ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. BB0903

FEDERAL AID PROJECT NO. NHPP-540-1(78)85

HWY. 71B INTCHNG. IMPVTS. (S)

STATE HIGHWAY I-49 SECTION 29

IN BENTON COUNTY

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June 16, 2016
Job No. 12-071

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Attn: Mr. Mike Burns, P.E.

RESULTS of GEOTECHNICAL INVESTIGATION
TASK ORDER No. B005
AHTD JOB BB0903 – HWY 71B INTERCHANGE IMPROVEMENTS
ROGERS/BENTONVILLE, BENTON COUNTY, ARKANSAS

INTRODUCTION

Submitted herein are the results of the geotechnical investigation performed for the Hwy 71B interchange improvements in Benton County, Arkansas. The study has been performed as part of Task Order No. B005. These services were authorized by the Crafton Tull & Associates, Inc. subconsultant agreement dated July 13, 2013. A geotechnical study was initially performed in 2015 and the results of that study were provided in the report dated August 24, 2015. Since that time the project concept has been revised.

We understand that the project consists of replacing the existing I-49 bridges (Bridges A5977 and B5977) over Hwy 71B. The replacement bridge will include three (3) lanes on both the southbound (A) and northbound (B) bridges over Hwy 71B (SE Walton Boulevard). The I-49 Bridge will include 10-ft-wide inside shoulders and a 16-ft-wide interior median will be included. A simple composite plate girder span with a total length of approximately 252 ft is planned. In addition, Hwy 71B will be widened. The widened roadway lanes will be accommodated by constructing MSE walls at the I-49 bridge ends, with the walls returning back along the interstate alignment to accommodate the grade changes.

The purposes of this geotechnical study were to explore subsurface conditions at the bridge widening location and in the wall alignments as needed to develop recommendations to guide design and construction of foundations and MSE walls. The data developed through the field and laboratory studies have been utilized in developing the conclusions and recommendations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface Investigation

Subsurface conditions were initially explored in June 2015 by drilling four (4) sample borings to 60- to 75-ft depth (Borings S1 through S4). Following notification of the change in the interchange concept, eight (8) additional borings (Borings W5 through W12) were drilled for use in developing geotechnical recommendations for the new walls. The boring locations were selected by the Engineer. The boring locations were then field adjusted as required for drill rig access. In some cases, the wall borings were offset to the toe of the existing embankments to facilitate safe equipment access. The bridge and wall boring locations were staked in the field by Grubbs, Hoskyn, Barton & Wyatt, Inc. (GHBW).

The project vicinity is shown on the attached Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2. Boring logs, showing descriptions of the soil and rock strata encountered and results of the field and laboratory tests, are included as Plates 3 through 14. The ground surface elevation at each boring location, as provided by the Engineer or inferred from the available topographic information, is also shown on each log. It must be recognized that the inferred elevations shown on the wall boring logs are approximate and actual elevations may vary. Keys to the terms and symbols used on the logs are presented on Plates 15 and 16 for soil and rock, respectively. The subsurface exploration program is summarized on Plate 17. A generalized subsurface profile is provided in Appendix A.

The borings were drilled with truck-mounted SIMCO 2800 and Mobil B-53 rotary-drilling rigs using a combination of dry-auger and rotary-wash drilling procedures. Soil samples were typically obtained in the borings at 2-ft intervals to a depth of 10 ft and at 5-ft intervals thereafter. Samples were typically obtained using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb safety hammer (with the Mobile B-53) or an automatic hammer (with the SIMCO 2800) with 30-in. drop 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). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column. Where rock hardness precluded recovery via the SPT, cuttings were obtained for use in visual classification.

Due to the rocky overburden soils, a core barrel could not be advanced downhole on the structural borings. Consequently, no rock coring was successfully performed.

All samples were extruded or otherwise removed from samplers in the field. Samples were visually classified by the geotechnical technician and placed in appropriate containers to prevent moisture loss and/or disturbance 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. Groundwater levels were measured during drilling operations. Observations regarding groundwater are noted in the lower-right portion of each log and are discussed in subsequent sections of this report.

LABORATORY TESTING

Laboratory testing was performed to evaluate pertinent physical and engineering characteristics of the subgrade and foundation soil and rock. The laboratory testing program included natural water content determinations and classification tests. A total of 87 natural water content determinations (AASHTO T 265) were performed to develop data on *in-situ* soil water contents 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, 26 liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90) and 28 sieve analyses (AASHTO T 88) 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 percent of soil passing the No. 200 Sieve is noted in the "No. 200%" column on the log forms. Classification test results, as well as soil classification by the Unified Soil Classification System (ASTM D-2487) and AASHTO classification system (AASHTO M 145), are summarized in Appendix B.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The project location is the I-49 and Hwy 71B (SE Walton Boulevard) interchange in Benton County, Arkansas. The existing structures are two-lane twin bridges. The bridges are constructed on earthen embankments with concrete riprap covered end slopes and grass-covered side slopes. Walton Boulevard at the I-49 bridge location is a five-lane major arterial roadway. The project locale is predominantly commercial development. As noted, the bridges cross over Walton Boulevard via embankments. The surrounding terrain is generally flat.

Site Geology

The Geologic Map of Arkansas[1] indicates that the I-49 bridge location is in the mapped outcrop of the early and middle Mississippian Period Boone Formation. The Boone Formation consists of limestone, chert and cherty limestone. The chert and limestone content varies widely, both horizontally and vertically. The limestone of the Boone is typically gray, compact, finely to coarsely crystalline, and massively bedded. The limestone of the Boone is nearly pure calcium carbonate and is soluble. As a result, sinkholes, caves and fissures can occur in the formation. The discontinuities in the rock mass are generally filled, or partially filled, with chert boulders, clay, and stalactitic and stalagmitic material. The chert in the Boone Formation is cryptocrystalline silica of organic origin. The chert may occur as widely separated nodules, connected nodules, in interbedded layers with limestone, and sometimes as beds. Unweathered chert is dense, hard, and brittle and exhibits a conchoidal fracture.

The Boone limestone typically weathers to red clay with numerous chert fragments, cobbles and boulders and discontinuous chert seams and layers (cherty clay). Though the residual clay often exhibits high plasticity, the residual soils typically classify as GC, clayey gravel, by the Unified Soil Classification System.

Seismic Conditions

Based on the site geology, the average soil and rock conditions revealed by the borings, and our experience in the area, a Seismic Site Class C (very dense soil and soft rock profile) is considered fitting for the widened I-49 bridge site with respect to the criteria of the AASHTO LRFD Bridge Design Specifications Seventh Edition 2014[2]. Given the bridge location and AASHTO code-based values, the 1.0-sec period spectral acceleration coefficient for Site Class B (S_1) is 0.051 and the 1.0-sec period spectral acceleration coefficient (S_{D1}) value for Site Class C is 0.087. Utilizing these parameters, Table 3.10.6-1[3] indicates that a Seismic Performance Zone 1 is fitting for the I-49 / Hwy 71B interchange bridge site. In reference to the 2014 edition of the AASHTO Guide Specifications, the Peak Ground Acceleration (PGA) having a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years) is predicted to be 0.059 for a Seismic Site Class C for the bridge location.

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[1] Geologic Map of Arkansas, Arkansas Geologic Commission and U.S. Geologic Survey; 1993

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

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

Subsurface Conditions

Based on the results of the borings performed at the I-49 / Hwy 71B interchange bridge site, the subsurface conditions at the bridge location may be generalized into the following strata.

Stratum I: The surface and near-surface soils are embankment fill and on-site fill. At the bridge end embankments the fill extends to 22- to 28-ft depth (approximately El 1284 to El 1280). At the roadway grades (see Borings S2 and S3), the fill extends from 2- to 4-ft depth. The fill is comprised of firm to very stiff red to red and brown clay with chert fragments. Minor amounts of fine gravel are also present in the fill. Localized very stiff gray silty clay with limestone fragment fill is also present at depth (see Boring S1). The cherty clay fill exhibits variable fair to good compaction. SPT N-values in the fill range from 10 to 30 blows per foot. The average N-value of 17 blows per ft indicates average good compaction. The fill has low compressibility and moderate shear strength. Fill depth, content, and compaction may vary across the site.

Stratum II: Below the fill is natural stiff to very stiff red clay with chert fragments and seams (cherty clay). The chert content is variable and chert layers are present at depth. The cherty clay represents residual soil weathered from the underlying cherty limestone bedrock. The clay fraction of the cherty clay has variable low to high plasticity. The cherty clay has moderate shear strength and low compressibility.

Stratum III: The basal stratum encountered in the borings is hard light gray and gray cherty limestone. The cherty limestone is strong. Minor amounts of drilling fluid loss indicate the possibility of open fractures or clay-filled voids in the limestone. However, no open voids or apparent karst zones were encountered or indicated by the borings in the limestone.

A Generalized Subsurface Profile projected to the bridge centerline is presented in Appendix A. It should be recognized that 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. Additionally, the natural transition between strata is generally gradual, and the stratigraphy described in the sections above may vary.

Groundwater Conditions

Groundwater was not encountered in the borings prior to the introduction of drilling fluids at 10- to 20-ft depths during drilling operations in June 2015. Groundwater was locally encountered in the wall borings at depths of 23 ft (±El 1275) to 26 ft (±El 1268) in May 2016. In addition, there is the potential for shallow perched water to develop, particularly during periods of high seasonal precipitation. Perched water may accumulate in the overburden soils and fractured rock zones. Groundwater levels will vary with seasonal precipitation and surface runoff and infiltration.

ANALYSES and RECOMMENDATIONS

Bridge Foundations

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

We recommend that the structural loads of the widened bridge be supported on pile foundation systems. Recommendations for piling are discussed in the following report sections.

Pile Foundations

We recommend that foundation loads of the I-49 / Hwy 71B interchange replacement bridge be supported on steel piles. The piles should extend through the embankment fill and overburden soils to bear at refusal in the competent hard light gray and gray cherty limestone (Stratum III). Piles should be driven to practical refusal. Steel HP14x73 or HP14x89 piles fitted with rock points are recommended.

Bearing capacities of piles driven to refusal must be determined using the AASHTO Load and Resistance Factor Design (LRFD) structural design procedure[4]. We recommend that nominal resistance (P_n) of steel piles be determined based on the yield strength of steel H piles (f_y) and the net end area (A_{net}) of the section. Given that the piles will be driven to refusal in rock with the potential for driving damage, we recommend a maximum allowable stress (σ_{all}) of 0.25 f_y. An effective resistance factor (φ) of 0.50 is recommended for end-bearing piles. This effective resistance factor for steel piles has been based on the assumption of difficult driving.

It has been our experience that allowable pile capacities of 134 tons are suitable for 14-in. dimension steel piles f_y 50 ksi steel. These capacities are based on allowable stress design (ASD). However, the appropriate factored bearing capacity as per LRFD criteria must be confirmed by the Engineer. Post-construction settlement of piles driven to refusal will be negligible. Given the age

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

of the existing embankment and the predominantly preconsolidated condition of the residual overburden soils, downdrag loads due to long-term embankment settlement are expected to be negligible.

We recommend that all piles extend through the embankment fill and overburden soils to bear in the competent rock. Estimated as-built pile tip elevations are expected to be on the order of El 1251 at both Bents 1 and 2. Depending on the embankment height and specific subsurface conditions encountered, preboring could be required to attain the recommended penetration through the cherty clay embankment fill. In addition, hard intervals of chert beds may warrant deeper preboring in some cases.

Post-construction settlement of piles driven to refusal will be negligible. As noted, downdrag loads due to long-term embankment settlement are expected to be negligible due to the age of the existing embankments and the expectation that no new embankments will be constructed.

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 recommend that the steel H-piles be driven with a hammer system capable of delivering at least 20,000 ft-lbs per blow for the 14-in. steel piles driven to refusal in rock. This value is based on the results of a drivability analysis using wave equation analyses (WEAP) methods and the computer program GRLWEAP 2010[5]. The results of the wave equation drivability analysis are provided in Appendix C.

As a minimum, safe bearing capacity of test piles and production piles should be determined by AHTD Standard Specifications Section 805.09, Method A. Driving records should be available for review by the Engineer during pile installation. All piles should be driven to practical refusal, typically defined as a penetration of 0.5 in. or less for the final 10 blows.

MSE Walls

General. The bridge ends and embankment sides will utilize MSE walls. This will include both Bents 1 and 2. The MSE walls are expected to have a maximum height of about 30 feet. The wall subgrade / leveling pad is planned at El 1284. It is understood that the MSE walls will be designed by Others on behalf of the Contractor. MSE wall backfill in the reinforced zone must

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[5] GRLWEAP 2010; Pile Dynamics, Inc.; 2010

comply with the Designer's specifications. As a minimum, the reinforced zone backfill is expected to comply with AHTD Standard Specifications Section 302, SM-1 or Section 303, Class 7.

The MSE wall bearing stratum at the plan bearing elevation is expected to be very stiff brownish gray to tan and gray silty clay to very stiff to hard reddish brown clay with numerous chert fragments. For the MSE wall supported in the anticipated bearing strata, a minimum nominal unit bearing resistance of 11,600 lbs per sq ft is recommended. A resistance factor (φ) of 0.65 is recommended for the MSE wall bearing. Consequently, a factored unit bearing resistance (q_R) of 7500 lbs per sq ft is anticipated.

The potential for undercut of the wall subgrade / foundation soils is considered low. All subgrade should be evaluated by the Engineer or Department during the work. Unstable or otherwise unsuitable foundation soils should be excavated to suitable materials.

Resistance to wall sliding can be evaluated using an ultimate friction factor (tan δ) value of 0.40 between the MSE wall reinforced zone base and the silty clay or cherty clay subgrade/bearing soil. A resistance factor (φ) of 1.0 is recommended for evaluation of sliding resistance. Long-term post-construction settlement of the wall foundation bearing stratum is expected to be negligible.

To evaluate global stability of the plan MSE wall configurations, slope stability analyses were performed. Given the similarity in maximum wall heights and in soil conditions, the stability analyses were limited to one (1) bridge end, The results of the stability analyses are provided in Appendix D. Stability analyses were performed using the computer program SLOPE/W 2007[6] and a Morgenstern-Price analysis. The loading conditions evaluated include the following.

- End of construction condition
- Long term condition
- Seismic condition assuming new embankments reinforced with geogrid and a horizontal acceleration coefficient (k_h) value of one-half of the peak ground acceleration value, i.e., 0.03, as per FHWA guidelines[7]

The stability analyses results are summarized and shown graphically in Appendix D. The soil parameters utilized in the analyses are shown on the result summary plate. Based on these results, global stability of the proposed walls is considered suitable.

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

[7] Design and Construction of Mechanistically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, Publication No. FHWA-NHI-10-025, FHWA, November 2009, Page 8-10.

Wingwall and Abutment Wall Lateral Earth Pressures

It is expected that wingwalls and abutment walls for the replacement bridges will be backfilled with unclassified borrow or select material. Recommendations related to lateral earth pressures for wingwalls and abutments 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:
  - Active condition for walls that are free to rotate, backfilled with unclassified borrow, and fully drained: 65 lbs per sq ft per ft depth.
  - Active condition for walls that are free to rotate backfilled with unclassified borrow and with no provision for internal drainage: 95 lbs per sq ft per ft depth.
- Total unit weight (γ) for SM-1: 125 lbs per cu ft
- Angle of internal friction (φ) for SM-1 backfill: 32°
- Equivalent fluid pressure for SM-1 backfill:
  - Active condition for walls that are free to rotate, backfilled with SM-1 or clean granular backfill, and fully drained: 40 lbs per sq ft per ft depth.
  - Active condition for walls that are free to rotate, backfilled with SM-1 or clean granular backfill, and with no provision for internal drainage: 85 lbs per sq ft per ft depth.
- Ultimate sliding resistance:
  - Interaction friction angle (δ) for concrete on stable bearing stratum of embankment fill or cherty clay: 19°.
  - Interaction friction factor (tan δ) for concrete on stable bearing stratum: 0.35.
  - The sliding resistance values above are nominal/ultimate values.
  - A resistance factor (φ) of 0.80 is recommended for sliding resistance.

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 or Department. Drainage zones should be fully isolated from the fine-grained cherty clay embankment fill and natural soils by an appropriate geotextile complying with the criteria of AHTD 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.

Site Grading Considerations

We expect that site grading will include significant cut of the existing embankments and fill placement. No substantial grading for new embankments is anticipated with this project. Site preparation in areas of incidental grading should begin with stripping the topsoil and any unsuitable surface soils. The stripping depth is expected to be on the order of 6 to 9 inches.

After stripping and performing any cut, and prior to placing fill, the subgrade should be evaluated by proof-rolling with a loaded tandem-wheel dump truck or similar equipment where accessible. Areas identified to be soft or that exhibit pumping should be undercut, processed and recompacted or replaced with suitable fill, whichever is appropriate. Based on the results of the borings, the potential for undercuts is considered low. Nevertheless, depending on seasonal site conditions and final grading plans, localized undercuts on the order of 2 ft below existing grades, more or less, could be warranted to stabilize localized areas of weak surface soils. Undercut requirements must be field verified by the Engineer or the Department during the work.

All new embankments should be constructed in accordance with AHTD criteria (AHTD Standard Specifications Section 210). Where localized seepage into undercuts or excavations is a problem, undercuts should be backfilled with AASHTO M43 #57 or stone backfill (AHTD Standard Specifications Section 207) fully encapsulated with an appropriate filter fabric (AHTD Standard Specifications Subsection 625.02, Type 2). The granular backfill should be vented to positive discharge if possible.

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. Fills placed against existing slopes should be benched into the existing slope face as new fill is constructed to facilitate placement of horizontal lifts. The in-place density and water content should be determined for each lift of fill and backfill. Each lift of backfill and fill should be tested and approved prior to placing subsequent lifts.

CONSTRUCTION CONSIDERATIONS

Positive surface drainage should be established at the start of the work, be maintained during construction and following completion of the project to prevent surface water ponding and subsequent saturation of subgrade soils. Density and water content of all earthwork should be maintained until the embankment and bridge work is completed. Subgrade soils that become saturated by ponding water or runoff should be excavated to undisturbed soils. Embankment areas where additional site grading is planned should be evaluated by the Engineer or Department during subgrade preparation and prior to starting embankment construction.

Shallow groundwater was not encountered in the borings drilled in June 2015. Minor seepage into isolated excavations can probably be controlled by ditching or sump-and-pump methods. If seepage into excavations becomes a problem, backfill should consist of clean crushed stone AASHTO M43 #57 or clean, crushed coarse stone (AHTD Standard Specifications Section 207). Sand or stone backfill should be encapsulated in filter fabric (AHTD Standard Specifications Subsection 625.02, Type 2) and vented to positive discharge at daylight or into storm drainage lines where possible.

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. In areas of seepage infiltration, the granular fill should be fully encapsulated with a filter fabric complying with AHTD Standard Specifications Subsection 625.02, Type 2.

Piles should be installed in compliance with AHTD Standard Specifications Section 805. Piles should be carefully examined prior to driving and piles with structural defects should be rejected. Any splices in steel piles should develop the full cross-sectional capacity of un-spliced piles. Some preboring may be required for pile installation. Depending on the specific location, rock drilling methods may be required for prebores advanced into limestone/cherty limestone. Prebores should have adequate width to accommodate the pile width. We recommend that after piles are installed in prebores, the annulus around piling be backfilled with sand grout, lean concrete, or an alternate approved by the Engineer or the Department.

Pile installation should be monitored by qualified personnel to maintain specific and complete driving records and to observe pile installation procedures. Driving records should be available for review by the Engineer and/or Department during pile installation.

We recommend that 14-in. steel piles be driven with a minimum 20,000 ft-lbs per blow hammer system. The pile-hammer system proposed by the Contractor should be specifically reviewed by the Engineer or Department prior to acceptance for the work. Blow counts on steel piles should be limited to about 20 blows per inch. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSING

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

The following illustrations are attached and complete this report.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 14	Boring Logs
Plates 15 and 16	Keys to Terms and Symbols
Plate 17	Summary of Subsurface Exploration Program
Appendix A	Generalized Subsurface Profile
Appendix B	Classification Test Results
Appendix C	Results of Drivability Analysis
Appendix D	Stability Analyses Results

* * * * *

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

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

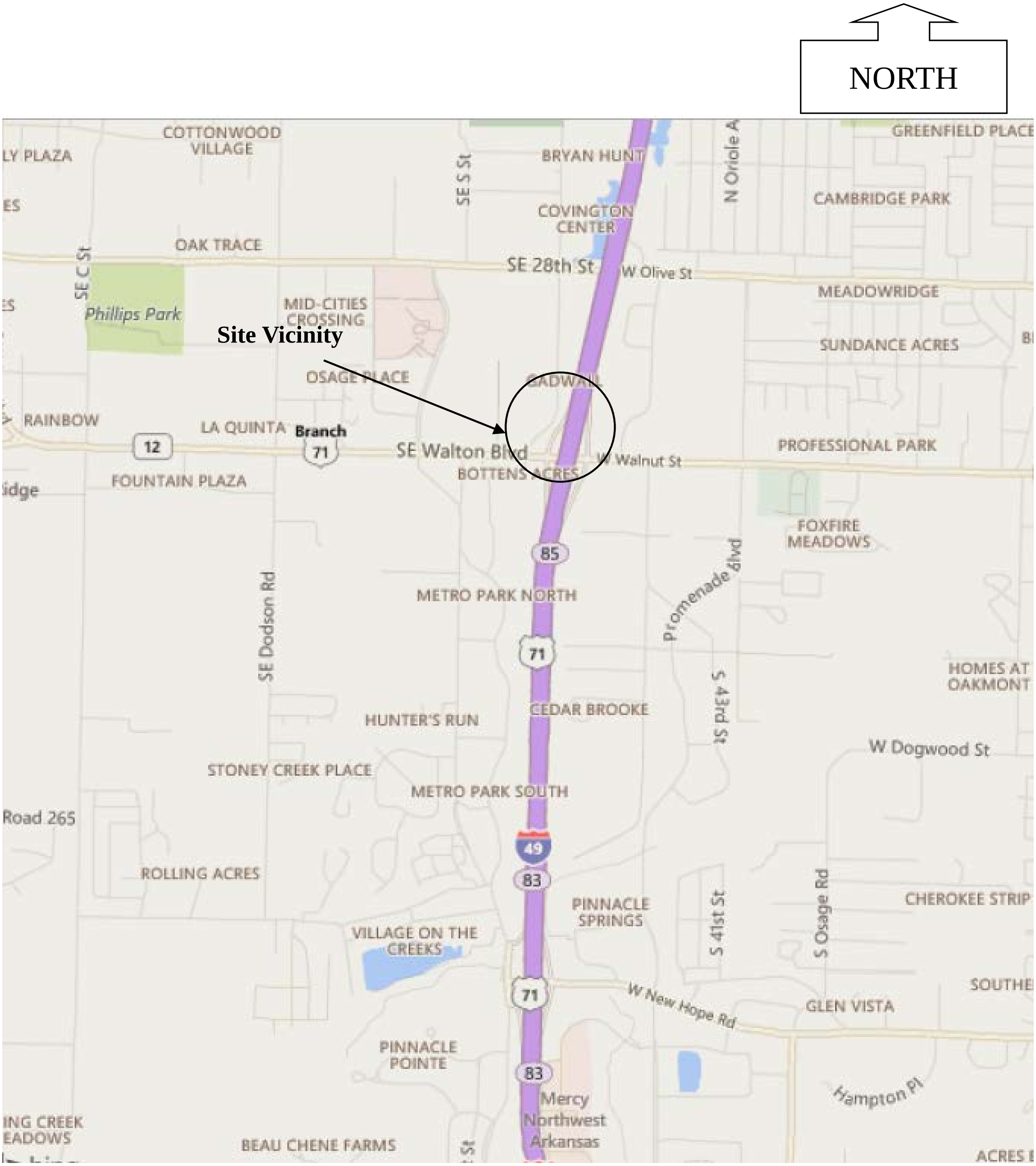
Blaine M. Orth, P.E.
Senior Project Engineer

Mark E. Wyatt, P.E.
President

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

BMO/MEW:jw

Copies Submitted: Crafton, Tull & Associates, Inc.
Attn: Mr. Mike Burns, P.E. (1+electronic)

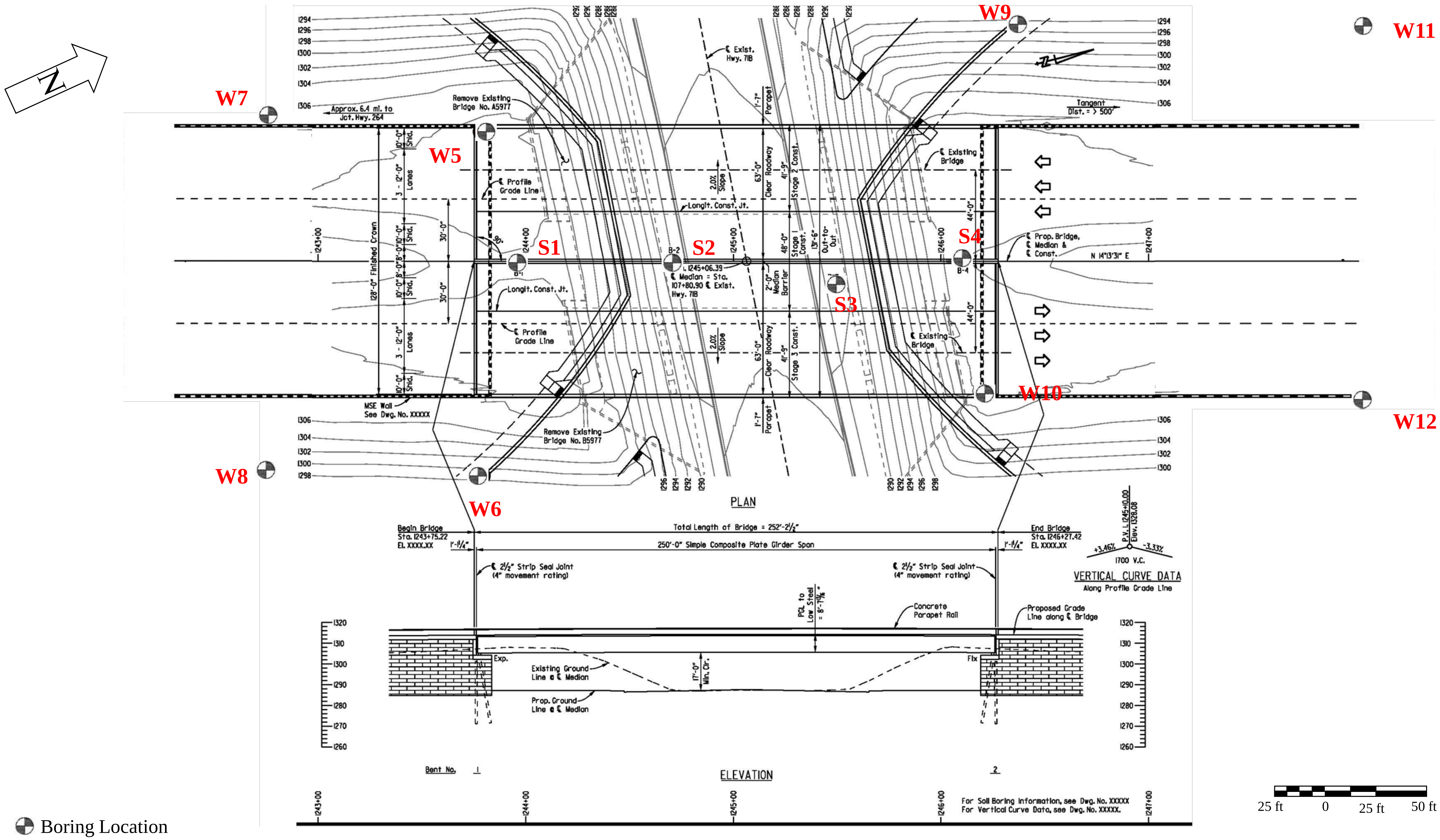


Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

SITE VICINITY MAP
BB0903 HWY 71B INTERCHANGE IMPROVEMENTS
Benton County, Arkansas

Job No. 12-071

Plate 1



Grubbs, Hoskyn, Barton & Wyatt, INC. CONSULTING ENGINEERS	**PLAN of BORINGS** **BB0903: Hwy 71B INTERCHANGE IMPROVEMENTS** **Benton County, Arkansas**	**Scale: As Shown** **Date: May 2016**	**Job No. 12-071**	**PLATE 2**

12-071

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

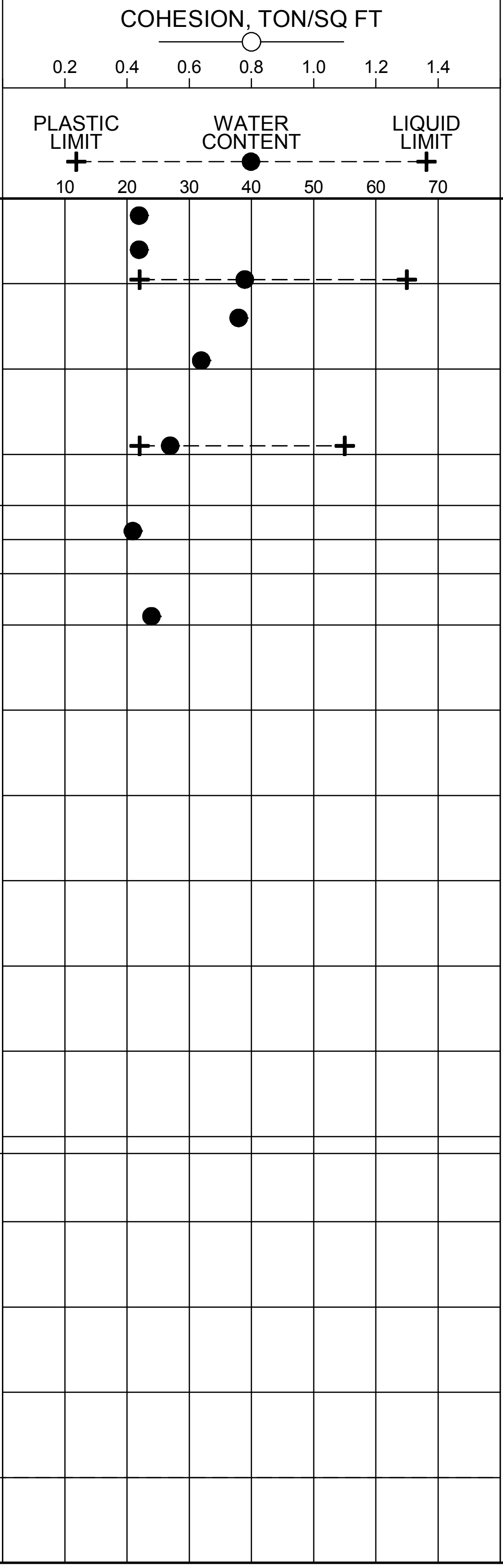
LOG OF BORING NO. S1

BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1243+96.80, 1.5 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 1307.8	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
			Stiff to very stiff red clay w/chert fragments (fill)	13		
				17		
5				22		72
				29		
10				29		
15			- with chert cobbles below 14 ft	30		63
20			Very stiff gray silty clay w/limestone fragments and trace organics (fill)	25		
25			Very stiff red clay w/numerous chert fragments (cherty clay)	37		
30			- with chert cobbles and seams below 28 ft	50/4"		
35			- with chert cobbles, seams and layers below 33 ft	50/3"		
40				25/0"		
45				25/0"		
50				25/0"		
55				50/11"		
60			Hard light gray and gray cherty limestone	25/0"		
65				25/0"		
70			- no loss of circulation in cherty limestone to completion depth	25/0"		
75				25/0"		



COMPLETION DEPTH: 75.0 ft
DATE: 6-23-15

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 6/23/2015

Form 108-6(74) Job No. 12-071

12-071

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

LOG OF BORING NO. S2

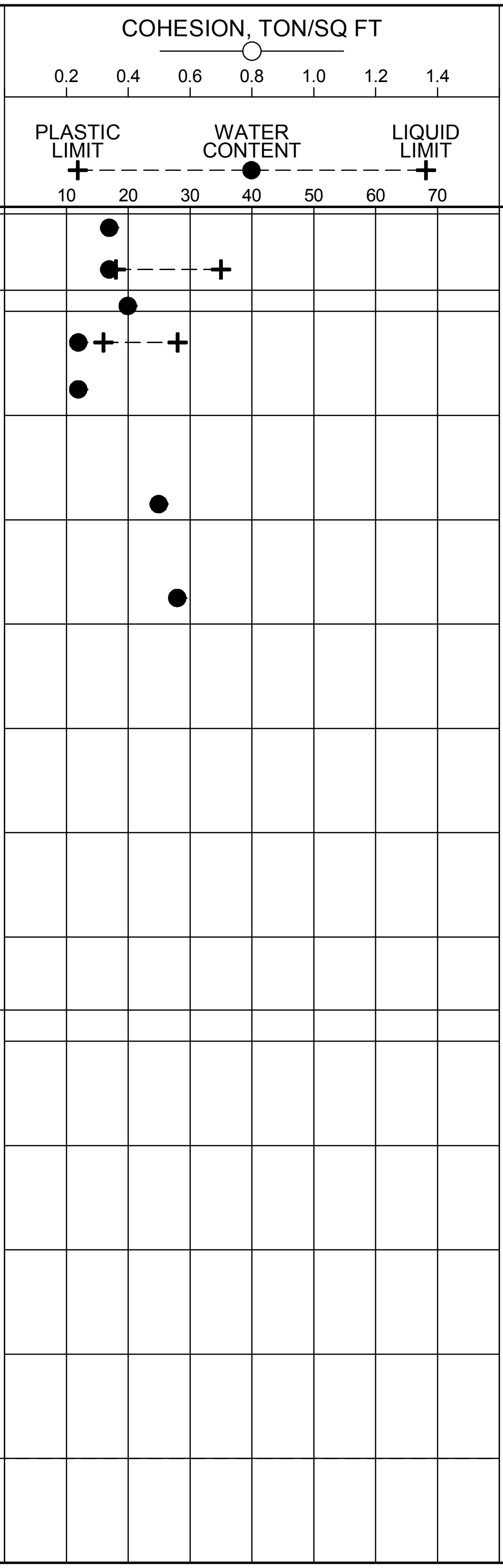
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1244+71.61, 1 ft Lt

SURF. EL: 1286.9

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
4 inches: Concrete Stiff brown silty clay w/a little fine gravel and chert fragments (fill)	11		
	14		80
Very stiff reddish tan clay w/chert nodules and fragments (cherty clay)	38		
	50/8"		39
	50/5"		
- red and tan with chert cobbles below 11 ft	25/0"		
	50/4"		
- with chert seams and layers below 22 ft	25/0"		
	25/0"		
- tripoli and clay at 32 ft	50/4"		
- 100% water loss at 36 ft, borehole backfilled with bentonite and re-drilled, circulation recovered	25/0"		
Hard light gray and gray cherty limestone			
- could not advance core barrel to cherty limestone, coring abandoned	25/0"		
	25/0"		
- no loss of circulation, no voids in cherty limestone to completion depth	25/0"		
	25/0"		



DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

COMPLETION DEPTH: 60.0 ft
DATE: 6-17-15

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 6/17/2015

LGBNEW 12-071.GPJ 6-16-16

12-071

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

LOG OF BORING NO. S3

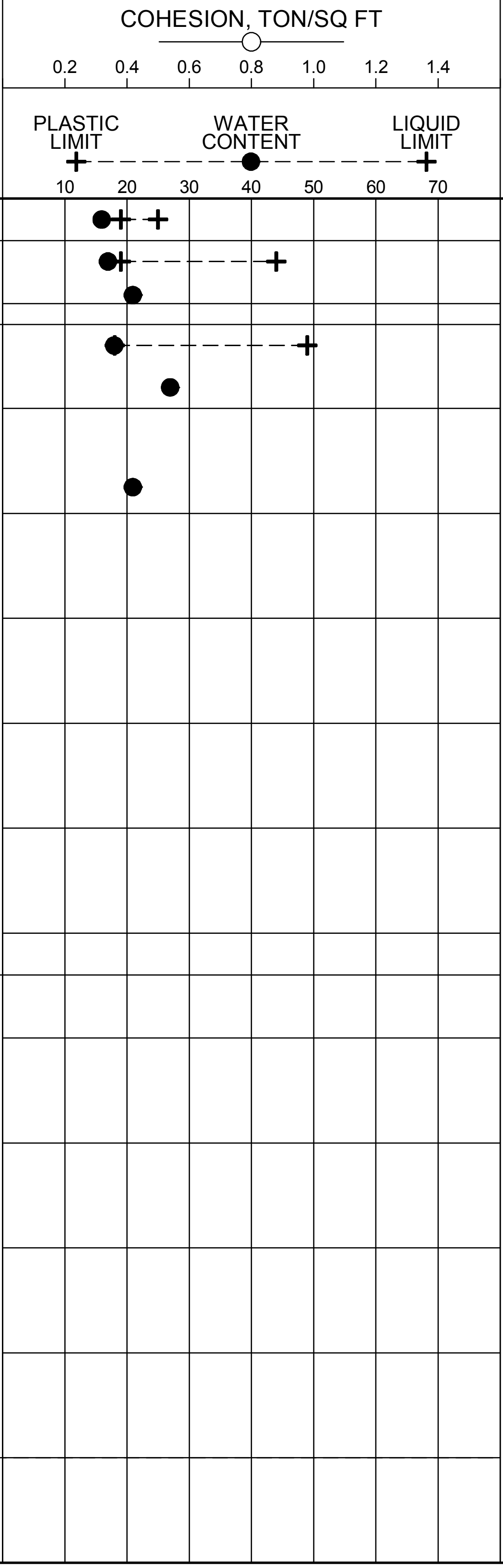
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1245+48.29, 12 ft Rt

SURF. EL: 1288.0

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brown silty clay w/a little fine gravel and occasional organics (fill)	14		85
	Firm to stiff reddish tan clay with chert fragments	10		81
5	- stiff, red, gray and tan below 4 ft	15		
	Very stiff red clay w/numerous chert nodules and fragments (cherty clay)	50/9"		53
		50/8"		
10	- with chert cobbles below 10 ft			
		50/6"		
15				
	- with medium close chert seams and layers below 16 ft			
		25/0"		
20				
		50/4"		
25				
		25/0"		
30				
		25/0"		
35				
	Hard gray and light gray cherty limestone	25/0"		
40	- 10% water loss at 40 ft; circulation recovered			
		25/0"		
45	- could not advance core barrel to cherty limestone due to chert fragment fall in, coring abandoned			
		25/0"		
50				
		25/0"		
55				
		25/0"		
60				



COMPLETION DEPTH: 60.0 ft
DATE: 6-16-15

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 6/16/2015

Form 108-6(74) Job No. 12-071

12-071

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

LOG OF BORING NO. S4

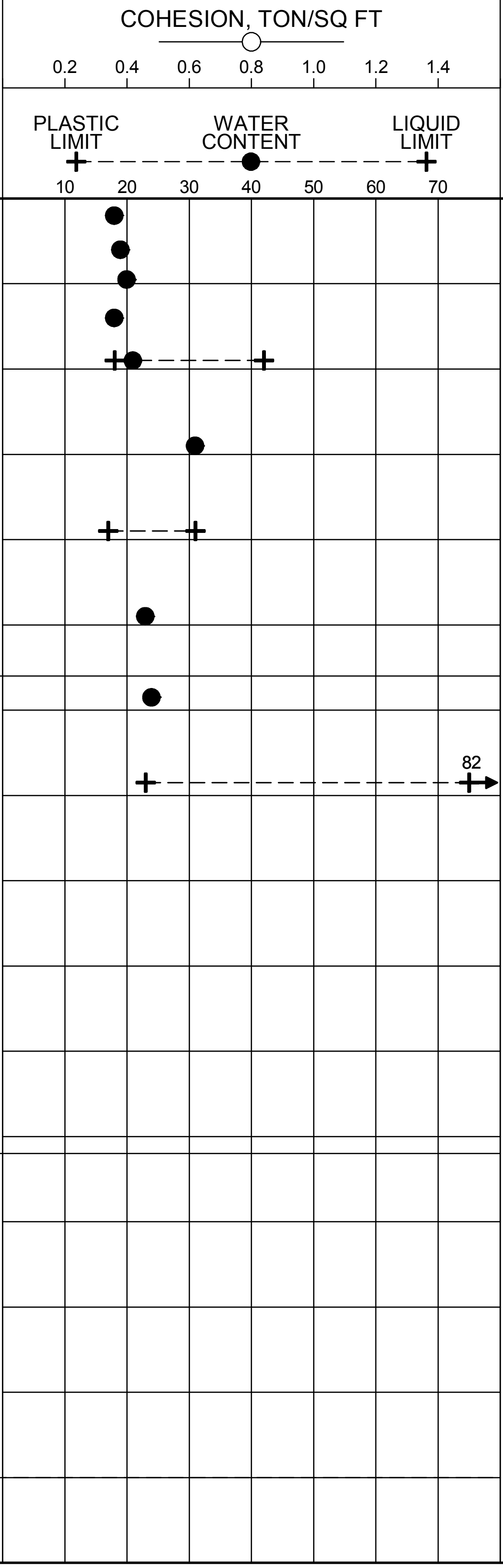
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1246+10.93, CL

SURF. EL: 1308.0

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brownish red and brown silty clay w/chert fragments (fill)	13		
	- firm to stiff at 2 to 4 ft	10		
5	- stiff below 4 ft	15		
		16		
10		14		72
15	- with more chert below 14 ft	16		
20	- red, gray and brown below 18 ft	14		74
25		16		
30	Very stiff red and tan clay w/numerous chert fragments and seams (cherty clay)	50/3"		
35		50/4"		53
40		50/6"		
45		50/9"		
50		50/5"		
55	- with more chert below 53 ft - 20% water loss at 55 ft	25/0"		
60	Hard light gray and gray cherty limestone - fluid circulation recovered at 48 ft	25/0"		
65		25/0"		
70				
75		25/0"		



LGBNEW 12-071.GPJ 6-16-16

COMPLETION DEPTH: 75.0 ft
DATE: 6-22-15

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 6/22/2015

12-071

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

LOG OF BORING NO. W5

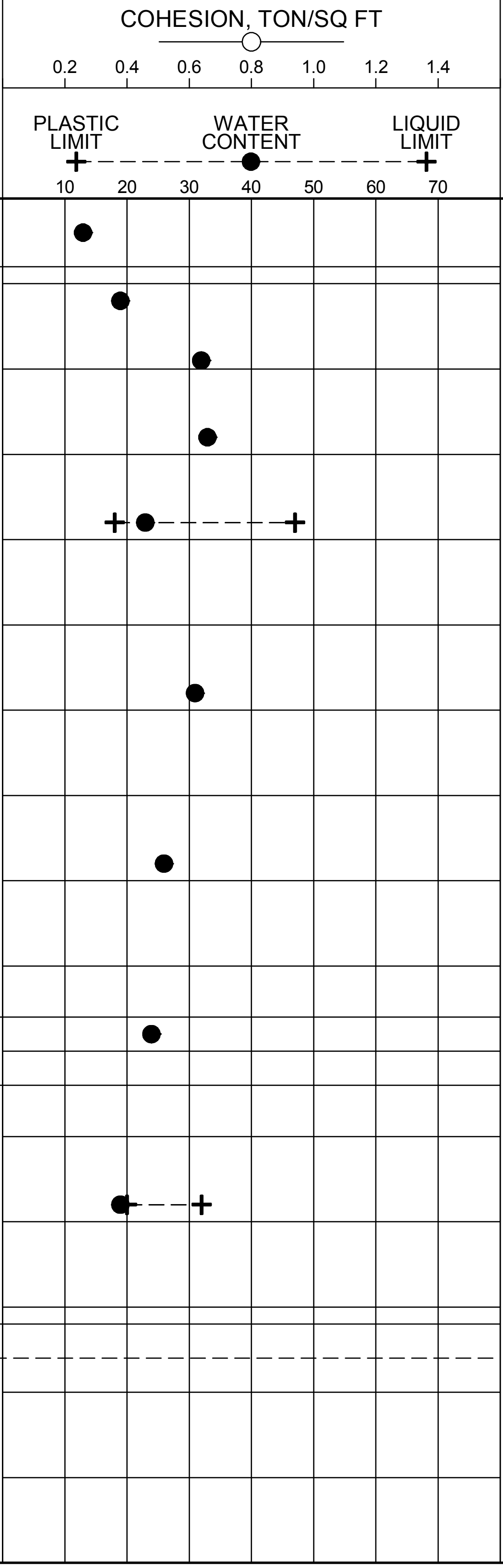
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1243+80, 62.3 ft Lt

SURF. EL: 1308±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff to very stiff brown silty clay w/fine sand and chert fragments and trace organics, dry (fill)	27		42
	Stiff brownish red clay w/chert fragments (fill)	37		
5		21		
	- stiff to very stiff below 6 ft	29		
10		26		70
15		36		
20		31		
25	Very stiff brownish gray silty clay	50/11"		
	Very stiff reddish brown, reddish tan and gray silty clay w/ferrous nodules, fine sand pockets and sandstone fragments			
30		40		70
	Hard light gray cherty limestone	25/0"		
35	- auger refusal on limestone at 34 ft			



COMPLETION DEPTH: 34.0 ft
DATE: 5-19-16

DEPTH TO WATER
IN BORING: Dry

DATE: 5/19/2016

Form 108-6(74) Job No. 12-071

12-071

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

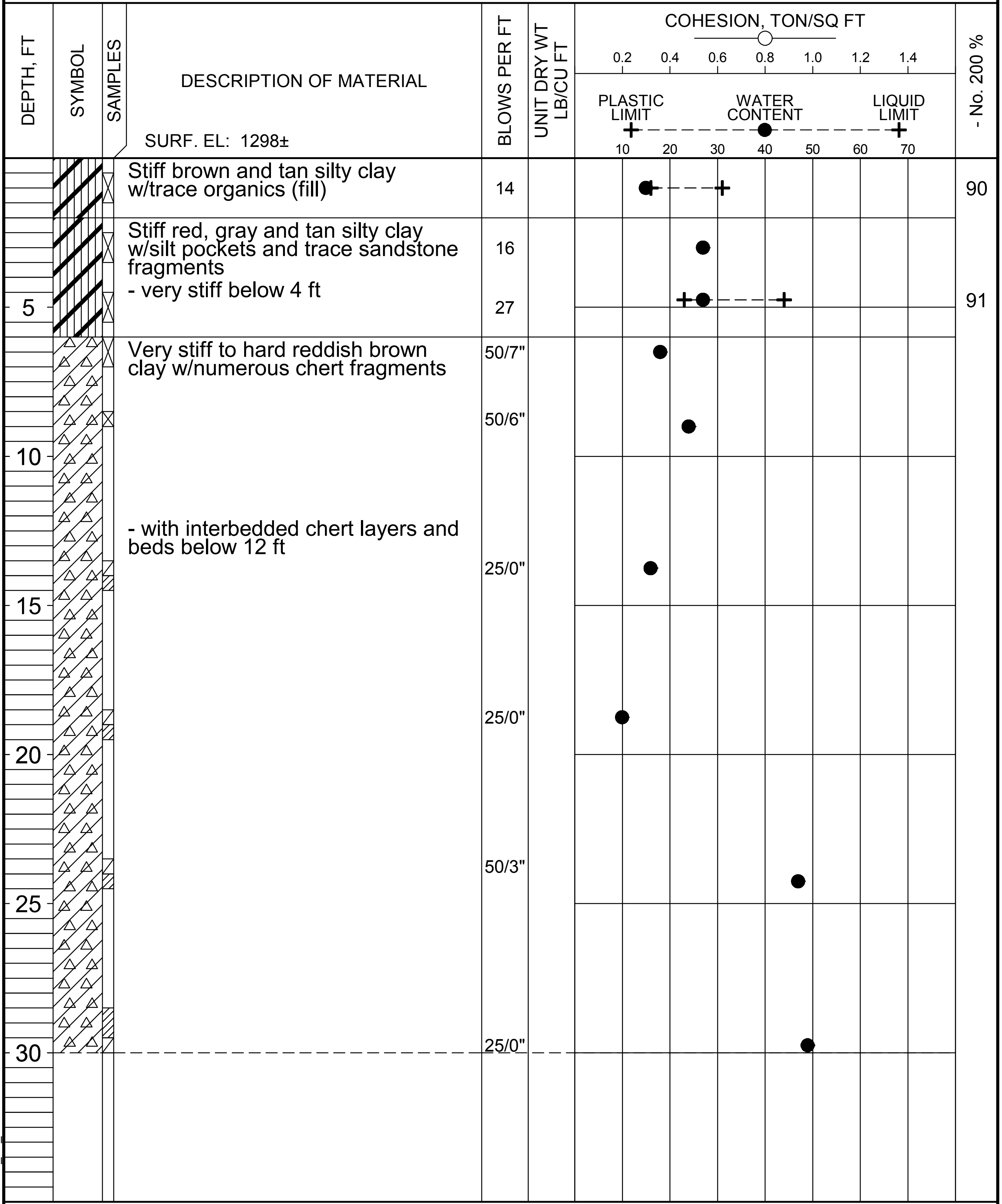
LOG OF BORING NO. W6

BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1243+77, 106 ft Rt

SURF. EL: 1298±



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Stiff brown and tan silty clay w/trace organics (fill)	14		90
Stiff red, gray and tan silty clay w/silt pockets and trace sandstone fragments	16		
- very stiff below 4 ft	27		91
Very stiff to hard reddish brown clay w/numerous chert fragments	50/7"		
	50/6"		
- with interbedded chert layers and beds below 12 ft	25/0"		
	25/0"		
	50/3"		
	25/0"		

LGBNEW 12-071 W_LOGS.GPJ 6-16-16

COMPLETION DEPTH: 30.0 ft
DATE: 5-24-16

DEPTH TO WATER
IN BORING: 23 ft

DATE: 5/24/2016

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

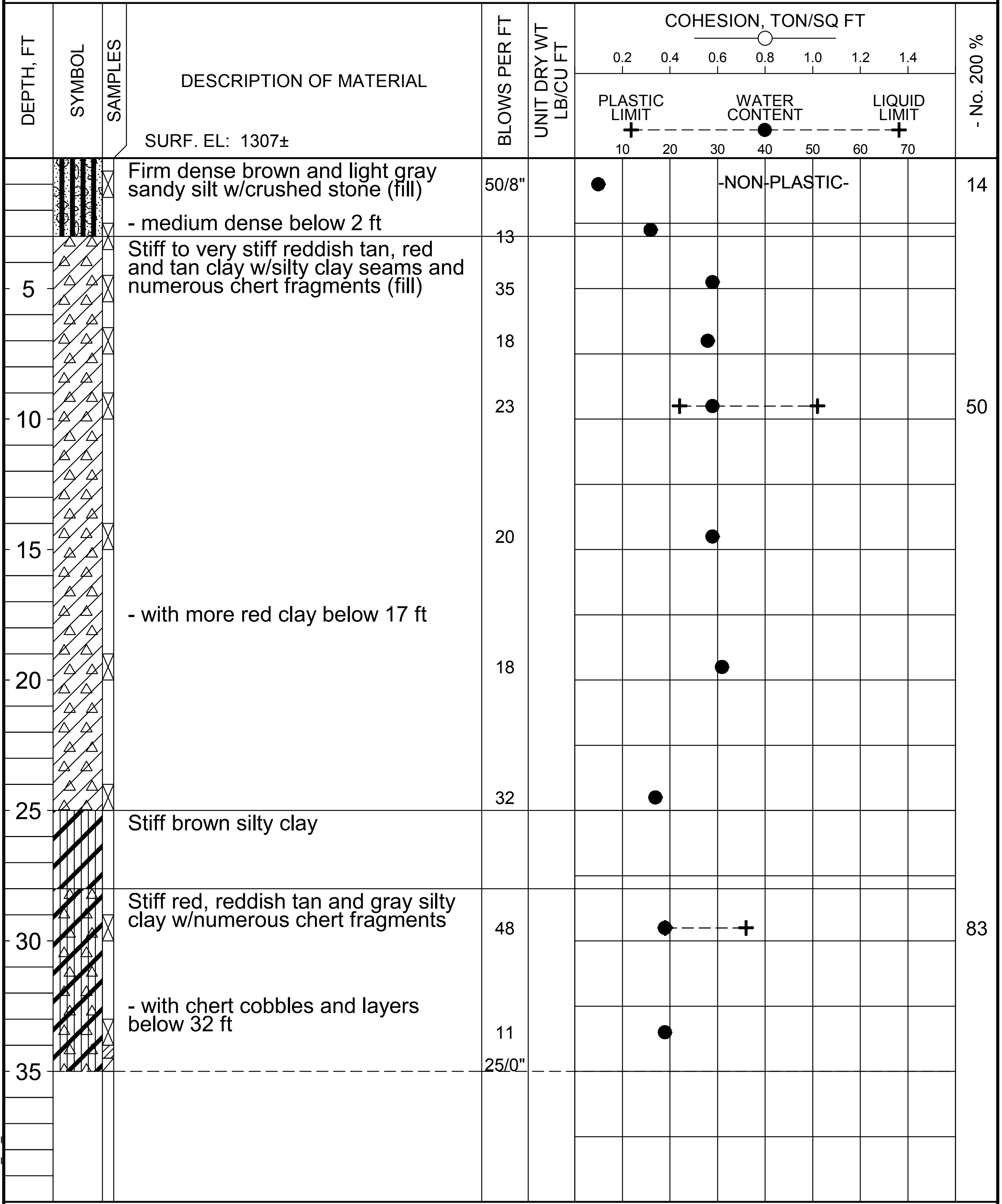
LOG OF BORING NO. W7

BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1242+77, 68.4 ft Lt

SURF. EL: 1307±



DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
Firm dense brown and light gray sandy silt w/crushed stone (fill)	50/8"	14
- medium dense below 2 ft	13	
Stiff to very stiff reddish tan, red and tan clay w/silty clay seams and numerous chert fragments (fill)	35	
	18	
	23	50
	20	
- with more red clay below 17 ft	18	
	32	
Stiff brown silty clay		
Stiff red, reddish tan and gray silty clay w/numerous chert fragments	48	83
- with chert cobbles and layers below 32 ft	11	
	25/0"	

COMPLETION DEPTH: 35.0 ft
DATE: 5-23-16

DEPTH TO WATER
IN BORING: Dry

DATE: 5/23/2016

Form 108-6(74) Job No. 12-071

12-071

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

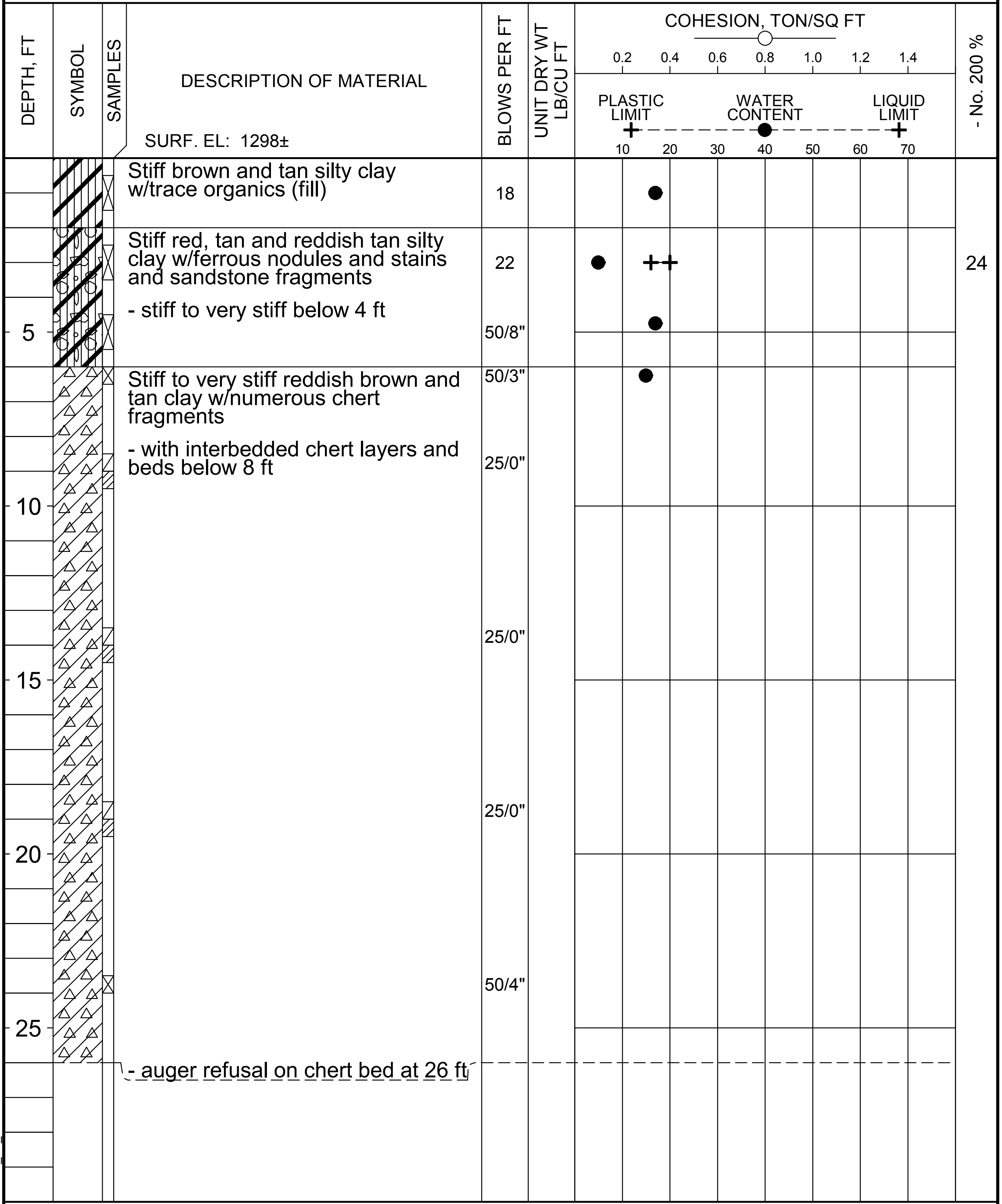
L O G O F B O R I N G N O. W8

BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1242+75, 99 ft Rt

SURF. EL: 1298±



Stiff brown and tan silty clay w/trace organics (fill)	18	
Stiff red, tan and reddish tan silty clay w/ferrous nodules and stains and sandstone fragments	22	24
- stiff to very stiff below 4 ft	50/8"	
Stiff to very stiff reddish brown and tan clay w/numerous chert fragments	50/3"	
- with interbedded chert layers and beds below 8 ft	25/0"	
	25/0"	
	25/0"	
	50/4"	
- auger refusal on chert bed at 26 ft		

LGBNEW 12-071 W_LOGS.GPJ 6-16-16

COMPLETION DEPTH: 26.0 ft
DATE: 5-24-16

DEPTH TO WATER
IN BORING: Dry

DATE: 5/24/2016

Form 108-6(74) Job No. 12-071

12-071

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

LOG OF BORING NO. W9

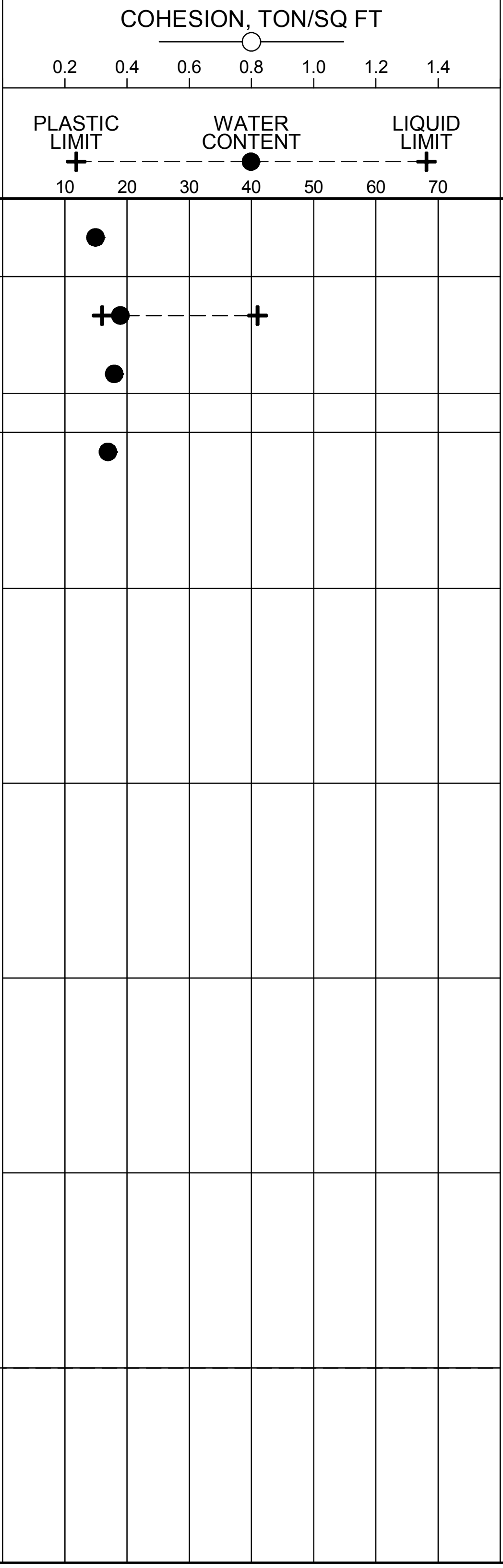
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1246+38, 113.2 ft Lt

SURF. EL: 1294±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
0	Firm to stiff brown silty clay (fill)	10		
2	Stiff reddish tan and light gray silty clay w/ferrous nodules and stains, sandstone fragments and fine sand pockets	12		89
	- stiff to very stiff with more nodules and sandstone fragments below 4 ft	50/8"		
6	Stiff to very stiff reddish brown clay w/numerous chert fragments	50/7"		
	- with interbedded chert layers and beds below 8 ft	25/0"		
		50/5"		
		50/3"		
		25/0"		
30		50/5"		



COMPLETION DEPTH: 30.0 ft
DATE: 5-24-16

DEPTH TO WATER
IN BORING: 26 ft

DATE: 5/24/2016

LGBNEW 12-071 W_LOGS.GPJ 6-16-16

Form 108-6(74) Job No. 12-071

12-071

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

LOG OF BORING NO. W10

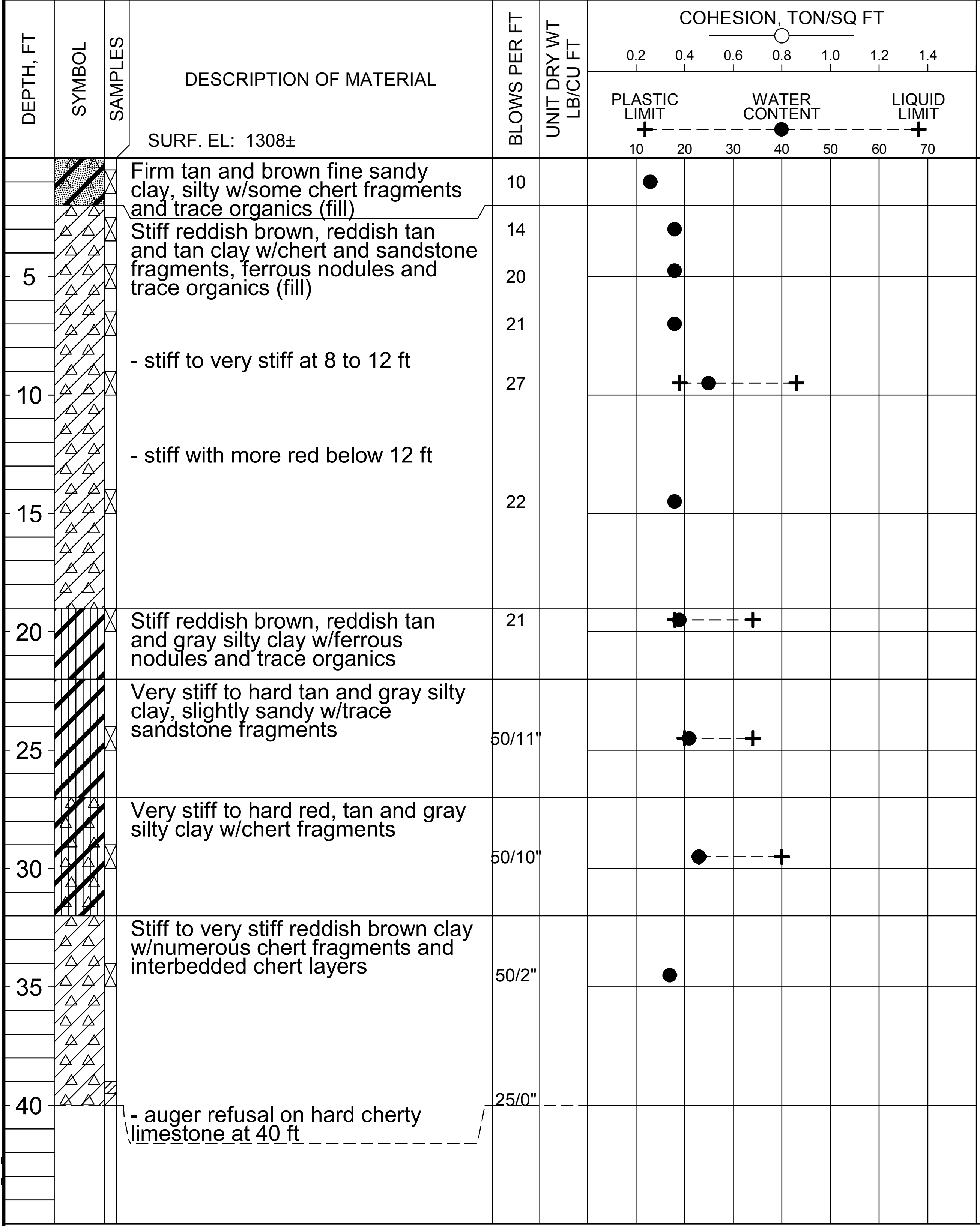
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1246+21, 63.7 ft Rt

SURF. EL: 1308±

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm tan and brown fine sandy clay, silty w/some chert fragments and trace organics (fill)	10		
Stiff reddish brown, reddish tan and tan clay w/chert and sandstone fragments, ferrous nodules and trace organics (fill)	14		
	20		
	21		
- stiff to very stiff at 8 to 12 ft	27		90
- stiff with more red below 12 ft	22		
Stiff reddish brown, reddish tan and gray silty clay w/ferrous nodules and trace organics	21		86
Very stiff to hard tan and gray silty clay, slightly sandy w/trace sandstone fragments	50/11"		87
Very stiff to hard red, tan and gray silty clay w/chert fragments	50/10"		70
Stiff to very stiff reddish brown clay w/numerous chert fragments and interbedded chert layers	50/2"		
- auger refusal on hard cherty limestone at 40 ft	25/0"		



COMPLETION DEPTH: 40.0 ft
DATE: 5-19-16

DEPTH TO WATER
IN BORING: Dry

DATE: 5/19/2016

LGBNEW 12-071 W_LOGS.GPJ 6-16-16

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Consulting Engineers

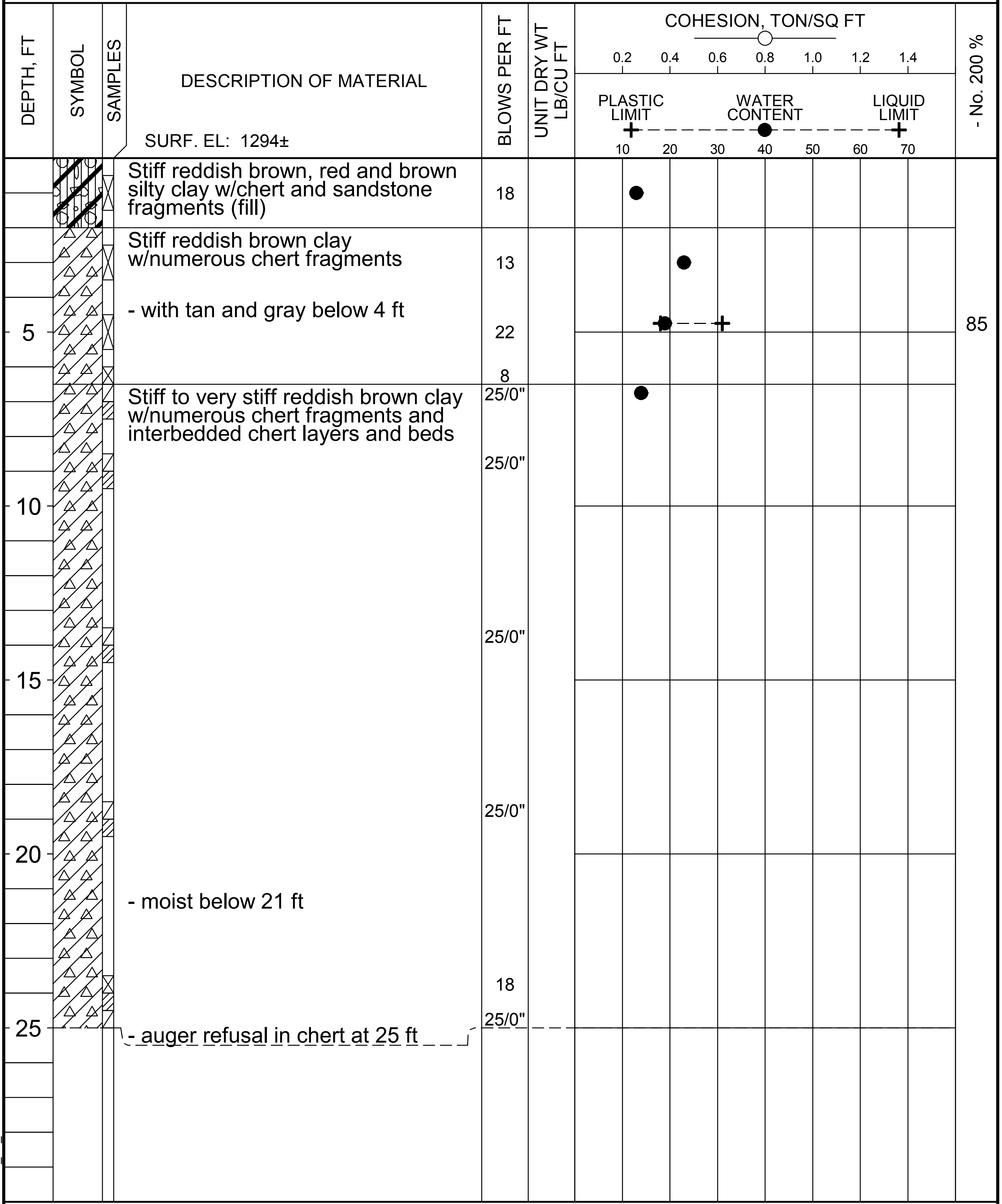
LOG OF BORING NO. W11

BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger

LOCATION: Approx Sta 1248+04, 110.8 ft Lt

SURF. EL: 1294±



Stiff reddish brown, red and brown silty clay w/chert and sandstone fragments (fill)

Stiff reddish brown clay w/numerous chert fragments

- with tan and gray below 4 ft

Stiff to very stiff reddish brown clay w/numerous chert fragments and interbedded chert layers and beds

- moist below 21 ft

- auger refusal in chert at 25 ft

COMPLETION DEPTH: 25.0 ft
DATE: 5-24-16

DEPTH TO WATER
IN BORING: 23 ft

DATE: 5/24/2016

12-071

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

LOG OF BORING NO. W12

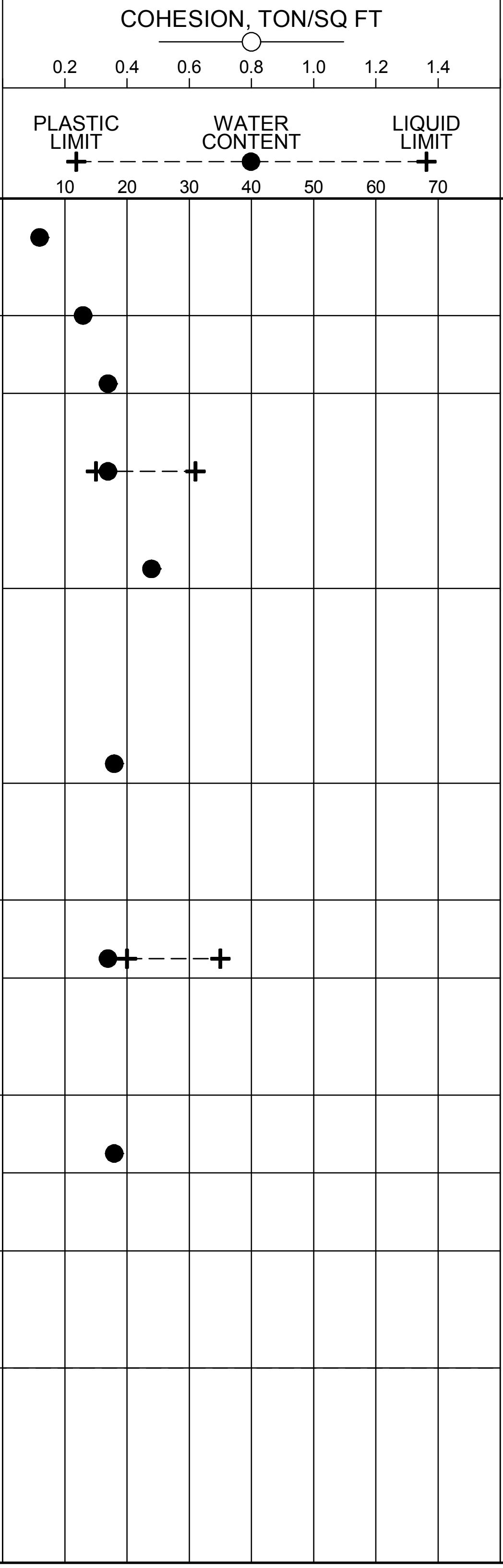
BB0903: Hwy 71B INTCHNG. IMPVTS.
Benton County, Arkansas

TYPE: Auger to 27 ft /Wash

LOCATION: Approx Sta 1248+04, 63.7 ft Rt

SURF. EL: 1307±

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Very stiff dark brown silty clay, sandy w/crushed stone and trace organics, dry (fill)	50/9"		13
Stiff reddish brown and reddish tan silty clay w/chert and sandstone fragments and ferrous nodules	17		
	17		
- stiff to very stiff below 6 ft	24		72
	28		
	23		
Stiff reddish brown, gray and brown fine sandy clay w/ferrous nodules	14		73
Very stiff to hard red, tan and gray clay w/trace sandstone fragments and numerous chert fragments	50/7"		
Moderately hard to hard light gray cherty limestone - 100% water loss at 27 ft	25/0"		



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

COMPLETION DEPTH: 30.0 ft
DATE: 5-19-16

DEPTH TO WATER
IN BORING: Dry to 27 ft

DATE: 5/19/2016

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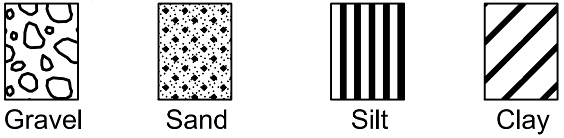
SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

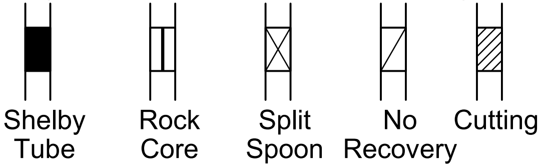
(SHOWN IN SYMBOLS COLUMN)



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 6-10-16

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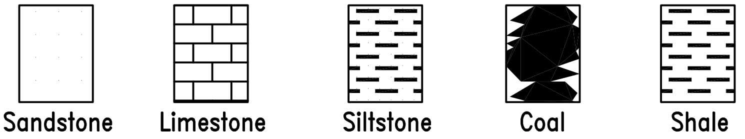
BORING LOG TERMS – ROCK

ROCK TYPES
(SHOWN IN SYMBOLS COLUMN)



Joint Characteristics –

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

Bedding Characteristics –

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

Lithologic Characteristics –

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

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

Hardness–

Soft (S) – Reserved for plastic material alone.

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

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

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

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

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

Texture –

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

Structure –

- Bedding
  - Flat – 0° – 5°
  - Gently Dipping – 5° – 35°
  - Moderately Dipping – 55° – 85°
  - Steeply Dipping – 55° – 85°
- Fractures, scattered
  - Open
  - Cemented or Tight
- Fractures, closely spaced
  - Open
  - Cemented or Tight
- Brecciated (Sheared and Fragmented)
  - Open
  - Cemented or Tight
- Joints
  - Faulted
  - Slickensides

Degree of Weathering –

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

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

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

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

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

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

Solution and Void Conditions –

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

Swelling Properties –

- Nonswelling
- Swelling

Slaking Properties –

- Nonslaking
- Slakes slowly on exposure
- Slakes readily on exposure

Rock Quality Designation (RQD) –

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

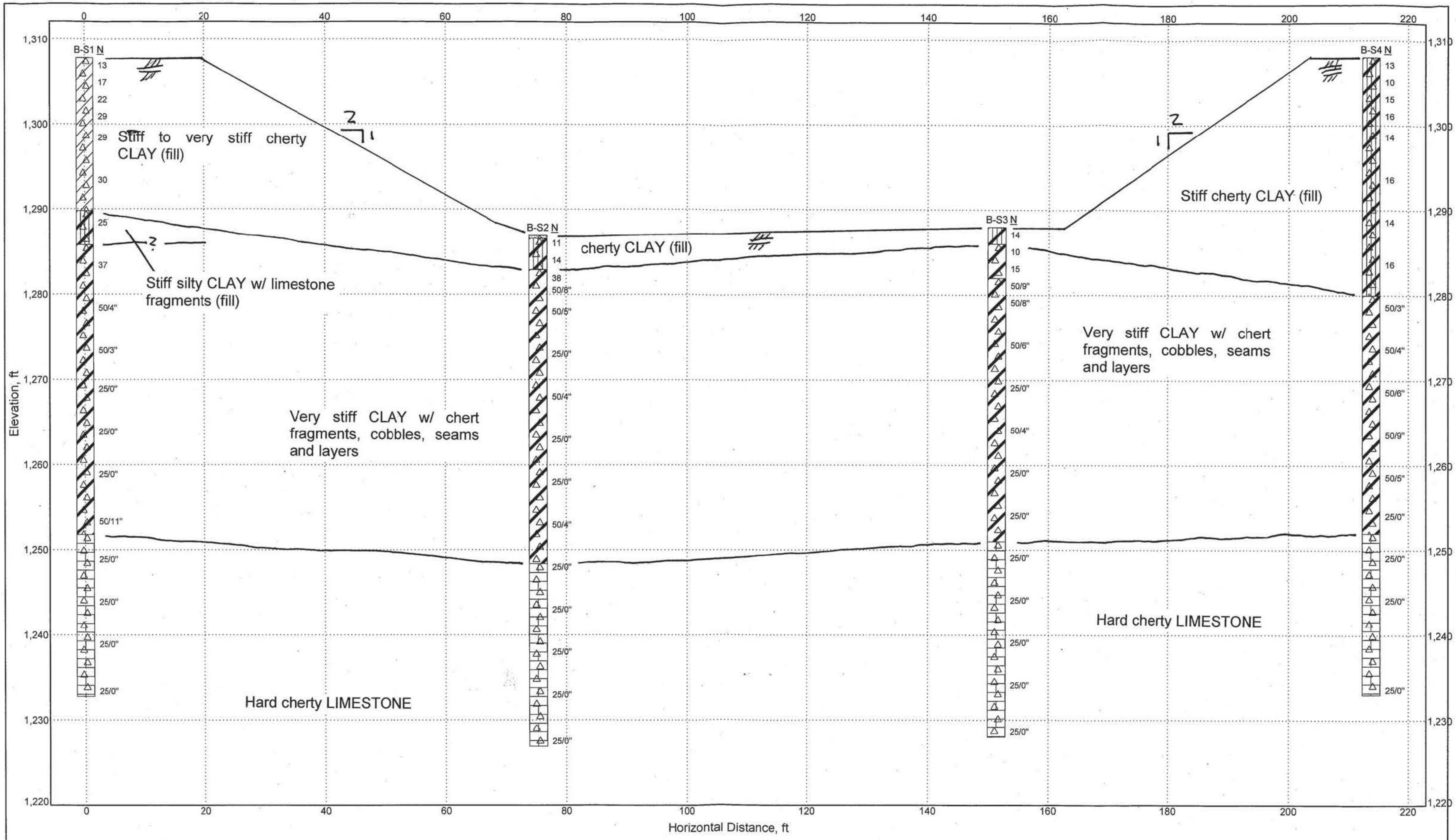
KEYROCK FHWA 3-2-12

SUMMARY of SUBSURFACE EXPLORATION

PROJECT: BB0903 Hwy 71B INTERCHANGE IMPROVEMENTS
LOCATION: Benton County, Arkansas
GHBW JOB No.: 12-071

Boring No.	Project Facet	Approx Sta	Approx. Offset, ft	Approx Surf El, ft	Comp Depth, ft
S1	Structure	1243+97	1.5R	1307.8	75
S2	Structure	1244+72	1.0L	1286.9	60
S3	Structure	1245+48	12R	1288	60
S4	Structure	1246+11	CL	1308	75
W5	Wall	1243+80	62.3L	1308	34
W6	Wall	1243+77	106R	1298	30
W7	Wall	1242+77	68.4L	1307	35
W8	Wall	1242+75	99R	1298	26
W9	Wall	1246+38	113.2L	1294	30
W10	Wall	1246+21	63.7R	1308	40
W11	Wall	1248+04	110.8L	1294	25
W12	Wall	1248+04	63.7R	1307	30

APPENDIX A



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

SCALE:
AS SHOWN

Generalized Subsurface Profile
BB0903 Hwy 71B INTERCHANGE IMPROVEMENTS
Benton County, Arkansas
Project Number: 12-071

PLATE

APPENDIX B

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT:BB0903 Hwy 71B INTERCHANGE IMPROVEMENTS
LOCATION: Benton County, Arkansas
GHBW JOB No.: 12-071

Boring No.	Sample Depth, ft	Water Content, %	ATTERBERG LIMITS			Percent Passing No. 200, %	UNIFIED CLASS.	AASHTO CLASS.
			Liquid Limit	Plastic Limit	Plasticity Index			
S1	4.5-5.5	39	65	28	37	72	CH	A-7-6
S1	14-15	27	55	22	33	63	CH	A-7-6
S2	2.5-3.5	17	25	18	7	80	CL-ML	A-4
S2	6-7	12	28	16	12	39	GC	A-6
S3	0.5-1.5	16	26	19	7	85	CL-ML	A-4
S3	2.5-3.5	19	44	17	27	81	CL	A-7-6
S3	6.5-7.5	18	49	18	31	53	CL	A-7-6
S4	9-10	21	42	18	24	72	CL	A-7-6
S4	19-20	17	31	17	14	74	CL	A-6
S4	33.5-34.5	----	82	23	59	53	CH	A-7-6

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: BB0903 Hwy 71B INTERCHANGE IMPROVEMENTS
LOCATION: Benton County, Arkansas
JOB No.: 12-071

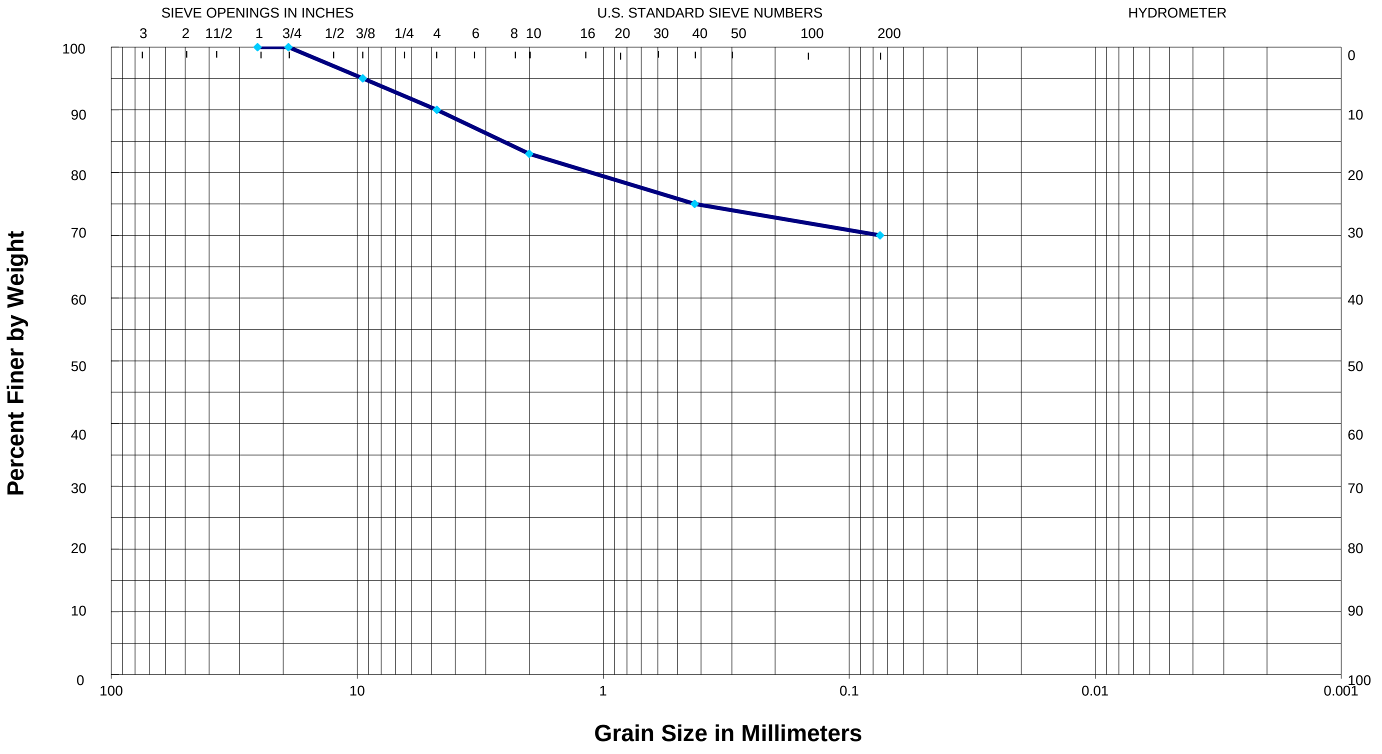
Boring No.	Sample Depth, ft	Water Content, %	ATTERBERG LIMITS			SIEVE ANALYSIS - PERCENT PASSING							UNIFIED CLASS.	AASHTO CLASS.
			Liquid Limit	Plastic Limit	Plasticity Index	1 in.	3/4 in.	3/8 in.	#4	#10	#40	#200		
W5	0.5 - 1.5	13	---	---	---	---	---	---	---	---	---	42	SM	A-4
W5	9 - 10	23	47	18	29	100	100	95	90	83	75	70	CL	A-7-6
W5	29 - 30	19	32	20	12	---	---	---	---	---	---	70	CL	A-7-6
W6	0.5 - 1.5	15	31	16	15	---	---	---	---	---	---	90	CL	A-6
W6	4.5 - 5.5	24	44	23	21	---	---	---	---	---	---	91	CL	A-7-6
W7	0.5 - 1.5	5	NP	NP	NP	---	---	---	---	---	---	14	SM	A-1-b
W7	9 - 10	29	51	22	29	---	---	---	---	---	---	50	CH	A-7-6
W7	29 - 30	19	36	19	17	---	---	---	---	---	---	83	CL	A-6
W8	2.5 - 3.5	5	20	16	4	---	---	---	---	---	---	24	GM-GC	A-2-4
W9	2.5 - 3.5	19	41	16	25	---	---	---	---	---	---	89	CL	A-7-6
W10	9 - 10	25	43	19	24	100	100	100	100	99	97	90	CL	A-7-6
W10	19 - 20	19	34	18	16	---	---	---	---	---	---	86	CL	A-7-6
W10	24 - 25	21	34	20	14	---	---	---	---	---	---	87	CL	A-7-6
W10	29 - 30	23	40	23	17	---	---	---	---	---	---	70	CL	A-7-6
W11	4.5 - 5.5	19	31	18	13	---	---	---	---	---	---	85	CL	A-6
W12	0.5 - 1.5	6	---	---	---	100	100	91	73	53	31	13	SM	A-1-b
W12	6.5 - 7.5	17	31	15	16	100	100	100	96	88	84	72	CL	A-7-6
W12	19 - 20	17	35	20	15	---	---	---	---	---	---	73	CL	A-7-6

Grubbs, Hoskyn,
Barton Wyatt, Inc.
CONSULTING ENGINEERS

12-071

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

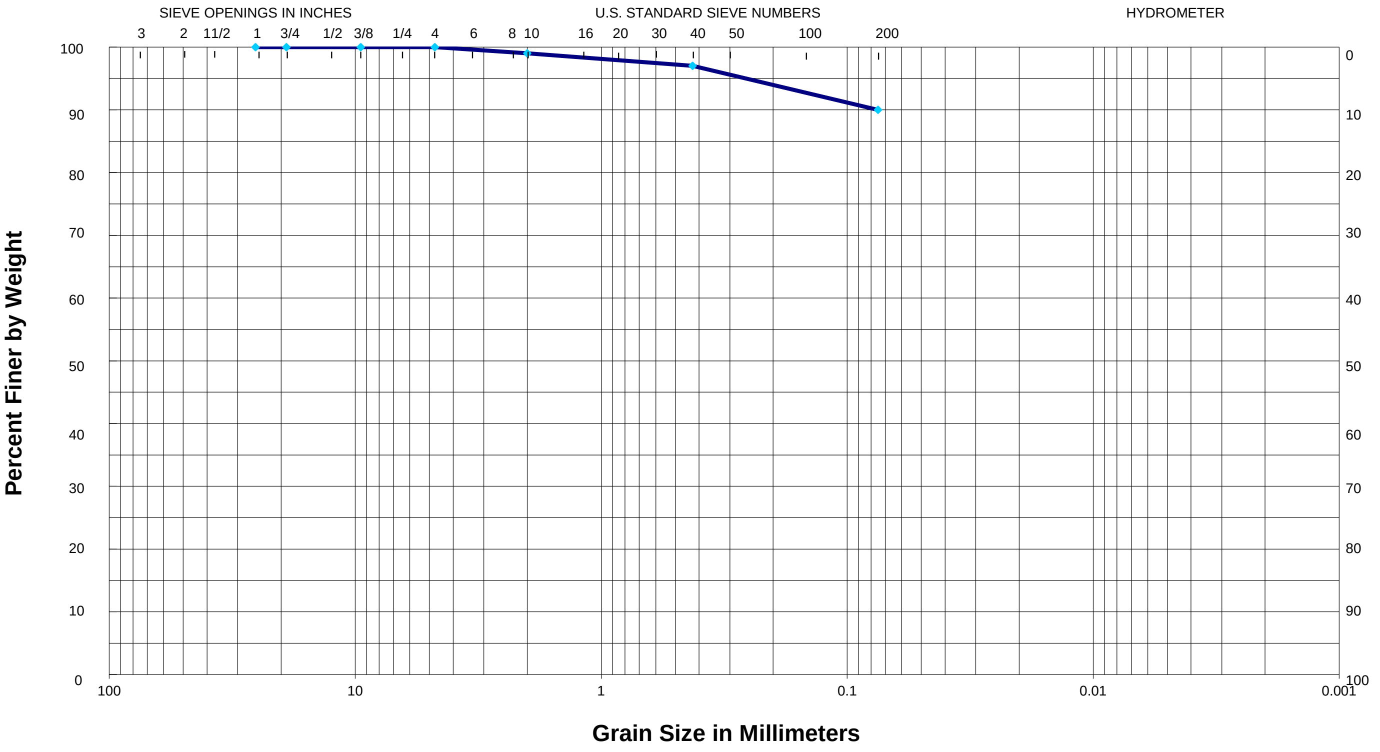
Sample: Boring W5, 9-10 ft
Description: Brownish red clay with chert fragments

USCS Classification: **CL**
AASHTO Classification: **A-7-6**

12-071

GRAIN SIZE CURVE

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



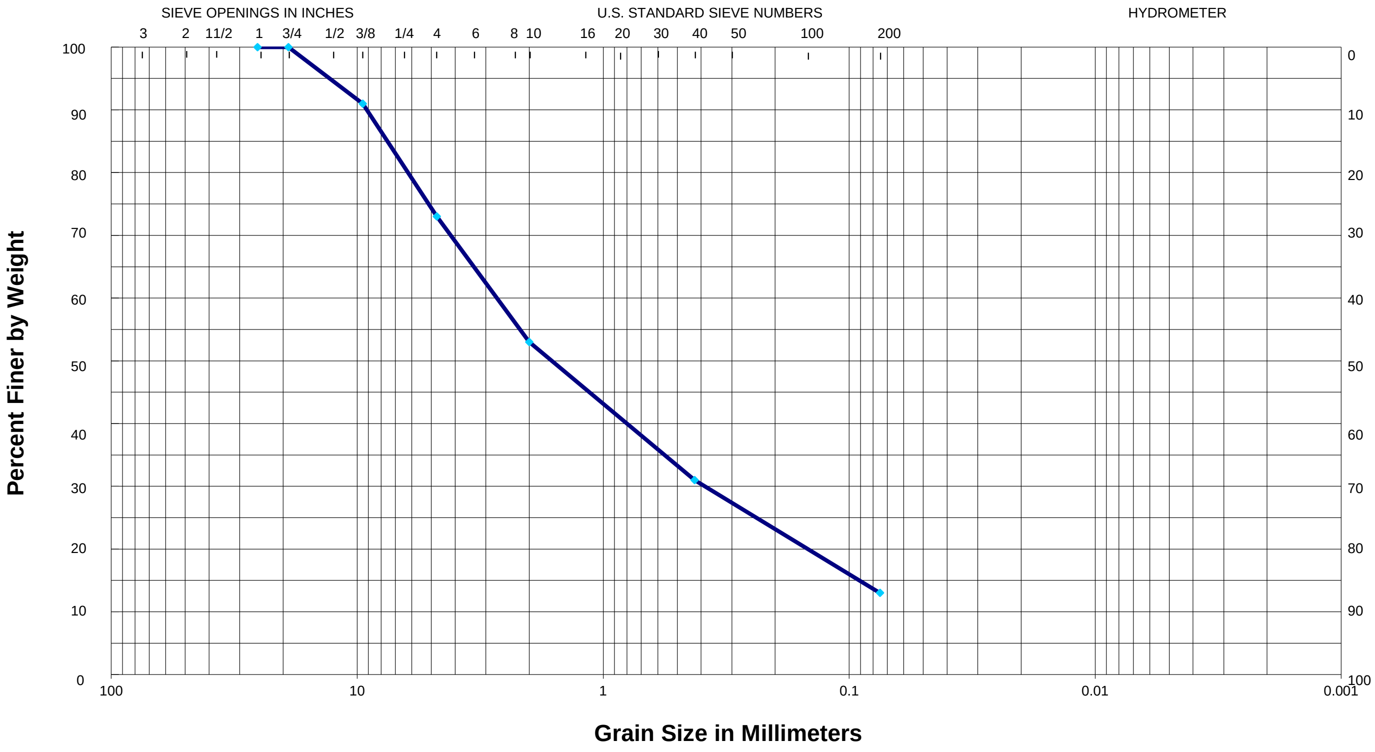
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring W10, 9-10 ft
Description: Reddish brown, reddish tan, and tan clay w/ chert and sandstone fragments, ferrous nodules and trace organics

USCS Classification: **CL**
AASHTO Classification: **A-7-6**

12-071

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

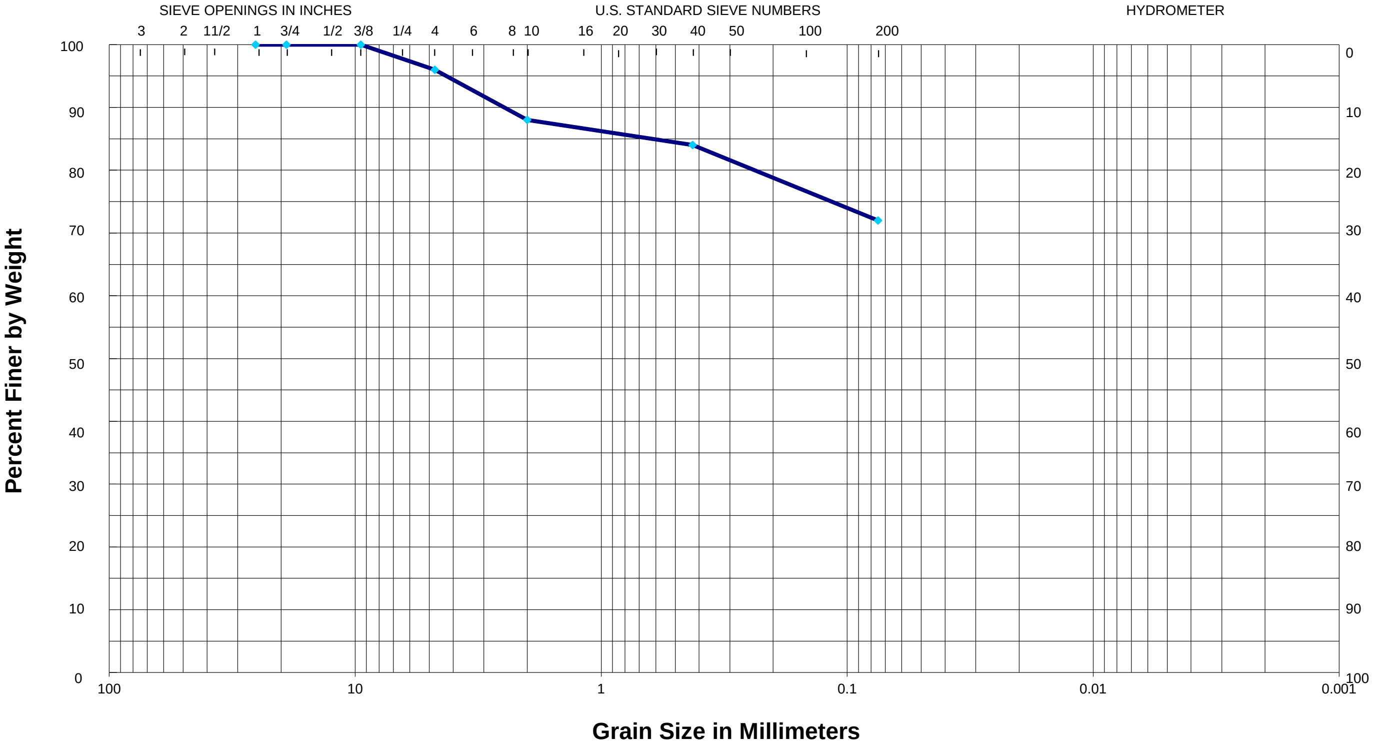
Sample: Boring W12, 0.5-1.5 ft
Description: Dark brown silty clay, sandy w/ crushed stone and trace organics

USCS Classification: **SM**
AASHTO Classification: **A-1-b**

12-071

GRAIN SIZE CURVE

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

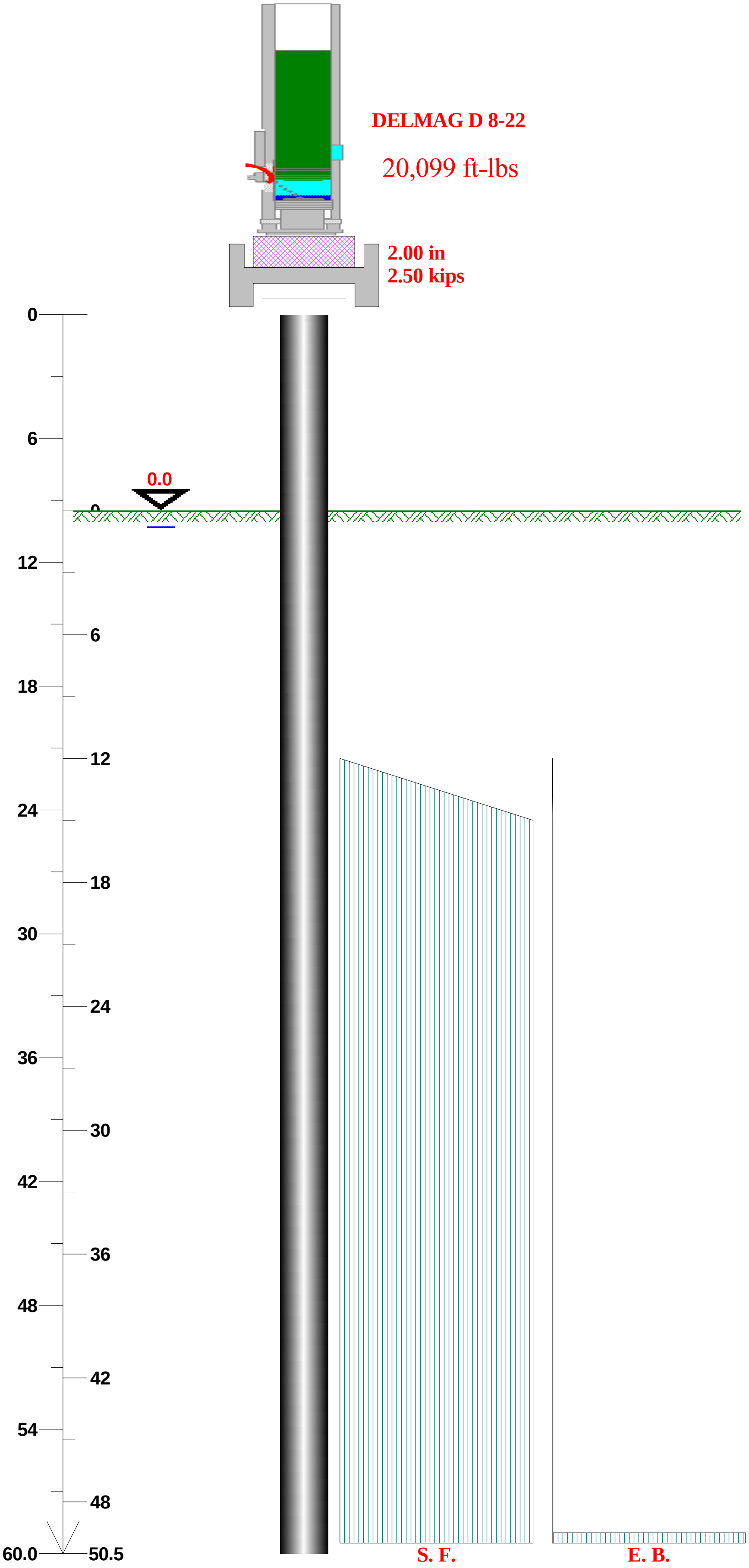


GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring W12, 6.5-7.5 ft
Description: Reddish brown and reddish tan silty clay w/ chert and sandstone fragments and ferrous nodules

USCS Classification: **CL**
AASHTO Classification: **A-7-6**

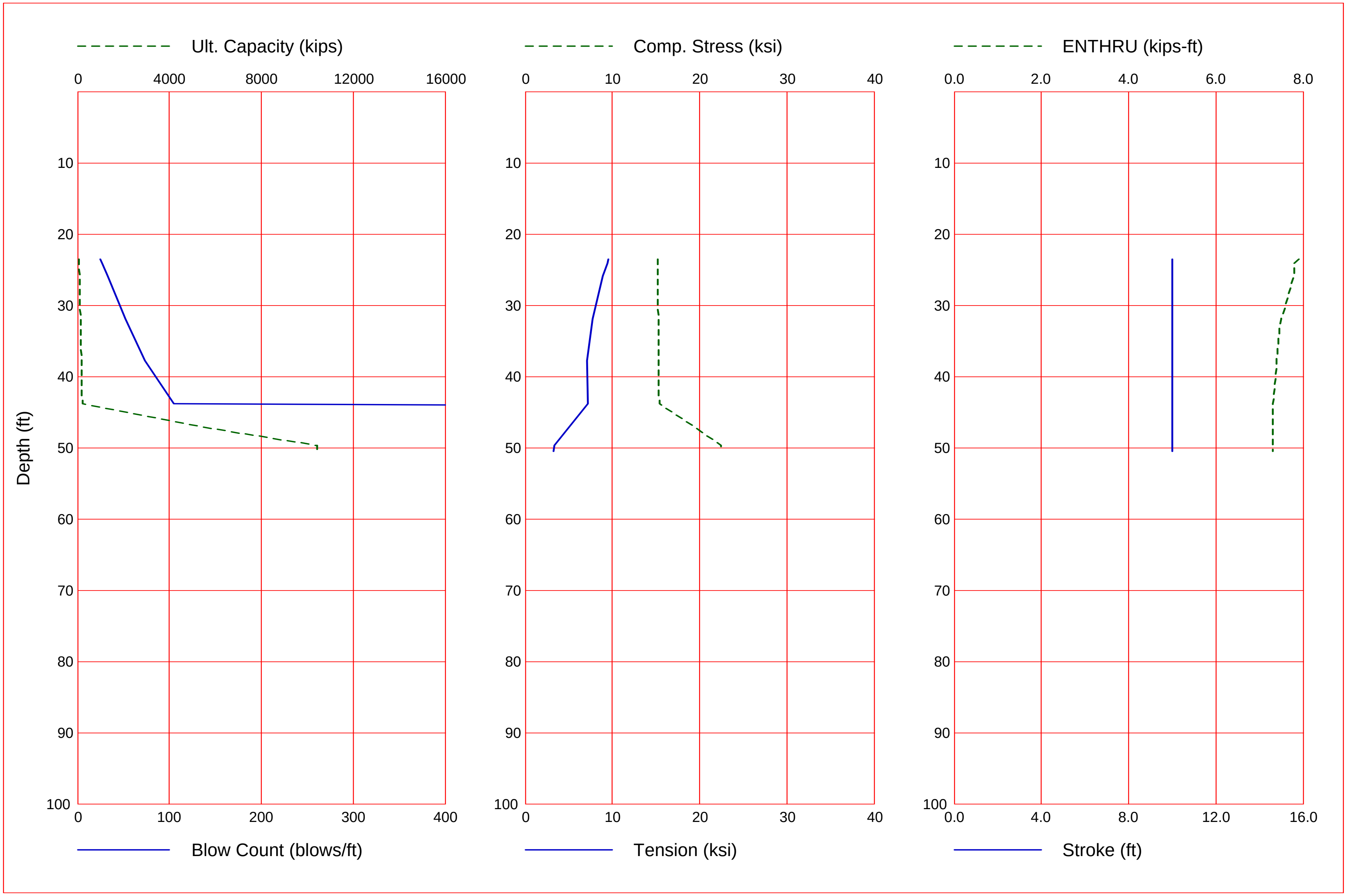
APPENDIX C


DELMAG D 8-22
20,099 ft-lbs
2.00 in
2.50 kips
0.0
0
6
12
18
24
30
36
42
48
54
60.0
0
6
12
18
24
30
36
42
48
50.5
S. F.
E. B.

Grubbs, Hoskyn, Barton & Wyatt, Inc.
BB0903 Bent 1 - D 8-22 (2) hammer

Jun 13 2016
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.500 / 1.000



Gain/Loss 1 at Shaft and Toe 0.500 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
23.5	76.3	71.4	4.9	25.0	15.189	-9.553	10.00	7.9
24.1	80.9	76.0	4.9	27.1	15.194	-9.387	10.00	7.8
25.9	93.1	88.2	4.9	33.4	15.201	-8.958	10.00	7.8
31.9	135.9	131.0	4.9	52.6	15.280	-7.781	10.00	7.5
37.8	178.7	173.9	4.9	73.9	15.310	-7.071	10.00	7.4
43.8	221.6	216.7	4.9	104.9	15.406	-7.190	10.00	7.3
49.7	10423.1	258.1	10165.0	9999.0	22.407	-3.337	10.00	7.3
50.5	10429.2	264.2	10165.0	9999.0	22.280	-3.255	10.00	7.3

Refusal occurred; no driving time output possible

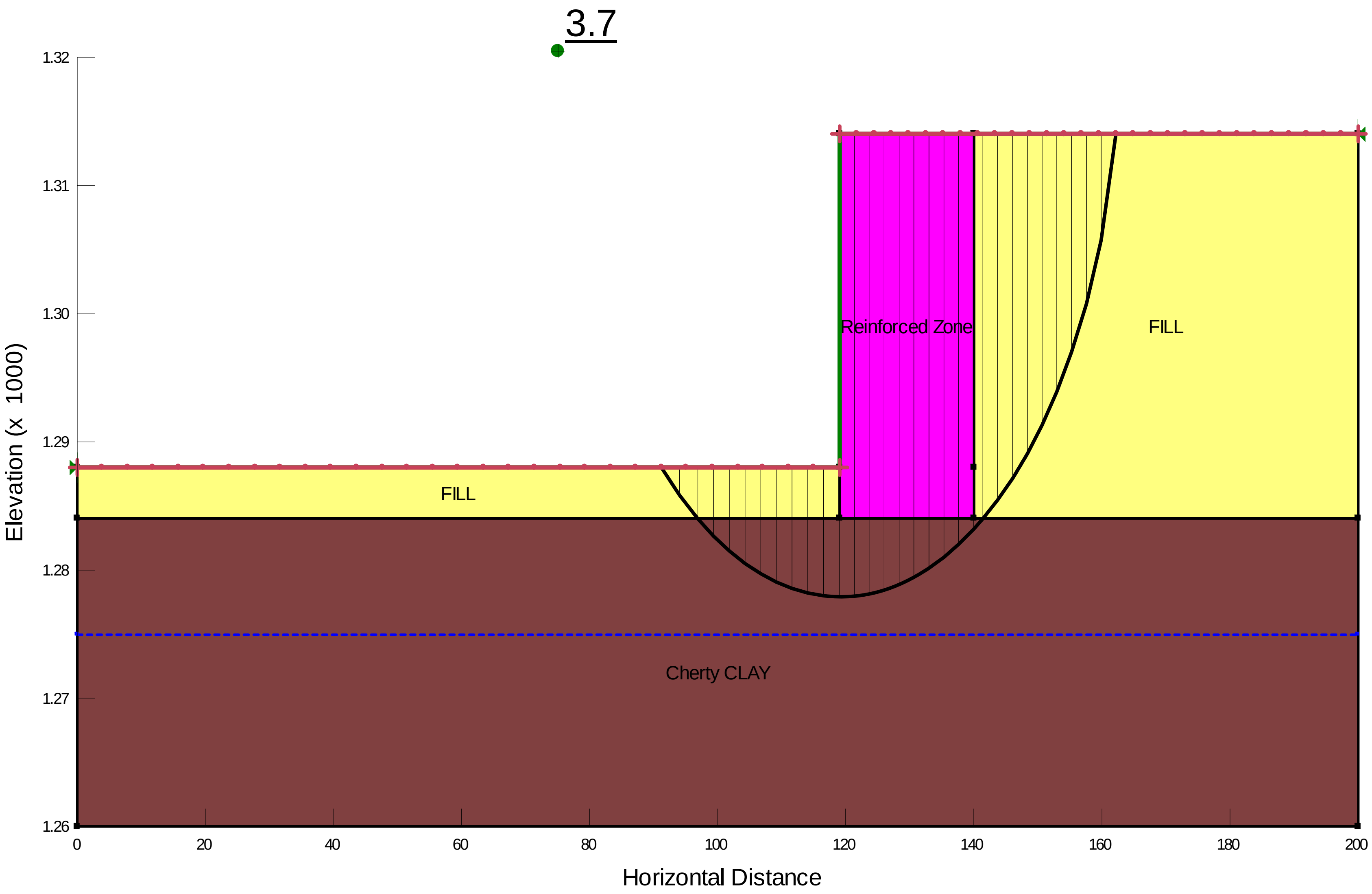
APPENDIX D

Summary of Global Stability Analysis Results
MSE Walls
AHTD Job No. BB0903 – HWY 71B INTERCHANGE IMPROVEMENTS

Design Loading Condition	Calculated Minimum Factor of Safety
End of Construction	3.7
Long Term	1.7
Seismic ($k_h = 0.5A_S = 0.03$)	1.6

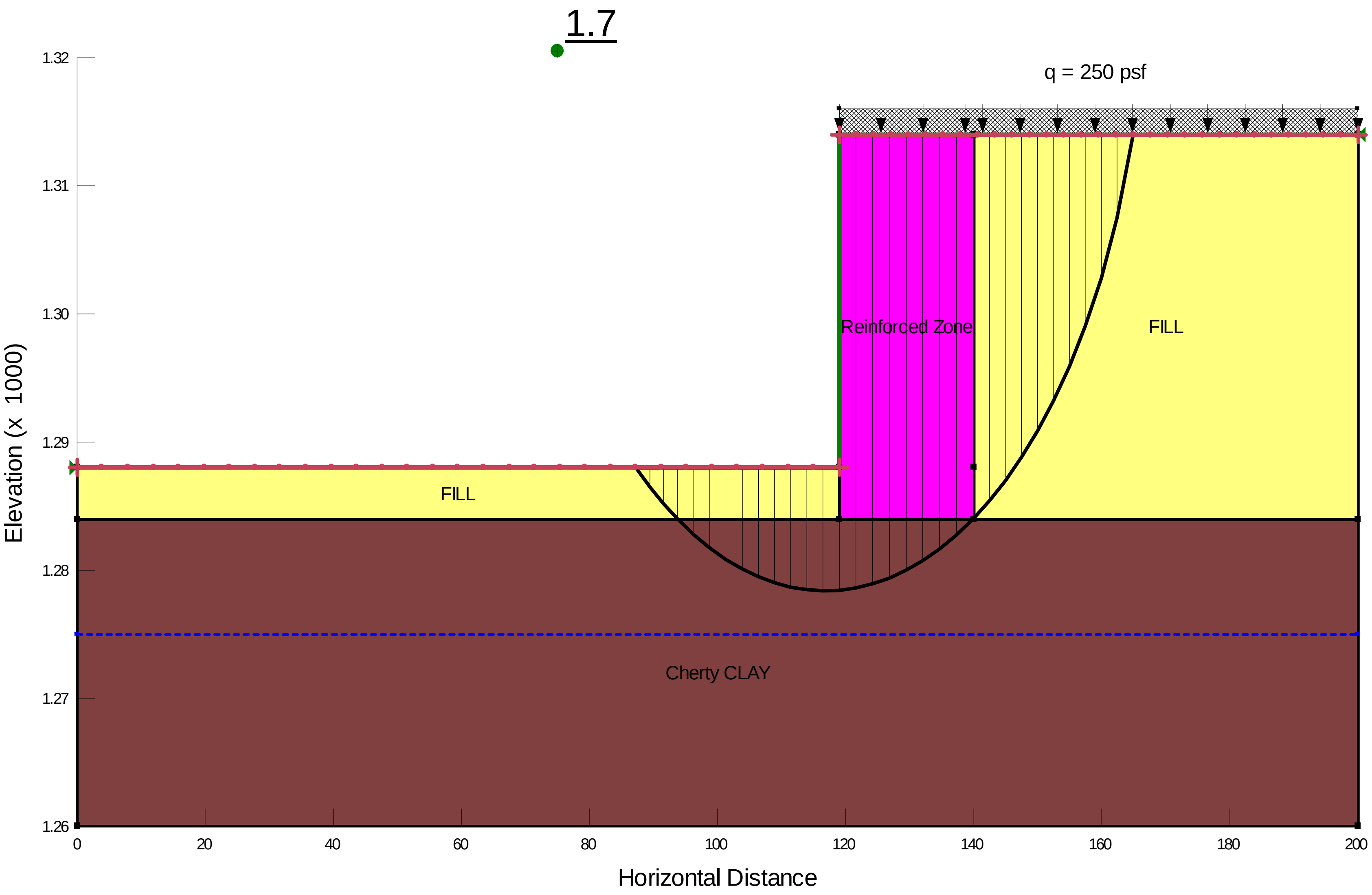
Summary of Soil Strength Parameters

Soil Description	Total Unit Weight (γ) pcf	Undrained Shear Strength (s_u) psf	Effective Cohesion (c') psf	Effective Friction Angle (φ') deg
Fill	125	2000	225	20
Cherty Clay	115	2250	225	24



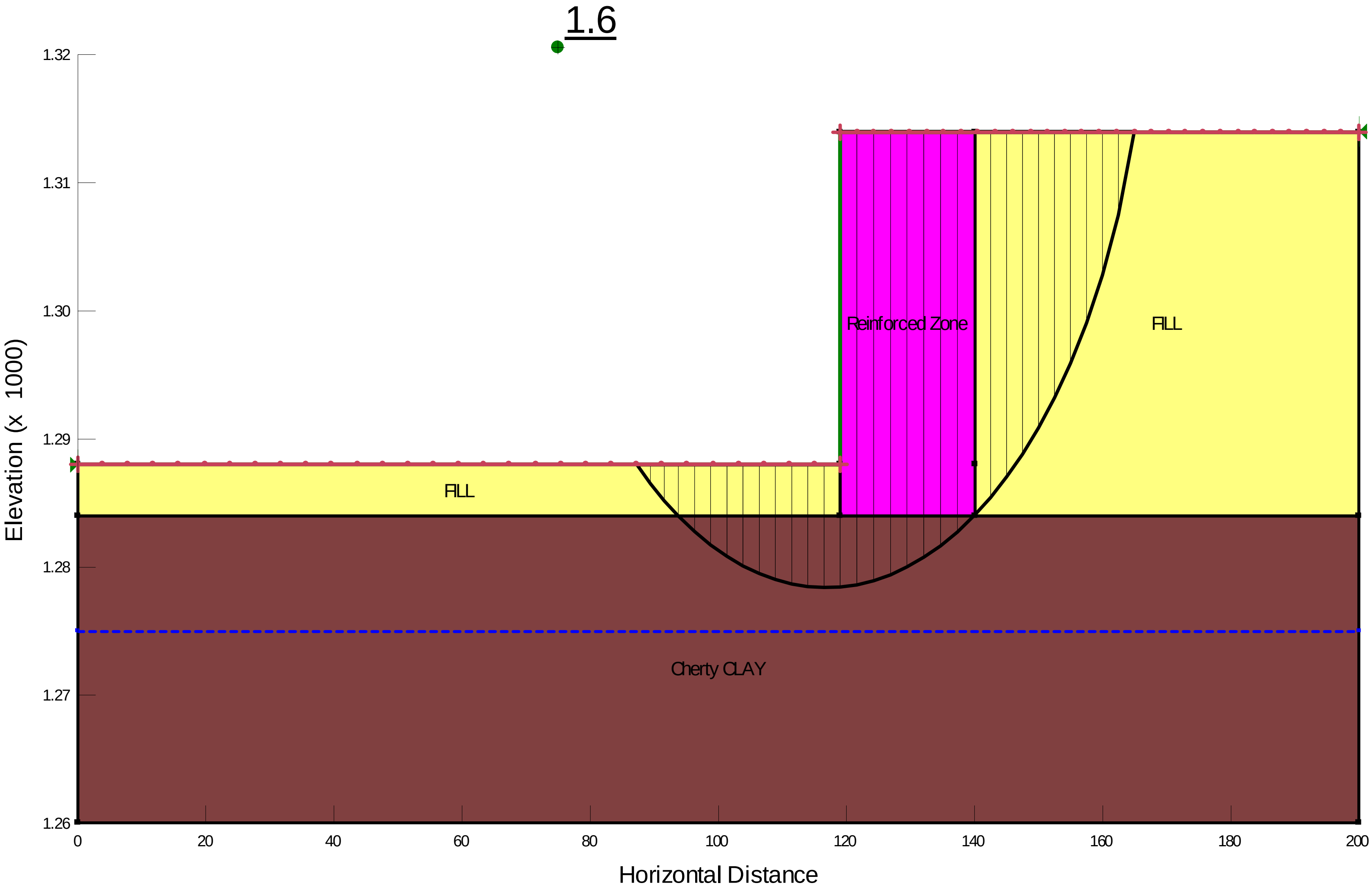
Results of Global Stability Analyses – End of Construction Condition
MSE Walls
AHTD Job No. BB0903 – HWY 71B INTERCHANGE IMPROVEMENTS

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



Results of Global Stability Analyses – Long Term Condition
MSE Walls
AHTD Job No. BB0903 – HWY 71B INTERCHANGE IMPROVEMENTS

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



Results of Global Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.03$)
MSE Walls
AHTD Job No. BB0903 – HWY 71B INTERCHANGE IMPROVEMENTS

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

Grubbs, Hoskyn, Barton & Wyatt, INC.
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P.O. Box 30970
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#1 Trigon Place 72209
(501) 455-2536
FAX (501) 455-4137

August 24, 2015
Job No. 12-071

Crafton Tull & Associates, Inc.
901 North 47th Street, Suite 200
Rogers, Arkansas 72756

Attn: Mr. Mike Burns, P.E.

GEOTECHNICAL INVESTIGATION
TASK ORDER No. B005
AHTD JOB No. 090305 – HWY. 71B INTCHNG. IMPVTS. (F)
BENTONVILLE, BENTON COUNTY, ARKANSAS

INTRODUCTION

This report presents the results of the geotechnical investigation performed for Arkansas State Highway and Transportation Department (AHTD) Job No. 090305: Hwy. 71B Intchng. Impvts. (F). This project consists of the proposed widening of the Interstate 49 bridges over Hwy 71B at Exit 35 in Bentonville, Benton County, Arkansas. These services were authorized by the Crafton Tull & Associates, Inc. subconsultant agreement dated July 13, 2013. Notice to proceed with this project phase was received on May 30, 2015.

It is understood that the project consists of widening the existing I-49 bridges (Bridges A5977 and B5977) over Hwy 71B. The widening plans include adding one (1) inside lane to both the southbound (A) and northbound (B) bridges over Hwy 71B (SE Walton Boulevard). The widened bridges will be approximately 185-ft-long, continuous composite W-beam units. The widened bridges will have four (2) bents with spans on the order of 44, 95, and 44 feet. The widened portion of the bridge decks will be 29.5 ft wide from the connection with the existing bridge to the outside of the deck. Preliminary plans are to support the widened sections on steel pile foundations. It is understood that the existing bridge foundations are steel piles at the bridge ends (Bents 1 and 4) and footings at the interior bents (Bents 2 and 3). Site grading associated with the widening project is expected to be minor. The existing bridge end slopes are configured at 2-horizontal to 1-vertical (2H:1V) with concrete riprap. The existing side slopes are configured at 3H:1V. It is also understood that the existing embankment slopes will be incorporated into the widened bridges.

The purposes of this geotechnical study were to explore subsurface conditions at the bridge widening location and to develop recommendations to guide design and construction of foundations for the widening. These purposes were achieved by a multi-phased study that has included:

- Drilling sample borings to evaluate soil, rock, and groundwater conditions at the bridge location and to obtain samples for laboratory testing.
- Performing laboratory tests to evaluate pertinent engineering properties of the foundation and subgrade strata.
- Analyzing field and laboratory data to develop recommendations for foundation design and construction considerations.

The data developed through the field and laboratory portions of this study have been considered in developing the conclusions and recommendations discussed in the following report sections.

SUBSURFACE EXPLORATION

Subsurface Investigation

Subsurface conditions were explored at the Hwy 71B bridge location by drilling four (4) sample borings to 60- to 75-ft depth. The boring locations were selected by the Engineer. The locations were then field adjusted as required for drill rig access. The bridge boring locations were staked in the field by Grubbs, Hoskyn, Barton & Wyatt, Inc. (GHBW). The subsurface exploration program is summarized below in Table 1.

Table 1: Summary of Subsurface Exploration Program

Boring No.	Project Feature / Location	Approximate Surface Elevation, ft	Boring Completion Depth, ft
S1	Bent 1, Sta 1243+96.80	1307.8	75
S2	Bent 2, Sta 1244+71.61	1286.9	60
S3	Bent 3, Sta 1245+48.29	1288.0	60
S4	Bent 4, Sta 1246+10.93	1308.0	75

The project site is shown on the attached Plate 1. The approximate boring locations are shown on the Plan of Borings, Plate 2. Boring logs, showing descriptions of the soil and rock strata encountered and results of the field and laboratory tests, are included as Plates 3 through 6. The ground surface elevation at each boring location, as provided by the Engineer, is also shown on each log. Keys to the terms and symbols used on the logs are presented on Plates 7 and 8 for soil and rock, respectively. A generalized subsurface profile is provided on Plate 9.

The borings were drilled with truck-mounted SIMCO 2800 and Mobil B-53 rotary-drilling rigs using a combination of dry-auger and rotary-wash drilling procedures. Soil samples were typically obtained in the borings at 2-ft intervals to a depth of 10 ft and at 5-ft intervals thereafter. Samples were typically obtained using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb safety hammer (with the Mobile B-53) or an automatic hammer (with the SIMCO 2800) with 30-in. drop 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). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column. Where rock hardness precluded recovery via the SPT, cuttings were obtained for use in visual classification.

Due to the rocky overburden soils, a core barrel could not be advanced downhole. Consequently, no rock coring was performed.

All samples were extruded or otherwise removed from samplers in the field. Samples were visually classified by the geotechnical technician and placed in appropriate containers to prevent moisture loss and/or disturbance 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. Groundwater levels were measured during drilling operations. Observations regarding groundwater are noted in the lower-right portion of each log and are discussed in subsequent sections of this report.

LABORATORY TESTING

Laboratory testing was performed to evaluate pertinent physical and engineering characteristics of the subgrade and foundation soil and rock. The laboratory testing program included natural water content determinations and classification tests. A total of 29 natural water content determinations (AASHTO T 265) were performed to develop data on *in-situ* soil water contents 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, ten (10) liquid and plastic (Atterberg) limit determinations (AASHTO T 89 and T 90) and ten (10) sieve analyses (AASHTO T 88) 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 percent of soil passing the No. 200 Sieve is noted in the “No. 200%" column on the log forms. Classification test results, as well as soil classification by the Unified Soil Classification System (ASTM D-2487) and AASHTO classification system (AASHTO M 145), are summarized in Appendix B.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The bridge location is the I-49 and Hwy 71B (SE Walton Boulevard) interchange in Bentonville, Benton County, Arkansas. The existing structures are two-lane twin bridges. The bridges are constructed on earthen embankments with concrete riprap covered end slopes and grass-covered side slopes. Walton Boulevard at the I-49 bridge location is a five-lane major arterial roadway. The project locale is predominantly commercial development. As noted, the bridges cross over Walton Boulevard via embankments. The surrounding terrain is generally flat.

Site Geology

The Geologic Map of Arkansas[1] indicates that the I-49 bridge location is in the mapped outcrop of the early and middle Mississippian Period Boone Formation. The Boone Formation consists of limestone, chert and cherty limestone. The chert and limestone content varies widely, both horizontally and vertically. The limestone of the Boone is typically gray, compact, finely to coarsely crystalline, and massively bedded. The limestone of the Boone is nearly pure calcium carbonate and is soluble. As a result, sinkholes, caves and fissures can occur in the formation. The discontinuities in the rock mass are generally filled, or partially filled, with chert boulders, clay, and stalactitic and stalagmitic material. The chert in the Boone Formation is cryptocrystalline silica of organic origin. The chert may occur as widely separated nodules, connected nodules, in interbedded layers with limestone, and sometimes as beds. Unweathered chert is dense, hard, and brittle and exhibits a conchoidal fracture.

The Boone limestone typically weathers to red clay with numerous chert fragments, cobbles and boulders and discontinuous chert seams and layers (cherty clay). Though the residual

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[1] Geologic Map of Arkansas, Arkansas Geologic Commission and U.S. Geologic Survey; 1993

clay often exhibits high plasticity, the residual soils typically classify as GC, clayey gravel, by the Unified Soil Classification System.

Seismic Conditions

Based on the site geology, the average soil and rock conditions revealed by the borings, and our experience in the area, a Seismic Site Class C (very dense soil and soft rock profile) is considered fitting for the widened I-49 bridge site with respect to the criteria of the 2011 Guide Specifications for LRFD Seismic Bridge Design[2]. Given the bridge location and AASHTO code-based values, the 1.0-sec period spectral acceleration coefficient for Site Class B (S_1) is 0.051 and the 1.0-sec period spectral acceleration coefficient (S_{D1}) value for Site Class C is 0.087. Utilizing these parameters, Table 3.10.6-1[3] indicates that a Seismic Performance Zone 1 is fitting for the I-49 / Hwy 71B interchange bridge site. In reference to the 2011 edition of the AASHTO Guide Specifications, the Peak Ground Acceleration (PGA) having a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years) is predicted to be 0.059 for a Seismic Site Class C for the bridge location.

Subsurface Conditions

Based on the results of the borings performed at the I-49 / Hwy 71B interchange bridge site, the subsurface conditions at the bridge location may be generalized into the following strata.

Stratum I: The surface and near-surface soils are embankment fill and on-site fill. At the bridge end embankments the fill extends to 22- to 28-ft depth (approximately El 1286 to El 1280). At the roadway grades (see Borings S2 and S3), the fill extends to variable 4-ft depth (El 1283) to 2-ft depth (El 1286). The fill is comprised of firm to very stiff red to red and brown clay with chert fragments. Minor amounts of fine gravel are also present in the fill. Localized very stiff gray silty clay with limestone fragment fill is also present at depth (see Boring S1). The cherty clay fill exhibits variable fair to good compaction. SPT N-values in the fill range from 10 to 30 blows per foot. The average N-value of 17 blows per ft indicates average good compaction. The fill has low compressibility and moderate shear strength. Fill depth, content, and compaction may vary across the site.

Stratum II: Below the fill is natural stiff to very stiff red clay with chert fragments and seams (cherty clay). The chert content is variable and chert layers are present at depth. The cherty clay represents residual soil weathered from the underlying cherty limestone bedrock. The clay fraction of the cherty clay has variable low to high plasticity. The cherty clay has moderate shear strength and low compressibility.

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[2] Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, Washington, DC, American Association of State Highway and Transportation Officials, 2011

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

Stratum III: The basal stratum encountered in the borings is hard light gray and gray cherty limestone. The cherty limestone is strong. Minor amounts of drilling fluid loss indicate the possibility of open fractures or clay-filled voids in the limestone. However, no open voids or apparent karst zones were encountered or indicated by the borings in the limestone.

A Generalized Subsurface Profile is presented on Plate 9. It should be recognized that 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. Additionally, the natural transition between strata is generally gradual, and the stratigraphy described in the sections above may vary.

Groundwater Conditions

Groundwater was not encountered in the borings prior to the introduction of drilling fluids at 10- to 20-ft depths during drilling operations in June 2015. Though not encountered in the borings, there is the potential for shallow perched water to develop, particularly during periods of high seasonal precipitation. Perched water may accumulate in the overburden soils and fractured rock zones. Groundwater levels will vary with seasonal precipitation and surface runoff and infiltration.

ANALYSES and RECOMMENDATIONS

Bridge Foundations

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

We recommend that the structural loads of the widened bridge be supported on pile foundation systems. Recommendations for piling are discussed in the following report sections.

Pile Foundations

We recommend that foundation loads of the I-49 / Hwy 71B interchange bridge be supported on steel piles. The piles may be driven to capacity in the overburden cherty clay (Stratum II) or to refusal in the hard cherty limestone (Stratum III). As a minimum, piles should extend through the embankment fill and on-site fill (Stratum I) into the stiff to very stiff cherty clay overburden soils (Stratum II). We understand that HP12x53 piles are planned. We also understand that a minimum safe bearing capacity of 96 tons will be specified for piles. Other pile sizes or types may be evaluated if desired. We recommend that all piles be fitted with rock points.

Ultimate 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 dated June 2015. Ultimate pile capacity curves are provided in Appendix C.

The ultimate axial capacities shown in Appendix C have been developed based on single, isolated foundations. Piles bearing in the overburden soils and spaced closer than six (6) pile widths may develop lower individual capacity due to group effects. Group reductions of end-bearing piles founded in rock are not expected to be a factor.

Based on AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.35 is recommended for evaluation of factored compression capacity for piles founded in the overburden cherty clay. 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, etc. resistance factors of 1.0 and 0.8 are recommended for evaluating compression and uplift capacities, respectively. Post-construction settlement of piles driven to the recommended factored capacities should be less than 0.5 inch. Downdrag loads due to long-term embankment settlement are expected to be negligible in light of the age of the existing embankments with minor site grading expected for the widening project.

Bearing capacities of piles driven to refusal must be determined using the AASHTO Load and Resistance Factor Design (LRFD) structural design procedure[4]. We recommend that nominal (ultimate) resistance (P_n) of steel piles be determined based on the yield strength (f_y) of steel H piles and the net end area (A_{net}) of the section. Given that the piles will likely be driven to refusal in hard rock with the potential for driving damage, we recommend a maximum allowable stress (σ_{all}) of $0.25f_y$. An effective resistance factor (φ_c) of 0.50 is recommended for end bearing steel piles. This effective resistance factor for steel piles has been based on the assumption of difficult driving. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

It has been our experience that allowable pile capacities on the order of 96 tons are determined for f_y 50 ksi steel HP12x53 piles. This capacity is based on allowable stress design (ASD). However, the appropriate factored bearing capacity must be confirmed by the Engineer.

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

Post-construction settlement of piles driven to refusal will be negligible. As noted, downdrag loads due to long-term embankment settlement are expected to be negligible due to the age of the existing embankments and the expectation that no new embankments will be constructed.

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.

Based on the understanding that a minimum safe bearing capacity of 96 tons will be required for all piles, it is anticipated that piles will be driven to refusal in the hard cherty limestone (Stratum III). Estimated as-built pile lengths and tip elevations for the various bents are summarized in the table below. These estimated lengths and tip elevations have been developed based on piling being driven to practical refusal. Some hard driving is likely in zones with higher chert content. Preboring could be required for pile installation. Comments on preboring are included in the table below. We recommend that prebores through the overburden soils have a maximum diameter equal to about 70 to 75 percent of the pile diagonal dimension. Prebores in rock must have a sufficient diameter to prevent damage to the pile, i.e., larger than the H-pile diagonal dimension.

Table 2: Estimated Pile Length and Tip Elevations

Bent No.	**Estimated Pile Tip Elevation, ft(1)**	**Estimated Pile Length, ft (below estimated plan footing or cap bottom)**	**Comments**
1	1244	56	Preboring could be required through the embankment fill, with prebores estimated to extend to ±El 1286
2	1248	31	Preboring could be required, with 10-ft prebore estimated, i.e., prebore to ±El 1269
3	1251	28	Preboring could be required, with 10-ft prebore estimated, i.e., prebore to ±El 1269

Bent No.	**Estimated Pile Tip Elevation, ft$^{(1)}$**	**Estimated Pile Length, ft (below estimated plan footing or cap bottom)**	**Comments**
4	1252	50	Preboring could be required through the embankment fill, with prebores estimated to extend to ±El 1280

Note 1: Pile tip elevations are estimates only based on the results of the borings and information provided on the bridge drawings. As-built pile tip elevation must be field verified by the Engineer or Department.

Based on the results of drivability analyses utilizing wave equation methods[5], we recommend that steel HP12x53 be driven with a hammer system capable of delivering at least 31,000 ft-lbs per blow. The results of the wave equation drivability analyses are provided in Appendix D.

As a minimum, safe bearing capacity of test piles and production piles should be determined by AHTD Standard Specifications Section 805.09, Method A. Driving records should be available for review by the Engineer during pile installation. All piles should be driven to practical refusal, typically defined as a penetration of 0.5 in. or less for the final 10 blows.

Embankments

We understand that the bridge widening project will have no new embankments. We also understand that the existing embankments are presently stable and have no history of sliding or unusual maintenance issues. Consequently, stability considerations related to new embankments have not been evaluated.

Wingwall and Abutment Wall Lateral Earth Pressures

It is expected that wingwalls and abutment walls for the widened section will be backfilled with unclassified borrow or select material. Recommendations related to lateral earth pressures for wingwalls and abutments are summarized below.

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[5] GRLWEAP, Wave Equation Analysis of Pile Driving, Version 2010-3; 1998-2010, Pile Dynamics, Inc.

- 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:
    - Active condition for walls that are free to rotate, backfilled with unclassified borrow, and fully drained: 65 lbs per sq ft per ft depth.
    - Active condition for walls that are free to rotate backfilled with unclassified borrow and with no provision for internal drainage: 95 lbs per sq ft per ft depth.
- Total unit weight (γ) for SM-1: 125 lbs per cu ft
- Angle of internal friction (φ) for SM-1 backfill: 32°
- Equivalent fluid pressure for SM-1 backfill:
    - Active condition for walls that are free to rotate, backfilled with SM-1 or clean granular backfill, and fully drained: 40 lbs per sq ft per ft depth.
    - Active condition for walls that are free to rotate, backfilled with SM-1 or clean granular backfill, and with no provision for internal drainage: 85 lbs per sq ft per ft depth.
- Ultimate sliding resistance:
    - Interaction friction angle (δ) for concrete on stable bearing stratum of embankment fill or cherty clay: 19°.
    - Interaction friction factor (tan δ) for concrete on stable bearing stratum: 0.35.
    - The sliding resistance values above are nominal/ultimate values.
    - A resistance factor (φ) of 0.80 is recommended for sliding resistance.

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 or Department. Drainage zones should be fully isolated from the fine-grained cherty clay embankment fill and natural soils by an appropriate geotextile complying with the criteria of AHTD 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.

Site Grading Considerations

We expect that site grading will include some minor cut and fill placement. No substantial grading for new embankments is anticipated with the widening project. Site preparation in areas of incidental grading should begin with stripping the topsoil and any unsuitable surface soils. The stripping depth is expected to be on the order of 6 to 9 inches.

After stripping and performing any cut, and prior to placing fill, the subgrade should be evaluated by proof-rolling with a loaded tandem-wheel dump truck or similar equipment where accessible. Areas identified to be soft or that exhibit pumping should be undercut, processed and recompacted or replaced with suitable fill, whichever is appropriate. Based on the results of the borings, the potential for undercuts is considered low. Nevertheless, depending on seasonal site conditions and final grading plans, localized undercuts on the order of 2 ft below existing grades, more or less, could be warranted to stabilize localized areas of weak surface soils. Undercut requirements must be field verified by the Engineer or the Department during the work.

The existing 2H:1V end slopes and any expansion or extension of these slopes will warrant erosion protection. This may include concrete riprap, dumped riprap, or other suitable systems. Concrete riprap (AHTD Standard Specifications Section 816) is recommended for slope configurations steeper than 2.5-horizontal to 1-vertical (2.5H:1V).

Embankments should be constructed in accordance with AHTD criteria (AHTD Standard Specifications Section 210). Where localized seepage into undercuts or excavations is a problem, undercuts should be backfilled with SM-1 (AHTD Standard Specifications Section 302) or stone backfill (AHTD Standard Specifications Section 207) fully encapsulated in an appropriate filter fabric (AHTD Standard Specifications Subsection 625.02, Type 2). The granular backfill should be vented to positive discharge if possible.

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. Fills placed against existing slopes should be benched into the existing slope face as new fill is constructed to facilitate placement of horizontal lifts. The in-place density and water content should be determined for each lift of fill and backfill. Each lift of backfill and fill should be tested and approved prior to placing subsequent lifts.

CONSTRUCTION CONSIDERATIONS

Positive surface drainage should be established at the start of the work, be maintained during construction and following completion of the project to prevent surface water ponding and subsequent saturation of subgrade soils. Density and water content of all earthwork should be maintained until the embankment and bridge work is completed. Subgrade soils that become saturated by ponding water or runoff should be excavated to undisturbed soils. Embankment areas where additional site grading is planned should be evaluated by the Engineer or Department during subgrade preparation and prior to starting embankment construction.

Shallow groundwater was not encountered in the borings drilled in June 2015. Minor seepage into isolated excavations can probably be controlled by ditching or sump-and-pump methods. If seepage into excavations becomes a problem, backfill should consist of clean sand (AHTD Standard Specifications Section 302, SM-1) or clean, crushed stone (AHTD Standard Specifications Section 207). Sand or stone backfill should be encapsulated in filter fabric (AHTD Standard Specifications Subsection 625.02, Type 2) and vented to positive discharge at daylight or into storm drainage lines where possible.

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. In areas of seepage infiltration, the granular fill should be fully encapsulated with a filter fabric complying with AHTD Standard Specifications Subsection 625.02, Type 2.

Piles should be installed in compliance with AHTD Standard Specifications Section 805. Piles should be carefully examined prior to driving and piles with structural defects should be rejected. Any splices in steel piles should develop the full cross-sectional capacity of un-spliced piles. Some preboring may be required for pile installation. Depending on the specific location, rock drilling methods may be required for prebores advanced into limestone/cherty limestone. Prebores should have adequate width to accommodate the pile width. We recommend that after piles are installed in prebores, the annulus around piling be backfilled with sand grout, lean concrete, or an alternate approved by the Engineer or the Department.

Pile installation should be monitored by qualified personnel to maintain specific and complete driving records and to observe pile installation procedures. Driving records should be available for review by the Engineer and/or Department during pile installation.

We recommend that HP12x53 be driven with a minimum 31,000 ft-lbs per blow hammer system. The pile-hammer system proposed by the Contractor should be specifically reviewed by the Engineer or Department prior to acceptance for the work. Blow counts on steel piles should be limited to about 20 blows per inch. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

CLOSING

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

The following illustrations are attached and complete this report.

Plate 1	Site Vicinity Map
Plate 2	Plan of Borings
Plates 3 through 6	Boring Logs
Plates 7 and 8	Keys to Terms and Symbols
Plate 9	Generalized Subsurface Profile
Appendix A	Bridge Layout Drawings
Appendix B	Classification Test Results
Appendix C	Ultimate Pile Capacity Curves
Appendix D	Results of Drivability Analysis

* * * * *

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

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

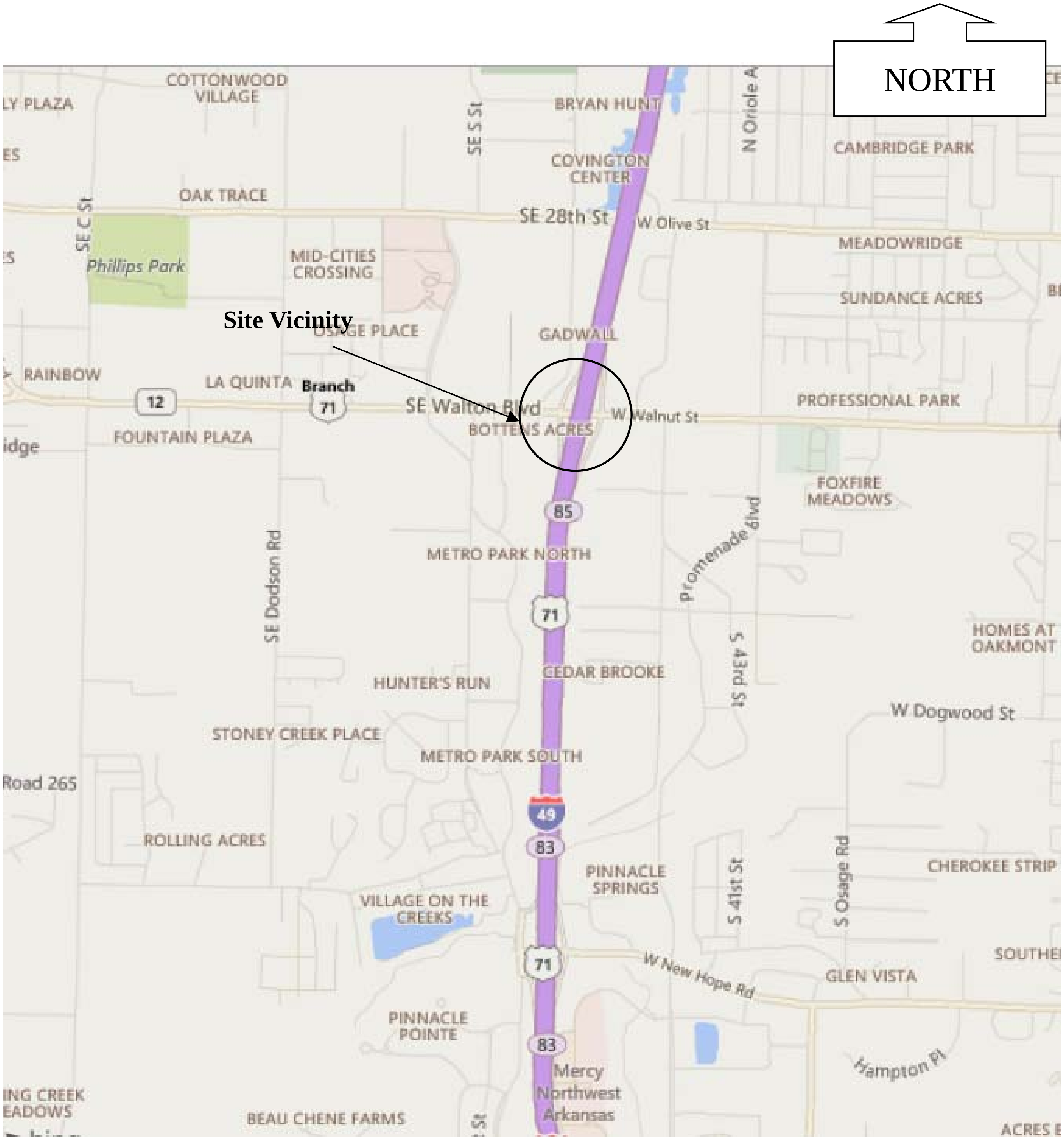
Matthew R. Satterfield, P.E.
Senior Project Engineer

Mark E. Wyatt, P.E.
President

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

MRS/MEW:jw

Copies Submitted: Crafton, Tull & Associates, Inc.
Attn: Mr. Mike Burns, P.E. (1+electronic)

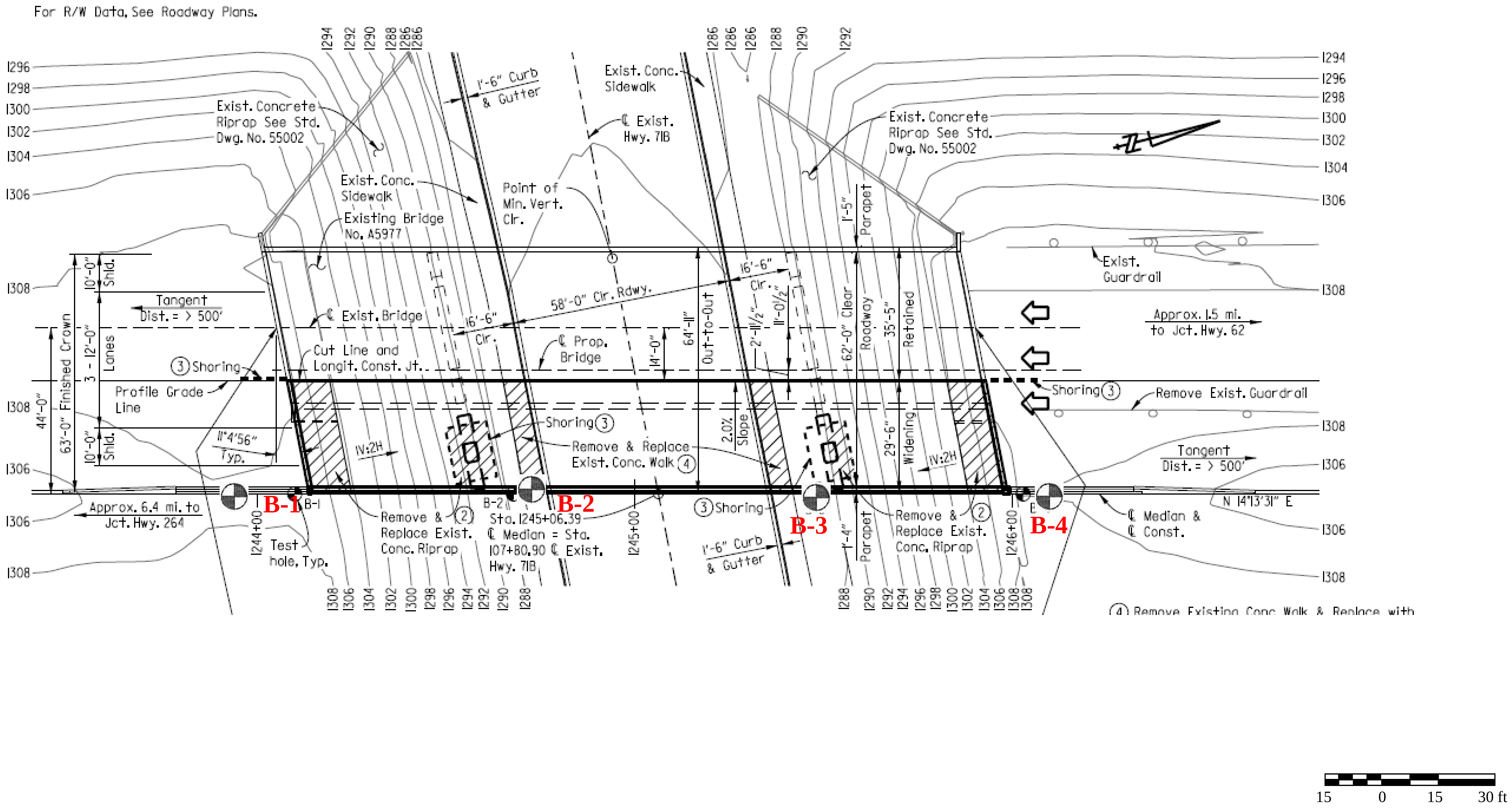


Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

SITE VICINITY MAP
090305: Hwy. 71B Intchng. Impvts. (F)
Benton County, Arkansas

Job No. 12-071

Plate 1



PLAN OF BORINGS
090305: Hwy. 71B Intchng. Impvts. (F)
BENTON COUNTY, ARKANSAS

Scale: As shown

Job No. 12-071

PLATE 2

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

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

LOG OF BORING NO. S1

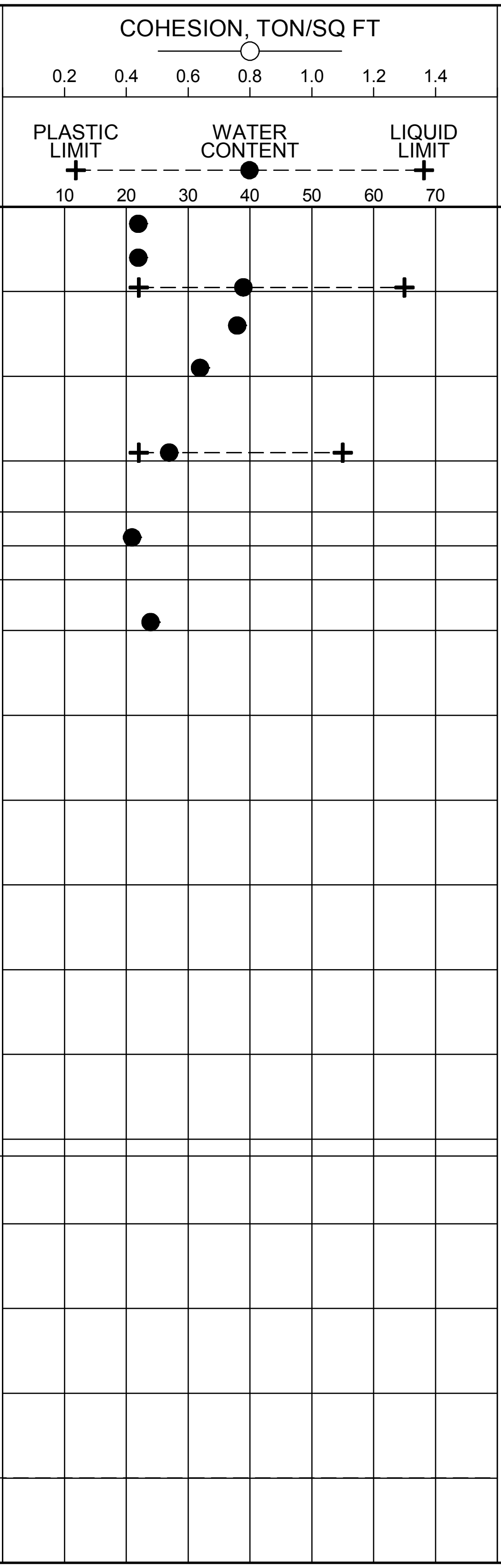
090305; I-49 over Hwy 71B
Benton County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1243+96.80, 1.5 ft Rt

SURF. EL: 1307.8

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff to very stiff red clay w/chert fragments (fill)	13		
		17		
5		22		72
		29		
10		29		
15	- with chert cobbles below 14 ft	30		63
20	Very stiff gray silty clay w/limestone fragments and trace organics (fill)	25		
25	Very stiff red clay w/numerous chert fragments (cherty clay)	37		
30	- with chert cobbles and seams below 28 ft	50/4"		
35	- with chert cobbles, seams and layers below 33 ft	50/3"		
40		25/0"		
45		25/0"		
50		25/0"		
55		50/11"		
	Hard light gray and gray cherty limestone			
60		25/0"		
65		25/0"		
70	- no loss of circulation in cherty limestone to completion depth	25/0"		
75		25/0"		



COMPLETION DEPTH: 75.0 ft
DATE: 6-23-15

DEPTH TO WATER
IN BORING: Dry to 20 ft
DATE: 6/23/2015

12-071

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

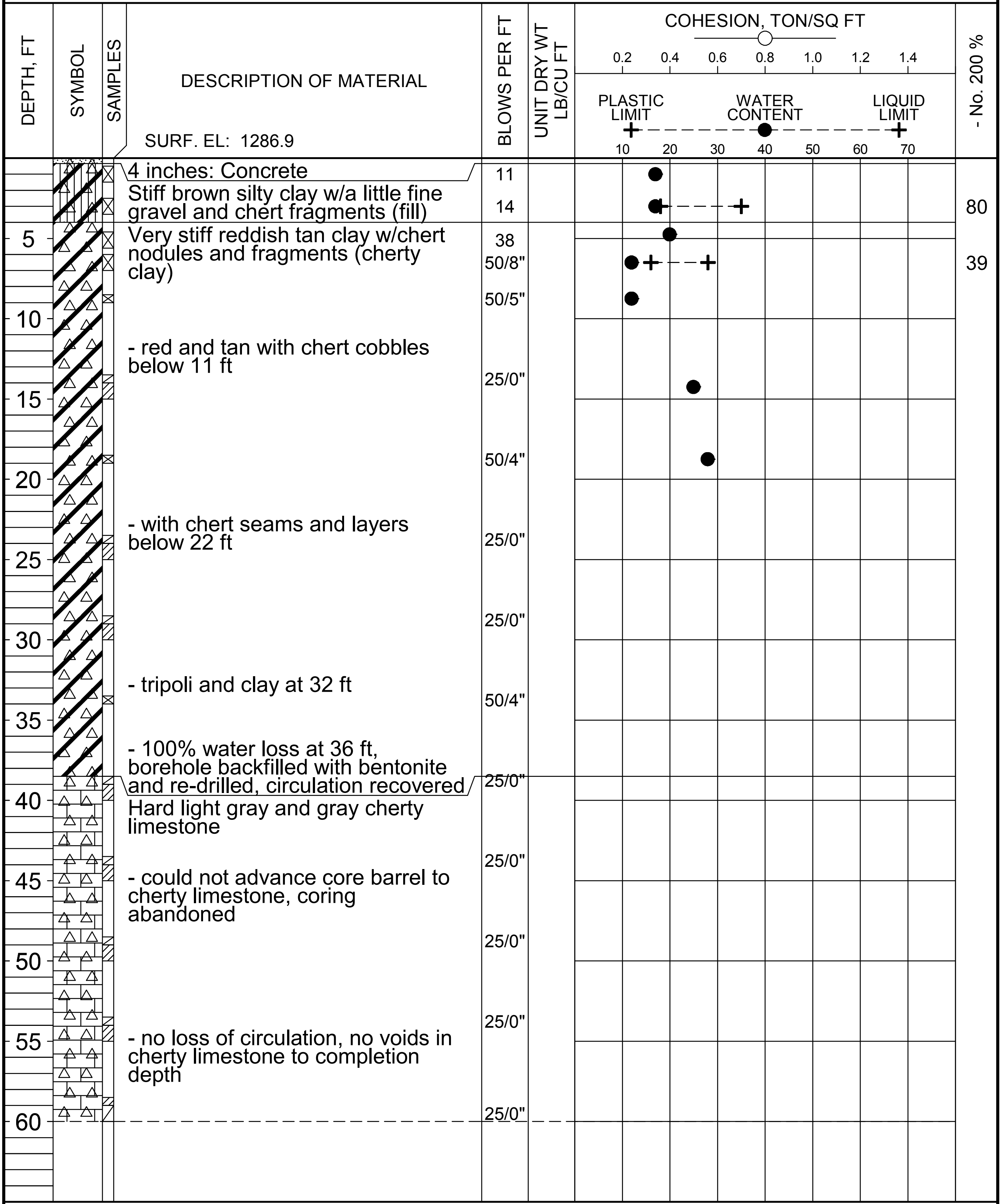
L O G O F B O R I N G N O. S2

090305; I-49 over Hwy 71B
Benton County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1244+71.61, 1 ft Lt

SURF. EL: 1286.9



DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
4 inches: Concrete		
Stiff brown silty clay w/a little fine gravel and chert fragments (fill)	11	
	14	80
Very stiff reddish tan clay w/chert nodules and fragments (cherty clay)	38	
	50/8"	39
	50/5"	
- red and tan with chert cobbles below 11 ft	25/0"	
	50/4"	
- with chert seams and layers below 22 ft	25/0"	
	25/0"	
- tripoli and clay at 32 ft	50/4"	
- 100% water loss at 36 ft, borehole backfilled with bentonite and re-drilled, circulation recovered	25/0"	
Hard light gray and gray cherty limestone		
- could not advance core barrel to cherty limestone, coring abandoned	25/0"	
	25/0"	
- no loss of circulation, no voids in cherty limestone to completion depth	25/0"	
	25/0"	

COMPLETION DEPTH: 60.0 ft
DATE: 6-17-15

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 6/17/2015

LGBNEW 12-071.GPJ 8-10-15

PLATE 4

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

LOG OF BORING NO. S3

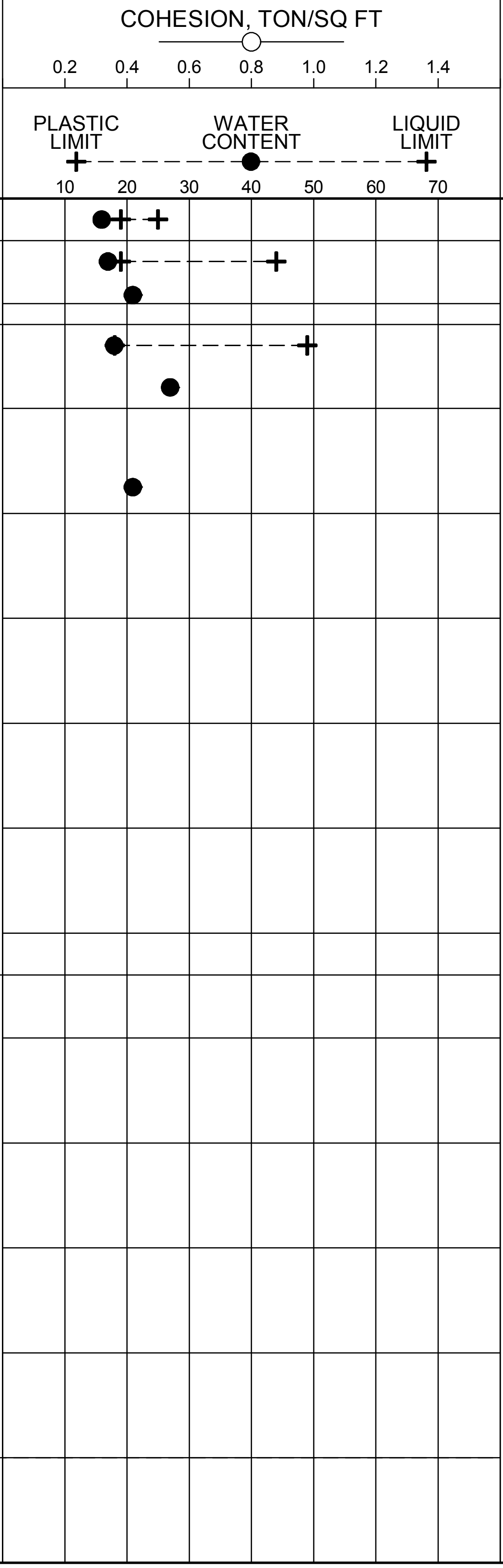
090305; I-49 over Hwy 71B
Benton County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1245+48.29, 12 ft Rt

SURF. EL: 1288.0

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Stiff brown silty clay w/a little fine gravel and occasional organics (fill)	14		85
Firm to stiff reddish tan clay with chert fragments - stiff, red, gray and tan below 4 ft	10		81
	15		
Very stiff red clay w/numerous chert nodules and fragments (cherty clay) - with chert cobbles below 10 ft - with medium close chert seams and layers below 16 ft	50/9"		53
	50/8"		
	50/6"		
	25/0"		
	50/4"		
	25/0"		
	25/0"		
Hard gray and light gray cherty limestone - 10% water loss at 40 ft; circulation recovered - could not advance core barrel to cherty limestone due to chert fragment fall in, coring abandoned	25/0"		
	25/0"		
	25/0"		
	25/0"		
	25/0"		



DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60

COMPLETION DEPTH: 60.0 ft
DATE: 6-16-15

DEPTH TO WATER
IN BORING: Dry to 10 ft
DATE: 6/16/2015

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

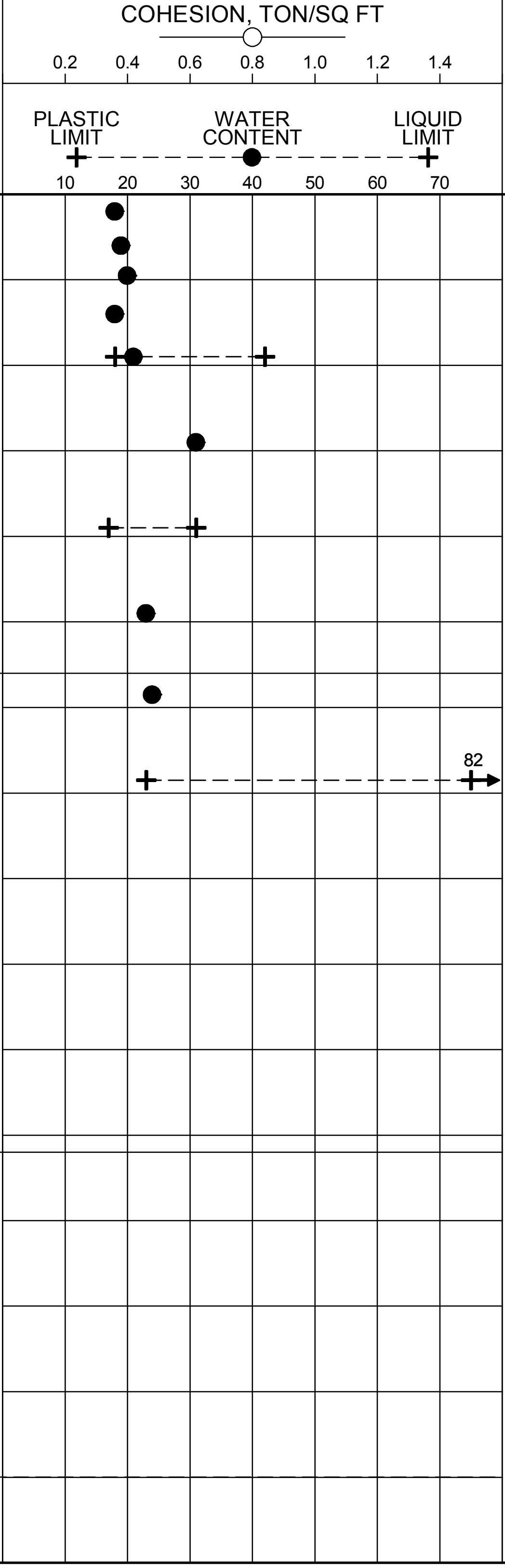
LOG OF BORING NO. S4

090305; I-49 over Hwy 71B
Benton County, Arkansas

TYPE: Auger to 20 ft /Wash
LOCATION: Sta 1246+10.93, CL

SURF. EL: 1308.0

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brownish red and brown silty clay w/chert fragments (fill)	13		
	- firm to stiff at 2 to 4 ft	10		
5	- stiff below 4 ft	15		
		16		
10		14		72
15	- with more chert below 14 ft	16		
20	- red, gray and brown below 18 ft	14		74
25		16		
30	Very stiff red and tan clay w/numerous chert fragments and seams (cherty clay)	50/3"		
35		50/4"		53
40		50/6"		
45		50/9"		
50		50/5"		
55	- with more chert below 53 ft - 20% water loss at 55 ft	25/0"		
60	Hard light gray and gray cherty limestone - fluid circulation recovered at 48 ft	25/0"		
65		25/0"		
70				
75		25/0"		



COMPLETION DEPTH: 75.0 ft
DATE: 6-22-15

DEPTH TO WATER
IN BORING: Dry to 20 ft
DATE: 6/22/2015

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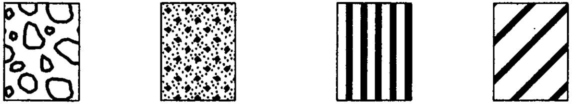
Barton & Wyatt, Inc.
Consulting Engineers

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

(SHOWN IN SYMBOLS COLUMN)

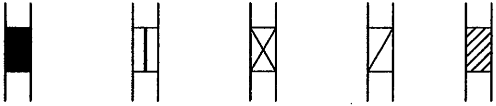


Gravel Sand Silt Clay

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 (l) 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 6-30-10

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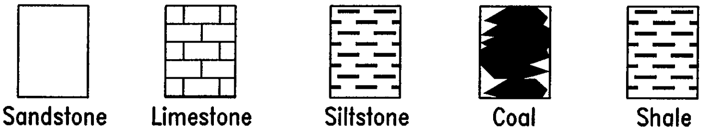
Consulting Engineers

BORING LOG TERMS – ROCK

ROCK TYPES
(SHOWN IN SYMBOLS COLUMN)



	Spacing	
Joint Characteristics –	Very Close	0.75 to 2.5 in.
	Close	2.5 to 8 in.
	Moderately Close	8 to 24 in.
	Wide	2 to 6 ft
	Very Wide	More than 6 ft
Bedding Characteristics –	Very Thin	0.75 to 2.5 in.
	Thin	2.5 to 8 in.
	Medium	8 to 24 in.
	Thick	2 to 6 ft
	Massive	More than 6 ft

Lithologic Characteristics –
- Clayey
- Shaly
- Calcareous (limy)
- Siliceous
- Sandy (Arenaceous)
- Silty
- Plastic Seams

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

Hardness–

Soft (S) – Reserved for plastic material alone.

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

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

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

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

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

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

Structure –
- Bedding
  - Flat – 0° – 5°
  - Gently Dipping – 5° – 35°
  - Moderately Dipping – 55° – 85°
  - Steeply Dipping – 55° – 85°
- Fractures, scattered
  - Open
  - Cemented or Tight
- Fractures, closely spaced
  - Open
  - Cemented or Tight
- Brecciated (Sheared and Fragmented)
  - Open
  - Cemented or Tight
- Joints
  - Faulted
  - Slickensides

Degree of Weathering –

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

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

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

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

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

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

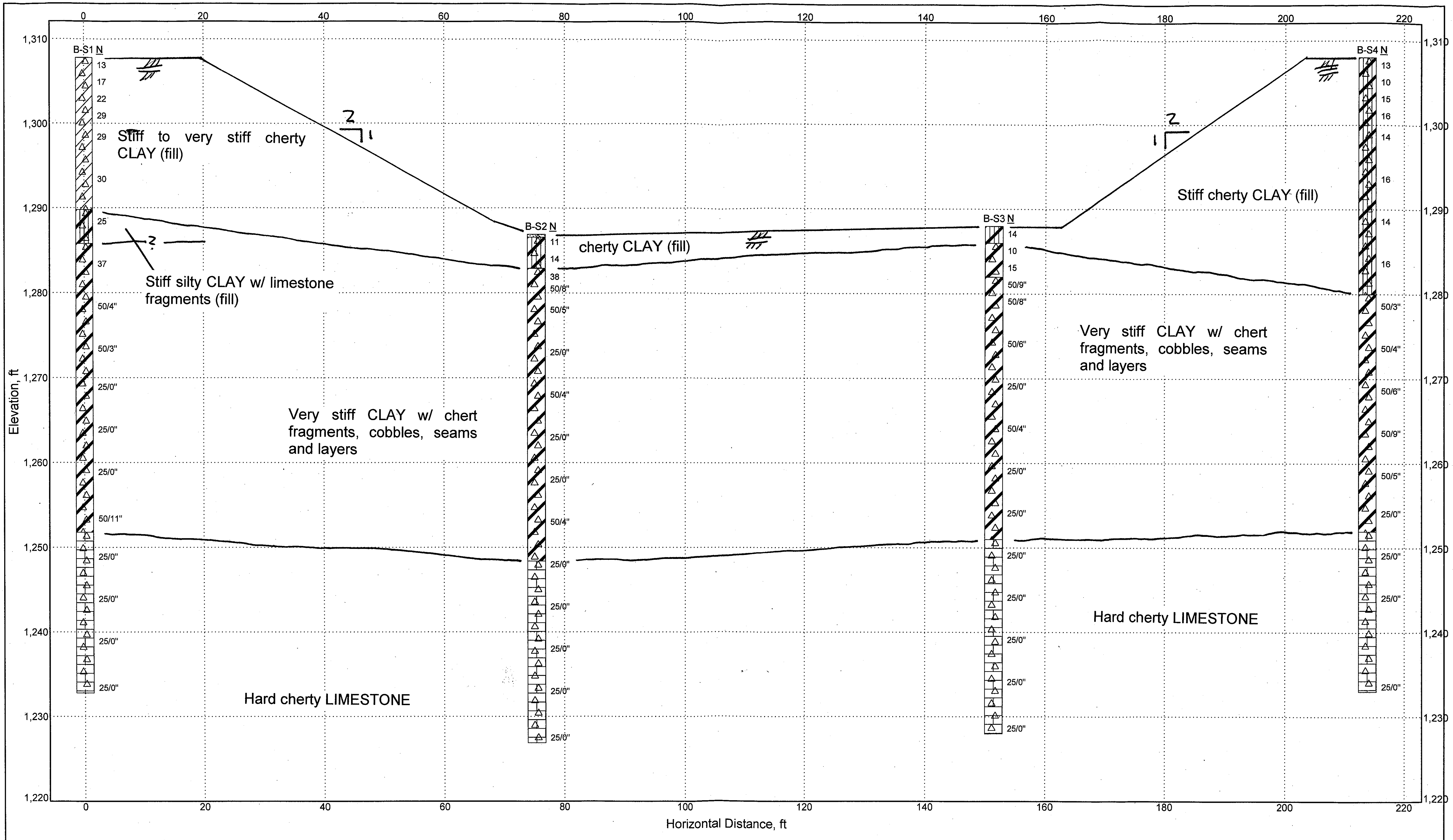
Solution and Void Conditions –
- Solid, contains no voids
- Vuggy (pitted)
- Vesicular (igneous)
- Porous
- Cavities
- Cavernous

Swelling Properties –
- Nonswelling
- Swelling

Slaking Properties –
- Nonslaking
- Slakes slowly on exposure
- Slakes readily on exposure

Rock Quality Designation (RQD) –

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



Grubbs, Hoskyn,
Barton & Wyatt, Inc.

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

SCALE:
AS SHOWN

Generalized Subsurface Profile
090305; I-49 over Hwy 71B
Benton County, Arkansas
Project Number: 12-071

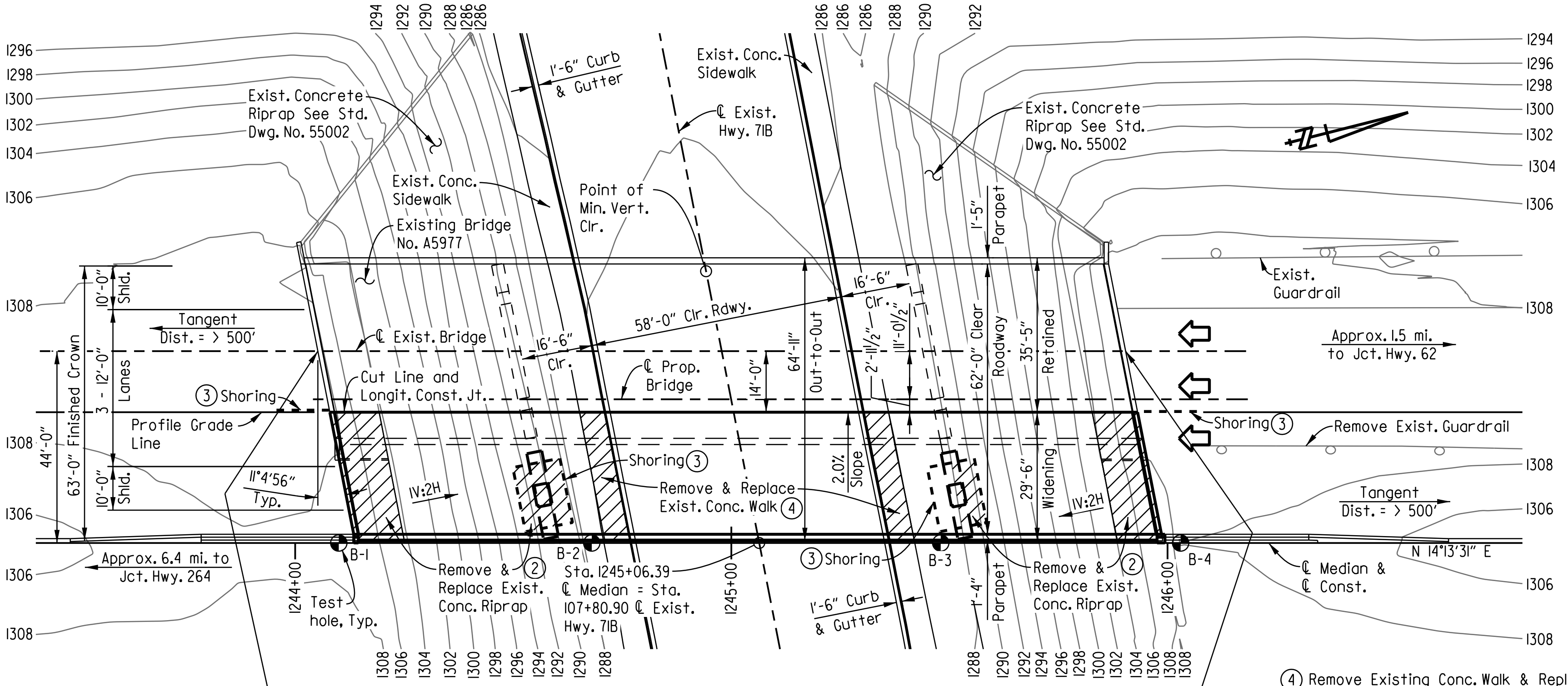
PLATE 9

APPENDIX A

Crafton, Tull & Associates Inc.	DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
					6	ARK.			
					JOB NO.		090305	1	6

① A5977 LAYOUT XXXXX

For R/W Data, See Roadway Plans.



PLAN

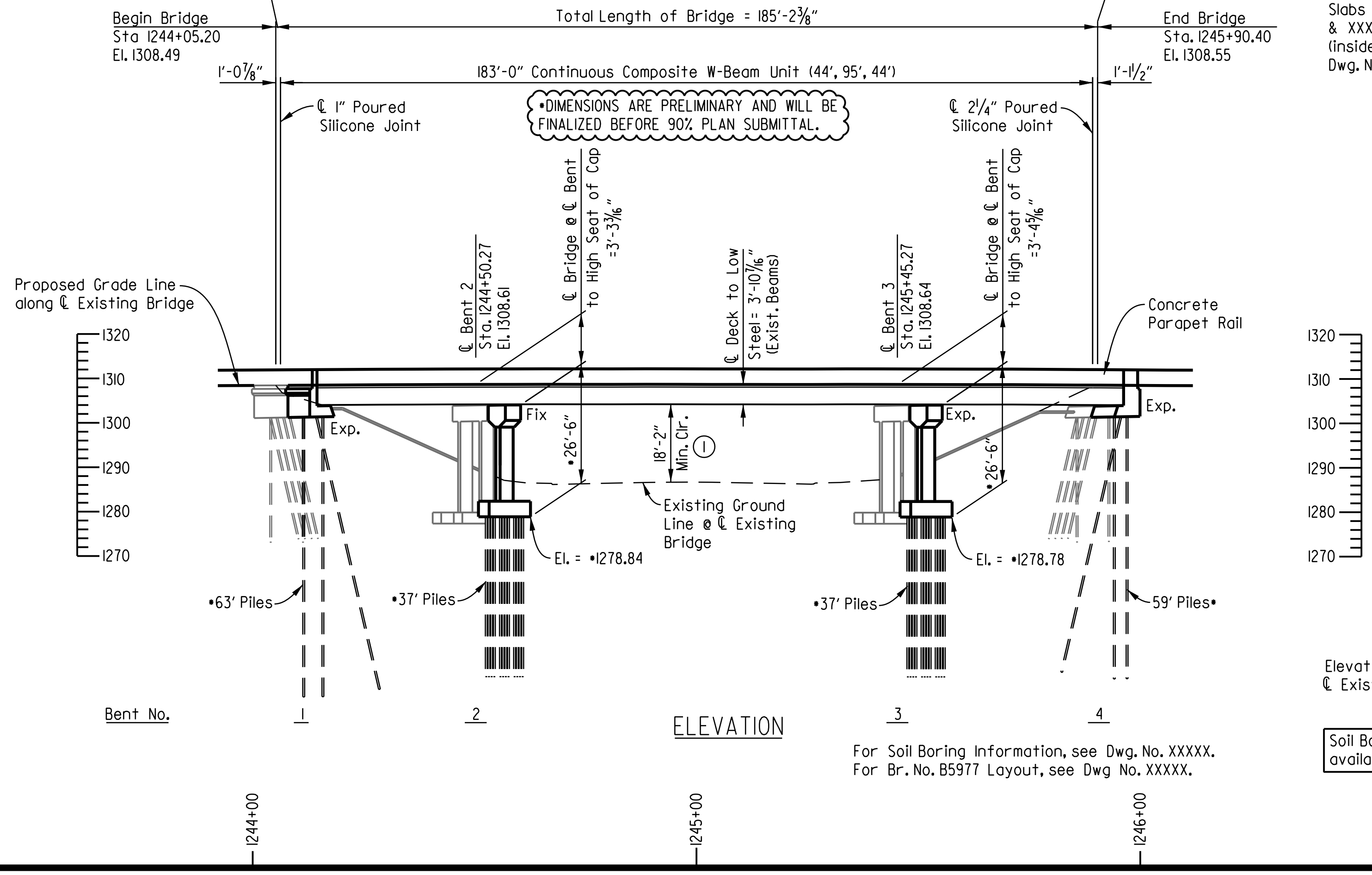
① Minimum Vertical Clearance

② Remove Existing Concrete Riprap & Replace with New Concrete Riprap as necessary for Bent construction (See Std. Dwg. No. 55002). Removal shall extend to the location of an existing construction joint. Payment for this work shall be considered subsidiary to the pay item "Modification of Existing Bridge Structure (Br. No. A5977)".

③ Shoring may be required during construction. Payment for this work, if required, shall be considered subsidiary to the pay item "Unclassified Excavation for Structures - Bridge". See SP Job No. 090305 "Shoring".

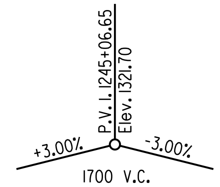
④ Remove Existing Conc. Walk & Replace with New Conc. Walk as necessary for Bent construction (See Std. Dwg. No. SI-1). Removal shall extend to the location of an existing construction joint. Payment for this work shall be considered subsidiary to the pay item "Modification of Existing Bridge Structure (Br. No. A5977)".

Remove Existing Approach Gutters (inside gutters) at both ends of Bridge. Use Type Special I Approach Slabs at both ends of Bridge, see Dwg.Nos. XXXXX & XXXXX. Use Type Special I Approach Gutters (inside gutters) at both ends of Bridge, see Dwg. No. XXXXX.



ELEVATION

For Soil Boring Information, see Dwg. No. XXXXX.
For Br. No. B5977 Layout, see Dwg No. XXXXX.



VERTICAL CURVE DATA

Along Profile Grade Line

Elevations and Stations shown are along ℄ Existing Bridge, unless noted otherwise.

Soil Boring Information not available at time of printing

GENERAL NOTES

BENCH MARK: ATHD Cap set in S.W. corner of Br. No. B5977, 68.69' Rt., Centerline, Sta. 1244+26.74, Elev. 1308.04.

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 Standard Specifications for Highway Bridges (17th Edition, 2002) with interim Specifications.

LIVE LOADING: HS20 DESIGN METHOD: LOAD FACTOR

SEISMIC PERFORMANCE CATERGORY: A

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (Grade 60 AASHTO M 31 or M 322 Type A)	fy = 60,000 psi
Structural Steel (AASHTO M 270, Gr. 50)	Fy = 50,000 psi
Structural Steel (AASHTO M 270, Gr. 36)	Fy = 36,000 psi

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

STEEL PILING: All piling shall be HP 12x53 (Gr. 50) and shall be driven with an approved air, steam, or diesel hammer to a minimum safe bearing capacity of 96 tons per pile into the material designated as XXXX XXXXXXX on the boring legend. Lengths shown are for estimating quantities and for use in determining payment for cut-off and build-up in accordance with the Standard Specifications. Piles in End Bent 1 & 4 to be driven after embankment to bottom of cap is in place. On all piles the contractor shall use approved steel H-Pile driving points.

PREBORING: Preboring will be required for all piling and shall extend from bottom of footing or bottom of cap to depth indicated on the plans. A preboring depth of 32' is anticipated at End Bents. A preboring depth of 8' is anticipated at Bents 2 and 3. The quantities for preboring shown are for bidding purposes only. The actual size and depth of preboring are to be determined in the field by the Engineer. The length of prebored hole shall be backfilled in accordance with Subsection 805.08(a) to completely fill the remaining voids. The Contractor shall be responsible for keeping prebored holes free from debris prior to back filling which may require the use of temporary casings or other methods. Pile casings and backfill will not be paid for directly but shall be considered subsidiary to the item "Preboring".

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.

CLASS 2 PROTECTIVE SURFACE TREATMENT: Class 2 Protective Surface Treatment shall be applied to the roadway surface including the existing deck surface to be retained, and roadway face and top of parapet rail of the new construction.

PAINT: All new Structural Steel except galvanized members, surfaces in contact with concrete, and as otherwise noted, shall be painted as specified in Subsection 807.75. The color of paint shall match existing beams.

	DRAWING NUMBER
DETAIL DRAWINGS:	XXXXX-XXXXX
Stage Construction	XXXXX-XXXXX
End Bents	XXXXX-XXXXX
Intermediate Bents	XXXXX
Elastomeric Bearings	XXXXX-XXXXX
Widening Details	XXXXX-XXXXX
Type Special Approach Slab	XXXXXX-XXXXX
Type Special Approach Gutters	

For Additional General Notes see Dwg. No. XXXXX

PRELIMINARY
SUBJECT
TO
REVISION

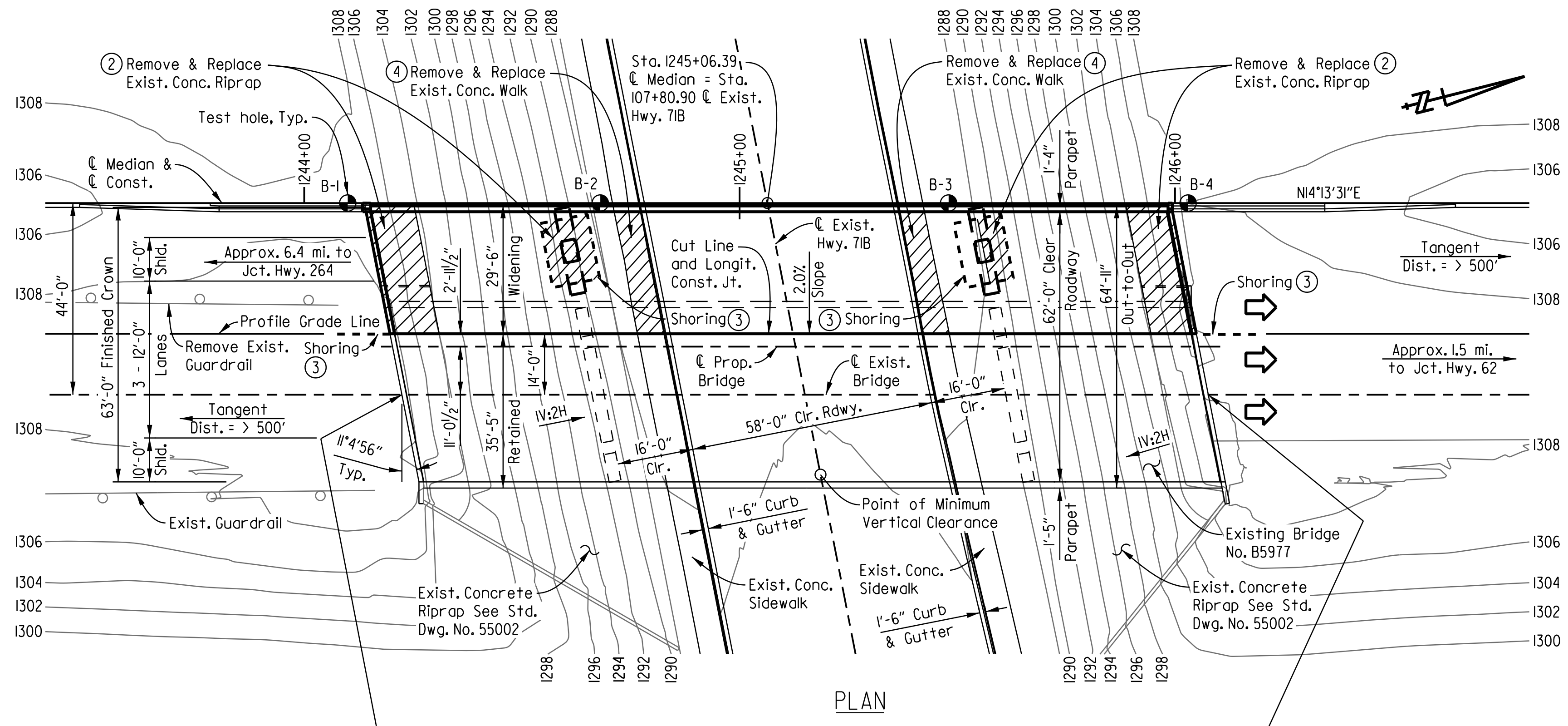
SHEET 1 OF X
LAYOUT OF BRIDGE 'A' OVER
HWY. 71B
NEW HOPE RD. - HWY. 62/102
WIDENING (ROGERS) (S)
BENTON COUNTY
ROUTE 49 SEC. 29
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: BWC DATE: 05-22-15 FILENAME: b090305al_11.dgn
CHECKED BY: CAW DATE: 06-04-15 SCALE: 1" = 20'
DESIGNED BY: KJC DATE: 05-15-15
BRIDGE NO. A5977 DRAWING NO. XXXXX

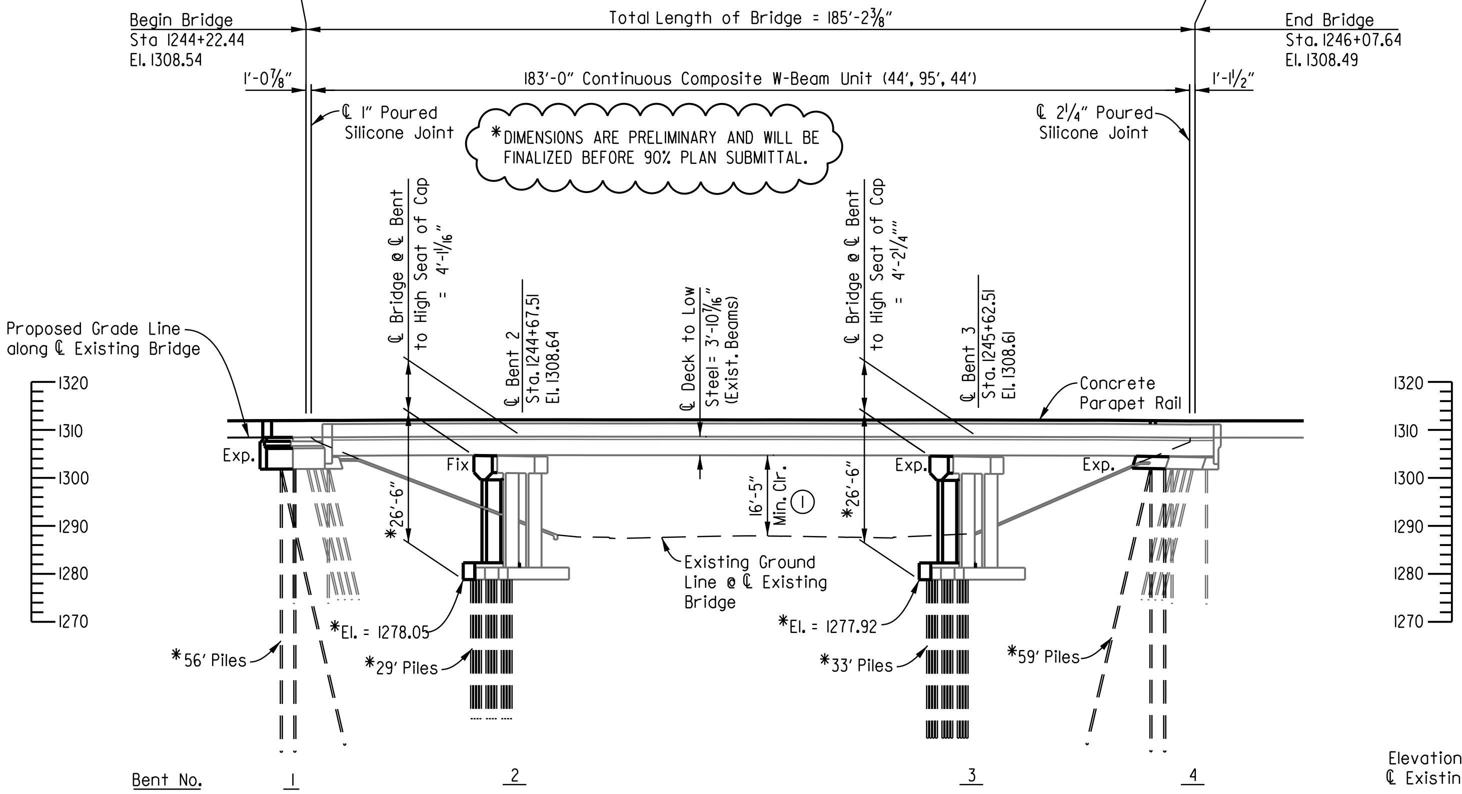
USER: bc5100
DESIGN FILE: G:\12103305_Hwy71Inchg\TRANSP\090305\dgn\bridge\b090305al_11.dgn
PLOTTED: 6/11/2015 16:22 SCALE: 40:1

Crafton, Tull & Associates Inc.	DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
					6	ARK.			
					JOB NO.		090305	2	6
					① B5977		LAYOUT		XXXXX

For R/W Data, See Roadway Plans.



PLAN



ELEVATION

GENERAL NOTES (con't)

THE PROPOSED WORK CONSISTS OF: Widening the existing bridge, modifying the existing end bents, constructing new intermediate bents to the median side of existing bents, removing and replacing portions of the concrete deck, removing and replacing portions of the existing concrete riprap, and replacing the joint seals. For additional requirements in conducting the work, see Section 821.

EXISTING BRIDGE: The Existing Bridge Nos. A&B5977 are approximately 42.8' wide and 185.2' long and consists of a 183' continuous composite W-beam and reinforced concrete slab, multi-column intermediate bents on spread footings, and steel pile end bents. Plans of the existing structure may be obtained upon request to the Construction Contract Procurement Section of the Program Management Division.

VERIFICATION: Except as noted, components of the existing bridge are to be retained and joined to proposed work. Information and dimensions shown are based on existing bridge plans. The Contractor is to adhere strictly to requirements for verification of the geometry of the existing bridge and its relationship to the proposed work described in Subsection 821.02 and make necessary adjustments to fit the proposed work to the existing structure. Payment for this work shall be considered subsidiary to the pay item "Modification of Existing Bridge Structure (Br. No. A5977)" and "Modification of Existing Bridge Structure (Br. No. B5977)".

REMOVAL AND SALVAGE: All material removed from the existing bridge under item 821 shall be disposed of according to Section 205. All material removed from the existing bridge shall become the property of the Contractor.

MAINTENANCE OF TRAFFIC: See Roadway Plans and Special Provisions for more information.

① Minimum Vertical Clearance

② Remove Existing Concrete Riprap & Replace with New Concrete Riprap as necessary for Bent construction (See Std. Dwg. No. 55002). Removal shall extend to the location of an existing construction joint. Payment for this work shall be considered subsidiary to the pay item "Modification of Existing Bridge Structure (Br. No. B5977)".

③ Shoring may be required during construction. Payment for this work, if required, shall be considered subsidiary to the pay item "Unclassified Excavation for Structures - Bridge". See SP Job No. 090305 "Shoring".

④ Remove Existing Conc. Walk & Replace with New Conc. Walk as necessary for Bent construction (See Std. Dwg. No. CDP-1). Removal shall extend to the location of an existing construction joint. Payment for this work shall be considered subsidiary to the pay item "Modification of Existing Bridge Structure (Br. No. B5977)".

Remove Existing Approach Gutters (inside gutter) at both ends of Bridge. Use Type Special I Approach Slabs at both ends of Bridge, see Dwg. Nos. XXXXX & XXXXX. Use Type Special I Approach Gutters (inside gutters) of both ends of Bridge, see Dwg. No. XXXXX.

Elevations and Stations shown are along ℄ Existing Bridge, unless noted otherwise.

For Soil Boring Information, see Dwg. No. XXXXX
For Br. No. A5977 Layout, see Dwg No. XXXXX.
For Vertical Curve Data, see Dwg. No. XXXXX.

Soil Boring Information not available at time of printing

PRELIMINARY SUBJECT TO REVISION

SHEET 2 OF X
LAYOUT OF BRIDGE 'B' OVER
HWY. 71B
NEW HOPE RD. - HWY. 62/102
WIDENING (ROGERS) (S)
BENTON COUNTY
ROUTE 49 SEC. 29
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: BWC DATE: 05-22-15 FILENAME: b090305bl_ll.dgn
CHECKED BY: CAW DATE: 06-04-15 SCALE: 1" = 20'
DESIGNED BY: KJC DATE: 05-15-15
BRIDGE NO. B5977 DRAWING NO. XXXXX

USER: bc5100
DESIGN FILE: G:\12103305_Hwy71Inchg\TRANSP\090305\dgn\bridge\b090305bl_ll.dgn SCALE: 40:1
PLOTTED: 6/11/2015 16:22

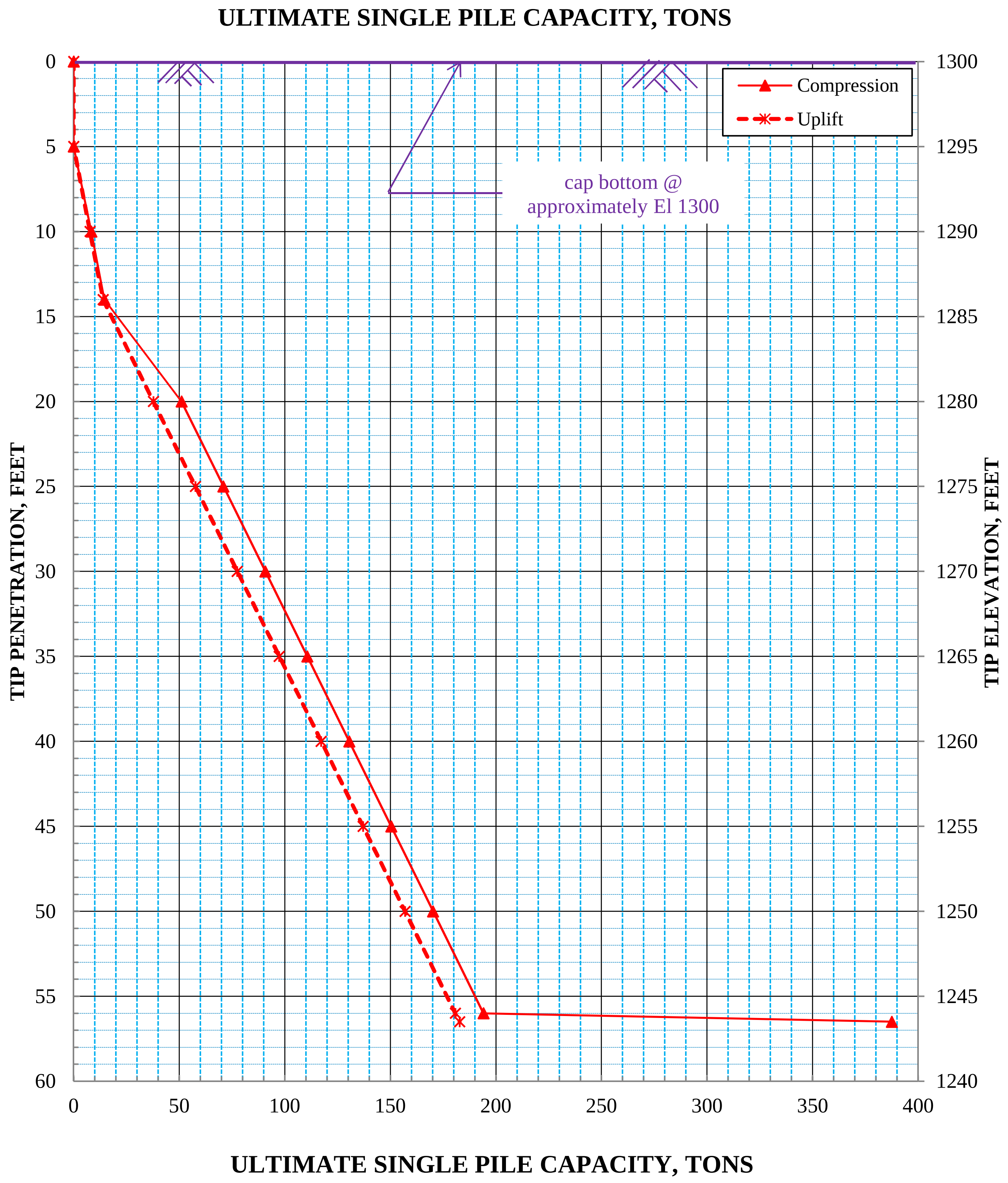
APPENDIX B

SUMMARY of CLASSIFICATION TEST RESULTS

Project: No. 090305 - Hwy. 71B Intchng. Impvts. (F)
Location: Bentonville, Benton County, Arkansas
GHBW Job Number: 12-071

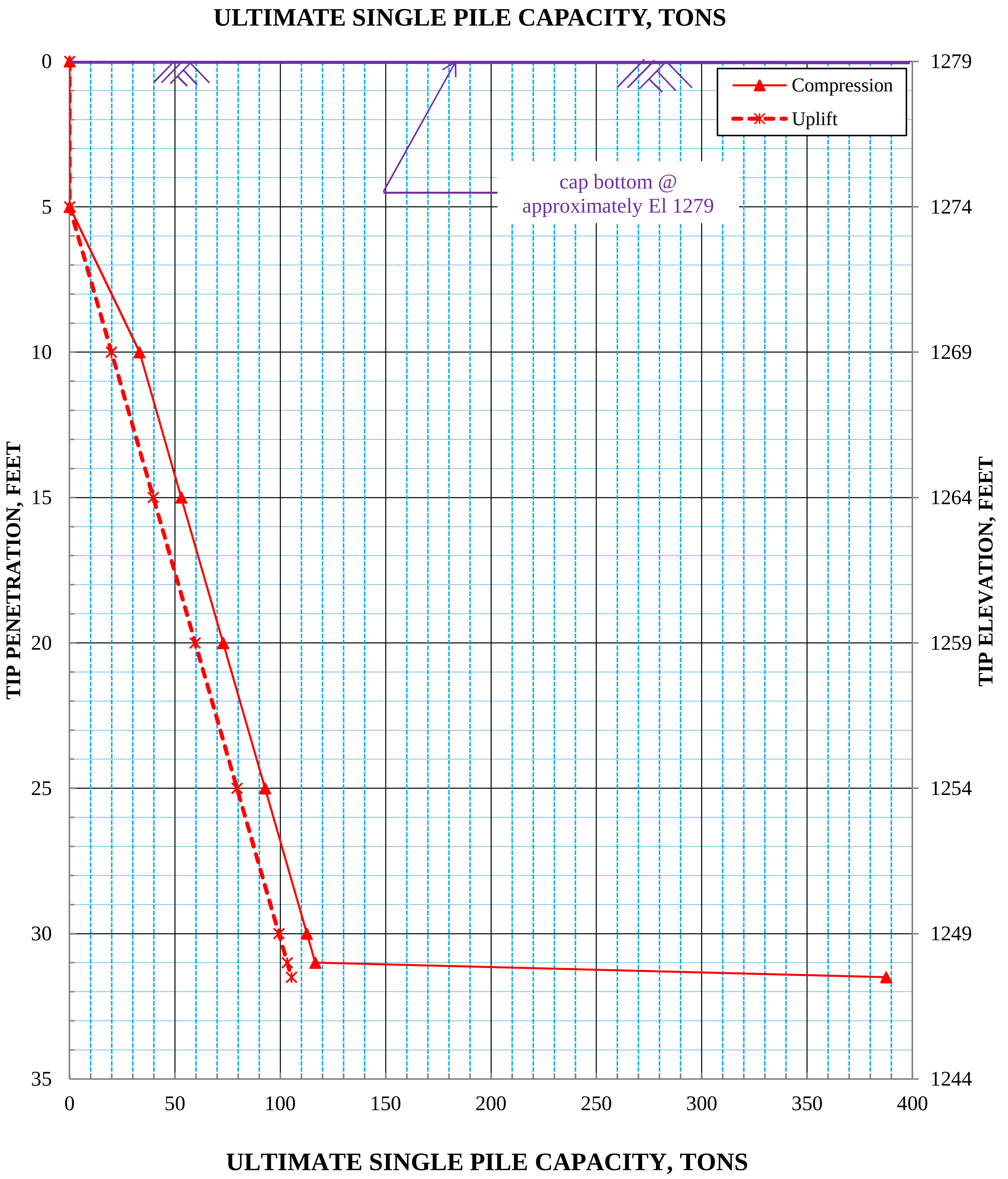
Boring No.	Sample Depth, ft	Water Content, %	ATTERBERG LIMITS			Percent Passing No. 200, %	UNIFIED CLASS.	AASHTO CLASS.
			Liquid Limit	Plastic Limit	Plasticity Index			
S1	4.5-5.5	39	65	28	37	72	CH	A-7-6
S1	14-15	27	55	22	33	63	CH	A-7-6
S2	2.5-3.5	17	25	18	7	80	CL-ML	A-4
S2	6-7	12	28	16	12	38	SC	A-4
S3	0.5-1.5	16	26	19	7	85	CL-ML	A-6
S3	2.5-3.5	19	44	17	27	81	CL	A-7-6
S3	6.5-7.5	18	49	17	32	53	CL	A-7-6
S4	9-10	21	42	18	24	72	CL	A-7-6
S4	19-20	17	31	17	14	74	CL	A-6
S4	33.5-34.5	---	82	23	59	53	CH	A-7-6

APPENDIX C

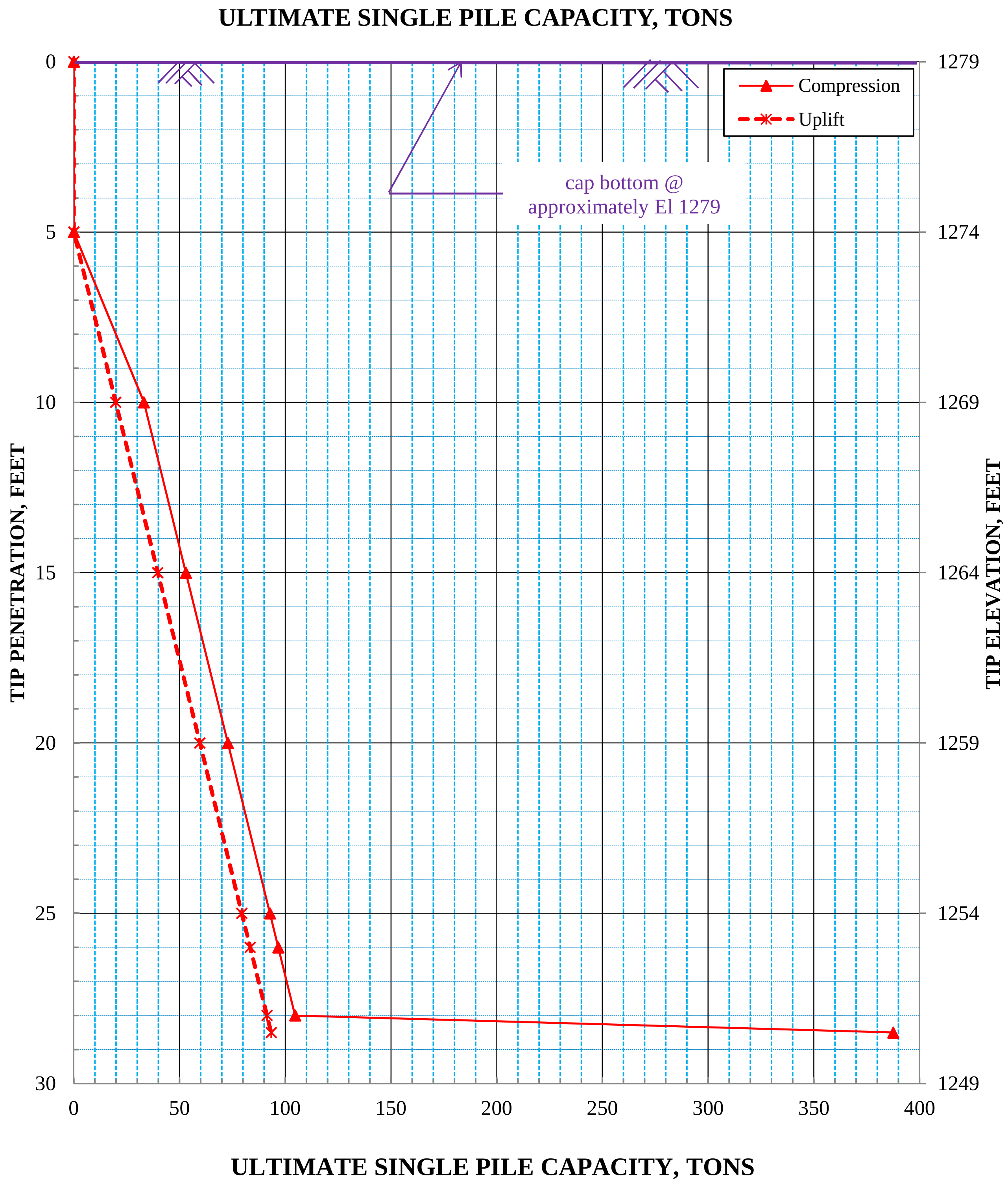


HP12x53 Steel Piles
090305 I-49 over Hwy 71B, Bent 1
Benton County, Arkansas

Note: Assumed pre-bore to El 1286

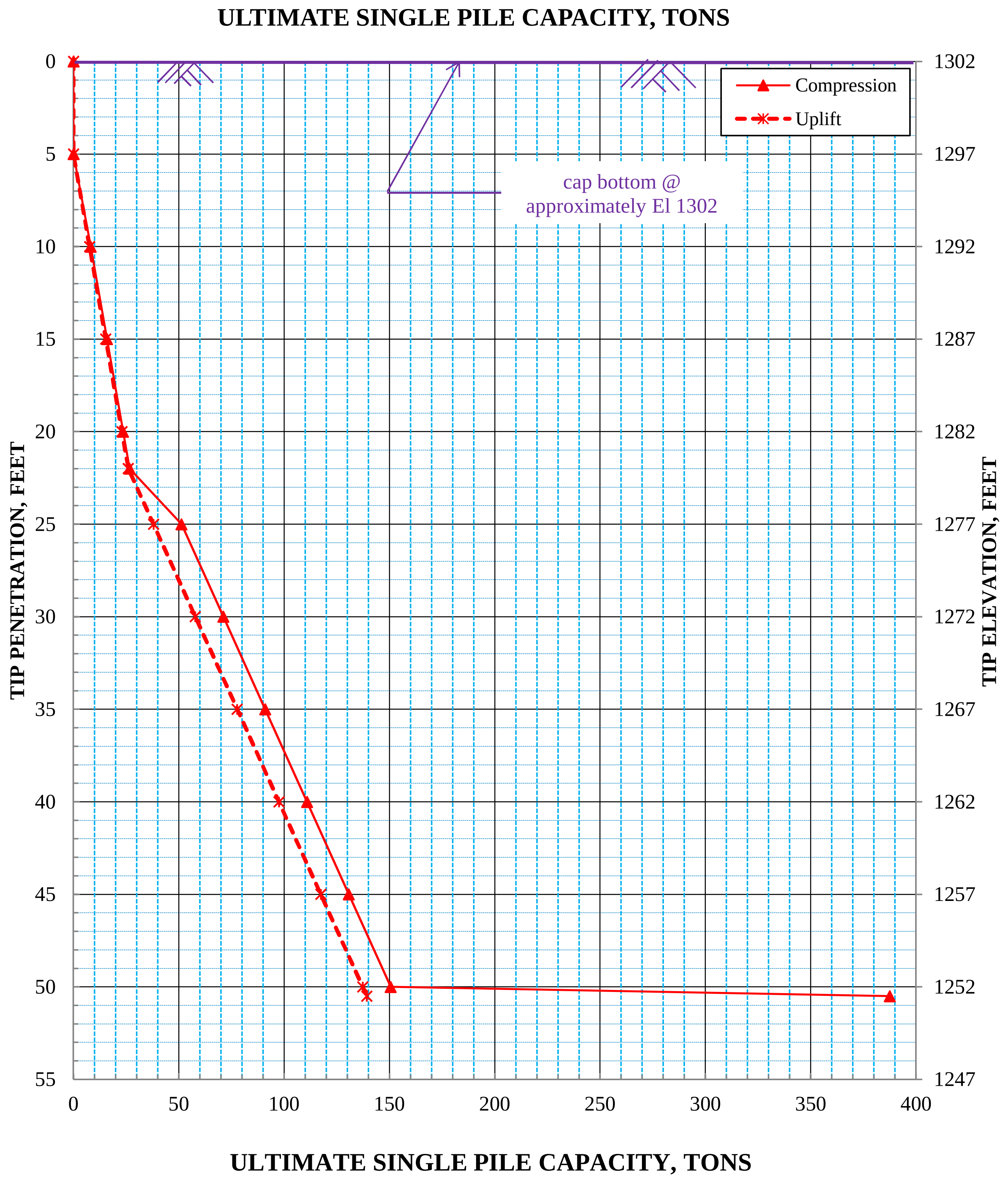


HP12x53 Steel Piles
090305 I-49 over Hwy 71B, Bent 2
Benton County, Arkansas



HP12x53 Steel Piles
090305 I-49 over Hwy 71B, Bent 3
Benton County, Arkansas

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

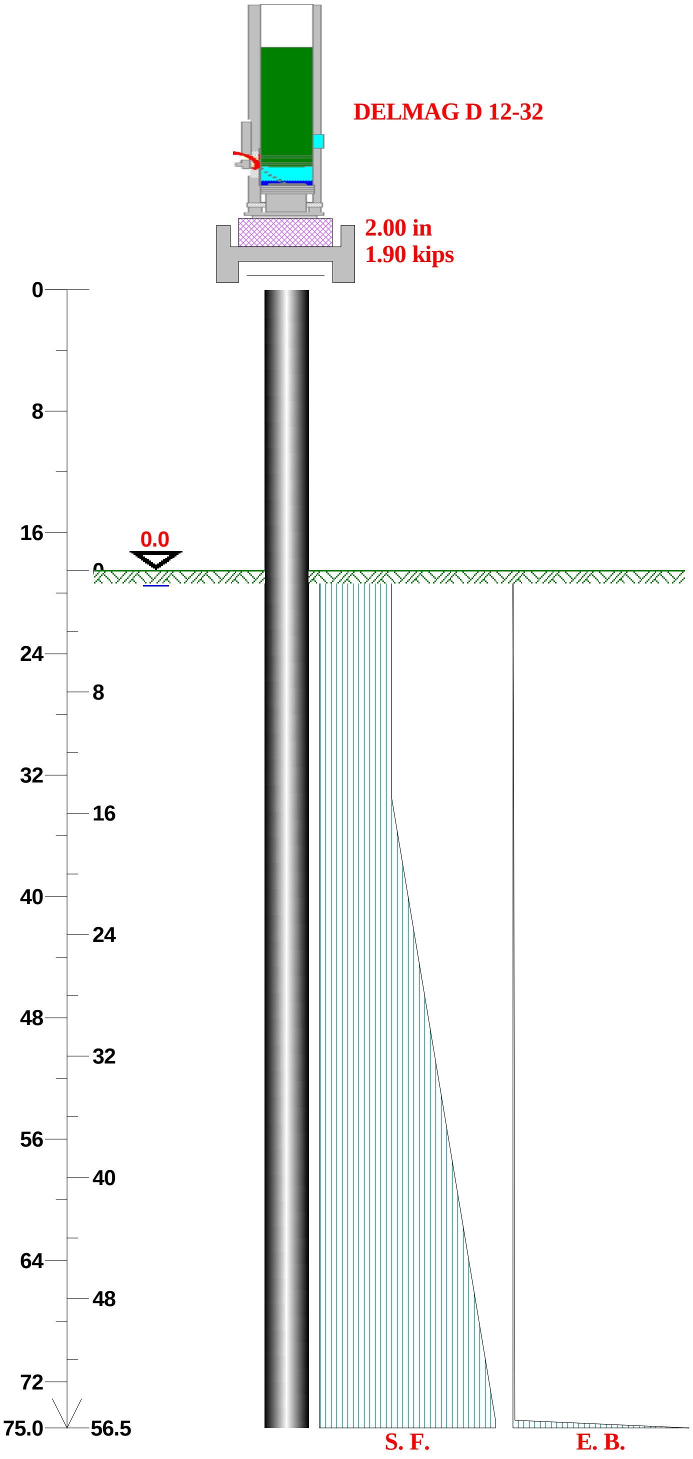


HP12x53 Steel Piles
090305 I-49 over Hwy 71B, Bent 4
Benton County, Arkansas

Note: Assumed pre-bore to El 1280

APPENDIX D

I-49 bridges (Bridges A5977 and B5977) over Hwy 71B
Bent 1 - HP12x53 Steel Pile

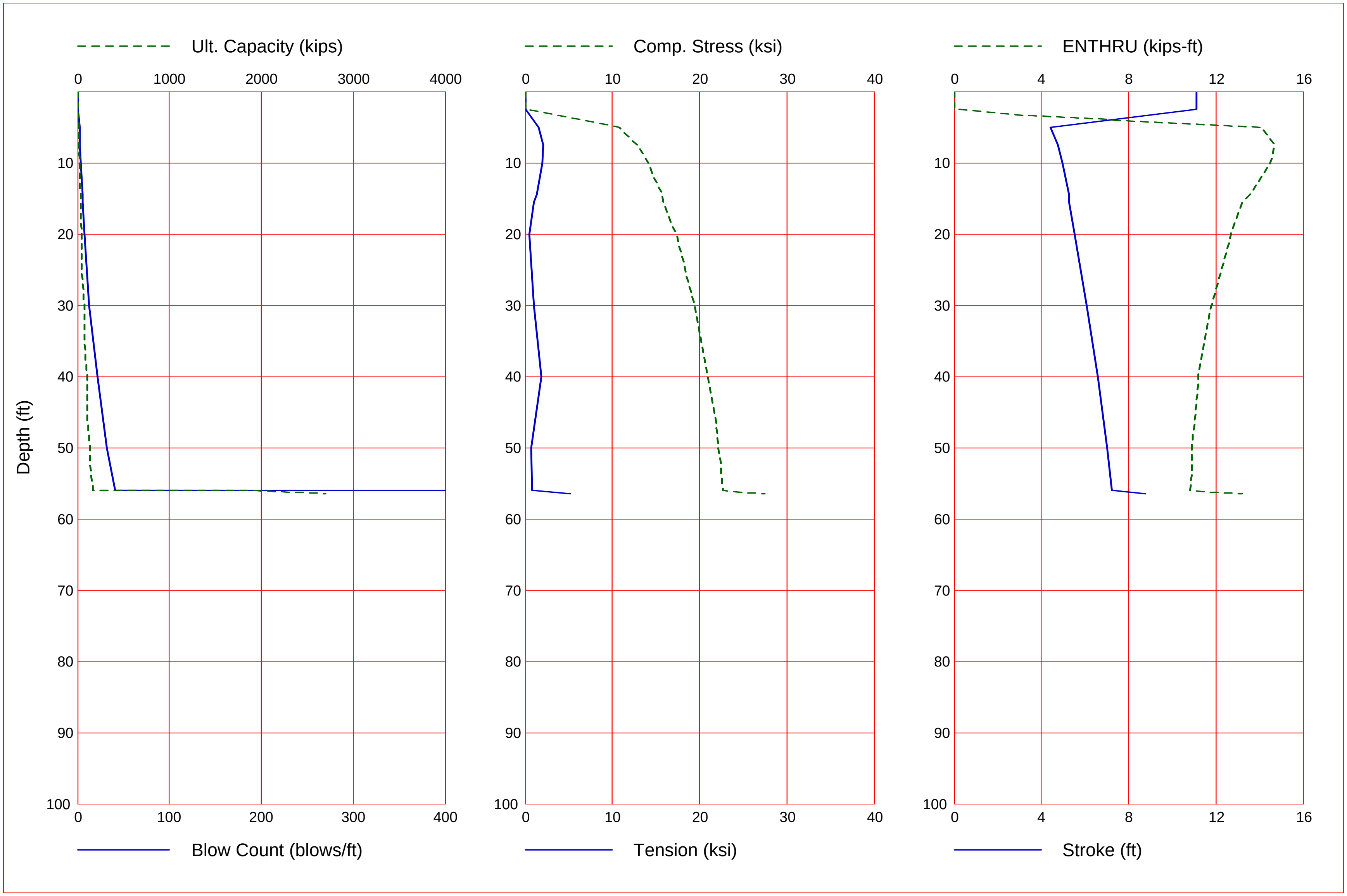


I-49 bridges (Bridges A5977 and B5977) over Hwy 71B
Bent 1 - HP12x53 Steel Pile

Grubbs, Hoskyn, Barton & Wyatt, Inc.
12-071_Driveability_Bent 1

Aug 22 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 0.500 / 1.000



DELMAG D 12-32
E = 31,330 ft-kips

I-49 bridges (Bridges A5977 and B5977) over Hwy 71B
Bent 1 - HP12x53 Steel Pile

Gain/Loss 1 at Shaft and Toe 0.500 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
0.1	0.2	0.2	0.1	0.0	0.000	0.000	11.11	0.0
2.5	6.0	4.1	1.9	0.0	0.000	0.000	11.11	0.0
5.0	12.0	8.1	3.9	2.1	10.790	-1.512	4.42	14.1
7.5	18.1	12.2	5.8	2.7	12.818	-2.055	4.74	14.7
10.0	24.1	16.3	7.8	3.3	14.108	-1.975	4.96	14.5
14.5	34.9	23.6	11.3	5.2	15.734	-1.289	5.28	13.6
15.5	37.1	25.2	11.9	5.7	15.893	-1.028	5.29	13.2
20.0	46.8	33.3	13.6	7.6	17.338	-0.486	5.54	12.7
30.0	72.6	55.3	17.3	13.1	19.469	-1.013	6.07	11.8
40.0	104.0	83.0	21.0	22.2	21.010	-1.870	6.59	11.2
50.0	141.2	116.4	24.8	31.5	22.100	-0.700	7.03	10.9
56.0	166.2	139.2	27.0	40.6	22.691	-0.844	7.25	10.8
56.5	2699.2	141.2	2558.0	9999.0	27.497	-5.180	8.75	13.2

Refusal occurred; no driving time output possible

DELMAG D 12-32
E = 31,330 ft-kips