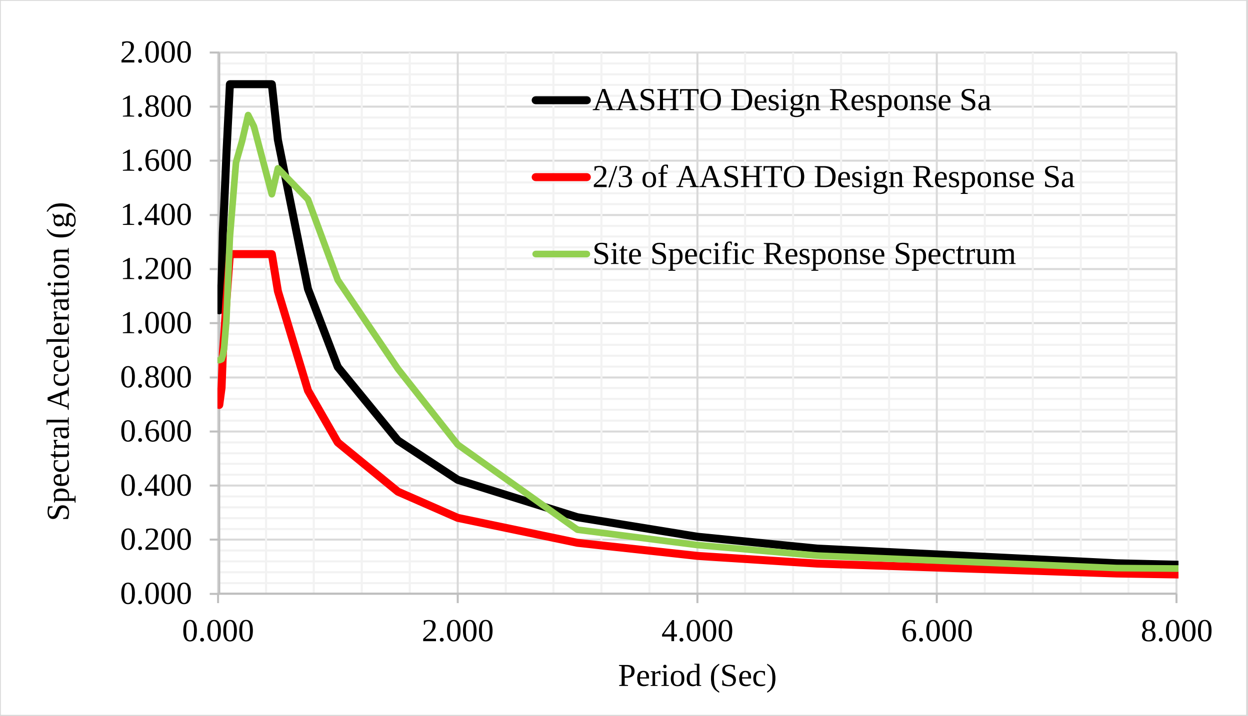


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

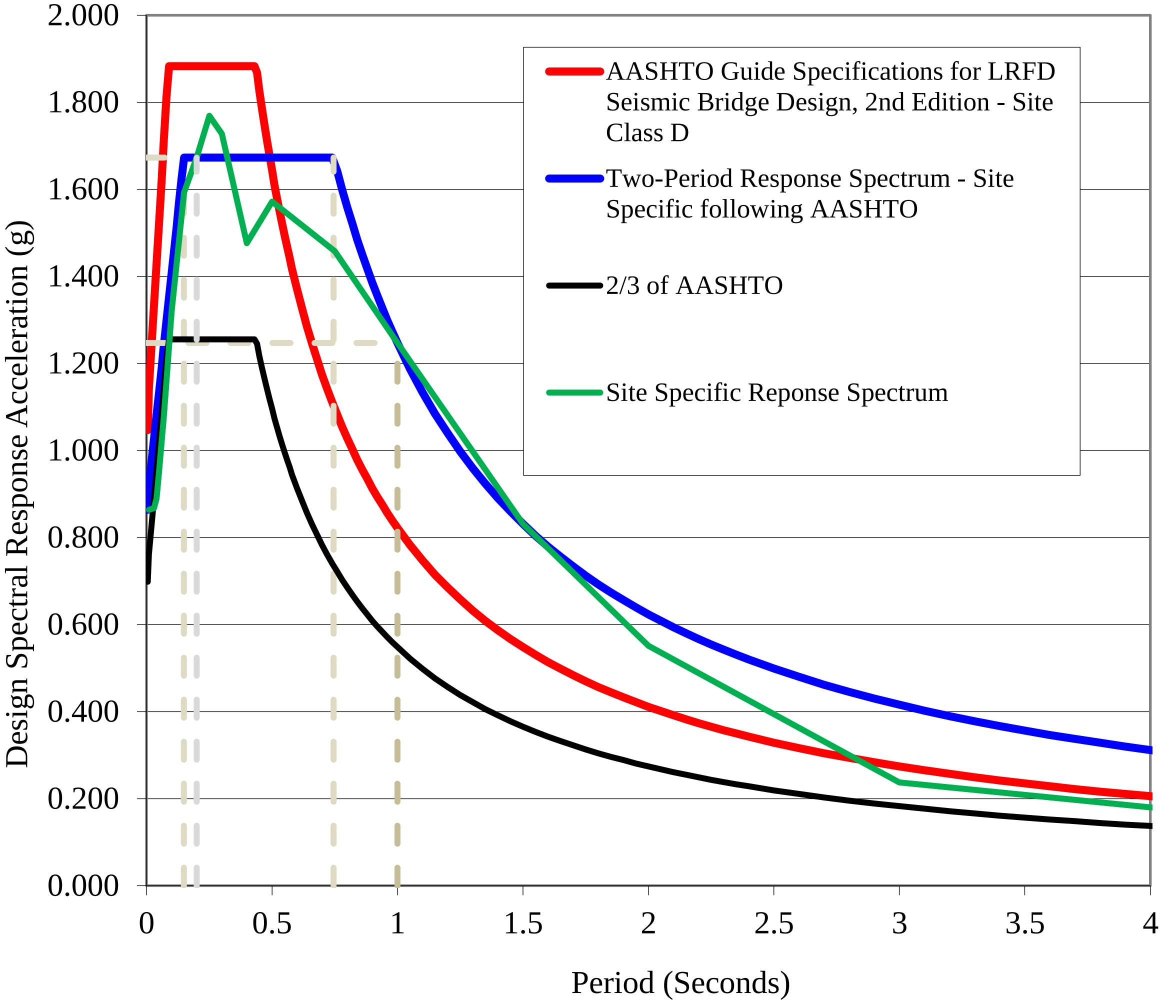


Figure 4. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

12.0 LIMITATIONS OF THE REPORT

The analyses, conclusions, and recommendations presented in this Report are professional opinions based on the site conditions and project layout described herein and further assume that the conditions provided in the geotechnical Report are representative of the subsurface conditions throughout the site, i.e., that the subsurface conditions elsewhere on the site are the same as those disclosed by the borings. If, during construction, subsurface conditions different from those encountered in the exploratory boring are observed or appear to be present, the Client must contact us immediately so that we can make changes to this Report if needed. The scope of our services did not include an assessment of the effects of flooding and natural erosion on the project site. No liquefaction studies were performed. This study is based on the condition that soil will not liquefy.

This Report is copy-righted and was prepared for the exclusive use of the owner, architect, and engineer to evaluate the project's design related to the ground response discussed in this Report.

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APPENDIX A. Site Location

Figure A.1. The Location of the Study Site.

Site-Specific Seismic Study
Tyronza River Site
Poinsett County, Arkansas

By

Shahram Pezeshk, Ph.D., P.E.
Email: s.pezeshk@aol.com
901-606-6934

June 17, 2023

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Site-Specific Seismic Study
Tyronza River Site
Poinsett County, Arkansas

1.0. EXECUTIVE SUMMARY

The executive summary provides an overview of my understanding of the project and recommendations. Information and recommendations presented in the executive summary should not be used without reviewing the entire Report.

- The location of the study site is 35.50571°N and 90.32291°W (See Appendix A).
- Based on the recommendations of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, A_S (zero-period), S_{DS} (short period), and S_{D1} (long period) are provided in Table 3.
- Site-specific recommendations following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions are provided in Table 5 and Table 6.

2.0. SCOPE OF WORK

The purpose of our study is to estimate the design spectra following the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. The structural design of new buildings allows two procedures for determining design ground motions:

1. *General Procedure*. In this method, the response spectrum is determined using the following steps: (1) develop the rock spectrum using seismic design maps for values of Peak Ground Acceleration (PGA) and spectral acceleration at periods of 0.2 and 1.0 seconds; (2) determine the Site Class using the shear-wave velocity (V_s) measurements from the upper 100 feet of the soil profile, and (3) adjust the rock spectrum for site class to develop the general response spectrum.

2. *Site-Specific Procedure*. In this method, the response spectrum is determined using a combination of probabilistic seismic hazard and site response analyses. The site-specific response spectrum may not be less than 2/3 of the general response spectrum.

Briefly, the scope of our services for the site-specific investigation included the following steps:

1. Perform probabilistic seismic hazard analysis (PSHA) to estimate ground motions in the rock underlying the site;
2. Determine Uniform Hazard Response Spectrum (UHRS) at the rock level;
3. Determine probabilistic consistent magnitude and distances from deaggregation;
4. Select ground motions consistent with magnitude and distances obtained in step 3;
5. Perform spectral matching to match the selected ground motions to the UHRS of Step 2;
6. Perform one-dimensional equivalent linear site-specific ground response analysis using the site-specific earthquake time histories by using the computer program SHAKE91 (Idriss and Sun, 1992) and considering the uncertainties associated with the shear-wave velocity and layer thicknesses for the soil profile; and
7. Develop site-specific response spectra for the existing subsurface conditions using the procedure outlined in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2022 Interim Revisions, based on 7 percent probability of exceedance in 75 years and 5 percent damping for a single degree of freedom (SDOF) structure.

3.0. SUBSURFACE CONDITIONS

This study is based on the available information on the soil stratigraphy provided by Geotechnology and the shear-wave velocity profile obtained using Seismic Cone Penetration Testing (SCPT).

4.0. SHEAR-WAVE VELOCITY PROFILE

Seismic Cone Penetration Testing (SCPT) was performed by Geotechnology (a UES Company). Table 1 provides the shear-wave velocity obtained from SCPT. To construct a base-case profile, the velocity profile was extemded to a deeper geologic unit that represents the reference site condition. To extend the shallower portion of the velocity profile to the deeper portion, the 3D velocity model developed for Central United States (CUS) was used. The CUS 3D velocity model has been developed by Ramirez-Guzman *et al.* (2012) and is a result of several efforts in previous years including Allen and Wald (2007), Chung and Rogers (2010), Cramer *et al.* (2004), Ginzburg *et al.* (1983), Gomberg *et al.* (2003), Mooney *et al.* (1983), Prodehl *et al.* (1984), and Stewart (1968).

Table 1. Shear-Wave Velocities Measured.

Depth1 (ft)	**Depth2 (ft)**	**V_s (ft/sec)**
1.61	4.92	695.20
4.92	8.20	599.26
8.20	11.51	680.57
11.51	14.79	498.56
14.79	18.07	425.19
18.07	21.35	392.85
21.35	24.67	485.57
24.67	27.91	543.14
27.91	31.19	544.55
31.19	34.41	551.01
34.41	37.65	676.83
37.65	40.90	741.38
40.90	44.15	771.46
44.15	47.40	758.11
47.40	50.64	718.42
50.64	54.02	768.83
54.02	57.27	683.22
57.27	60.55	840.17
60.55	63.76	807.24
63.76	67.01	859.29
67.01	70.26	1022.93
70.26	73.50	962.42
73.50	76.82	916.73
76.82	80.16	936.51
80.16	83.34	943.89
83.34	86.66	917.48
86.66	89.94	939.95
89.94	93.25	995.28
93.25	96.53	929.58
96.53	100.01	894.10

5.0 GENERAL INFORMATION

For this project, we have been requested to perform a site-specific seismic study to produce the ground surface response spectrum and a set of time series based on the seismic parameters used in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions, which include: seismic hazards related to 7 percent probability of exceedance in 75 years and 5 percent damping for SDOF structure.

6.0. REGIONAL SEISMICITY

Petersen et al. (2019) used fault models from the 2014 NSHM to model large earthquakes and apply gridded, smoothed seismicity models from an earthquake catalog to account for smaller earthquakes on and off the faults. They developed new seismicity catalogs for the CEUS and WUS, including earthquakes from 2013 through 2017 that occurred since the last model was constructed. Between 2013, when the catalog was last updated, and 2018, strongly felt earthquakes (magnitude 4+) occurred in almost half of the states in the United States. Figure 1 shows the USGS 2018 declustered catalog for CEUS.

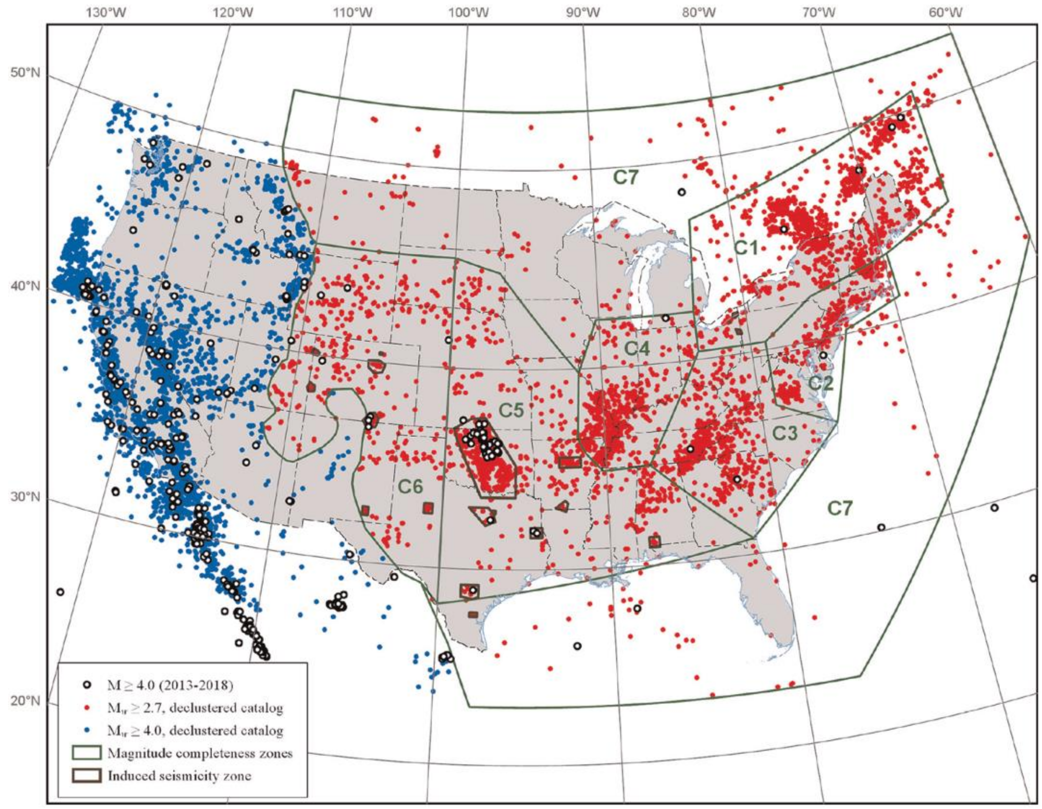


Figure 1. The 2018 NSHM Declustered Catalog for Central and Eastern United States (red) and Western United States (blue).

7.0. SEISMIC HAZARD ANALYSIS

A PSHA was performed to estimate the seismic ground motions for a rock site condition. The analytical model used for the PSHA is based on models developed initially by Cornell (1968). These models' underlying assumption is that earthquakes occur in space and time within a particular seismic zone is entirely random (i.e., a Poisson process). This type of probabilistic model is commonly used for seismic hazard analyses of essential facilities throughout the world.

The two primary components of the probabilistic model are:

1. The seismic source models specify the spatial, temporal, and magnitude distribution of earthquake occurrences expected in each of the seismic sources, and

2. The ground-motion attenuation models which determine the distribution of ground motions expected at the site for a potential earthquake occurrence (characterized by magnitude and location, and usually by other factors) on a seismic source.

The above two components comprise the inputs to the PSHA. In the PSHA, probability-of-exceedance rates (hazard curves) are computed for a range of horizontal ground motions. These ground motions are expressed in terms of peak ground acceleration (PGA) and 5 percent-damped pseudo absolute spectral accelerations (S_a) at various single-degree-of-freedom oscillator periods. From the probability-of-exceedance rates, the Uniform Hazard Response Spectrum (UHRS) corresponding to average return periods of 7% probability of exceedance in 75 years is computed.

7.1. SEISMIC SOURCE MODELS

The USGS seismic source models have been used for this project. The USGS addressed the causes of earthquakes in the Central and Eastern United States in two ways: (1) earthquake fault; and (2) background or smoothed seismicity models, which forecast the occurrence rates and magnitudes of potential seismic events.

7.2. GROUND MOTION MODELS

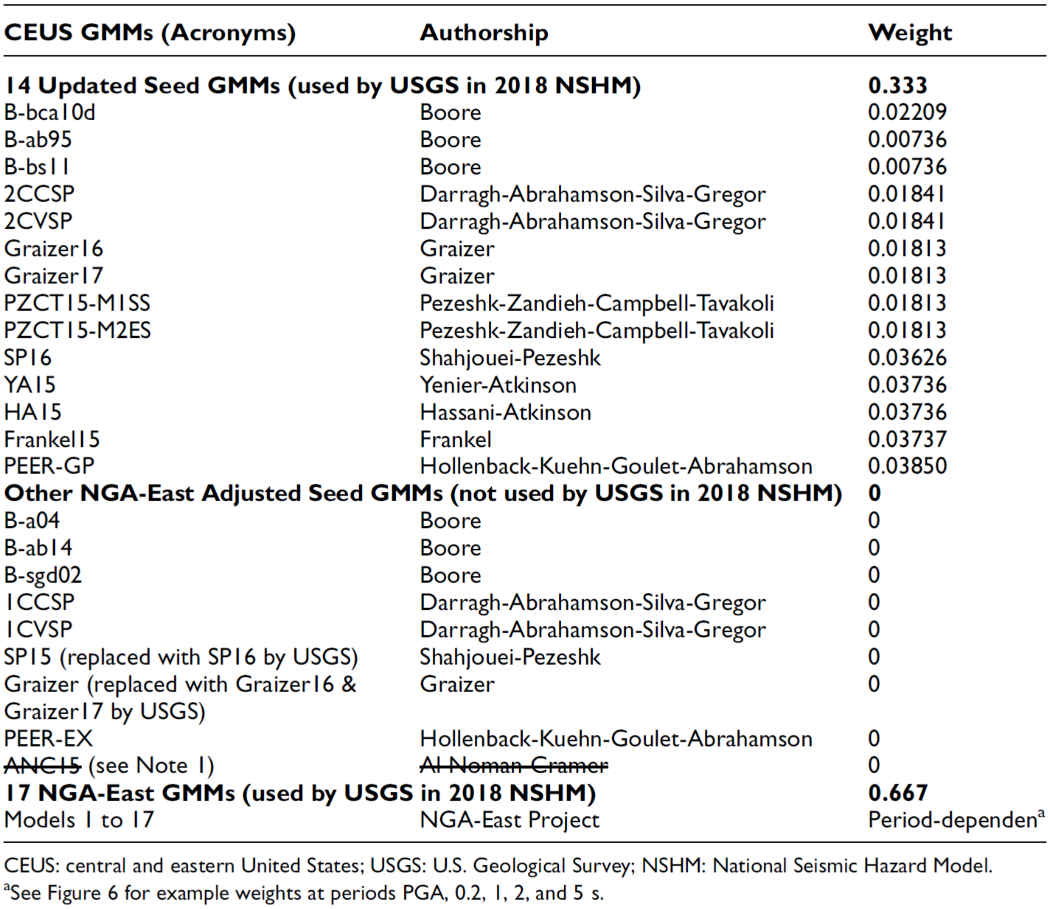
In general, the characteristics of the fault source, such as distance, type, magnitude, and site conditions, are used to estimate the magnitude of an earthquake parameter (spectral acceleration, peak ground acceleration, etc.) via ground-motion models (GMMs) or ground-motion prediction equations (GMPEs), also known as attenuation relationships. Various attenuation relationships have developed for specific regions using a database of appropriate ground motion records.

Petersen et al. (2020a) presented only a summary of the CEUS GMM updates, which included comparisons of the 2018 weighted median GMMs to the 2014 National Seismic Hazard Model (NSHM) and an overview of the aleatory variability (GMM standard deviation) and site-effect models. Rezaeian et al. (2021) discuss the CEUS GMM updates and implementation in the 2018 NSHM in detail. These updates consist of (1) 31 new GMMs, including the state-of-the-art Next Generation Attenuation relationships for central and eastern North America (NGA-East) (Goulet et al., 2018, 2017, 2021; Pacific Earthquake Engineering Research Center (PEER), 2015a), (2) an associated model of aleatory variability (based on Al Atik, 2015; Goulet et al., 2017; Stewart et al., 2019), and (3) a new site-effect model (for amplification or deamplification) specific to the CEUS (Hashash et al., 2020; Stewart et al., 2020). In the following, we discuss the individual GMMs in terms of their medians, assigned weights, weighted averages, attenuations with distance, and epistemic uncertainty.

According to Rezaeian et al. (2021), NSHM 2018 was updated to generate national seismic hazard maps for the Central and Eastern United States. The logic tree weights are based on the distance and the geometric spreading term used by each model. The models with a faster geometric spreading term are given more weight. The New Madrid seismic zone is the most likely seismic source that could affect the considered site. NSHM removed the attenuation relationships not applicable beyond 500 km, and weights were renormalized.

Table 2 lists the selected GMMs from the NSHM 2018 models with their associated weights. Three of the models were developed by Pezeshk and his colleagues [Pezeshk et al. 2015; 2018 (PZCT15-M1SS, PZCT15-M2ES), Shajouei and Pezeshk (2016) (SP16)].

Table 2. Ground Motion Models (GMMs).



CEUS GMMs (Acronyms)	**Authorship**	**Weight**
14 Updated Seed GMMs (used by USGS in 2018 NSHM)		**0.333**
B-bca10d	Boore	0.02209
B-ab95	Boore	0.00736
B-bs11	Boore	0.00736
2CCSP	Darragh-Abrahamson-Silva-Gregor	0.01841
2CVSP	Darragh-Abrahamson-Silva-Gregor	0.01841
Graizer16	Graizer	0.01813
Graizer17	Graizer	0.01813
PZCT15-M1SS	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
PZCT15-M2ES	Pezeshk-Zandieh-Campbell-Tavakoli	0.01813
SP16	Shahjouei-Pezeshk	0.03626
YA15	Yenier-Atkinson	0.03736
HA15	Hassani-Atkinson	0.03736
Frankel15	Frankel	0.03737
PEER-GP	Hollenback-Kuehn-Goulet-Abrahamson	0.03850
Other NGA-East Adjusted Seed GMMs (not used by USGS in 2018 NSHM)		**0**
B-a04	Boore	0
B-ab14	Boore	0
B-sgd02	Boore	0
1CCSP	Darragh-Abrahamson-Silva-Gregor	0
1CVSP	Darragh-Abrahamson-Silva-Gregor	0
SP15 (replaced with SP16 by USGS)	Shahjouei-Pezeshk	0
Graizer (replaced with Graizer16 & Graizer17 by USGS)	Graizer	0
PEER-EX	Hollenback-Kuehn-Goulet-Abrahamson	0
~~ANC15~~ (see Note 1)	~~Al Noman-Cramer~~	0
17 NGA-East GMMs (used by USGS in 2018 NSHM)		**0.667**
Models 1 to 17	NGA-East Project	Period-dependen[a]

CEUS: central and eastern United States; USGS: U.S. Geological Survey; NSHM: National Seismic Hazard Model.

[a]See Figure 6 for example weights at periods PGA, 0.2, 1, 2, and 5 s.

7.3. TREATMENT OF UNCERTAINTIES

Seismic-hazard studies distinguish between two types of uncertainty, namely epistemic and aleatory. Aleatory uncertainty is probabilistic variability that results from a natural physical process. For example, the size, location, and time of the next earthquake on a fault and the details of the ground motion are considered aleatory uncertainties. In advanced seismic hazard studies, integration is performed over aleatory uncertainties to get a single hazard curve—the epistemic uncertainty results from a lack of knowledge about earthquakes and their effects. In principle, epistemic uncertainties are addressed by multiple models and parameters. The most well-known epistemic uncertainties associated with the input parameters in seismic hazard analysis include the uncertainties in seismic source models (i.e., tectonic stresses, geological features, geometries, etc.), seismicity (i.e., activity rate, slip rate, etc.), and attenuation relationships (source, path, and site effects). The USGS 2014 procedure (Petersen *et al.*, 2014) is followed in this project to address the uncertainty in seismic-source characterization, which is quantified by considering alternative geometries, multiple magnitude-recurrence parameters, and multiple maximum magnitudes.

8.0. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Time-averaged shear-wave velocity in the top 100 ft (30 m) is defined as V_{S30}. The V_{S30} for the study site is determined to be 701 ft/sec, which according to the Guide Specifications, the study site is determined to be a Site Class "D" (Table 3.4.2.1-1, Site Class Definitions). Site coefficients F_{pga}, F_a, and F_v for the study site following Tables 3.4.2.3-1 and 3.4.2.302 mapped spectral acceleration are summarized in Table 3.

8.1. Dynamic Soil Properties

Low-strain soil shear modulus and damping are the required dynamic soil properties for seismic ground response analysis. A brief discussion of these properties is given below.

8.1.1. Low Strain Soil Shear Modulus

A key parameter necessary to evaluate the dynamic response of soils is the dynamic shear modulus, $G_{s,}$ or shear wave velocity, which is also related to the dynamic shear modulus. Values of shear wave velocity or shear modulus can be determined either by measuring in the laboratory on undisturbed soil samples or by performing seismic field tests. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than 10^{-4} percent), referred to as G_{max}, is the desired design parameter.

Laboratory measurement of shear wave velocity or low-strain soil shear modulus was beyond the scope of our services. Various correlations and typical values are available in the literature to estimate the approximate value of shear-wave velocity and G_{max}.

8.1.2. Damping

The inelastic behavior of soil (discussed later) also gives rise to the energy absorption characteristics of soil, known as material damping. Damping is generally expressed as a percentage of critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the study based on the strain levels in the soil, as explained in subsequent sections of this Report.

8.1.3. Effect of Strain on Dynamic Soil Properties

It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus is not a constant value but degrades nonlinearly with increasing strain in the soil. Dynamic analyses considering the true nonlinear behavior of soil are complicated and are an active and current research area. Accordingly, an equivalent linear analysis is typically used in practice. Equivalent linear analyses consist of performing a series of linear analyses in an iterative process, using, for each analysis, soil properties consistent with the strains resulting from the previous one. An equivalent linear site response analysis is used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain.

9.0. CODE-BASED DESIGN APPROACH

9.1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2022 Interim Revisions

Using the United States Geological Survey (USGS) Hazard Maps and the project location, the mapped 0.2-second spectral response acceleration (S_s) and the mapped 1.0-second spectral response acceleration (S_1) are provided in Table 3. Based on the average shear-wave velocities of the top 100 ft of soil, the site class has been determined to be site class “D.” Based on the mapped spectral acceleration and site class D, the site coefficients F_{PGA}, $F_{a,}$ and F_v are provided in Table 3. provides a summary of these parameters.

Table 3. Mapped Provisional Design Response Spectrum Parameters at 5% Damping.

Parameter	Value
F_a	1.000
F_v	1.545
F_{PGA}	1.000
S_S	1.726
S_1	0.455
S_{DS}	1.726
S_{D1}	0.703
PGA	0.978
A_s	0.978

10.0. SITE-SPECIFIC PROCEDURE

The probabilistic seismic hazard analysis (PSHA) considers all potential earthquake sources that will contribute to hazards at a specific site. The PSHA factors in contributions from all magnitudes, distances, and probability of occurrence for all sources. This study used PSHA to estimate PGA and spectral acceleration at various periods for a B/C NEHRP site condition for a 7% probability of exceedance in 75 years.

The PSHA was performed to obtain a uniform hazard response spectrum (UHRS). The PSHA and de-aggregation results were used to select earthquakes for the site response analyses. Eleven horizontal components (total of 11) of previously recorded earthquakes within the range of de-aggregation magnitudes and distances were selected.

Table *4* provides the mean and the modal deaggregation magnitude and distances for various periods. The UHRS was selected as the target spectrum, and the chosen time histories were matched with the target spectrum. As an example, acceleration, velocity, and displacement time histories for a typically selected earthquake are illustrated in Figure 2. The same process was repeated for all eleven earthquakes for both components.

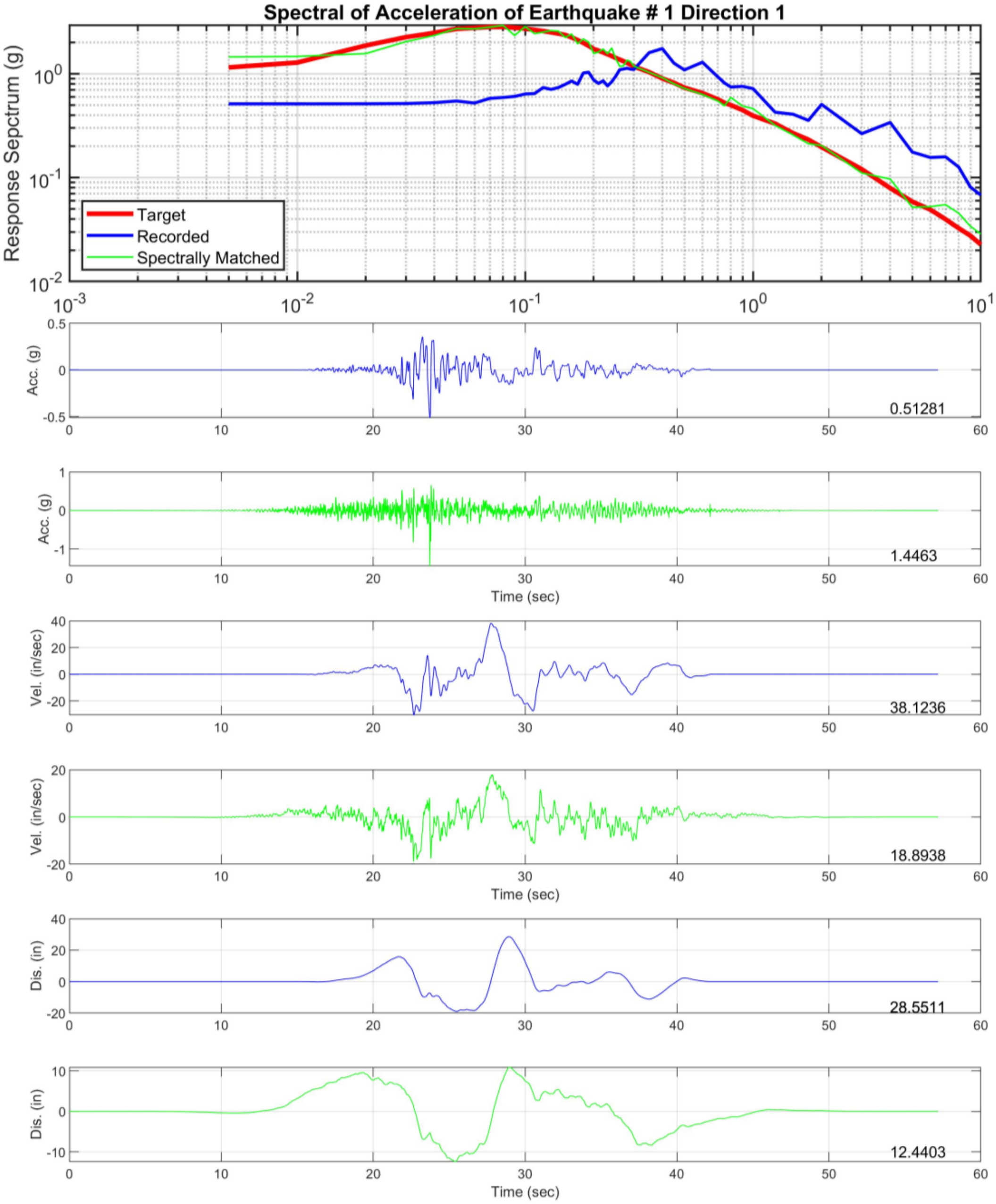


Figure 2. Time Histories Before and After the Spectral Matching Process for Earthquake #1. The numbers Shown in the Bottom right of Each Figure Represent the Absolute Maximum Value of the Graph.

Table 4. Deaggregation.

Mean and Mode Deaggregation Parameter at 1,033 Years					
Mean			**Mode**		
Period	**M**	**R (km)**	**Period**	**M**	**R (km)**
PGA	7.33	15.45	PGA	7.53	13.87
0.01	7.34	15.55	0.01	7.55	13.25
0.02	7.32	15.76	0.02	7.55	13.20
0.03	7.33	15.78	0.03	7.55	13.39
0.05	7.34	16.67	0.05	7.54	13.42
0.075	7.35	17.09	0.075	7.54	13.35
0.10	7.37	17.94	0.10	7.54	13.42
0.20	7.42	19.56	0.20	7.55	13.09
0.50	7.47	22.10	0.50	7.56	12.97
0.75	7.49	23.78	0.75	7.56	12.81
1.00	7.51	25.47	1.00	7.55	13.23
2.00	7.55	28.68	2.00	7.54	13.46
3.00	7.58	30.12	3.00	7.55	13.28
4.00	7.60	30.91	4.00	7.54	13.44
5.00	7.61	31.70	5.00	7.54	13.49
7.50	7.63	32.24	7.50	7.54	13.58
10.00	7.64	33.11	10.00	7.54	13.44

10.1. Seismic Hazard Analysis

The uniform hazard response spectrum (UHRS) and the magnitude and distance deaggregation for a 7 percent probability of exceedance in 75 years (equivalent to a return period of about 1033 years) are calculated from the PSHA. The seismic hazard is calculated for the uniform firm site condition with 760 m/s shear-wave velocity in the upper 30 m (V_{s30}), representing the boundary between NEHRP site classes B and C.

10.2. Variability in Soil's Shear-Wave and Thickness Profile

A probabilistic characterization of the soil shear-wave velocity profile was used to simulate shear-wave profiles. Two separate components; one for the thickness of each layer called the layering model that captures the variability in the thickness of soil layers, and one for the shear-wave velocity associated with each layer called the velocity model to account for the variability in the shear-wave velocity of each layer are used. A non-homogeneous Poisson model is used with a depth-dependent rate to account for the fact that the soil thickness of layers increases with depth.

In this project, the variability in the shear-wave velocity are considered. The model used statistically captures the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth. A total of 60 cases were generated. These 60 soil profiles are used to capture the soil layer shear-wave velocity and thickness uncertainties and their correlation with depth.

10.3. Site-Specific Results

Following the procedure outlined above, the site-specific response spectra were obtained, analyzing sixty profiles for each matched ground motion with the UHRS.

The site-specific results were obtained by performing PSHA using all seismic sources and faults and appropriate and recent ground motion prediction equations for Central and Eastern United States following the provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition with 2022 Interim Revisions. All uncertainties associated with each aspect of the site-specific analysis were carefully considered. Figure 3 shows the design response spectra, Guide Specifications, and 2/3 of Guide Specifications design spectra. In this figure, the site-specific spectrum is not limited to 2/3 of the Guide Specifications response spectrum for illustration.

Site-specific seismic design recommendations following the Guide Specifications provisions are provided in Table 5 and Table 6. The recommendation is to use the design Sa values provided in Table 5. Figure 4 shows the design response spectra, Guide Specifications, 2/3 of Guide Specifications design spectra, and the site-specific design spectrum constructed based on three periods of PGA, 0.2 sec and 1 sec. In Figure 4, the site-specific response spectrum is adjusted not to be less than 2/3 of the Guide Specifications design response spectrum.

11.0. DESIGN RESPONSE SPECTRAL PARAMETERS

The design spectral response acceleration parameters listed in Table 5 were developed following Guide Specifications.

Table 5. Site-Specific Spectral Acceleration Considering 5% Damping following the Guide Specifications.

Period	Site-Specific Response Spectra
(s)	(g)
0.010	0.769
0.030	0.771
0.040	0.779
0.050	0.797
0.070	0.856
0.100	1.001
0.150	1.182
0.200	1.565
0.250	1.474
0.300	1.739
0.400	1.621
0.500	1.730
0.750	1.480
1.000	1.200
1.500	0.800
2.000	0.539
3.000	0.260
4.000	0.157
5.000	0.123
7.500	0.095
10.000	0.074

Table 6. Site-Specific Response Accelerations Considering 5% Damping.

PARAMETER	DESIGN ACCELERATION PARAMETERS (g)
S_{DS}	1.565
S_{D1}	1.197
S_{MS}	1.565
S_{M1}	1.200
MCE_G	0.769

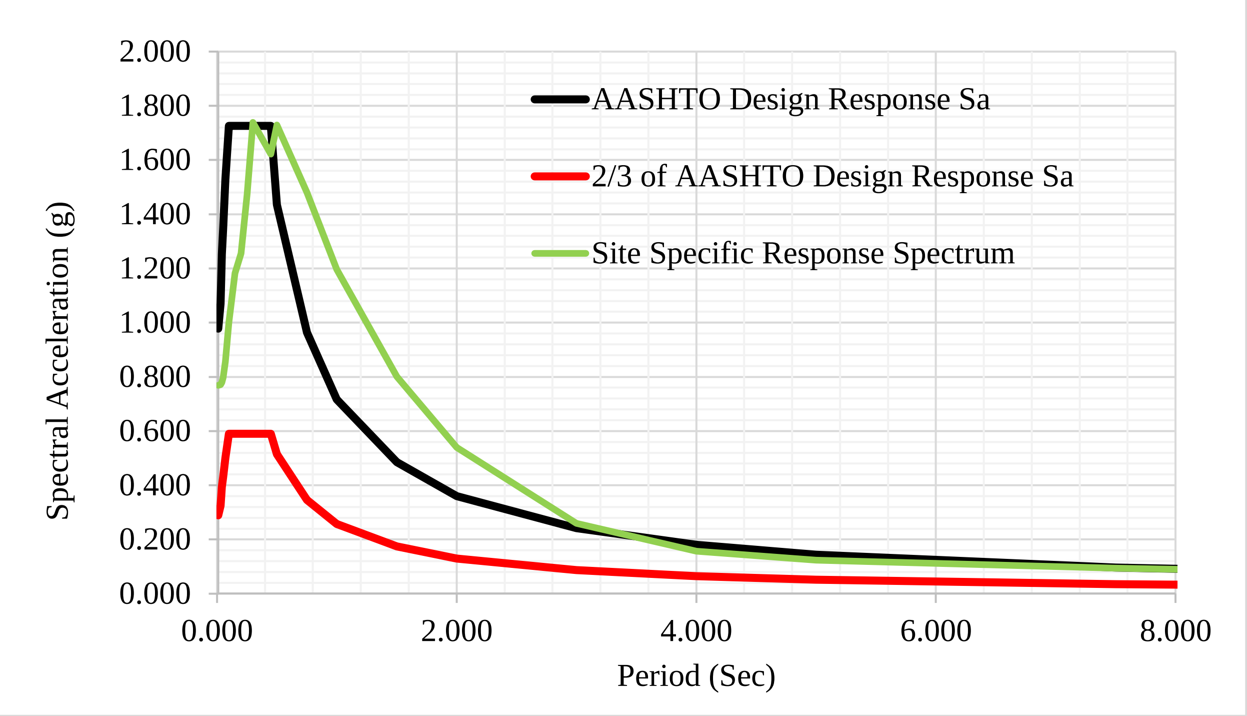


Figure 3. Site-Specific Design Response Spectrum, AASHTO Guide Specifications Design Response Spectrum, and 2/3 of the AASHTO Guide Specifications Design Response Spectrum.

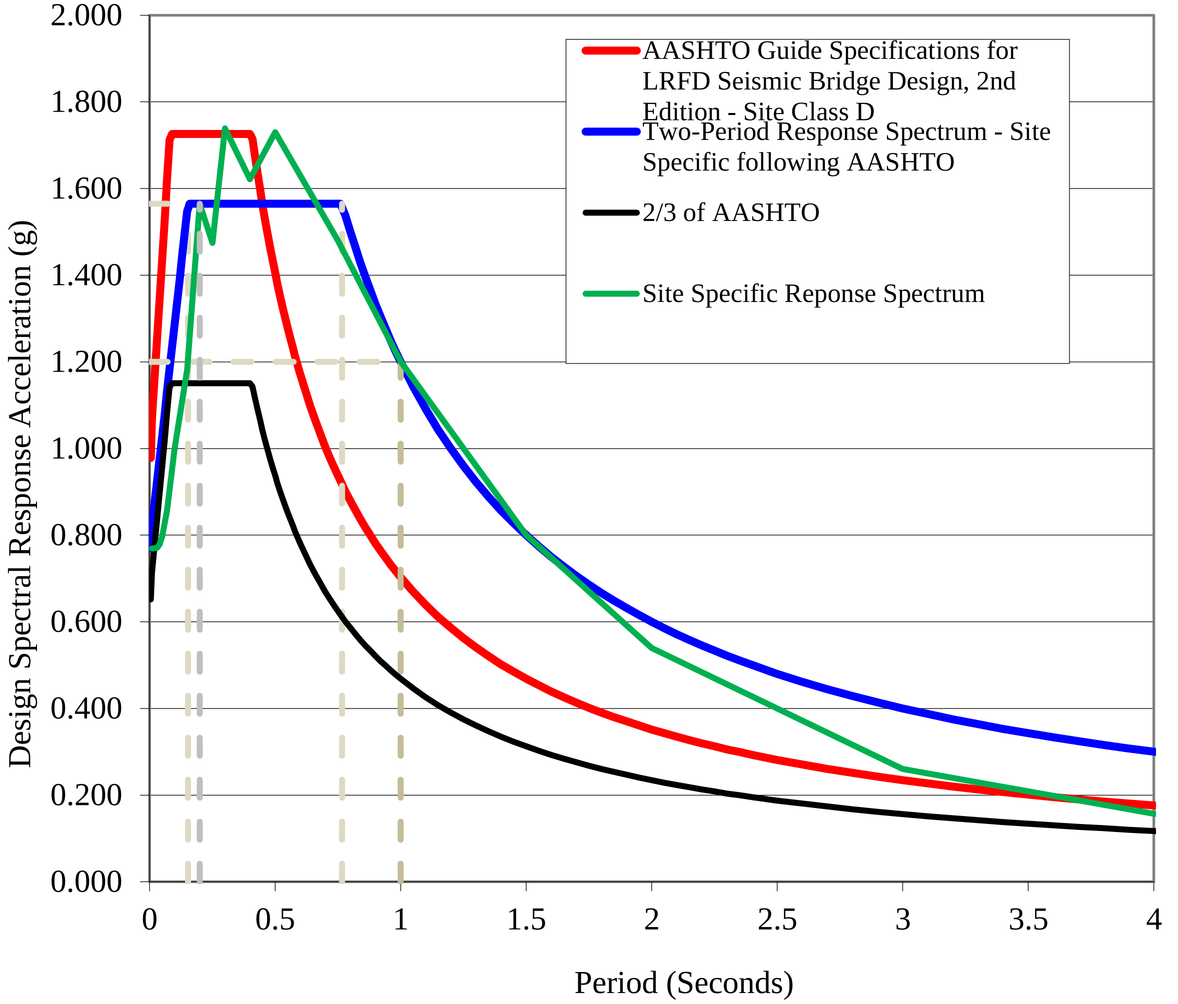


Figure 4. Design Response Spectrum based on AASHTO Guide Specifications, 2/3 of the AASHTO Guide Specifications Site-Specific, and Design Response Spectrum Based on PGA, 0.2, and 1 Second.

12.0 LIMITATIONS OF THE REPORT

The analyses, conclusions, and recommendations presented in this Report are professional opinions based on the site conditions and project layout described herein and further assume that the conditions provided in the geotechnical Report are representative of the subsurface conditions throughout the site, i.e., that the subsurface conditions elsewhere on the site are the same as those disclosed by the borings. If, during construction, subsurface conditions different from those encountered in the exploratory boring are observed or appear to be present, the Client must contact us immediately so that we can make changes to this Report if needed. The scope of our services did not include an assessment of the effects of flooding and natural erosion on the project site. No liquefaction studies were performed. This study is based on the condition that soil will not liquefy.

This Report is copy-righted and was prepared for the exclusive use of the owner, architect, and engineer to evaluate the project's design related to the ground response discussed in this Report.

13.0 REFERENCES

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APPENDIX A. Site Location

Figure A.1. The Location of the Study Site.