ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

SUBSURFACE INVESTIGATION

STATE JOB NO. BB0610

FEDERAL AID PROJECT NO. NHPP-40-4(81)200

WHITE RIVER STR. & APPRS. (F)

STATE HIGHWAY 40 SECTION 42

IN PRAIRIE COUNTY

LETTING OF NOVEMBER 2, 2016

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GEOTECHNICAL INVESTIGATION
AHTD JOB BB0610: WHITE RIVER STR. & APPRS. (F)
I-40 over WHITE RIVER
PRAIRIE COUNTY, ARKANSAS

INTRODUCTION

Provided herein are the final results of the geotechnical investigation performed for the replacement bridge and associated approaches for the Interstate-40 (I-40) over the White River project planned in Prairie County, Arkansas. This project has been designated as White River Str. & Apprs. (F) project (AHTD Job No. BB0610). These services were authorized by the Indefinite Delivery Subconsulting Agreement No. WLXM2600-I11-0004 between Jacobs Engineering Group Inc. (Jacobs/the Engineer) and Grubbs, Hoskyn, Barton & Wyatt, Inc. (GHBW/the Geotechnical Engineer) and our subsequent discussions with Jacobs and the Arkansas State Highway and Transportation Department (AHTD/the Department). Interim results and recommendations have been provided throughout the course of this study.

The replacement bridge will be constructed on the upstream side (north) of the existing bridge structure (AHTD Bridge No. 3713). The replacement bridge will have five (5) continuous composite steel plate girder units and a total of 17 bents (Bents 1 through 17) and two (2) piers (Piers 1 and 2 between Bents 9 and 10) at the river navigation channel. The total length of the bridge will be about 2842 feet. Steel shells are expected to be utilized to support the foundation loads at each bent/pier.

The project also includes reconstruction of the approaches to the replacement bridge. Fill embankments are planned at the bridge abutments. We understand that the existing embankments will be incorporated into the new approach embankments. A 3-horizontal to 1-vertical (3H:1V) configuration is planned for the end slopes and the side slopes of the new embankment. The outer faces of the end slopes will be armored by dumped riprap for erosion protection. Maximum embankment height will be on the order of 35 ft at the west bridge end and 32 ft at the east bridge end.

The purposes of this study phase were to explore subsurface conditions at the replacement bridge location and to develop recommendations to guide design and construction of foundations and earthwork. These purposes have been achieved by a multi-phased study that included the following:

- Drilling soil sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing;
- Conducting a site-specific ground motion response analysis to assist in evaluating seismic site class, determining design spectral acceleration coefficients, and developing conclusions regarding seismic performance zone / seismic design category;
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata; and
- Analyzing field and laboratory data and incorporating the results of site-specific analysis to develop recommendations for seismic site class, liquefaction potential, foundation design, embankment configurations, and construction considerations.

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

SUBSURFACE EXPLORATION

Subsurface conditions in the new bridge and approach embankment alignment were evaluated by drilling a total of 26 sample borings (Borings 1 through 25 and Boring 13A) to 70- to 150-ft depth. Boring 13 was advanced at the planned location to 78-ft depth and was abandoned due to borehole caving. Consequently, Boring 13A was performed at an adjacent offset location to 150-ft depth. The subsurface investigation program also included two (2) sample borings (Borings 29 and 30) performed in the existing bridge abutments to 30- to 35-ft depth to evaluate the existing embankment fill.

The site vicinity is shown on Plate 1. The approximate locations of the borings are shown on the overall Plan of Borings, Plate 2a. The locations of the structural borings (Borings 4 through 14, Boring 13A, and Borings 18 through 25) are also shown specifically on the Plans of Structure

Borings which were developed utilizing bridge layout drawings. These are attached as Plates 2b through 2f.

The subsurface conditions encountered in the borings, and the results of the field and laboratory tests, are shown on the boring logs, Plates 3 through 58. The approximate centerline station and offset of the boring locations are noted on the logs. Surveyed ground surface elevations, as provided by Jacobs, are also indicated on the logs. Where the surveyed ground surface elevation was not available, the approximate ground surface elevation has been inferred from the available topographic information. It must be noted that the inferred ground surface elevations shown are approximate and actual surface or channel bottom elevations may vary. A key to the terms and symbols used on the logs is presented as Plate 59. A summary of the subsurface exploration program is provided in Appendix A. A subsurface profile is provided in Appendix B.

Most borings were drilled with a truck-mounted Mobile B-53 rotary-drilling rig using a combination of dry-auger and rotary wash drilling methods. Borings 29 and 30 were drilled with a truck-mounted SIMCO 2400 rotary-drilling rig using dry-auger drilling procedures. Soil samples were typically obtained using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb safety hammer dropped 30 in. in accordance with Standard Penetration Test (SPT) procedures. The number of blows required to drive the standard split-barrel sampler the final 12 in. of an 18-in. total drive, or portion thereof, is defined as the Standard Penetration Number (N). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column.

Selected undisturbed samples of cohesive soils were obtained using a 3-in.-diameter thin-walled tube (Shelby Tube) hydraulically advanced into the soil. Undrained shear strength of cohesive soils obtained using tubes were estimated in the field using a calibrated hand penetrometer. Estimated shear strength values are plotted on the log forms, in tons per sq ft, as circles enclosing an “x”.

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

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

LABORATORY TESTING

To evaluate pertinent physical and engineering characteristics of the foundation and subgrade soils encountered in the borings, laboratory tests consisting of natural water content determinations, classification tests, shear strength measurements, and consolidation tests were performed on selected representative samples. The laboratory testing program included the following.

- Soil water content (AASHTO T-265)
- Liquid limit, plastic limit, and plasticity index (AASHTO T-89 and T-90)
- Grain size analyses (AASHTO T-88)
- Specific gravity of soils (AASHTO T-100)
- Unconfined compressive strength of soils (AASHTO T-208)
- Consolidated-drained direct shear tests (AASHTO T-236)
- Unconfined undrained triaxial compressive strength of soils (AASHTO T-296)
- One-dimensional consolidation tests (AASHTO T-216)

Water Content Determinations

A total of 226 natural water content determinations were performed to complete soil water content profiles for each boring. Water content results are plotted on the log forms in accordance with the scale and symbols shown in the legend located in the upper-right corner of the logs.

Classification Tests

To verify field classifications and to evaluate soil plasticity, 56 Atterberg (liquid and plastic) limit determinations and 145 sieve analyses, including eleven (11) hydrometer tests were performed on selected representative soil samples. The Atterberg limits of the soil samples are plotted on the logs as plus signs connected with a dashed line. The percentage by weight of soil passing the No. 200 sieve is noted in the "- No. 200%" column on the far right side of the log forms. In addition, specific gravity was measured for use in each hydrometer analysis of particle size distribution. A summary of laboratory test results and classification by the Unified Soil Classification System and AASHTO classification is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

Soil Strength Measurements

Soil shear strength was estimated in the field using hand penetrometer and/or SPT results. Laboratory soil strength testing included six (6) consolidated-drained direct shear tests performed on representative undisturbed samples of cohesive soils and a remolded, cohesive soil sample of silty fine sand. The results of the direct shear tests are provided graphically in Appendix C.

In addition, twelve (12) unconsolidated-undrained triaxial compression tests and one (1) unconfined compression test were performed on undisturbed samples to measure undrained shear strength. Undrained shear strength (cohesion) determined from the results of the compression tests are plotted on the logs at the appropriate depth, in tons per sq ft, as an open triangle or an open circle for the triaxial compressive strength and unconfined compressive strength, respectively. Unit dry weight and natural water content were also determined as a part of each shear strength test. The unit dry weight, in lb per cubic ft is noted in the "UNIT DRY WT" column on the log forms.

Consolidation Tests

To evaluate consolidation characteristics of the foundation soils for use in determining the settlement potential, three (3) one-dimensional standard consolidation tests were performed on three (3) undisturbed silty clay samples. In these tests, a representative sample was placed in a consolidometer and loaded incrementally to an appropriate pressure. The sample was subsequently unloaded in increments and the rebound measured. The results of the consolidation tests are presented in Appendix C. Dry unit weight was also determined as part of these tests and is noted in the "UNIT DRY WT" column on the boring logs.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The replacement bridge is planned on the north side of the existing I-40 bridge over the White River. The existing I-40 bridge (Bridge No. 03713) over the White River is located in Prairie County, Arkansas (approximate GPS coordinates: 34.839616, -91.457113), about 60 miles east of Little Rock, Arkansas. We understand that the existing bridge structure will be demolished. However, the approach embankments will be incorporated into the new bridge alignment.

The White River at the plan replacement bridge location flows from the north to the south. The river has a wide flood plain with a channel width on the order of 500 ft at the normal pool elevation (El 164). Average water depth in the water channel was estimated on the order of 30 ft at the time of field study (March 2014). The reported historical high water is at El 192.9, about 29 ft higher than the normal pool elevation. The project alignment is in a flat, low-lying flood plain with poor surface drainage. This area is generally a fallow ground with a combination of tall grass, bushes, and small to medium trees.

Site Geology

The site is in the flood plain of the White River. Recent (Quaternary) Alluvium in the area is typically underlain by Quaternary Terrace Deposits. The alluvial deposits are comprised of variable sand, silt, gravel and clay units, and mixtures of any or all of these clastic materials. The granular units are often water bearing. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The Alluvium is comprised of recent stream-deposited alluvial sediments which include gravel, sand, silt, clay and mixtures of all components. The thickness of the Terrace and Alluvial deposits is variable. Depth of bedrock in this area is estimated to exceed 1300 feet.

Seismic Conditions

Seismic Site Class. A site-specific ground motion response analysis has been performed for this project by the University of Texas at Austin[1]. The results of the site-specific analysis are provided in Appendix D. The site-specific analysis included measurement of shear wave velocity (V_s) on both sides of the White River utilizing a combination of active-source and ambient-wavefield surface wave methods. The active-source method (i.e., Multiple-Channel Analysis of Surface Wave or MASW method) was used to generate a more detailed shear wave velocity profile of the near-surface soils. The ambient-wavefield method (i.e., microtremor array measurements or MAM method) was used to measure shear wave velocity of the deeper subsurface soils. The shear wave velocity profiling on both sides of the White River extended to at least 1300-ft depth where a measured shear wave velocity in excess of 2500 ft per sec (i.e., Seismic Site Class B rock) was determined.

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[1] Site-Specific Ground Motion Response Analysis Results for the I-40 Replacement Bridge over the White River, Final Report; Brady R. Cox et al.; The University of Texas; May 27, 2014.

Based on the results of the site-specific analysis, an average V_s value of 560 ft/sec was calculated for the upper 100 ft of the subsurface conditions on the west side of the White River. This V_s value of 560 ft per sec corresponds to a Site Seismic Class E (soft soil profile) with respect to the criteria of the 2012 AASHTO LRFD Bridge Design Specifications[2] and those of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design[3].

On the east side of the river, an average V_s value of 680 ft per sec was measured in the upper 100 ft, indicating a Site Seismic Class D (stiff soil profile). It should be noted that these average shear wave velocity values were determined for the top 100 ft below the existing ground surface (December 2013).

The combined results of soil shear wave velocity (V_s) measurements for the project alignment on both the west and east sides of the river channel indicate an overall average V_s value of 620 ft per second. These data indicate a Site Seismic Class D (stiff soil profile).

Furthermore, it must be noted the replacement bridge over the White River is relatively long (approximately 2840 ft) with multiple bents/spans. For this type of bridge, AASHTO LRFD seismic design guides suggest that the average shear wave velocity should be determined for the top 100 ft of subsurface soils below the "depth to motion (Z_{DTM})". For deep foundations comprised of driven piles with a pile cap/footing, AASHTO guidelines suggest that the depth to motion should be defined at the pile cap bottom. We understand that the pile cap bottom for the replacement bridge is typically planned at a depth of 6 to 7 ft or deeper below the existing ground surface. For the bent/piers (Bent 9 and Piers 1 and 2) in or near the river channel, pile cap bottom is planned at deeper depth up to 80 ft deep below the existing grade. Based on the shear wave velocity profile data developed in the site-specific seismic study and a conservatively assumed Z_{DTM} of 6 ft, an average V_s value of 670 ft per sec has been determined. This shear wave velocity value also corresponds to a Seismic Site Class D.

Based on the results of the borings performed for the geotechnical investigation on this project, an average Standard Penetration Test (SPT) N-value of 32 blows per ft on the west side of the river channel and 28 blows per ft was calculated for the top 100 ft on the east side of the river channel. These SPT N-values correspond to a Seismic Site Class D (stiff soil profile) in accordance with the criteria of AASHTO LRFD seismic bridge design guides.

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[2] AASHTO LRFD Bridge Design Specifications, AASHTO, 2012.

[3] AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition; AASHTO; 2011.

In light of the relatively uniform subsurface conditions revealed by the borings and the average results of the shear wave velocity measurements, it is our recommendation that a **Seismic Site Class D (stiff soil profile)** be utilized for design in accordance with the criteria of AASHTO LRFD seismic bridge design guides.

Seismic Performance Zone / Seismic Design Category. Based on the bridge location and utilizing the code-based procedure of the AASHTO LRFD seismic bridge design guides, the mapped 1.0-sec period spectral acceleration coefficient (S_1) for a Seismic Site Class B is 0.119. This mapped S_1 value is based on a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years). The site coefficient (F_v) for S_1 adjusted for Seismic Site Class D is 2.32. Accordingly, the calculated design 1.0-sec period spectral acceleration coefficient (S_{D1}) is 0.28 for the replacement bridge site in accordance with the code-based procedure.

The site-specific ground motion response analysis has determined an S_{D1} value of 0.302 on the west side of the bridge alignment and an S_{D1} value of 0.275 on the east side. Consequently, a design 1.0-sec period spectral acceleration coefficient **(S_{D1}) of 0.29**, i.e., average value of 0.302 and 0.275, is considered appropriate for the bridge alignment. This is the larger of two-thirds (2/3) of the value determined based on AASHTO LRFD code-based procedure (S_{D1} of 0.19) or the value determined utilizing the site-specific procedure (average S_{D1} of 0.29).

For an S_{D1} value of 0.29, Table 3.10.6-1 of the 2012 AASHTO LRFD Bridge Design Specifications indicates that a **Seismic Performance Zone 2** is fitting for the bridge site. Per Table 3.5-1 of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design, a **Seismic Design Category B** is considered appropriate for the replacement bridge site.

Design Peak Ground Acceleration. The code-based procedure of the AASHTO LRFD seismic bridge design guides indicates 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 0.196. For a Seismic Site Class D, the Site Coefficient for the PGA, F_{PGA} is determined to be 1.41. Consequently, a design PGA (A_s) value of 0.28 is considered appropriate in accordance with code-based procedure.

The site-specific analysis has determined an average A_s value of 0.19. Consequently, a design PGA **(A_s) value of 0.19**, which is the larger of two-thirds (2/3) of the value determined based on AASHTO LRFD code-based procedure (A_s value of 0.19) and the value determined utilizing the site-specific procedure (average A_s value of 0.19), is considered to be appropriate.

Design Earthquake Moment Magnitude. Based on the United States Geology Survey (USGS) 2002 interactive deaggregations data[4], an earthquake moment magnitude (M_w) value of 7.7 is determined to be fitting for the replacement bridge site. The M_w value of 7.7 is also recommended in the final report of the site-specific analysis. Consequently, an earthquake moment magnitude of 7.7 is considered to be suitable for seismic analysis at the project site.

Results of Liquefaction Analyses. Liquefaction analyses have been performed to evaluate the liquefaction potential of the foundation soils. These analyses were performed utilizing a Microsoft Excel© spreadsheet which is developed based on the methodology and procedures proposed by Idriss and Boulanger[5] in 2008. The spreadsheet was provided by the Department. An earthquake Moment Magnitude (M_w) value of 7.7 and a design PGA (A_S) value of 0.19 were utilized in the liquefaction analyses. Three (3) generalized subsurface models have been utilized for these analyses:

- A generalized subsurface model based on the results of the borings drilled on the west land side of the river (Borings 1 through 12);
- A generalized subsurface model based on the results of the borings performed in the river channel (Borings 13 and 13A); and
- A generalized subsurface model based on the results of the borings advanced on the east side of the river (Borings 14 through 25).

The results of liquefaction analyses are presented in Appendix E as plots of calculated factors of safety against liquefaction versus depth. The calculated factor of safety against liquefaction was determined as the ratio of cyclic shear stress required to cause liquefaction (soil strength) to cyclic shear stress induced by an earthquake (earthquake loading). The liquefaction analysis results indicate that for a design earthquake with a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years), the liquefaction potential is generally low for the I-40 over White River replacement bridge site.

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[4] http://geohazards.usgs.gov/deaggint/2002/

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

Subsurface Conditions

Based on the results of the borings, the subsurface conditions are relatively uniform and consistent. The stratigraphy may be summarized into six (6) general strata as follows:

Stratum I: Existing bridge end embankments are on-site fill (see Borings 29 and 30). The existing embankments are comprised primarily of firm to very stiff tan and gray to reddish brown silty clay and clay. The embankment fill also includes localized layers of firm to stiff reddish brown clayey silt and medium dense tan silty fine sand. The predominant clay and silty clay embankment fill has moderate to high plasticity. The subordinate clayey silt and silty fine sand are of low-plasticity or are non-plastic. The embankment fill is relatively compact with overall moderate shear strength. The fill encountered in the borings extends to about 24- to 26-ft depth (approximately El 181 to El 174). Fill content, depth, and compaction will vary with location.

Stratum II: The surface and near-surface soils in the new bridge alignment are generally very soft to stiff brown and gray silty clay with subordinate units of clay and clayey silt. There are also minor, localized and discontinuous units of silty fine sand and clayey fine sand. The predominant cohesive surface and near-surface soils typically extend to about 8- to 13-ft depth. Locally, this stratum is deeper and extends to about 35- to 42-ft depth (see Borings 18, 19, and 21). The natural silty clay and clay also have moderate to high plasticity. These moderate- to high-plasticity soils classify as A-6 to A-7-6 by the AASHTO soil classification system and exhibit poor to very poor pavement subgrade support characteristics. The low-plasticity clayey silt is generally classified as A-4 by the AASHTO system. However, this soil is highly moisture-sensitive and potentially unstable. SPT N-values in this stratum range from 2 to 44 blows per ft (average 10 blows per ft), indicating variable and overall low to moderate shear strength and moderate to low compressibility.

Stratum III: Loose to very dense tan and reddish tan to gray and brown silty fine sand is below the predominantly cohesive soils of Stratum II. Thickness of the non-plastic silty fine sand stratum encountered in the borings varies widely, ranging from absence to about 39 ft and averaging about 8 feet. The Stratum III silty fine sand exhibits variable and typically increasing low to high relative density with depth. SPT N-values range from 5 blows per ft to in excess of 50 blows per ft and average 25 blows per feet. The silty fine sand extends to variable depths of 7 to 66 feet below existing grades.

Stratum IVa: The silty fine sand is underlain below about 7- to 66-ft depth by medium dense to dense tan, gray, and brown fine sand and fine to medium sand. The fine sand and fine to medium sand exhibit moderate to high relative density and low compressibility with field SPT-N values averaging 38 blows per foot. Outside the river channel, this stratum extends to about 22- to 66-ft depth (approximately El 158 to El 111) and an average depth of 40 ft (average El 136). In the river channel, the medium dense to dense fine sand and fine to medium sand extend to approximately El 107.

Stratum IVb: The Stratum IVa fine to medium sand grades to dense to very dense gray and brown fine to coarse sand with variable amounts fine to coarse gravel (Stratum IVb). The Stratum IVb fine to medium sand exhibits high relative density and low compressibility. SPT-N values in the Stratum IVb sand units are typically greater than 50 blows per foot.

Stratum V: The basal stratum encountered within the exploration depths of the borings is dense to very dense gray gravelly fine to coarse sand and sandy fine to coarse gravel. The basal gravelly sand/sandy gravel exhibits high relative density with SPT-N values greater than 50 blows per foot.

Generalized Subsurface Profile. To aid in visualizing the subsurface conditions in the replacement bridge alignment, A Generalized Subsurface Profile is developed utilizing the results of the borings drilled for the bridge foundations (Borings 4 through 12, 13A, and 14 through 25). As noted, this profile is included in Appendix B. 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 subsurface conditions, variations from the stratigraphy illustrated by the profiles 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 encountered in the borings at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014. Groundwater levels will vary with seasonal precipitation, surface infiltration, and stream levels of the White River. Seasonal seeps and springs are likely to be present.

Significant Conditions

The significant site and subsurface conditions considered pertinent to design and construction of the replacement bridge over the White River project are summarized below:

- The existing bridge structure to be removed with existing approach embankments to be incorporated into the new embankment.
- The relatively wide, well-defined White River channel with a channel width of about 500 ft and an average water depth of about 30 ft in February to March 2014.
- The generally low-lying and flat terrain of the flood plain at the White River banks.
- The generally low liquefaction potential.

- The primarily cohesive existing embankment fill (Stratum I) comprised of clay, silty clay, and clayey silt and exhibiting overall fair compaction and moderate shear strength.
- The cohesive surface and near-surface soils in the new bridge alignment, comprised of very soft to stiff silty clay, clay, and clayey silt (Stratum II), extending to about 8- to 13-ft depth or deeper, and exhibiting overall low shear strength.
- The predominance of granular soils (Strata III, IVa, IVb, and V) present below the Stratum II cohesive soils and extending in excess of the maximum 150 ft exploration depth of the borings.
- The generally increasing relative density and decreasing compressibility of the granular soils (Strata III, IVa, IVb, and V) with depth.
- The dense to very dense fine to medium sand (Stratum IVb) underlain by very dense gravelly fine to coarse sand/sandy fine to coarse gravel (Stratum V), present below about El 158 to El 107 (average El 136) and exhibiting high to very high relative density.
- The groundwater encountered in the borings at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014.

The relationships of these factors to design and construction of the bridge and approaches have been considered in developing the conclusions and recommendations discussed in the following report sections.

ANALYSES and RECOMMENDATIONS

Foundation Design

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

In light of the predominance of granular subsurface soils revealed by the results of the borings, the anticipated heavy bridge foundation loads, and our understanding of the project, deep foundations comprised of steel shell piling are recommended to support the bridge foundation loads.

Piling

Axial Pile Capacities. We recommend that the foundation loads of the I-40 Bridge over the White River be supported on a deep foundation system comprised of steel shell piling. We understand that 18-in.-diameter and 24-in.-diameter steel shells are planned. Ultimate single pile capacity curves for the planned 18-in.-diameter and 24-in.-diameter steel shell piles are included in Appendix F for each bridge bent and pier. Plan pile cap bottom elevation is also shown on the ultimate pile capacities curves. Capacities for alternative pile types and sizes can be provided, if desired. We also understand that both the 18-in.- and 24-in.-diameter steel shell piles will be equipped with closure plates. All the steel shell piles will be filled with concrete after installation.

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 dated December 16, 2013. Normal pool was assumed in developing these axial pile capacity curves.

Based on the results of the borings, jetting or a combination of pre-boring and jetting is anticipated to be required at the interior bents and piers to facilitate penetration of the closed-ended steel shell piles into the dense to very dense sand units. Pre-boring is expected to be more effective in the surface and near-surface cohesive soils (Strata I and II). Jetting will be effective in the cohesionless soil strata (Strata III through V) but will be less effective in cohesive soils. The ultimate axial pile capacity curves of the interior bents and piers included in Appendix F have been developed based on the assumption that the steel shells will be installed using jetting. We recommend that the final 5 ft of pile penetration be achieved by driving. A closure plate is recommended for the steel shell piles as per AHTD criteria.

The ultimate axial capacities shown in Appendix F are based on single, isolated foundations. Piles spaced closer than six (6) pile diameters may develop lower individual capacity due to group effects. We understand that the 18-in.-diameter steel shell piles will be spaced 54 to 72 in. (4.5 to 6 ft) on center. This spacing is greater than two-and-half (2.5) diameters of the 18-in.-diameter piles. The 24-in.-diameter piles at the bents for the main bridge spans (Bents 9 and 10 and Piers 1 and 2) will be spaced 72 in. (6 ft) on center, also greater than 2.5 pile diameters. In light of the predominance of granular foundation soils and the pile spacing greater than 2.5 pile diameters, the reduction in axial compressive pile capacity due to pile grouping interaction is considered negligible. As per AASHTO LRFD Bridge Design Specifications, a group efficiency factor (η) of 1.0 is considered to be fitting for both pile sizes. Our analyses indicate that the factored uplift resistance of a pile group as a block exceeds the sum of the factored individual pile uplift resistance. Consequently, we recommend individual pile uplift resistance be utilized for foundation design.

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 AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.35 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading, vessel collision, check flooding, etc. resistance factors of 1.0 and 0.8 are recommended for evaluating compression and uplift capacities, respectively. Post-construction settlement of piles installed 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 predominance of granular subsurface soils and the anticipated construction sequence as specified by AHTD Standard Specifications Section 805.02.

WEAP Driveability Analyses. To evaluate suitable driving equipment, driveability analyses have been performed for representative bents utilizing wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2010[6]. In the driveability analyses, both the 18-in.- and 24-in.-diameter steel shell piles were assumed to be installed close-ended. Preliminary information regarding the steel shell pile foundations, as provided by the Engineer, is summarized in Table 1a below. The results of driveability analyses for representative bents are also indicated in this table.

Table 1a: Summary of Piling Foundations

Bent No.	Pile Diameter, in.	Plan Pile Cap El, ft	Plan Pile Tip El, ft	Plan Pile Length	Representative Bent Selected for Analysis
Bent 1 (W Abut)	18	200	137	63	Bent 1
Bent 2	18	163	122	41	Bent 3
Bent 3	18	163	122	41	

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[6] GRLWEAP 2010; Pile Dynamics, Inc.

Bent No.	**Pile Diameter, in.**	**Plan Pile Cap El, ft**	**Plan Pile Tip El, ft**	**Plan Pile Length**	**Representative Bent Selected for Analysis**
Bent 4	18	165	118	47	Bent 5
Bent 5	18	167	112	55	
Bent 6	18	168	118	50	
Bent 7	18	169	119	50	Bent 7
Bent 8	18	160	105	55	Bent 8
Bent 9	24	149	116	33	Bent 10
Bent 10	24	165	107	58	
Pier 1	24	111	56	55	Pier 2
Pier 2	24	111	47	64	
Bent 11	18	168	107	61	Bent 11
Bent 12	18	168	120	48	Bent 13
Bent 13	18	168	120	48	
Bent 15	18	176	132	44	
Bent 16	18	176	135	41	
Bent 14	18	170	120	50	Bent 14
Bent 17 (E Abut)	18	205	152	53	Bent 17

We understand that the steel shell piles will conform to ASTM A252, Grade 3 with a minimum yield strength (f_y) of 45 kips per sq inch. Wall thickness (t) of ½ in. is planned for the 18-in.-diameter steel shell piles while a wall thickness of ¾ in. is planned for the 24-in.-diameter steel shell piles. The steel shell will be filled with Class S concrete (AHTD Standard Specifications Sub-Section 802.04) with a minimum compressive strength (f'_c) of 3500 lbs per sq inch. These pile dimensions and properties have been assumed in the driveability analyses of the piling foundations.

DELMAG diesel hammers was utilized for the driveability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in.- and 24-in-diameter piles are assumed to be installed close-ended. In the analyses, the piles at the abutments (Bents 1 and 17) are assumed to be driven from the plan pile cap bottom elevations to the plan tip elevations. The piles at the interior bents and piers are assumed to be jetted to 5 ft above the plan tip elevations. The final 5 ft of penetration is assumed to be achieved by impact hammer. Graphical results of the drivability analyses are provided in Appendix G. The results of the drivability analyses are summarized in Table 1b below.

Table 1b: Results of Driveability Analyses

Bent No.	Pile Diameter, in.	Pile Penetration, ft	Hammer Energy, ft-kips	Max Blow Count, Blows/ft	Max Comp Stress, ksi	Comments
Bent 1	18	63	66.2	209	31.1	Drivable
Bent 3	18	41	51.2	231	31.6	Drivable
Bent 5	18	55	73.8	203	37.1	Drivable
Bent 7	18	50	59.7	183	35.6	Drivable
Bent 8	18	55	66.2	240	39.0	Drivable
Bent 10	24	58	96.5	177	30.9	Drivable
Pier 2	24	64	125	236	37.1	Drivable
Bent 11	18	61	75.4	222	38.6	Drivable
Bent 13	18	48	51.2	214	28.5	Drivable
Bent 14	18	50	59.7	155	32.5	Drivable
Bent 17	18	53	66.2	223	31.7	Drivable

With the recommended hammer energy, the required number of hammer blows indicated by the WEAP analyses is typically limited to 20 blows per in. (240 blows per ft) for the steel shell piles. The calculated compressive and tensile stresses in the piles determined from the WEAP analyses are also in the acceptable range, less than 90 percent of the yield strength of 45 ksi (40.5 ksi), as per AHTD Standard Specifications Section 805.07. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed by the Engineer or Department (AHTD) prior to hammer acceptance and beginning of driving.

Piling Construction. We recommend a minimum of four (4) test piles be installed at the bridge location prior to installation of production piles. One (1) test pile should be installed at a bent location at each side of the navigation channel and at least one (1) test pile should be performed at each pier location. Test piles should have a length at least 10 ft longer than that anticipated for production piles.

Piles should be installed in compliance with AHTD Standard Specifications Section 805. Jetting may be required for pile installation. If jetting is to be utilized, containment of jetted materials must be provided unless approved by environmental agencies or other authorities. Safe bearing capacity of test piles and production piles should be determined by AHTD Standard Specifications Section 805.09, Method B. Driving records should be available for review by the Engineer or Department during pile installation.

Pile points are recommended at the pile tips to facilitate pile penetration. Jetting is expected to be utilized to assist pile penetration at the interior bents /piers. However, the final 5 ft of pile penetration must be achieved by driving. The jetting pressure and flow rate through jet pipes will directly affect jetting effect. Too much flow and pressure may result in poor controllability and poor alignment of the pile being worked and/or misalignment and compromising of the adjacent piles. Too little water flow or pressure could make the jetting technique ineffective. The Contractor should have demonstrable experience in installing steel shell piles of similar sizes in subsurface conditions similar to those at this site. The Contractor should have appropriate equipment with sufficient jetting pressure / flow rate and adequate hammer energy to install piles to the plan tip elevation.

Lateral Load Analyses

To evaluate foundation soil stiffness and responses to lateral loads and to determine lateral resistance, lateral load analyses were performed on representative bents and pier using the results of the borings and reactions provided by the Engineer. These representative bents and pier are selected by the Engineer and summarized in Table 2a below. As noted, the steel shell piles will conform to ASTM A252, Grade 3 with a minimum yield strength (f_y) of 45 kips per sq inch. Wall thickness (t) of ½ in. is planned for the 18-in.-diameter steel shell piles while a wall thickness of ¾ in. is planned for the 24-in.-diameter steel shell piles. The steel shells will be filled with Class S concrete (AHTD Standard Specifications Section 805.03) with a minimum compressive strength at 28 days of 3500 psi (AHTD Standard Specifications Section 802.05).

Lateral load analyses of single piles were performed using the computer program LPILE[7]. Preliminary analyses were first performed to evaluate stiffness of the foundation soils using assumed pile head reactions. We understand that the calculated foundation soil stiffness values were utilized by the Engineer as input parameters to determine pile head reactions. The calculated pile head reactions, as provided by the Engineer, are also summarized in Table 2a. These reactions include calculated lateral loads (shear loads in transverse and longitudinal directions) and axial compression load at the plan cap bottom elevations of single piles.

A fixed-head boundary condition was used in the analyses. The effects of group interaction of the piles at each bent were modelled by applying a P-multiplier to p (P_m) to define

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[7] LPILE Plus, Version 5.0; Lymon C. Reese and Shin Tower Wang; Ensoft, 2004.

the foundation p-y relationship. The P_m values are determined based on the pile space-diameter ratio and in accordance with the AASHTO LRFD Bridge Design Specifications. The top 5 ft of pile penetration is neglected from lateral resistance evaluation. At Pier 1, the results of scour analysis performed by the Engineer indicate the potential localized scour at the design flood (100-year flood) is at El 76, about 35 ft below the plan pile footing bottom elevation (El 111). To model the lateral resistance of the foundation soils above the localized, temporary scour depth, a reduced shear strength parameter, i.e., an internal friction angle (ϕ') of 30° which is typical of loose sand, is utilized in the analysis.

Table 2a: Representative Bents Lateral Load Analyses

Bent No.	Pile Diameter, in.	Plan Pile Length, ft	Lateral Load Direction	Pile Spacing, ft	Lateral Load, kips	Axial Load, kips
1	18	63	Long	4.5	20.5	159
2	18	41	Trans	5	2.2	164
			Long	5	2.5	164
3	18	41	Trans	5	2.4	160
			Long	5	2.7	160
8	18	55	Trans	6	37.8	351
			Long	4.5	19.0	351
9	24	33	Trans	6	35.9	348
			Long	6	18.2	348
Pier 1	24	55	Trans	6	22.6	380
			Long	6	11.3	380
10	24	58	Trans	8	36.2	343
			Long	6	18.4	343
17	18	53	Long	4.5	20.5	159

Graphical plots of pile head deflection versus depth, unfactored bending moment versus depth, and shear force versus depth are provided in Appendix H for these representative bridge bents and pier. The results of the lateral load analyses are summarized below in Table 2b.

Table 2b: Lateral Load Analysis Results

Bent No.	**Lateral Load Direction**	**Lateral Load, kips**	**Calculated Maximum Pile-Head Deflection, in.**	**Calculated Maximum Unfactored Bending Moment, in.-kips**
1	Long	20.5	0.86	2650
2	Trans	2.2	0.04	230
	Long	2.5	0.05	250
	Resultant	3.4	0.06	340
3	Trans	2.4	0.04	230
	Long	2.7	0.04	260
	Resultant	3.6	0.06	350
8	Trans	37.8	0.48	4600
	Long	19.0	1.13	2200
	Resultant	42.3	1.31	5200
9	Trans	35.9	0.37	4400
	Long	18.2	0.15	2100
	Resultant	40.3	0.44	5100
Pier 1	Trans	22.6	0.38	3200
	Long	11.3	0.18	1600
	Resultant	25.2	0.43	3600
10	Trans	36.2	0.41	4700
	Long	18.4	0.16	2200
	Resultant	40.6	0.47	5300
18	Long	20.5	1.07	2850

Wingwall and Abutment Wall Lateral Earth Pressures

It is expected that wingwalls and abutment walls, if planned, will be backfilled with unclassified borrow or select material. Recommendations regarding lateral earth pressures for wingwalls and abutments are summarized below.

- Total unit weight (γ) for unclassified backfill: 130 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: 96 lbs per sq ft per ft depth.
- Angle of internal friction (ϕ) for SM-1 backfill: 32°
- Total unit weight (γ) for SM-1: 125 lbs per cu ft
- Equivalent fluid pressure for SM-1 backfill:
    - 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.
- Nominal/ultimate sliding resistance:
    - Interaction friction angle (δ) for concrete on stable bearing stratum: 19°.
    - Interaction friction factor (tan δ) for concrete on stable bearing stratum: 0.34.
    - A resistance factor (φ) for sliding resistance: 0.8

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

Approach Embankments

General. Preliminary stability analyses were initially performed to evaluate various alternative embankment configurations (i.e., various embankment height, slope, and geosynthetics reinforcement layout, etc.) at the west and east bridge ends. We understand that the existing embankments will be utilized to the extent possible and the south portion of the plan embankments will incorporate the existing embankment fill to some degree. Consequently, two (2) embankment sections had been evaluated for each bridge end in the preliminary analyses. These included: (1) an embankment section comprised completely of new embankment fill (new embankment section) and, (2) an embankment section incorporating the existing embankment fill (composite embankment section). Based on the results of these preliminary stability analyses, it had been concluded that the composite embankment section is generally more critical with lower factors of safety against slope instability for all the loading conditions analyzed and for both bridge end slopes. Consequently, the final analyses evaluate/verify only composite embankment sections.

As noted, the bridge layout dated December 16, 2013, as provided by the Engineer, has been utilized to develop Plans of Borings that are included in this final report as Plates 2b through 2f. In addition, some representative cross sections of the approach embankments were also provided by the Engineer. These cross sections are included in Appendix I. The bridge layout and embankment cross sections indicate that geogrid-reinforced embankments with maximum embankment height of up to about 35 ft and a 3-horizontal to 1-vertical (3H:1V) slope configuration have been adopted by the Engineer. It is understood the bridge layout and embankment cross sections were developed based on the results of preliminary stability analyses.

The purposes of the final stability analyses are to verify stability of the design embankment configuration with respect to shear strength of embankment fill and foundation soils. To model the lower strength boundary of unclassified embankment fill placed for new embankments, a cohesion value of 750 lbs per sq ft and an internal friction angle (φ) of 0° were assumed. Cohesive soils were assumed for the embankment fill. Cohesive soils are recommended for embankment fill due to improved response to seismic loading as compared to cohesionless soils. The *in-situ* soil properties have been modeled for use in stability analyses based on the results of the laboratory testing program and our experience with similar soils. For the purposes of stability analyses, a uniform surcharge of 250 lbs per sq ft has been included to accommodate vehicle traffic loads.

Stability analyses have been performed using the computer program SLOPE/W 2007[8] and a Morgenstern-Price analysis. The loading conditions evaluated for the maximum 35-ft embankments include the following.

- End of construction with total stresses.
- Long term with effective stresses and groundwater at the embankment toe.
- Long term with effective stresses and the embankment saturated to a groundwater level approximately equal to the design 500-yr flood at El 192.9.
- Seismic condition with effective stresses and groundwater at the embankment toe.

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

  - As per FHWA guidelines[9] utilized by AHTD and assuming an embankment reinforced with geogrid, the analyses for the seismic condition have utilized a horizontal acceleration coefficient (k_h) value of one-half of the peak ground acceleration value, i.e., 0.10.
  - For analyses of unreinforced embankments, a horizontal acceleration coefficient (k_h) value of 0.19, which is the peak ground acceleration value, was utilized.
- Rapid drawdown with effective stresses and a saturated embankment at the design 500-yr flood of El 192.9 and groundwater at the embankment toe elevation. Only the upstream slopes (i.e., end slopes and north side of the side slopes) were evaluated in stability analysis of rapid drawdown condition.

Geosynthetic (biaxial geogrid) internal reinforcement is incorporated into embankment model and stability analyses. For the purposes of the analyses, the internal reinforcement is assumed to be structural geogrids with a minimum allowable tensile strength of 4000 lbs/ft. The geogrid reinforcement has been assumed to be spaced at 3-ft vertical intervals starting at the ground level and continuing to within three (3) ft of the final grade. Laterally, the reinforcement has been assumed to extend from one side slope face to the other side slope face, consistent with the construction requirement. The longitudinal extent of the reinforcement was determined through an iterative process by performing stability analyses on unreinforced slopes of decreasing height. The results of detailed stability analyses are discussed in the following report sections.

Stability analyses have been performed to verify the suitability of the plan approach embankments. Results of the stability analyses performed on six (6) embankment sections are provided in this final report. These sections, as well as location and plan height of the slopes, are summarized in Table 3 below. Section view drawings, with material parameters shown on, have been developed for these sections to facilitate stability analysis modeling. These sections are included in respective appendices containing the results of stability analyses (see Table 3).

Table 3: Summary of Embankment Sections Utilized for Stability Analyses

Location	Slope	Reinforcement Condition	Height, ft	Appendix
West Abutment (Sta 1883+41.92)	3H:1V End Slope	Geogrid-Reinforced	35	J
	3H:1V Side Slopes	Geogrid-Reinforced	15 to 35	K
East Abutment (Sta 1911+84.08)	3H:1V End Slope	Geogrid-Reinforced	32	L

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[9] 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.

Location	Slope	Reinforcement Condition	Height, ft	Appendix
	3H:1V Side Slopes	Geogrid-Reinforced	10 to 32	M
Sta 1875+00	3H:1V Side Slopes	Unreinforced	17 to 25	N
Sta 1913+00	3H:1V Side Slopes	Unreinforced	10 to 29	O

Results of Stability Analyses. The results of stability analyses are summarized in Tables 4 through 9. Detailed graphical results for each case analyzed are provided in appropriate appendices summarized in Table 3 above.

Table 4: Stability Analysis Results – End Slope @ West Abutment (H = 35 ft)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ embankment toe	2.1
Long Term	Groundwater @ embankment toe	1.8
Long Term	Historical high water @ El 192.9	1.9
Seismic ($kh = 0.5A_S = 0.10$)	Groundwater @ embankment toe	1.3
Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

Table 5: Stability Analysis Results – Side Slopes @ West Abutment (H = 15 to 35 ft)

Design Loading Condition	Embankment Side	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	North	Groundwater @ embankment toe	2.2
	South		4.1
Long Term	North	Groundwater @ embankment toe	1.9
	South		2.0
	North	Historical high water @ El 192.9	1.9
	South		2.0
Seismic ($k_h = 0.5A_S = 0.10$)	North	Groundwater @ embankment toe	1.4
	South		1.5
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.5

Table 6: Stability Analysis Results – End Slope @ East Abutment (H = 32 ft)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ embankment toe	2.8
Long Term	Groundwater @ embankment toe	2.5
Long Term	Historical high water @ El 192.9	2.4
Seismic ($k_h = 0.5A_S = 0.10$)	Groundwater @ embankment toe	1.8
Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.3

Table 7: Stability Analysis Results – Side Slopes @ East Abutment (H = 10 to 32 ft)

Design Loading Condition	Embankment Side	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	North	Groundwater @ embankment toe	2.5
	South		4.5
Long Term	North	Groundwater @ embankment toe	2.3
	South		3.4
	North	Historical high water @ El 192.9	2.3
	South		3.4
Seismic ($k_h = 0.5A_S = 0.10$)	North	Groundwater @ embankment toe	1.7
	South		1.9
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	2.1

Table 8: Stability Analysis Results – Side Slopes @ Sta 1875+00 (H = 17 to 25 ft)

Design Loading Condition	Embankment Side	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	North	Groundwater @ embankment toe	2.9
	South		3.4
Long Term	North	Groundwater @ embankment toe	2.2
	South		2.4
	North	Historical high water @ El 192.9	3.0
	South		3.2
Seismic ($k_h = 1.0A_S = 0.19$)	North	Groundwater @ embankment toe	1.1
	South		1.2
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.6

Table 9: Stability Analysis Results – Side Slopes @ Sta 1913+00 (H = 10 to 29 ft)

Design Loading Condition	Embankment Side	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	North	Groundwater @ embankment toe	2.9
	South		3.4
Long Term	North	Groundwater @ embankment toe	1.8
	South		4.3
	North	Historical high water @ El 192.9	1.8
	South		3.9
Seismic ($k_h = 1.0A_S = 0.19$)	North	Groundwater @ embankment toe	1.1
	South		1.6
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.6

Conclusions and Recommendations. The results of slope stability analyses summarized in Tables 4 through 7 indicate acceptable factors of safety against sliding for the geogrid-reinforced end and side slopes with maximum height of 32 to 35 ft and a simple 3H:1V slope configuration for all the loading conditions analyzed and for both end and side slopes evaluated at the bridge ends. Consequently, the design slope configuration is considered to be adequate and suitable in conjunction with the assumed geogrid reinforcement properties and arrangement.

The embankment alignments which may be unreinforced was determined through an iterative process by performing stability analyses on unreinforced slopes of decreasing height. A trial-and-error procedure was utilized in the analyses to determine the maximum unreinforced embankment height until the calculated minimum factor of safety is just greater than 1.0. Based on the results of these analyses summarized in Tables 8 and 9, we recommend the embankment alignment between Sta 1875+00 and Sta 1883+42 (Begin Bridge) and the alignment between Sta 1911+84 (End Bridge) and Sta 1913+00 be reinforced with geogrid. Beyond these limits, geogrid reinforcement is not required. Recommended limits of geogrid-reinforced embankment alignments are shown on the site layout included in Appendix P.

Embankment Fill Considerations. We recommend that embankment fill consist of cohesive soils. Suitable borrow includes sandy clay, clayey sand, silty clay, and clay. We recommend the following properties for imported borrow for use as embankment fill.

- Maximum liquid limit of 50
- Minimum plasticity index (PI) of 10
- A minimum of 60 percent passing the No. 4 sieve (4.76mm)

- A minimum of 35 percent passing the No. 200 sieve (0.074mm)
- Non-dispersive

We recommend that the top 24 in. of embankment fill in slopes have a maximum liquid limit of 40 and a PI between 5 and 18. All fill and backfill must be free of organic materials. Maximum particle size in embankment fill should be limited to about 6 inches.

Where fill is placed against existing embankment slopes, short vertical cuts should be benched into the existing slope faces to facilitate application of geogrid reinforcement and bonding of horizontal fill lifts. Maximum bench height should be limited to 3 feet. A typical bench width of 9 ft is recommended. Detailed benching pattern during construction must be based on specific site and construction conditions. A nominal horizontal bench width of 8 to 12 ft and a maximum bench height of 4 ft are recommended.

Stability of East River Bank near Pier 2

The east river bank where Pier No. 2 is planned is relatively high (±51 feet) and steep. The upper half portion of the slope is steeper and has a slope configuration of approximately 1.1H:1V. Consequently, engineering analyses were performed to evaluate stability of the river bank at the east side of the White River. Like those analyses performed for the approach embankments, stability analyses were also performed on the existing east river bank using the computer program SLOPE/W 2007 and a Morgenstern-Price method. Likewise, four (4) loading conditions were evaluated. These loading conditions are: 1) long term condition with groundwater at El 164, 2) long term condition with historical high water at El 192.9, 3) seismic condition with groundwater at El 164, and 4) rapid drawdown from El 192.9 to El 159 (design low water).

The results of stability analyses performed on the east river bank are summarized in Table 10 below. Detailed graphical results are provided in Appendix Q.

Table 10: Stability Analysis Results – Existing East River Bank (H = 51 ft)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
Long Term	Groundwater @ El 164	1.3
Long Term	Historical high water @ El 192.9	1.5
Seismic ($k_h = 1.0A_S = 0.19$)	Groundwater @ El 164	0.7
Rapid Drawdown	Drawdown from 192.9 to El 159	0.8

The results of the slope stability analyses summarized above indicate that the east river bank is generally stable for long term conditions, but with marginally acceptable factors of safety against sliding. For seismic and rapid drawdown conditions, the calculated factors of safety are less than 1.0, indicating a substantial risk of sliding with these loading conditions. Consequently, design will be required to maintain stability of the river bank and to protect the planned Pier 2 from lateral loads due to sliding.

Detailed design for the remediation of the east river bank is not presently available. However, we understand that the preliminary concept is to design Pier 2 as a retaining wall-like structure or to include a pressure relief wall. The pier or retaining wall will extend from the top of pile footing (approximately El 116) up to some elevation (e.g., El 186) to retain ground and resist the lateral earth pressure. We also understand that individual columns will be utilized above the top of the retaining wall elevation to support the bridge superstructure.

The thick retaining wall structure is expected to be fixed at the top by the superstructure with the columns and will be designed to allow minimal deflection. Consequently, at-rest earth pressure will be mobilized on the retained side. On the retaining side, passive earth pressure will be developed to resist wall deflection. Recommendations regarding lateral earth pressures for the Pier 2 retaining wall are summarized below.

- Retained side (behind retaining wall) – at-rest earth pressure
    - Design (equivalent) surface elevation of retained soils: El 175
    - Effective unit weight (γ'): 56 lbs per cu ft
    - Angle of internal friction (ϕ): 24°
    - Coefficient of static at-rest earth pressure (K_0): 0.60
    - Coefficient of total (static and seismic) at-rest earth pressure (K_{0e}): 1.02
- Retaining side (in front of retaining wall) – passive earth pressure
    - Design (equivalent) surface elevation of retaining soils: El 145
    - Effective unit weight (γ'): 66 lbs per cu ft
    - Angle of internal friction (ϕ): 36°
    - Coefficient of static passive earth pressure (K_p): 3.85
    - Coefficient of total (static and seismic) at-rest earth pressure (K_{pe}): 3.32
- Nominal/ultimate sliding resistance:
    - Interaction friction angle (δ) for concrete (pile footing) on stable bearing stratum: 19°
    - Interaction friction factor ($\tan \delta$) for concrete (pile footing) on stable bearing stratum: 0.34
    - A resistance factor (φ) for sliding resistance: 0.8

Stability analyses were performed to verify global stability of the east river bank with the retaining structure. Two (2) loading conditions, i.e., seismic and rapid drawdown, were evaluated. The soil on the retaining side was conservatively assumed to slide or be eroded by scour to the dense silty fine sand stratum near El 145. In addition, the retaining structure was assumed to be rigid enough to drive the failure surface below El 110. The retaining structure design must include verification of internal stability of the retaining wall.

The results of stability analyses performed on the east river bank with a retaining structure are summarized in Table 11. Detailed graphical results are also provided in Appendix Q. The results shown in Table 11 indicate acceptable factors of safety against sliding with respect to seismic and rapid drawdown conditions. The factors of safety for long term conditions are expected to be greater and will be acceptable.

Table 11: Stability Analysis Results –East River Bank Remedied by Retaining Structure

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
Seismic ($k_h = 1.0A_S = 0.19$)	Groundwater @ El 164	1.9
Rapid Drawdown	Drawdown from 192.9 to El 159	5.9

Pavement Design Subgrade Support Parameters

It is understood that the new approach roads will be constructed mostly on embankments. In light of the roadway on the embankment, surface drainage in the new roadway alignment is expected to be good.

Based on the results of the borings performed in the existing embankment (i.e., Borings 29 and 30), the subgrade soils are expected to be predominantly silty clay and clay (AASHTO A-6 and A-7-6). Locally available borrow for use as unclassified embankment fill is expected to be comprised of similar soils.

We recommend that A-7-5 and A-7-6 silty clay and clay be excluded within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18. The as-built pavement subgrade should be approved by the Engineer based on site observations. Areas of unsuitable subgrade should be improved by undercut or treatment with hydrated lime or Portland cement.

Based on correlation with the AASHTO classification of the anticipated subgrade soils, subgrade support characteristics are expected to be fair. For the purposes of developing an effective resilient modulus (M_R) for use in pavement design, correlation has been based on the fair subgrade support expected for a subgrade comprised of a minimum 18 in. of low-plasticity soils. We recommend an effective resilient modulus (M_R) value of 3200 lb per sq in. for use in approach road pavement design. An effective subgrade modulus (k) value of 165 lbs per cu in. is also recommended.

Scour Analysis Parameters

Scour protection will be warranted to limit the potential for loss of ground and reduction in foundation capacities. We understand that specific scour analyses will be performed by the Engineer. For use in scour analyses, the particle size through which 50 percent of soil by weight passing, D_{50}, are indicated on the generalized subsurface profile and respective grain size distribution curves included in Appendix R.

Site Grading and Earthwork

We understand that the replacement bridges will incorporate the existing embankments to the extent possible. Some site grading/reshaping of the existing embankments will be required. After bridge demolition, site grading and subgrade preparation should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. Where fill depths in excess of 3 ft are planned, stumps may be left after close cutting trees to grade, as per AHTD criteria. Otherwise, the tree stumps must be completely excavated and properly backfilled. The depth of stripping will be variable, with deeper stripping depths in the low-lying, poorly drained, and/or wooded areas, and less stripping required in the higher-terrain areas. In general, the stripping depth is estimated to be about 6 to 12 in. in cleared areas, but may be 18 to 24 in. or more in the localized wooded areas. The zone of organic surface soils should be completely stripped in the embankment footprints.

Where the existing shoulder pavements are within 3 ft of the plan subgrade elevation, the existing pavement surface should be scarified to a minimum depth of 6 inches. The scarified soil should be recompacted to a stable condition. Where pavements are to be demolished, consideration may be given to utilizing the processed asphalt concrete, Portland cement concrete, and/or aggregate base for embankment fill in areas/zones where piling is not planned. In this case, the demolished materials should be thoroughly blended and processed to a reasonably well-graded mixture with a maximum particle size of 2 inches.

Following demolition, stripping and grubbing, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and unsuitable soils should be determined. Proof-rolling is recommended to evaluate subgrade stability. Proof-rolling should be performed with a loaded tandem-wheel dump truck or similar equipment. Unstable soils exhibiting a tendency to rut and/or pump should be undercut and replaced with suitable fill. Care should be taken that undercuts, stump holes, and other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, undercuts on the order of 2 to 4 ft, more or less, will be required at the bridge abutments. It should be noted that the estimated undercut depths are below existing grade and are based on the results of the borings and test pits. Required as-built depth of undercut will vary with seasonal site conditions and final grading plans. As-built undercut requirements must be field verified by the Engineer or Department.

Undercuts for embankments may be backfilled with suitable embankment fill. Should excavations or deeper undercuts encounter shallow water or seepage, or if areas of seepage are encountered during the work, backfill should consist of clean sand (AHTD Standard Specifications Section 302, SM-1 with less than 10 percent passing the No. 200 sieve), stone backfill (AHTD Standard Specifications, Section 207), or clean aggregate (AHTD Standard Specifications Subsections 403.01 and 403.02 Class 3 mineral aggregate) extending up to an elevation above the inflow of seepage. 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.

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

Subgrade preparation and mass undercuts should extend at least 10 ft beyond the embankment toes to the extent possible. Subgrade preparation in roadway areas should extend at least 3 ft outside pavement shoulder edges to the extent possible. The existing drainage features should be completely mucked out and all loose and/or organic soils removed prior to fill placement.

Fill and backfill may consist of unclassified borrow free of organics and other deleterious materials as per AHTD Standard Specifications Subsection 210.06. Granular soils must be protected from erosion with a minimum 18-in.-thick armor of clayey soil with a PI in the range of 5 to 18.

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

CONSTRUCTION CONSIDERATIONS

Groundwater and Seepage Control

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

Groundwater was encountered at 4- to 16-ft depth or at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014. In addition, shallow perched groundwater may be encountered in the near-surface soils. Seepage into excavations and cuts can typically be controlled by ditching or sump-and-pump methods. If seepage into excavations becomes a problem, backfill should consist of clean sand (AHTD Standard Specifications

Section 302, SM-1 with less than 10 percent passing the No. 200 sieve), stone backfill (AHTD Standard Specifications Section 207), or clean aggregate (AHTD Standard Specifications Subsections 403.01 and 403.02 Class 3 mineral aggregate) to an elevation above the inflow of seepage. In areas of seepage infiltration, the granular fill should be encapsulated with a filter fabric complying with AHTD Standard Specifications Subsection 625.02, Type 2 and vented to positive discharge.

Where perched water is 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. We also recommend that blanket drains be constructed in any drainage features encountered during the grading work.

Piling

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.

Pile installation should be monitored by qualified personnel to maintain specific and complete driving records and observe pile installation procedures. Driving records should be available for review by the Engineer during pile installation. Compatible driving equipment should be utilized based on the results of drivability analyses performed by the Department. Blow counts for steel piles should be limited to about 20 blows per inch. As-built pile capacities should be evaluated by use of wave equation analysis of piles (WEAP) in accordance of AHTD Standard Specifications, Section 805.09, Method B.

CLOSURE

The Engineer 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 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 final report:

Plate 1	Site Vicinity Map
Plates 2a through 2f	Plans of Borings
Plates 3 through 58	Boring Logs
Plate 59	Key to Terms and Symbols
Appendix A	Summary of Subsurface Exploration Program
Appendix B	Generalized Subsurface Profiles
Appendix C	Results of Laboratory Test Results
Appendix D	Results of Site-Specific Ground Motion Response Analyses
Appendix E	Results of Liquefaction Analyses
Appendix F	Ultimate Single Pile Capacity
Appendix G	Results of WEAP Driveability Analyses
Appendix H	Results of Lateral Load Analyses
Appendix I	Relevant Cross Sections
Appendix J	Results of Stability Analyses – Reinforced End Slope at West Abutment
Appendix K	Results of Stability Analyses – Reinforced Side Slopes at West Abutment
Appendix L	Results of Stability Analyses – Reinforced End Slope at East Abutment
Appendix M	Results of Stability Analyses – Reinforced Side Slopes at East Abutment
Appendix N	Results of Stability Analyses – Conventional Side Slopes at Sta 1875+00
Appendix O	Results of Stability Analyses – Conventional Side Slopes at Sta 1913+00
Appendix P	Recommended Approximate Limits of Geogrid-Reinforced Embankments
Appendix Q	Stability of East River Bank near Pier 2
Appendix R	Scour Analysis Parameters

* * * * *

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

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

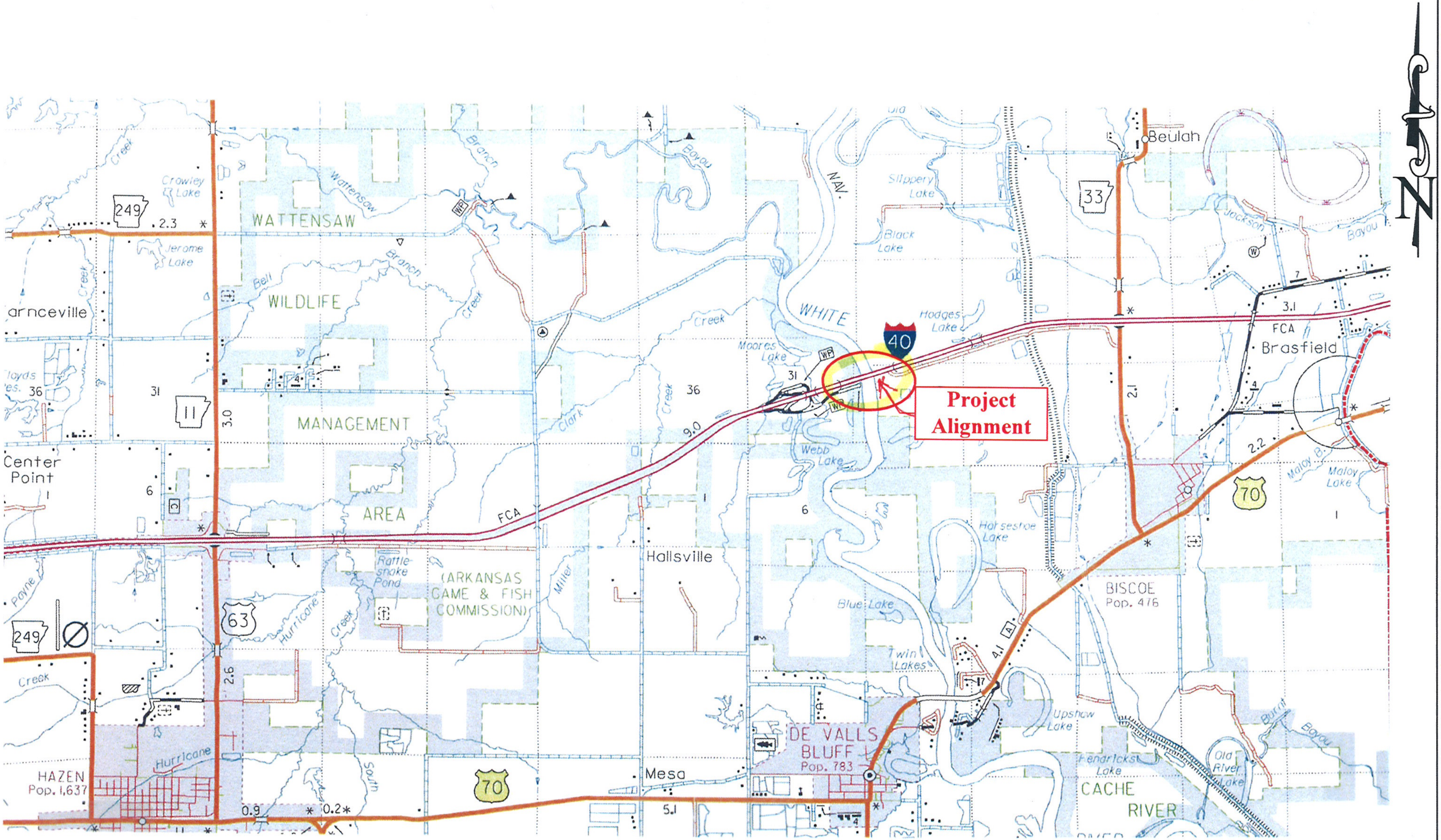
Yongsheng Zhao, Ph.D., P.E.
Project Engineer

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

Mark E. Wyatt, P.E.
President

YZ/MEW:jw

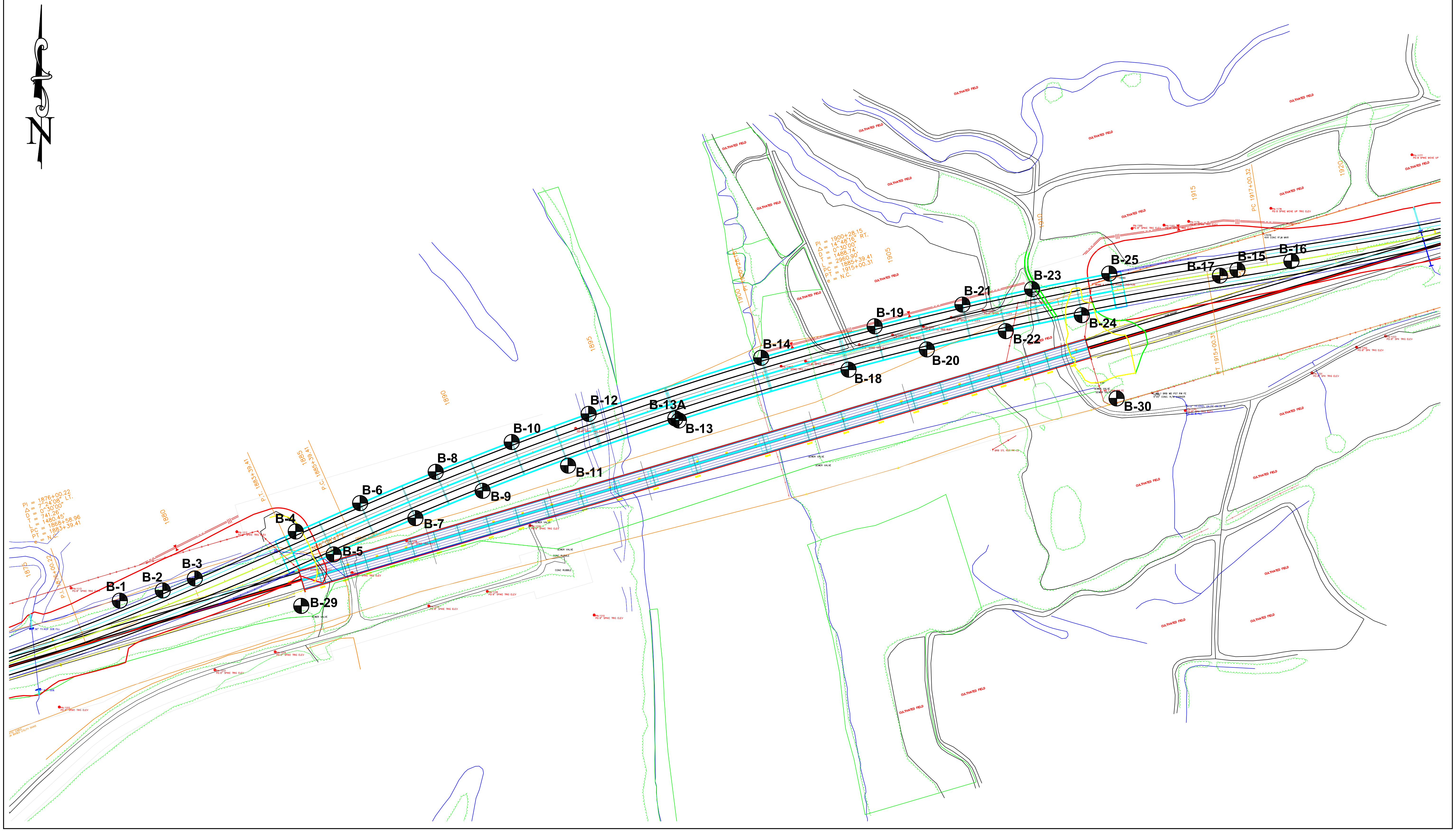
Copies submitted: Jacobs Engineering Group Inc.
Attn: Mr. Mark A. Asher, P.E. (2+electronic)
Attn: Mr. Chris Criswell, P.E. (1-electronic)
Attn: Mr. Mark Schurk, P.E. (1-electronic)
Attn: Mr. Charles Wise, P.E. (1-electronic)



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

Plan of Borings
I-40 over White River
AHTD Job No. BB0610
Prairie County, Arkansas

Job No.: 13-017	As Shown
June 2013	Plate 1



Note: Base drawing provided by Jacobs.

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

Plan of Borings
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Job No.: 13-017

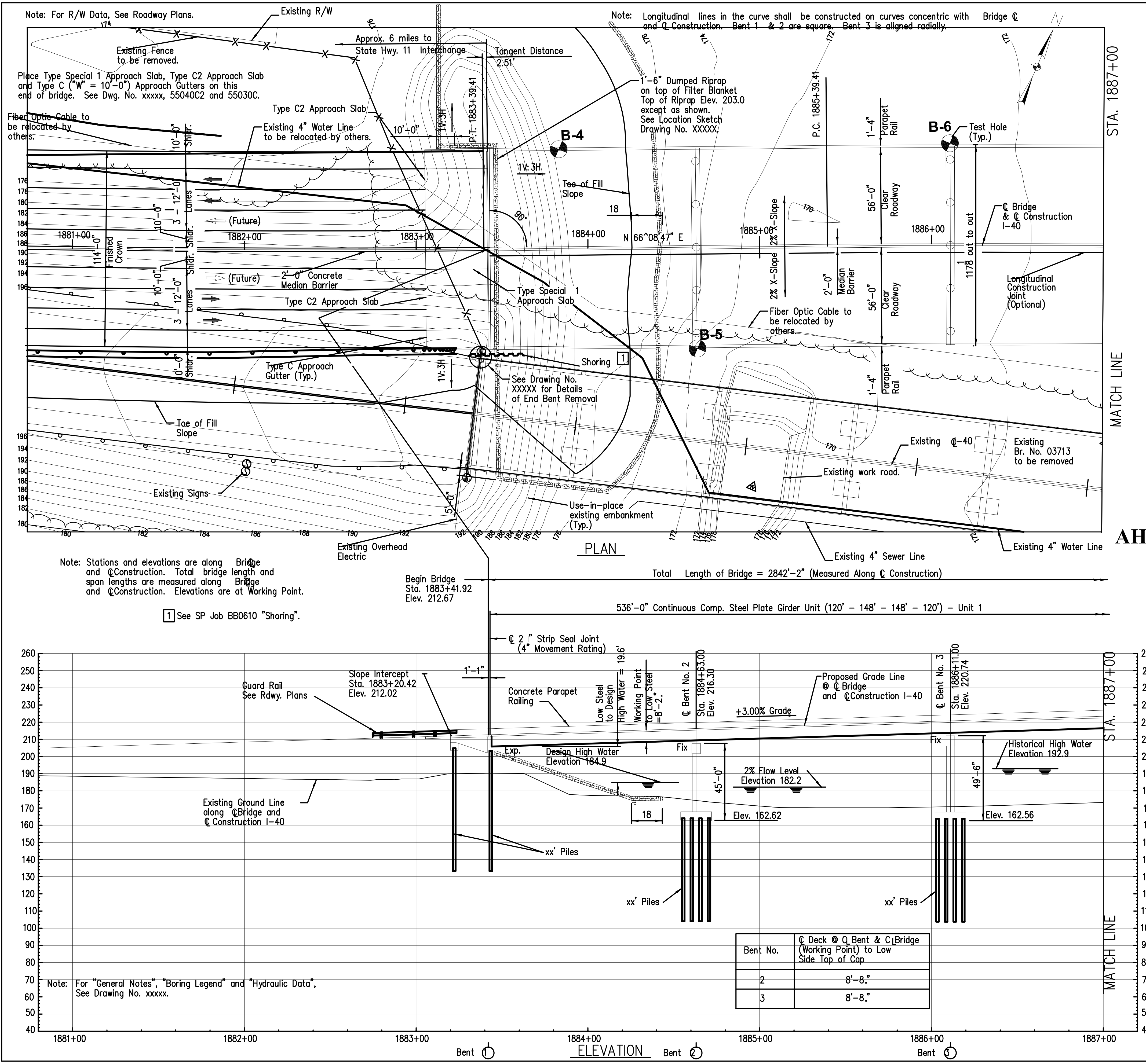
Scale: As Shown

March 26, 2014

Plate 2a

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
2	8'-8"
3	8'-8"

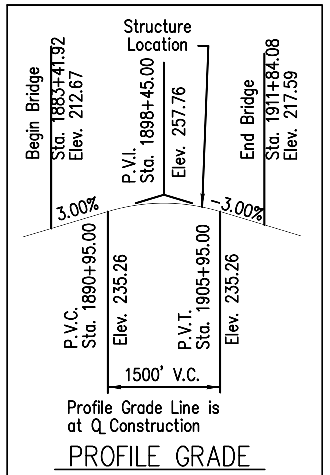
Curve Data

℄C Construction
HORIZONTAL CURVE DATA
PI = 1876+00.22
R = 11,459.16'
Δ = 7^24'08" Lt.
D = 0^30'00"
T = 741.26'
L = 1480.45'
PC = 1868+58.96
PT = 1883+39.41
e = Normal Crown

Curve Data

℄C Construction
HORIZONTAL CURVE DATA
PI = 1900+28.15
R = 11,459.16'
Δ = 14^48'16" Rt.
D = 0^30'00"
T = 1488.74'
L = 2960.90'
PC = 1885+39.41
PT = 1915+00.31
e = Normal Crown

Profile Grade Detail



Profile Grade Line is at ℄ Construction

PROFILE GRADE

Note: All elevations in detail above are shown at working point.

"N" VALUES

Sta. 1883+42 – 60' Left of ℄C Construction

Sta. 1884+63 – 60' Right of ℄C Construction

Sta. 1886+11 – 60' Left of ℄C Construction

Plan of Borings - Plate 2b
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

SHEET 1 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

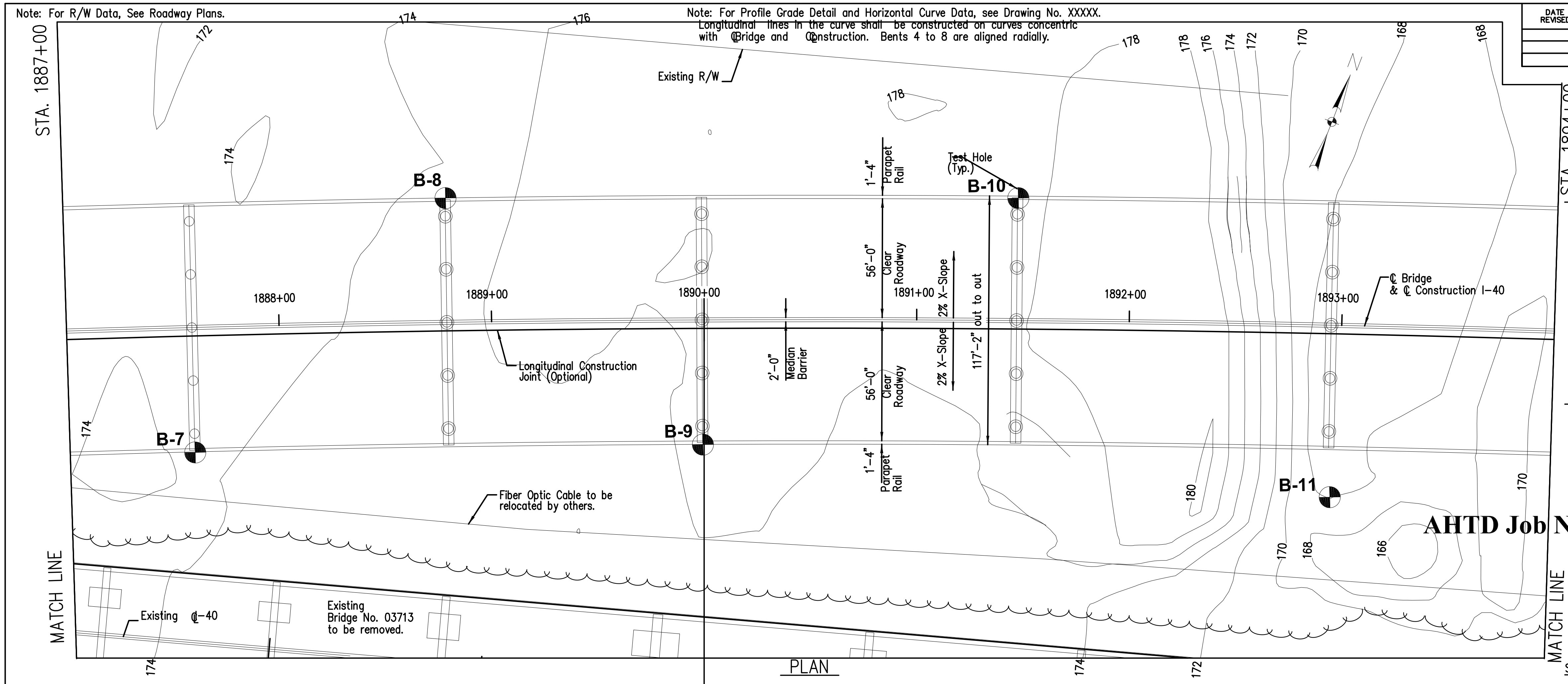
Note: Layout provided by Jacobs.

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

Note: For R/W Data, See Roadway Plans.

Note: For Profile Grade Detail and Horizontal Curve Data, see Drawing No. XXXXX. Longitudinal lines in the curve shall be constructed on curves concentric with ℄Bridge and ℄Construction. Bents 4 to 8 are aligned radially.



Plan of Borings - Plate 2c
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

30 ft 0 30 60 ft

"N" VALUES

Sta. 1887+59 – 60' Right of ℄ Construction

Sta. 1888+79 – 60' Left of ℄ Construction

Sta. 1891+47 – 60' Left of ℄ Construction

Sta. 1889+99 – 60' Right of ℄ Construction

Sta. 1889+99 – 60' Right of ℄ Construction

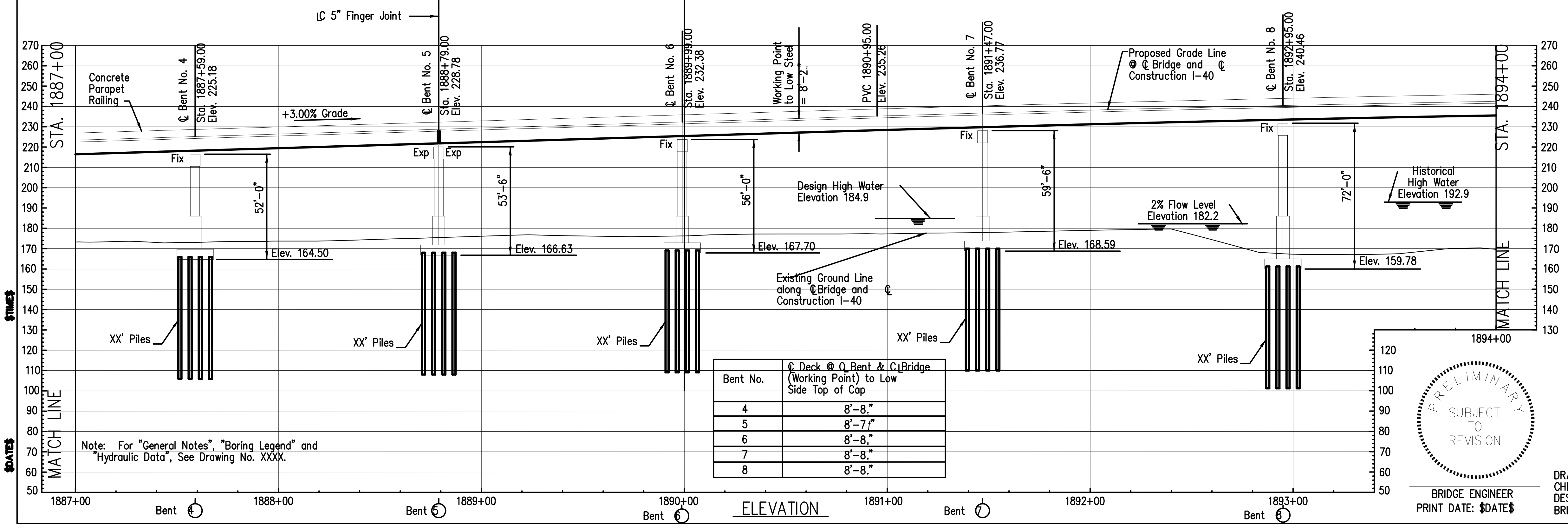
Sta. 1891+47 – 60' Left of ℄ Construction

Sta. 1892+95 – 60' Right of ℄ Construction

Total Length of Bridge = 2842'-2" (Measured Along ℄ Construction)

536'-0" Continuous Comp. Steel Plate Girder Unit (120' – 148' – 148' – 120') – Unit 1

536'-0" Continuous Comp. Steel Plate Girder Unit (120' – 148' – 148' – 120') – Unit 2



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
4	8'-8"
5	8'-7ƒ"
6	8'-8"
7	8'-8"
8	8'-8"

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. XXXX.

Note:
Stations and elevations are along ℄Bridge and ℄Construction. Total bridge length and span lengths are measured along ℄Bridge and ℄Construction. Elevations are at Working Point.

PRELIMINARY SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 2 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

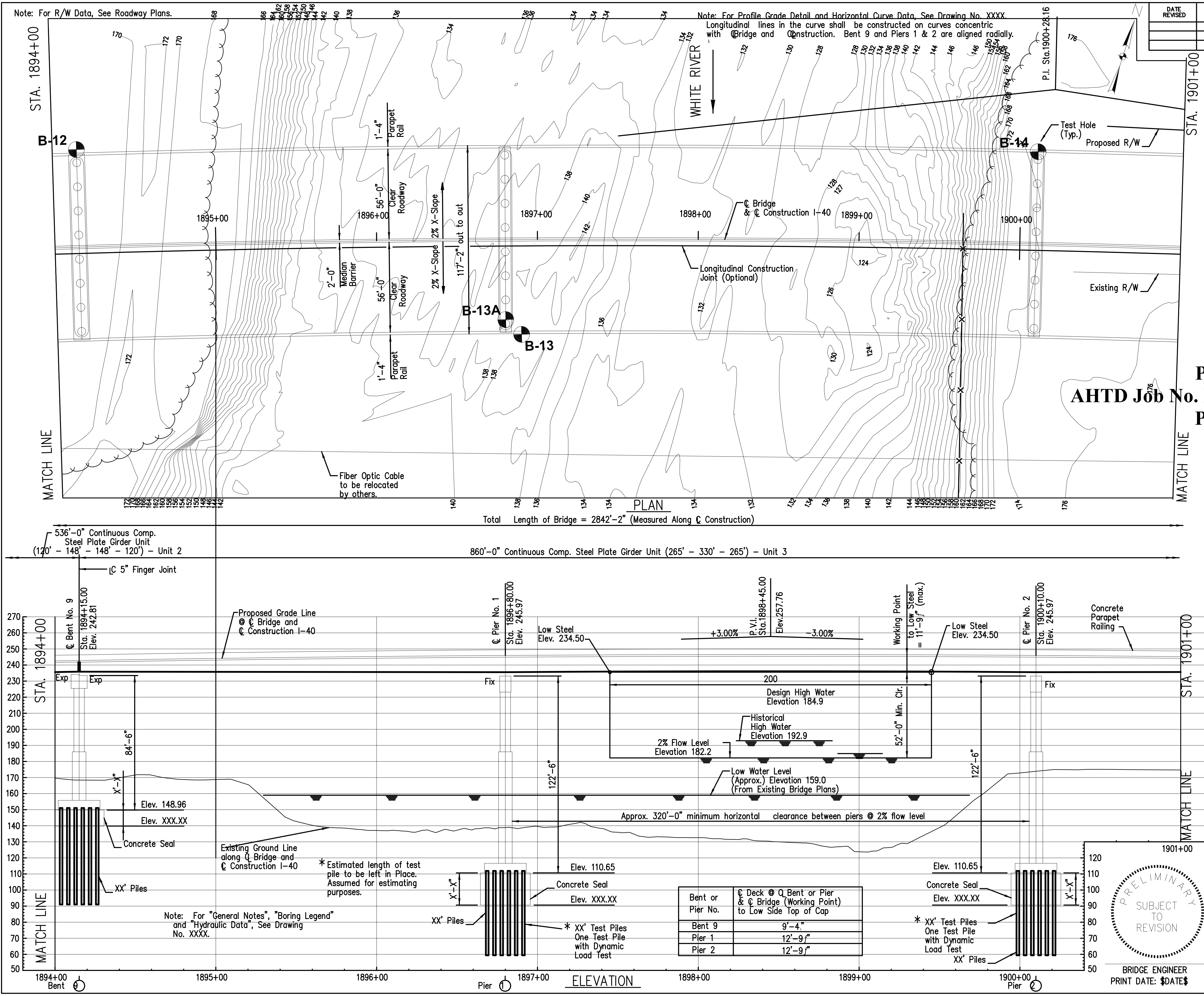
DRAWN BY: LHG DATE: 12/30/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

Note: Layout provided by Jacobs.

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX



Bent or Pier No.	℄ Deck @ ℄ Bent or Pier & ℄ Bridge (Working Point) to Low Side Top of Cap
Bent 9	9'-4"
Pier 1	12'-9⅞"
Pier 2	12'-9⅞"

"N" VALUES

Sta. 1894+15 – 60' Left of ℄ Construction

Sta. 1896+80 – 60' Right of ℄ Construction

Plan of Borings - Plate 2d
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

30 ft 0 30 60 ft

Boring No. 14
Sta. 1900+10 – 60' Left of ℄ Construction

.05–1.5, N=2
2.5–3.5, N=7
4.5–5.5, N=5
6.5–7.5, N=4
9.0–10.0, N=8
14.0–15.0, N=6
19.0–20.0, N=5
24.0–25.0, N=14
29.0–30.0, N=32
34.0–35.0, N=36
39.0–40.0, N=40
44.0–45.0, N=41
49.0–50.0, N=45
58.0–59.0, N=50(11")
68.0–69.0, N=50(11")
78.0–79.0, N=50(10")
88.0–88.5, N=50(6")
97.0–97.5, N=50(7")
108.0–108.5, N=50(6")
118.0–118.5, N=50(4")

Note: Stations and elevations are along ℄ Bridge and ℄ Construction. Total bridge length and span lengths are measured along ℄ Bridge and ℄ Construction. Elevations are at Working Point.

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 3 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO.XXXXX DRAWING NO.XXXXX

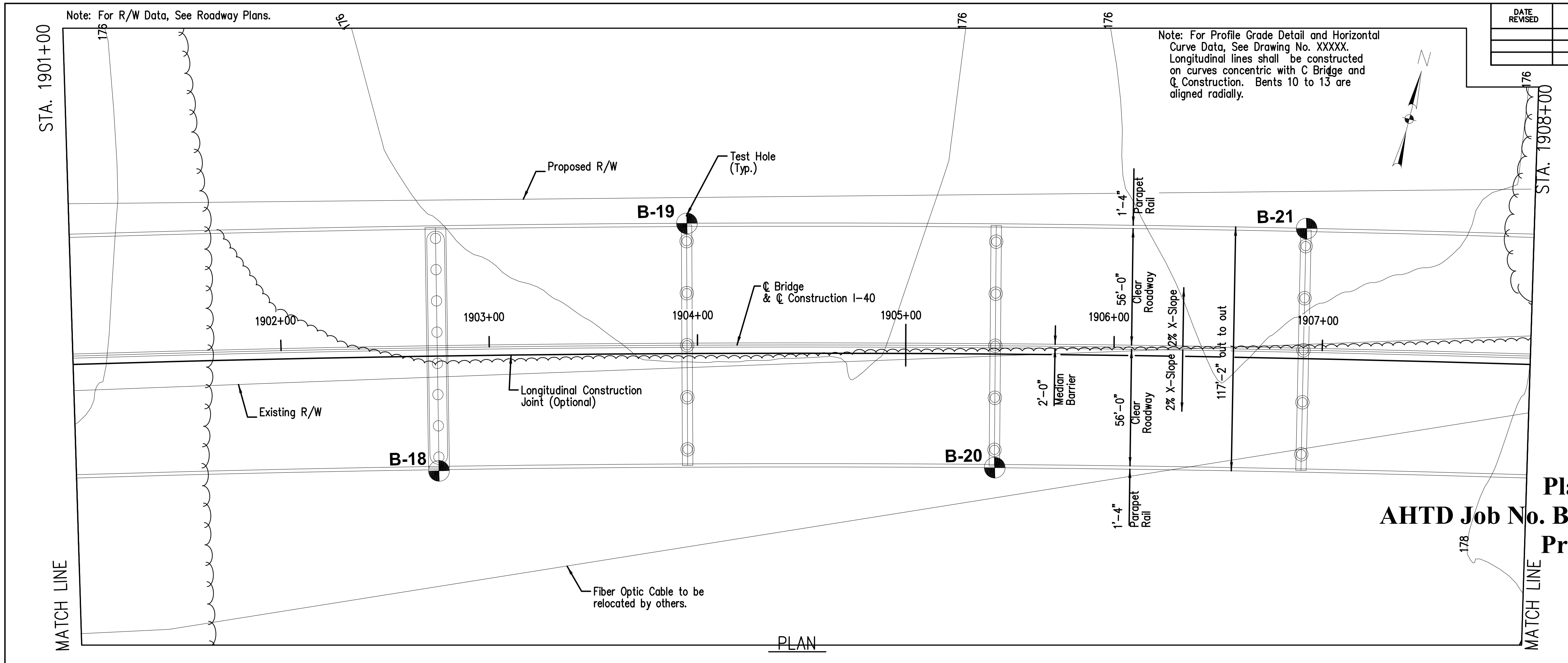
Note: Layout provided by Jacobs.

JACOBS

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

Note: For R/W Data, See Roadway Plans.



Note: For Profile Grade Detail and Horizontal Curve Data, See Drawing No. XXXXX. Longitudinal lines shall be constructed on curves concentric with ℄ Bridge and ℄ Construction. Bents 10 to 13 are aligned radially.

Plan of Borings - Plate 2e
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

"N" VALUES

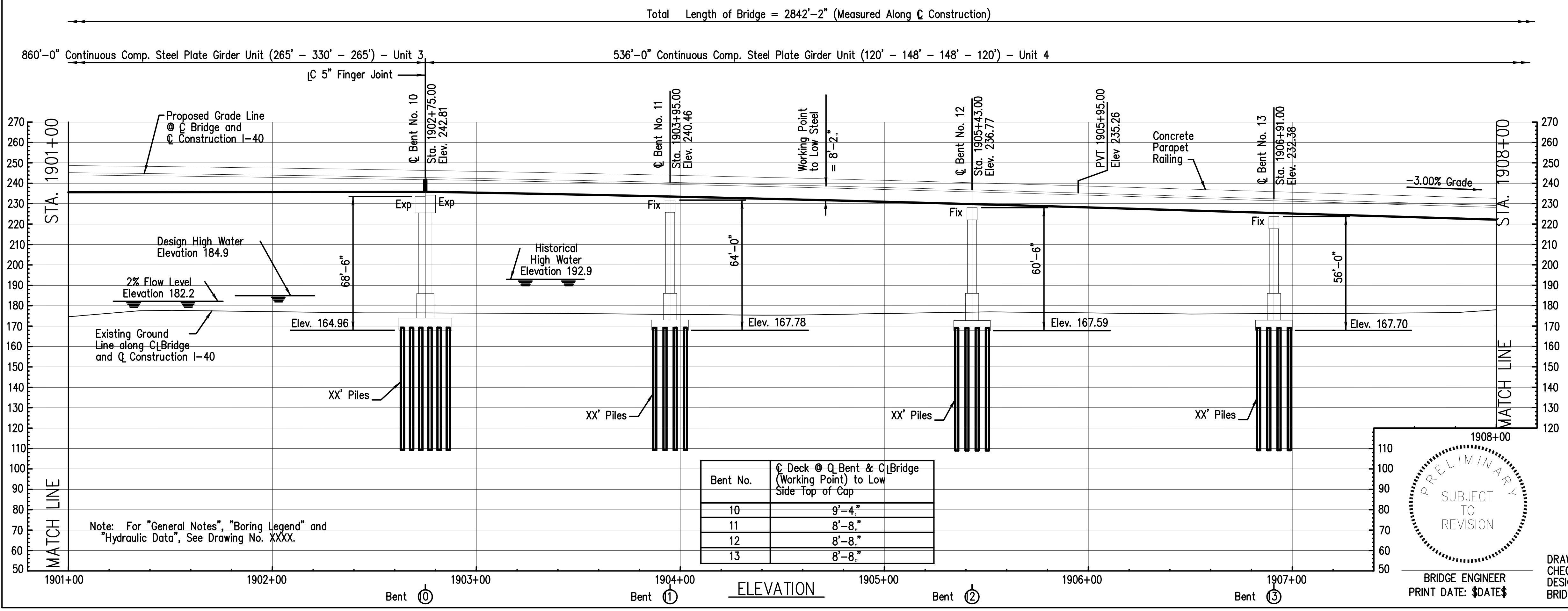
Sta. 1902+75 – 60' Right of ℄ Construction

Sta. 1903+95 – 60' Left of ℄ Construction

Sta. 1905+43 – 60' Right of ℄ Construction

30 ft 0 30 60 ft

Sta. 1906+91 – 60' Left of ℄ Construction



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
10	9'–4"
11	8'–8"
12	8'–8"
13	8'–8"

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. XXXX.

Note: Stations and elevations are along ℄ Bridge ℄ Construction. Total bridge length and span lengths are measured along ℄ Bridge and ℄ Construction. Elevations are at Working Point.

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 4 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'–0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

$FILE$
$TIME$
$DATE$

Note: Layout provided by Jacobs.

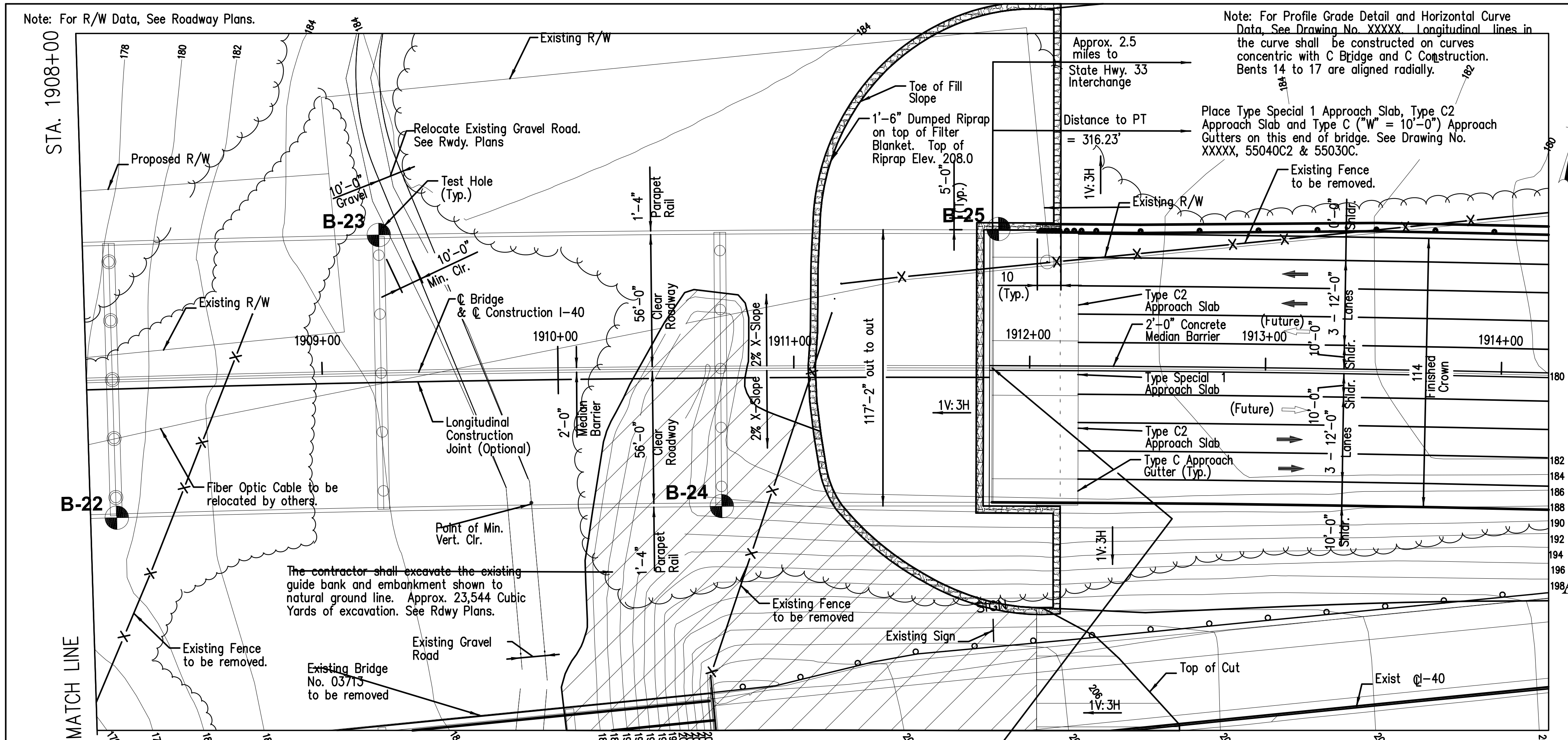
JACOBS

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

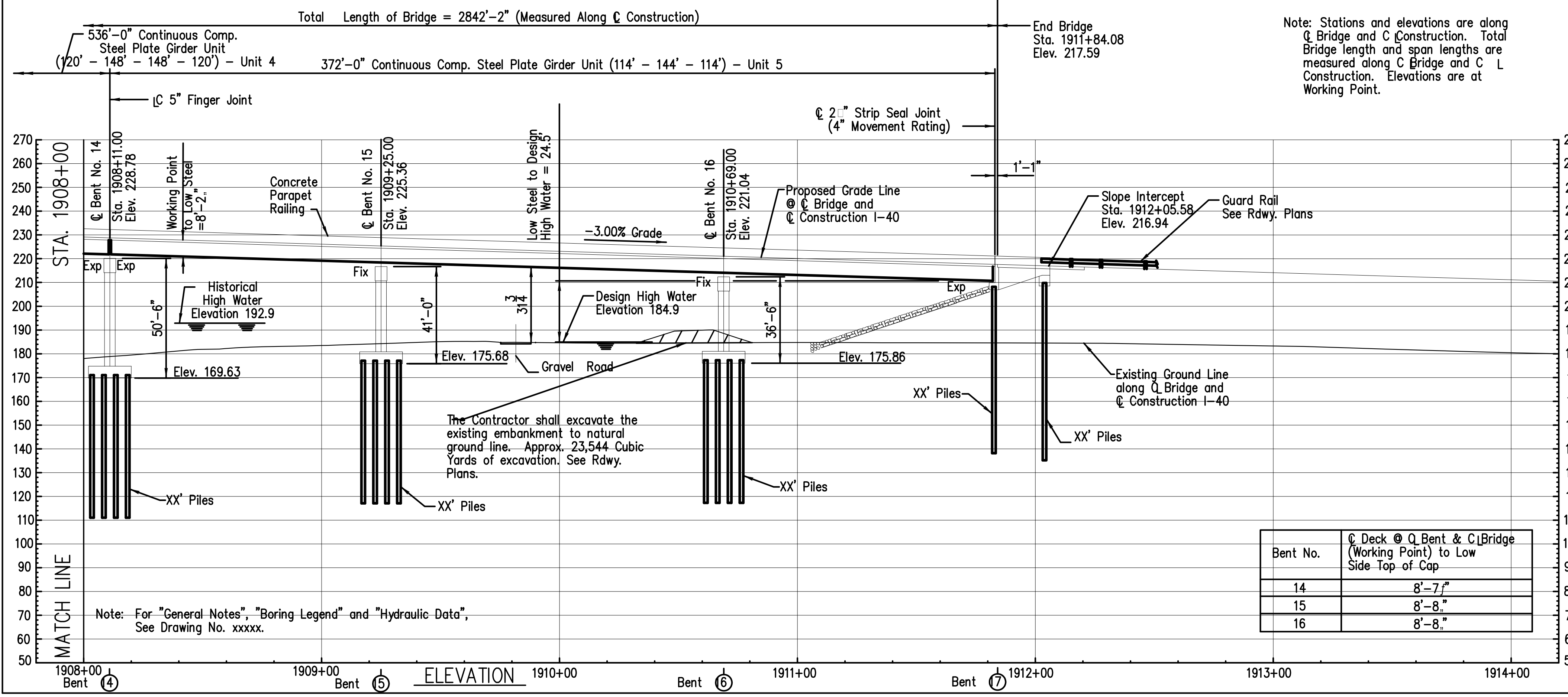
Note: For R/W Data, See Roadway Plans.

Note: For Profile Grade Detail and Horizontal Curve Data, See Drawing No. XXXXX. Longitudinal lines in the curve shall be constructed on curves concentric with C Bridge and C Construction. Bents 14 to 17 are aligned radially.



PLAN

Note: Stations and elevations are along ℄ Bridge and C Construction. Total Bridge length and span lengths are measured along C Bridge and C L Construction. Elevations are at Working Point.



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
14	8'-7ƒ"
15	8'-8„"
16	8'-8„"

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. xxxxx.

ELEVATION

"N" VALUES

Sta. 1908+11 – 60' Right of ℄ Construction	Sta. 1909+25 – 60' Left of ℄ Construction

Plan of Borings - Plate 2f
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Sta. 1910+69 – 60' Right of ℄ Construction	Sta. 1911+84 – 60' Left of ℄ Construction

30 ft 0 30 60 ft

SHEET 5 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

PRELIMINARY SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

$FILE$ $TIME$ $DATE$

Note: Layout provided by Jacobs.

JACOBS

13-017

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

LOG OF BORING NO. 1

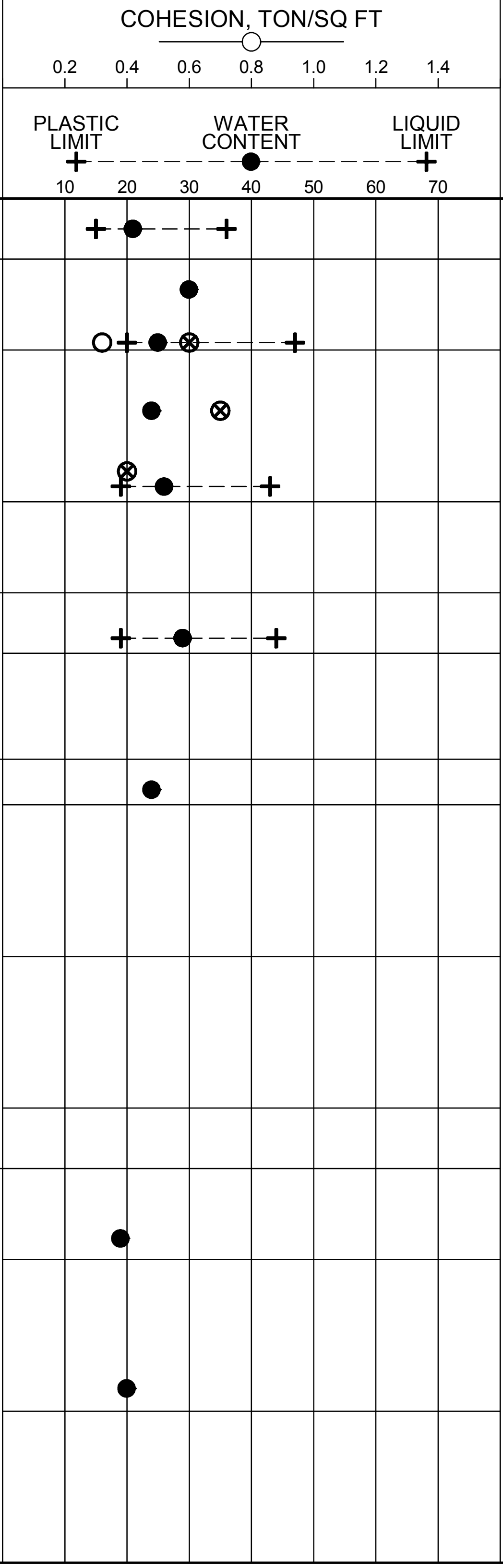
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1877+60, 70 ft Lt

SURF. EL: 184.3

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm reddish brown silty clay w/clayey silt pockets (fill)	8		82
	Soft to firm brown and gray silty clay w/ferrous stains - firm to stiff at 2 to 4 ft	5		
5	- with organic stains to 3 ft - water at 4 ft - stiff at 4 to 8 ft		96	96
	- firm below 8 ft			
10			95	97
15	Soft brownish gray silty clay, moist	6		97
20	Medium dense brown and gray silty fine sand w/occasional silty clay pockets	11		13
25	- dense, less silty and coarser below 24 ft	35		
30		41		
35	Dense brown and gray fine to medium sand, slightly silty	50/7"		6
40	- with clayey fine sand seams at 39 to 45 ft	50/6"		16
		41		



COMPLETION DEPTH: 70.0 ft
DATE: 4-30-13

DEPTH TO WATER
IN BORING: 4 ft

DATE: 4/30/2013

LGBNEW 13-017.GPJ 12-2-13

13-017

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

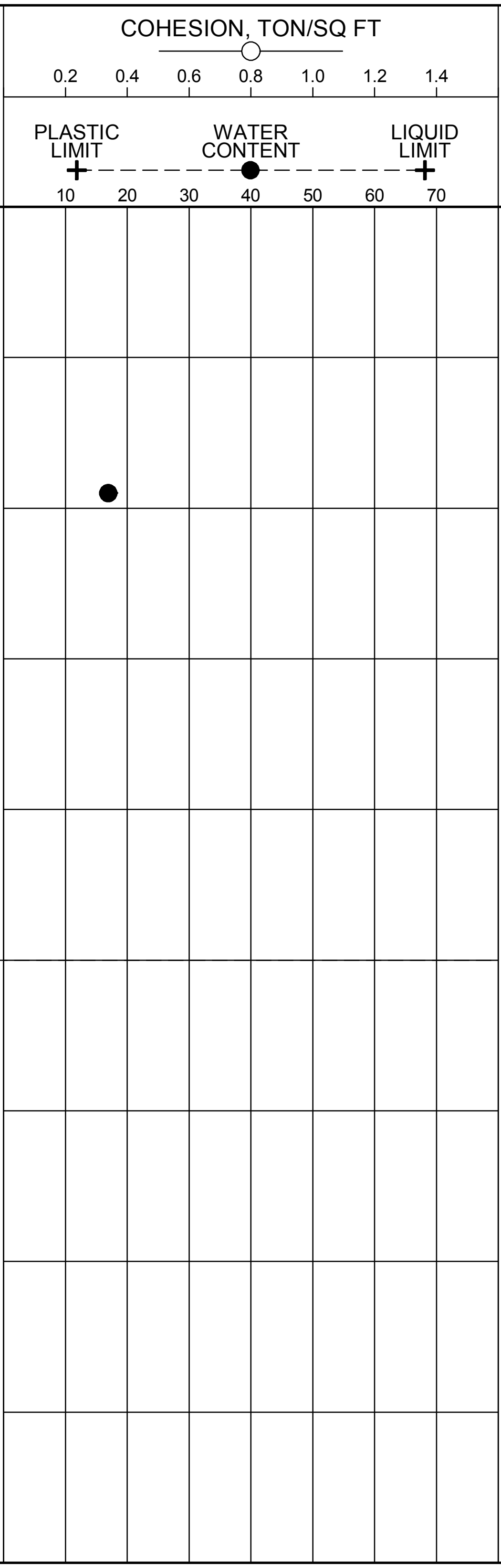
LOG OF BORING NO. 1

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1877+60, 70 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒		30		
			- dense to very dense with trace fine to coarse gravel below 52 ft			
55		☒		50/8"		5
		☒		50/8"		
60		▨				
65		☒		50/8"		
70		☒		50/8"		
75						
80						
85						



COMPLETION DEPTH: 70.0 ft
DATE: 4-30-13

DEPTH TO WATER
IN BORING: 4 ft

DATE: 4/30/2013

LGBNEW 13-017.GPJ 12-2-13

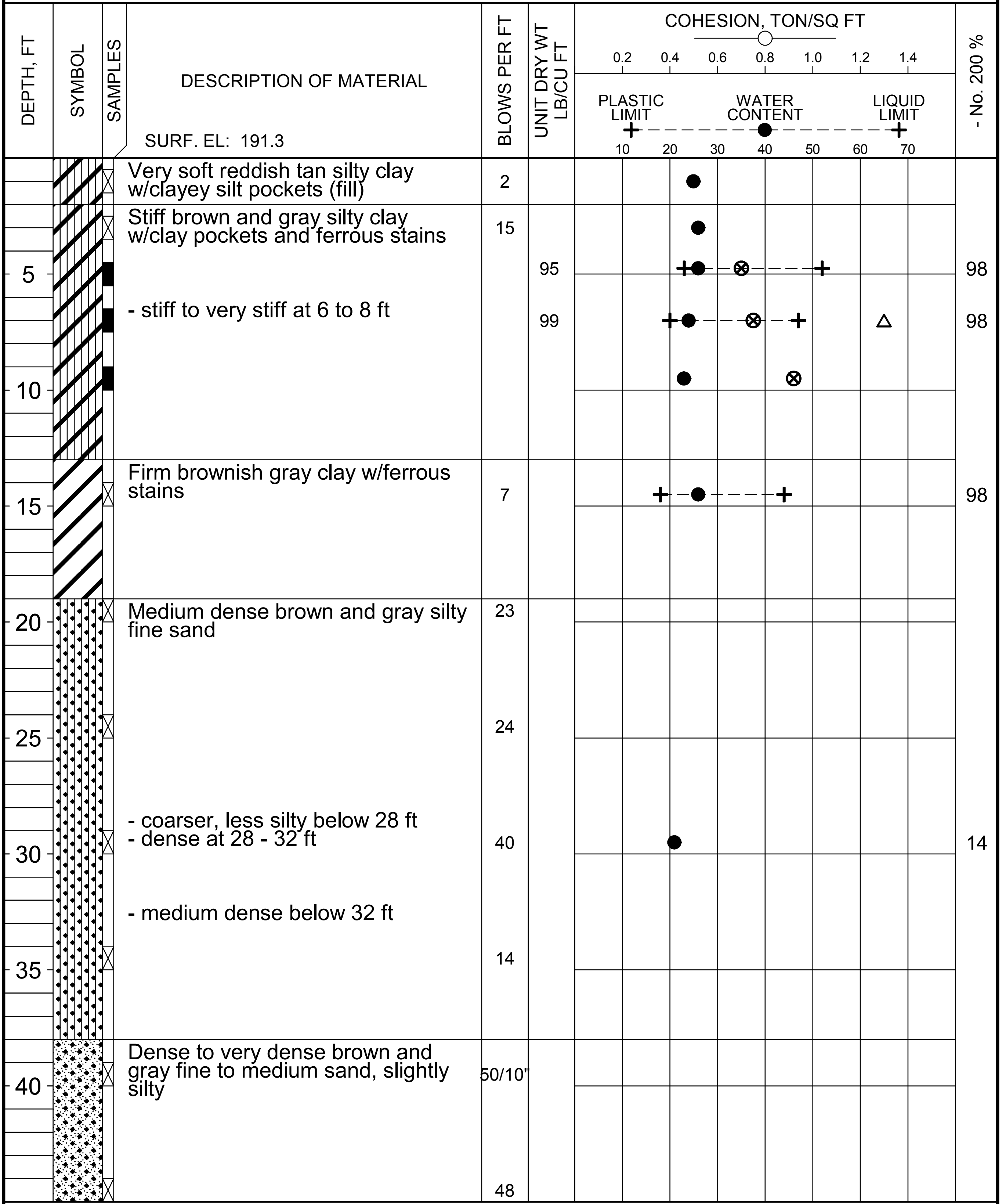
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 2

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 13.5 ft /Wash

LOCATION: Sta 1879+00, 50 ft Lt



SURF. EL: 191.3

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Very soft reddish tan silty clay w/clayey silt pockets (fill)	2		
Stiff brown and gray silty clay w/clay pockets and ferrous stains	15		
		95	98
- stiff to very stiff at 6 to 8 ft		99	98
Firm brownish gray clay w/ferrous stains	7		98
Medium dense brown and gray silty fine sand	23		
	24		
- coarser, less silty below 28 ft - dense at 28 - 32 ft	40		14
- medium dense below 32 ft	14		
Dense to very dense brown and gray fine to medium sand, slightly silty	50/10"		
	48		

COMPLETION DEPTH: 70.0 ft
DATE: 5-1-13

DEPTH TO WATER
IN BORING: Dry to 13.5 ft

DATE: 5/1/2013

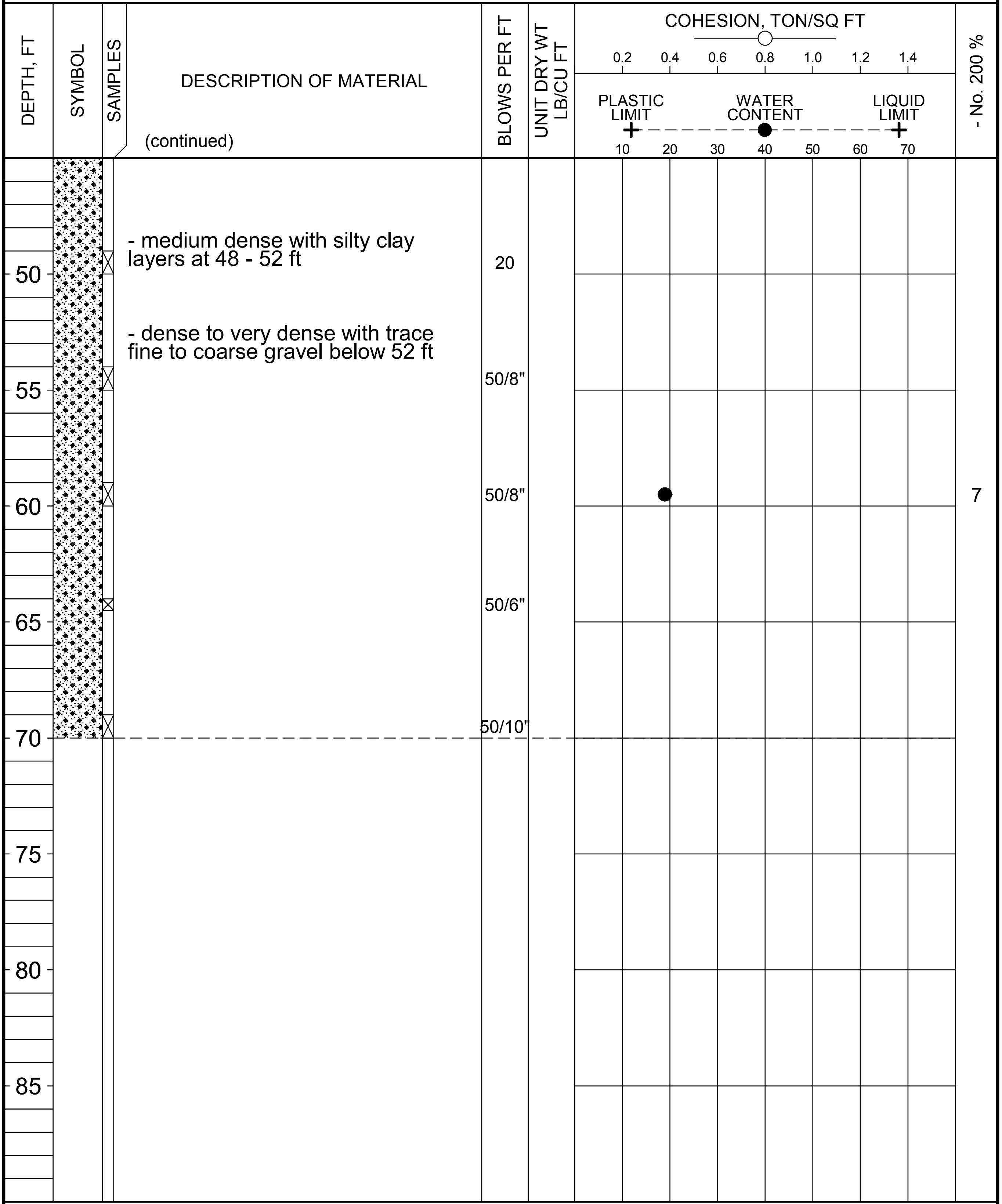
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

L O G O F B O R I N G N O. 2

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 13.5 ft /Wash

LOCATION: Sta 1879+00, 50 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	- medium dense with silty clay layers at 48 - 52 ft	20		
50				
	- dense to very dense with trace fine to coarse gravel below 52 ft			
		50/8"		
55				
		50/8"		7
60				
		50/6"		
65				
		50/10"		
70				
75				
80				
85				

COMPLETION DEPTH: 70.0 ft
DATE: 5-1-13

DEPTH TO WATER
IN BORING: Dry to 13.5 ft

DATE: 5/1/2013

13-017

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

LOG OF BORING NO. 3

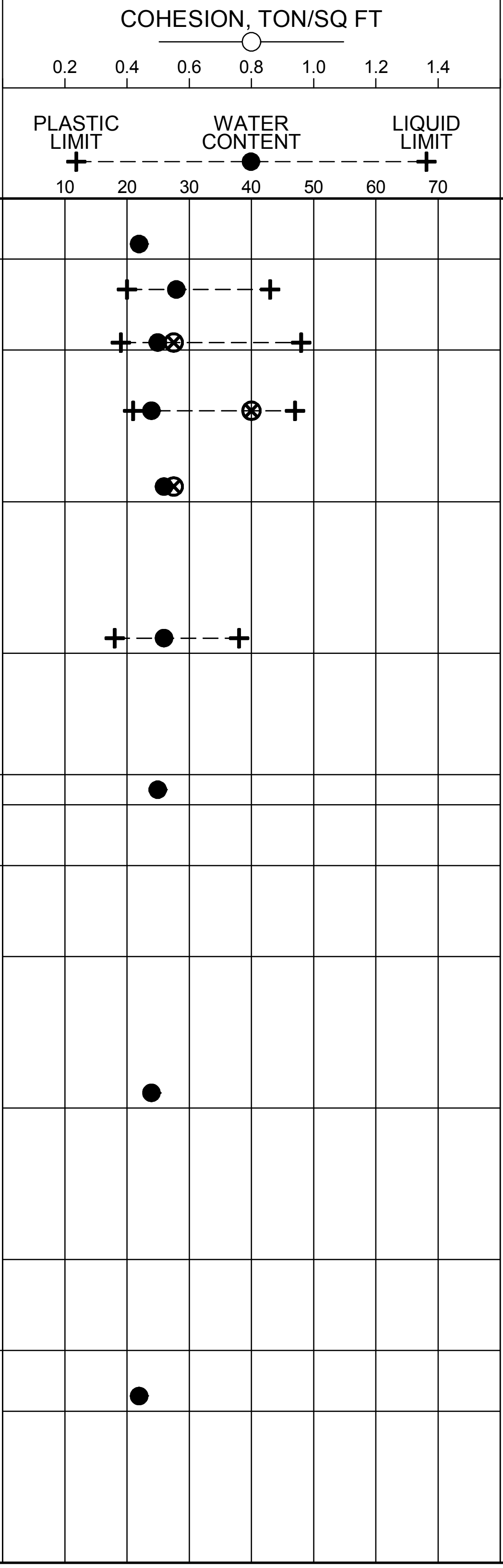
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1880+15 , 50 ft Lt

SURF. EL: 190.5

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft reddish tan and brown clay w/clayey silt pockets and surface organics	4		
	Soft brown and gray silty clay w/ferrous stains - stiff at 4 - 12 ft	6		97
5			99	95
			100	95
10				
	- firm to stiff below 12 ft			
15		10		93
20	Medium dense clayey fine sand, wet	15		39
	Dense brown and tan silty fine sand w/trace silty clay pockets, wet			
25		39		
	- brown and gray below 28 ft			
30		31		15
35		30		
	Medium dense brown and gray fine to medium sand, slightly silty			
40		22		6
	- dense below 42 ft			
		39		



COMPLETION DEPTH: 70.0 ft
DATE: 5-2-13

DEPTH TO WATER
IN BORING: Dry to 8 ft

DATE: 5/2/2013

13-017

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

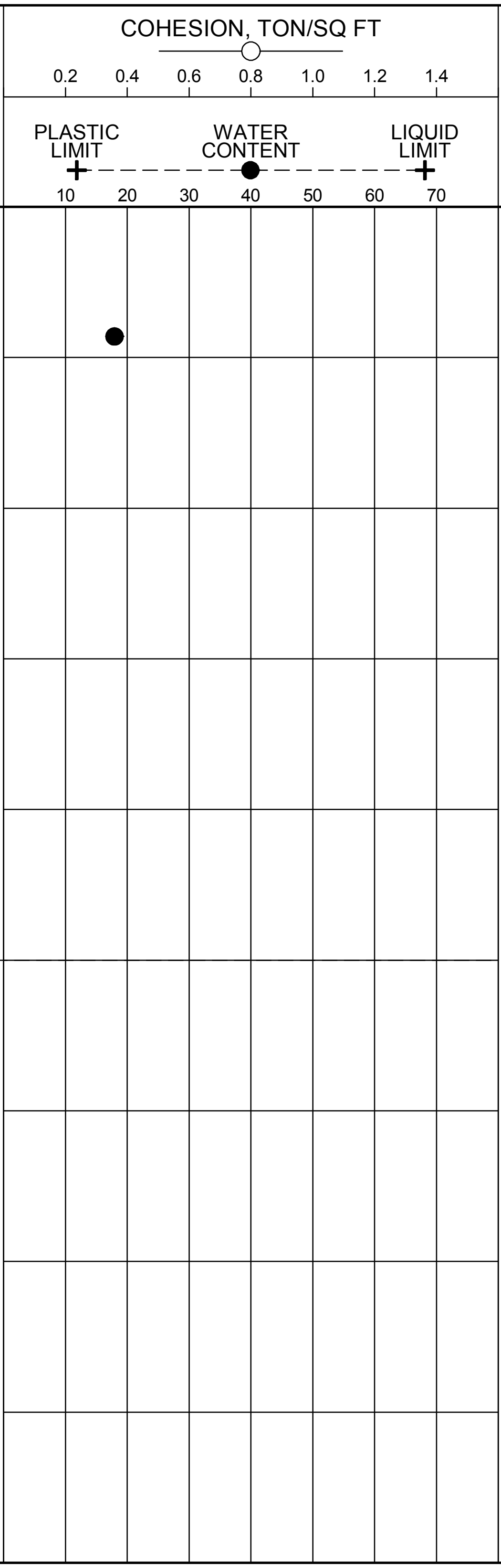
LOG OF BORING NO. 3

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1880+15 , 50 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
			- dense to very dense below 48 ft			
		☒		50/7"		11
50						
			- with trace fine to coarse gravel below 52 ft			
		☒		50/8"		
55						
		☒		50/6"		
60						
		☒		50/8"		
65						
70		☒		50/6"		
75						
80						
85						



COMPLETION DEPTH: 70.0 ft
DATE: 5-2-13

DEPTH TO WATER
IN BORING: Dry to 8 ft

DATE: 5/2/2013

LGBNEW 13-017.GPJ 12-2-13

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

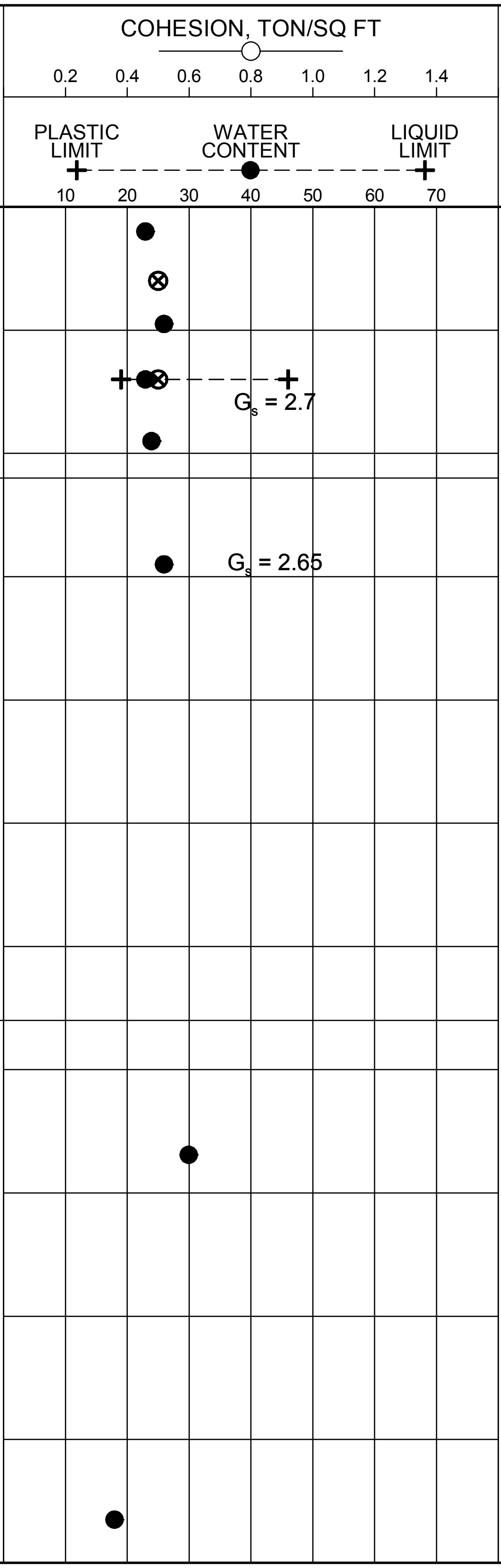
LOG OF BORING NO. 4

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1883+83, 60 ft Lt

SURF. EL: 171.9

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm gray and brown silty clay w/silt pockets and ferrous nodules and stains	9		
5	- stiff below 4 ft	13		
			100	37
10	- water at 8 ft	23		
	Medium dense gray and brown fine to medium sand, slightly silty			
15		22		5
20		23		
25		26		
30	- dense below 28 ft	45		
	Dense gray fine to medium sand	50		
35				
	- dense to very dense below 38 ft	50/9"		4
40				
		50/8"		
45				
		50/9"		
50				
		50/6"		10



COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

LGBNEW 13-017.GPJ 4-7-14

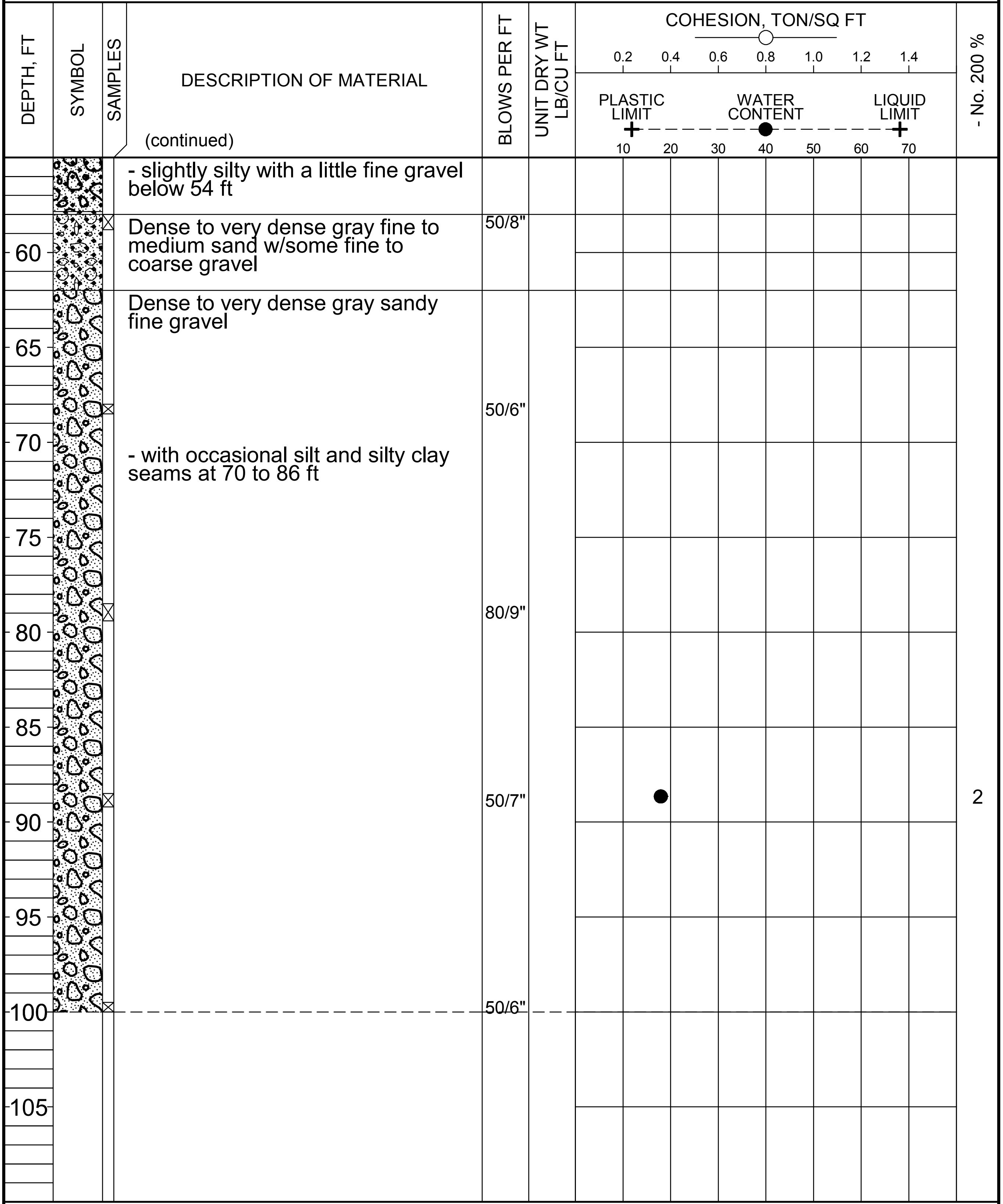
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 4

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1883+83, 60 ft Lt



DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
- slightly silty with a little fine gravel below 54 ft			
Dense to very dense gray fine to medium sand w/some fine to coarse gravel	50/8"		
Dense to very dense gray sandy fine gravel			
	50/6"		
- with occasional silt and silty clay seams at 70 to 86 ft			
	80/9"		
	50/7"		2
	50/6"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

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

LOG OF BORING NO. 5

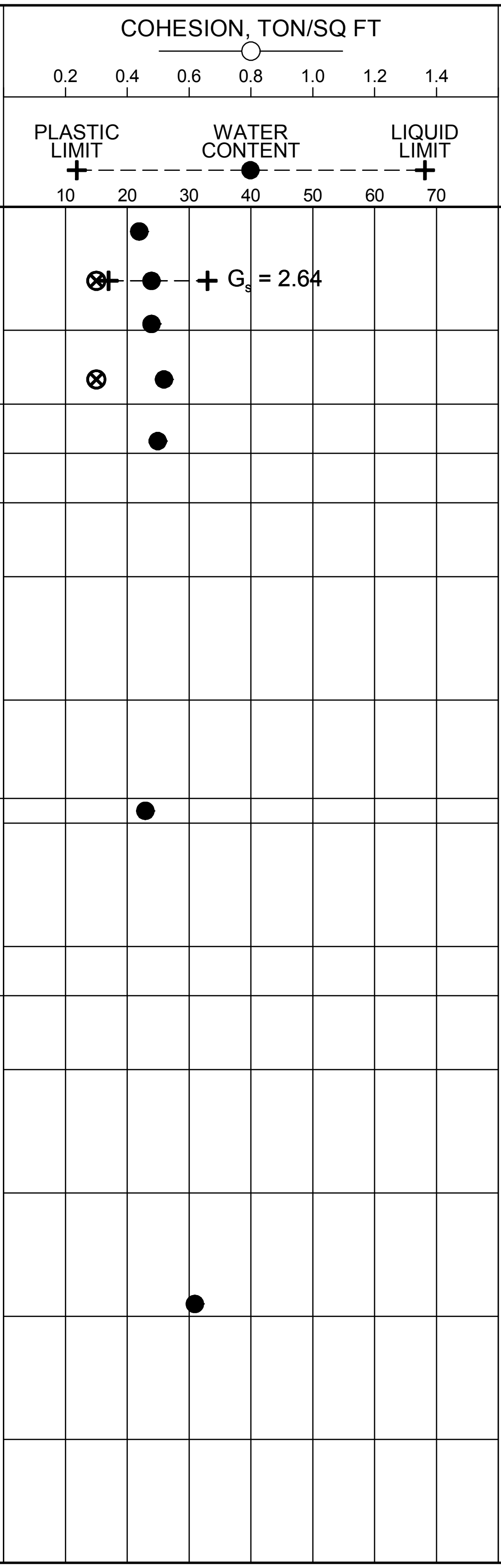
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1884+63, 60 ft Rt

SURF. EL: 171.7

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm dark gray and brown silty clay, slightly sandy w/silt pockets and ferrous nodules and stains	9		
			87
	8		
- water at 8 ft			
Loose gray and brown silty fine sand, slightly clayey, wet	10		31
Medium dense reddish tan and tan silty fine sand	27		
- dense below 18 ft	35		
- brown below 22 ft			
Dense to very dense gray and tan fine sand w/trace medium sand	50/10"		2
	50/8"		
Medium dense gray fine to medium sand w/occasional organic inclusions	29		
- dense to very dense below 38 ft	50/9"		
	50/10"		4
	50/9"		
- with some fine to coarse gravel below 52 ft	70/3"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-25-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/25/2014

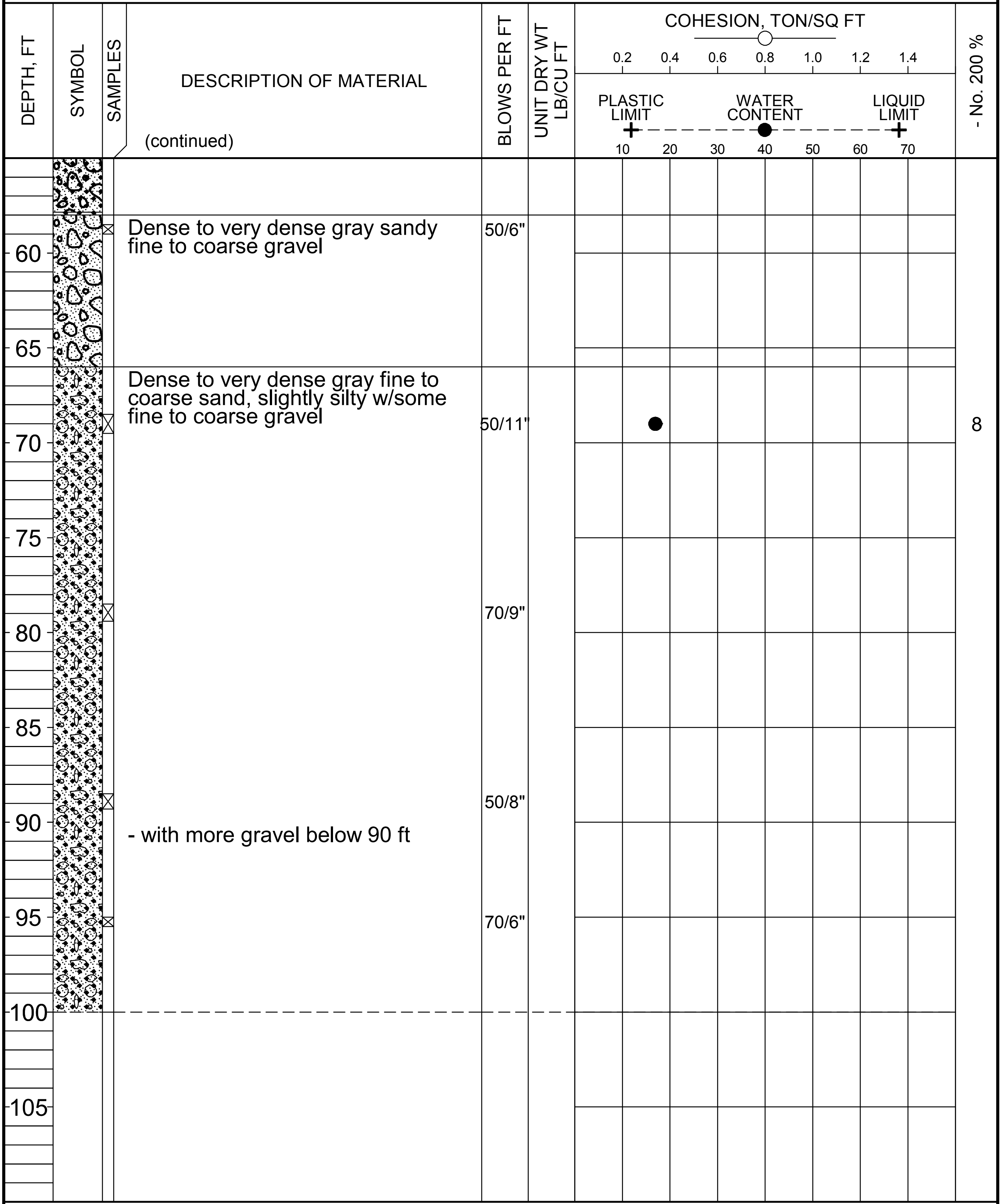
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 5

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1884+63, 60 ft Rt



DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Dense to very dense gray sandy fine to coarse gravel	50/6"		
Dense to very dense gray fine to coarse sand, slightly silty w/some fine to coarse gravel	50/11"		8
	70/9"		
	50/8"		
- with more gravel below 90 ft			
	70/6"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-25-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/25/2014

13-017

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

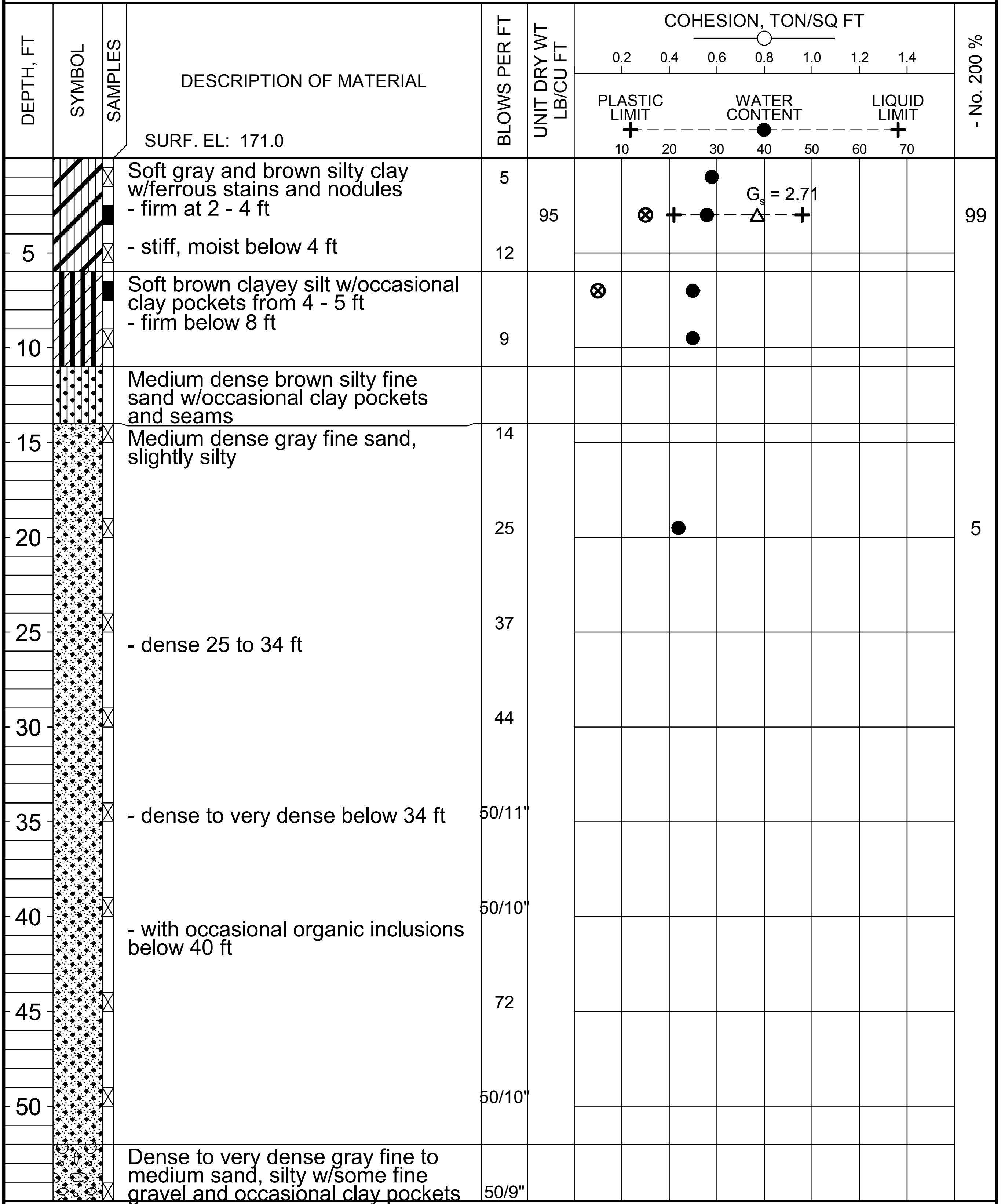
LOG OF BORING NO. 6

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1886+11, 60 ft Lt

SURF. EL: 171.0



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft gray and brown silty clay w/ferrous stains and nodules	5		
	- firm at 2 - 4 ft		95	99
5	- stiff, moist below 4 ft	12		
	Soft brown clayey silt w/occasional clay pockets from 4 - 5 ft - firm below 8 ft			
10		9		
	Medium dense brown silty fine sand w/occasional clay pockets and seams			
15	Medium dense gray fine sand, slightly silty	14		
20		25		5
25	- dense 25 to 34 ft	37		
30		44		
35	- dense to very dense below 34 ft	50/11"		
40	- with occasional organic inclusions below 40 ft	50/10"		
45		72		
50		50/10"		
	Dense to very dense gray fine to medium sand, silty w/some fine gravel and occasional clay pockets	50/9"		

COMPLETION DEPTH: 100.0 ft
DATE: 3-11-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/11/2014

LGBNEW 13-017.GPJ 4-7-14

13-017

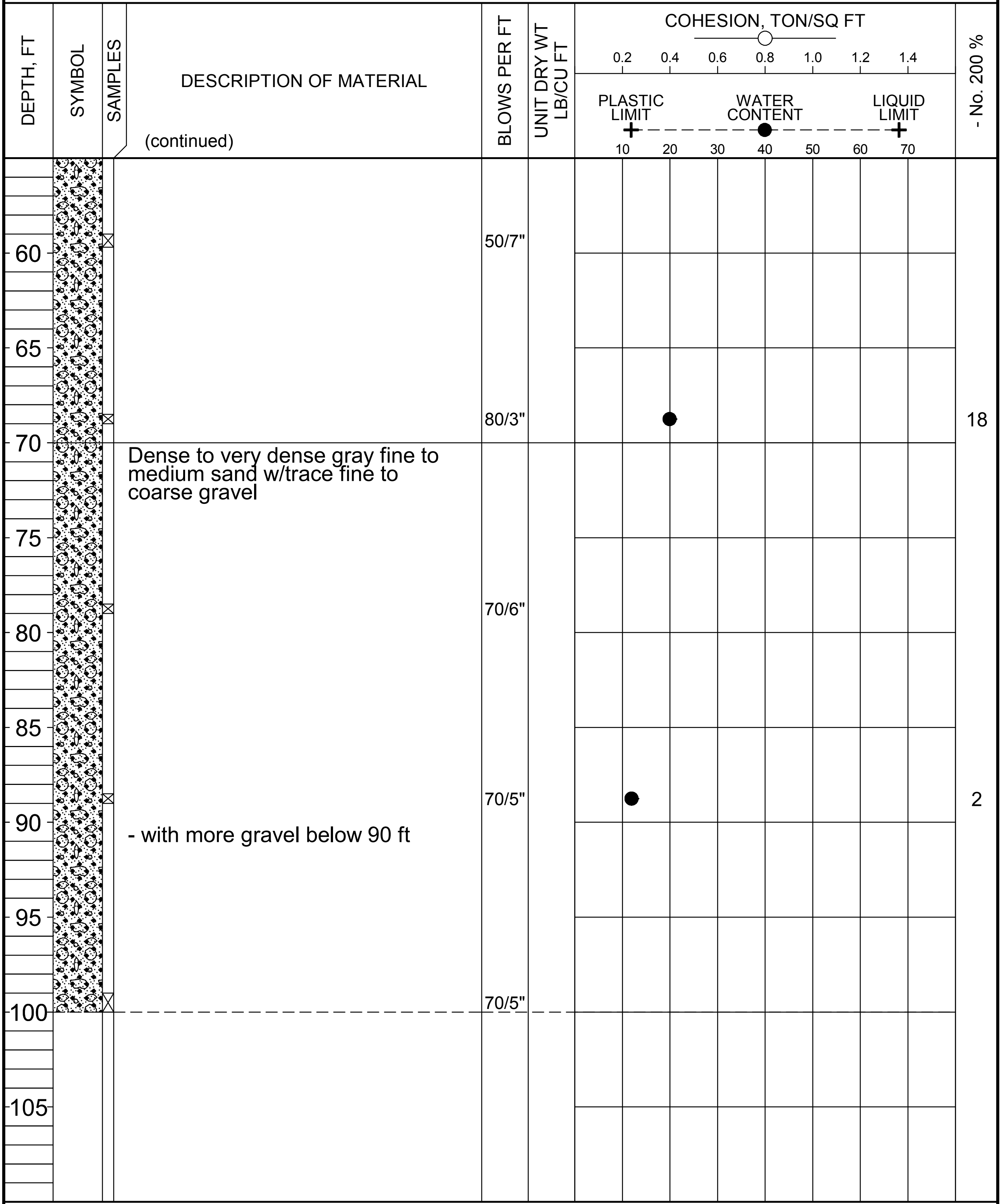
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 6

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1886+11, 60 ft Lt



Dense to very dense gray fine to medium sand w/trace fine to coarse gravel

- with more gravel below 90 ft

COMPLETION DEPTH: 100.0 ft
DATE: 3-11-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/11/2014

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

LOG OF BORING NO. 7

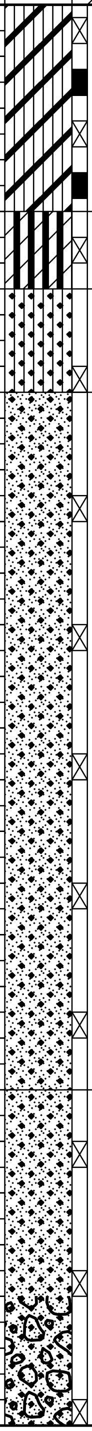
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

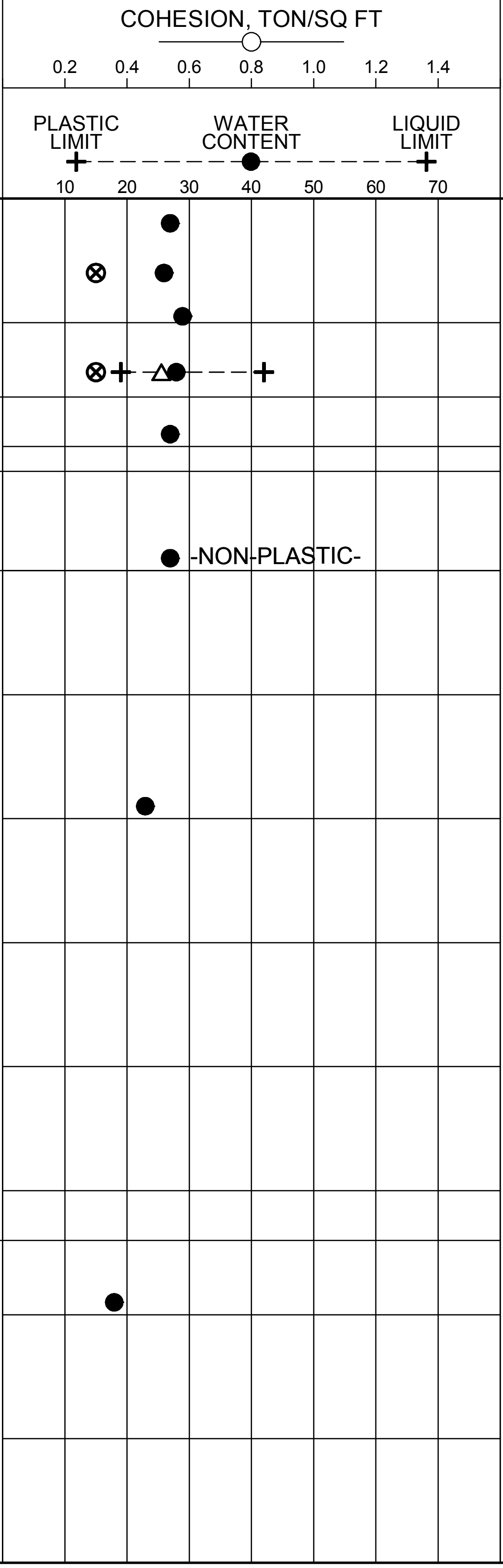
TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1887+59, 60 ft Rt

SURF. EL: 173.5

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm brown silty clay w/ferrous stains and nodules and some organics - gray and brown below 2 ft	7		
5		9		
			94	97
10	Soft brown clayey silt w/occasional silty clay pockets, moist	6		
	Medium dense brown and gray silty fine sand			
15		14		41
	Medium dense gray fine sand, slightly silty			
20		27		
25	- dense at 24 - 33 ft - with trace fine gravel at 25 - 30 ft	44		9
30		50		
35	- dense to very dense below 33 ft	50/9"		
40		50/9"		
	Dense to very dense gray fine to medium sand, silty w/occasional clay seams and layers			
45		50/8"		19
50	- with some fine to coarse gravel below 50 ft	50/11"		
		80/8"		





COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/12/2014

LGBNEW 13-017.GPJ 4-7-14

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

LOG OF BORING NO. 7

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1887+59, 60 ft Rt

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2–1.4) / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT (10–70)	- No. 200 %
		Dense gray sandy fine to coarse gravel				
60	☒		80/8"			
65		- with occasional fine to medium sand layers at 65 - 75 ft				
70	☒		80/8"			
75						
80	☒		80/7"			
85						
90	☒		80/6"		● 20	4
95						
100	☒		80/6"			
105						

COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/12/2014

13-017

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

LOG OF BORING NO. 8

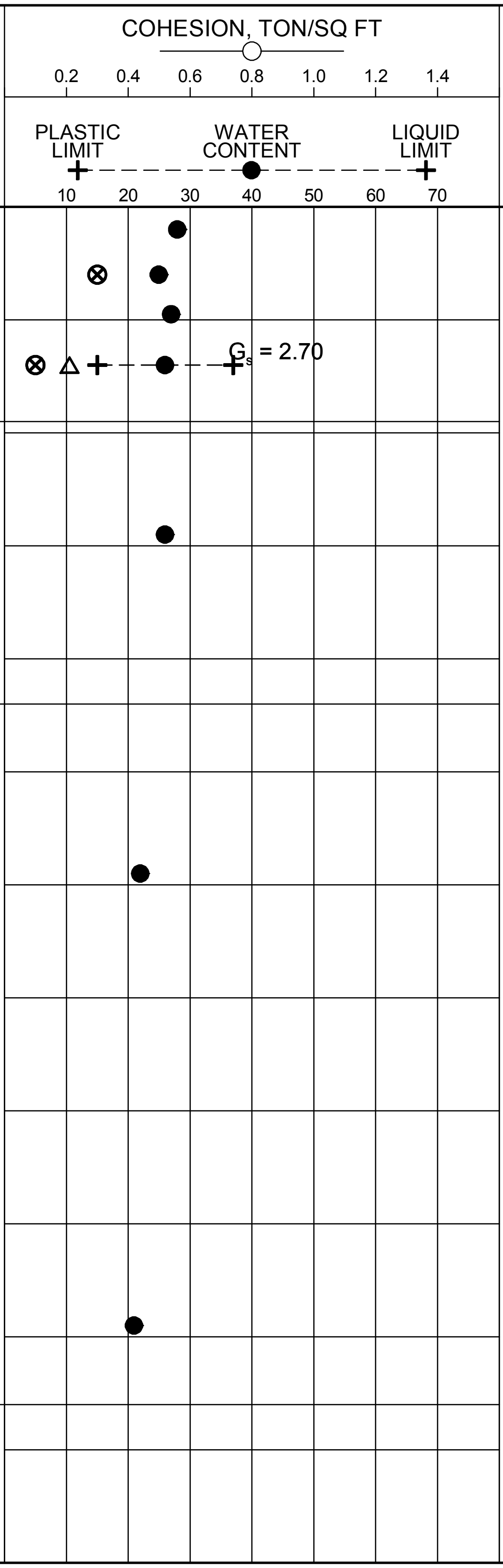
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1888+79, 60 ft Lt

SURF. EL: 174.4

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm to stiff gray and brown silty clay, sandy w/occasional silt pockets and ferrous nodules and stains	10		
	14		
- soft below 6 ft			81
- water at 8 ft	16		
Medium dense brown silty fine sand			
- brown and tan below 13 ft - loose at 13 - 18 ft	10		29
- medium dense below 18 ft	21		
Dense gray fine sand, slightly silty	52		
- dense to very dense at 28 to 43 ft	50/11"		5
- with organic inclusions below 34 ft	50/8"		
	50/11"		
- dense with trace fine gravel below 43 ft	50		
	49		12
Dense to very dense gray fine to medium sand w/trace fine to coarse gravel	50/10"		
	50/9"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

LGBNEW 13-017.GPJ 4-7-14

13-017

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

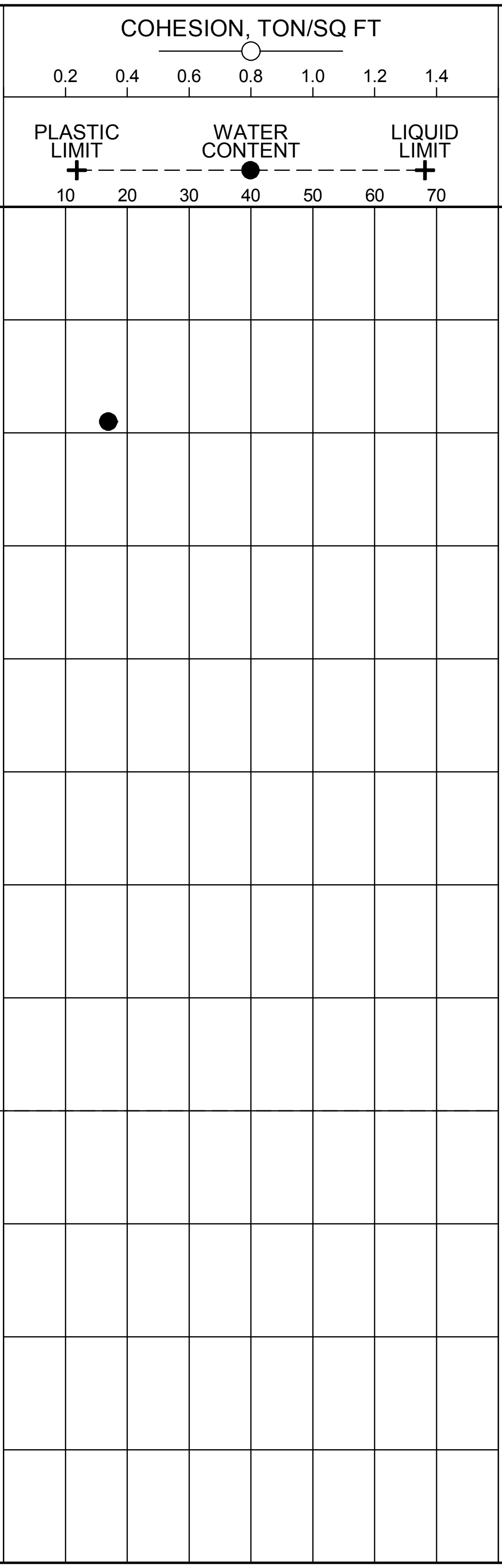
LOG OF BORING NO. 8

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1888+79, 60 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Dense to very dense tan and gray sandy fine to coarse gravel			
65				
70		70/9"		4
75				
80		90/9"		
85				
90		70/8"		
95				
100		70/8"		
105				
110				
115				



COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

LGBNEW 13-017.GPJ 4-7-14

13-017

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

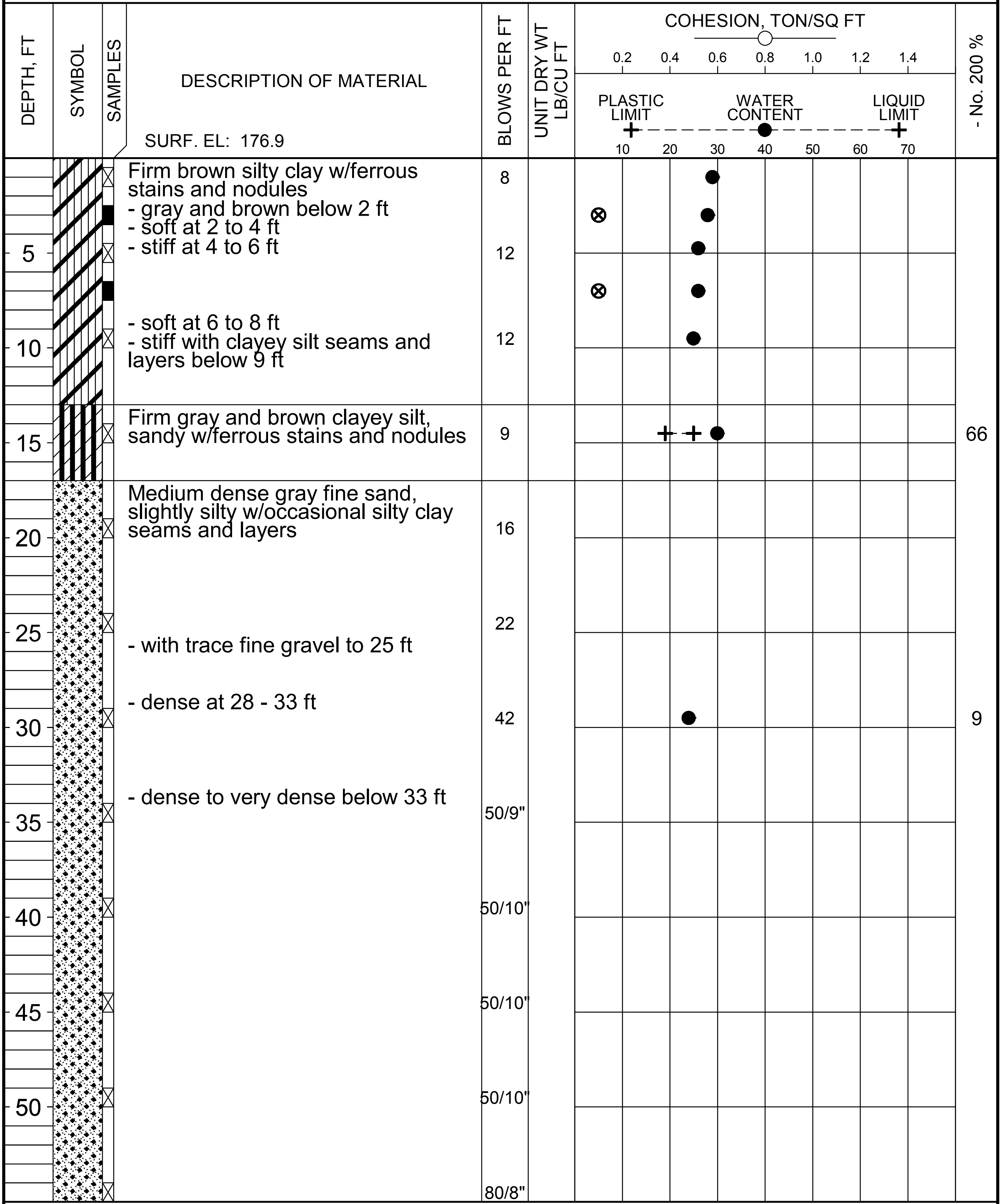
L O G O F B O R I N G N O. 9

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1889+99, 60 ft Rt

SURF. EL: 176.9



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm brown silty clay w/ferrous stains and nodules - gray and brown below 2 ft - soft at 2 to 4 ft - stiff at 4 to 6 ft - soft at 6 to 8 ft - stiff with clayey silt seams and layers below 9 ft	8 12 12		
Firm gray and brown clayey silt, sandy w/ferrous stains and nodules	9		66
Medium dense gray fine sand, slightly silty w/occasional silty clay seams and layers - with trace fine gravel to 25 ft - dense at 28 - 33 ft - dense to very dense below 33 ft	16 22 42 50/9" 50/10" 50/10" 50/10" 80/8"		9

COMPLETION DEPTH: 100.0 ft
DATE: 3-10-14

DEPTH TO WATER
IN BORING: 9 ft

DATE: 3/10/2014

LGBNEW 13-017.GPJ 4-7-14

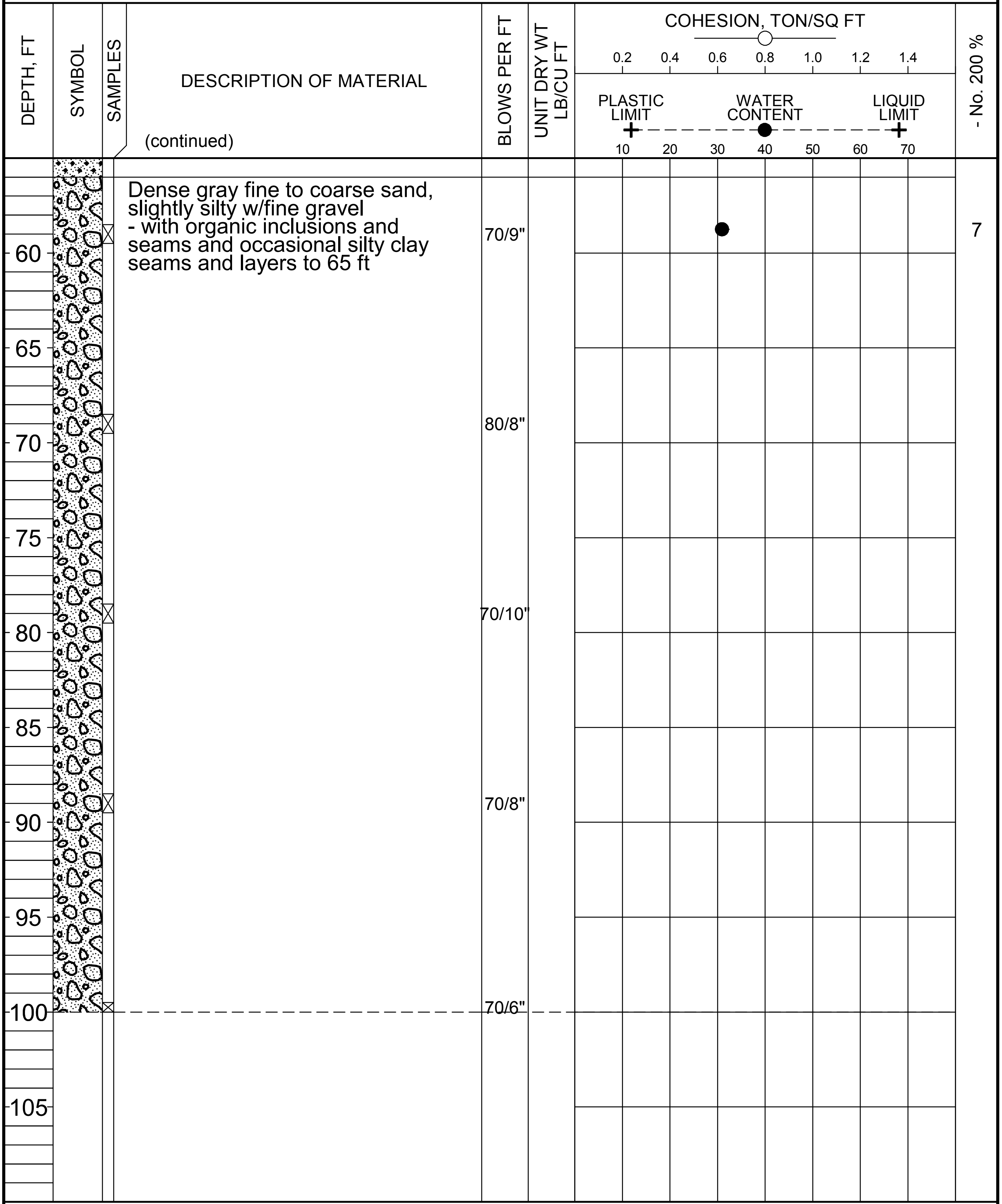
Grubbs, Hoskyn,
Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 9

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1889+99, 60 ft Rt



DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Dense gray fine to coarse sand, slightly silty w/fine gravel - with organic inclusions and seams and occasional silty clay seams and layers to 65 ft	70/9"		7
	80/8"		
	70/10"		
	70/8"		
	70/6"		

COMPLETION DEPTH: 100.0 ft
DATE: 3-10-14

DEPTH TO WATER
IN BORING: 9 ft

DATE: 3/10/2014

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

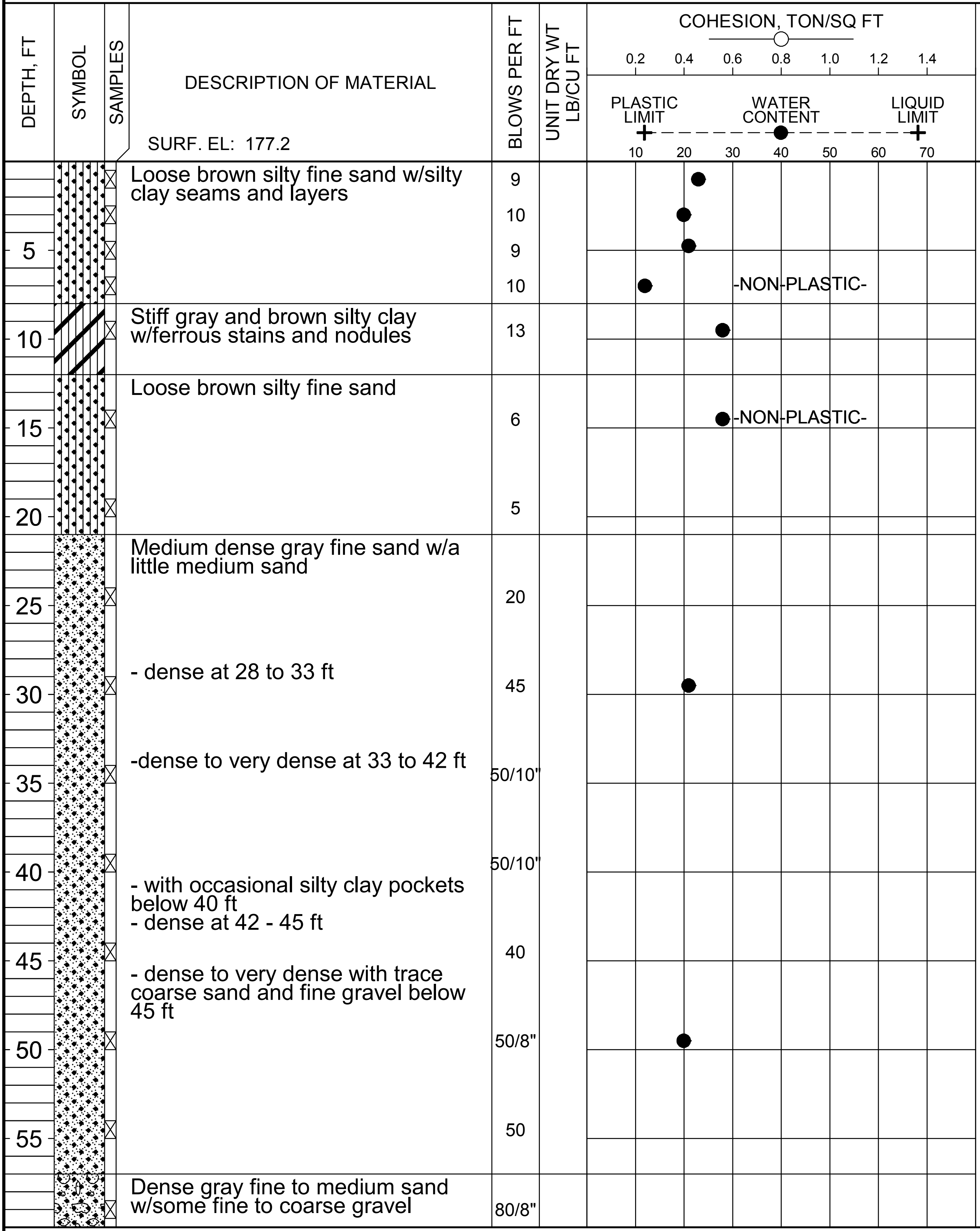
LOG OF BORING NO. 10

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1891+47, 60 ft Lt

SURF. EL: 177.2

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Loose brown silty fine sand w/silty clay seams and layers	9		
	10		
	9		
	10		20
Stiff gray and brown silty clay w/ferrous stains and nodules	13		
Loose brown silty fine sand	6		38
	5		
Medium dense gray fine sand w/a little medium sand	20		
- dense at 28 to 33 ft	45		4
-dense to very dense at 33 to 42 ft	50/10"		
	50/10"		
- with occasional silty clay pockets below 40 ft - dense at 42 - 45 ft	40		
- dense to very dense with trace coarse sand and fine gravel below 45 ft	50/8"		4
	50		
Dense gray fine to medium sand w/some fine to coarse gravel	80/8"		



COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 3/12/2014

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

LOG OF BORING NO. 10

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1891+47, 60 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2 0.4 0.6 0.8 1.0 1.2 1.4); PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT (10 20 30 40 50 60 70)	- No. 200 %
65					
70		80/8"			
	Dense gray sandy fine to coarse gravel				
75					
80		80/7"			
85					
90		80/7"			
95					
100		80/7"			
105					
110					
115					

COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 3/12/2014

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

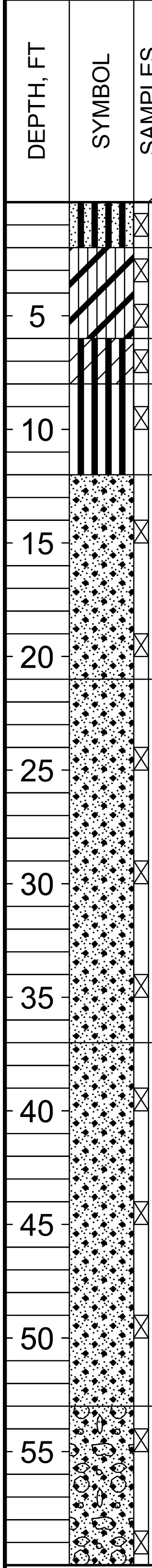
LOG OF BORING NO. 11

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

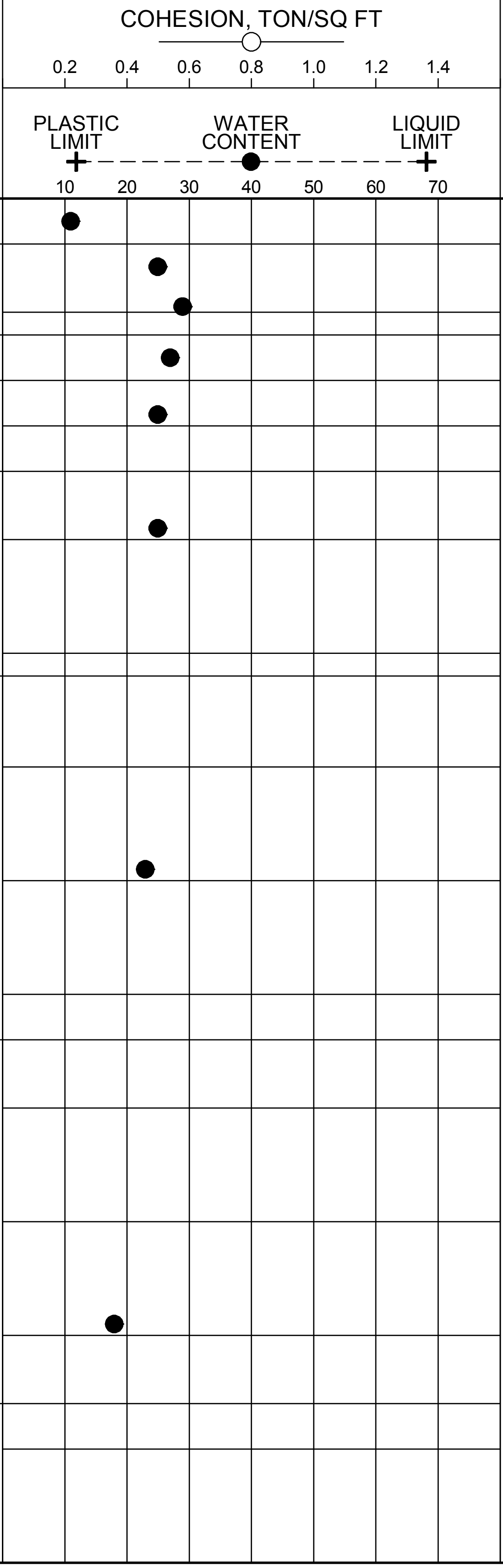
TYPE: Auger to10 ft /Wash

LOCATION: Sta 1892+95, 80 ft Rt

SURF. EL: 167.5



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Loose brown fine sandy silt w/occasional clay pockets (fill)	8		
Firm gray and brown silty clay w/ferrous stains and nodules	9		
	8		
Soft brown clayey silt w/occasional silty clay pockets	6		
Loose brown silt w/occasional clay pockets	7		
Medium dense reddish brown fine sand, slightly silty	26		5
	25		
Medium dense gray fine sand	20		
- dense at 27 to 33 ft	40		2
- dense to very dense below 33 ft	50/11"		
Dense to very dense gray fine to medium sand	50/9"		
	50/8"		
	70/8"		2
Dense to very dense gray fine to coarse sand w/fine to coarse gravel and trace clay pockets	70/8"		
	80/8"		



COMPLETION DEPTH: 100.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/13/2014

13-017

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

LOG OF BORING NO. 11

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to10 ft /Wash
LOCATION: Sta 1892+95, 80 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
65							
		☒		80/7"			
70							
75							
		☒		80/8"			
80							
85							
		☒		80/10"			
90							
95			Dense gray sandy fine to coarse gravel				
100		☒		80/6"			
105							
110							
115							

LGBNEW 13-017.GPJ 4-7-14

COMPLETION DEPTH: 100.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/13/2014

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

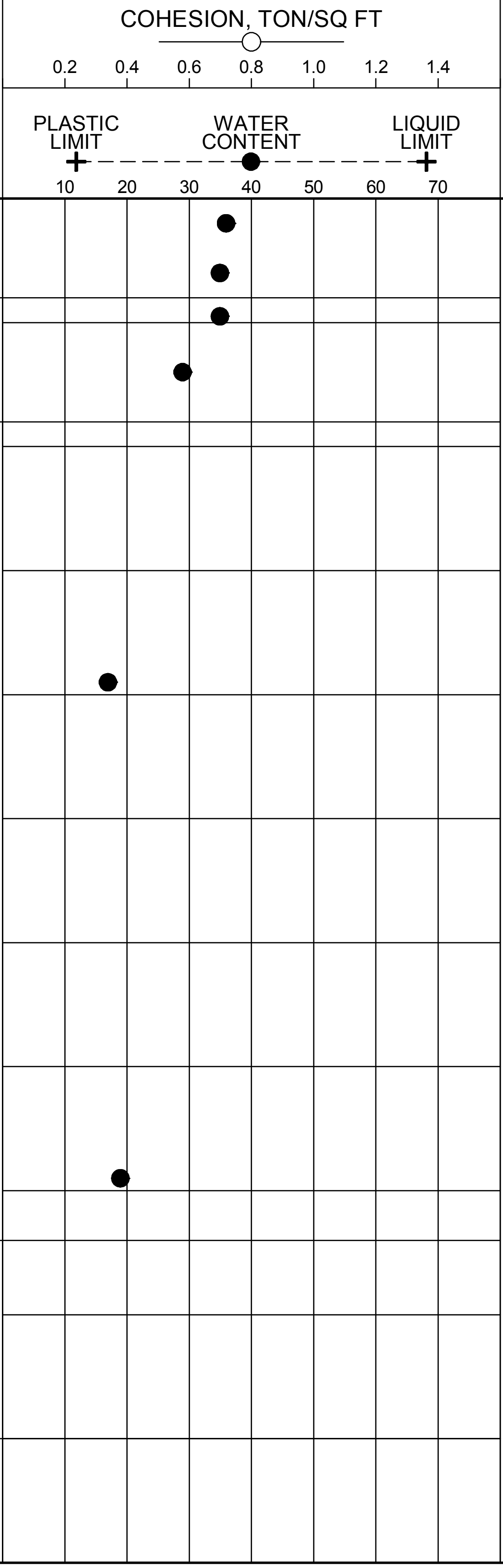
LOG OF BORING NO. 12

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1894+15, 60 ft Lt

SURF. EL: 168

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very soft gray and brown silty clay w/occasional silt pockets and seams	2		
		2		
5	Very soft brown clayey silt w/silty clay seams	3		
		3		
10	Medium dense brown and gray fine sand - gray below 13 ft	18		
15		18		
20		29		2
25		29		
30	- with occasional organic inclusions to 35 ft - dense at 28 to 34 ft	46		
35	- dense to very dense with some medium sand below 34 ft	55		
40		50/11"		3
45	Dense gray silty fine sand w/trace medium sand and occasional silty clay seams	32		
50	- with wood fragments at 50 ft	45		16
	Dense to very dense gray sandy fine gravel w/occasional fine to medium sand seams and layers	50/11"		



COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

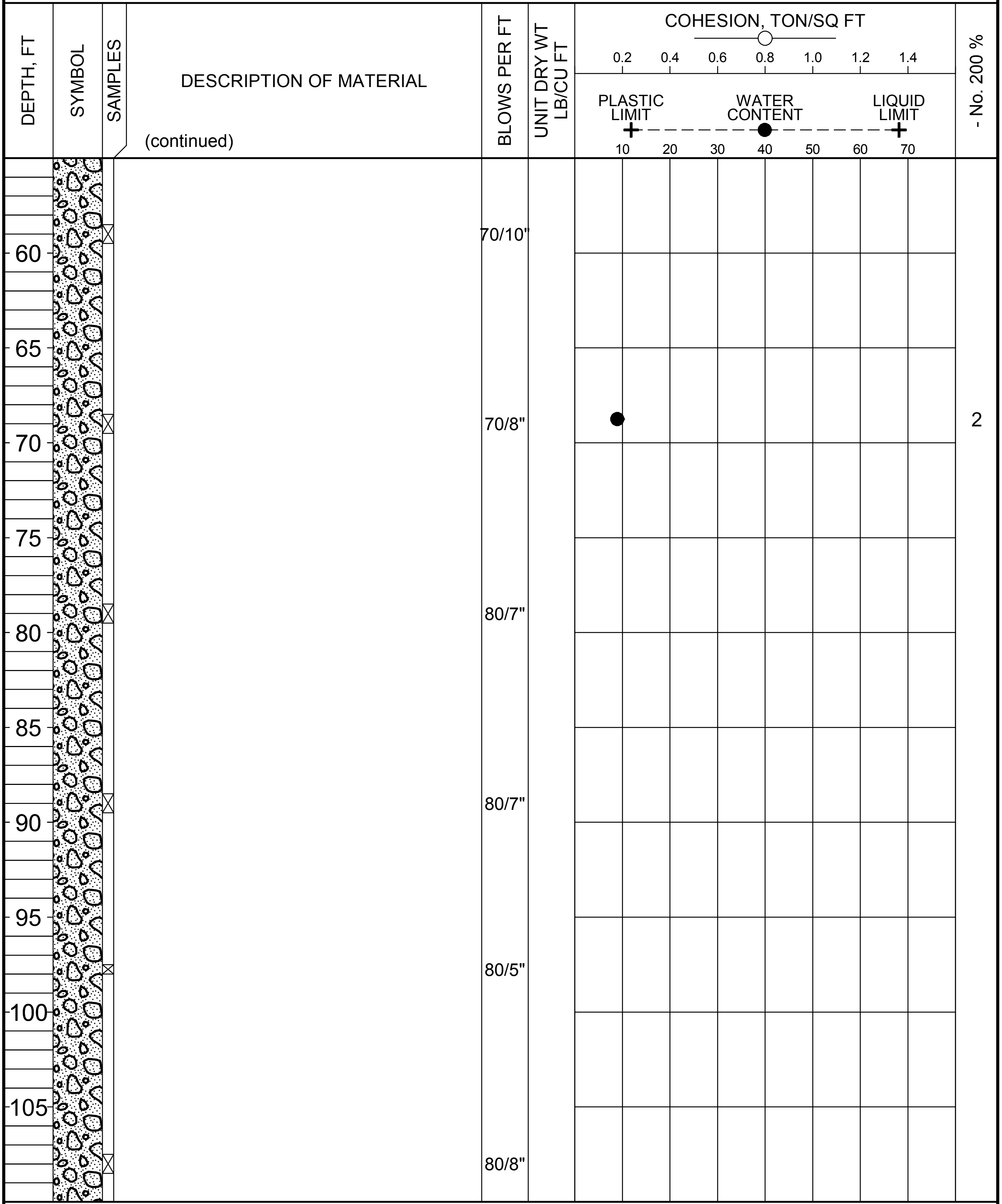
DATE: 3/13/2014

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

LOG OF BORING NO. 12

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1894+15, 60 ft Lt



COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

DATE: 3/13/2014

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

LOG OF BORING NO. 12

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1894+15, 60 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
			- borehole caving at 110 ft				
115							
120							
125							
130							
135							
140							
145							
150							
155							
160							

COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

DATE: 3/13/2014

13-017

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

LOG OF BORING NO. 13

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+90, 60 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 137±	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ (0.2 0.4 0.6 0.8 1.0 1.2 1.4) PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + (10 20 30 40 50 60 70)	- No. 200 %
	Medium dense brown and gray silty fine sand w/a little medium sand	15			
5					
		17		● -NON-PLASTIC- G_s = 2.66	31
		15			
10		16			
15	- dense at 15 to 20 ft				
		30			
20	- dense to very dense with some fine to coarse gravel below 20 ft				
		75			
25					
30					
		63			
35					
		100/11"			
40	Dense to very dense brownish gray fine to medium sand w/some fine to coarse gravel				
		75			

COMPLETION DEPTH: 78.0 ft
DATE: 2-28-14

DEPTH TO WATER
IN BORING: NA

DATE: 2/28/2014

LGBNEW 13-017.GPJ 4-8-14

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

LOG OF BORING NO. 13

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+90, 60 ft Rt

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION / WATER CONTENT	- No. 200 %
		☒		73			
50							
		☒		75			
55							
		☒		100/6"			
60			Dense to very dense gray fine to coarse sand w/some fine to coarse gravel				
65							
		☒		100/5"			
70							
75			- brown and gray below 75 ft				
		☒	- 100% water loss at 78 ft	100/5"			
80			NOTE 1: Water depth: 27 ft. NOTE 2: Set 30 ft casing to 3 ft below mudline. NOTE 3: Boring caving. Abandoned at 78 ft				
85							

COMPLETION DEPTH: 78.0 ft
DATE: 2-28-14

DEPTH TO WATER
IN BORING: NA

DATE: 2/28/2014

13-017

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

LOG OF BORING NO. 13A

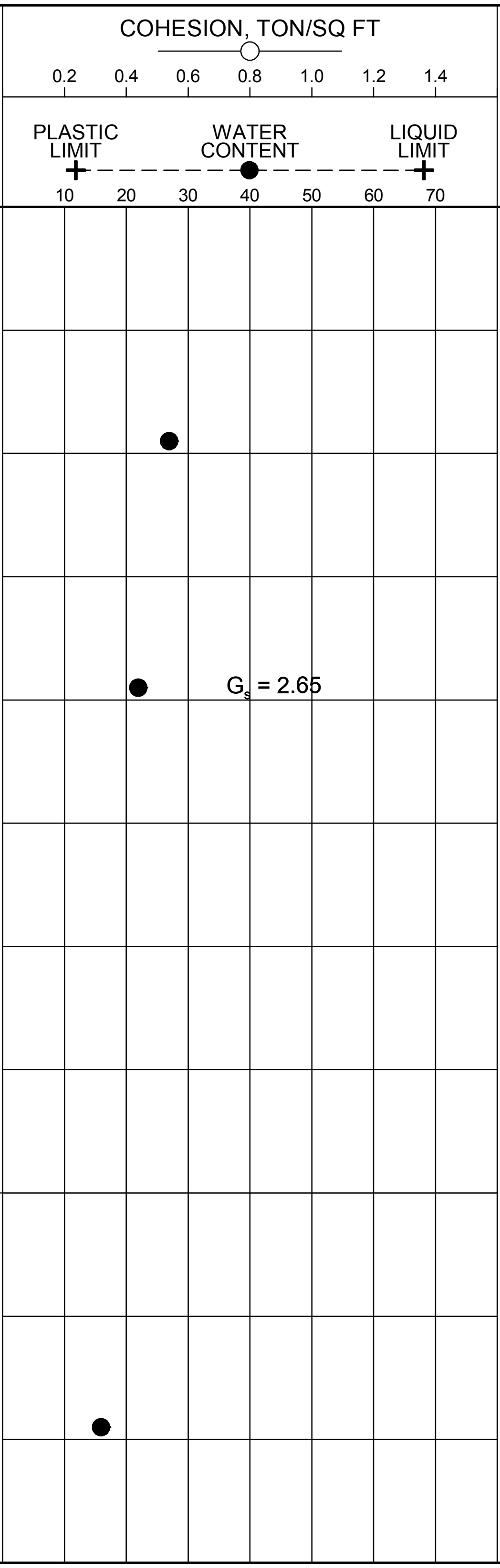
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt

SURF. EL: 137±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Dense brown and tan fine sand w/a little medium sand	50/8"		
5				
10		45		2
15				
20		36		4
25				
30	- dense to very dense with some gravel below 30 ft	50/10"		
35				
		100/7"		
40	Dense to very dense gray fine to coarse sand, slightly silty w/fine gravel			
45				
50		100/8"		6
		50/8"		



COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

LGBNEW 13-017.GPJ 4-8-14

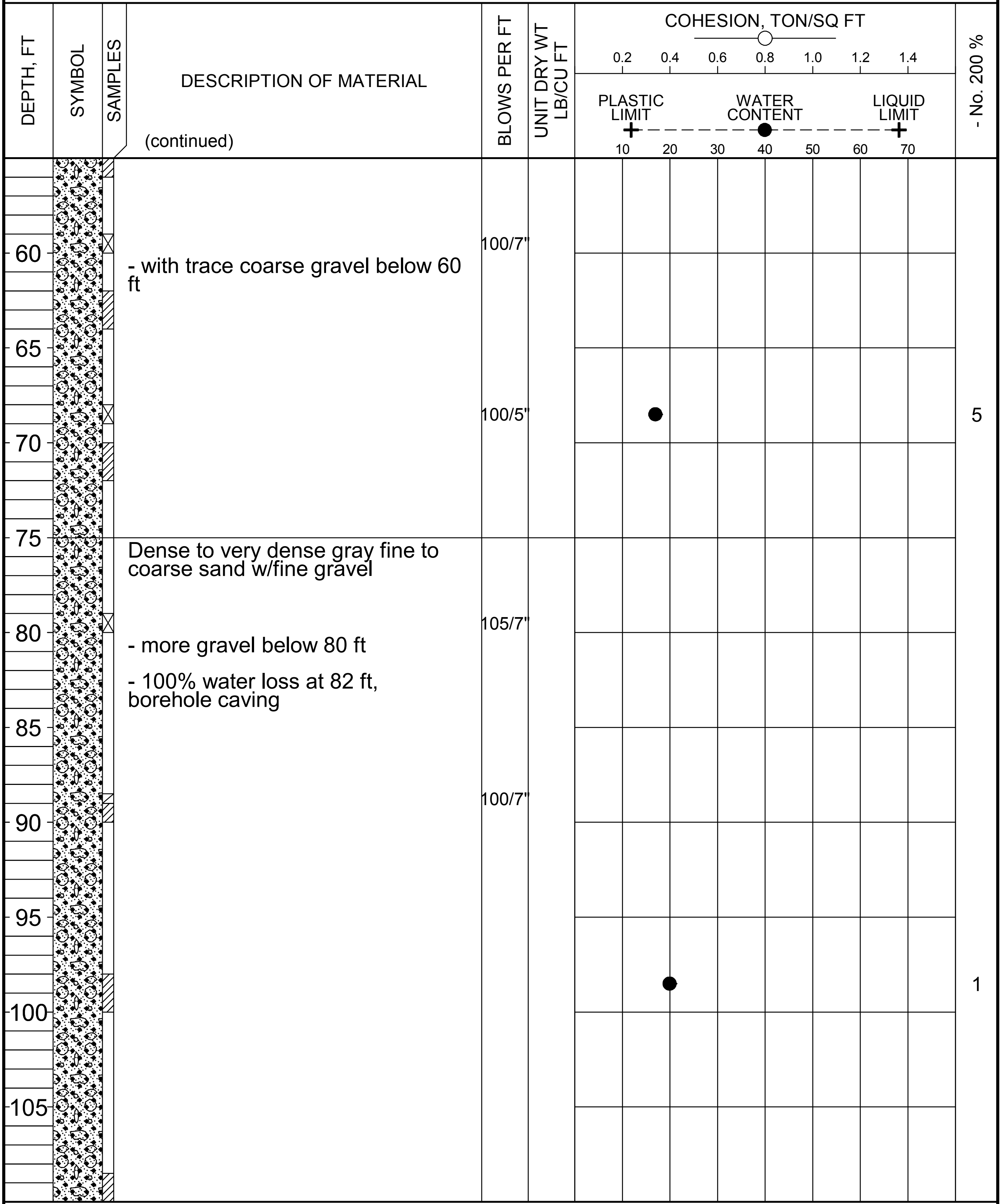
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 13A

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt



DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
	100/7"	
- with trace coarse gravel below 60 ft		
	100/5"	5
Dense to very dense gray fine to coarse sand w/fine gravel		
	105/7"	
- more gravel below 80 ft		
- 100% water loss at 82 ft, borehole caving		
	100/7"	
		1

COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

LGBNEW 13-017.GPJ 4-8-14

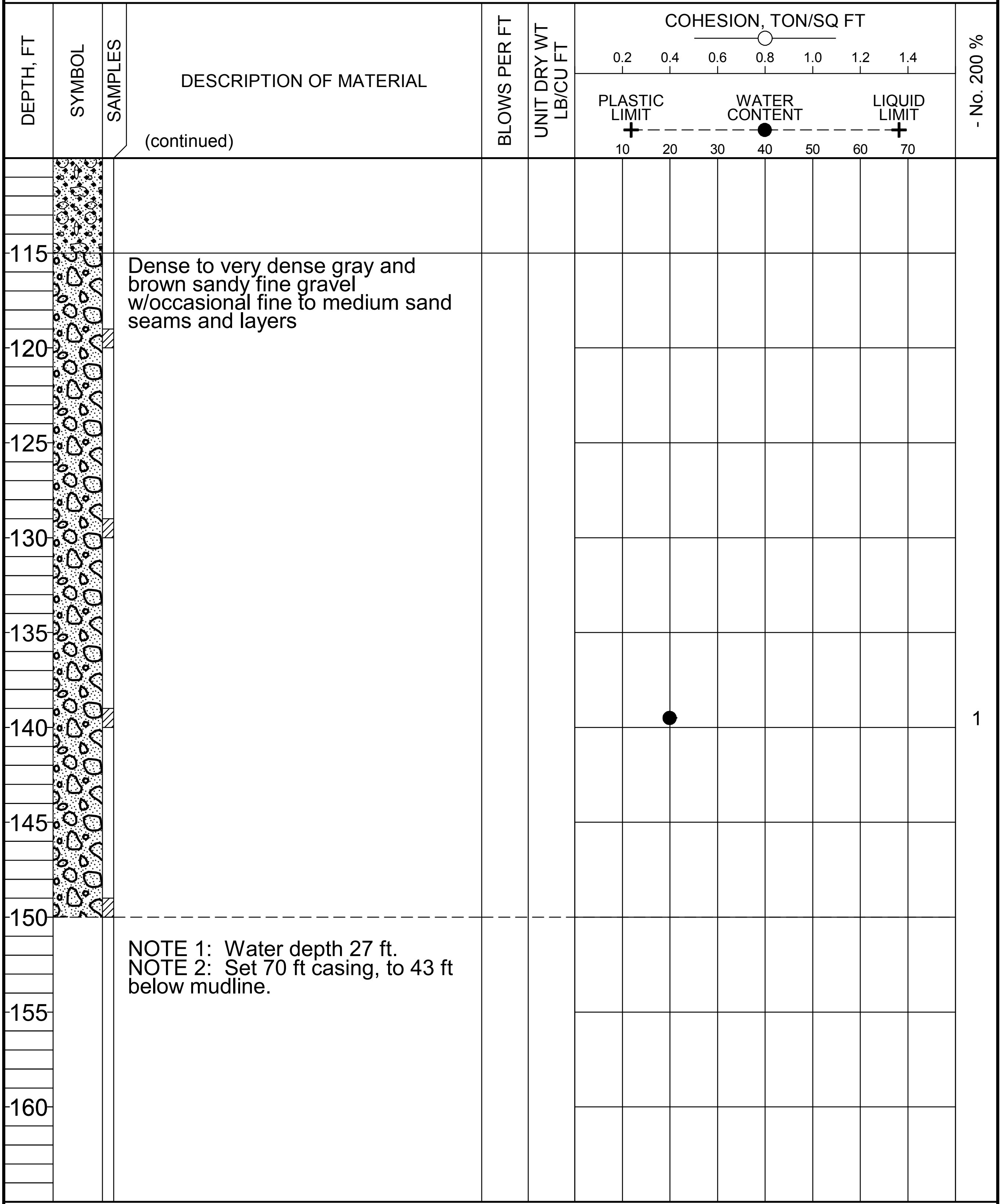
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 13A

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	- No. 200 %
115	Dense to very dense gray and brown sandy fine gravel w/occasional fine to medium sand seams and layers	
140		1
150	NOTE 1: Water depth 27 ft. NOTE 2: Set 70 ft casing, to 43 ft below mudline.	

COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

13-017

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

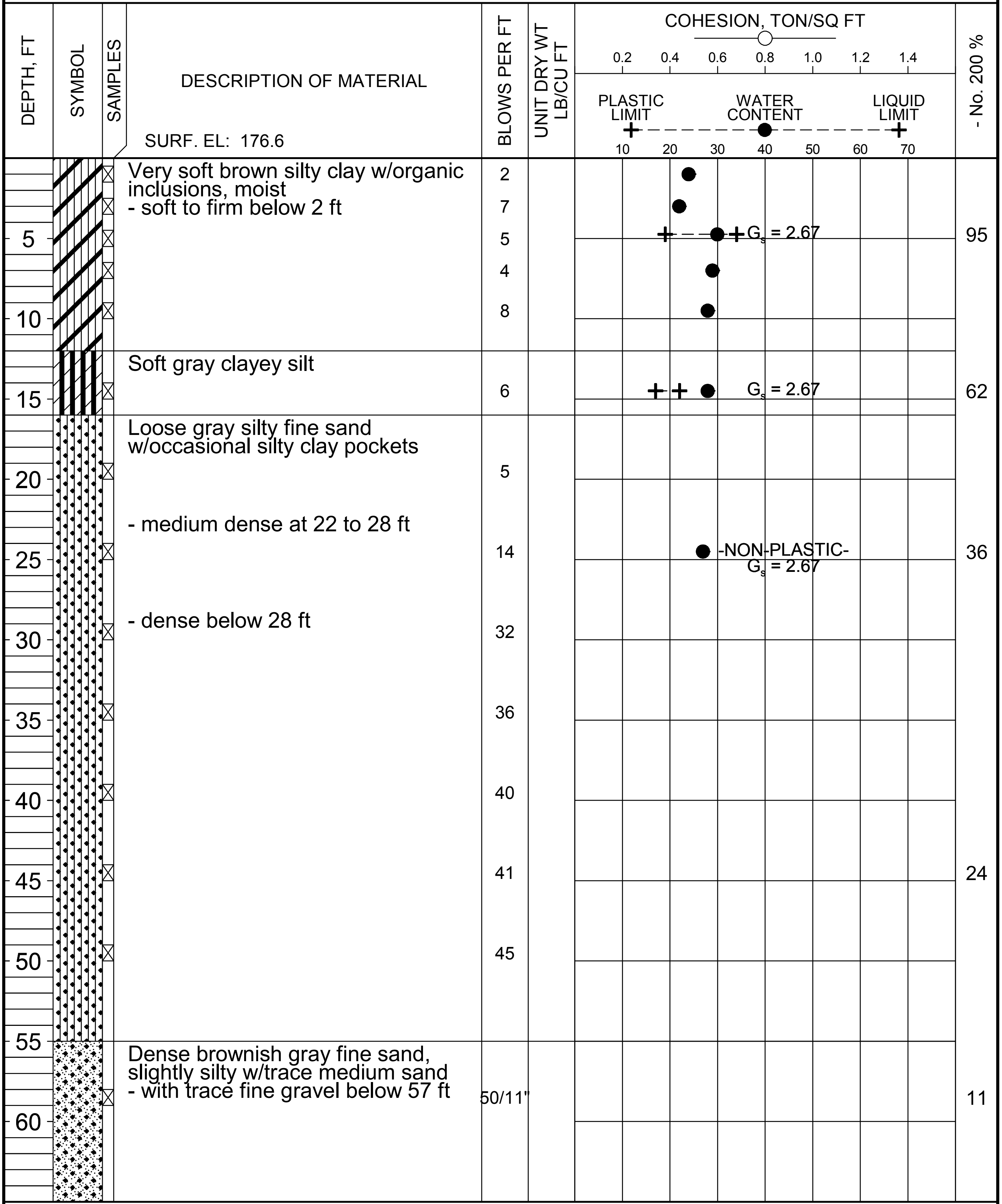
LOG OF BORING NO. 14

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1900+10, 60 ft Lt

SURF. EL: 176.6



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	- No. 200 %
Very soft brown silty clay w/organic inclusions, moist	2			
- soft to firm below 2 ft	7			
	5		G_s = 2.67	95
	4			
	8			
Soft gray clayey silt	6		G_s = 2.67	62
Loose gray silty fine sand w/occasional silty clay pockets	5			
- medium dense at 22 to 28 ft	14		-NON-PLASTIC- G_s = 2.67	36
- dense below 28 ft	32			
	36			
	40			
	41			24
	45			
Dense brownish gray fine sand, slightly silty w/trace medium sand - with trace fine gravel below 57 ft	50/11"			11

COMPLETION DEPTH: 125.0 ft
DATE: 2-13-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/13/2014

LGBNEW 13-017.GPJ 4-8-14

13-017

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

LOG OF BORING NO. 14

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1900+10, 60 ft Lt

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION / WATER CONTENT	- No. 200 %
		☒		50/11"			
70			Dense to very dense brown sandy fine gravel				
75							
		☒		50/10"			
80							
85							
		☒		50/6"			
90							
95							
		☒		50/7"			
							2
100							
105							
		☒		50/6"			
110							
115							
		☒		50/4"			
120							
125							

COMPLETION DEPTH: 125.0 ft
DATE: 2-13-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/13/2014

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

LOG OF BORING NO. 15

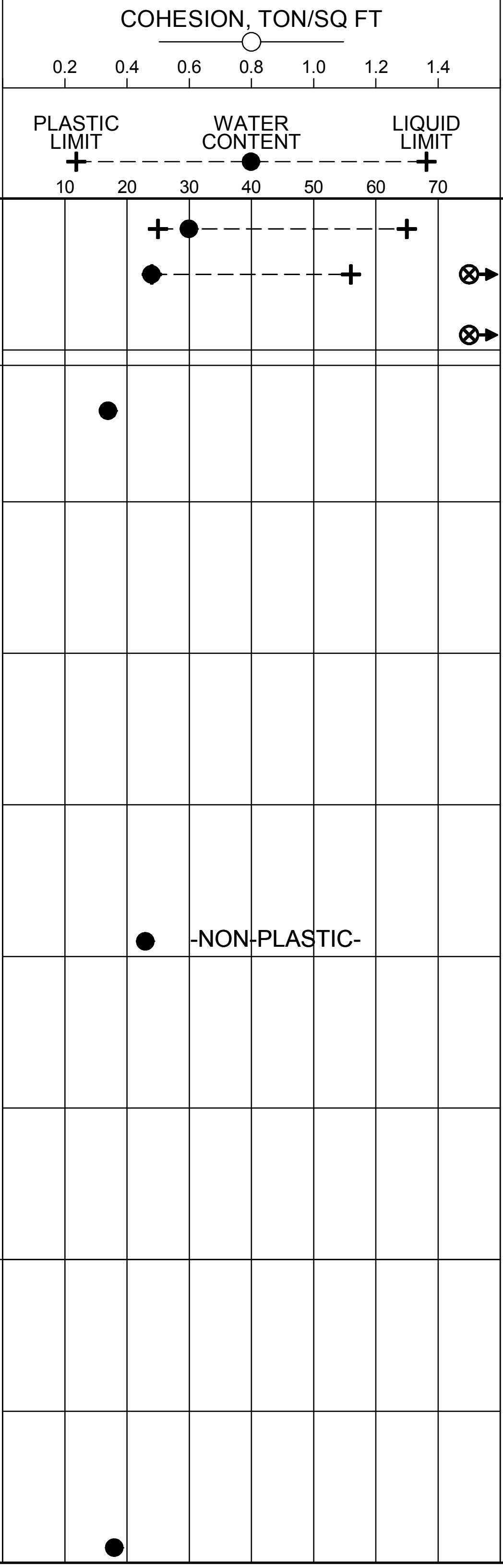
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1916+00, CL

SURF. EL: 180.1

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brown clay	14		99
	- very stiff below 2 ft			94
5	- with fine sandy silt pockets below 4 ft			
	Medium dense brown silty fine sand	23		
10	- loose to medium dense at 8 - 12 ft	10		
	- medium dense at 12 - 18 ft			
15		18		
20	- wet at 9 - 18 ft - dense at 18 - 28 ft	36		
25	- with silty clay pockets below 24 ft	42		12
30	- dense to very dense at 28 - 32 ft	50/8"		
35	- dense, grayish brown, wet below 32 ft	35		
	Dense brown and gray fine to medium sand, slightly silty w/silty clay pockets and trace fine gravel			
40		45		
		38		9



COMPLETION DEPTH: 70.0 ft
DATE: 6-24-13

DEPTH TO WATER
IN BORING: Dry to 15 ft
DATE: 6/24/2013

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

LOG OF BORING NO. 15

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1916+00, CL

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

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

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50	- dark gray and brown with numerous organic inclusions below 48 ft	34		
	Dense to very dense gray fine sand, slightly silty			
55	- dense, coarser below 55 ft	50/9"		
60		37		6
65	Dense to very dense brown and gray fine to medium sand, slightly silty	50/8"		
70		50/10"		5
75				
80				
85				

COMPLETION DEPTH: 70.0 ft
DATE: 6-24-13

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 6/24/2013

13-017

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

LOG OF BORING NO. 16

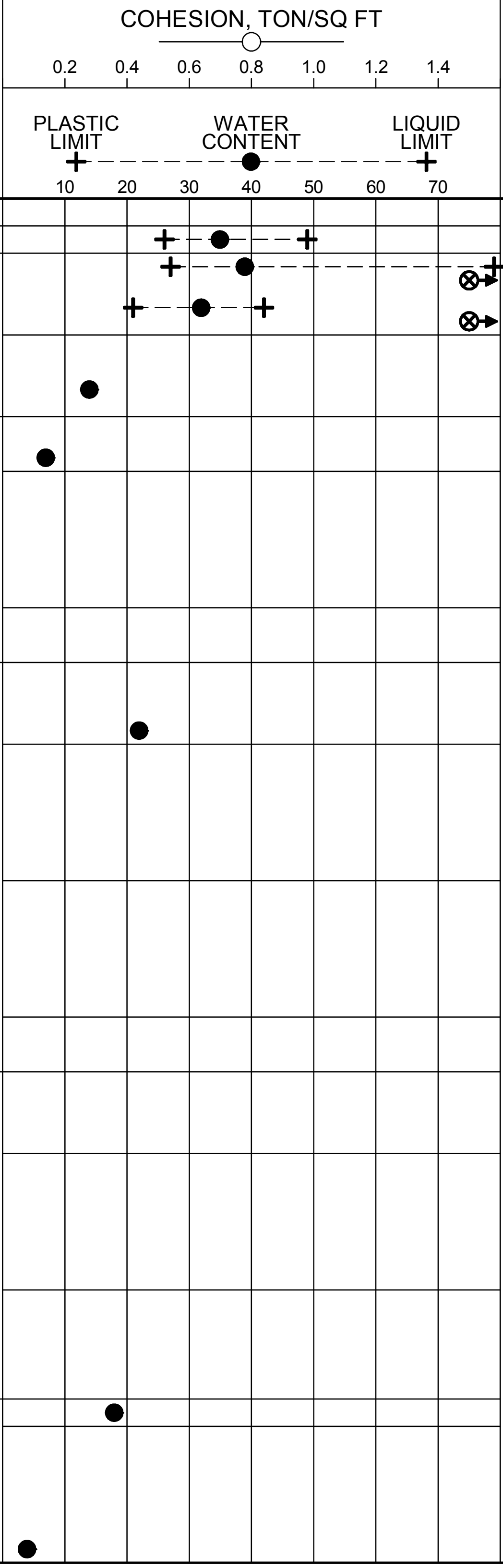
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1917+80, CL

SURF. EL: 185.1

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft dark brown clayey silt w/surface organics	3		86
	Very soft grayish brown silty clay			99
5	Very stiff brown clay and silty clay w/silt pockets			96
		42		
10	Medium dense brown silty fine sand	20		29
15		13		
20	Dense brown fine sand, slightly silty	34		9
25	- damp at 22 - 32 ft - medium dense at 22 - 26 ft	20		
	- dense below 26 ft			
30		38		
35	Loose dark gray and brown silty fine sand w/numerous organic inclusions	9		
40	- medium dense, wet below 38 ft	14		20
45	Dense brown and gray fine to medium sand, slightly silty	36		
		38		



COMPLETION DEPTH: 70.0 ft
DATE: 6-19-13

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 6/19/2013

LGBNEW 13-017.GPJ 4-8-14

13-017

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

LOG OF BORING NO. 16

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash LOCATION: Sta 1917+80, CL

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT WATER CONTENT LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
		☒		30			
55			Dense gray and brown silty fine sand, wet				
60		☒		47			13
			- dense to very dense below 63 ft				
65		☒		50/9"			
			Dense to very dense brown and gray fine to medium sand w/trace fine gravel				
70		☒		50/10"			4
75							
80							
85							
90							
95							

COMPLETION DEPTH: 70.0 ft
DATE: 6-19-13

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 6/19/2013

LGBNEW 13-017.GPJ 4-8-14

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

LOG OF BORING NO. 17

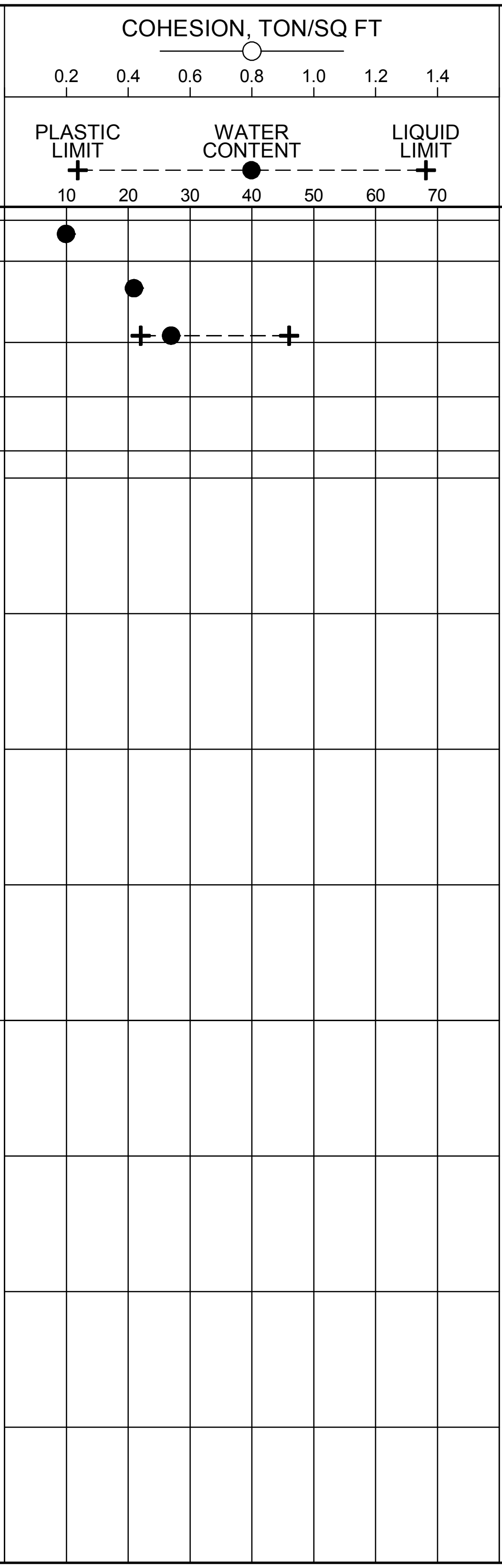
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1915+40, 10 ft Rt

SURF. EL: 178.4

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very stiff brown silty clay	16		
	Medium dense brown silty fine sand	14		81
5	Stiff dark brown silty clay w/organic stains - light gray and tan below 4.5 ft	15		88
	Medium dense brown silty fine sand	11		
10	Loose brown and tan fine sand, slightly silty	7		
15	- medium dense at 12 - 18 ft	13		
20	- dense at 18 to 23 ft	35		
25	- dense to very dense at 23 to 28 ft	50/11"		
30	- medium dense below 28 ft	16		9
35	Dense gray and brown fine to medium sand, slightly silty	39		
40	- medium dense below 38 ft	28		
45	- gray below 42 ft	19		6
	- gray and dark gray with organic inclusions below 47 ft	16		



COMPLETION DEPTH: 88.0 ft
DATE: 6-26-13

DEPTH TO WATER
IN BORING: 8 ft

DATE: 6/26/2013

13-017

Grubbs, Hoskyn, Barton & Wyatt, Inc.
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L O G O F B O R I N G N O. 17

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1915+40, 10 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	- dense below 52 ft			
55	- with silty clay pockets and seams at 54 - 56 ft	38		17
60		50		
65	Dense gray and brown fine to medium sand	38		
	- dense to very dense below 67 ft			
70		50/8"		4
75		50/8"		
	Dense to very dense brown and gray sandy fine to coarse gravel	50/8"		
80				
		50/8"		3
85				
90				
95				

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

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

COMPLETION DEPTH: 88.0 ft
DATE: 6-26-13

DEPTH TO WATER
IN BORING: 8 ft

DATE: 6/26/2013

LGBNEW 13-017.GPJ 4-8-14

13-017

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

LOG OF BORING NO. 18

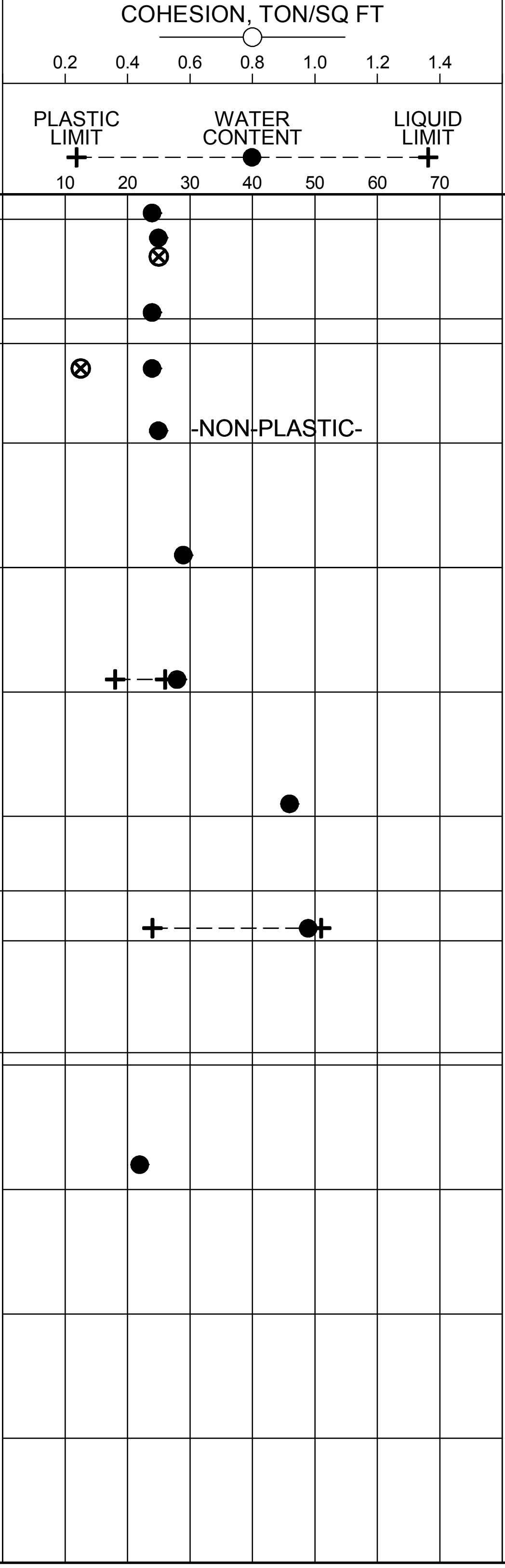
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1902+75, 60 ft Rt

SURF. EL: 177.1

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft dark brown silty clay			
	Firm to stiff brown clay	8		
5	- stiff below 4 ft	16		
	Soft brown and light brown silty clay			
10	- with interbedded fine sandy silt layers below 8.5 ft	9		51
	- firm at 9 - 10 ft			
	- soft below 12 ft			
15		4		
	Very soft brownish gray fine sandy clay, silty			
20		3		77
	- firm below 23 ft			
25		8		
	Stiff brown clay			
30		21		99
	- very stiff below 33 ft	44		
35	Dense gray silty fine sand			
	- dense to very dense below 38 ft	50/11"		20
40				
45		50/11"		
50		50/10"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-14-14

DEPTH TO WATER
IN BORING: 7 ft

DATE: 2/14/2014

LGBNEW 13-017.GPJ 4-8-14

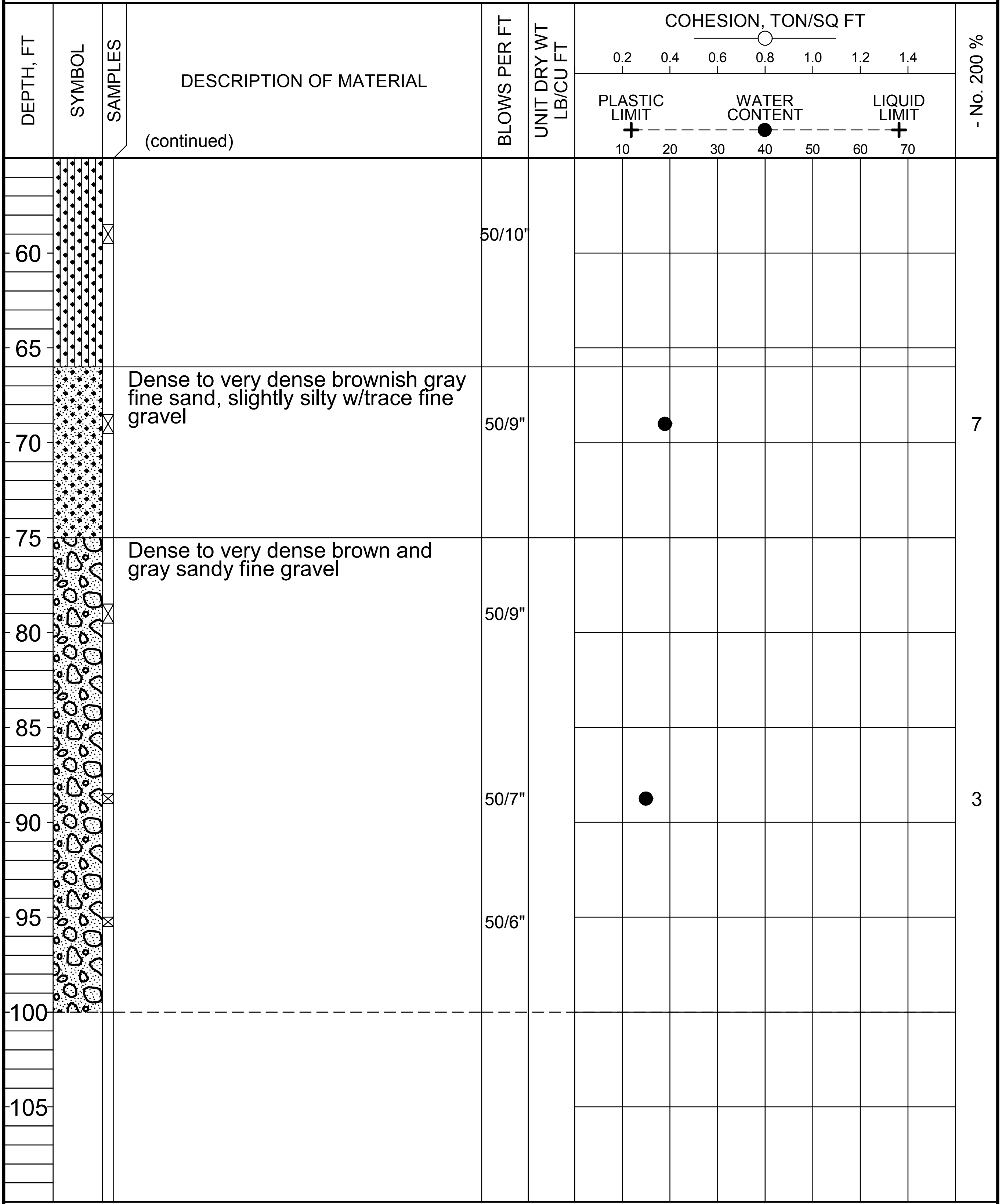
13-017

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

LOG OF BORING NO. 18

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1902+75, 60 ft Rt



Dense to very dense brownish gray fine sand, slightly silty w/trace fine gravel

Dense to very dense brown and gray sandy fine gravel

DEPTH, FT	BLOWS PER FT	- No. 200 %
59	50/10"	
69	50/9"	7
79	50/9"	
88.5	50/7"	3
95	50/6"	

COMPLETION DEPTH: 100.0 ft
DATE: 2-14-14

DEPTH TO WATER
IN BORING: 7 ft

DATE: 2/14/2014

13-017

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

LOG OF BORING NO. 19

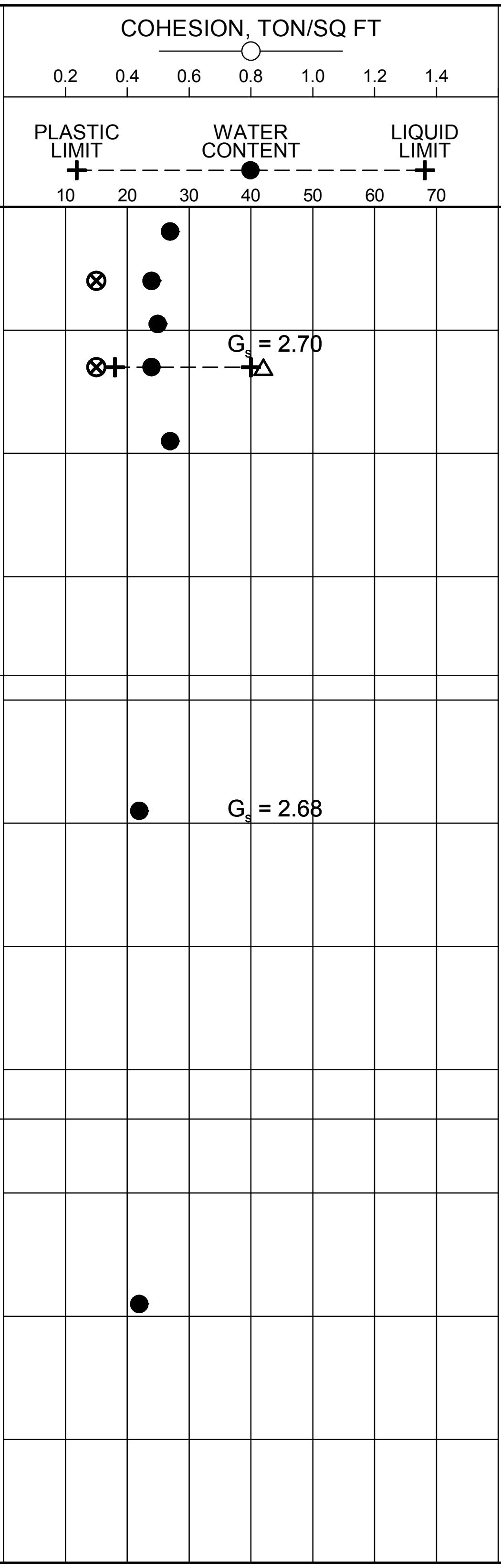
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1903+95, 60 ft Lt

SURF. EL: 174.5

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm dark brown and brown silty clay w/ferrous stains and nodules	8		
	- firm to stiff at 2 - 6 ft			
	- gray and brown below 4 ft			
5		13		
	- firm below 6 ft		102	96
10		9		
15		7		
	- gray below 15 ft			
20	Medium dense gray fine sand, slightly silty	23		
	- dense below 23 ft			
25		43		8
30		48		
35		55		
	Dense to very dense brownish gray fine sand, slightly silty			
40		50/9"		
45		50/10"		8
50		50/10"		
	- with some organic inclusions, occasional silty clay pockets and a little fine to coarse gravel below 50 ft			



COMPLETION DEPTH: 100.0 ft
DATE: 2-17-14

DEPTH TO WATER
IN BORING: 13 ft

DATE: 2/17/2014

LGBNEW 13-017.GPJ 6-13-14

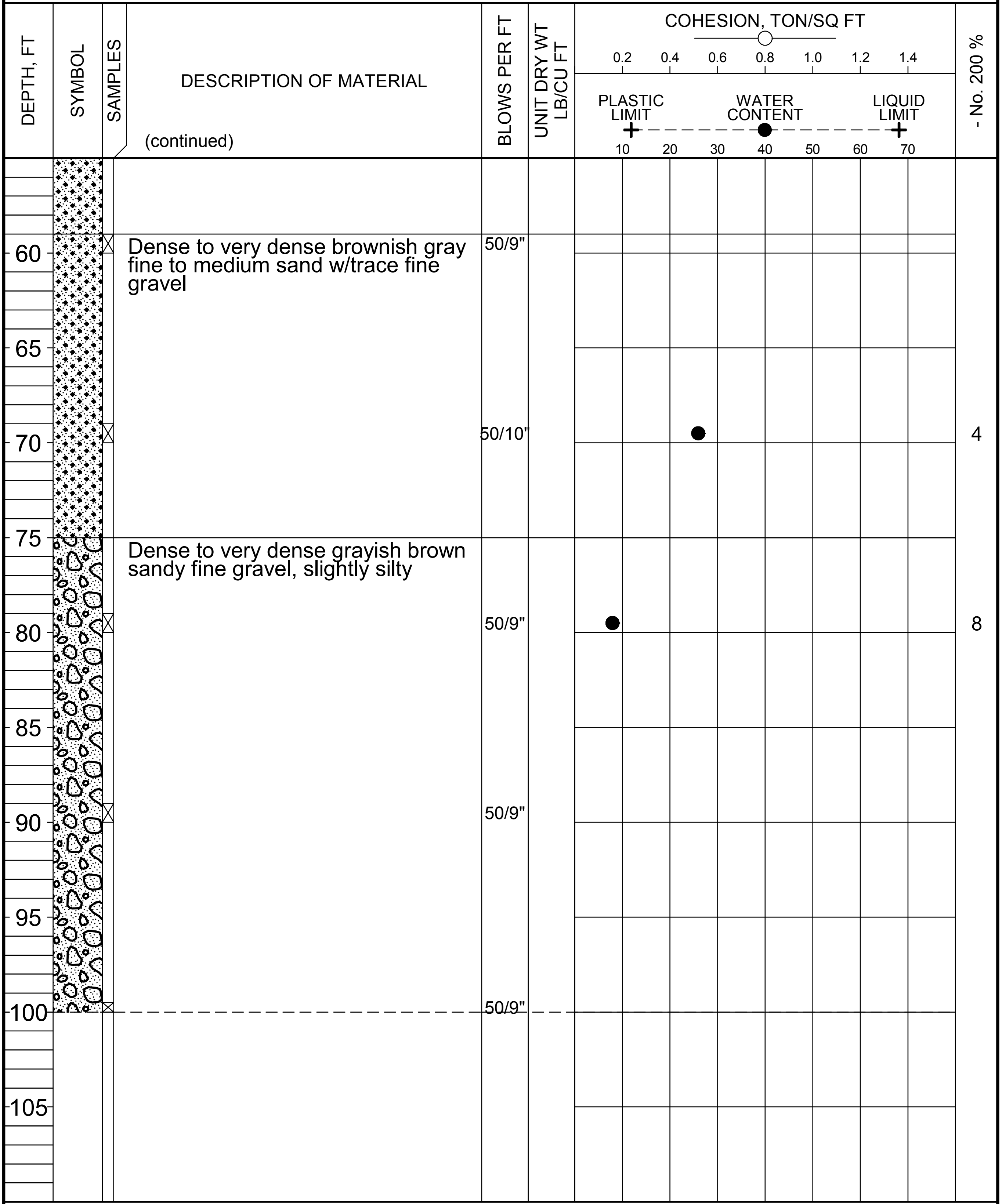
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 19

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1903+95, 60 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
60	Dense to very dense brownish gray fine to medium sand w/trace fine gravel	50/9"	
70		50/10"	4
75	Dense to very dense grayish brown sandy fine gravel, slightly silty		
80		50/9"	8
90		50/9"	
100		50/9"	

COMPLETION DEPTH: 100.0 ft
DATE: 2-17-14

DEPTH TO WATER
IN BORING: 13 ft

DATE: 2/17/2014

13-017

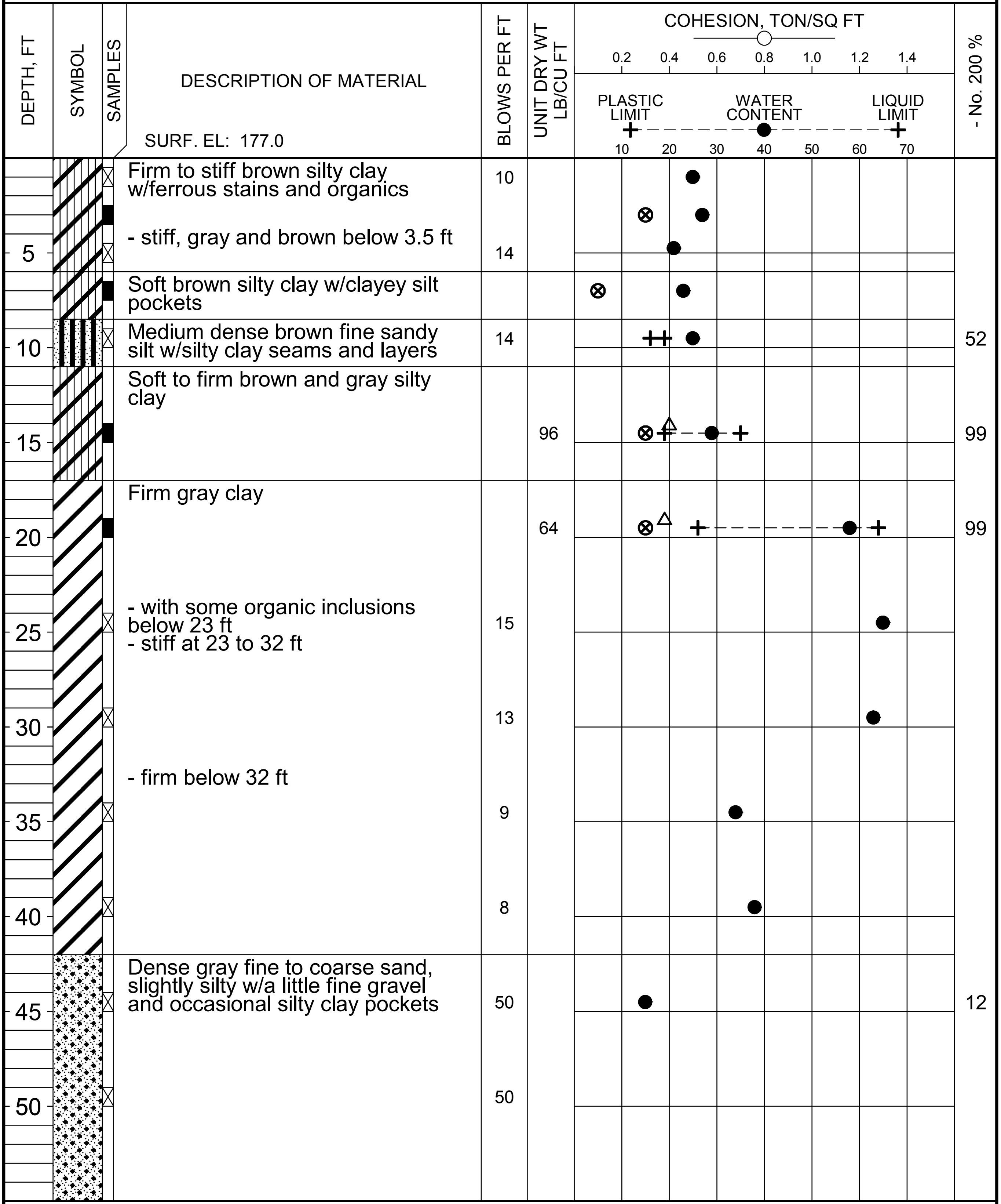
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 20

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1905+43, 60 ft Rt



DESCRIPTION OF MATERIAL (SURF. EL: 177.0)	**BLOWS PER FT**	**UNIT DRY WT LB/CU FT**	**- No. 200 %**
Firm to stiff brown silty clay w/ferrous stains and organics	10		
- stiff, gray and brown below 3.5 ft	14		
Soft brown silty clay w/clayey silt pockets			
Medium dense brown fine sandy silt w/silty clay seams and layers	14		52
Soft to firm brown and gray silty clay		96	99
Firm gray clay		64	99
- with some organic inclusions below 23 ft - stiff at 23 to 32 ft	15 13		
- firm below 32 ft	9 8		
Dense gray fine to coarse sand, slightly silty w/a little fine gravel and occasional silty clay pockets	50 50		12

COMPLETION DEPTH: 100.0 ft
DATE: 2-18-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/18/2014

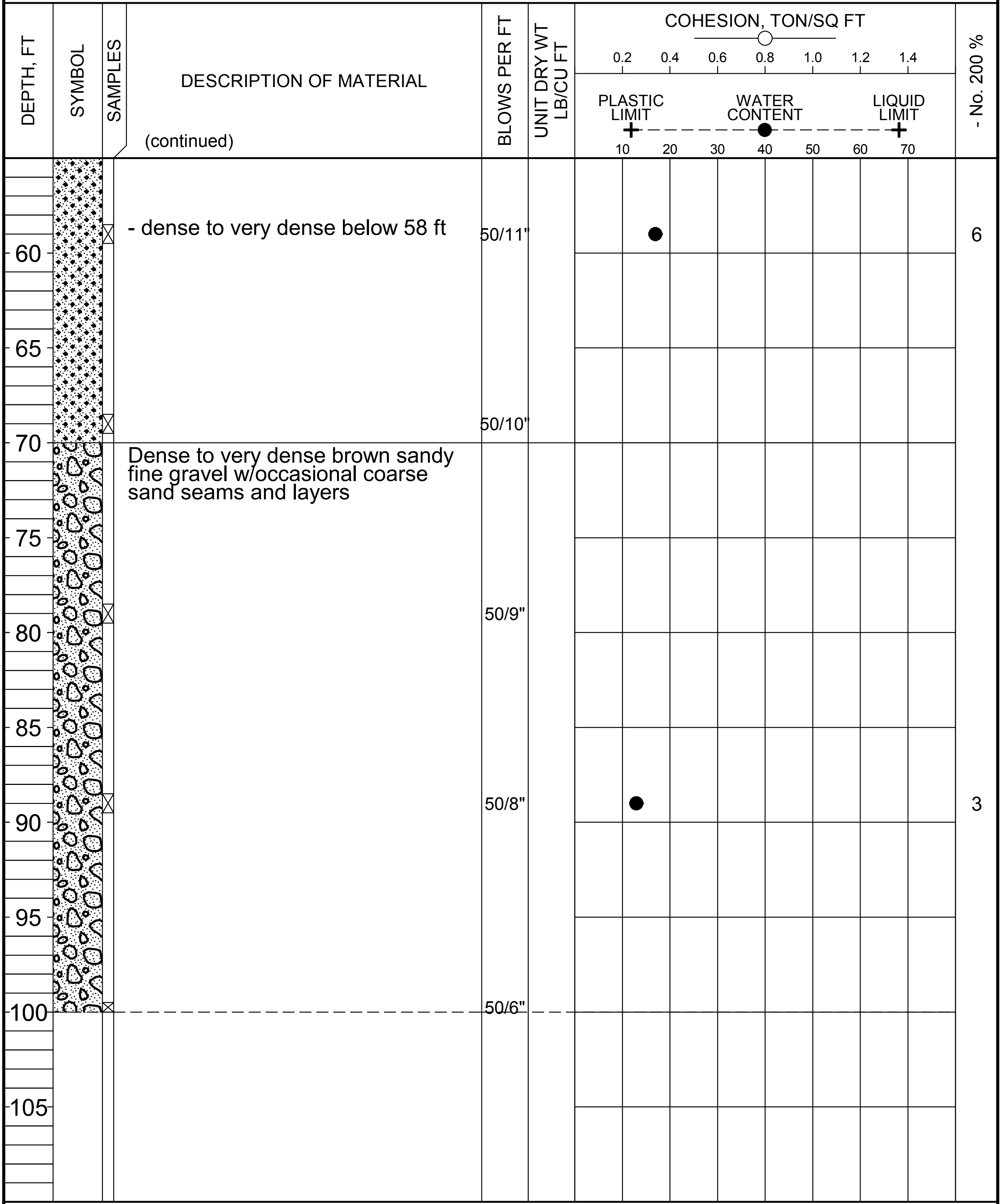
LGBNEW 13-017.GPJ 6-13-14

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

LOG OF BORING NO. 20

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1905+43, 60 ft Rt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	- No. 200 %
58	- dense to very dense below 58 ft	50/11"	6
68		50/10"	
70	Dense to very dense brown sandy fine gravel w/occasional coarse sand seams and layers		
78		50/9"	
88		50/8"	3
100		50/6"	

COMPLETION DEPTH: 100.0 ft
DATE: 2-18-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/18/2014

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

LOG OF BORING NO. 21

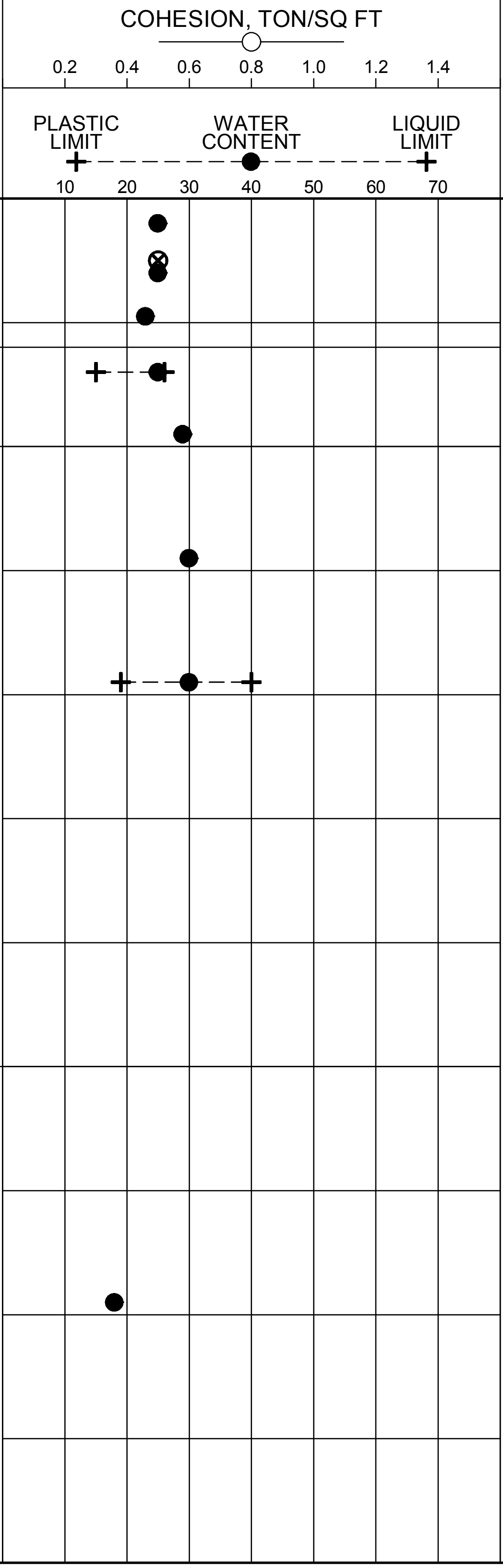
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1906+91, 60 ft Lt

SURF. EL: 175.8

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm tan and brown clay	8		
- stiff below 2 ft			
	11		
- water at 6 ft	7		46
Loose tan and brown clayey fine sand, silty			
- very loose below 8 ft	3		
Firm gray silty clay w/occasional fine sand partings and seams			
	8		
	7		97
- soft below 23 ft	6		
	5		
	4		
Medium dense tan and gray fine to coarse sand w/a little fine gravel	20		
- dense to very dense below 38 ft	50/9"		
	50		4
	50/10"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-21-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/21/2014

13-017

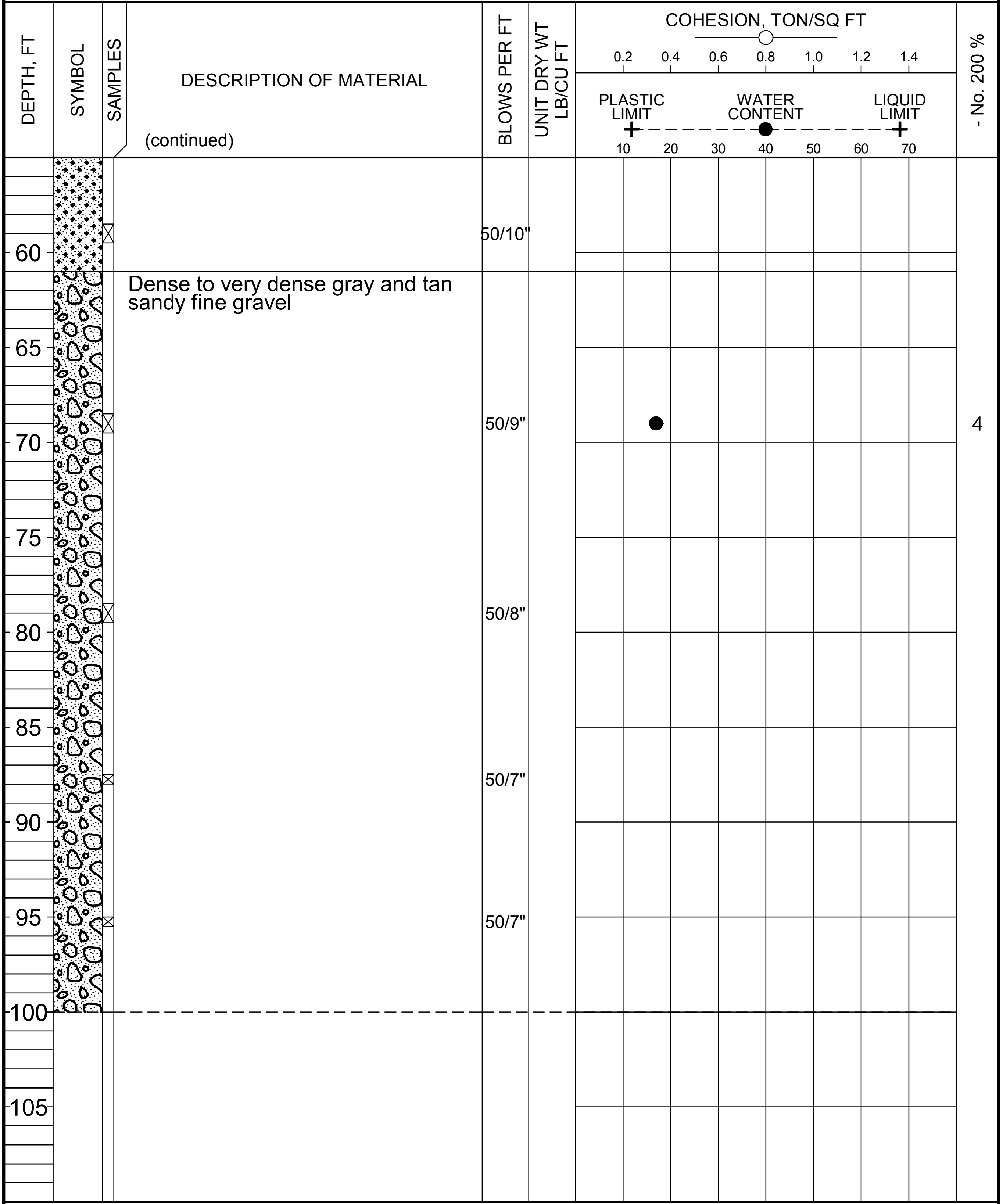
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 21

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1906+91, 60 ft Lt



DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	50/10"		
Dense to very dense gray and tan sandy fine gravel	50/9"		4
	50/8"		
	50/7"		
	50/7"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-21-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/21/2014

LGBNEW 13-017.GPJ 4-8-14

13-017

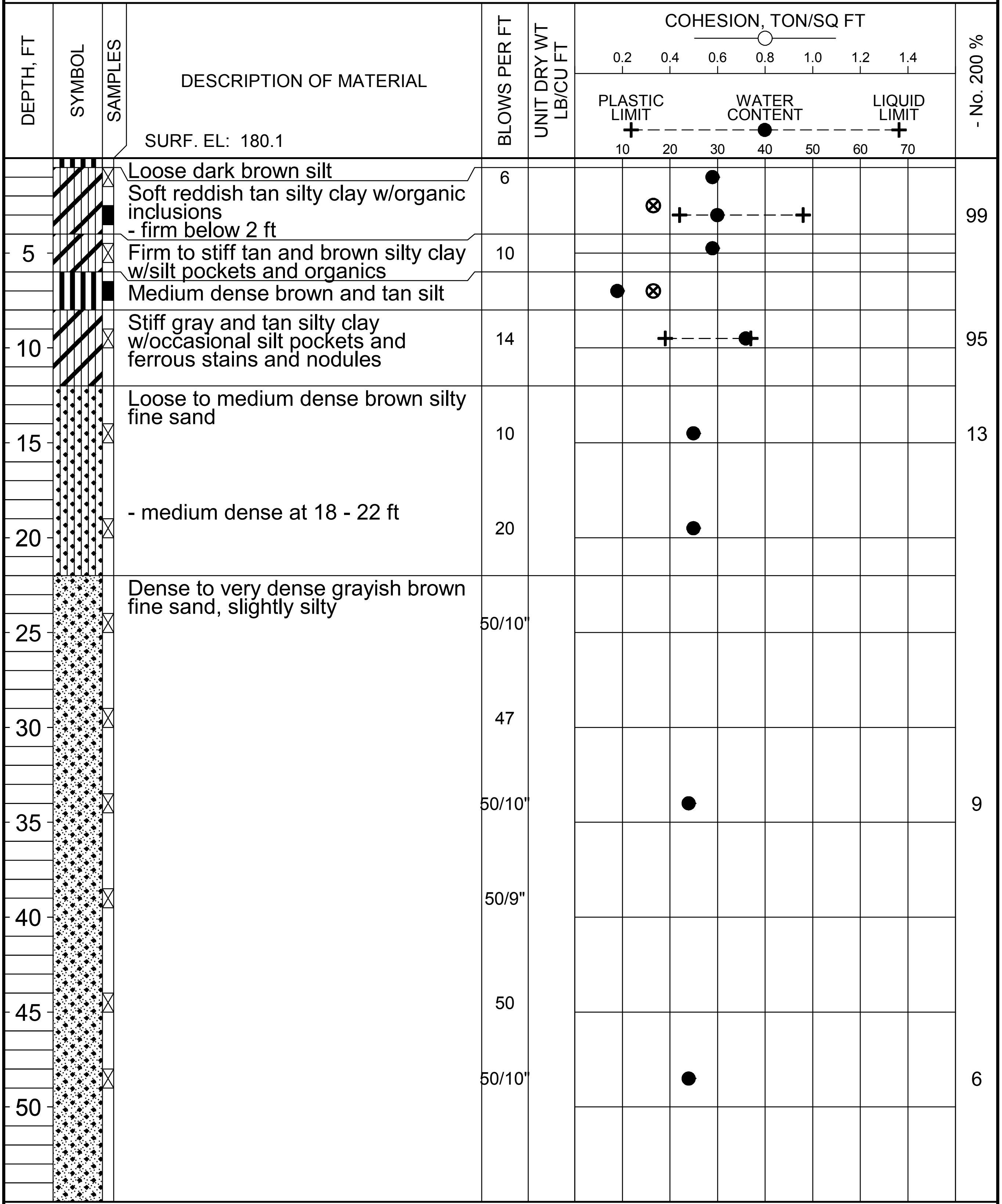
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 22

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1908+11, 60 ft Rt



SURF. EL: 180.1

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Loose dark brown silt	6		
Soft reddish tan silty clay w/organic inclusions - firm below 2 ft			99
Firm to stiff tan and brown silty clay w/silt pockets and organics	10		
Medium dense brown and tan silt			
Stiff gray and tan silty clay w/occasional silt pockets and ferrous stains and nodules	14		95
Loose to medium dense brown silty fine sand	10		13
- medium dense at 18 - 22 ft	20		
Dense to very dense grayish brown fine sand, slightly silty	50/10"		
	47		
	50/10"		9
	50/9"		
	50		
	50/10"		6

COMPLETION DEPTH: 100.0 ft
DATE: 2-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 2/12/2014

LGBNEW 13-017.GPJ 4-8-14

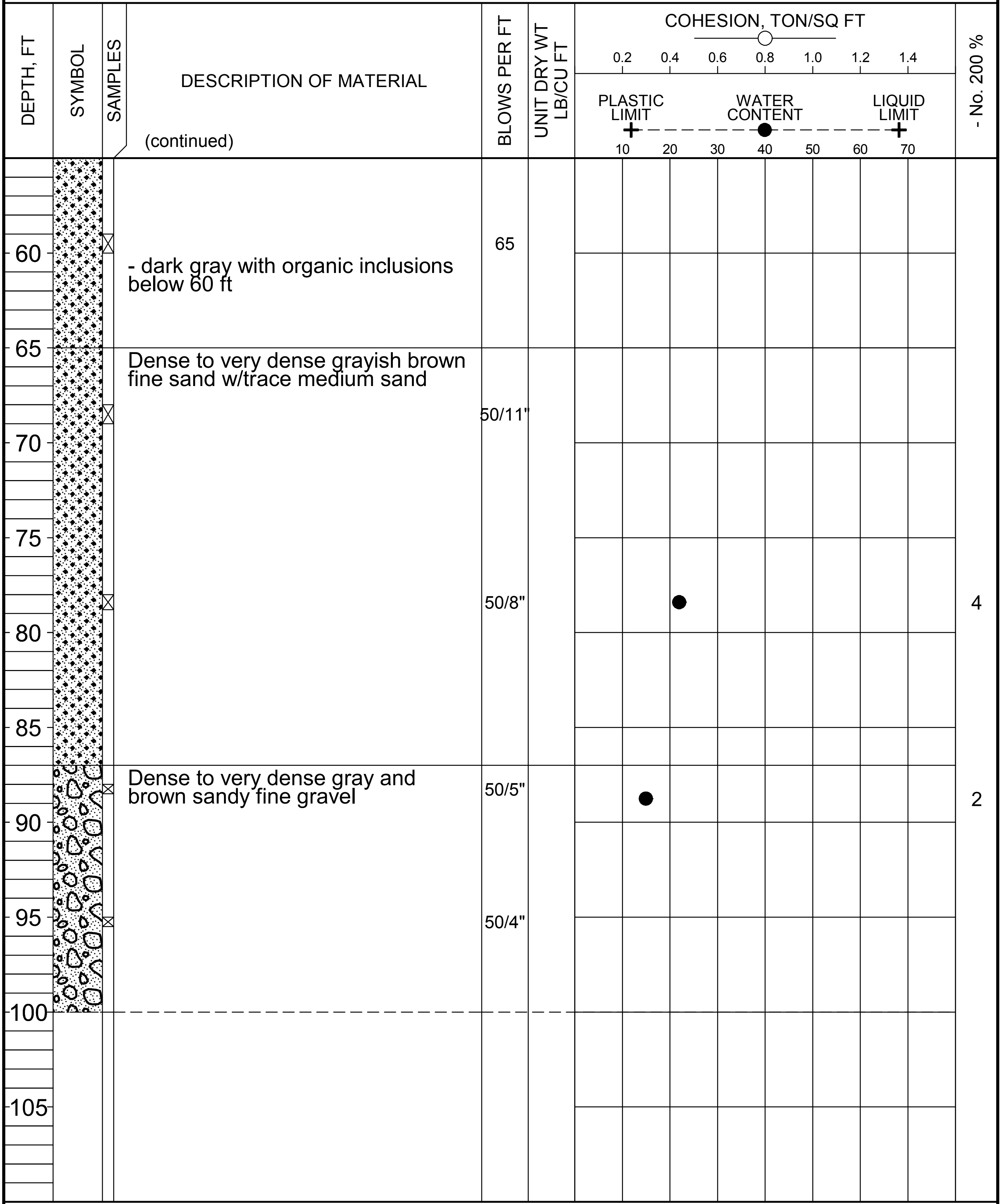
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 22

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1908+11, 60 ft Rt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
60	- dark gray with organic inclusions below 60 ft	65		
65	Dense to very dense grayish brown fine sand w/trace medium sand			
		50/11"		
		50/8"		4
	Dense to very dense gray and brown sandy fine gravel	50/5"		2
95		50/4"		
100				

COMPLETION DEPTH: 100.0 ft
DATE: 2-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 2/12/2014

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

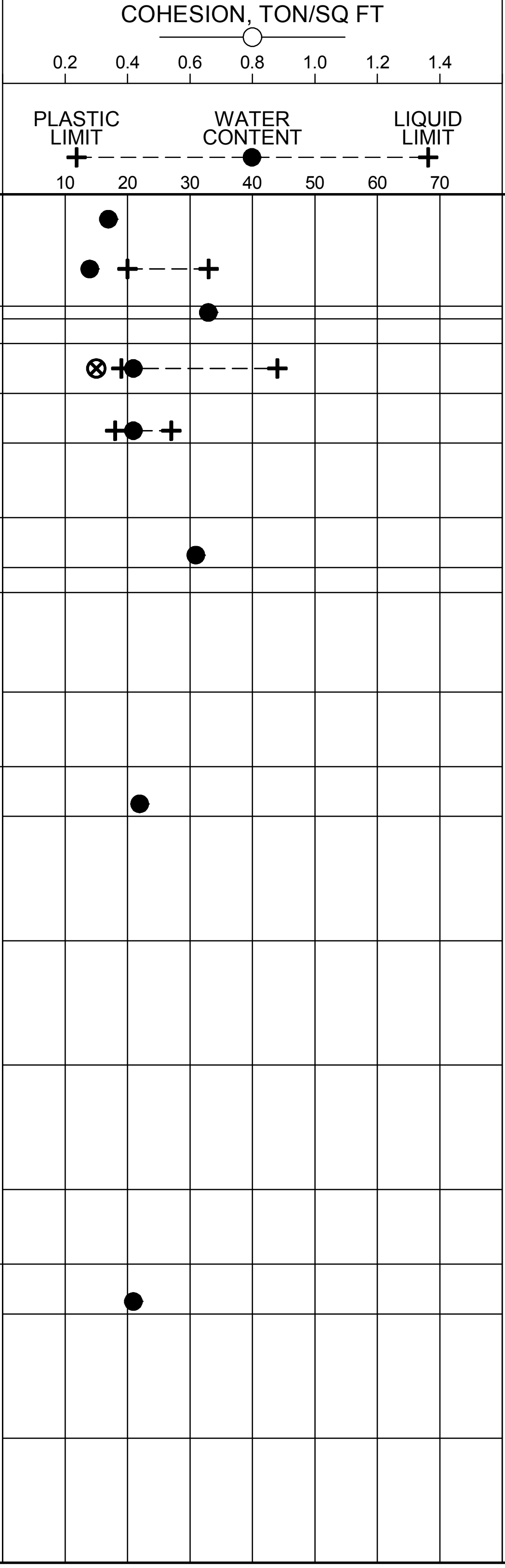
LOG OF BORING NO. 23

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash
LOCATION: Sta 1909+25, 60 ft Lt

SURF. EL: 185.4

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brown and light brown silty clay w/organics	15		
		20		87
5	Stiff light tan clay	11		
	Firm brown and tan silty clay w/ferrous stains and nodules		100	97
10	Stiff brown clayey silt	22		71
15	Loose to medium dense gray fine sandy silt w/occasional silty clay pockets	10		76
20	Loose brown silty fine sand	6		
25	Dense to very dense grayish brown fine sand	50/10"		4
30		43		
35		50/7"		
40		50		
45	Dense brownish gray fine to medium sand, slightly silty	40		6
50	- dense to very dense below 48 ft - with organic inclusions below 49 ft	50/11"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-11-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/11/2014

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

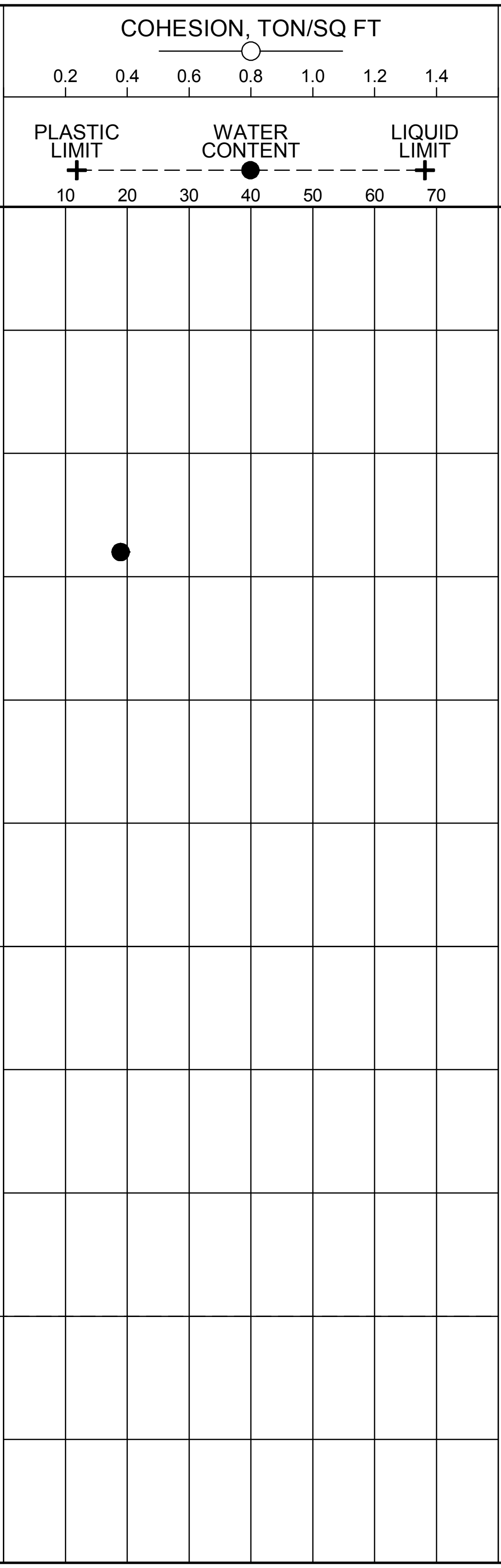
LOG OF BORING NO. 23

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1909+25, 60 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
60	- with a little fine to coarse sand and gravel below 59 ft	50/7"		
65				
70		50		3
75				
80		50/10"		
85	Dense to very dense grayish brown sandy fine to coarse gravel			
90		25/0"		
95		50/2"		
100				
105				



COMPLETION DEPTH: 100.0 ft
DATE: 2-11-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/11/2014

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

LOG OF BORING NO. 24

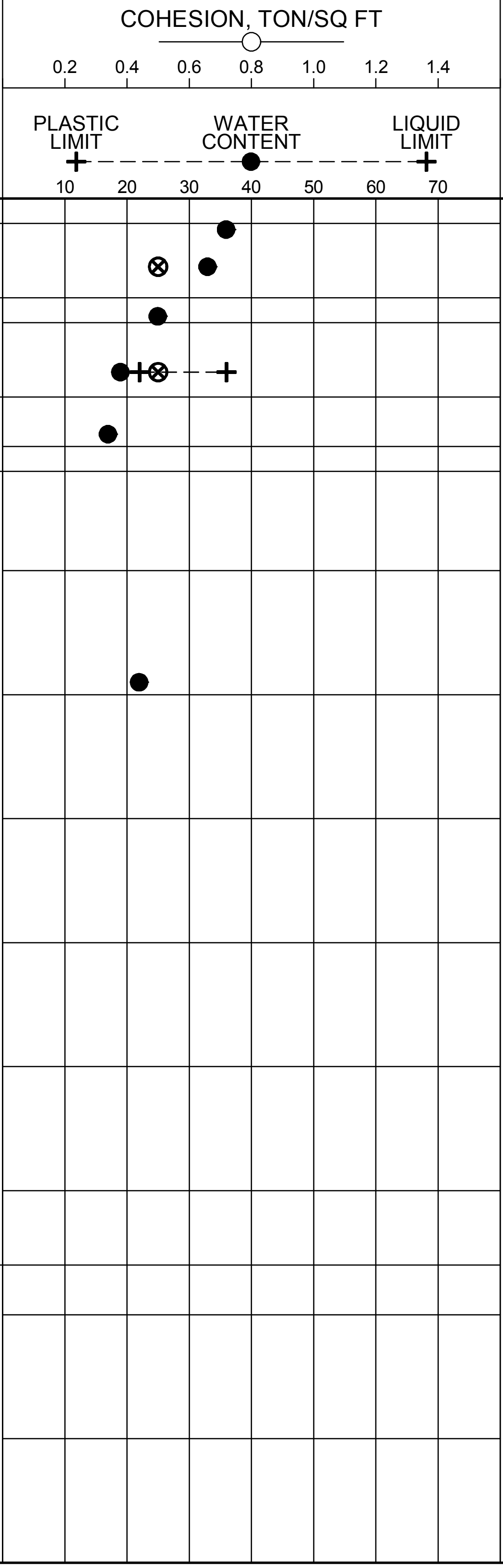
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1910+69, 60 ft Rt

SURF. EL: 187.3

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown silty clay	8		
	Firm brown clay			
	- firm to stiff below 2.5 ft			
5	Very stiff tan and brown silty clay	27		
	- stiff below 6 ft			88
10	Medium dense tan and brown fine sandy silt	28		67
	Medium dense brown and tan fine sand, slightly silty			
15		28		
	- dense below 18 ft			
20		38		7
25		50		
30		50		
35		32		
40		46		
45	Dense to very dense tan and gray fine sand, slightly silty	50/8"		
50		50/10"		
		50/8"		



COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 2/20/2014

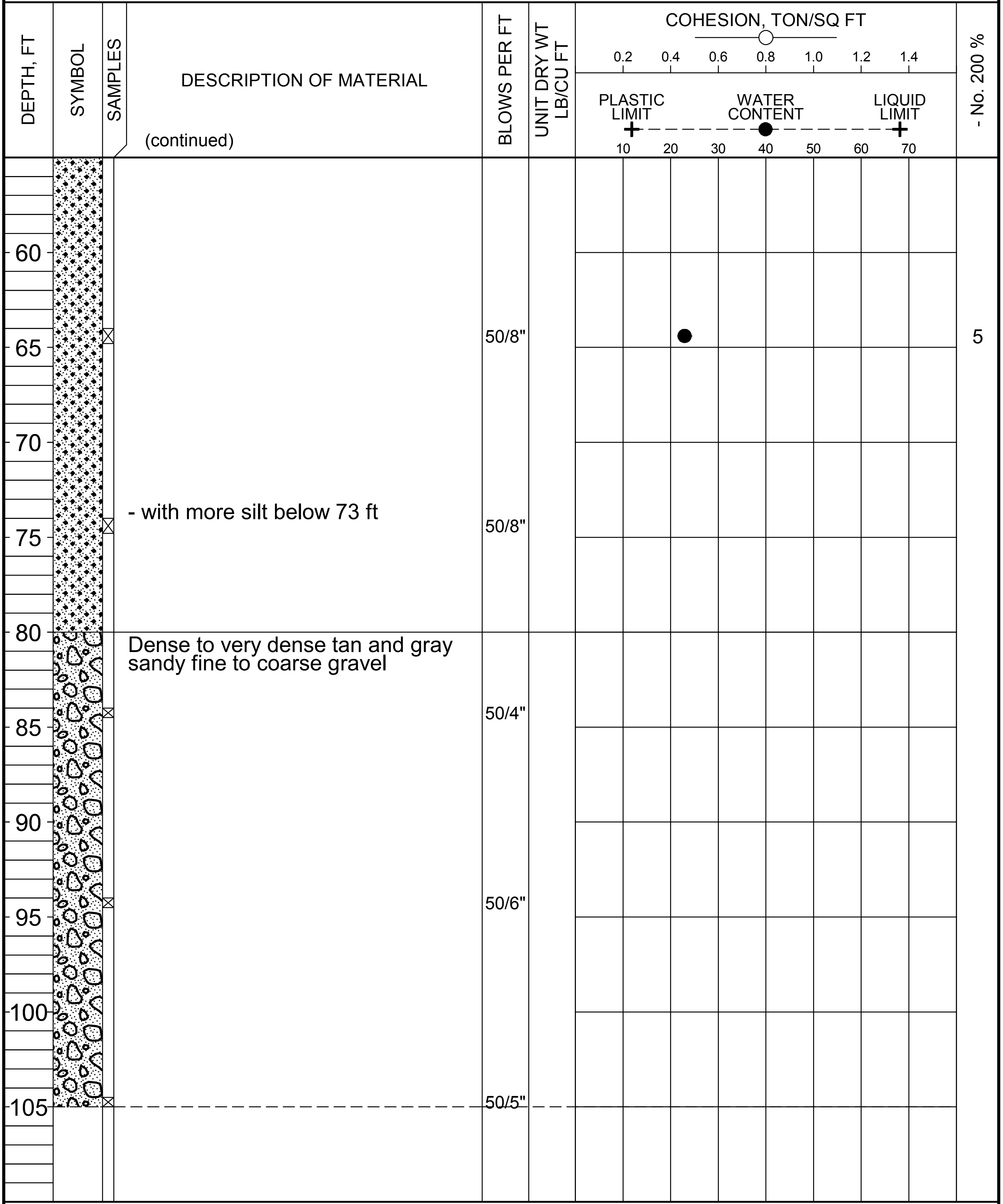
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 24

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1910+69, 60 ft Rt



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT	- No. 200 %
	(continued)				
64.5		50/8"		●	5
74.5	- with more silt below 73 ft	50/8"			
80	Dense to very dense tan and gray sandy fine to coarse gravel				
84.5		50/4"			
94.5		50/6"			
105		50/5"			

COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: Dry to 10 ft
DATE: 2/20/2014

13-017

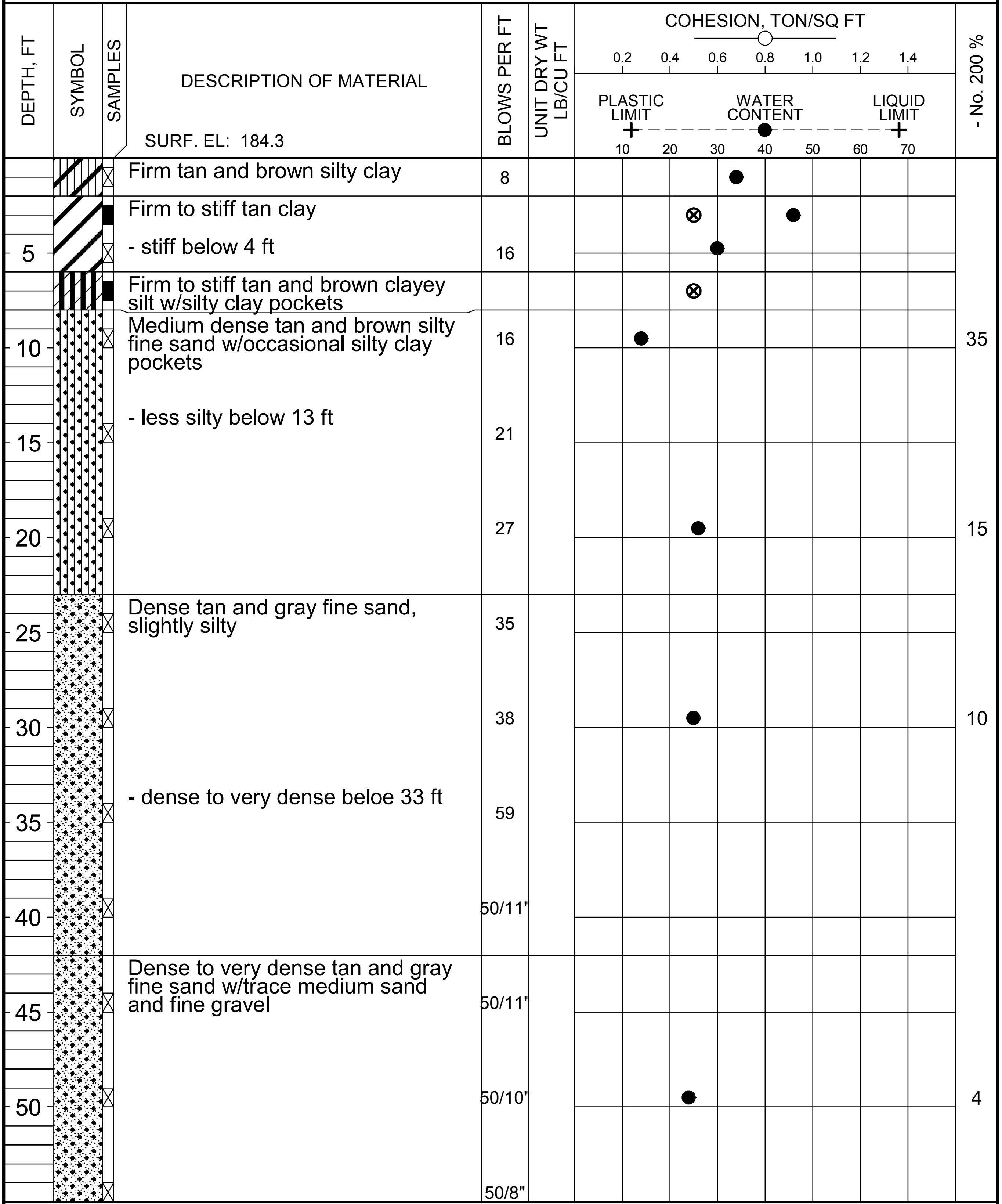
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 25

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1911+84, 60 ft Lt



SURF. EL: 184.3

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm tan and brown silty clay	8		
Firm to stiff tan clay			
- stiff below 4 ft	16		
Firm to stiff tan and brown clayey silt w/silty clay pockets			
Medium dense tan and brown silty fine sand w/occasional silty clay pockets	16		35
- less silty below 13 ft	21		
	27		15
Dense tan and gray fine sand, slightly silty	35		
	38		10
- dense to very dense beloe 33 ft	59		
	50/11"		
Dense to very dense tan and gray fine sand w/trace medium sand and fine gravel	50/11"		
	50/10"		4
	50/8"		

COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/20/2014

LGBNEW 13-017.GPJ 4-8-14

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

LOG OF BORING NO. 25

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1911+84, 60 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
60							
65		☒		50/11"			
70							
75		☒		50/3"			
80			Dense to very dense brown and gray sandy fine to coarse gravel				
85		☒		50/4"			
90							
95		☒		50/5"			
100							
105		☒		50/4"			

COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/20/2014

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

LOG OF BORING NO. 29

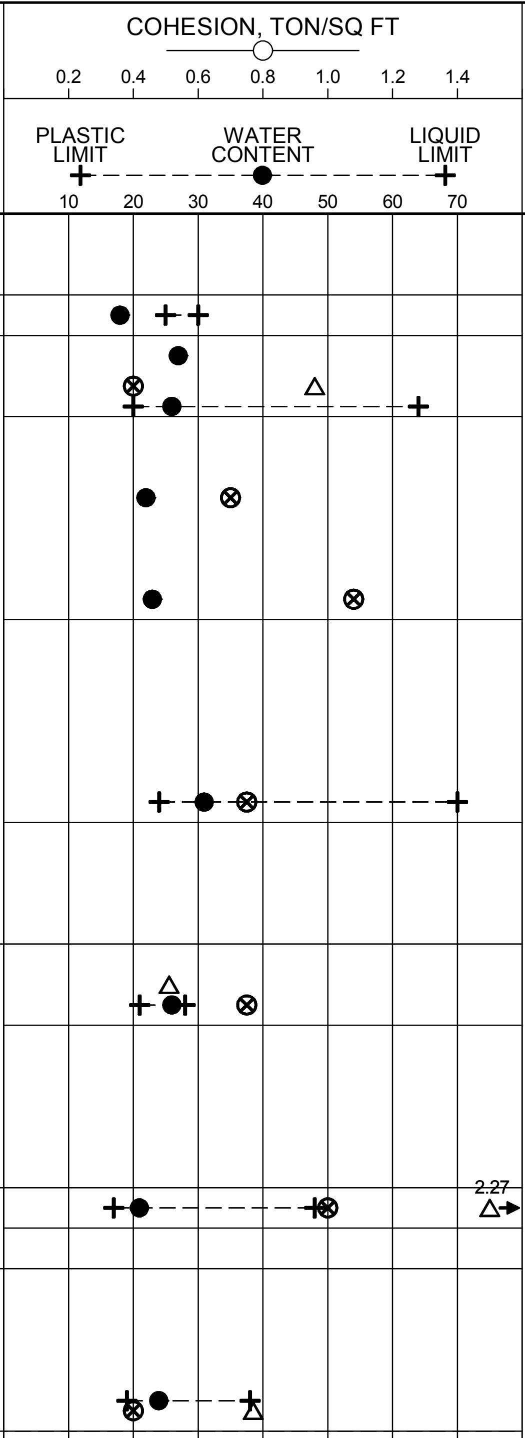
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger

LOCATION: Sta 1883+15, 80 ft Rt - Existing West Bridge End

SURF. EL: 205±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	10 inches: Asphalt Concrete			
	14 inches: Crushed Stone Base			
	Stiff reddish brown clayey silt, sandy (fill)	15		82
	Firm to stiff tan and gray clay w/fine sandy silt pockets (fill)	7		
5			101	96
	- perched water at 5.5 ft - stiff at 6 to 8 ft			
	- very stiff at 8 to 12 ft			
10				
	- stiff, reddish brown at 12 - 24 ft			
15			90	99
20	Firm to stiff reddish brown clayey silt (fill)		95	97
25	Stiff to very stiff tan and gray silty clay		106	97
	Firm to stiff brownish gray silty clay w/organic inclusions			
30			100	97
	NOTE: Borehole backfilled with cement-sand mix, pavement patched.			



COMPLETION DEPTH: 30.0 ft
DATE: 4-15-13

DEPTH TO WATER
IN BORING: Dry

DATE: 4/15/2013

13-017

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

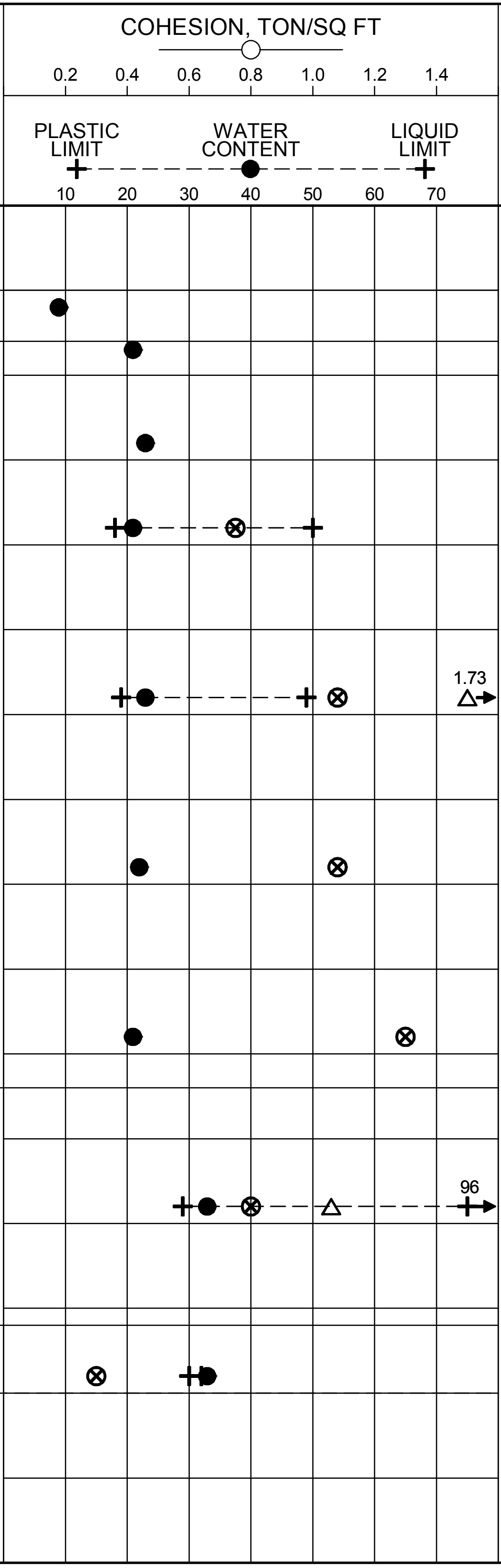
LOG OF BORING NO. 30

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger

LOCATION: Sta 1910+85, 185 ft Rt - Existing East Bridge End

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 200±	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	8 inches: Asphalt Concrete 22 inches: Crushed Stone Base			
	Medium dense tan silty fine sand (fill)	11		
5	Stiff tan and gray clay w/fine sandy silt pockets (fill)	12		
	- firm at 6 - 8 ft	8		
10	- stiff at 8 to 13 ft		104	87
15	- very stiff below 13 ft		108	95
20				
25				
30	Stiff to very stiff gray clay w/organic inclusions		89	99
35	Medium dense brownish tan silt, slightly clayey w/organic stains			99
	NOTE: Borehole backfilled with cement-sand mix, pavement patched.			



COMPLETION DEPTH: 35.0 ft
DATE: 4-15-13

DEPTH TO WATER
IN BORING: Dry

DATE: 4/15/2013

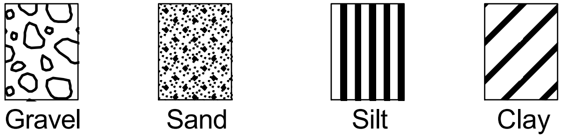
LGBNEW 13-017.GPJ 4-8-14

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

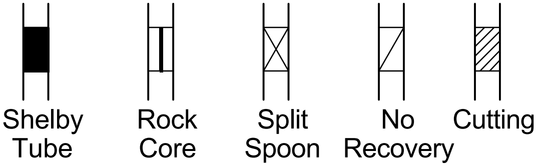
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



TERMS DESCRIBING CONSISTENCY OR CONDITION

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

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

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

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

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

TERMS CHARACTERIZING SOIL STRUCTURE

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

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

KEY 3-14-12

APPENDIX A

Summary of Subsurface Exploration Program

Project: AHTD Job No. BB0610: White River Str. & Apprs. (F)

Location: Prairie County, Arkansas

GHBW Job No.: 13-017

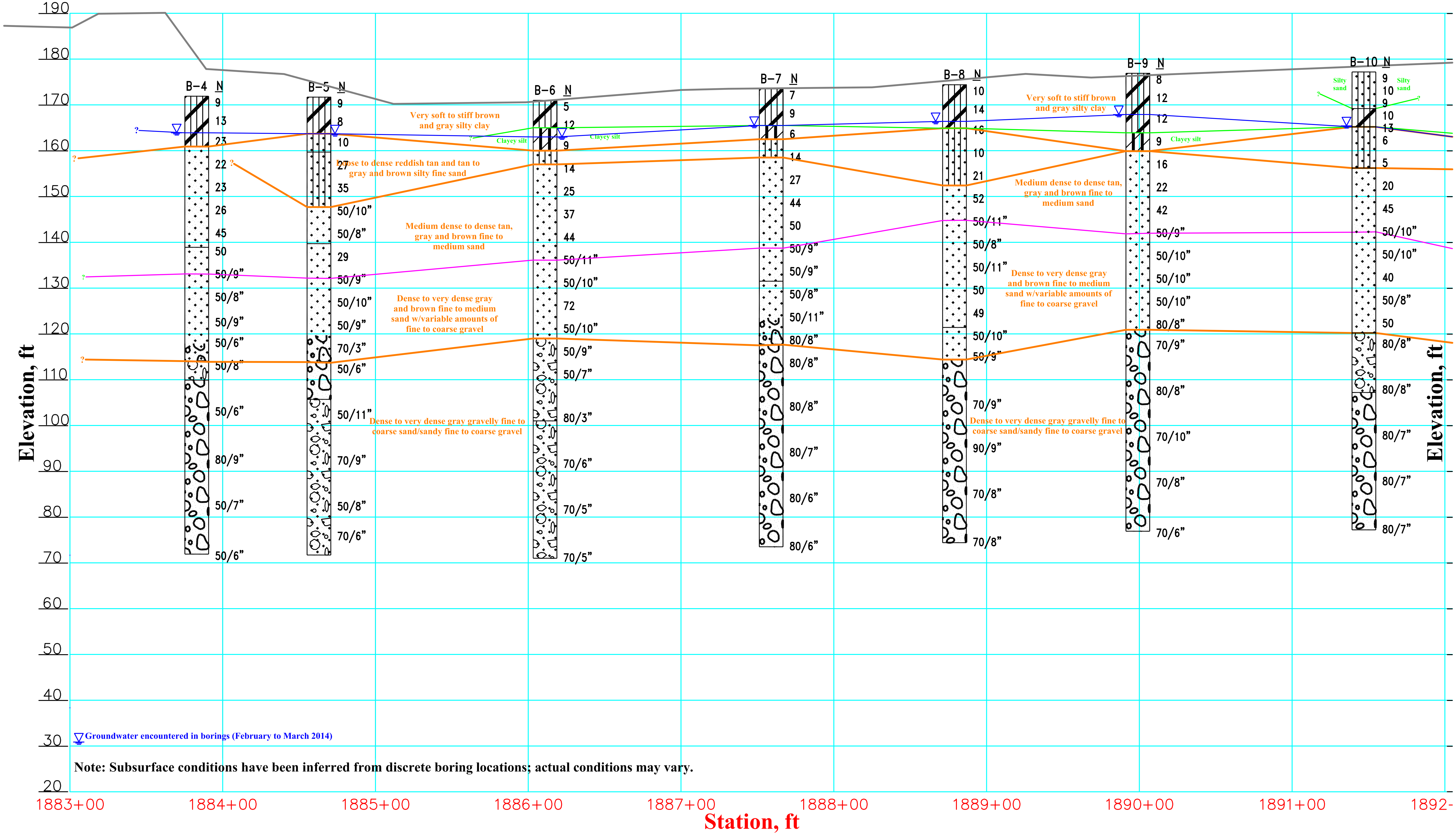
Boring No.	Approximate Station	Approximate Offset	Ground Surface El, ft	Completion Depth, ft	Project Feature
1	1877+60	70' Lt	184.3	70	West Bridge End
2	1879+00	50' Lt	191.3	70	West Bridge End
3	1880+15	50' Lt	190.5	70	West Bridge End
4	1883+83	60' Lt	171.9	100	Bent 1
5	1884+63	60' Rt	171.7	100	Bent 2
6	1886+11	60' Lt	171.0	100	Bent 3
7	1887+59	60' Rt	173.5	100	Bent 4
8	1888+79	60' Lt	174.4	100	Bent 5
9	1889+99	60' Rt	176.9	100	Bent 6
10	1891+47	60' Lt	177.2	100	Bent 7
11	1892+95	80' Rt	167.5	100	Bent 8
12	1894+15	60' Lt	168±	130	Bent 9
13	1896+90	60' Rt	137±	78	Pier 1
13A	1896+80	50' Rt	137±	150	Pier 1

Summary of Subsurface Exploration Program

Project: AHTD Job No. BB0610: White River Str. & Apprs. (F)
Location: Prairie County, Arkansas
GHBW Job No.: 13-017

14	1900+10	60' Lt	176.6	125	Pier 2
15	1916+00	CL	180.1	70	East Bridge End
16	1917+80	CL	185.1	70	East Bridge End
17	1915+40	10' Rt	178.4	88	East Bridge End
18	1902+75	60' Rt	177.1	100	Bent 10
19	1903+95	60' Lt	174.5	100	Bent 11
20	1905+43	60' Rt	177.0	100	Bent 12
21	1906+91	60' Lt	175.8	100	Bent 13
22	1908+11	60' Rt	180.1	100	Bent 14
23	1909+25	60' Lt	185.4	100	Bent 15
24	1910+69	60' Rt	187.3	105	Bent 16
25	1911+81	60' Lt	184.3	105	Bent 17
29	1883+15	80' Rt	205±	30	Existing West Bridge End
30	1910+85	185' Rt	200±	35	Existing East Bridge End

APPENDIX B



Grubbs, Hoskyn,
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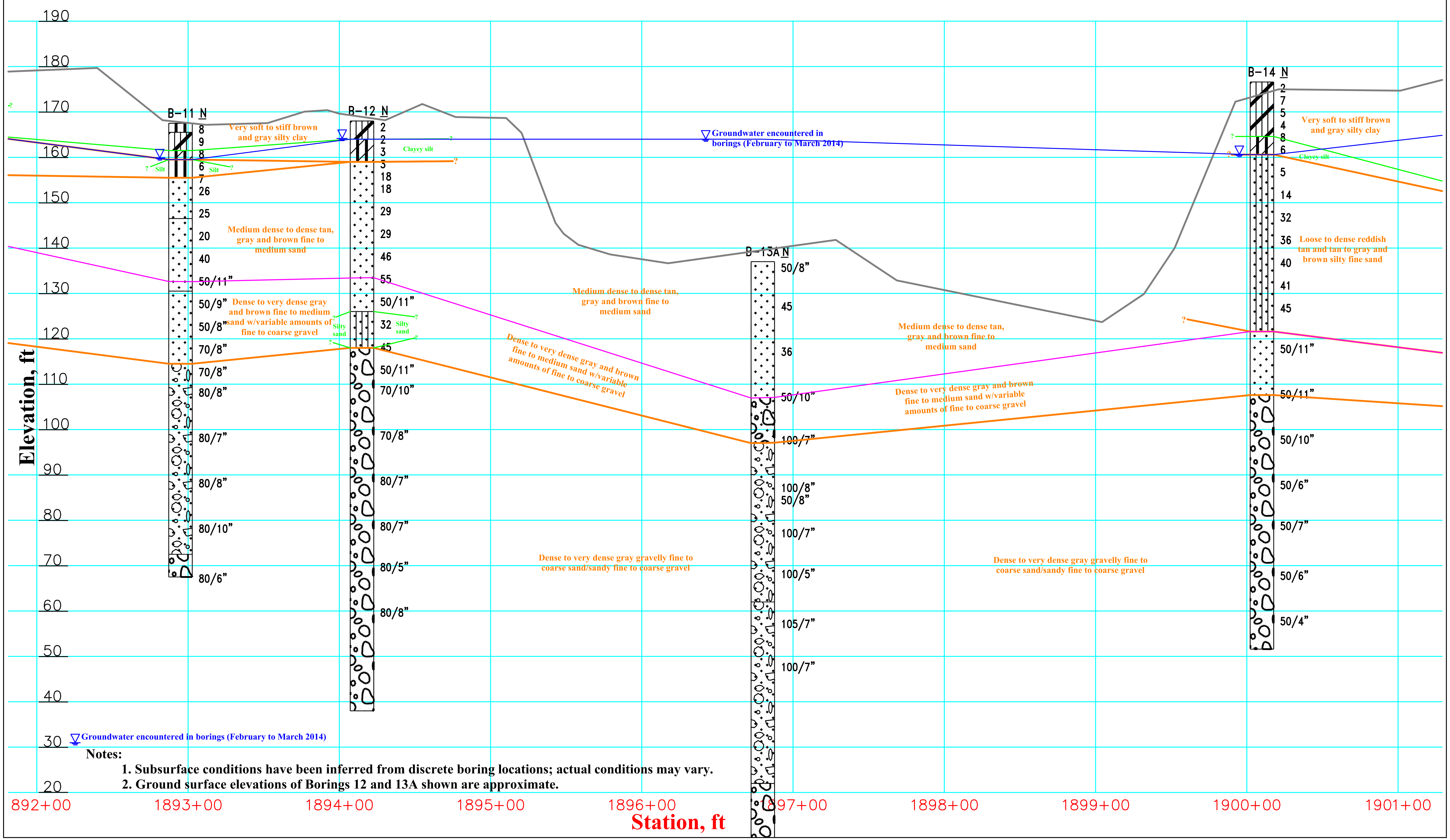
Generalized Subsurface Profile - Page 1 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

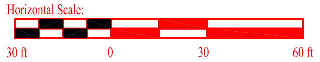
April 8, 2014

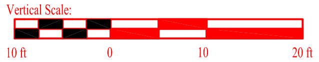
Plate



Notes:

1. Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.
2. Ground surface elevations of Borings 12 and 13A shown are approximate.





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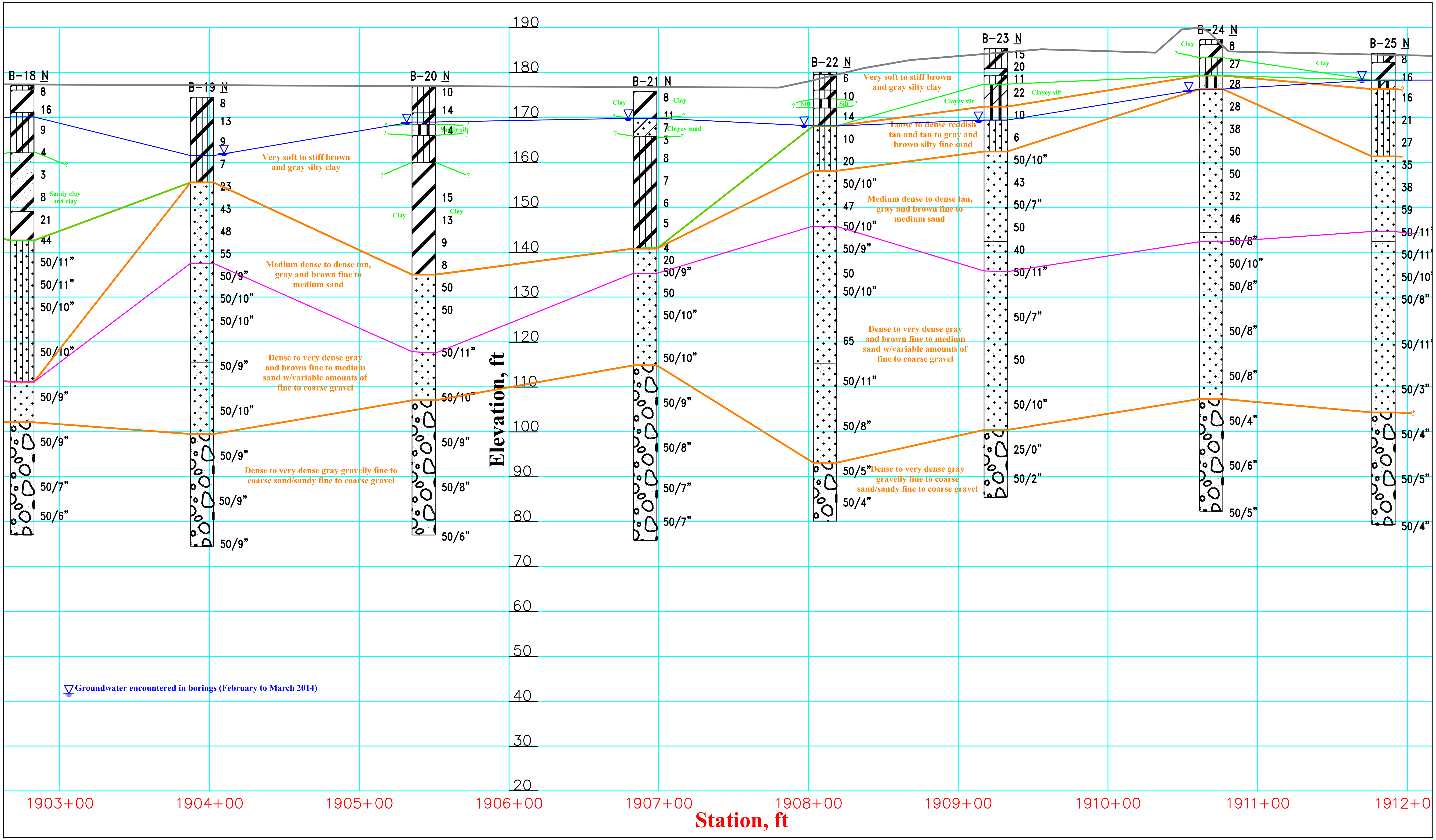
Generalized Subsurface Profile - Page 2 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	Scale: As Shown
April 8, 2014	Plate



Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

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Generalized Subsurface Profile - Page 3 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

April 8, 2014

Plate

APPENDIX C

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
1	0.5-1.5	21	36	15	21	----	----	----	----	----	----	82	CL	A-6
1	5-5.5	25	47	20	27	----	----	----	----	----	----	96	CL	A-7-6
1	9.5-10	26	43	19	24	100	100	100	100	100	98	97	CL	A-7-6
1	14-15	29	44	19	25	---	---	---	---	---	---	97	CL	A-7-6
1	19-20	24	----	----	----	100	100	100	100	100	96	13	SM	A-2-4
1	34-35	19	----	----	----	100	100	100	100	100	73	6	SP-SM	A-3
1	39-40	20	----	----	----	100	100	100	100	100	57	16	SM	A-2-4
1	54-55	17	----	----	----	100	100	96	95	94	62	5	SP-SM	A-3
2	4.5-5.5	25	52	23	29	100	100	100	100	100	99	98	CH	A-7-6
2	7-7.5	24	47	20	27	----	----	----	----	----	----	98	CL	A-7-6
2	14-15	26	44	18	26	----	----	----	----	----	----	98	CL	A-7-6
2	29-30	21	----	----	----	100	100	100	100	100	82	14	SM	A-2-4
2	59-60	19	----	----	----	100	100	92	89	88	52	7	SP-SM	A-3

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
3	2.5-3.5	28	43	20	23	----	----	----	----	----	----	97	CL	A-7-6
3	4.5-5	25	48	19	29	100	100	100	100	99	97	95	CL	A-7-6
3	7-7.5	24	47	21	26	100	100	100	100	99	98	95	CL	A-7-6
3	14-15	26	38	18	20	----	----	----	----	----	----	93	CL	A-6
3	19-20	25	19	11	8	----	----	----	----	----	----	39	SC	A-4
3	29-30	24	----	----	----	100	100	100	100	100	76	15	SM	A-2-4
3	39-40	22	----	----	----	100	100	100	100	100	34	6	SP-SM	A-1-b
3	49-50	18	----	----	----	100	100	98	98	95	57	11	SP-SM	A-2-4
4	7-7.5	23	46	19	27	100	100	100	100	100	99	97	CL	A-7-6
4	14-15	26	----	----	----	100	100	100	100	100	60	5	SP-SM	A-3
4	38.5-39.5	30	----	----	----	100	100	100	100	100	77	4	SP	A-3
4	53.5-54	18	----	----	----	100	100	89	87	86	58	10	SP-SM	A-3
4	88.5-89.5	18	----	----	----	100	100	92	51	26	7	2	GW	A-1-a

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
5	3-3.5	24	33	17	16	100	100	100	100	100	100	87	CL	A-6
5	9-10	25	----	----	----	----	----	----	----	----	----	31	SM	A-2-4
5	24-25	23	----	----	----	100	100	100	100	100	95	2	SP	A-3
5	43.5-44.5	31	----	----	----	100	100	100	100	100	77	4	SP	A-3
5	68.5-69.5	17	----	----	----	100	96	88	80	73	54	8	SP-SM	A-3
6	3-3.5	28	48	21	27	100	100	100	100	100	100	99	CL	A-7-6
6	19-20	22	----	----	----	100	100	100	100	100	99	5	SP-SM	A-3
6	58.5-59.5	20	----	----	----	100	100	89	81	75	51	18	SM	A-2-4
6	88.5-89	12	----	----	----	100	100	100	100	100	77	4	SP	A-3
7	7-7.5	28	42	19	23	----	----	----	----	----	----	97	CL	A-7-6
7	14-15	27	----	----	----	----	----	----	----	----	----	41	SM	A-2-4
7	24-25	23	----	----	----	100	100	100	100	100	90	9	SP-SM	A-3
7	43.5-44.5	18	----	----	----	100	100	97	94	91	42	19	SM	A-1-b

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
7	88.5-89	20	----	----	----	100	100	99	77	54	24	4	SW	A-1-b
8	7-7.5	26	37	15	22	100	100	100	100	100	100	81	CL	A-6
8	14-15	26	----	----	----	100	100	100	100	100	100	29	SM	A-2-4
8	29-30	22	----	----	----	100	100	100	100	100	95	5	SP-SM	A-3
8	49-50	21	----	----	----	100	100	100	100	100	95	12	SP-SM	A-2-4
8	68.5-69.5	17	----	----	----	100	100	95	85	74	44	4	SP	A-1-b
9	14-15	30	25	19	6	----	----	----	----	----	----	66	CL-ML	A-4
9	29-30	24	----	----	----	100	100	100	100	100	95	9	SP-SM	A-3
9	58.5-59.5	31	----	----	----	100	100	99	81	49	24	7	SP-SM	A-1-a
10	6.5-7.5	12	Non-plastic			----	----	----	----	----	----	20	SM	A-2-4
10	14-15	28	Non-plastic			----	----	----	----	----	----	38	SM	A-4
10	29-30	21	----	----	----	100	100	100	100	99	83	4	SP	A-3
10	49-50	20	----	----	----	100	100	100	100	98	77	4	SP	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
11	14-15	25	----	----	----	----	----	----	----	----	----	5	SP-SM	A-3
11	29-30	23	----	----	----	100	100	100	100	100	98	2	SP	A-3
11	48.5-49.5	18	----	----	----	100	100	100	98	97	31	2	SP	A-1-b
12	19-20	17	----	----	----	----	----	----	----	----	----	2	SP	A-3
12	38.5-39.5	19	----	----	----	100	100	100	99	98	66	3	SP	A-3
12	49-50	----	----	----	----	100	100	100	100	100	93	16	SM	A-2-4
12	68.5-69.5	9	----	----	----	100	100	90	45	18	6	2	GW	A-1-a
13	6.5-7.5	24	Non-plastic			100	100	100	100	100	88	31	SM	A-2-4
13A	9-10	27	----	----	----	100	100	100	98	98	85	1	SP	A-3
13A	19-20	22	----	----	----	100	97	86	84	84	79	4	SP	A-3
13A	48.5-49.5	16	----	----	----	100	100	94	81	76	26	6	SW-SM	A-1-b
13A	68.5-69	17	----	----	----	100	92	84	74	66	32	5	SW-SM	A-1-b

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
13A	99-100	20	----	----	----	100	100	75	70	65	36	1	SP	A-1-b
13A	139-140	20	----	----	----	100	100	86	68	55	29	1	SP	A-1-b
14	4.5-5.5	30	34	19	15	100	100	100	100	100	100	95	CL	A-6
14	14-15	28	22	17	5	100	100	100	100	100	100	62	CL-ML	A-4
14	24-25	27	Non-plastic			100	100	100	100	100	100	36	SM	A-4
14	44-45	----	----	----	----	100	100	100	100	100	99	24	SM	A-2-4
14	58.5-59.5	----	----	----	----	100	100	100	100	100	90	11	SP-SM	A-2-4
14	100-118	----	----	----	----	100	100	88	50	29	14	2	GP	A-1-a
15	0.5-1.5	30	65	25	40	----	----	----	----	----	----	89	CH	A-7-6
15	2-2.5	24	56	24	32	----	----	----	----	----	----	94	CH	A-7-6
15	6.5-7.5	17	----	----	----	100	100	100	100	100	100	13	SM	A-2-4
15	24-25	23	Non-plastic			100	100	100	100	100	99	12	SP-SM	A-2-4
15	44-45	18	----	----	----	100	95	95	92	90	46	9	SP-SM	A-1-b
15	59-60	----	----	----	----	100	100	100	100	100	98	6	SP-SM	A-3

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
15	69-70	----	----	----	----	100	100	95	94	93	60	5	SP-SM	A-3
16	1-2	35	49	26	23	----	----	----	----	----	----	86	CL	A-7-6
16	2-2.5	39	79	27	52	----	----	----	----	----	----	99	CH	A-7-6
16	4.5-5	32	42	21	21	----	----	----	----	----	----	96	CL	A-7-6
16	9-10	7	----	----	----	100	100	100	100	100	99	29	SM	A-2-4
16	19-20	----	----	----	----	100	100	100	100	100	100	9	SP-SM	A-3
16	39-40	----	----	----	----	100	100	100	100	100	90	19	SM	A-2-4
16	49-50	----	----	----	----	100	100	100	99	98	48	4	SP	A-1-b
16	59-60	----	----	----	----	100	100	100	100	100	98	13	SM	A-2-4
16	69-70	----	----	----	----	100	100	93	90	82	39	4	SP	A-1-b
17	2.5-3.5	21	----	----	----	----	----	----	----	----	----	81	CL	A-7-6
17	4.5-5.5	27	46	22	24	----	----	----	----	----	----	88	CL	A-7-6
17	6.5-7.5	22	Non-plastic			----	----	----	----	----	----	42	SM	A-4
17	9-10	----	----	----	----	----	----	----	----	----	----	3	SP	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
17	14-15	----	----	----	----	----	----	----	----	----	----	5	SP-SM	A-3
17	29-30	----	----	----	----	100	100	100	100	100	86	9	SP-SM	A-3
17	44-45	----	----	----	----	100	100	100	100	100	87	6	SP-SM	A-3
17	54-55	----	----	----	----	100	100	100	100	100	98	17	SM	A-2-4
17	68.5-69	----	----	----	----	100	100	100	100	99	74	3	SP	A-3
17	83.5-84	----	----	----	----	100	94	78	59	41	18	3	SW	A-1-a
18	9-10	25	Non-plastic			----	----	----	----	----	----	51	ML	A-4
18	19-20	28	26	18	8	----	----	----	----	----	----	77	CL	A-4
18	29-30	49	51	24	27	----	----	----	----	----	----	99	CH	A-7-6
18	38.5-39.5	22	----	----	----	100	100	100	100	100	97	20	SM	A-2-4
18	68.5-69.5	19	----	----	----	100	100	94	90	89	84	7	SP-SM	A-3
18	88.5-89.5	15	----	----	----	100	100	78	35	19	11	3	GP	A-1-a
19	6.5-7	24	40	18	22	100	100	100	100	100	99	96	CL	A-6
19	24-25	22	----	----	----	100	100	100	100	100	90	8	SP-SM	A-3

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
19	43.5-44.5	22	----	----	----	100	100	100	100	100	82	8	SP-SM	A-3
19	68.5-69.5	26	----	----	----	100	100	100	100	99	69	4	SP	A-3
19	78.5-79.5	11	----	----	----	100	100	96	79	58	35	8	SP-SM	A-1-b
20	9-10	25	19	16	3	----	----	----	----	----	----	52	ML	A-4
20	14-14.5	29	35	19	16	----	----	----	----	----	----	99	CL	A-6
20	19-19.5	53	64	26	38	----	----	----	----	----	----	99	CH	A-7-6
20	44-45	15	----	----	----	100	100	90	86	78	47	12	SP-SM	A-1-b
20	58.5-59.5	17	----	----	----	100	100	98	94	90	63	6	SP-SM	A-3
20	88.5-89.5	13	----	----	----	100	100	93	56	39	20	3	SW	A-1-a
21	6.5-7.5	25	26	15	11	----	----	----	----	----	----	46	SC	A-6
21	19-20	30	40	19	21	----	----	----	----	----	----	97	CL	A-6
21	44-45	18	----	----	----	100	100	100	97	87	55	4	SP	A-3
21	68.5-69.5	17	----	----	----	100	100	100	73	38	8	4	SP	A-1-b

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
22	2.5-3.5	30	48	22	26	----	----	----	----	----	----	99	CL	A-7-6
22	9-10	36	37	19	18	----	----	----	----	----	----	95	CL	A-6
22	14-15	25	----	----	----	100	100	100	100	100	99	13	SM	A-2-4
22	33.5-34.5	24	----	----	----	100	100	100	100	100	100	9	SP-SM	A-3
22	48.5-49.5	24	----	----	----	100	100	100	100	100	100	6	SP-SM	A-3
22	78.5-79.5	22	----	----	----	100	100	100	100	100	92	4	SP	A-3
22	88.5-89	15	----	----	----	100	100	87	71	65	46	2	SP	A-1-b
23	2.5-3.5	14	33	20	13	----	----	----	----	----	----	87	CL	A-6
23	7-7.5	21	44	19	25	----	----	----	----	----	----	97	CL	A-7-6
23	9-10	21	27	18	9	----	----	----	----	----	----	71	CL	A-4
23	14-15	31	----	----	----	100	100	100	100	99	95	76	ML	A-4
23	24-25	22	----	----	----	100	100	100	100	100	95	4	SP	A-3
23	44-45	21	----	----	----	100	100	100	100	99	75	6	SP-SM	A-3
23	68.5-69.5	19	----	----	----	100	94	87	86	85	75	3	SP	A-3

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
24	6.5-7.5	19	36	22	14	----	----	----	----	----	----	88	CL	A-6
24	9-10	17	----	----	----	----	----	----	----	----	----	67	ML	A-4
24	19-20	22	----	----	----	100	100	96	96	96	96	7	SP-SM	A-3
24	64-65	23	----	----	----	100	100	100	100	100	92	5	SP-SM	A-3
25	9-10	14	----	----	----	----	----	----	----	----	----	35	SM	A-2-4
25	19-20	26	----	----	----	100	100	100	100	100	98	15	SM	A-2-4
25	29-30	24	----	----	----	100	100	100	100	100	100	10	SP-SM	A-3
25	48.5-49.5	24	----	----	----	100	100	100	100	99	96	4	SP	A-3
29	1.5-2.5	18	30	25	5	----	----	----	----	----	----	82	CL-ML	A-4
29	5-5.5	26	64	20	44	----	----	----	----	----	----	96	CH	A-7-6
29	14.5-15	30	70	24	46	----	----	----	----	----	----	99	CH	A-7-6
29	19.5-20	26	28	21	7	----	----	----	----	----	----	97	CL-ML	A-4
29	24.5-25	21	48	17	31	----	----	----	----	----	----	97	CL	A-7-6
29	29.5-30	24	38	19	19	----	----	----	----	----	----	97	CL	A-6

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
30	9.5-10	21	50	18	32	----	----	----	----	----	----	87	CH	A-7-6
30	14.5-15	23	49	19	30	----	----	----	----	----	----	95	CL	A-7-6
30	29.5-30	33	85	26	59	----	----	----	----	----	----	99	CH	A-7-6
30	34-34.5	33	32	30	2	----	----	----	----	----	----	99	ML	A-4

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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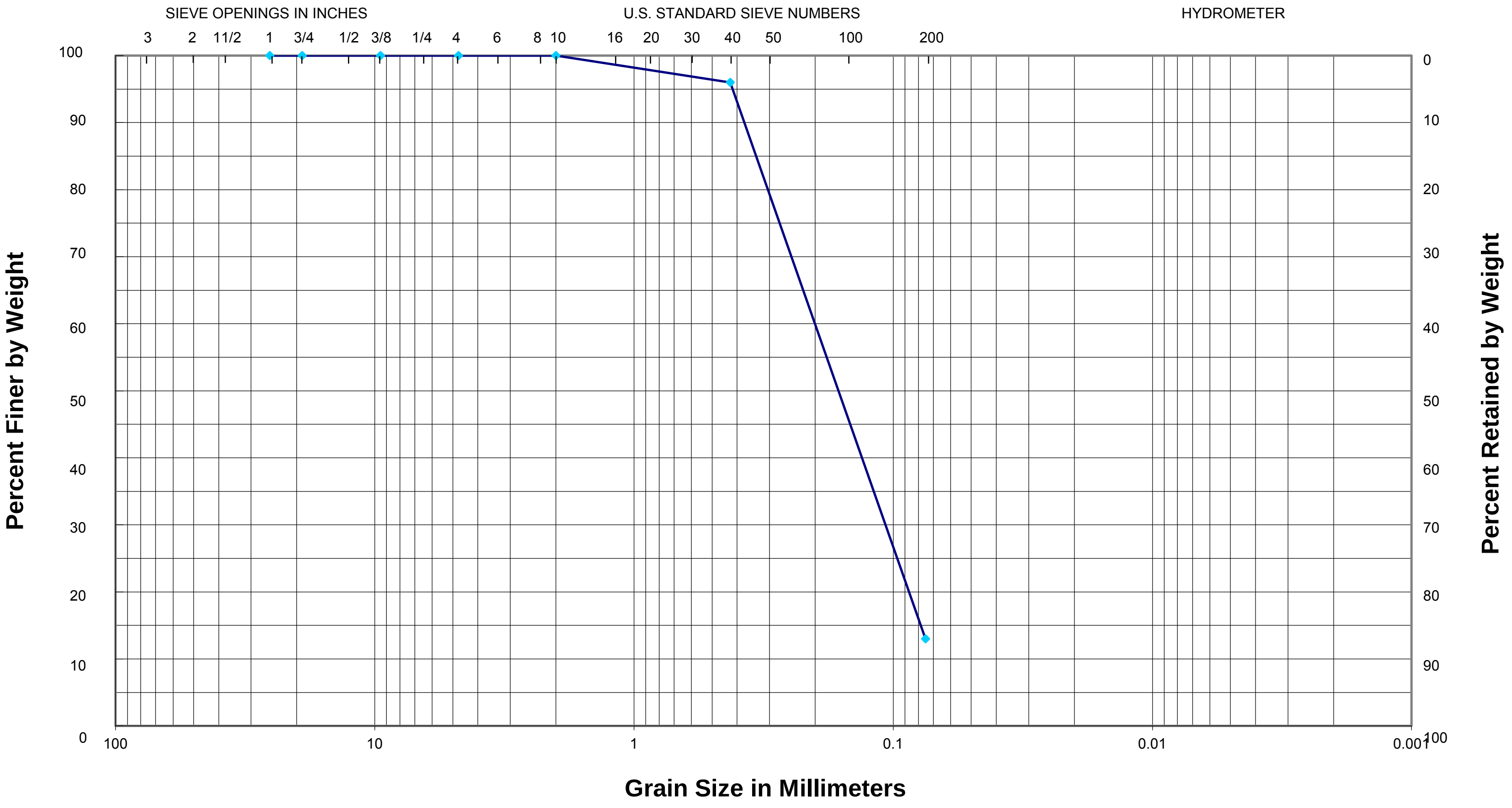
13-017

GRAIN SIZE CURVE



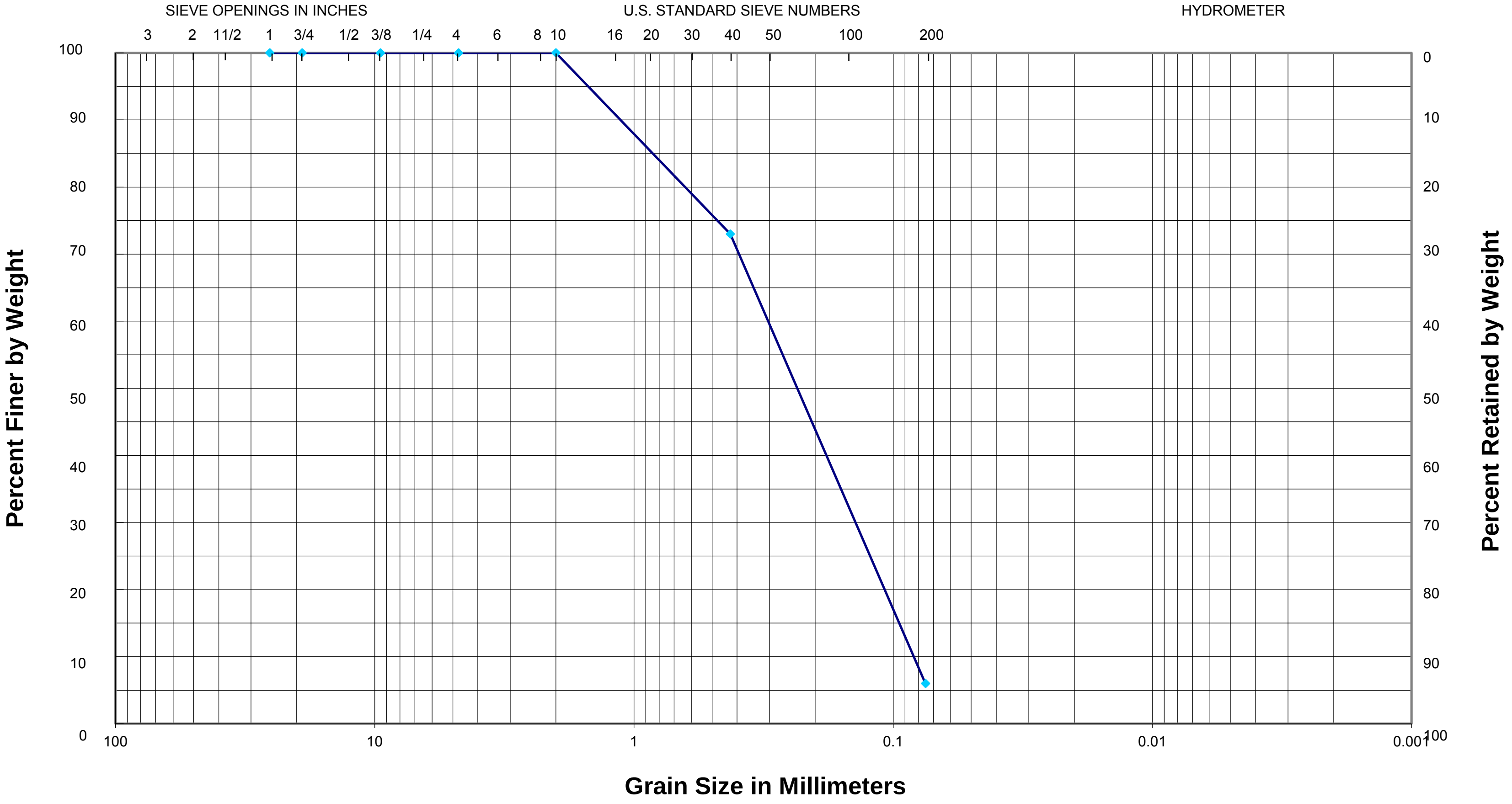
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 34-35 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

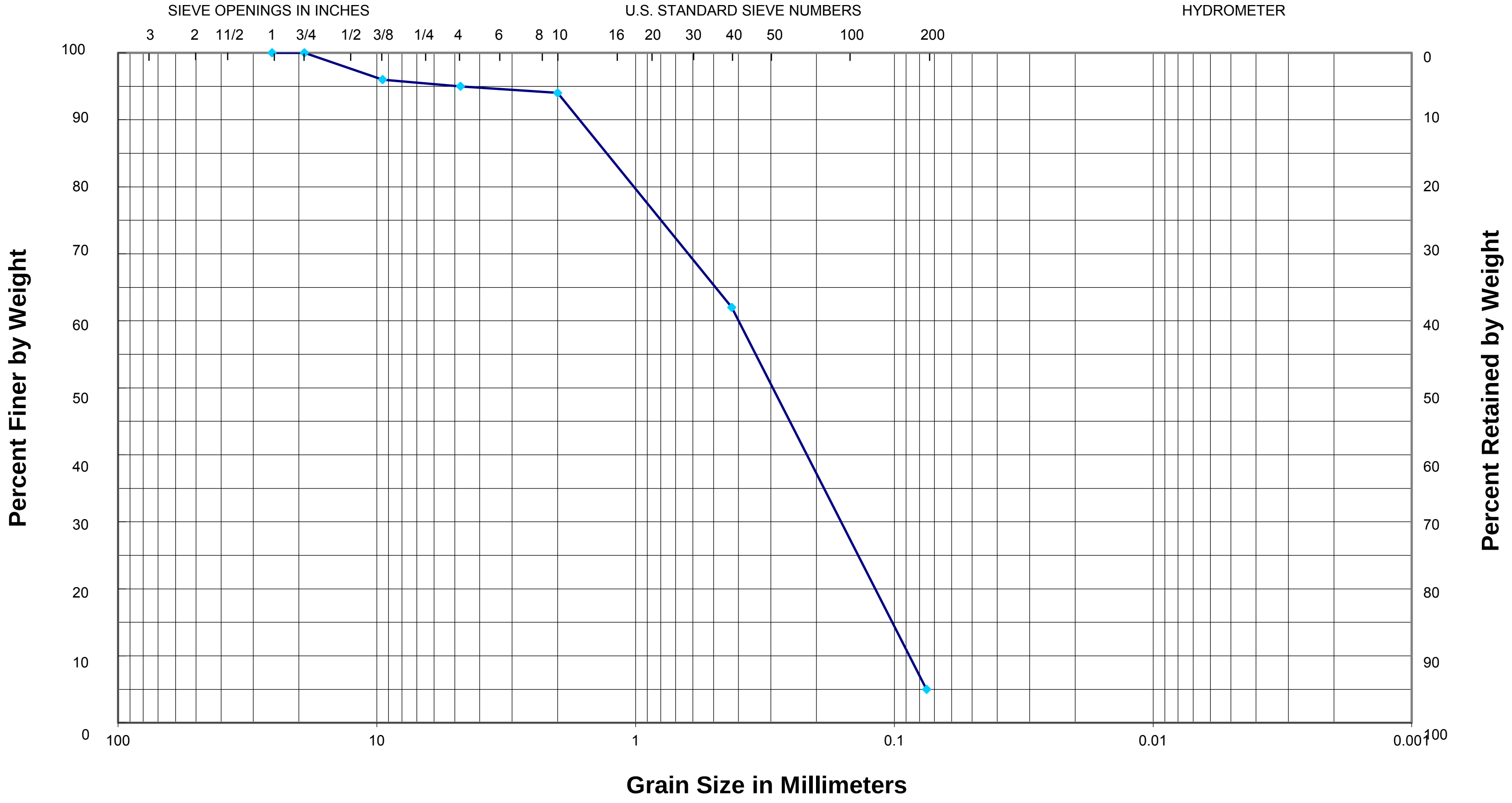
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 39-40 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand w/clayey fine sand seams
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



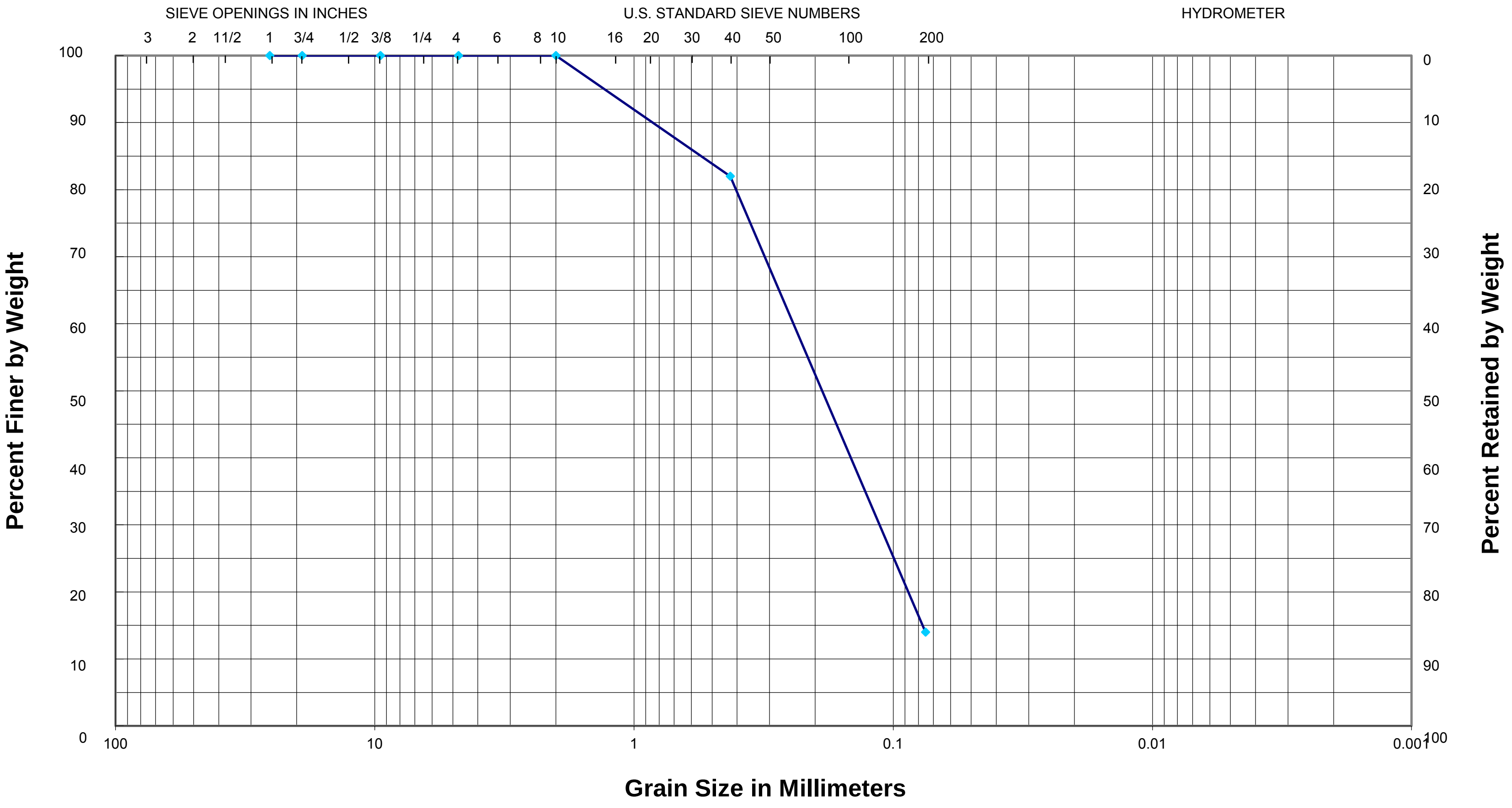
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 54-55 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



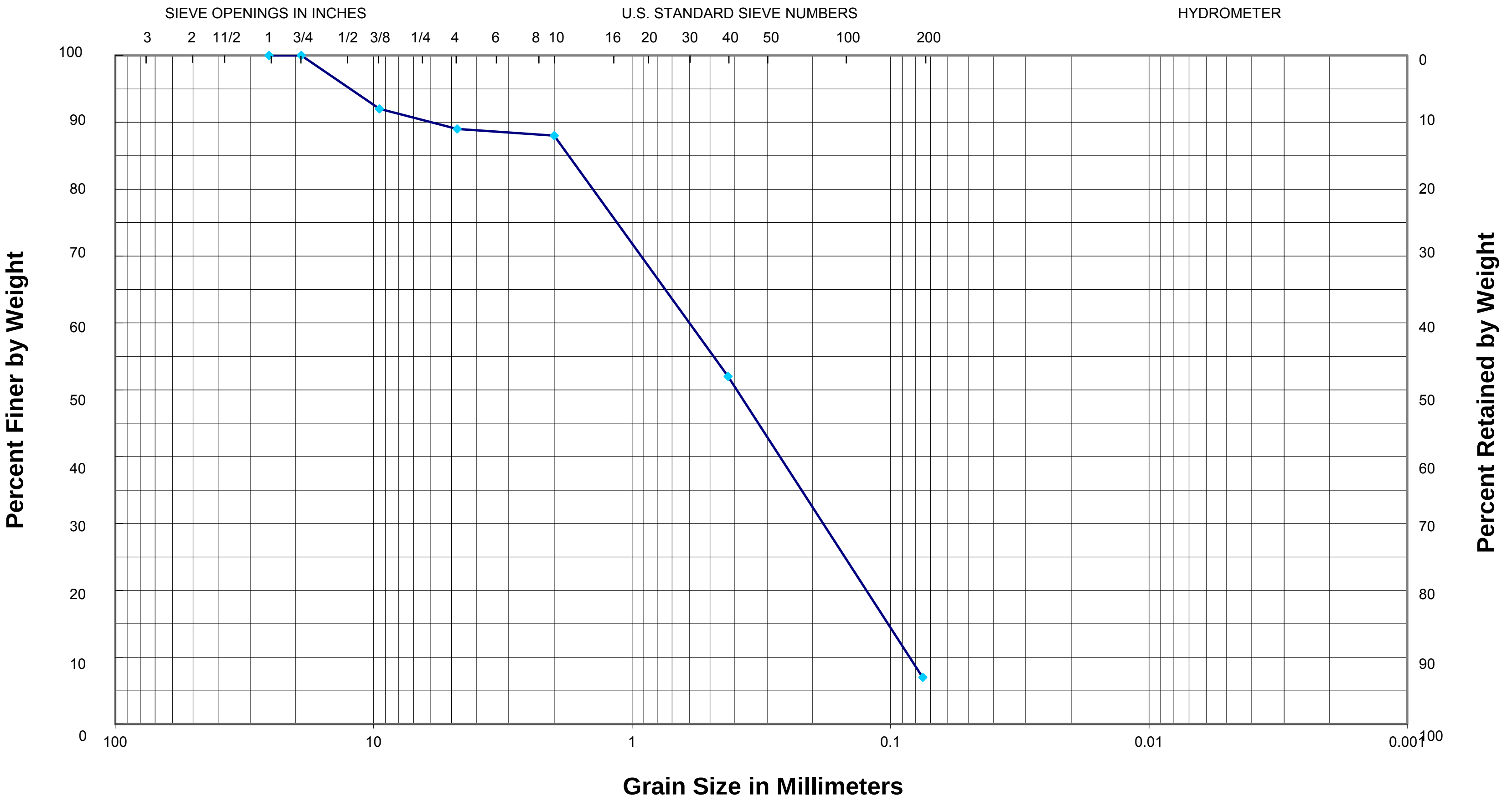
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 2, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 2, 59-60 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

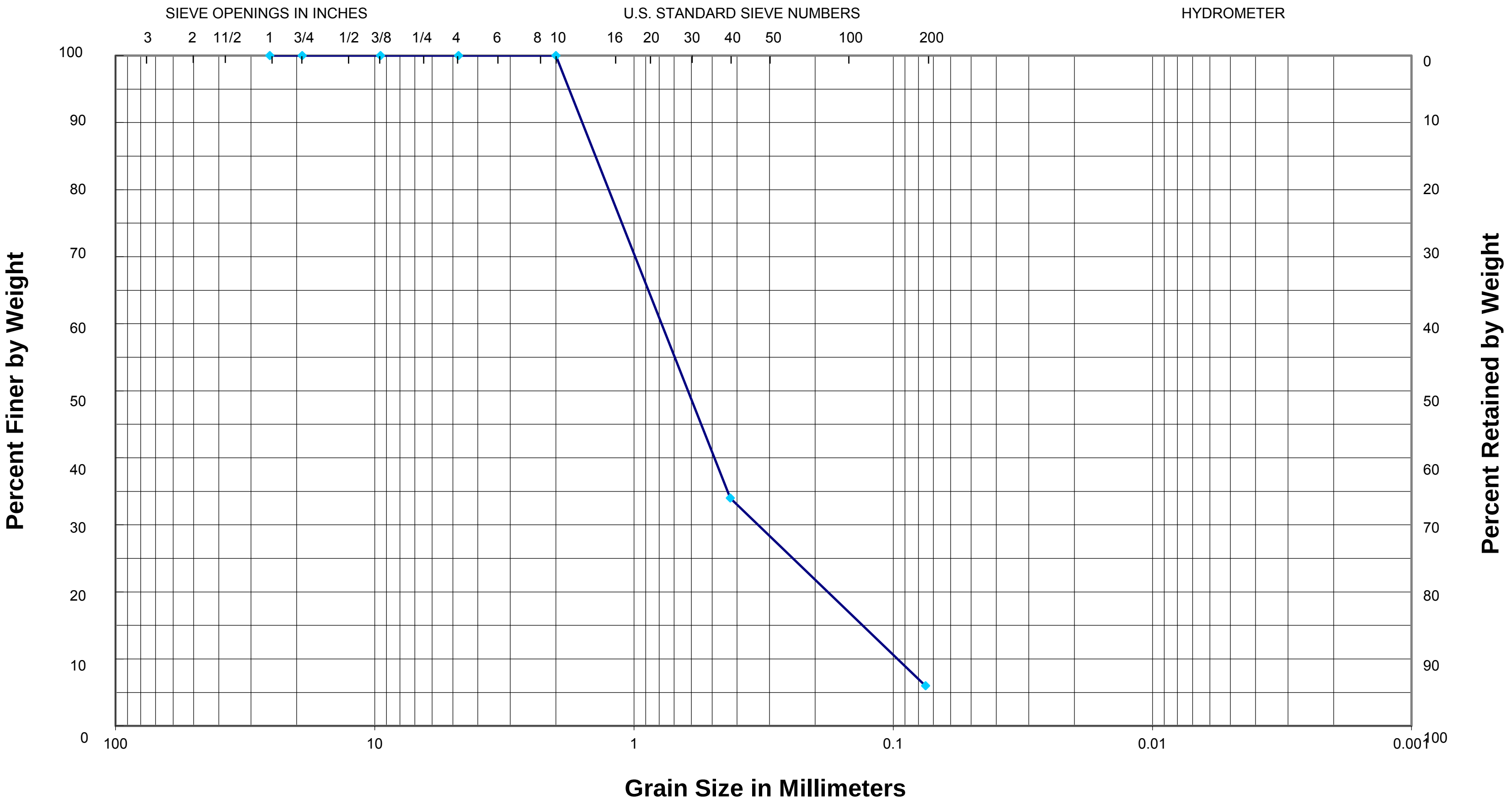
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 3, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



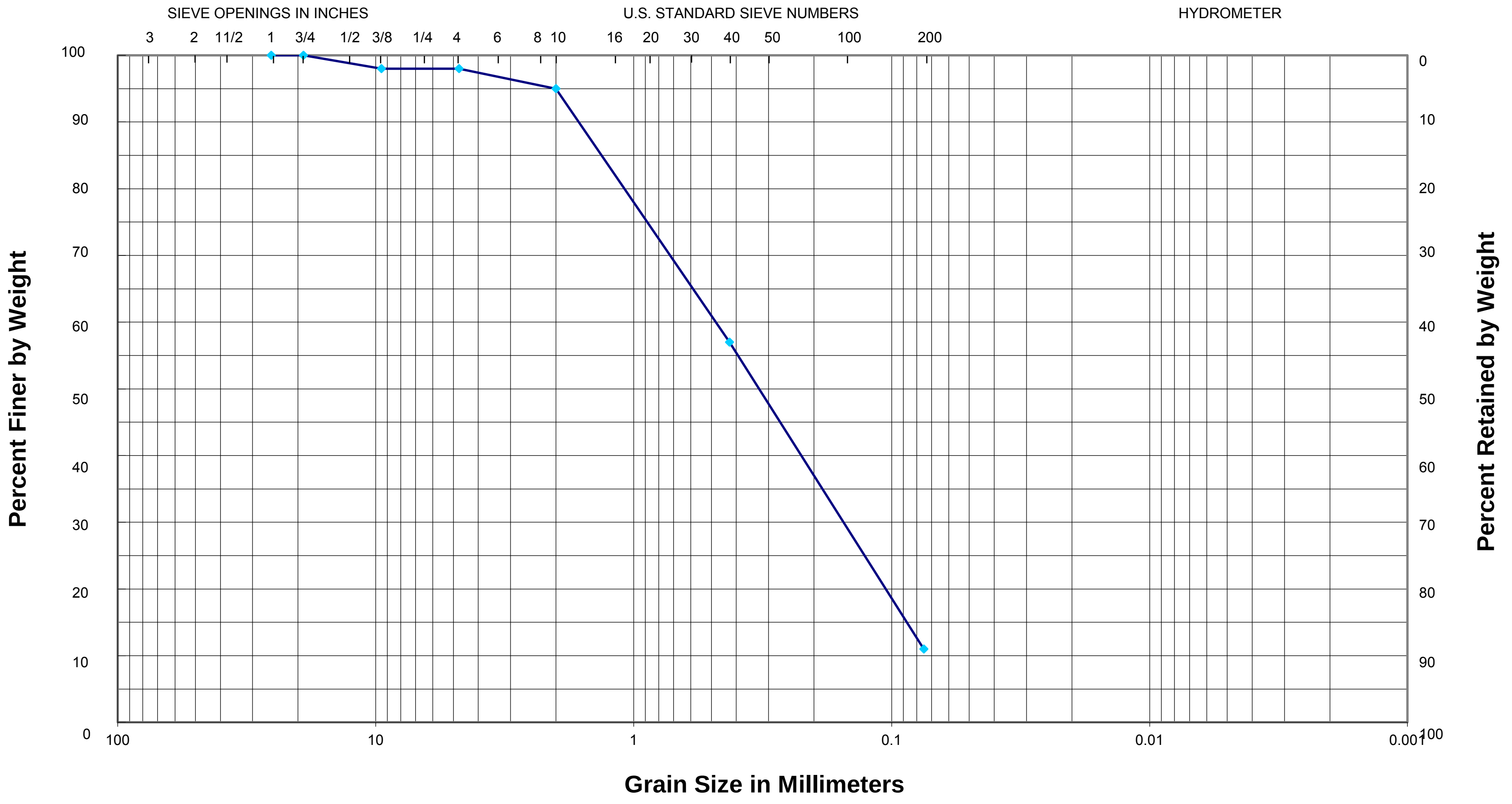
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 3, 39-40 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

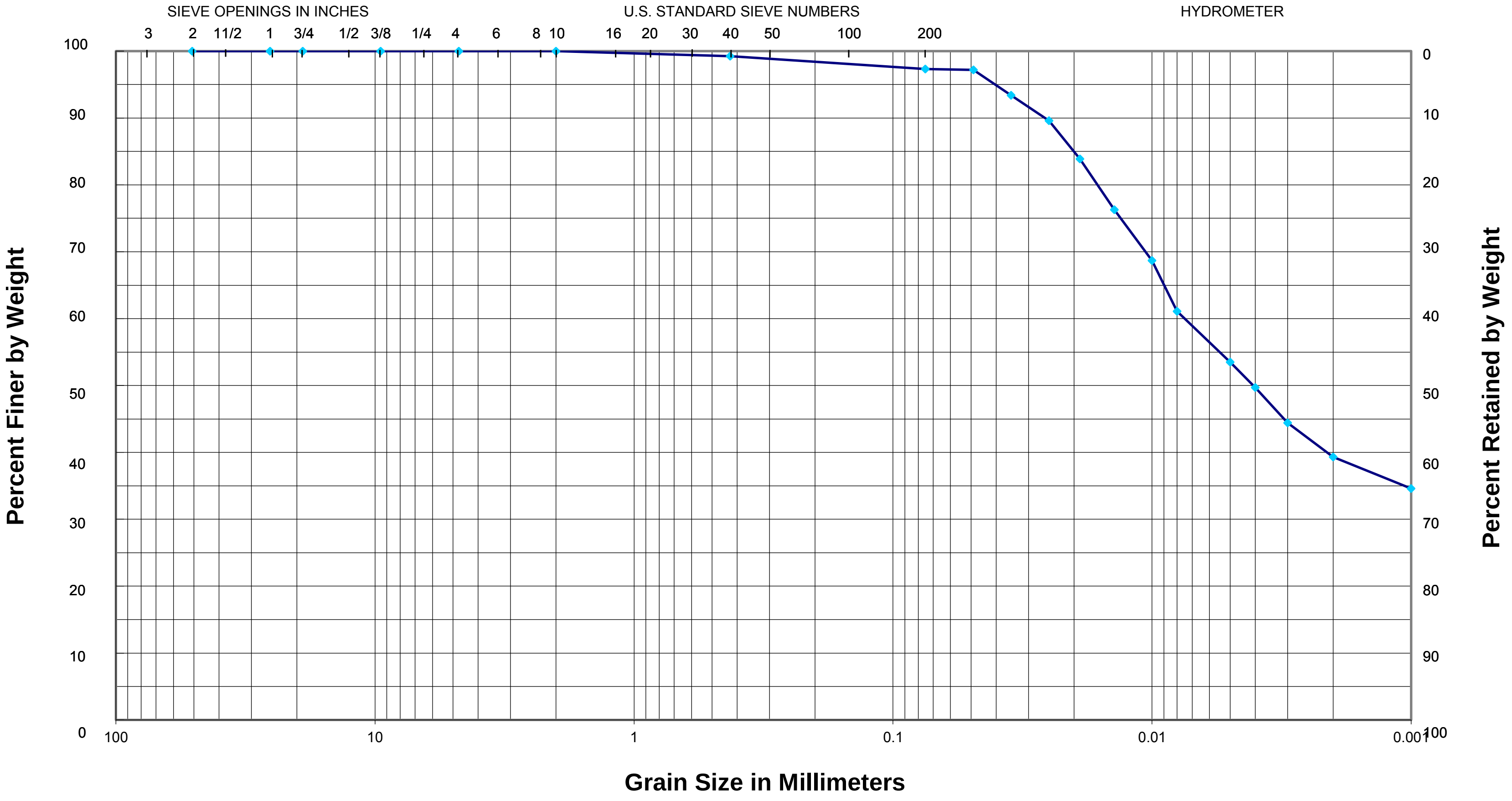
Sample: Boring 3, 49-50 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

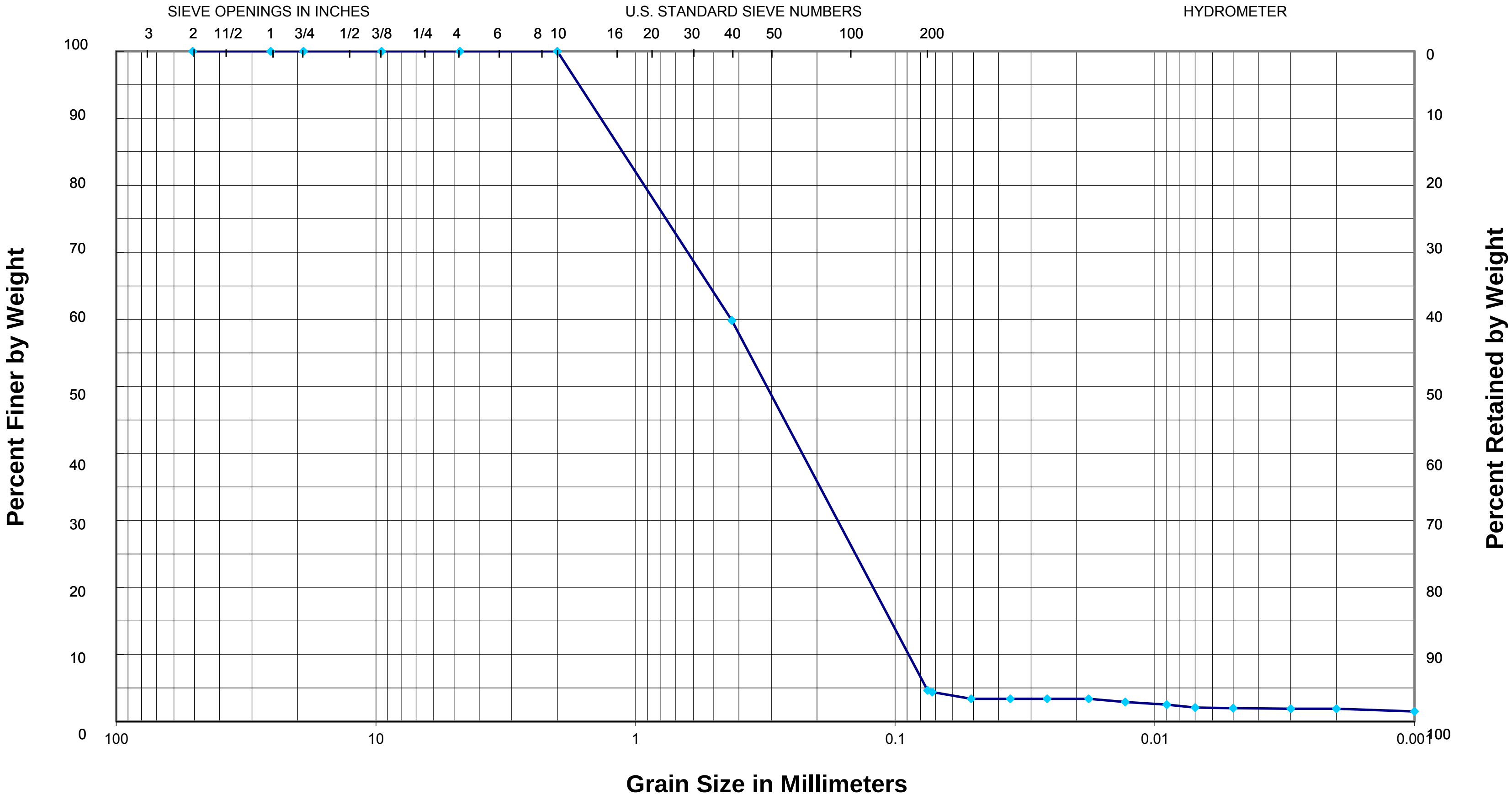
Sample: Boring 4, 7-7.5 ft
Properties: G_s = 2.700; LL = 46, PL = 19, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE

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



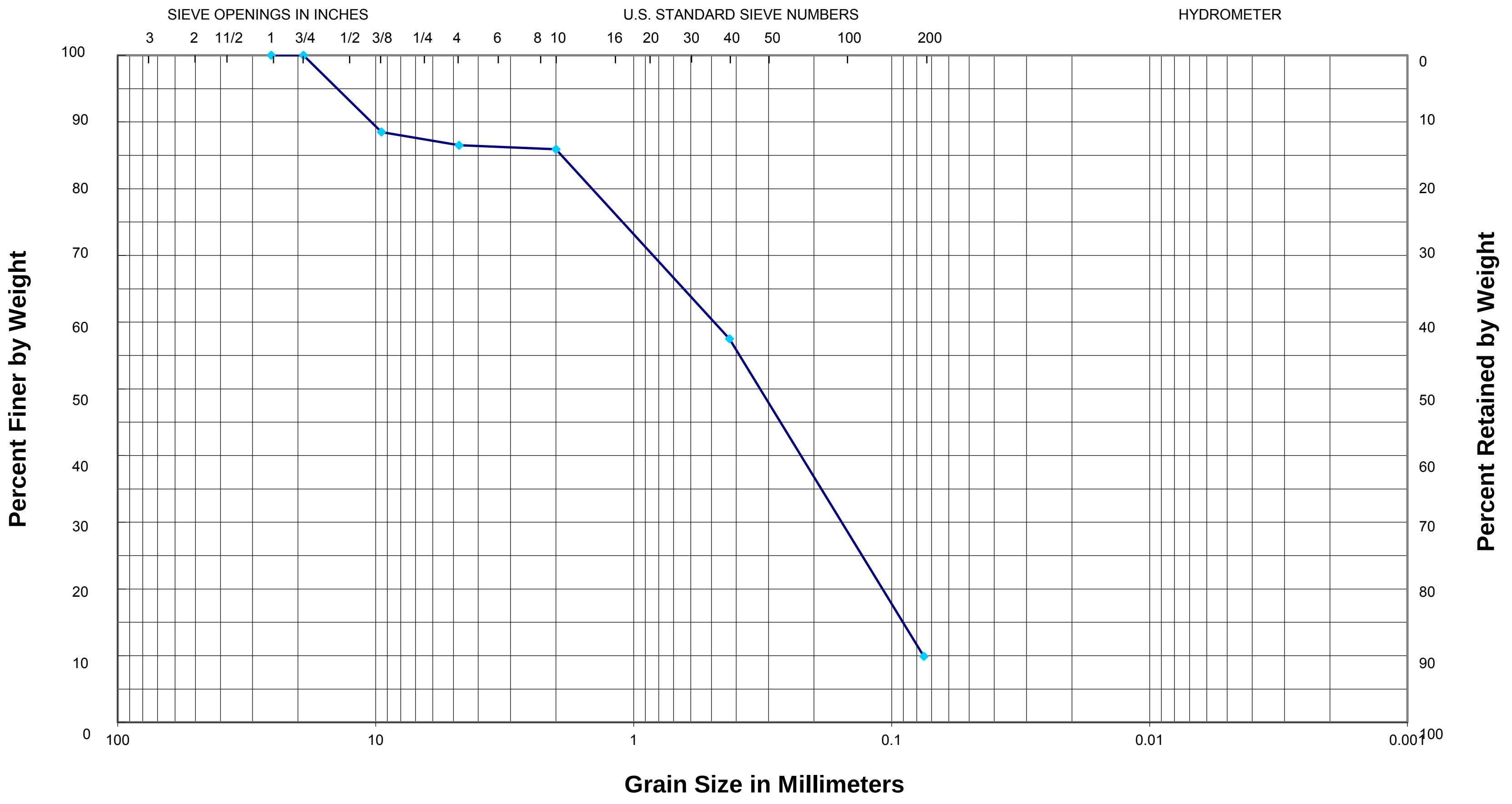
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 4, 14-15 ft
Properties: G_s = 2.653; Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 4, 53.5-54 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand w/a little fine gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

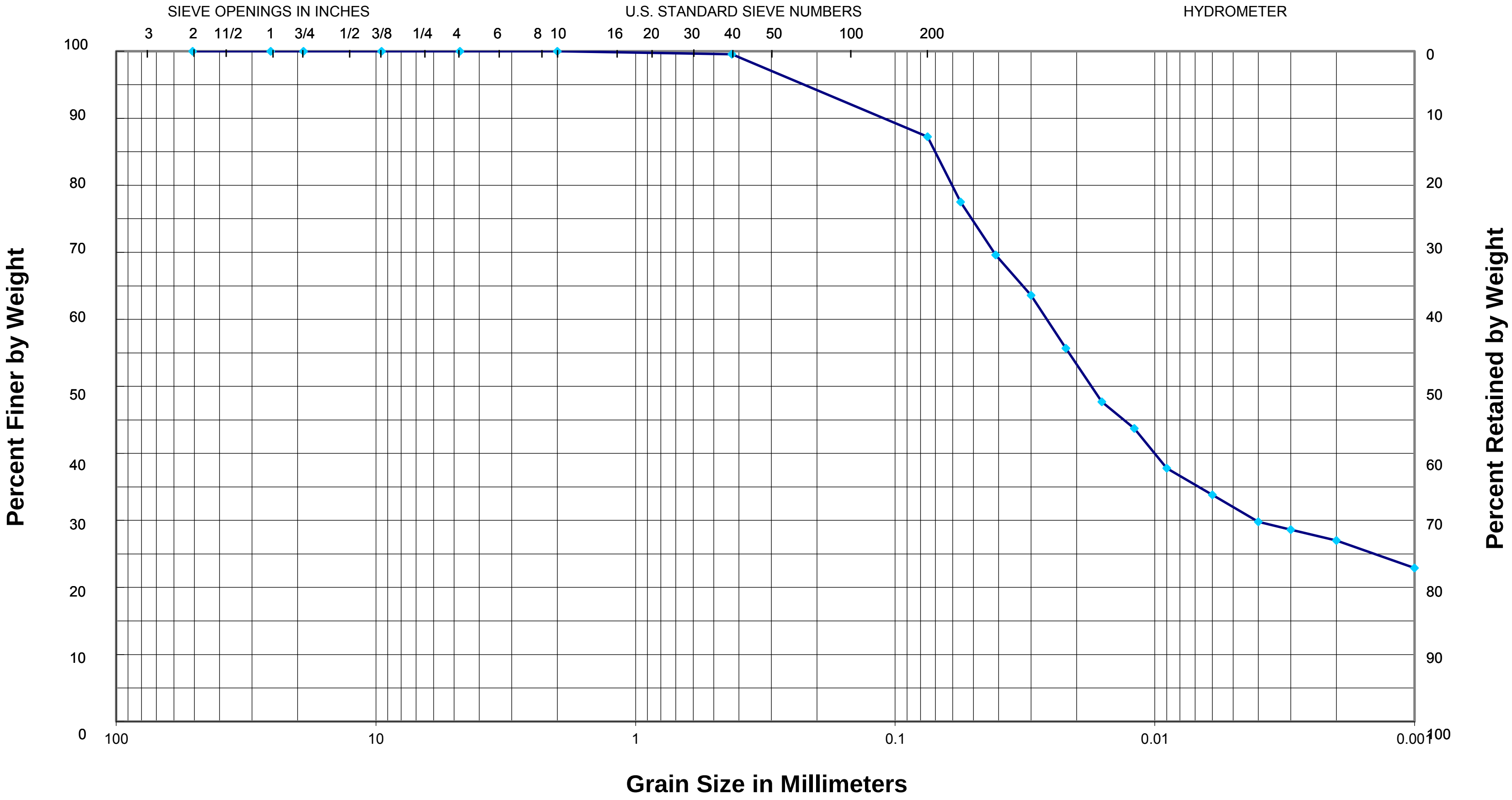
Sample: Boring 4, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Gray sandy fine gravel
Classification: USCS = GW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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



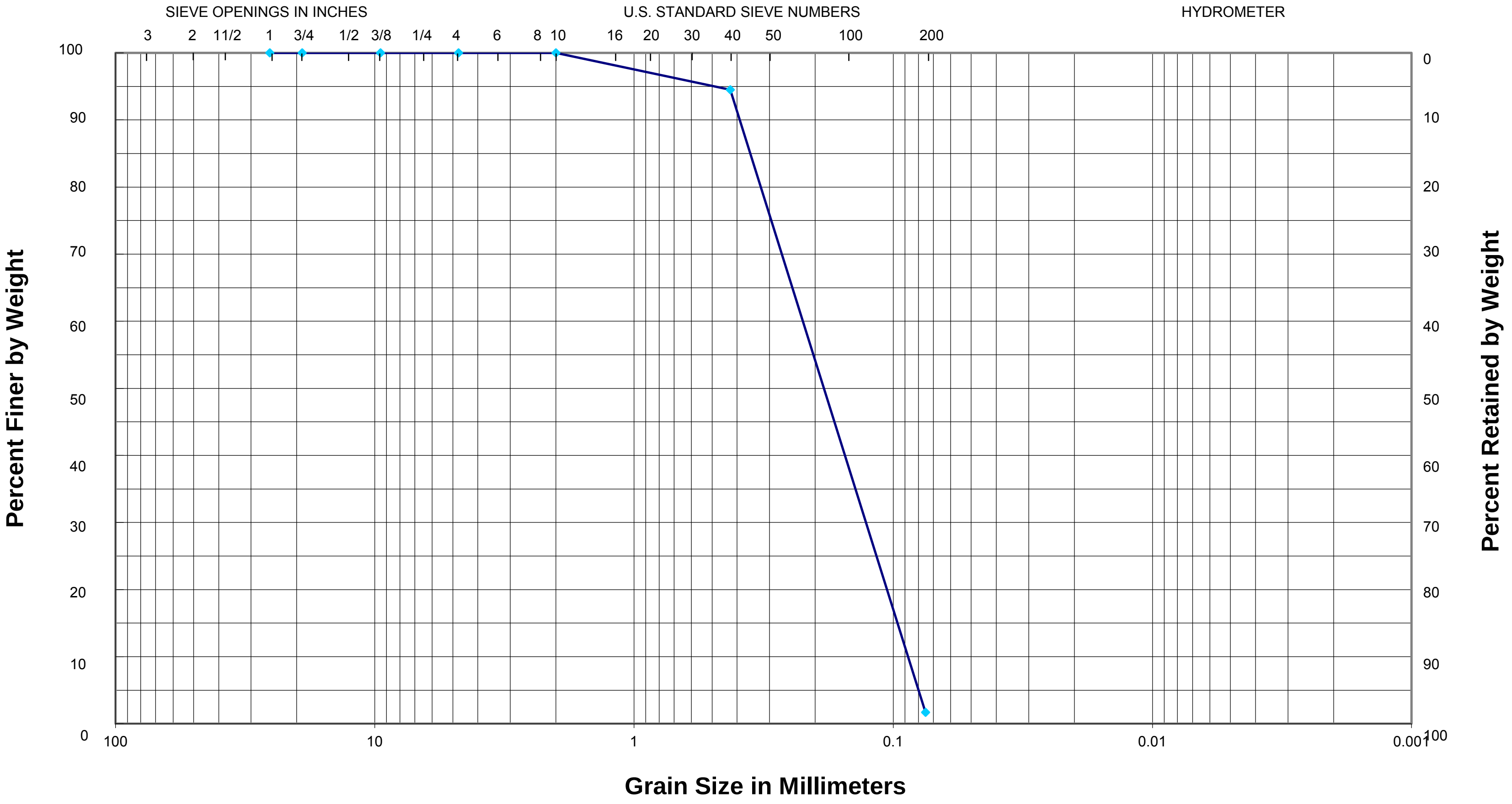
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 5, 3-3.5 ft
Properties: G_s = 2.637; LL = 33, PL = 17, PI = 16

Description: Dark gray and brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



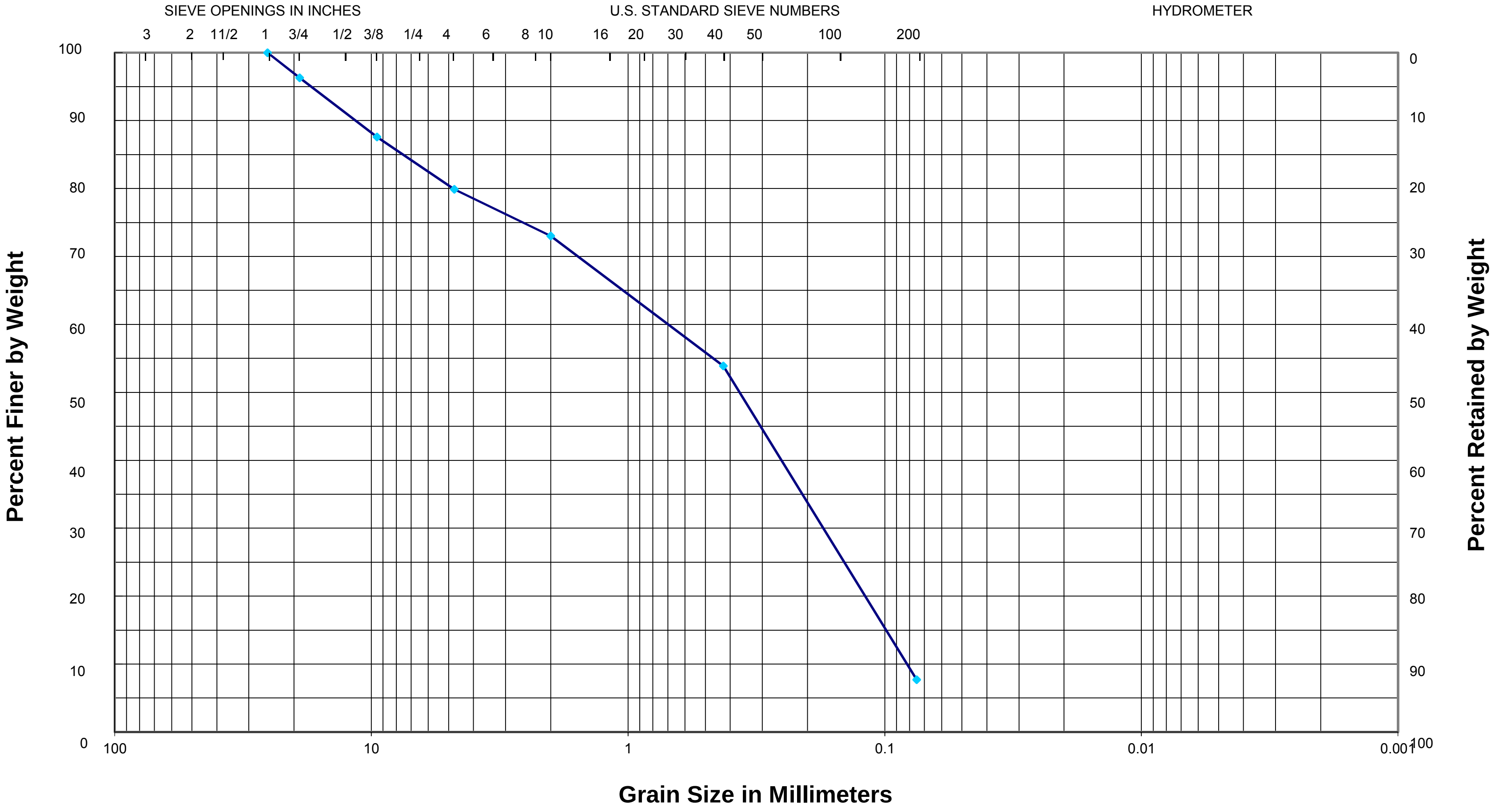
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 5, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray and tan fine sand w/trace medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



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<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

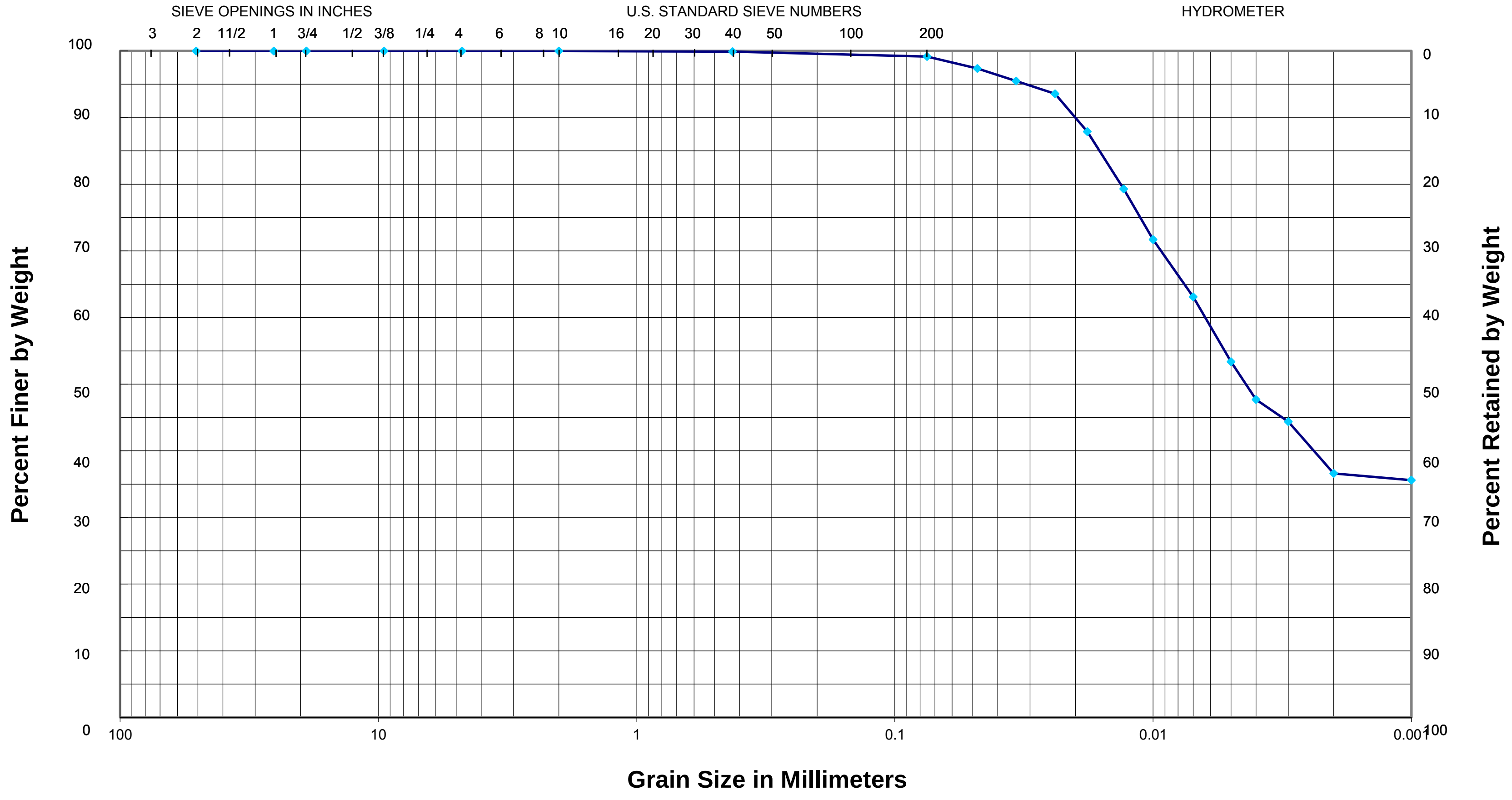
Sample: Boring 5, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/fine to coarse gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



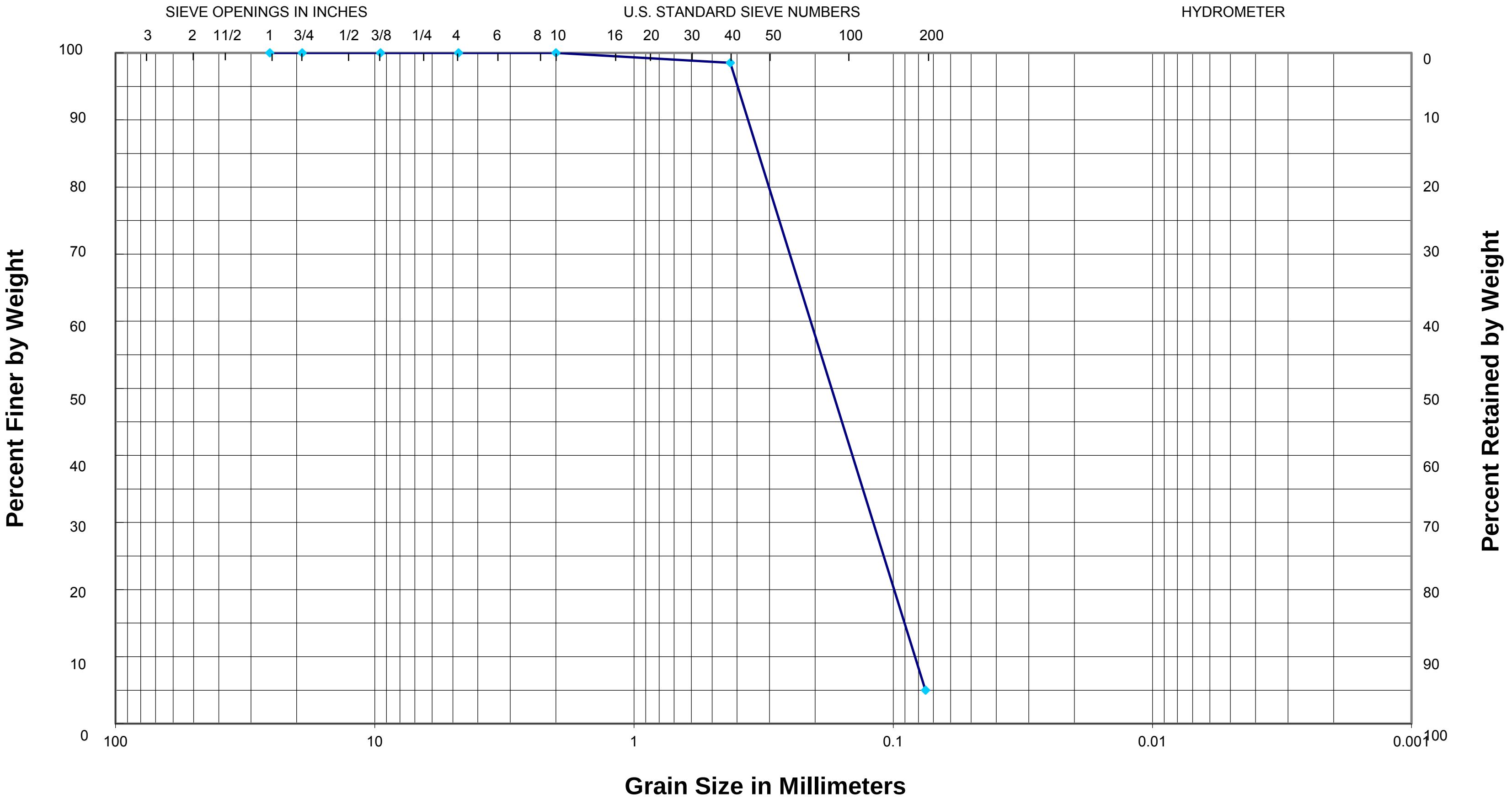
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 6, 3-3.5 ft
Properties: G_s = 2.707; LL = 48, PL = 21, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 6, 19-20 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

<table>
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<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Sample: Boring 6, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand, silty w/some fine gravel
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

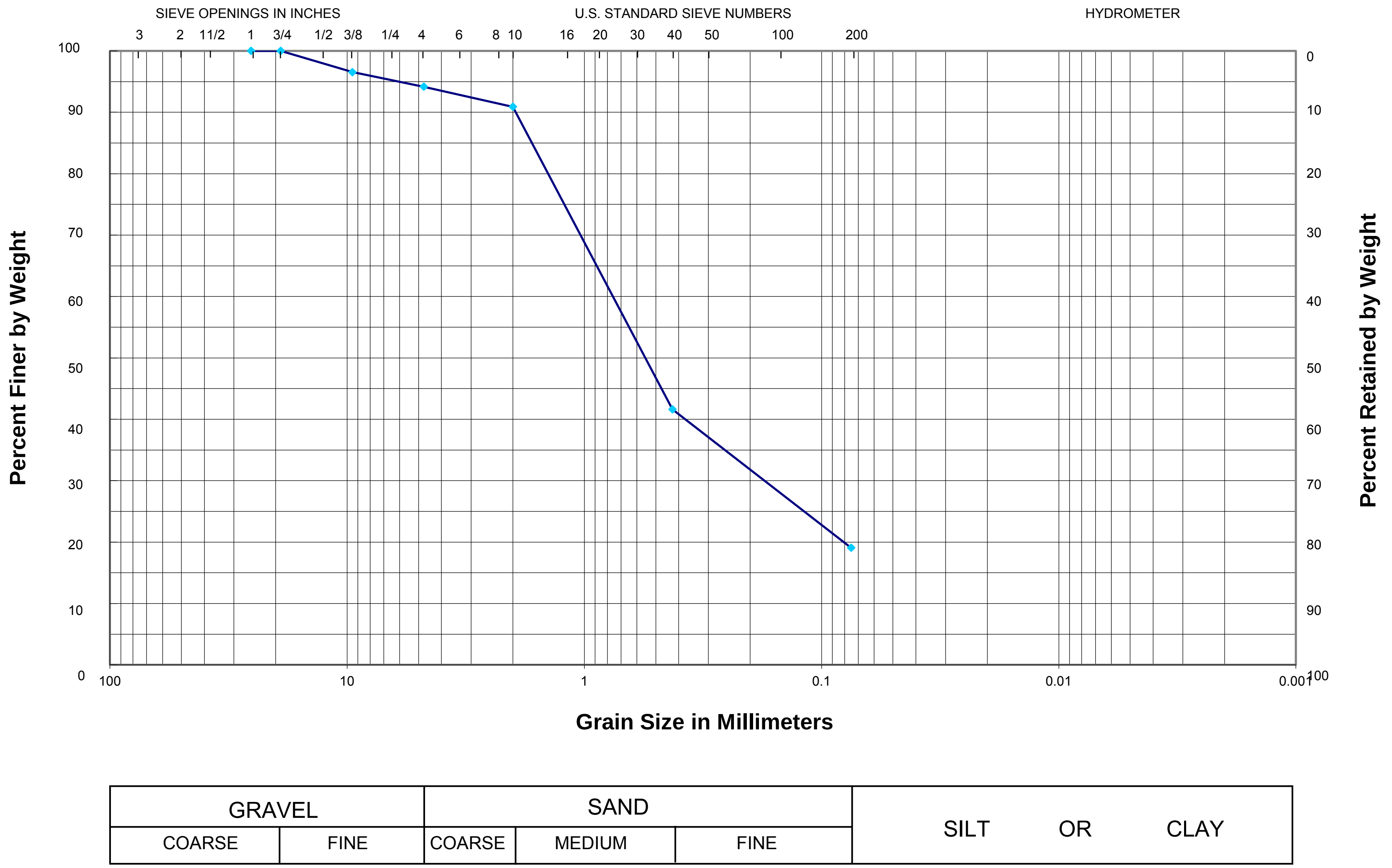
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 7, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

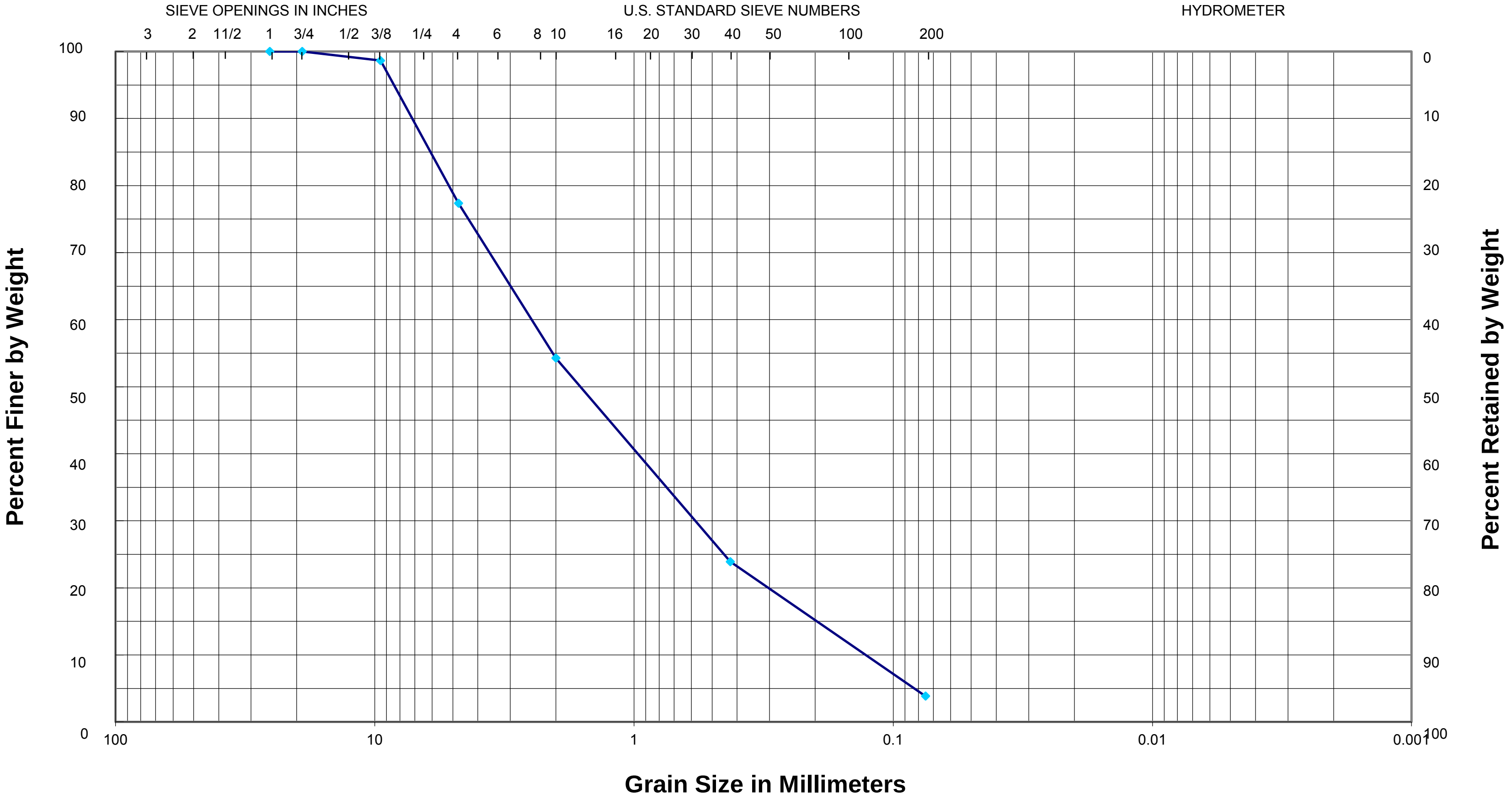


Sample: Boring 7, 43.5-44.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand, silty
Classification: USCS = SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



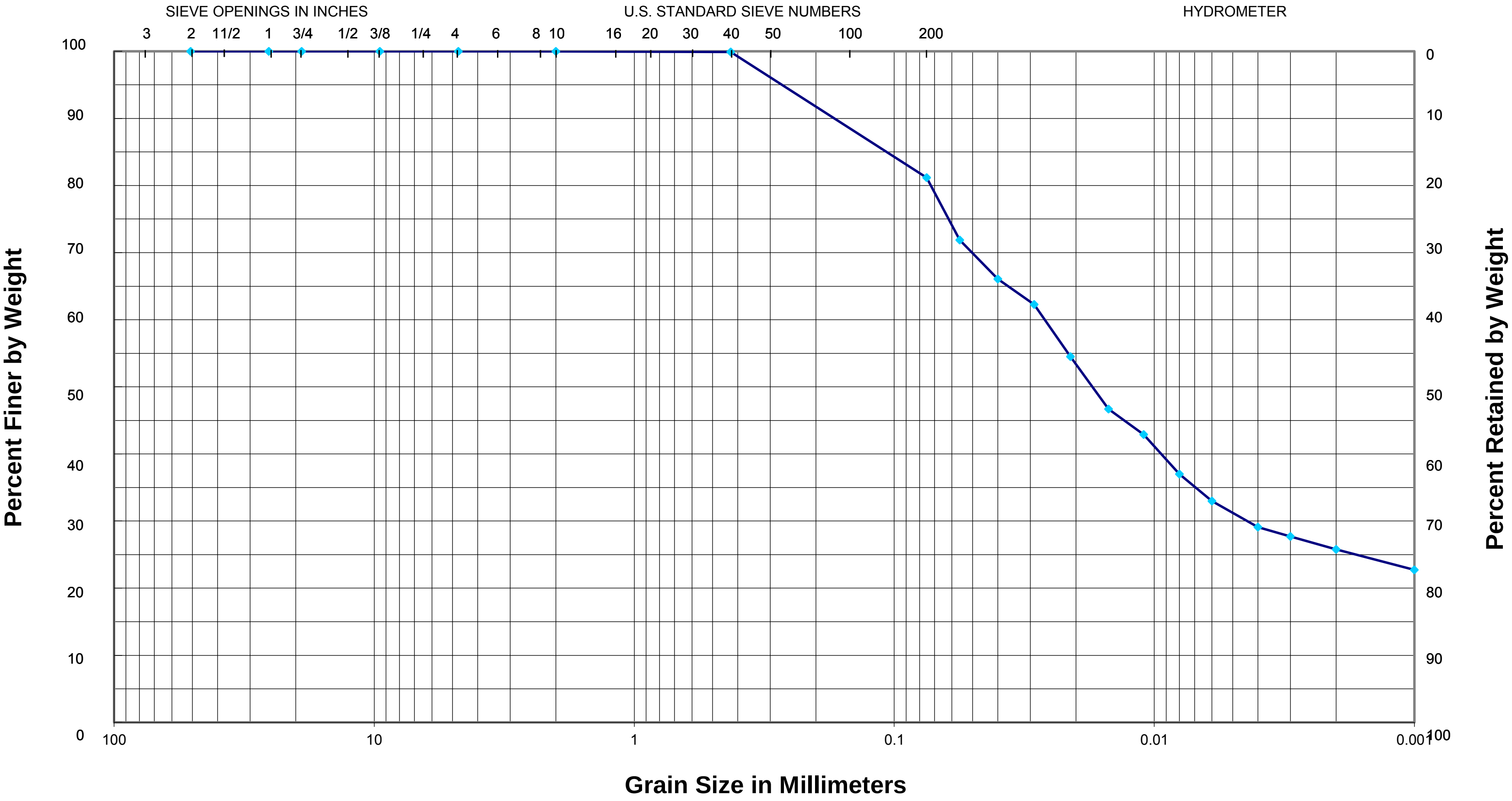
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 7, 88.5-89 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SW; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



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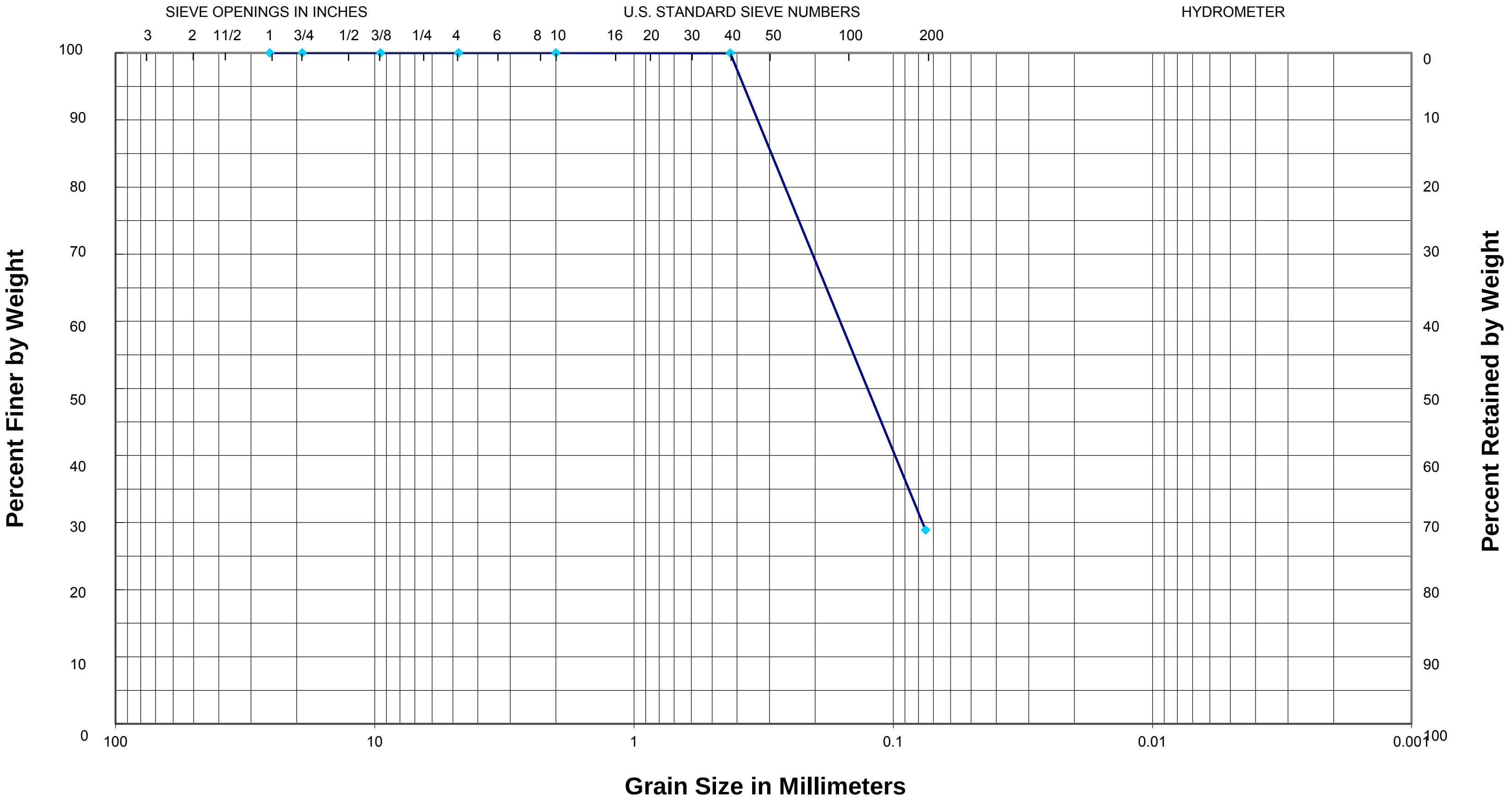
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Sample: Boring 8, 7-7.5 ft
Properties: G_s = 2.697; LL = 37, PL = 15, PI = 12

Description: Gray and brown silty clay, sandy
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



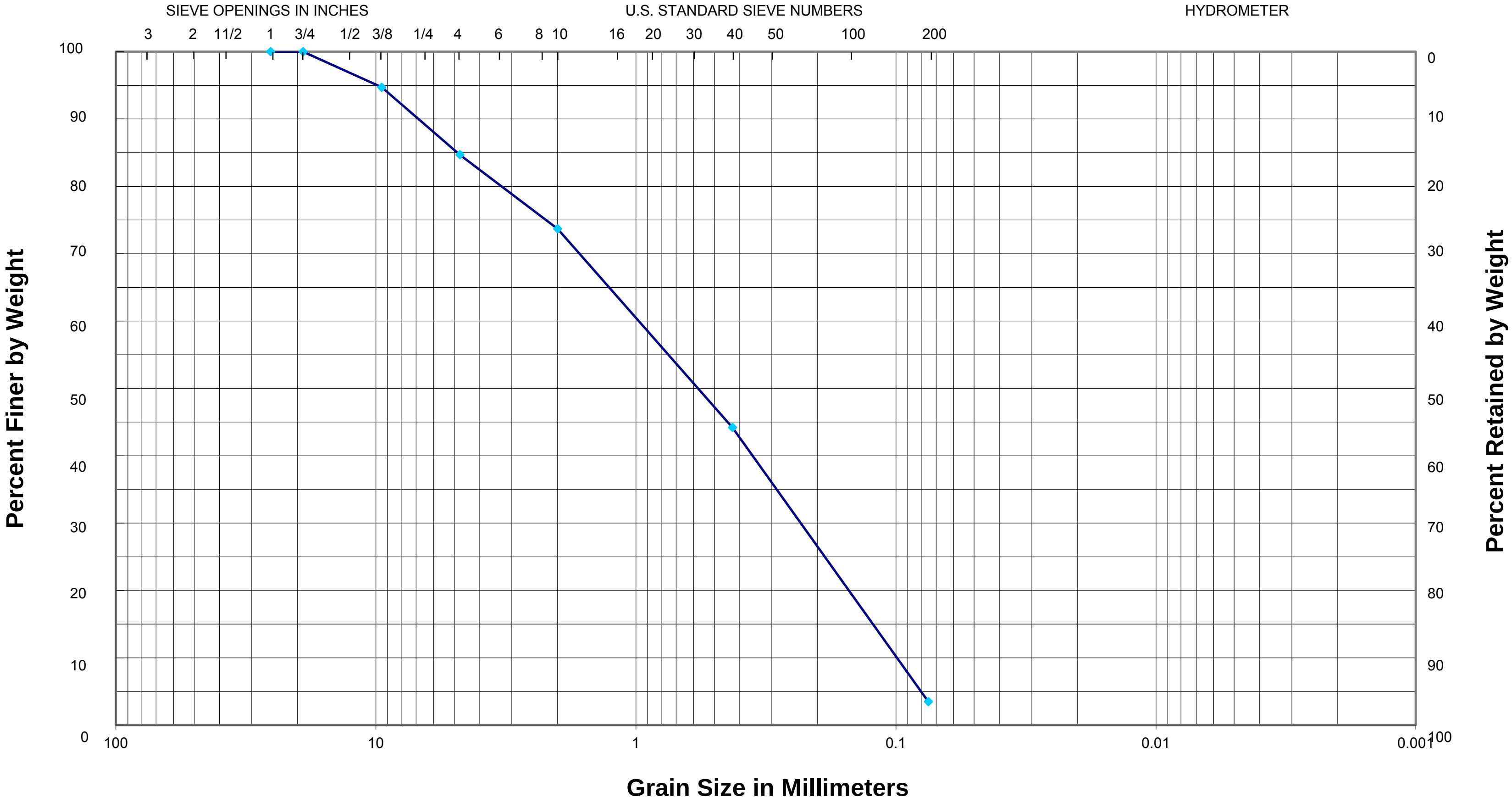
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine to coarse sand w/fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

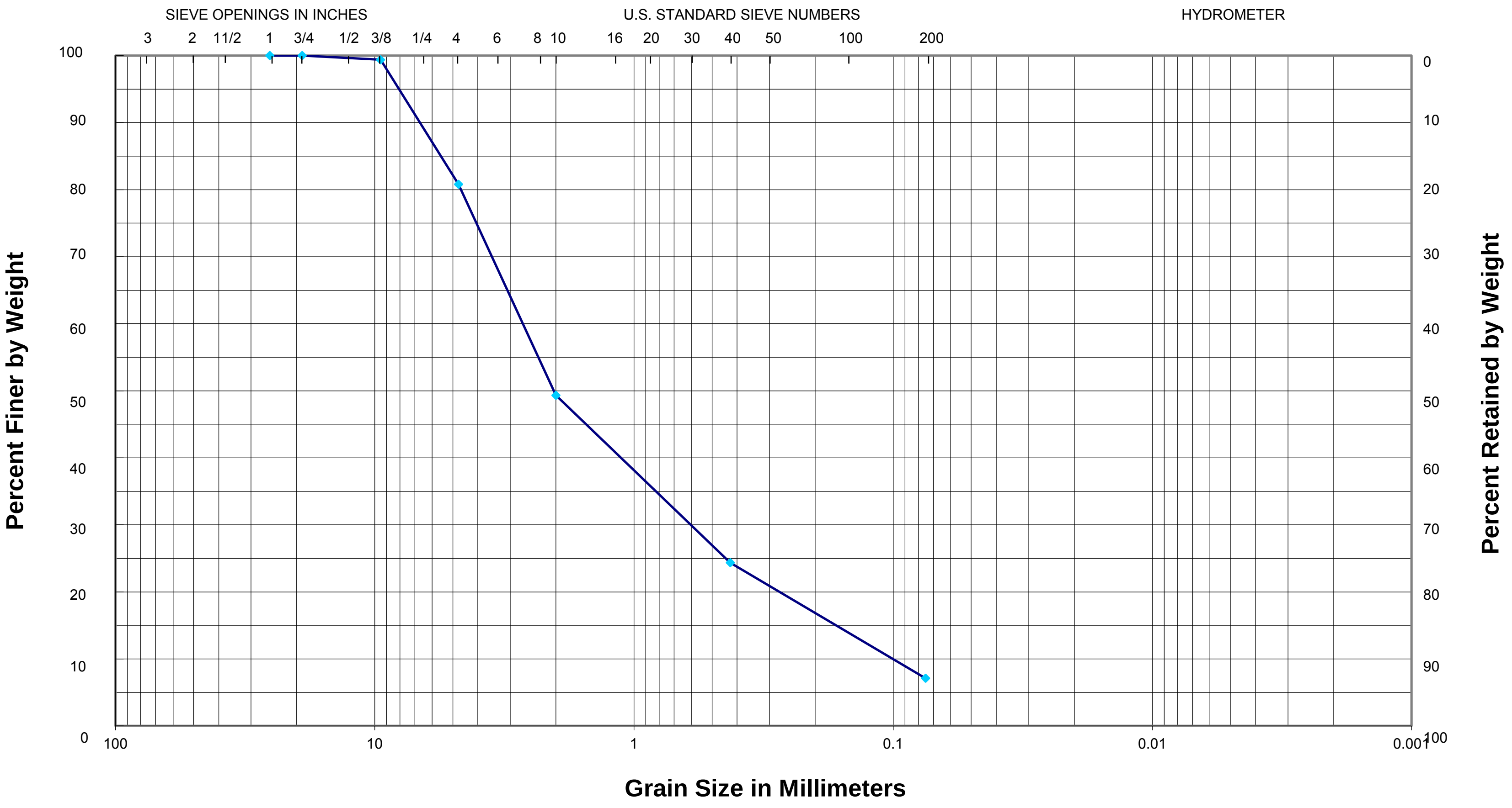
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 9, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



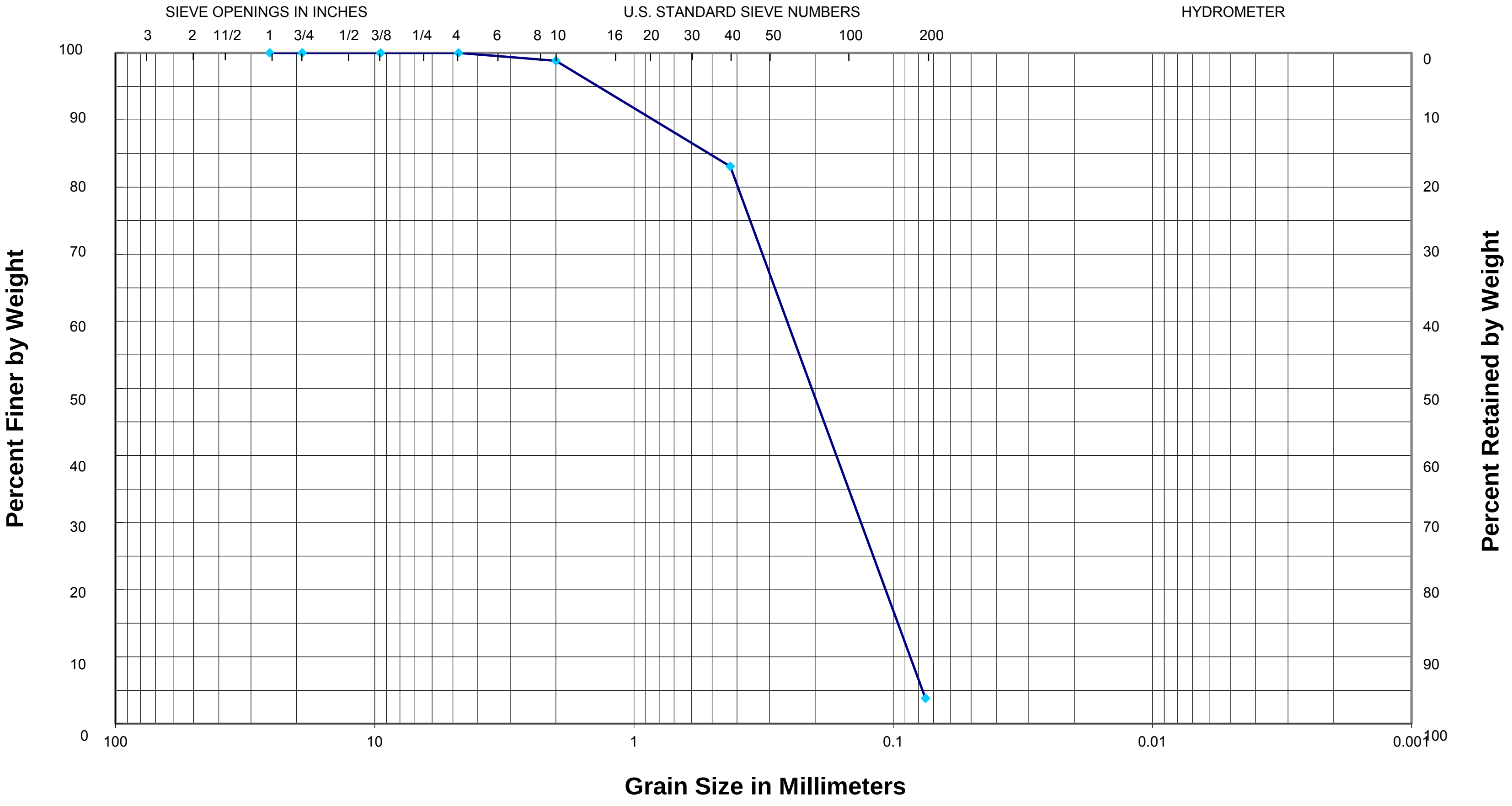
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 9, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/fine gravel
Classification: USCS = SP-SM; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



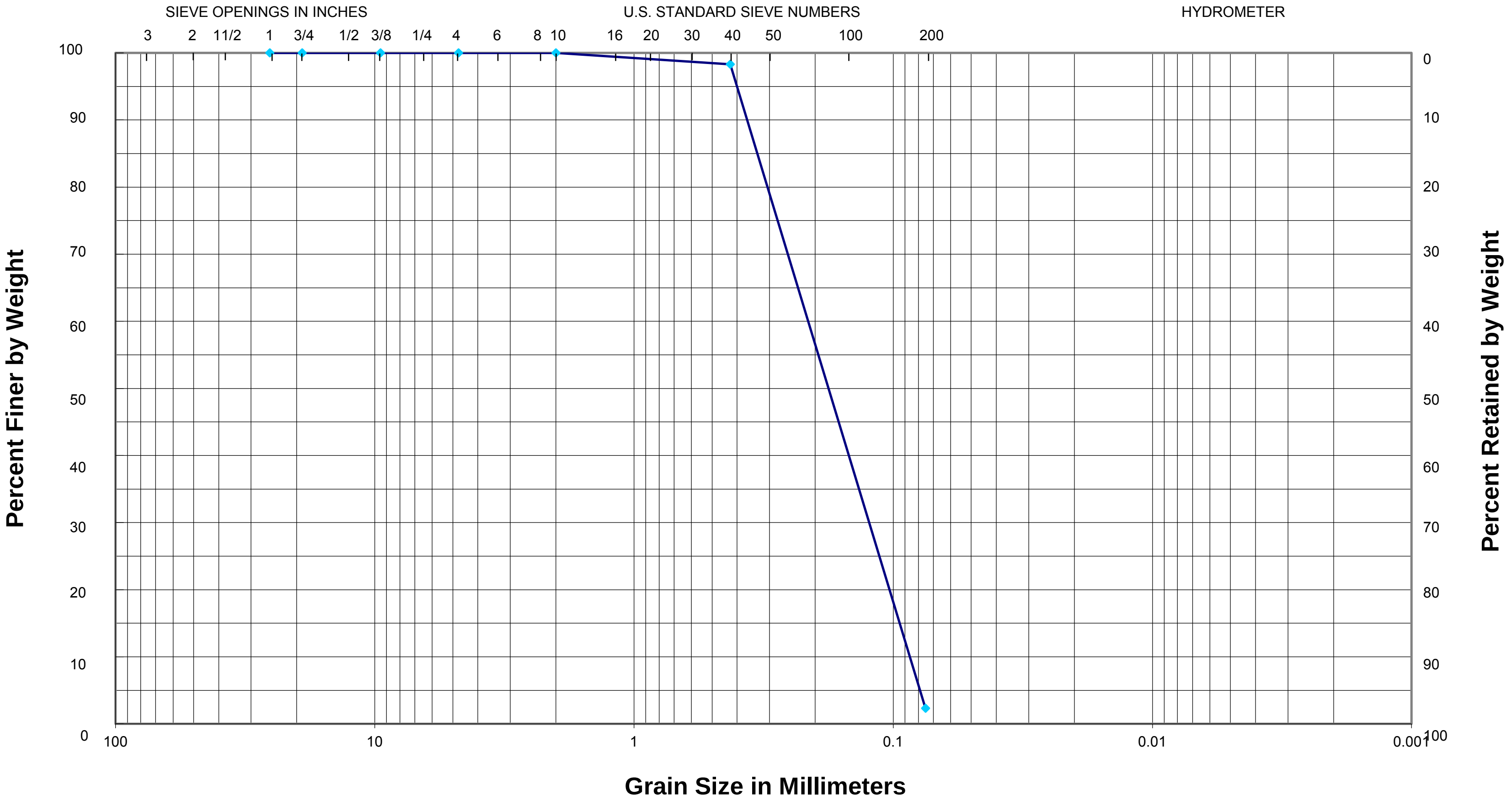
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 10, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



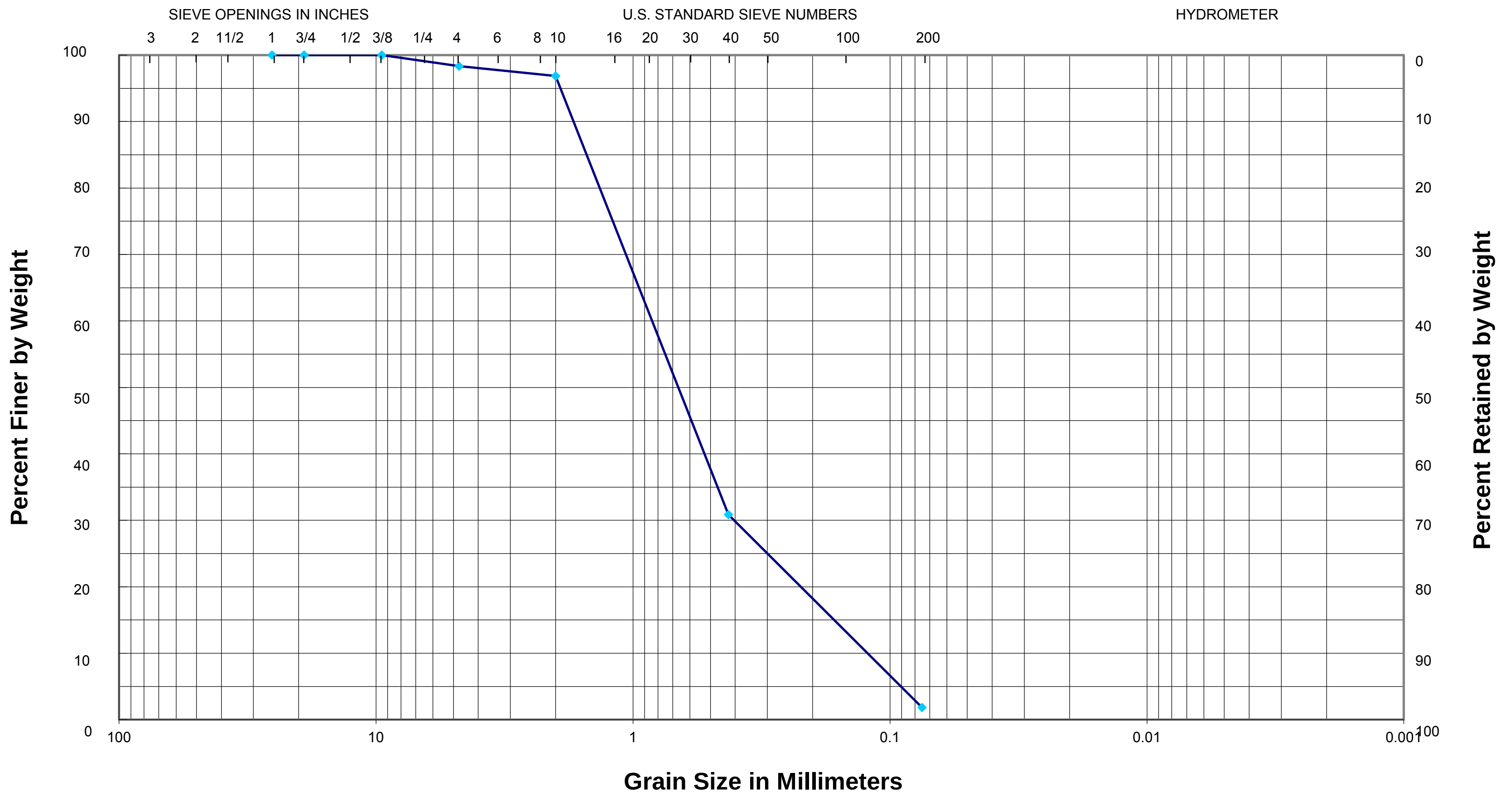
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 11, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 11, 48.5-49.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 12, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray sandy fine gravel
Classification: USCS = GW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13, 6.5-7.5 ft
Properties: G_s = 2.661; Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

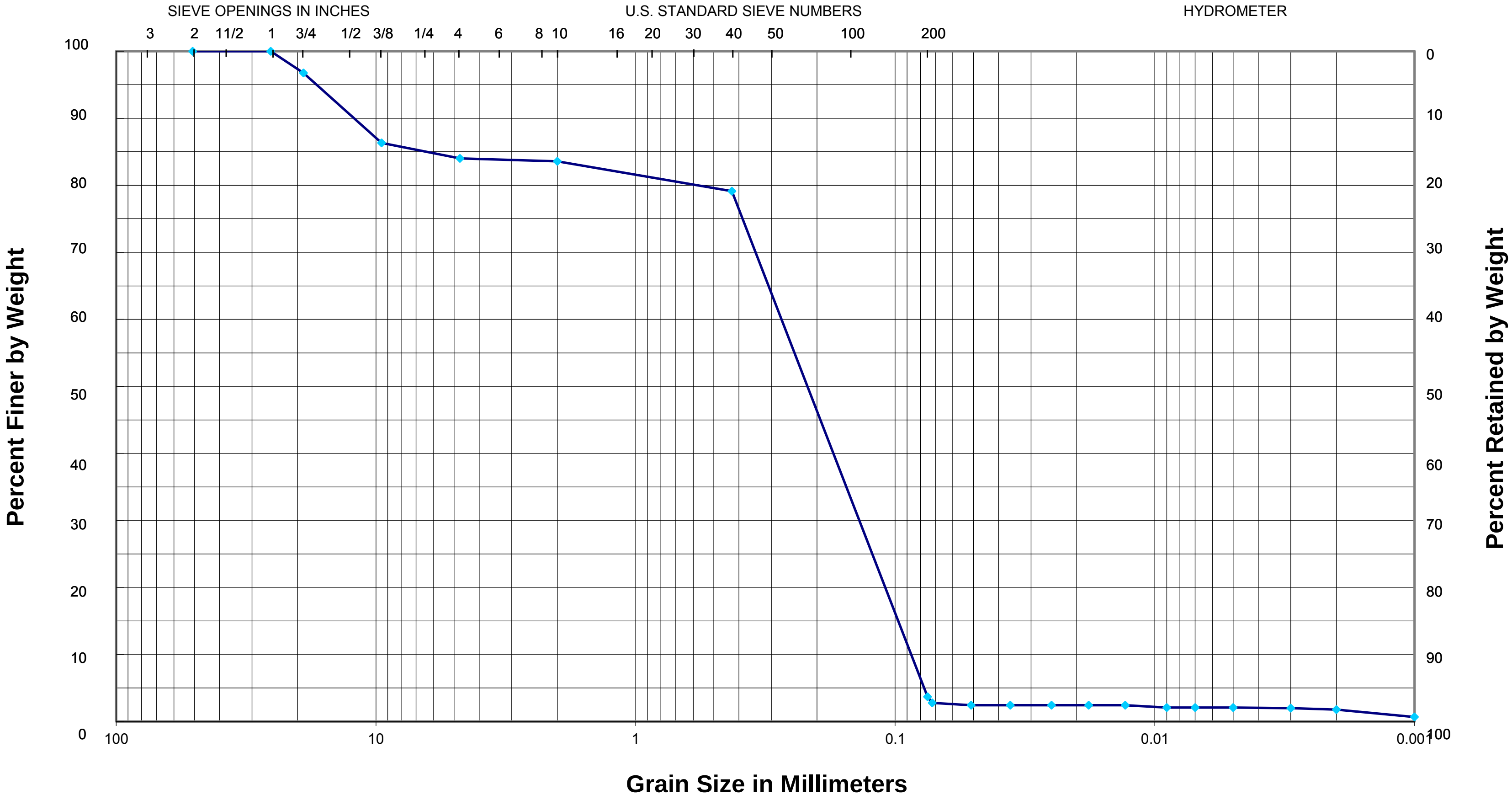
Sample: Boring 13A, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13A, 19-20 ft
Properties: G_s = 2.654; Non-plastic

Description: Brown and tan fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

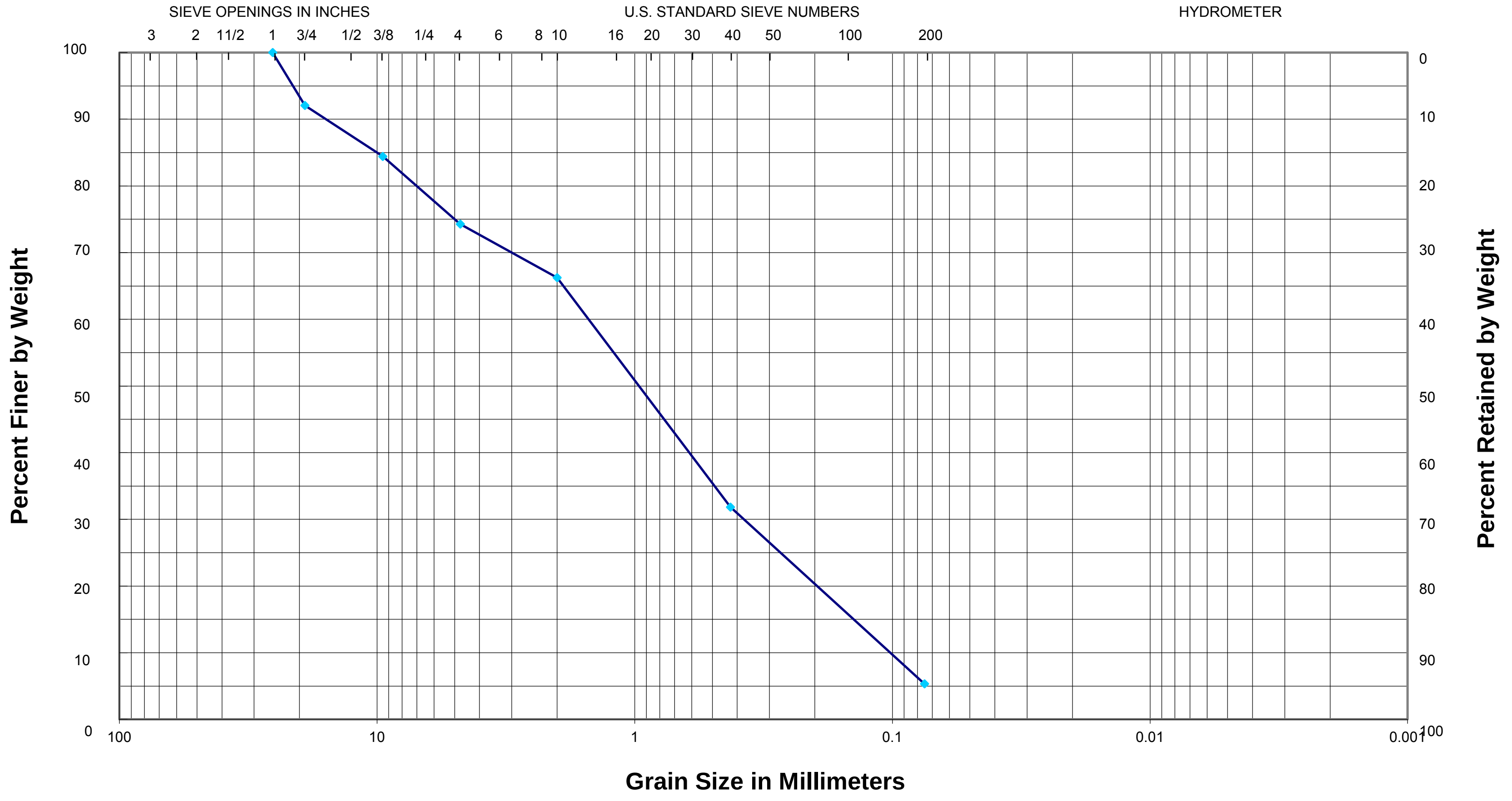
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 48.5-49.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/some fine gravel
Classification: USCS = SW-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/some fine to coarse gravel
Classification: USCS = SW-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

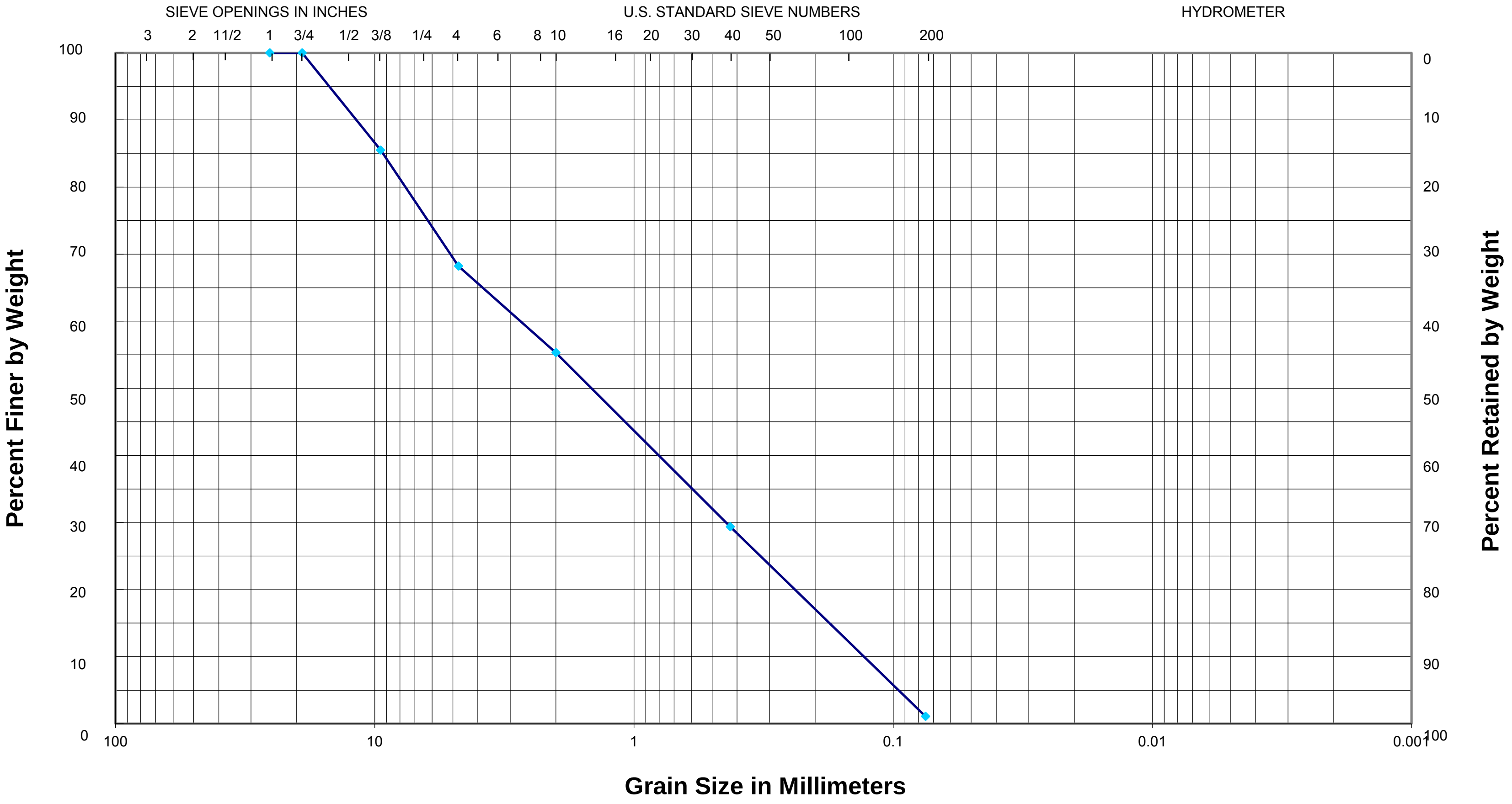
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 99-100 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

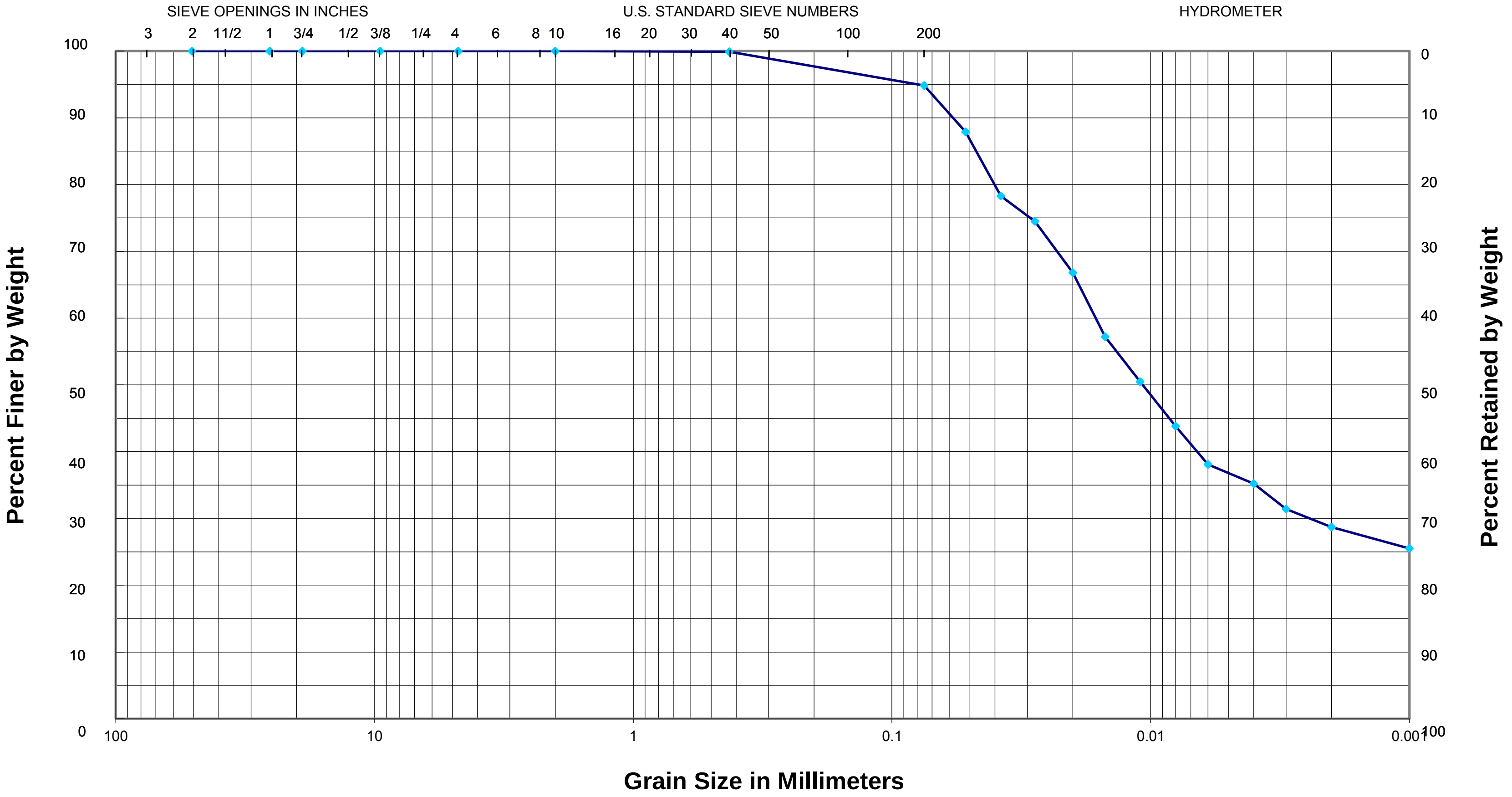
Sample: Boring 13A, 139-140 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

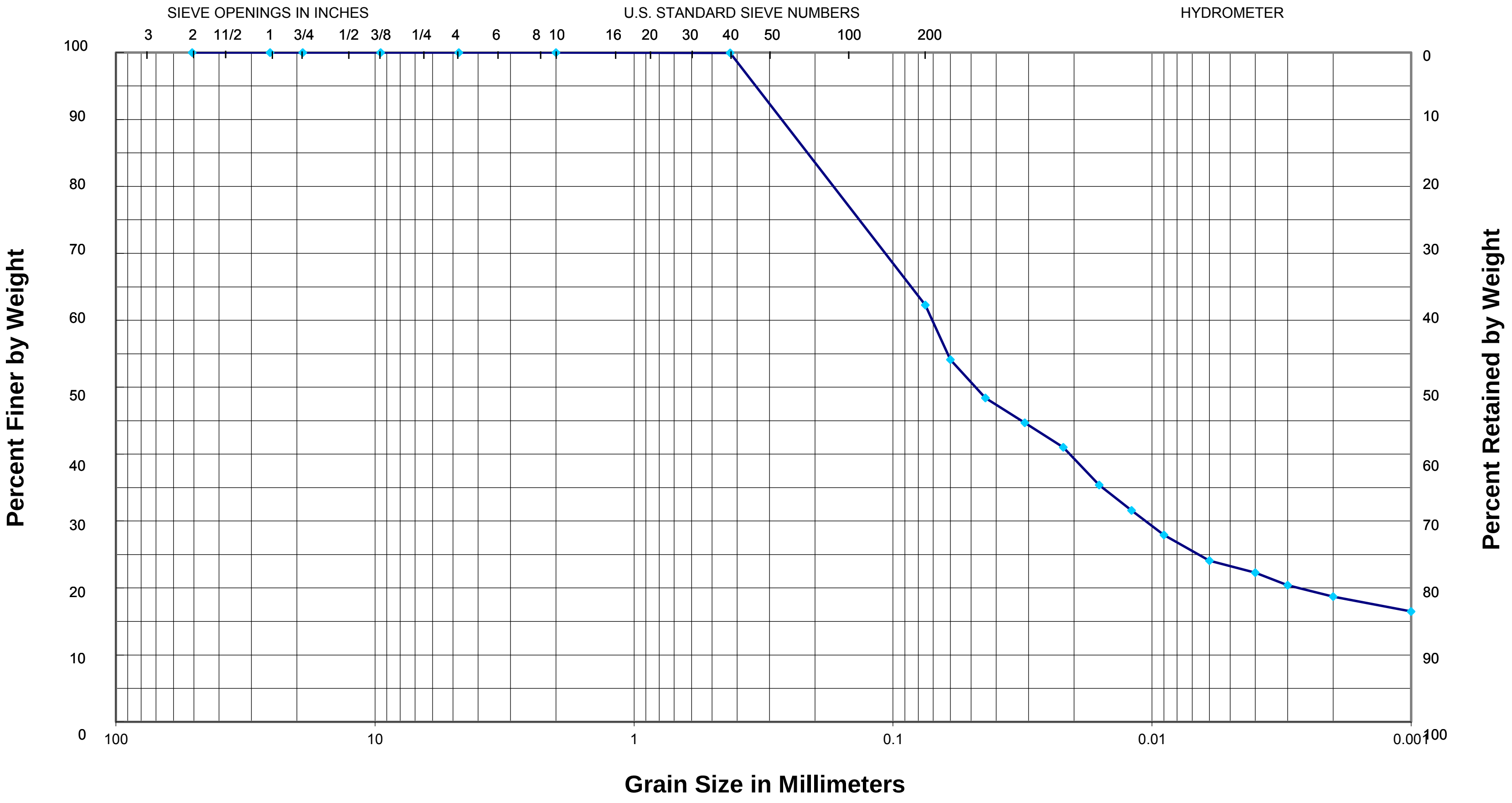
Sample: Boring 14, 4.5-5.5 ft
Properties: G_s = 2.669; LL = 34, PL = 19, PI = 15

Description: Brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

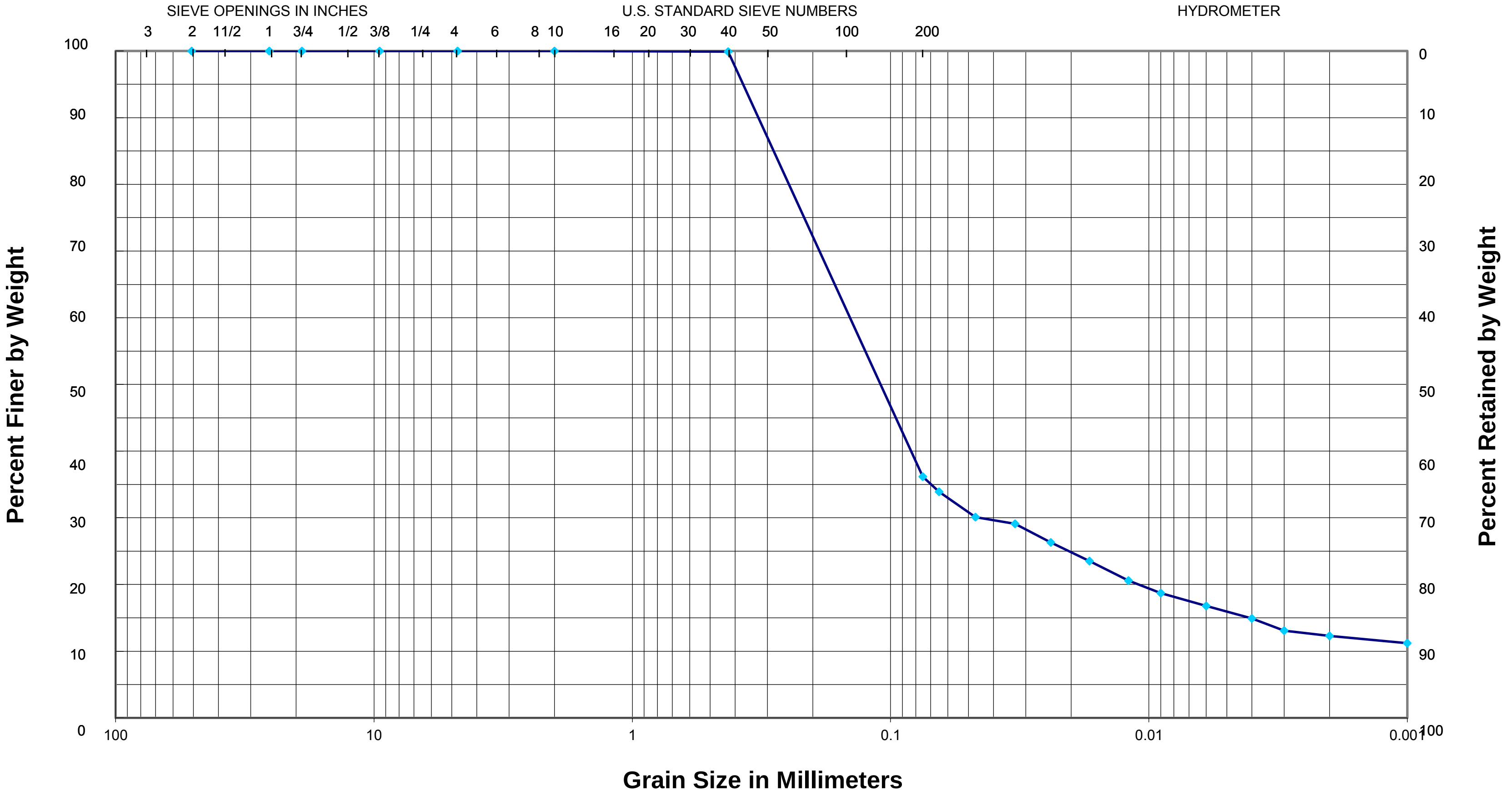
Sample: Boring 14, 14-15 ft
Properties: G_s = 2.668; LL = 22, PL = 17, PI = 5

Description: Gray clayey silt
Classification: USCS = CL-ML; AASHTO = A-4

13-017

GRAIN SIZE CURVE

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



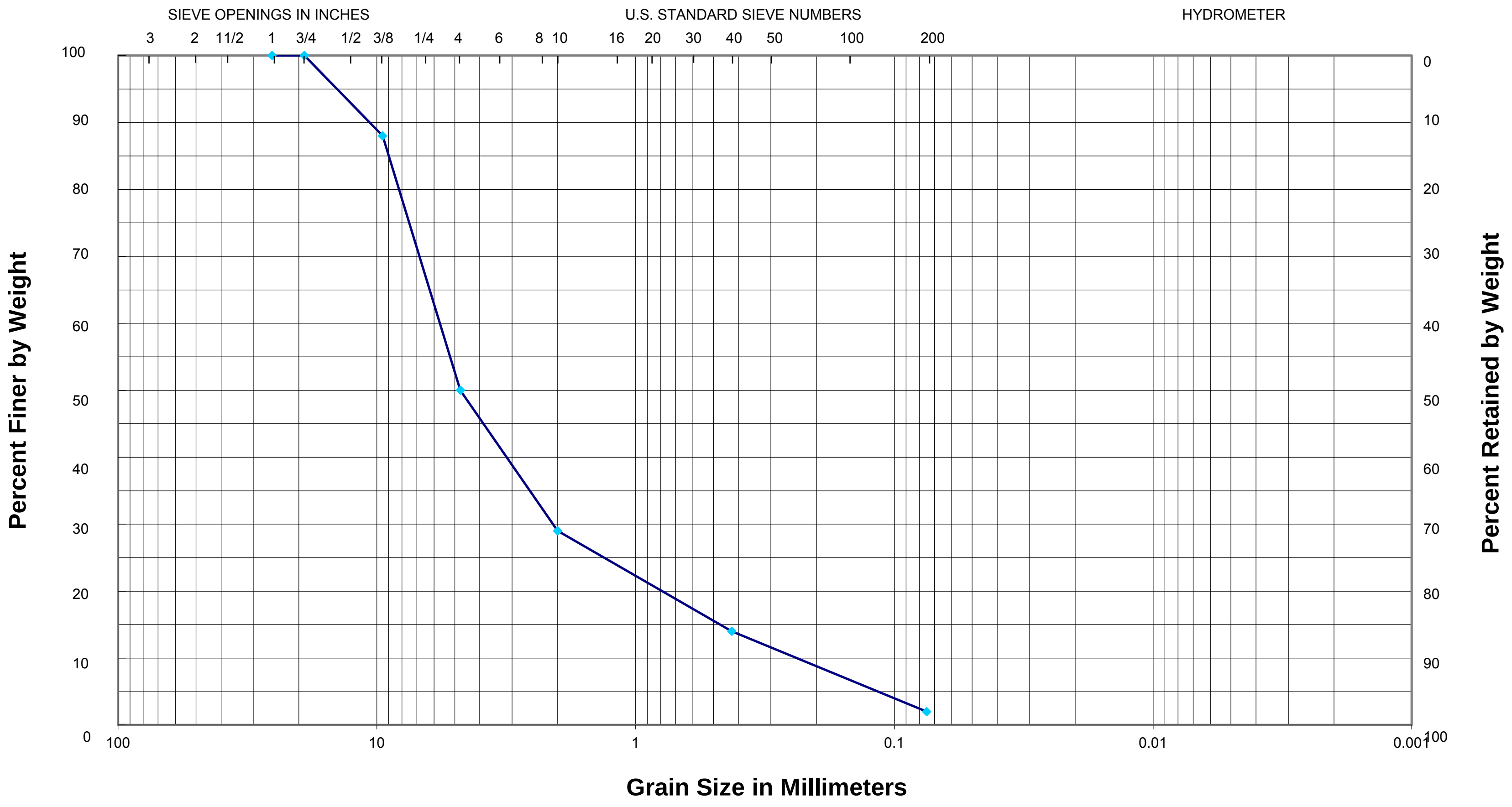
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 14, 24-24.5 ft
Properties: G_s = 2.668; Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-4

13-017

GRAIN SIZE CURVE



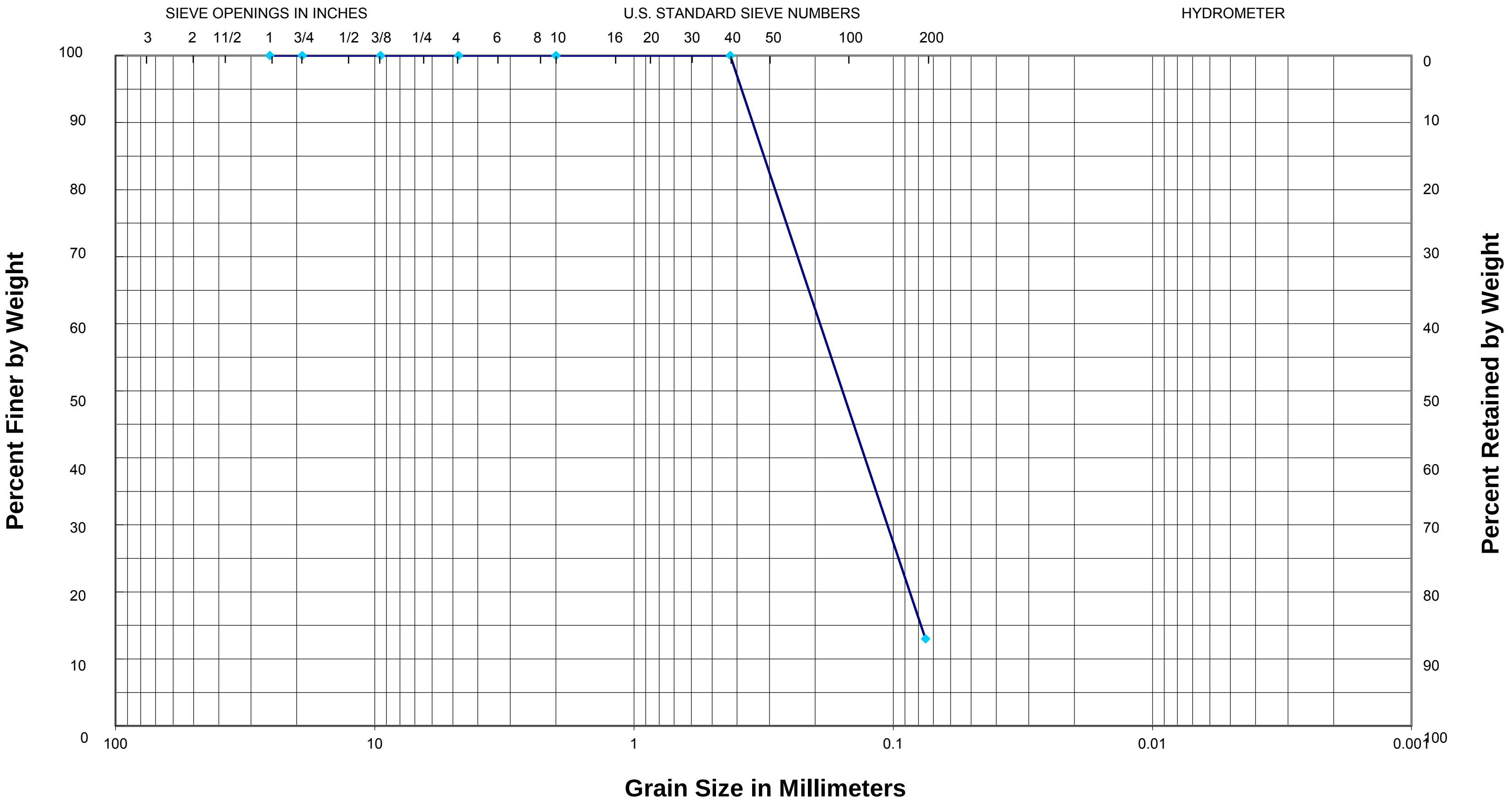
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 14, 100-118 ft (composite)
Atterberg Limits: Non-plastic

Description: Brown sandy fine gravel
Classification: USCS = GP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



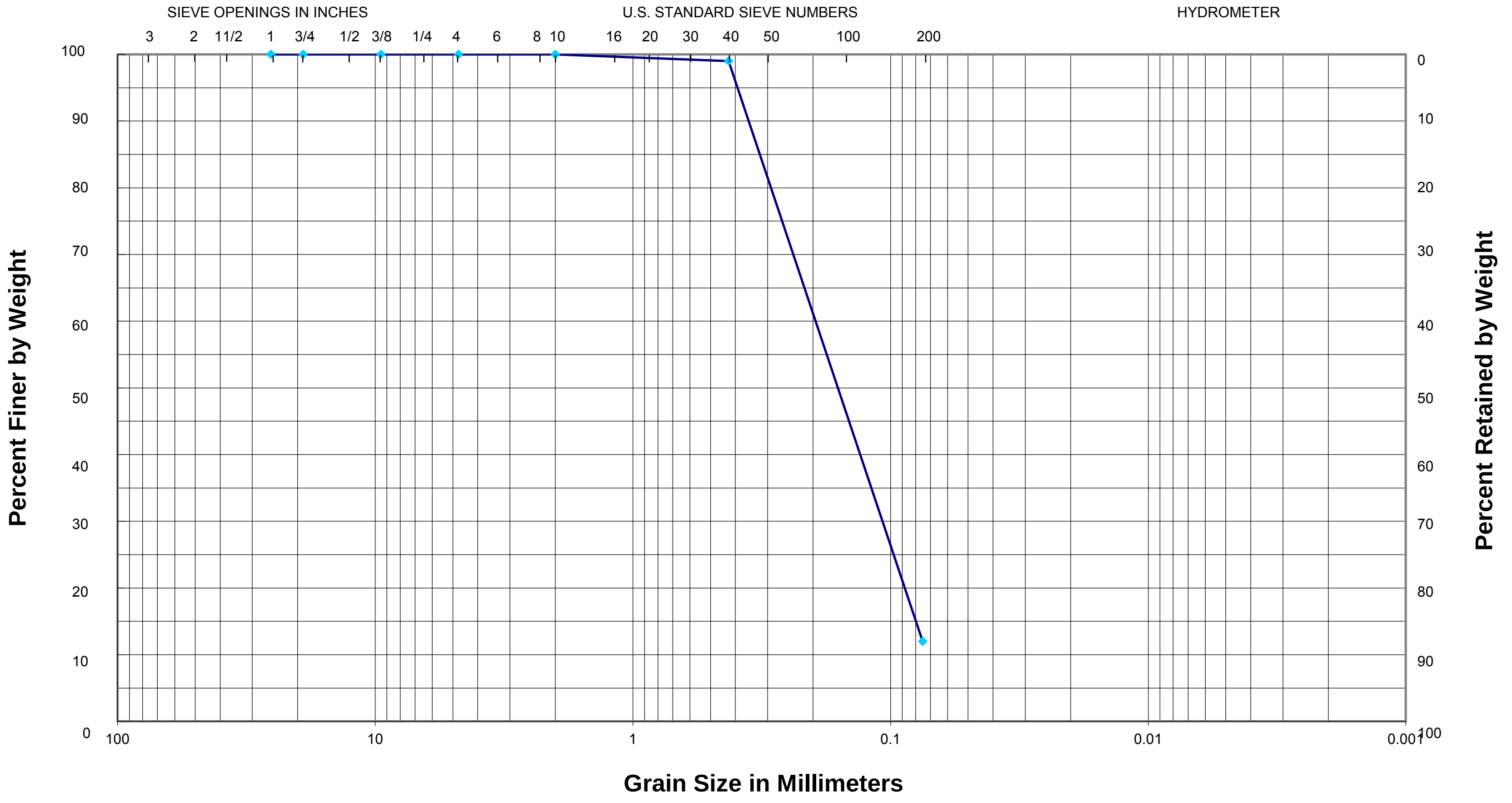
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 6.5-7.5 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



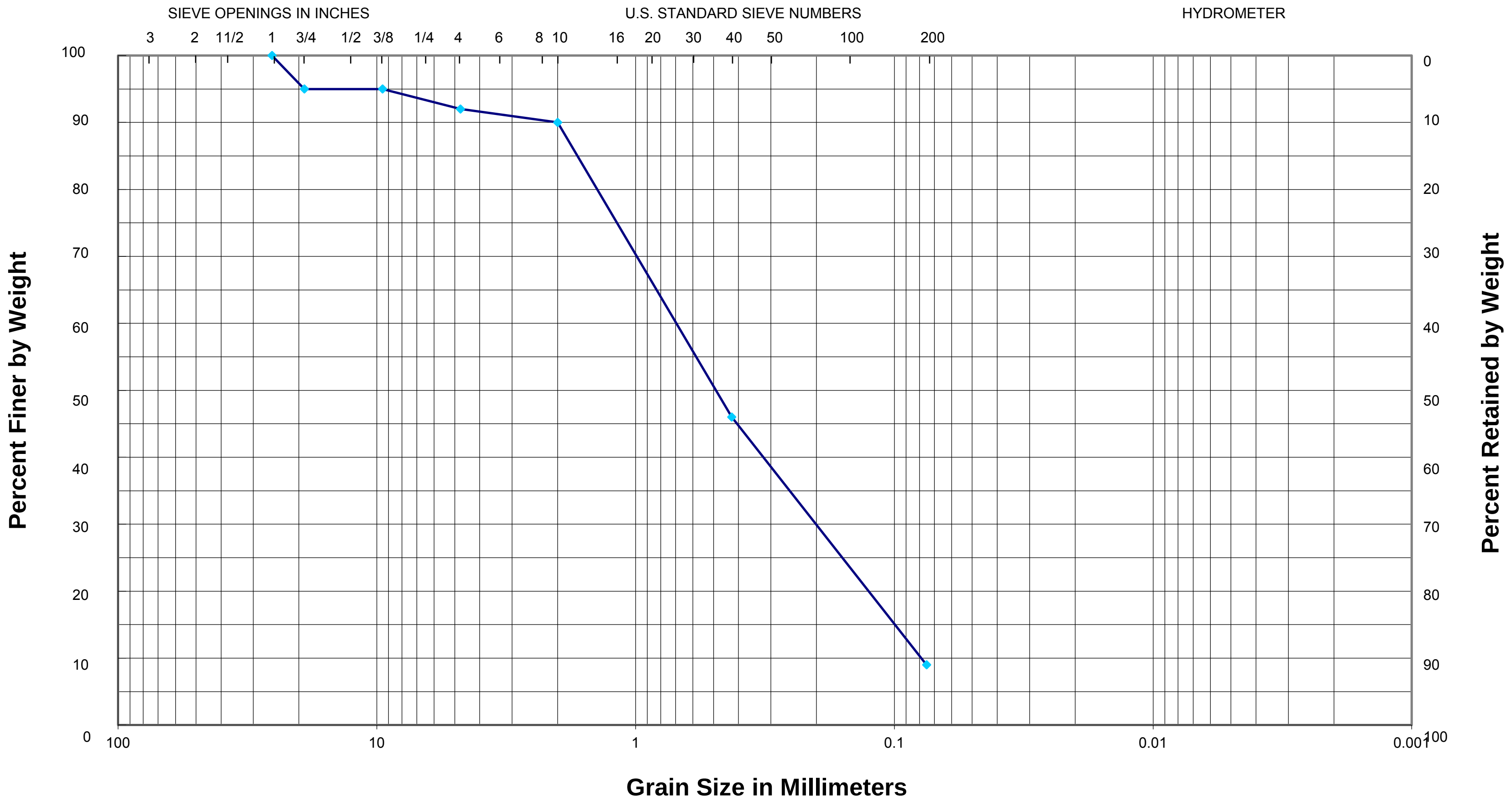
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 24-25 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



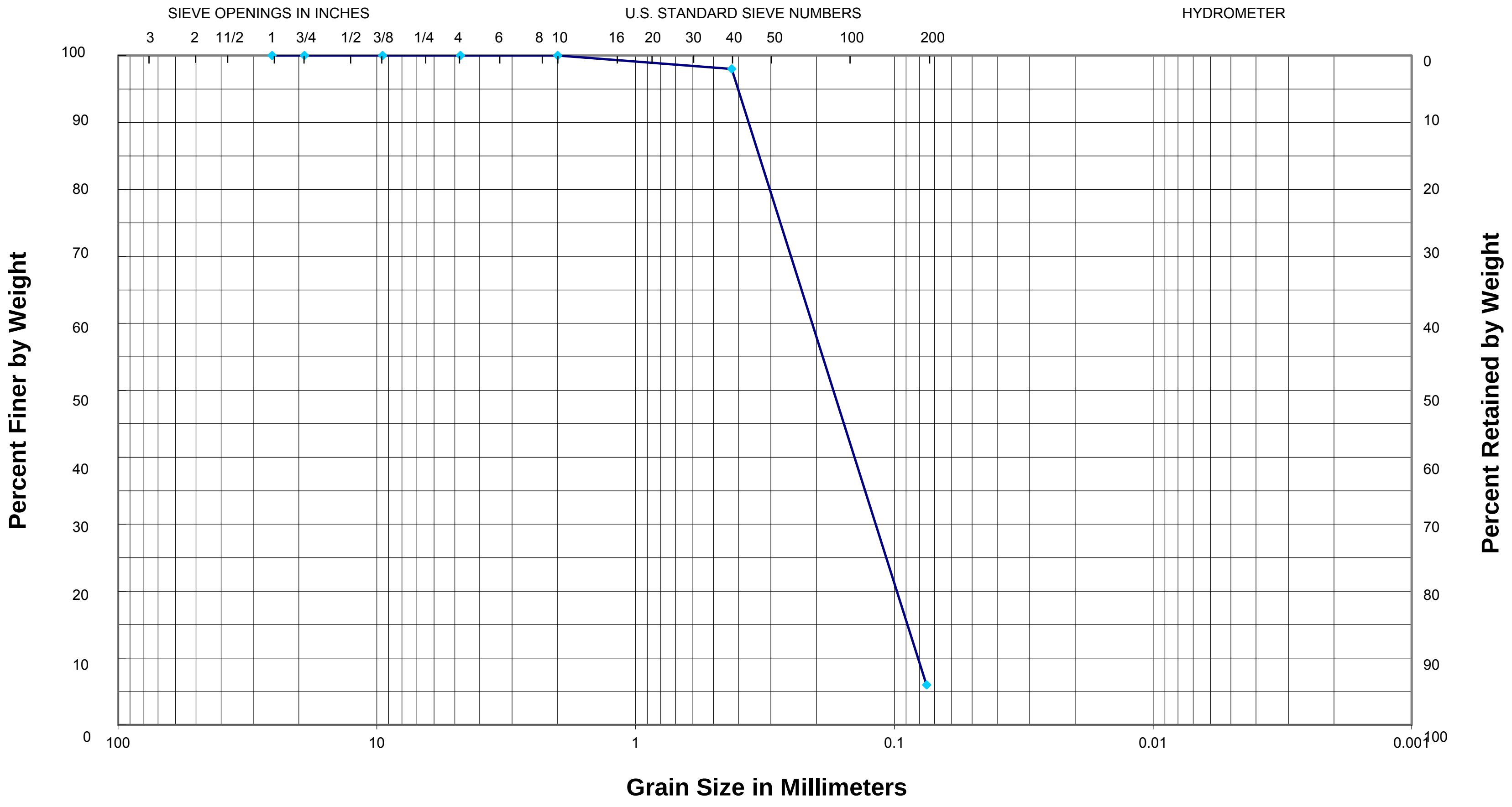
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 44-45 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



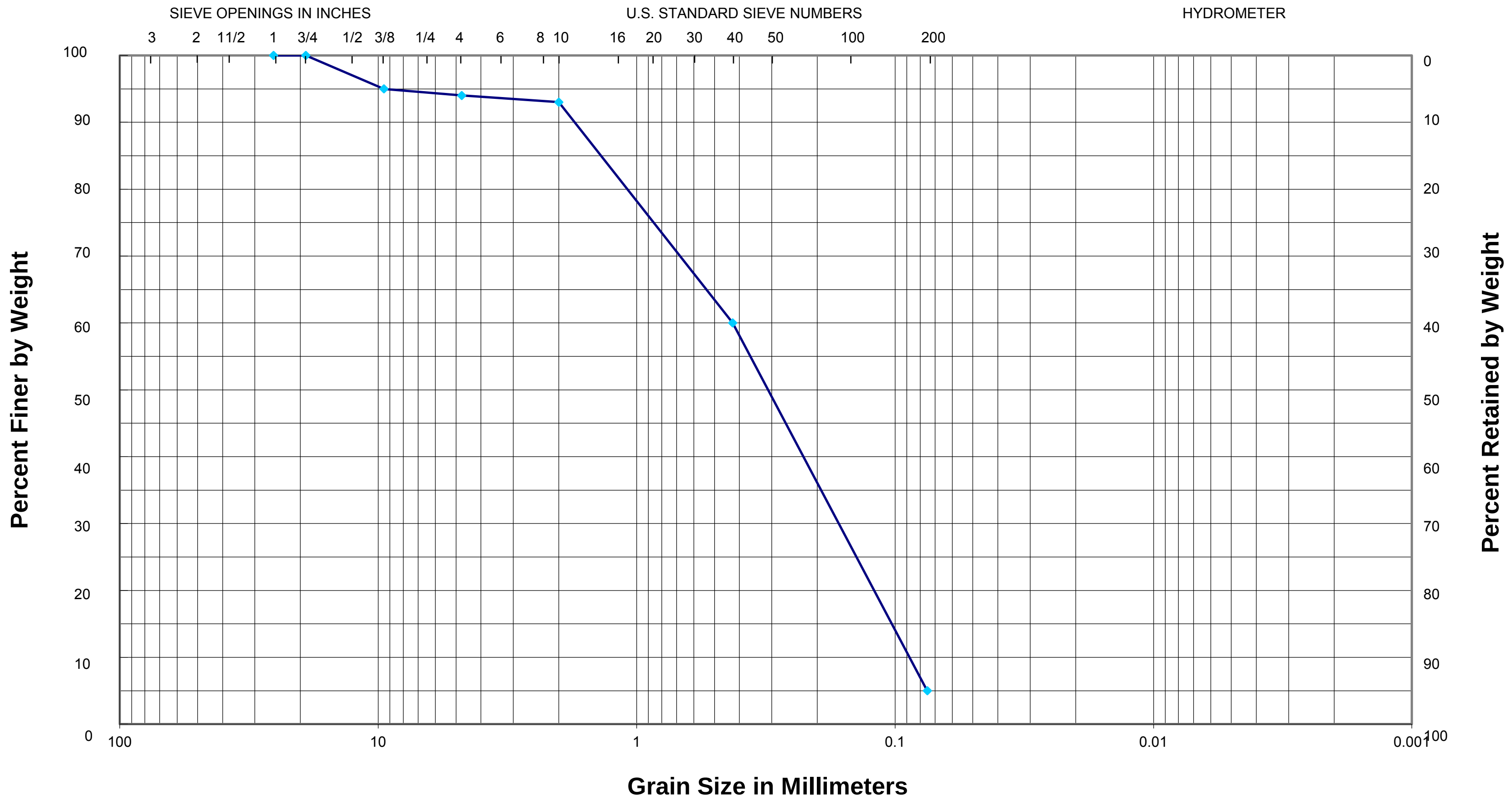
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 59-60 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



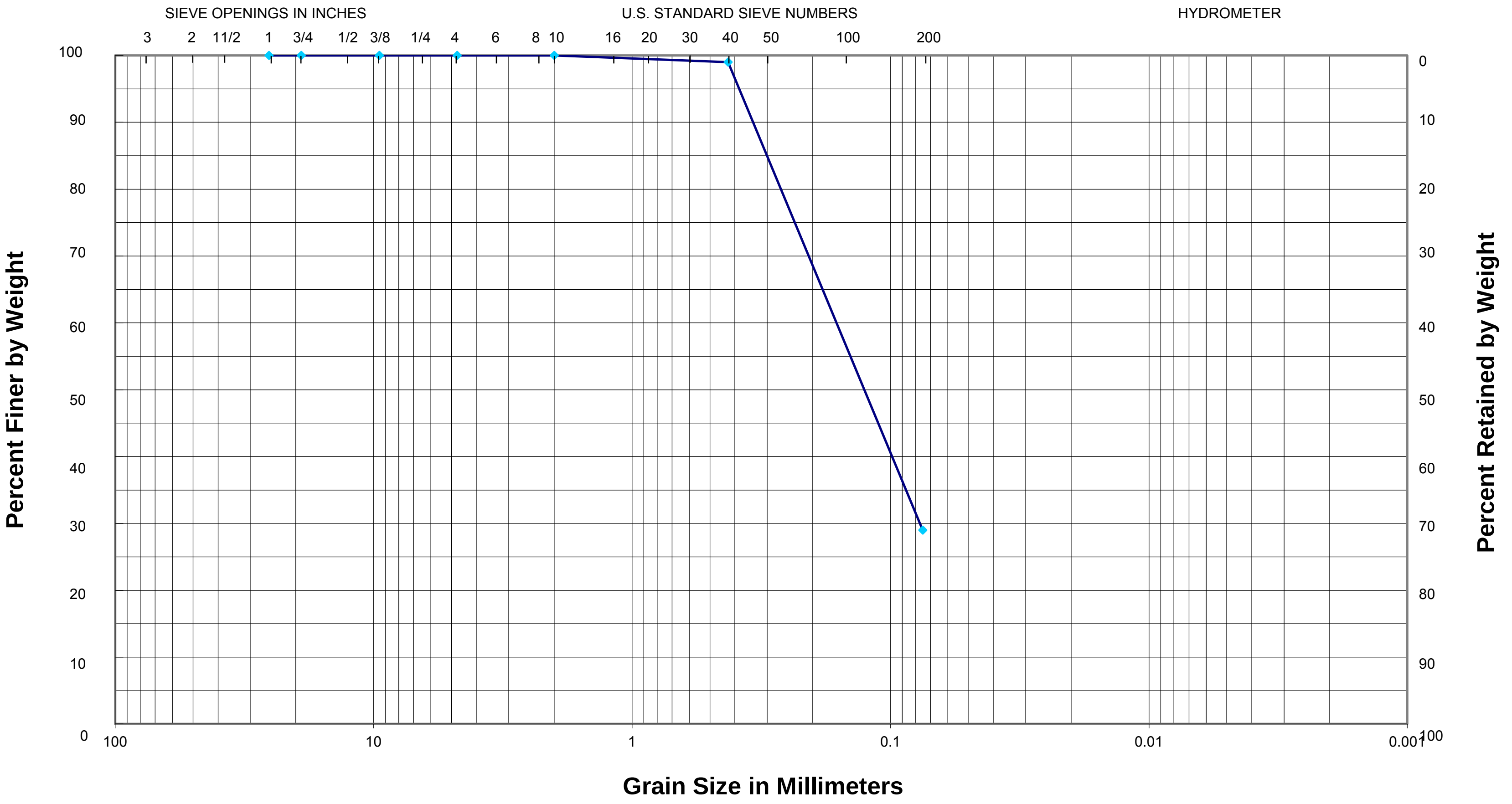
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 69-70 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



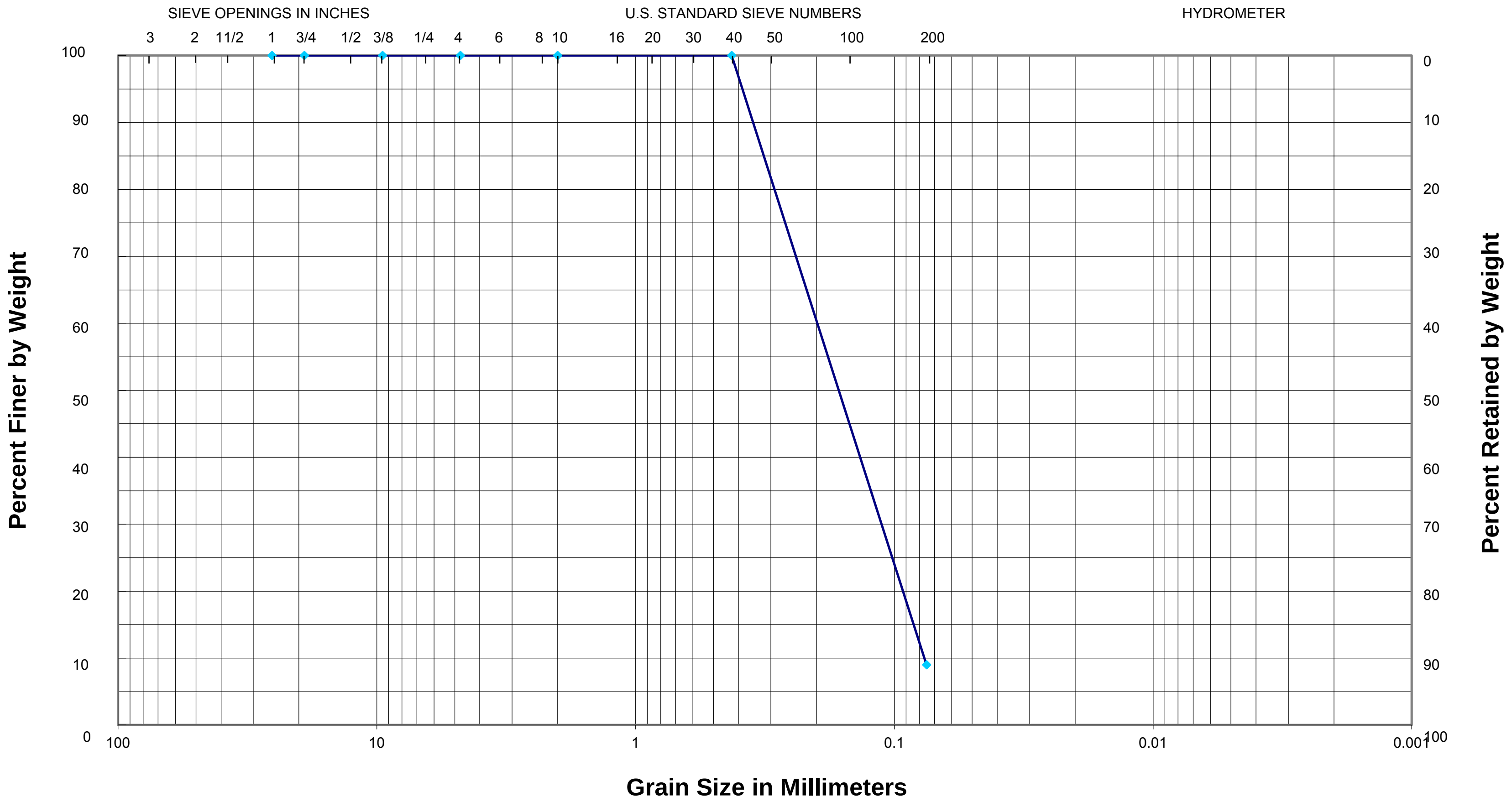
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



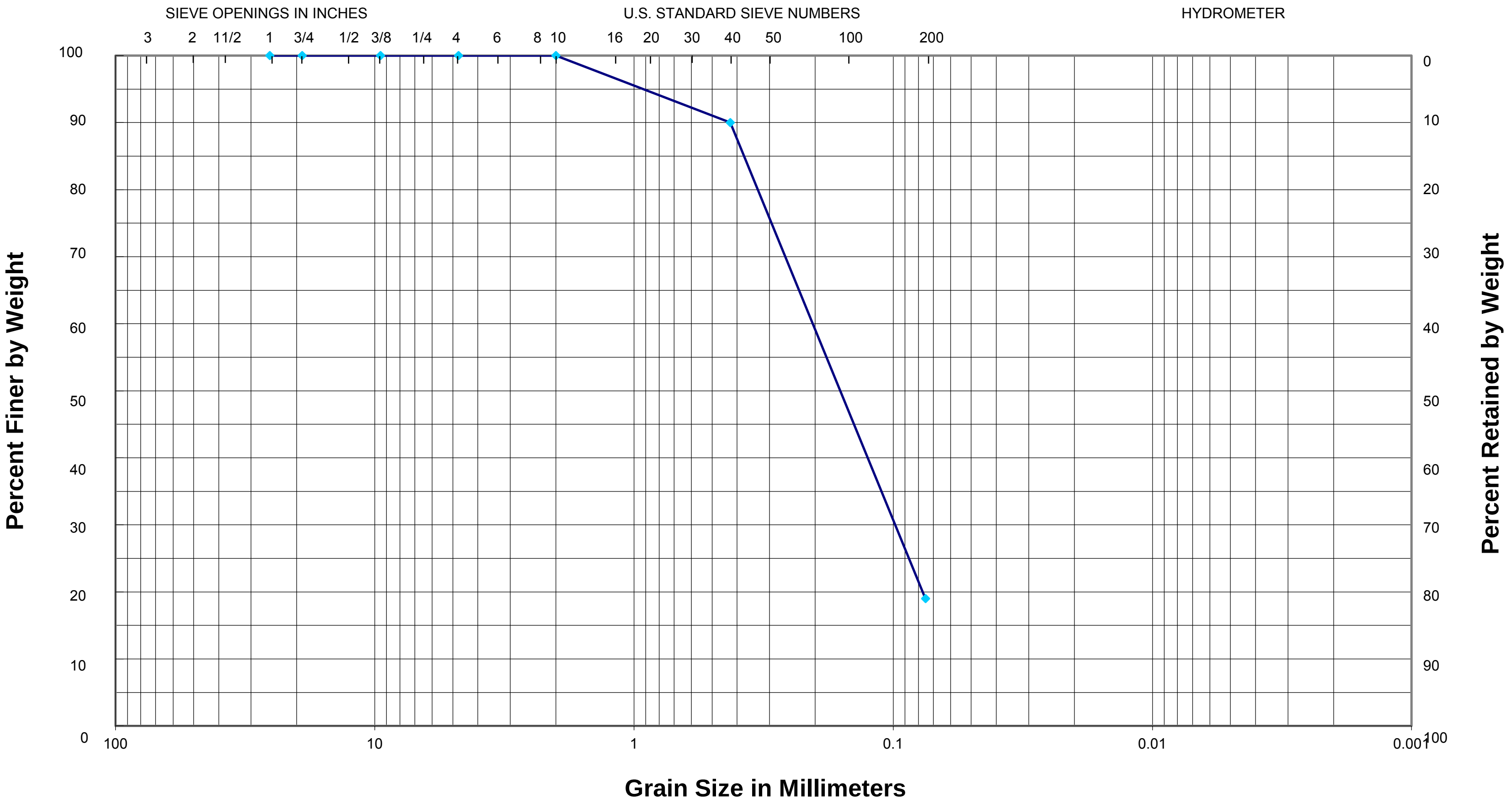
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 39-40 ft
Atterberg Limits: Non-plastic

Description: Dark gray and brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

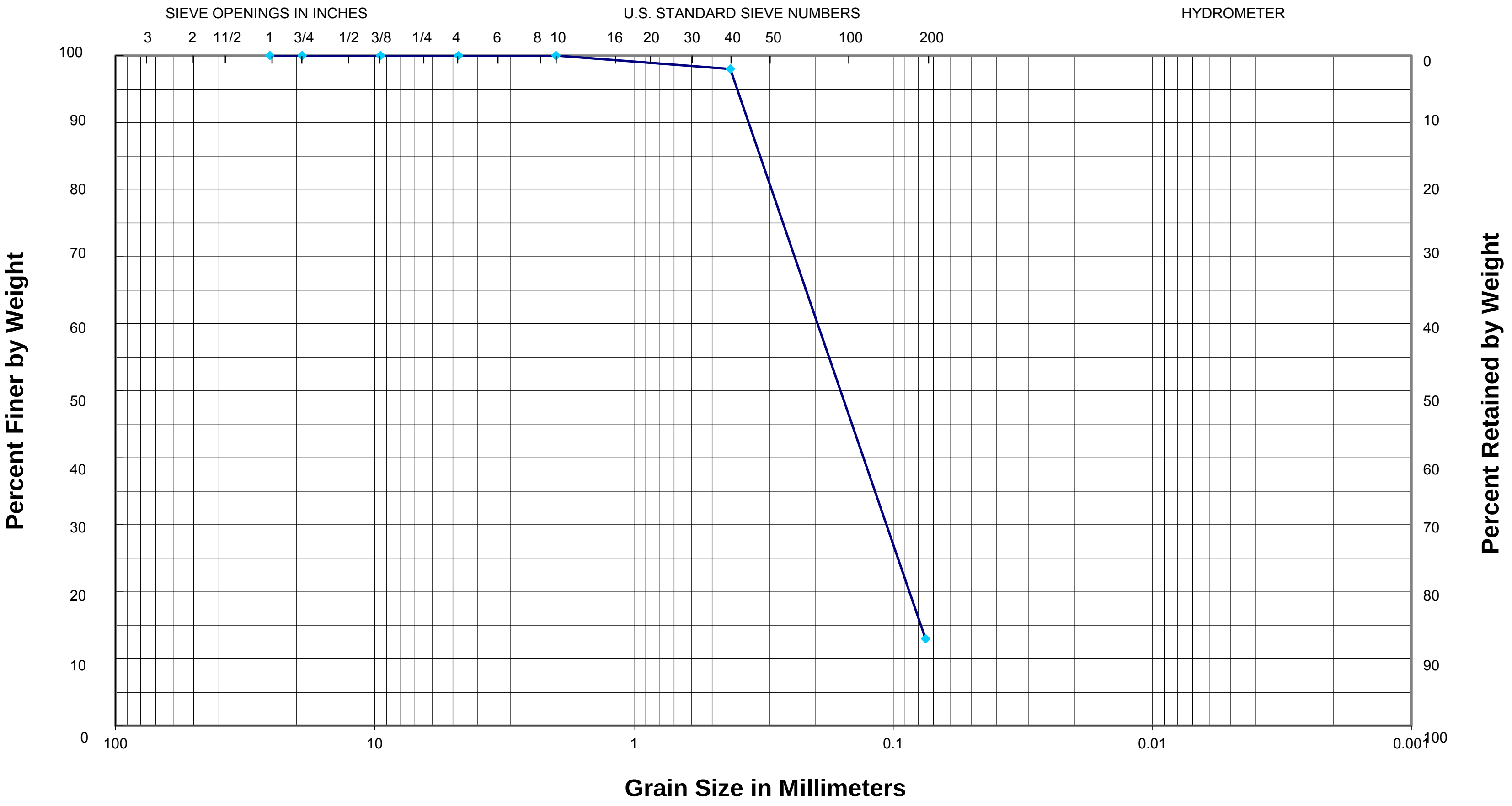
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 49-50 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



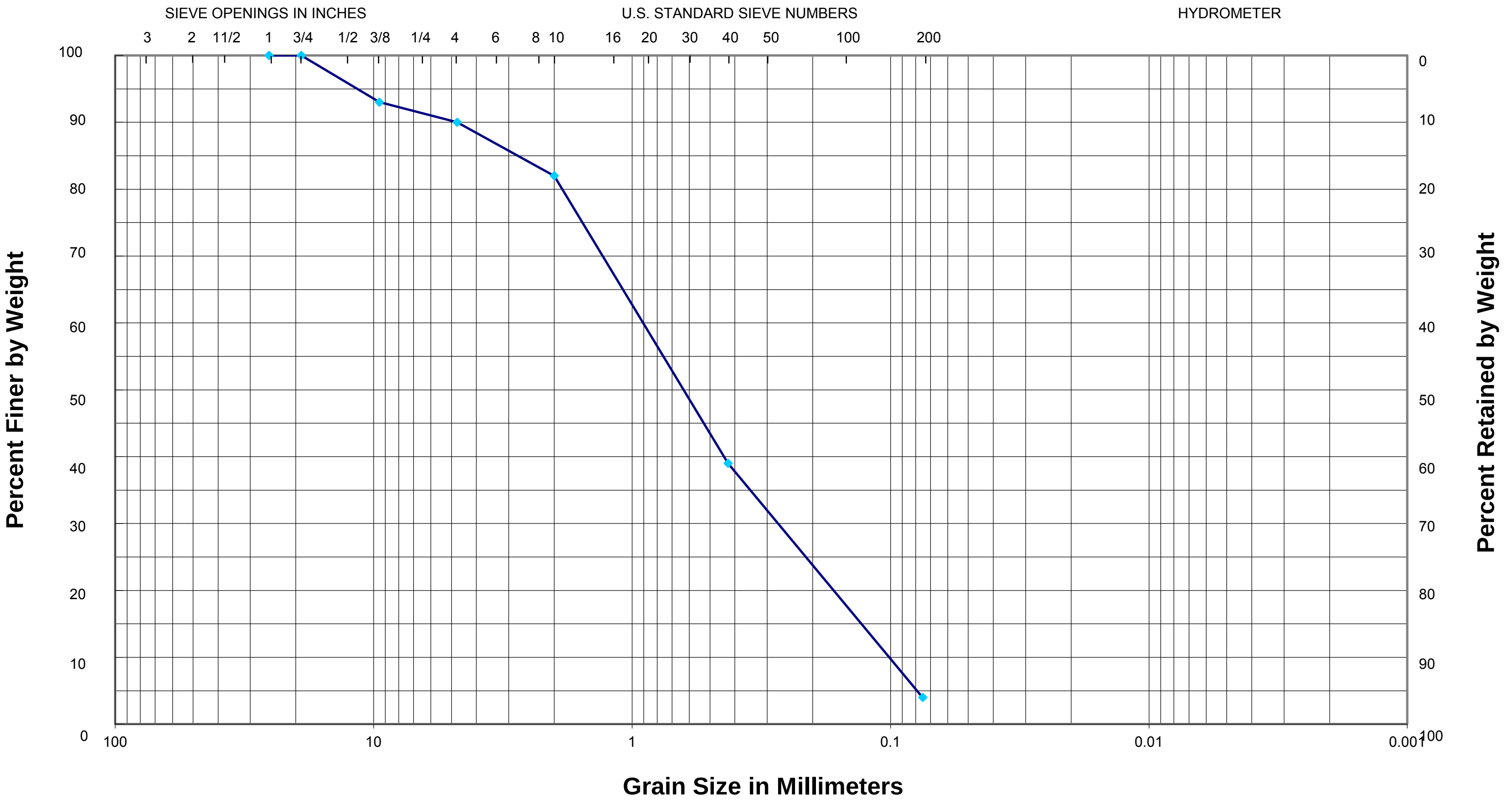
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 59-60 ft
Atterberg Limits: Non-plastic

Description: Gray and brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



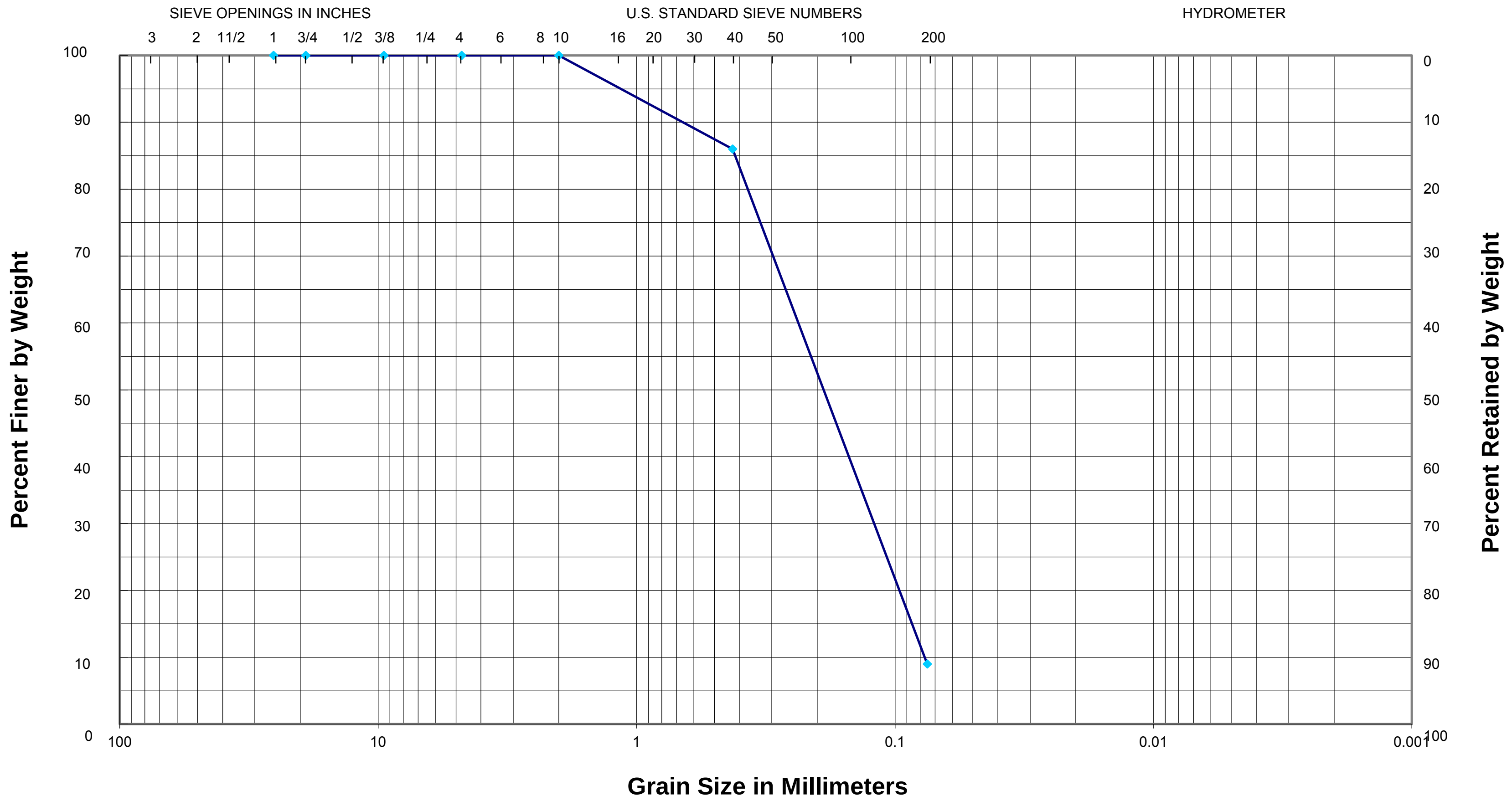
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 69-70 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand w/trace fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



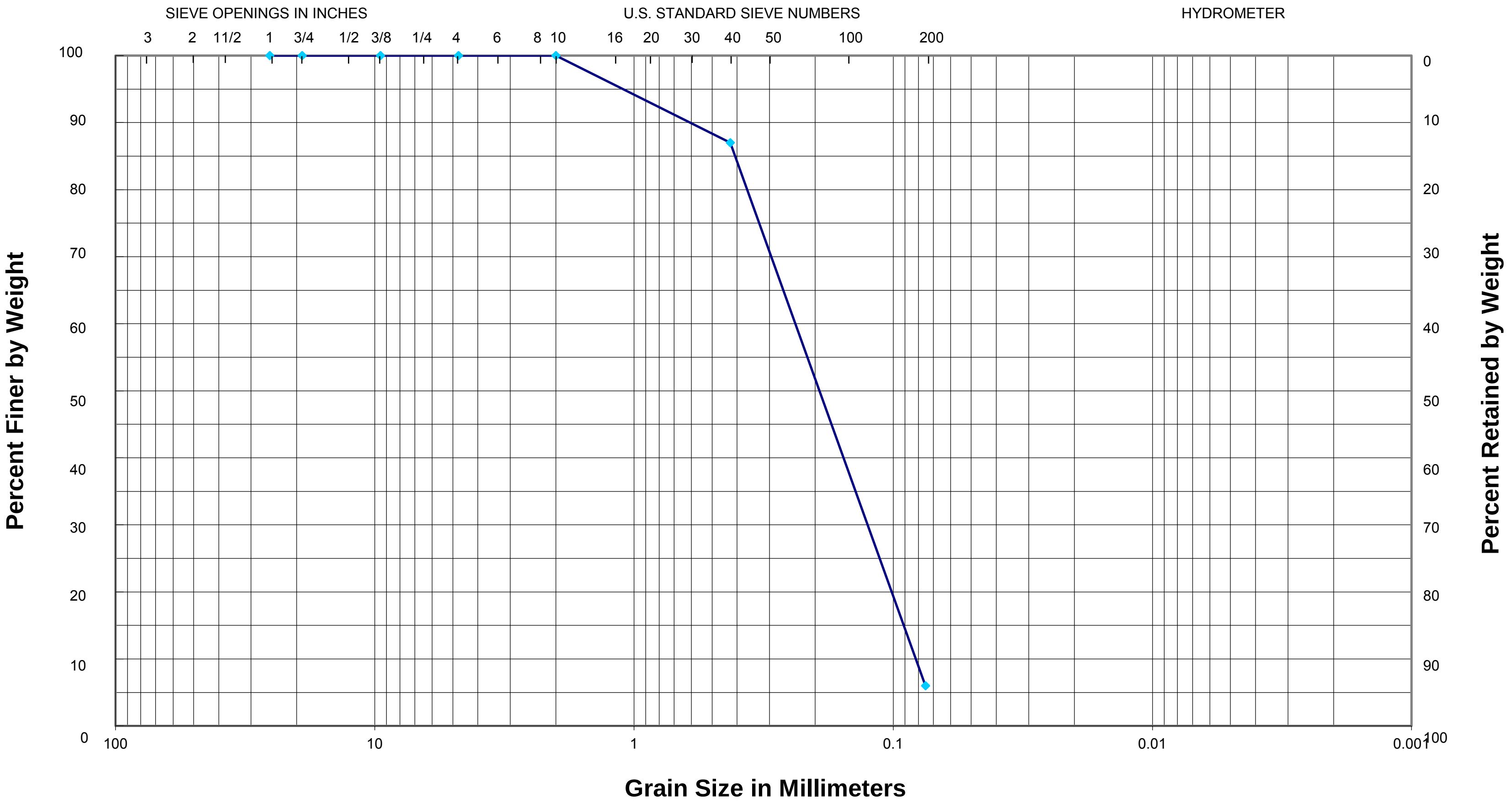
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 44-45 ft
Atterberg Limits: Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

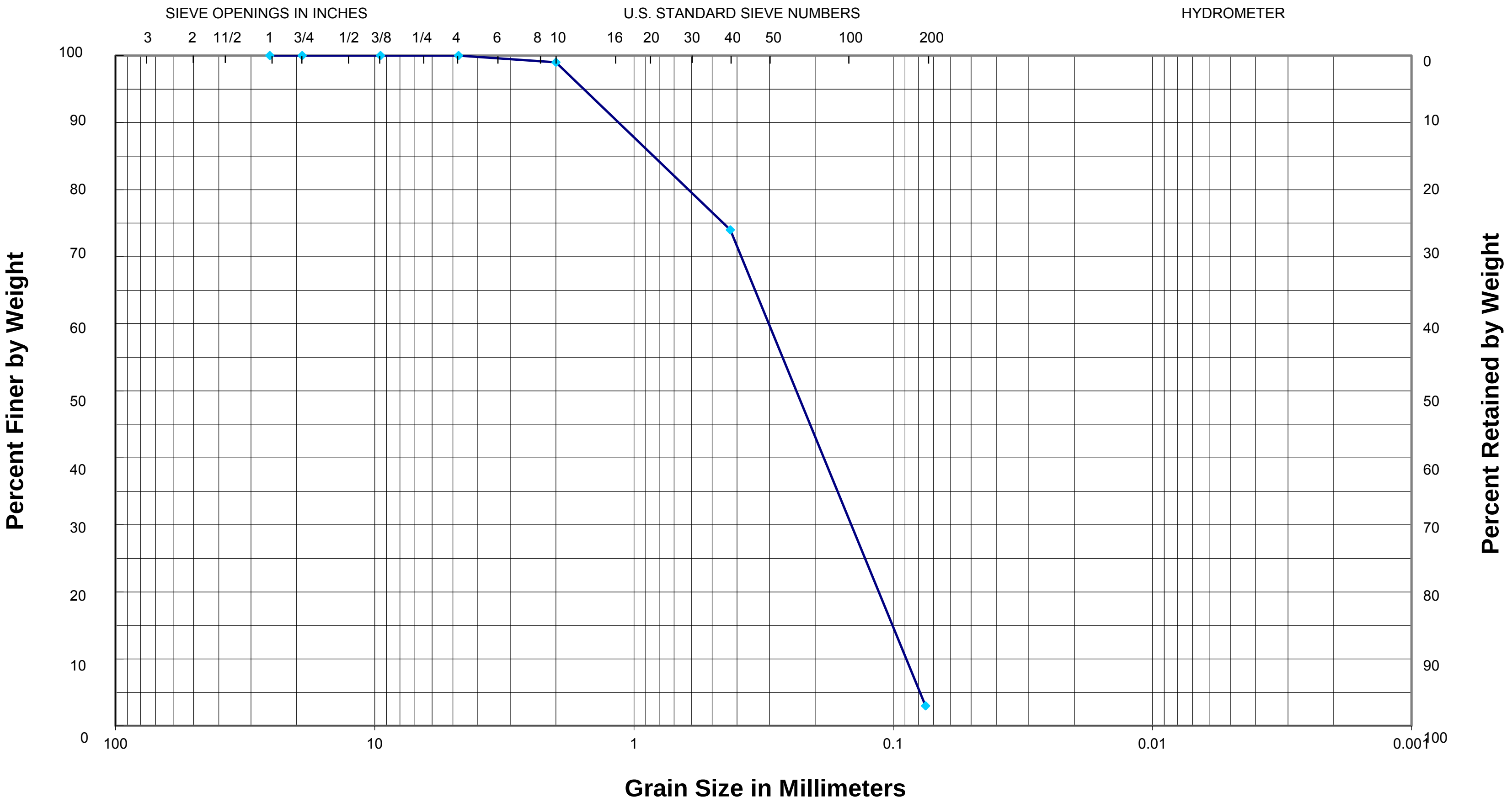
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 54-55 ft
Atterberg Limits: Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



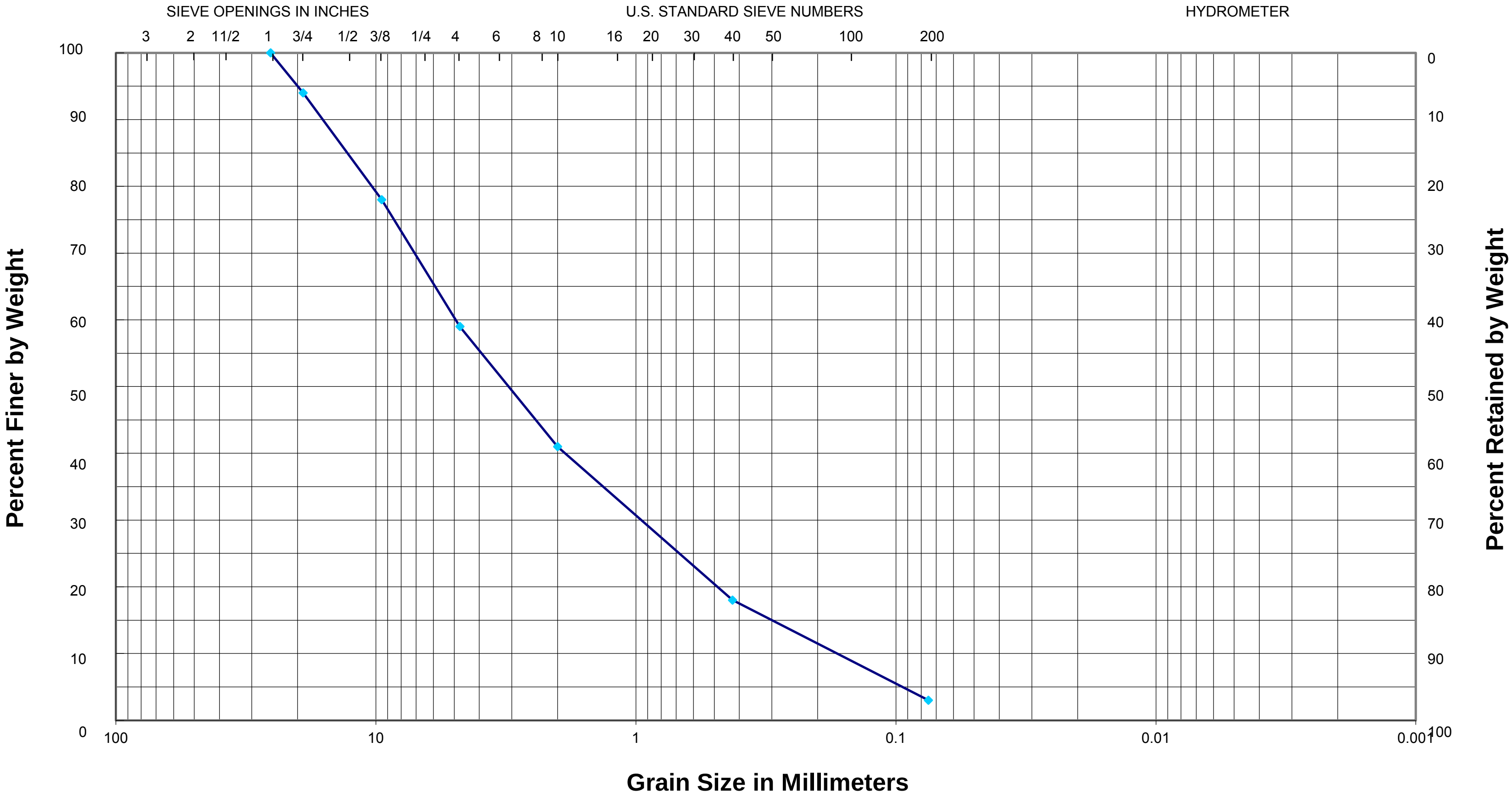
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 68.5-69 ft
Atterberg Limits: Non-plastic

Description: Brown fine to medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 83.5-84 ft
Atterberg Limits: Non-plastic

Description: Brown and gray sandy fine to coarse gravel
Classification: USCS = SW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

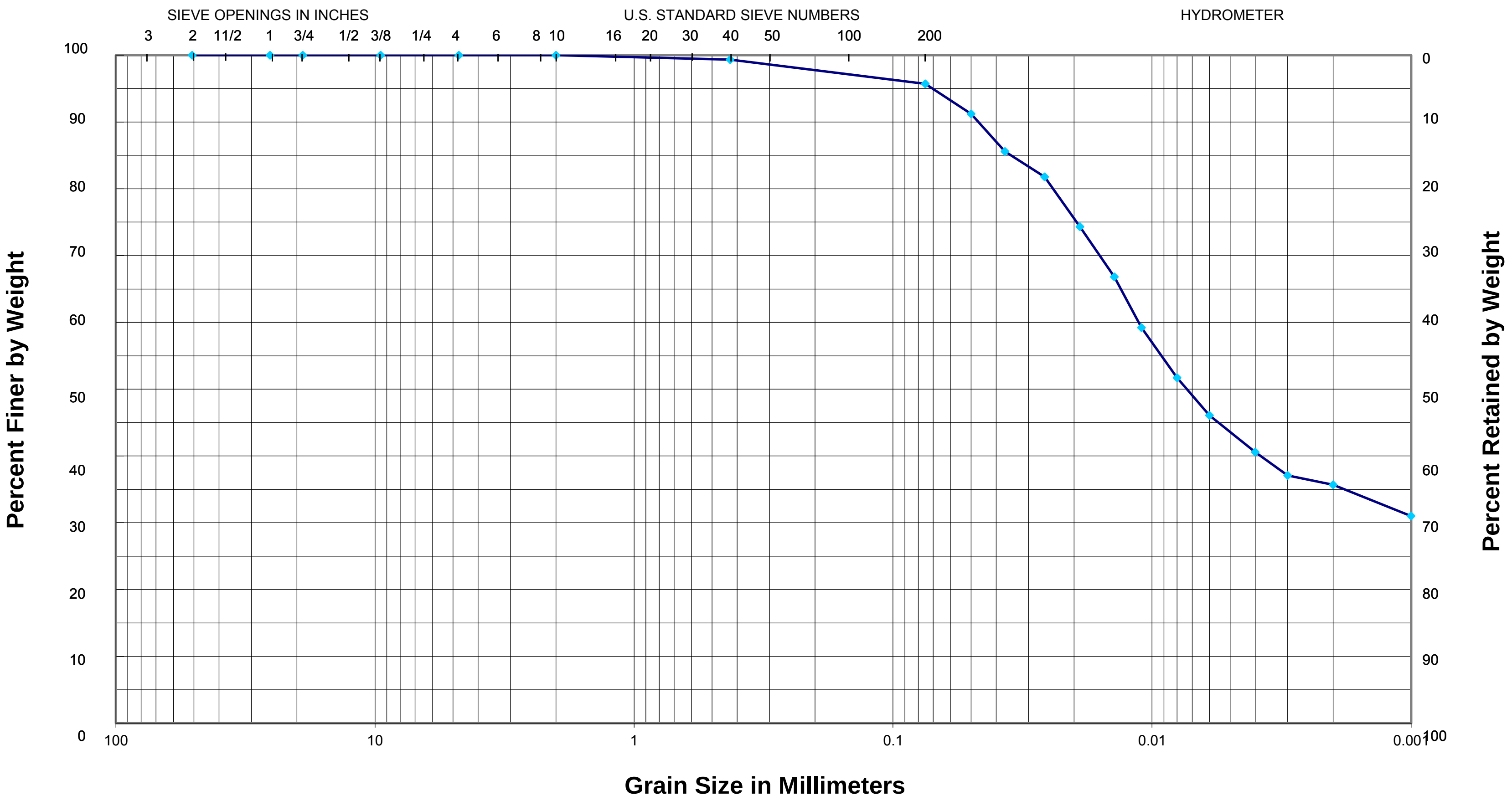
Sample: Boring 18, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Brown and gray sandy fine gravel
Classification: USCS = GP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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



<table>
<tr><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT</td><td rowspan="2">CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

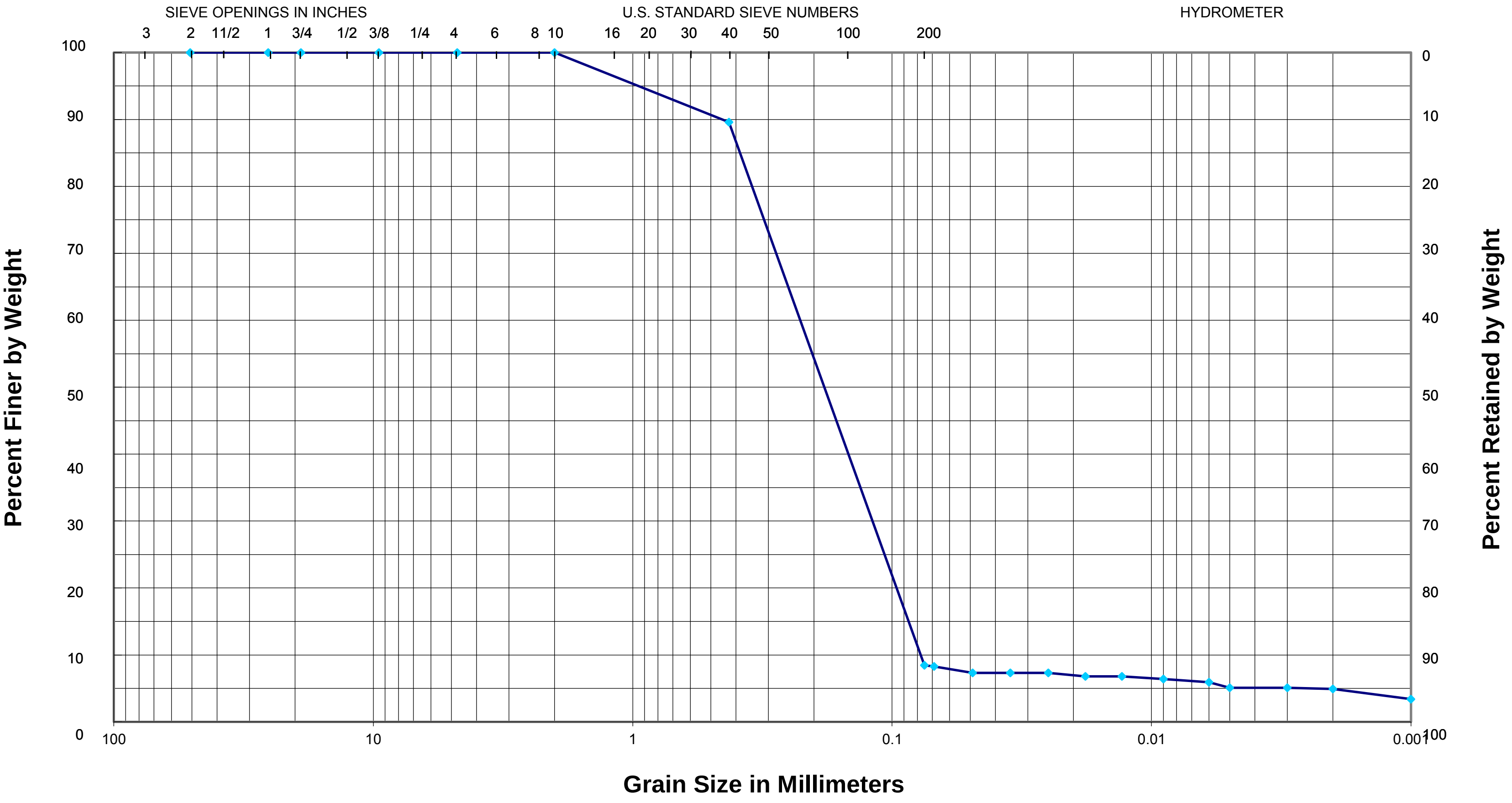
Sample: Boring 19, 6.5-7 ft
Properties: G_s = 2.696; LL = 40, PL = 18, PI = 22

Description: Gray and brown silty clay w/ferrous stains and nodules
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



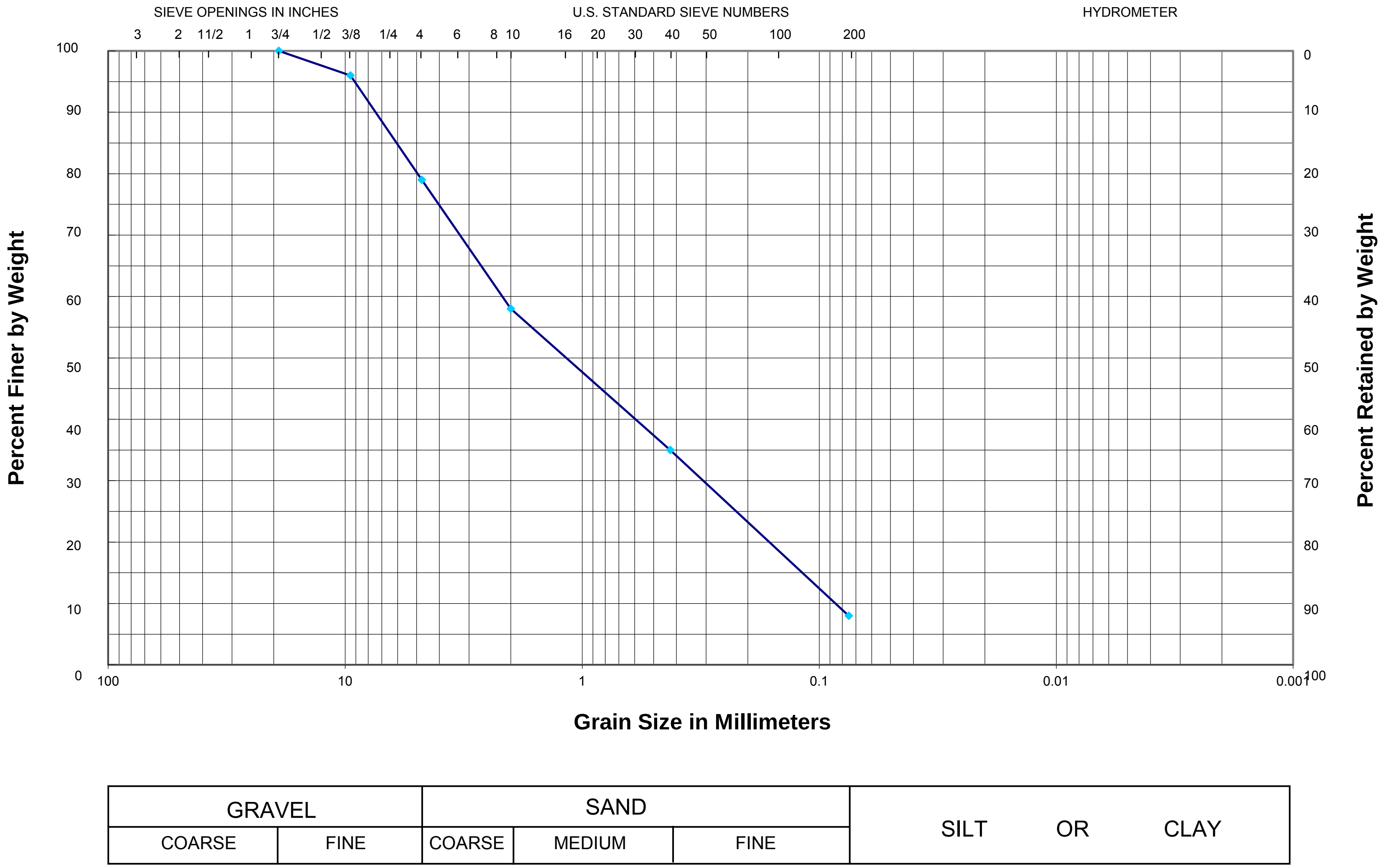
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 19, 24-25 ft
Properties: G_s = 2.678; Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = sp-sm; AASHTO = A-3

13-017

GRAIN SIZE CURVE

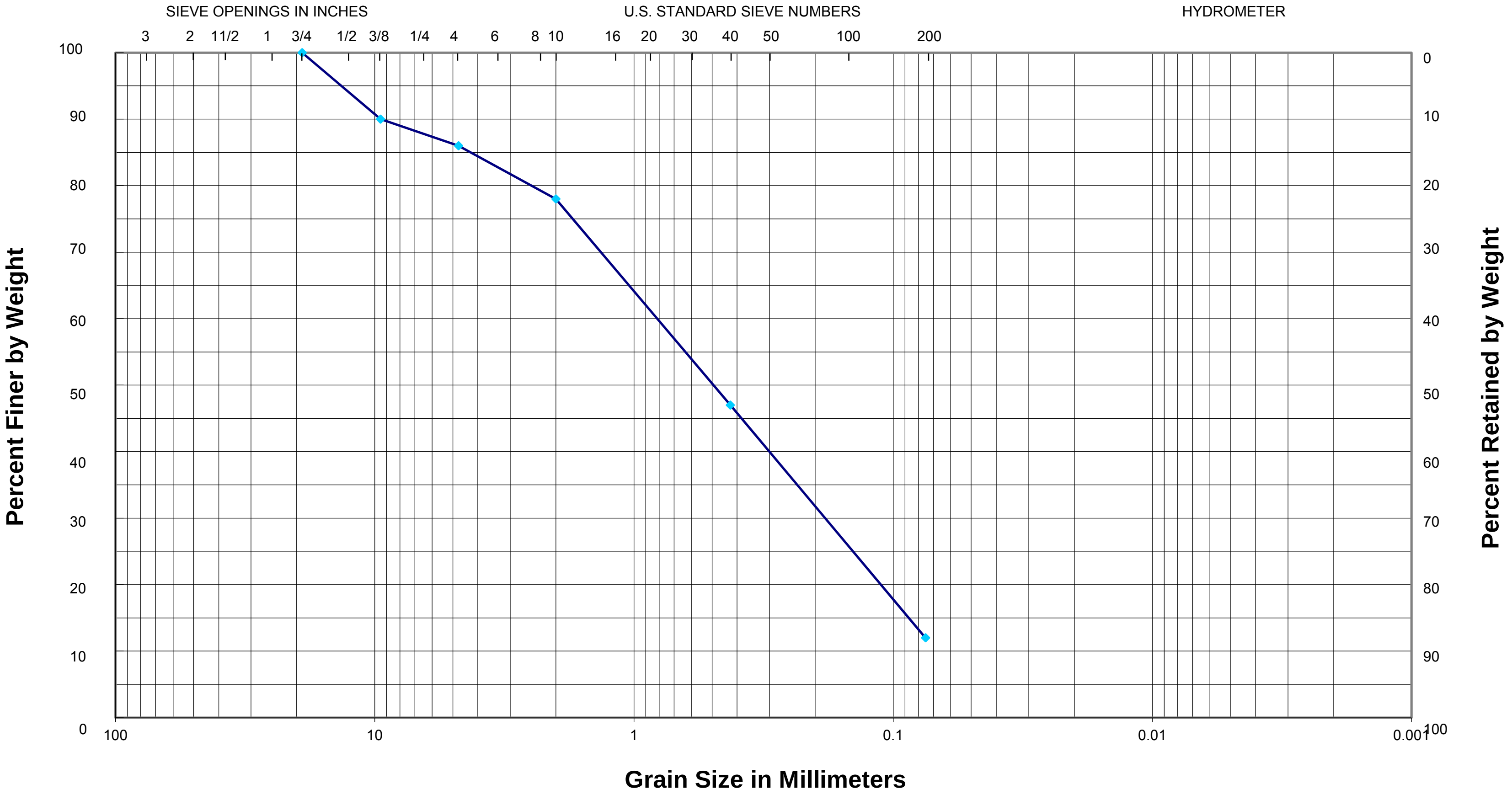


Sample: Boring 19, 78.5-79.5 ft
Atterberg Limits: Non-plastic

Description: Grayish brown sandy fine gravel, slighly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



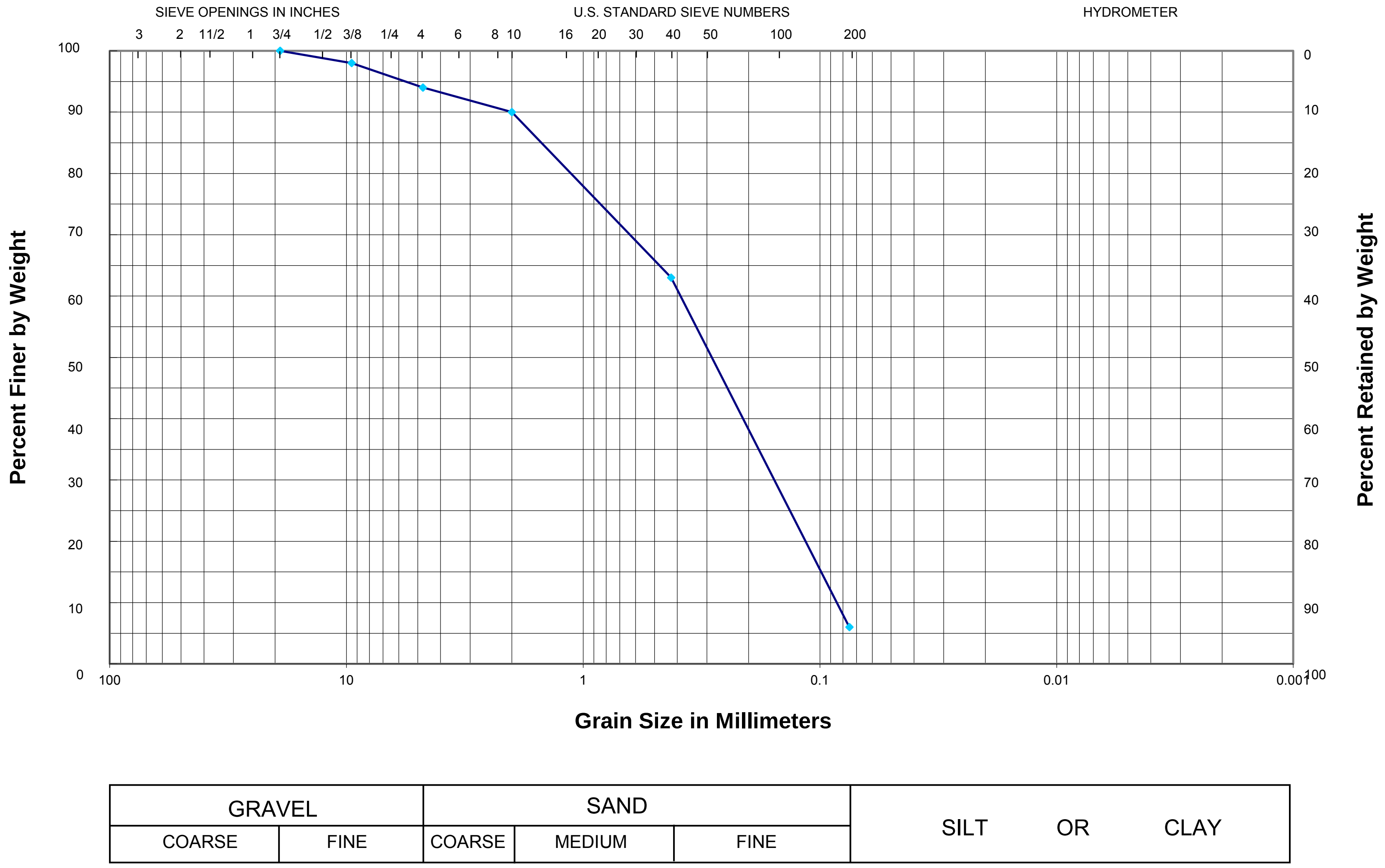
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 20, 44-45 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand with a little fine gravel
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

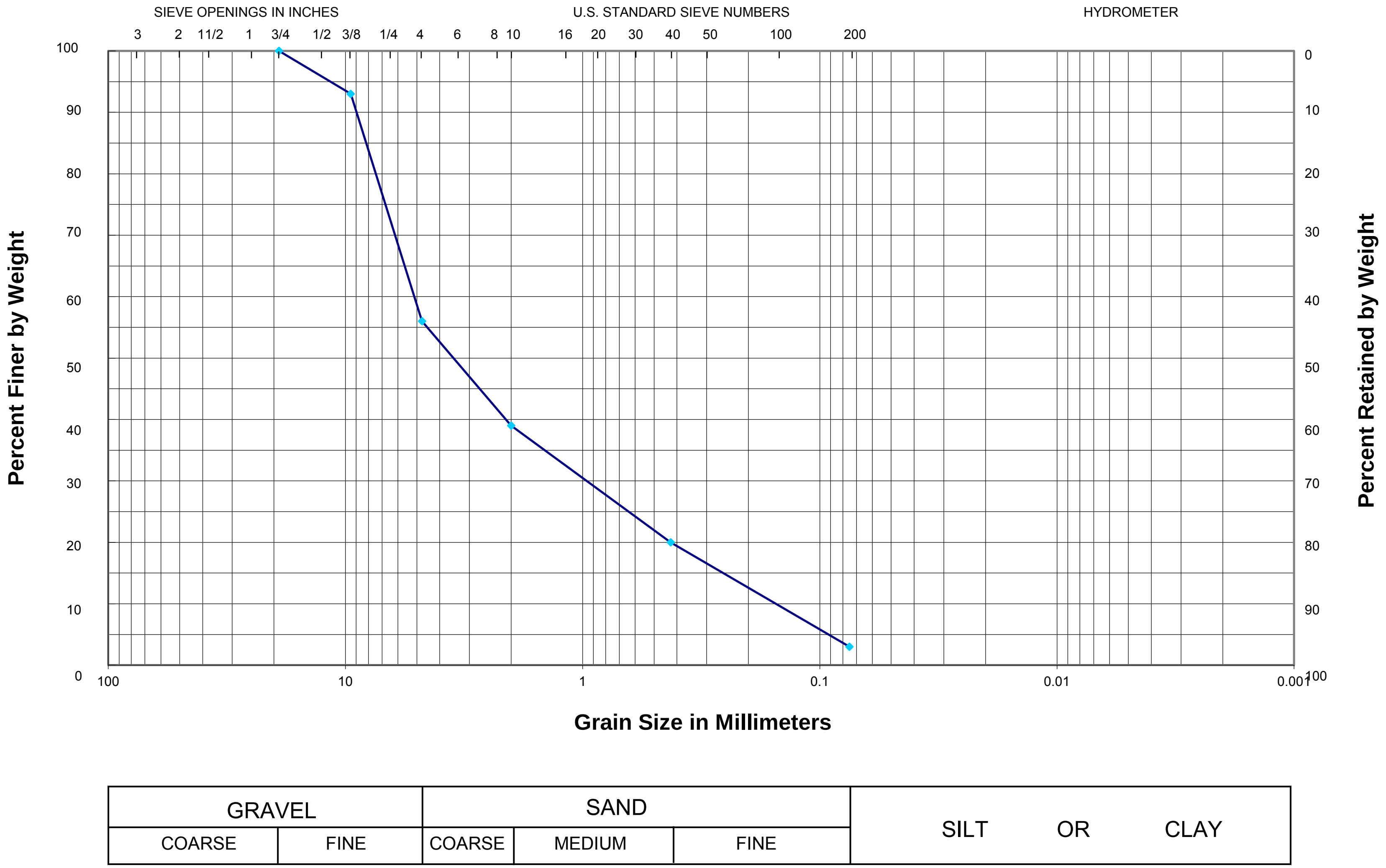


Sample: Boring 20, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand with trace fine gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

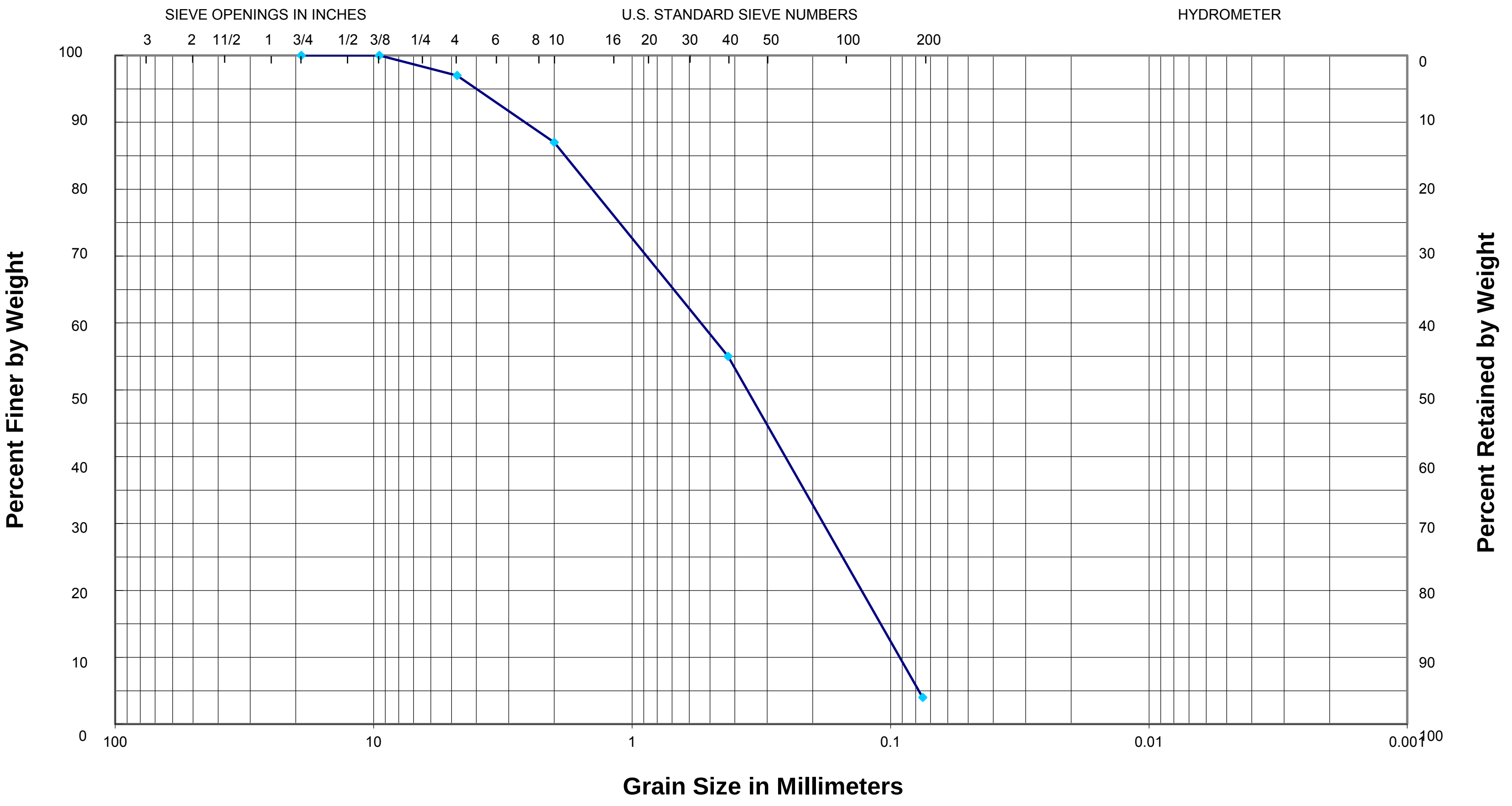


Sample: Boring 20, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Brown sandy fine gravel
Classification: USCS = SW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



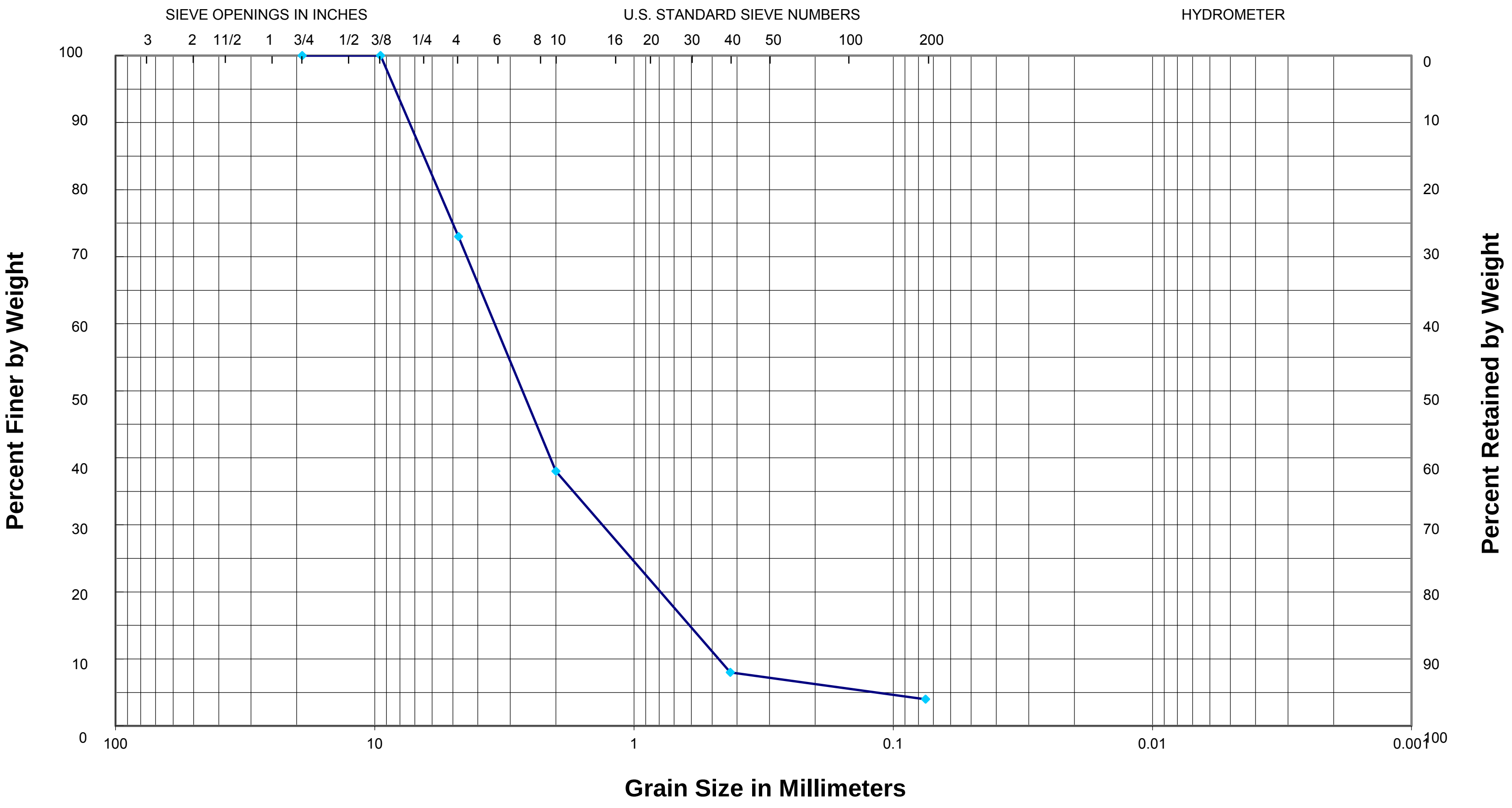
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 21, 44-45 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine to coarse sand w/a little fine gravel
Classification: USCS = SP; AASHTO = A-1-3

13-017

GRAIN SIZE CURVE



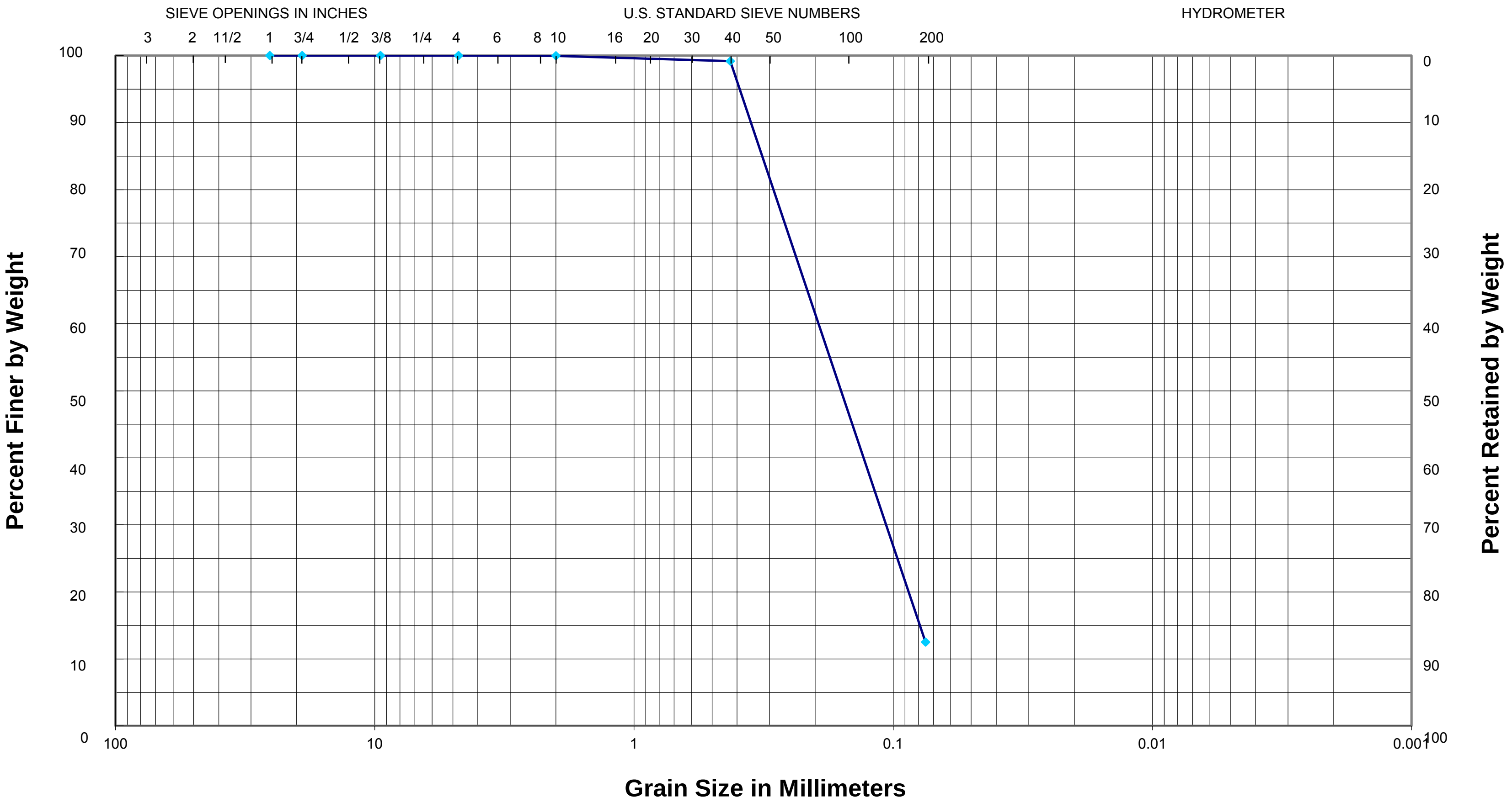
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 21, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray and tan sandy fine gravel
Classification: USCS = SP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



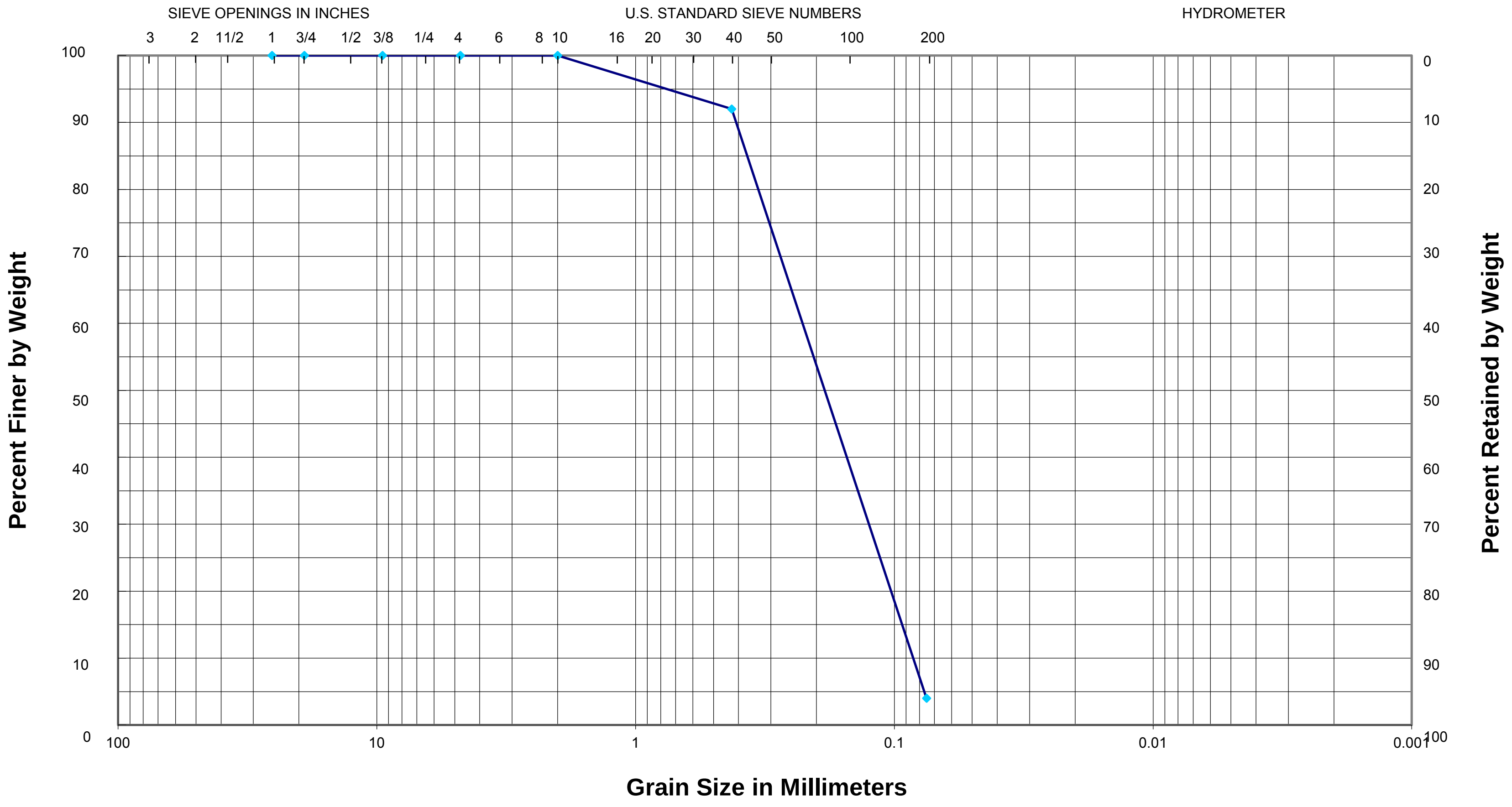
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



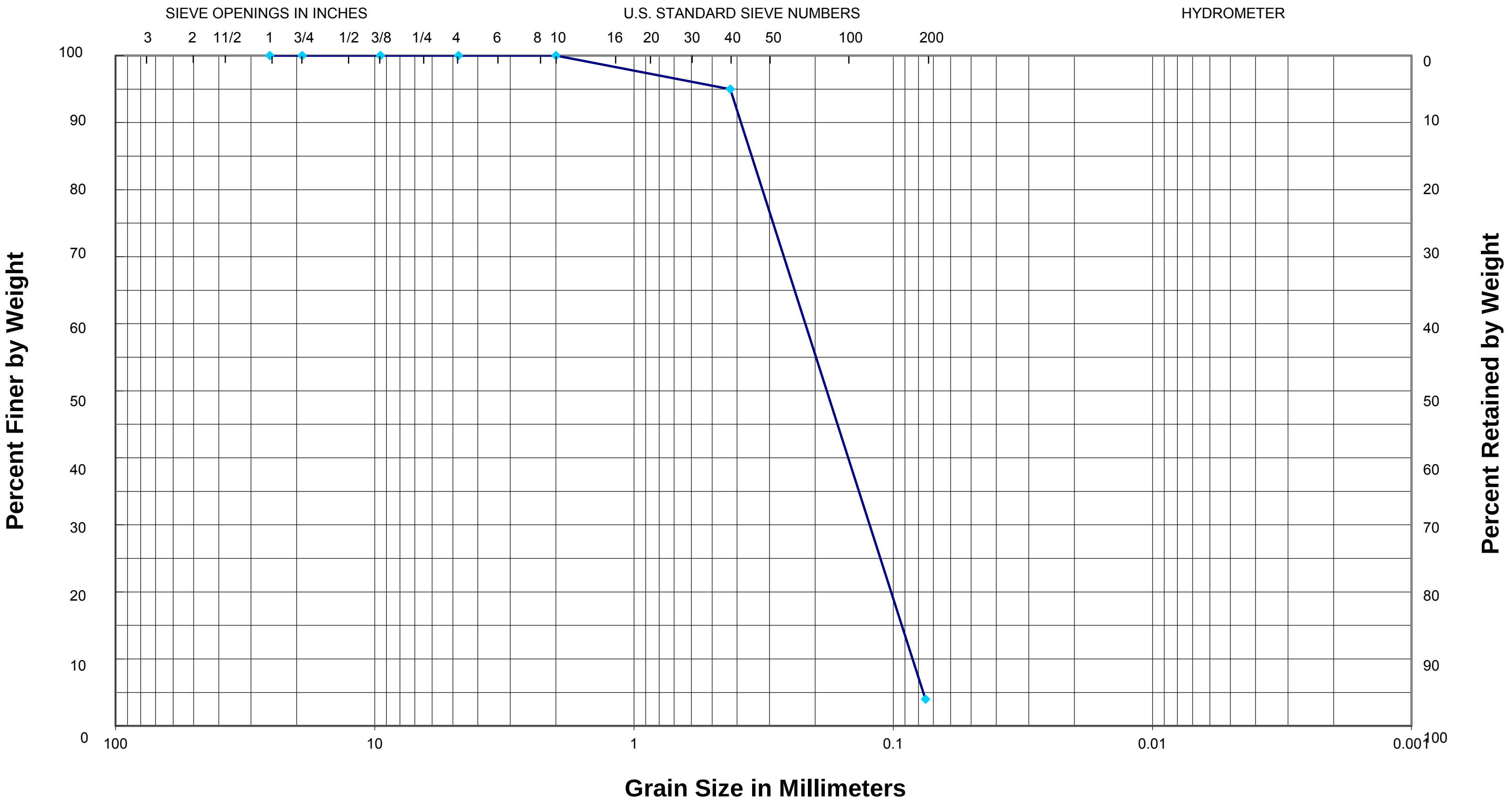
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Gray and brown sandy fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



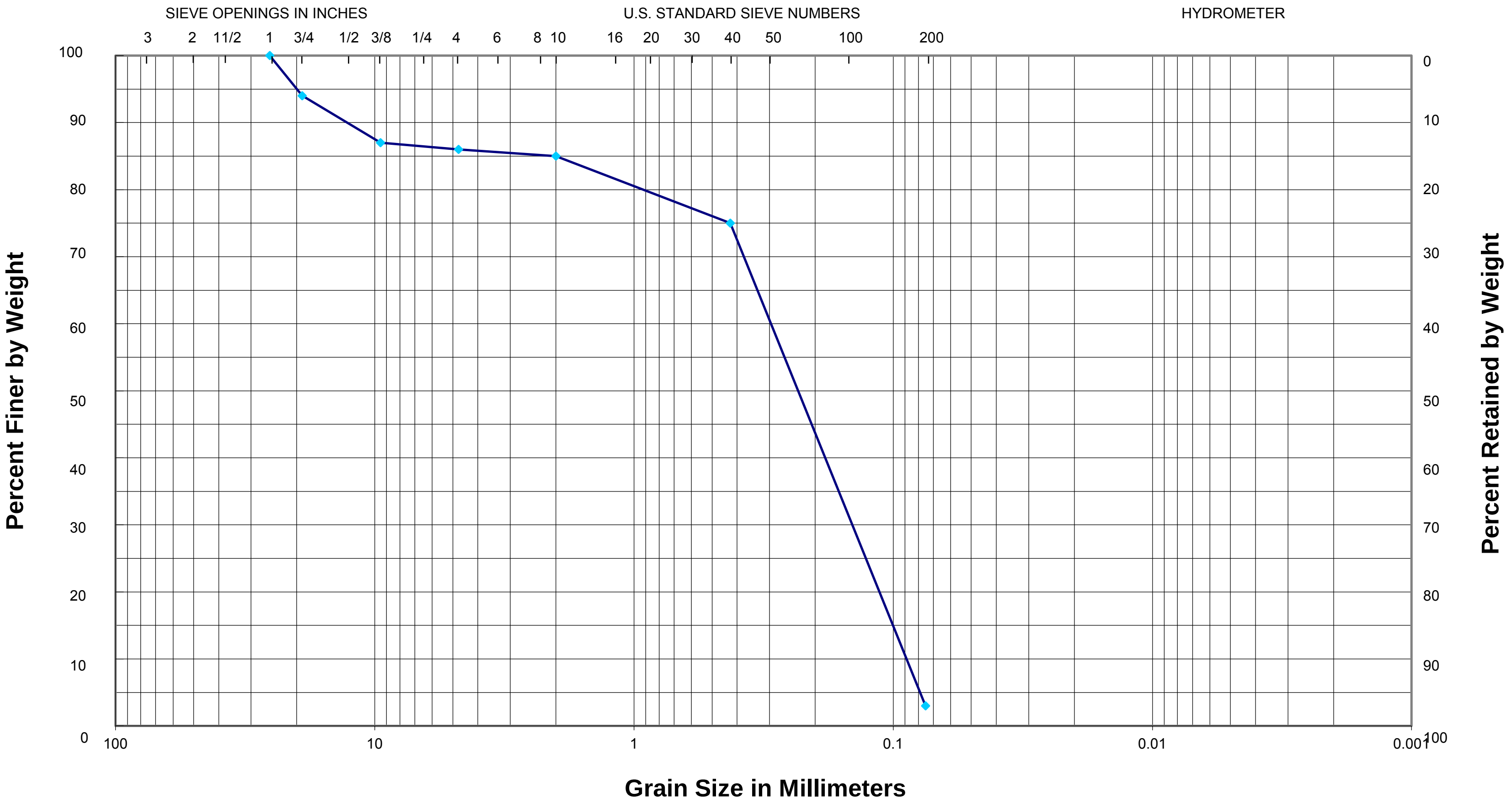
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 24-25 ft
Atterberg Limits: Non-plastic

Description: Grayish brown fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



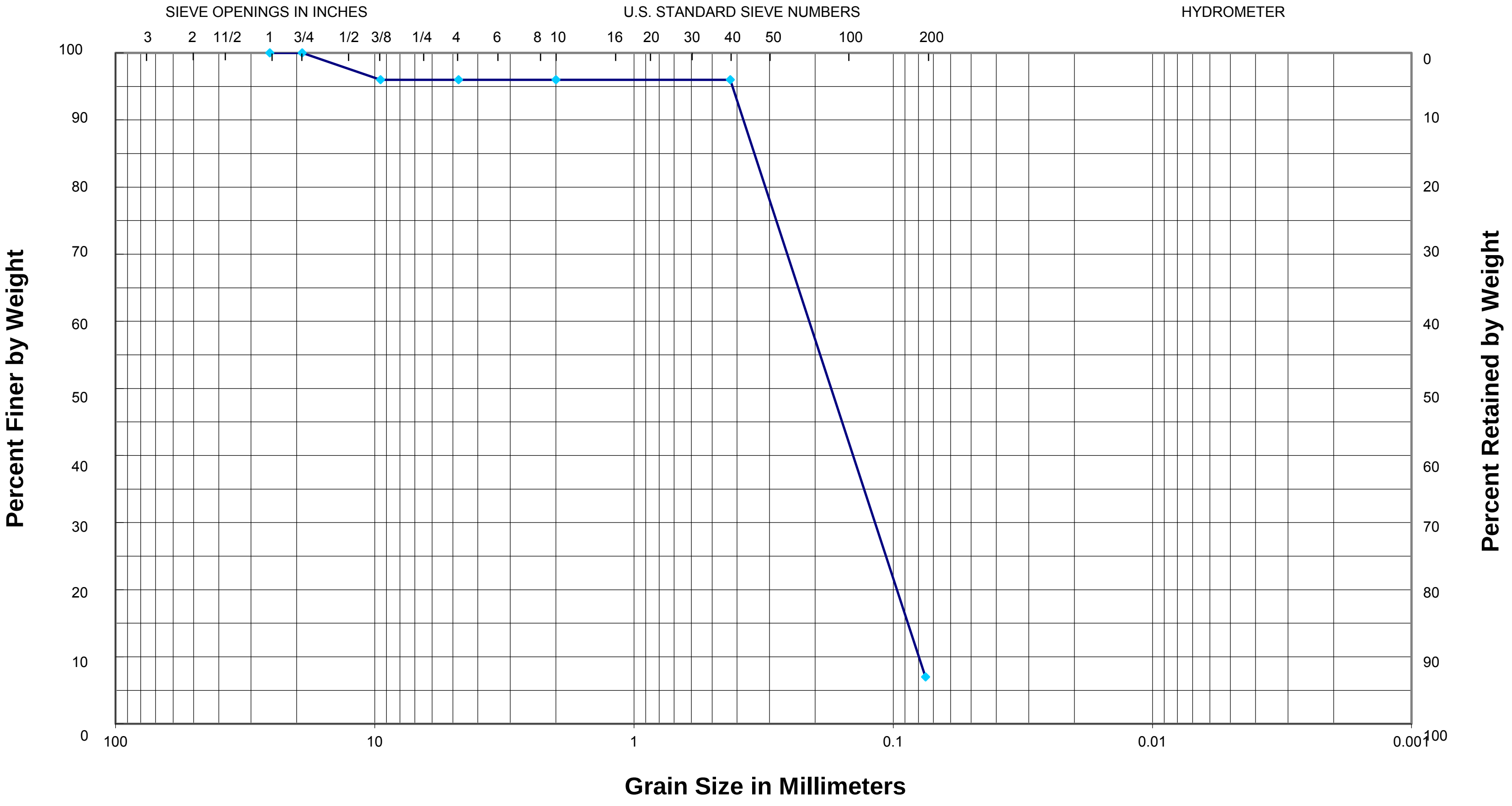
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Brownish gray fine to medium sand w/a little coarse sand and fine gravel
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



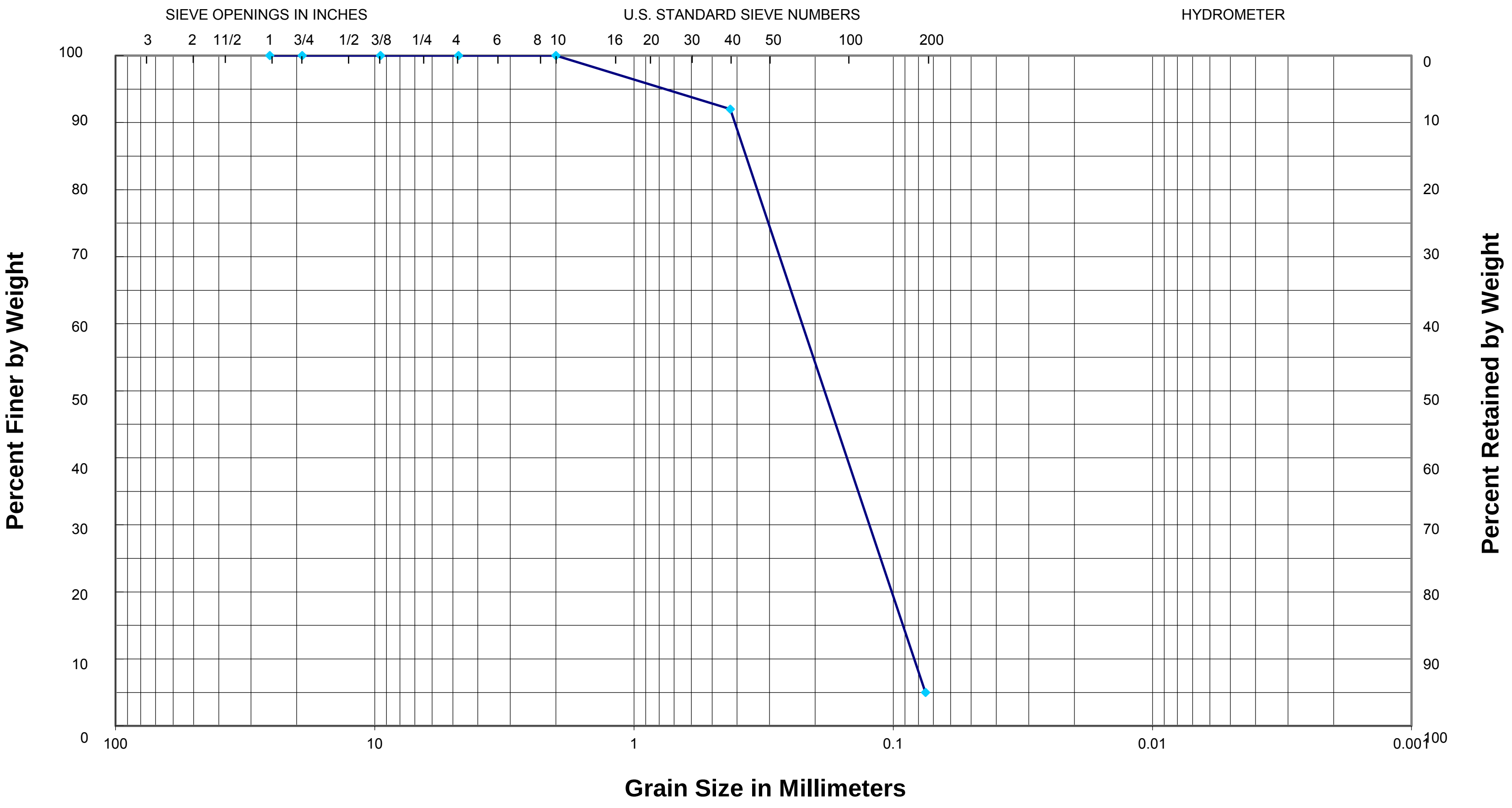
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



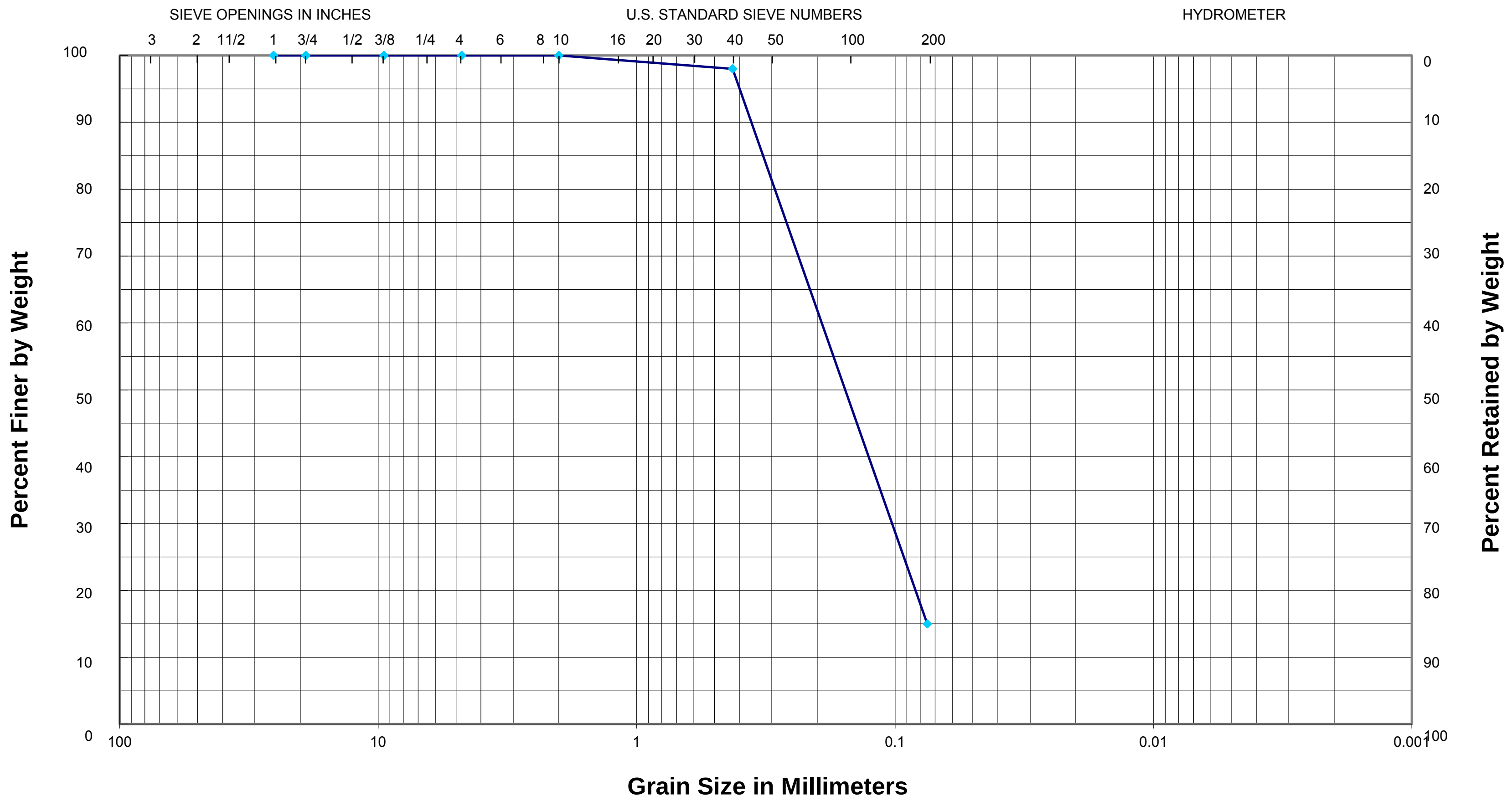
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 64-65 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 25, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.: 13-017

Project: I-40 over White River

Prairie County, Arkansas

Boring No.: 2

Depth, ft: 4.5-5

Sample Description: Brown and gray clay with clay pockets and ferrous stains

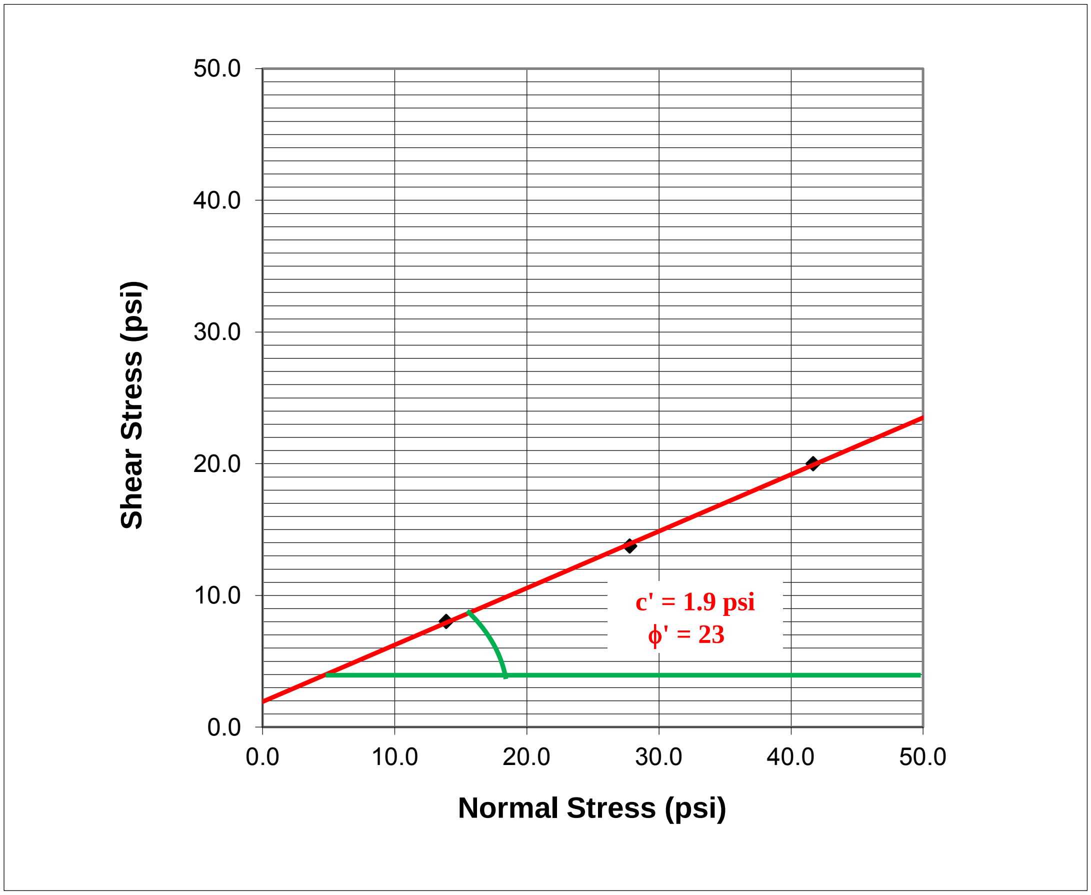
Tested By: MM

Reported by: YZ

Test Date: 5/21/2013

Material Properties: LL = 52, PL = 23, PI = 29; Minus No. 200 Sieve = 98%; USCS = CH; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	13.9	8.0	93.6	27.7
2	27.8	13.8	96.5	25.0
3	41.7	20.0	95.4	26.6



Grubbs, Hoskyn, Barton & Wyatt, Inc.

CONSULTING ENGINEERS

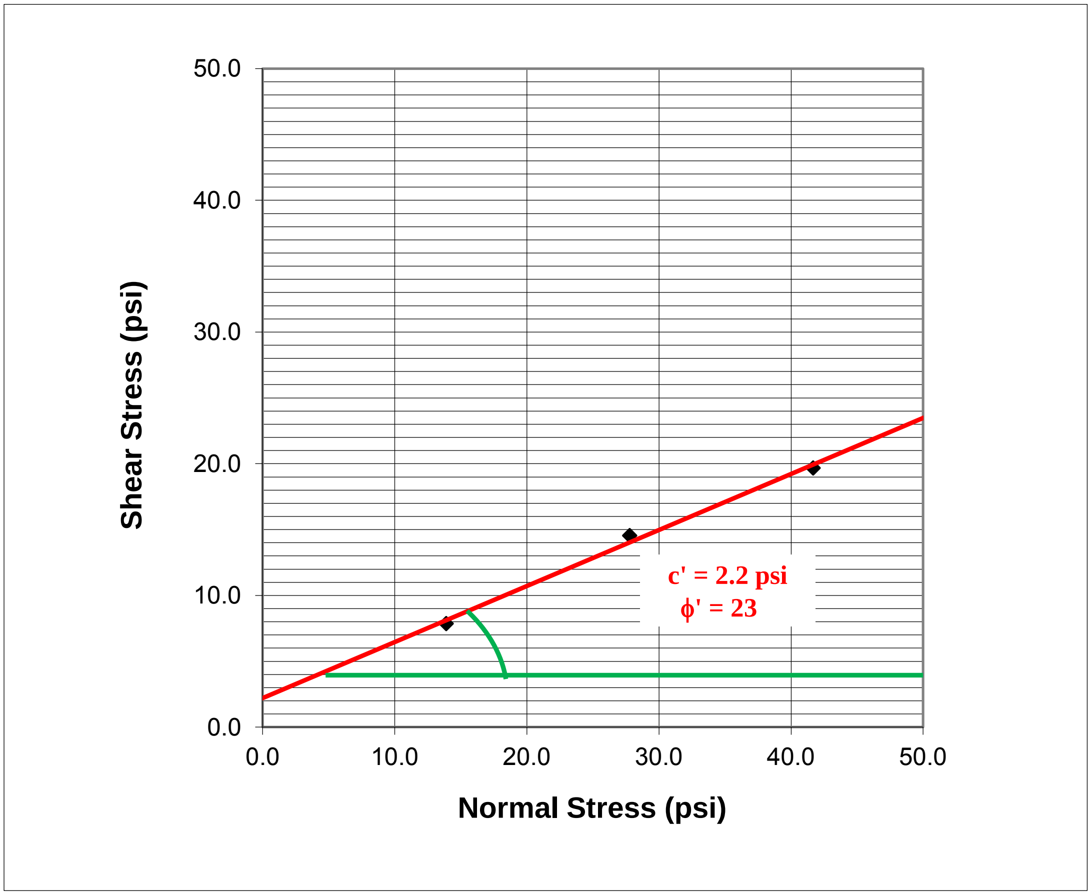
DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.:	13-017	Tested By:	MM
Project:	I-40 over White River	Reported by:	YZ
	Prairie County, Arkansas	Test Date:	5./30/2013
Boring No.:	3		
Depth, ft:	7-7.5		
Sample Description:	Brown and gray silty clay with ferrous stains		

Material Properties: LL = 47, PL = 21, PI = 26; Minus No. 200 Sieve = 95%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	13.9	7.9	99.0	23.7
2	27.8	14.5	99.3	24.0
3	41.7	19.7	100.6	22.9



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DIRECT SHEAR (AASHTO T-236) TEST RESULTS

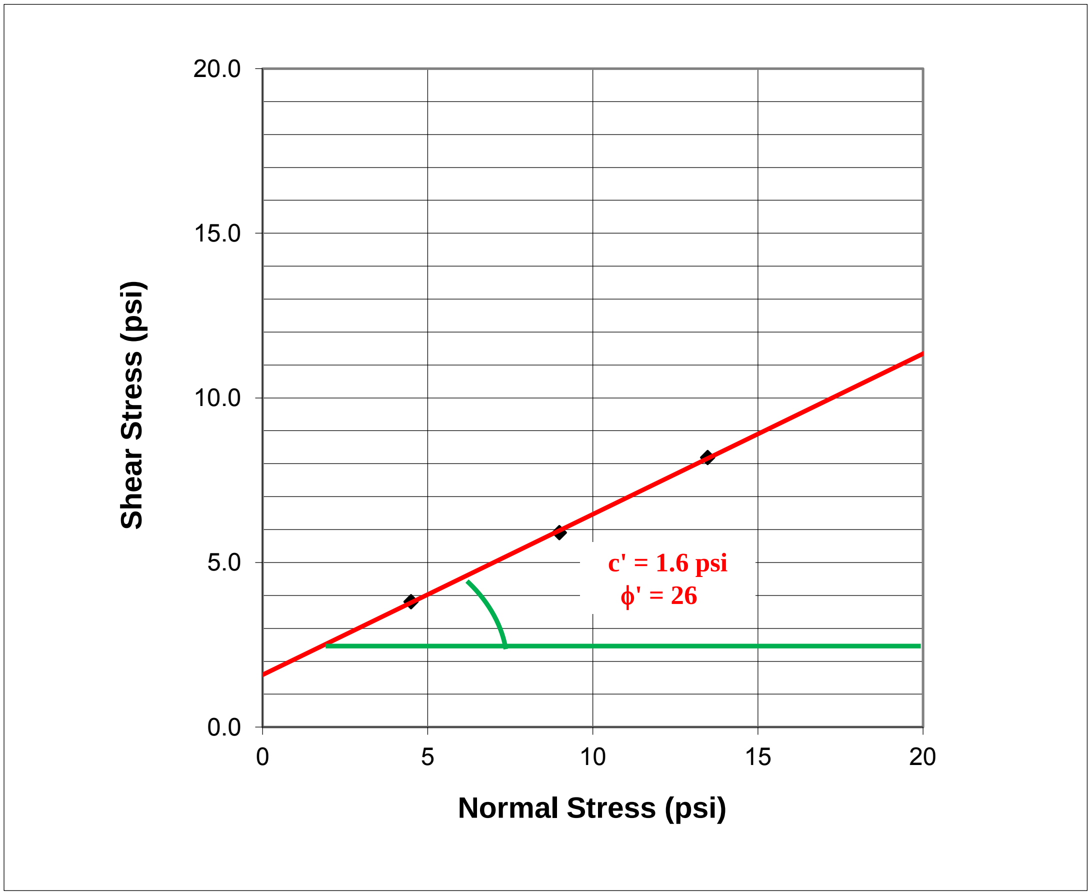
(Consolidated Drained)

Job No.: 13-017
Project: I-40 over White River
Prairie County, Arkansas
Boring No.: 4
Depth, ft: 7-7.5
Sample Description: Gray and brown silty clay

Tested By: MM
Reported by: YZ
Test Date: 3/12/2014

Material Properties: LL = 46, PL = 19, PI = 27; Minus No. 200 Sieve = 97%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	3.8	100.2	23.5
2	9.0	5.9	99.2	23.5
3	13.5	8.2	99.0	23.5



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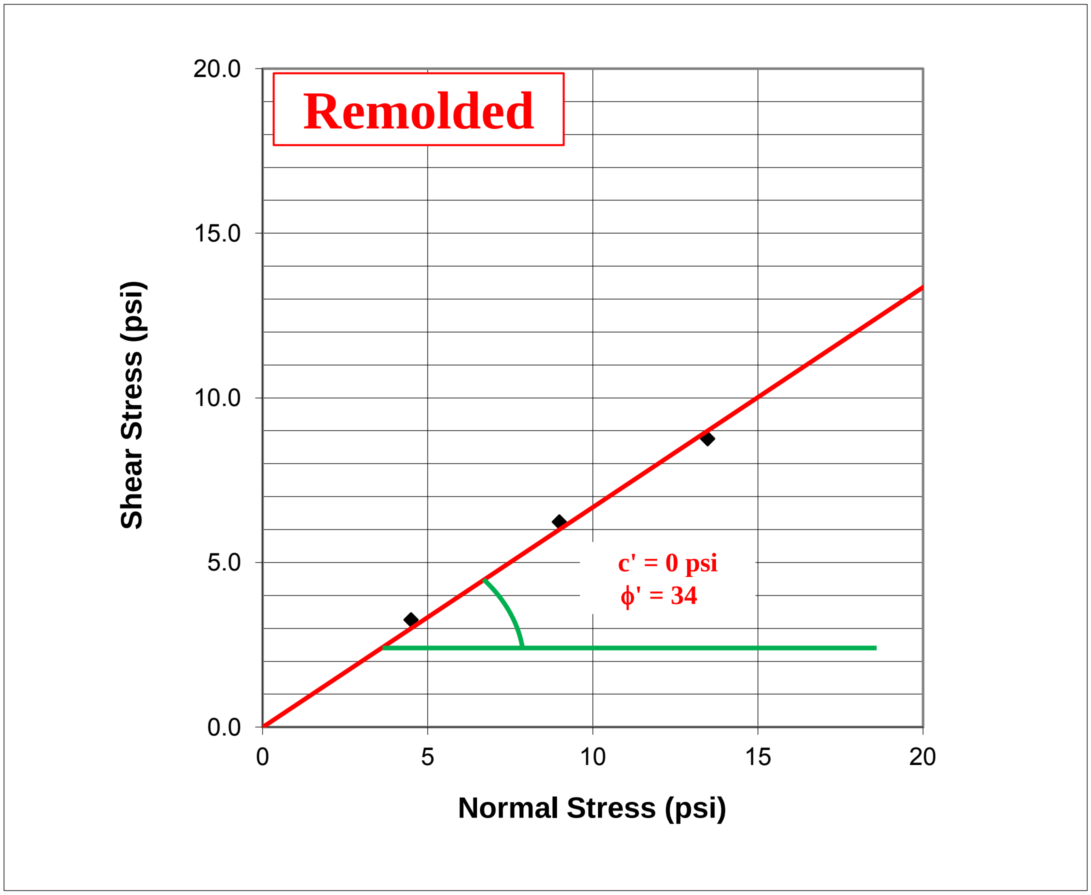
DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.:	13-017	Tested By:	MM
Project:	I-40 over White River	Reported by:	YZ
	Prairie County, Arkansas	Test Date:	2/25/2014
Boring No.:	22		
Depth, ft:	14-25		
Sample Description:	Brown silty fine sand		

Material Properties: Non-plastic; Minus No. 200 Sieve = 13%; USCS = SM; AASHTO = A-4

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	3.3	96.8	24.5
2	9.0	6.2	97.4	24.5
3	13.5	8.8	97.8	24.5



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DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.: 13-017

Project: I-40 over White River

Prairie County, Arkansas

Boring No.: 29

Depth, ft: 14-14.5

Sample Description: Reddish brown clay

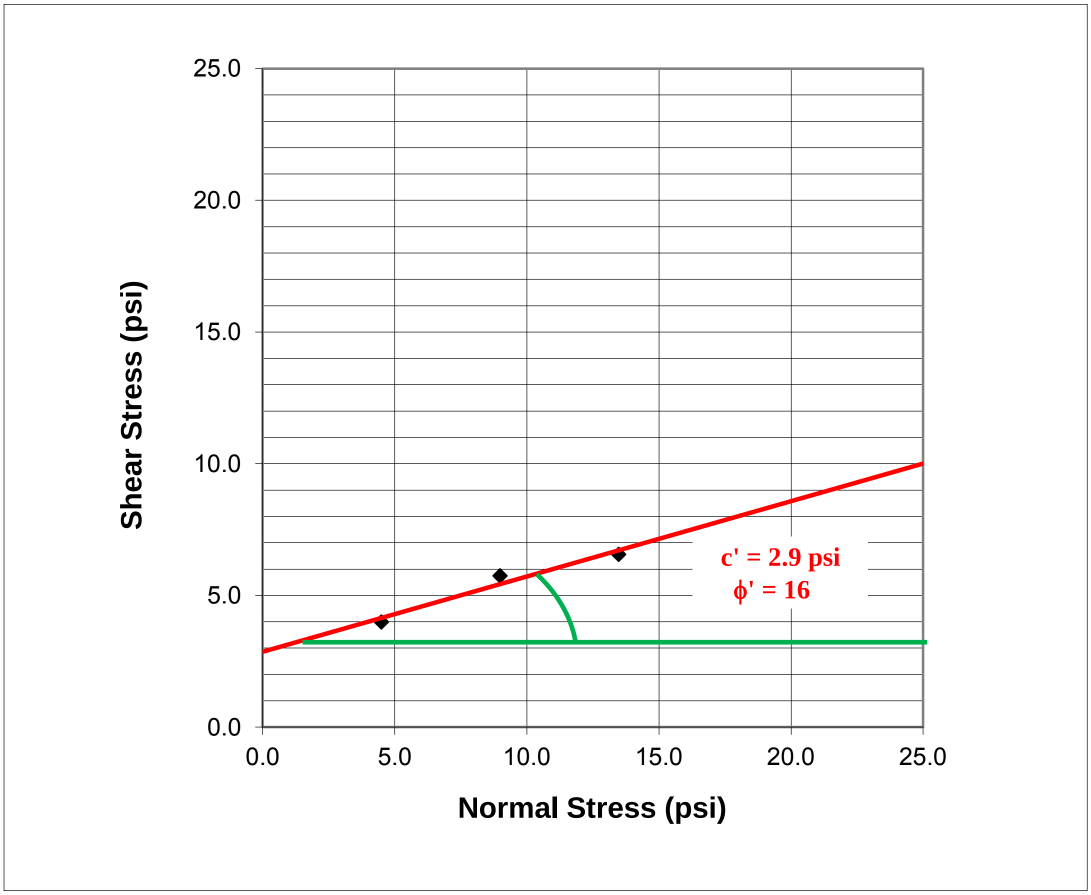
Tested By: MM

Reported by: YZ

Test Date: 5/13/2013

Material Properties: LL = 70, PL = 24, PI = 46; Minus No. 200 Sieve = 99%; USCS = CH; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	4.0	90.2	30.5
2	9.0	5.7	89.5	31.5
3	13.5	6.6	91.6	30.3



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DIRECT SHEAR (AASHTO T-236) TEST RESULTS

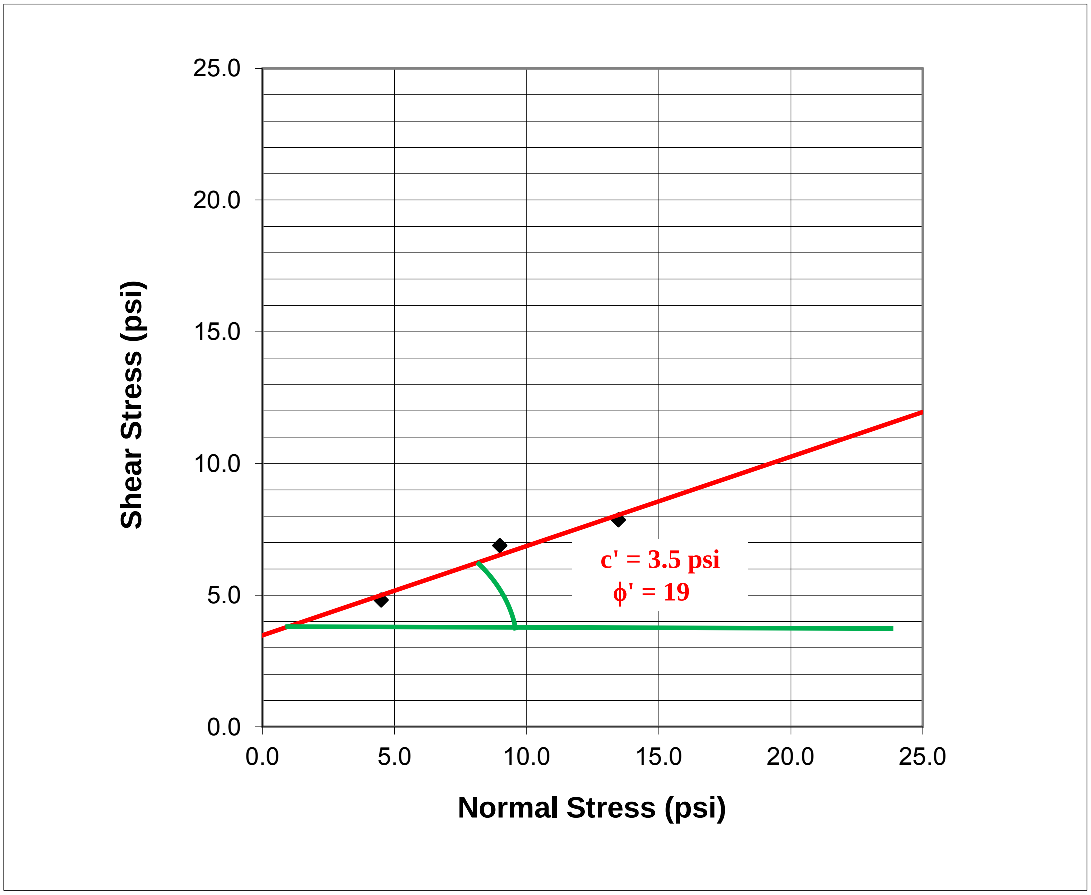
(Consolidated Drained)

Job No.: 13-017
Project: I-40 over White River
Prairie County, Arkansas
Boring No.: 30
Depth, ft: 9.5-10
Sample Description: Tan and gray clay with fine sandy silt pockets

Tested By: MM
Reported by: YZ
Test Date: 5/13/2013

Material Properties: LL = 50, PL = 18, PI = 32; Minus No. 200 Sieve = 87%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	4.8	101.0	24.0
2	9.0	6.9	105.6	19.5
3	13.5	7.9	104.4	20.5



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CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 1
Depth: 9.5-10 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 95.0 pcf
Initial Water Content: 26.4%
Final Water Content: 24.5%
Liquid Limit: 43
Plastic Limit: 19
Minus #200: 97%

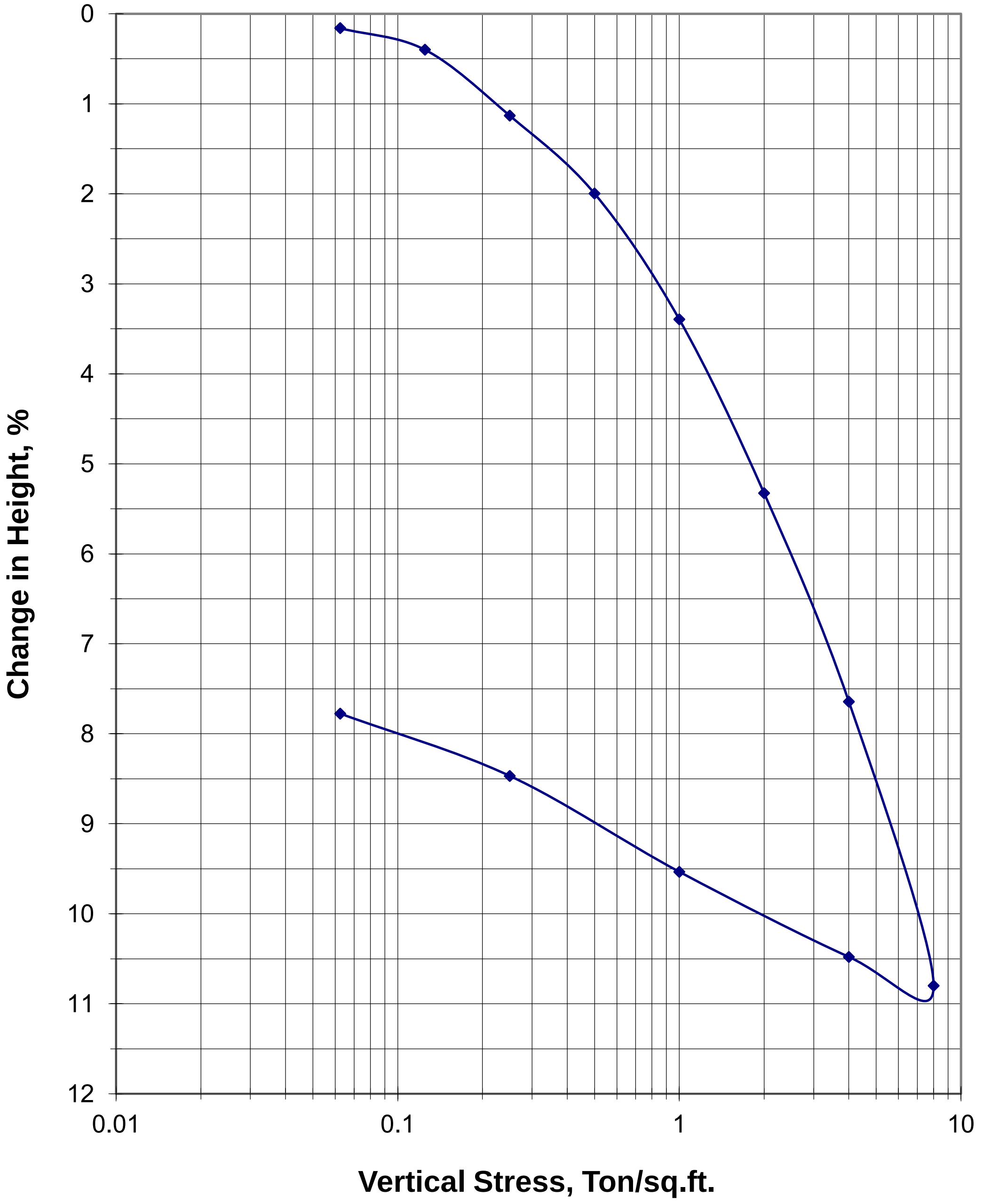

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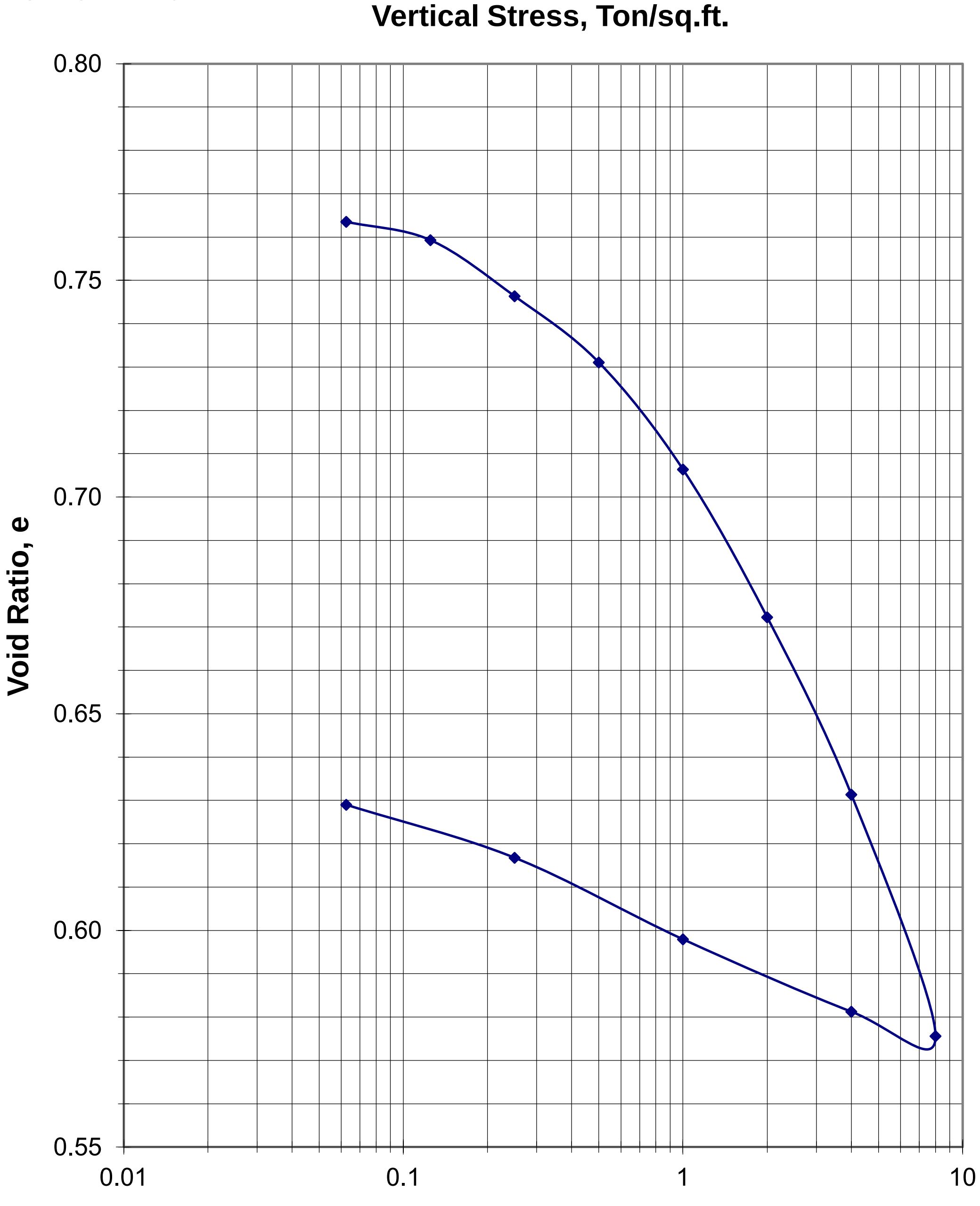
PLATE

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 1
Depth: 9.5-10 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

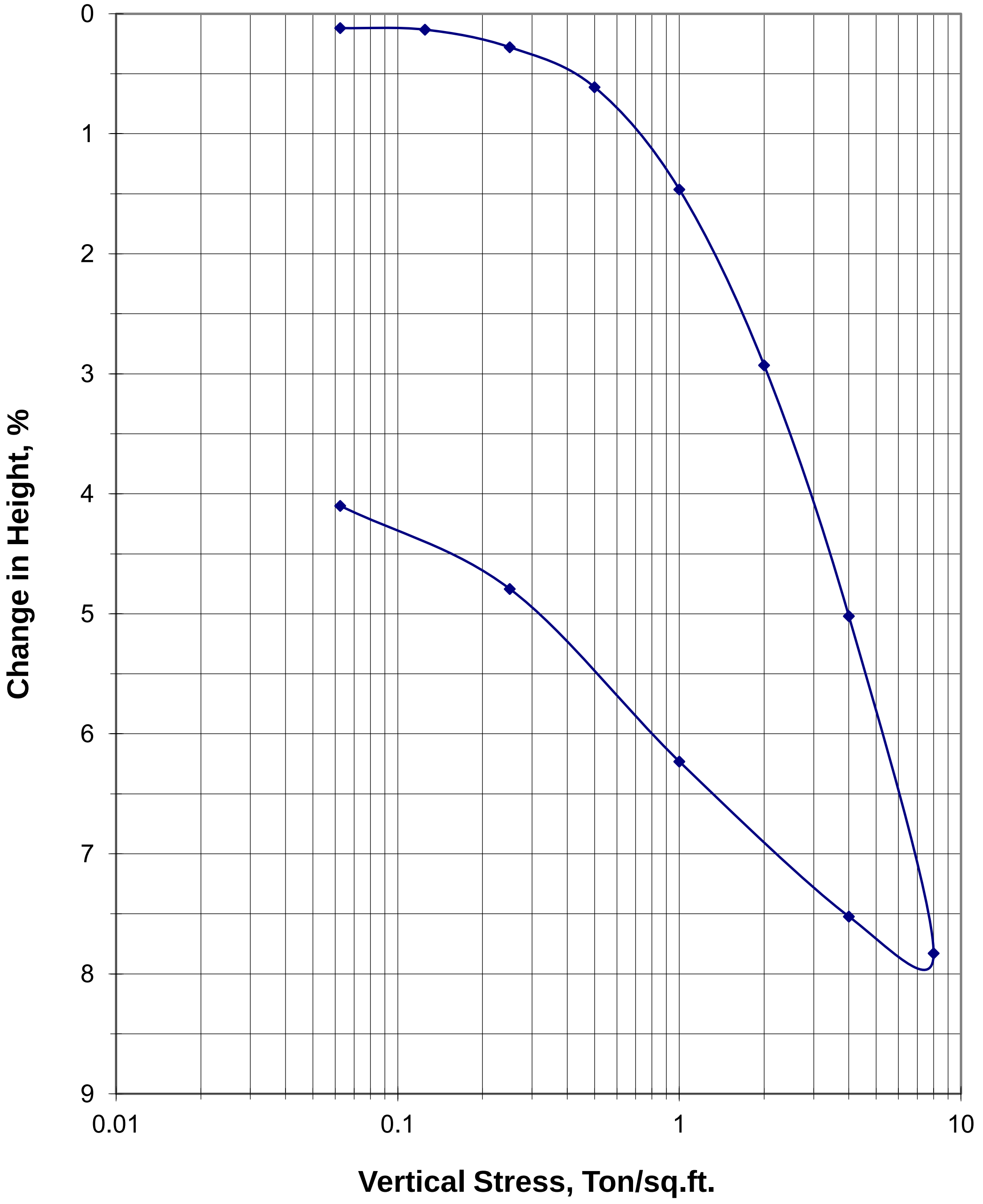
Unit Dry Weight: 95.0 pcf
Initial Water Content: 26.4%
Final Water Content: 24.5%
Liquid Limit: 43
Plastic Limit: 19
Minus #200: 97%



CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 3
Depth: 4.5-5 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

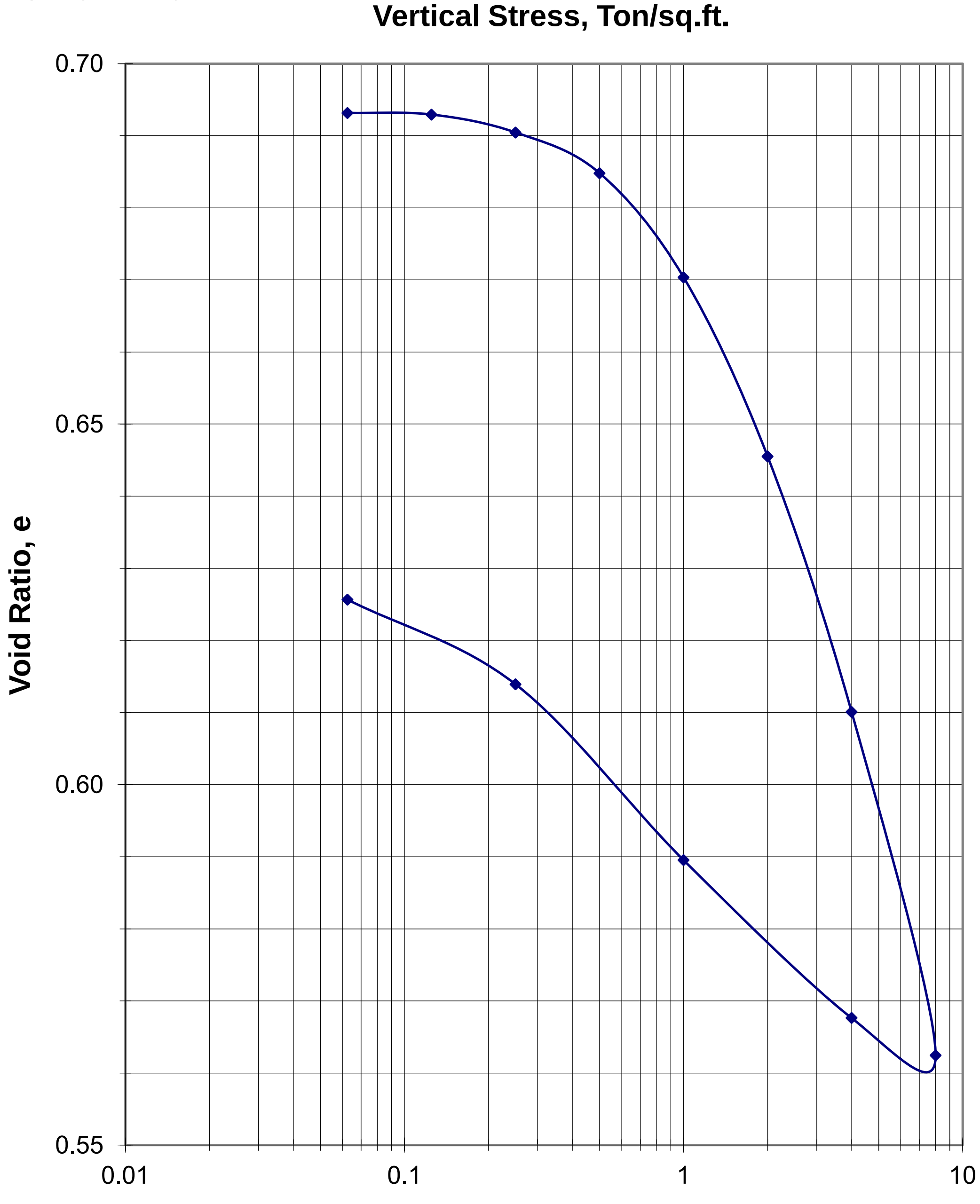
Unit Dry Weight: 99.0 pcf
Initial Water Content: 24.5%
Final Water Content: 25.1%
Liquid Limit: 48
Plastic Limit: 19
Minus #200: 95%



CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 3
Depth: 4.5-5 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 99.0 pcf
Initial Water Content: 24.5%
Final Water Content: 25.1%
Liquid Limit: 48
Plastic Limit: 19
Minus #200: 95%



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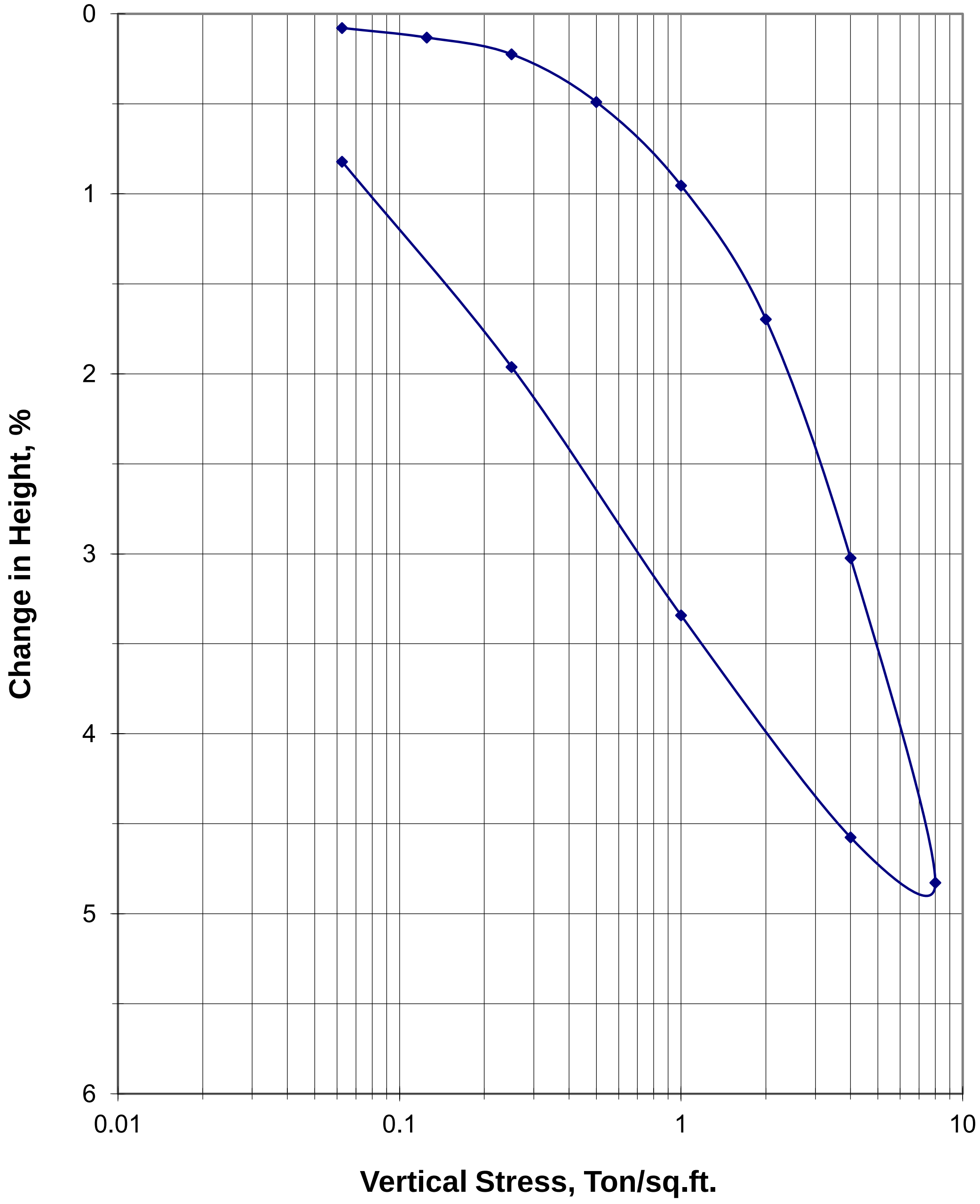
CONSULTING ENGINEERS

PLATE

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 23
Depth: 7-7.5 ft
Description: Brown and tan silty clay
USCS = CL
AASHTO = A-7-6

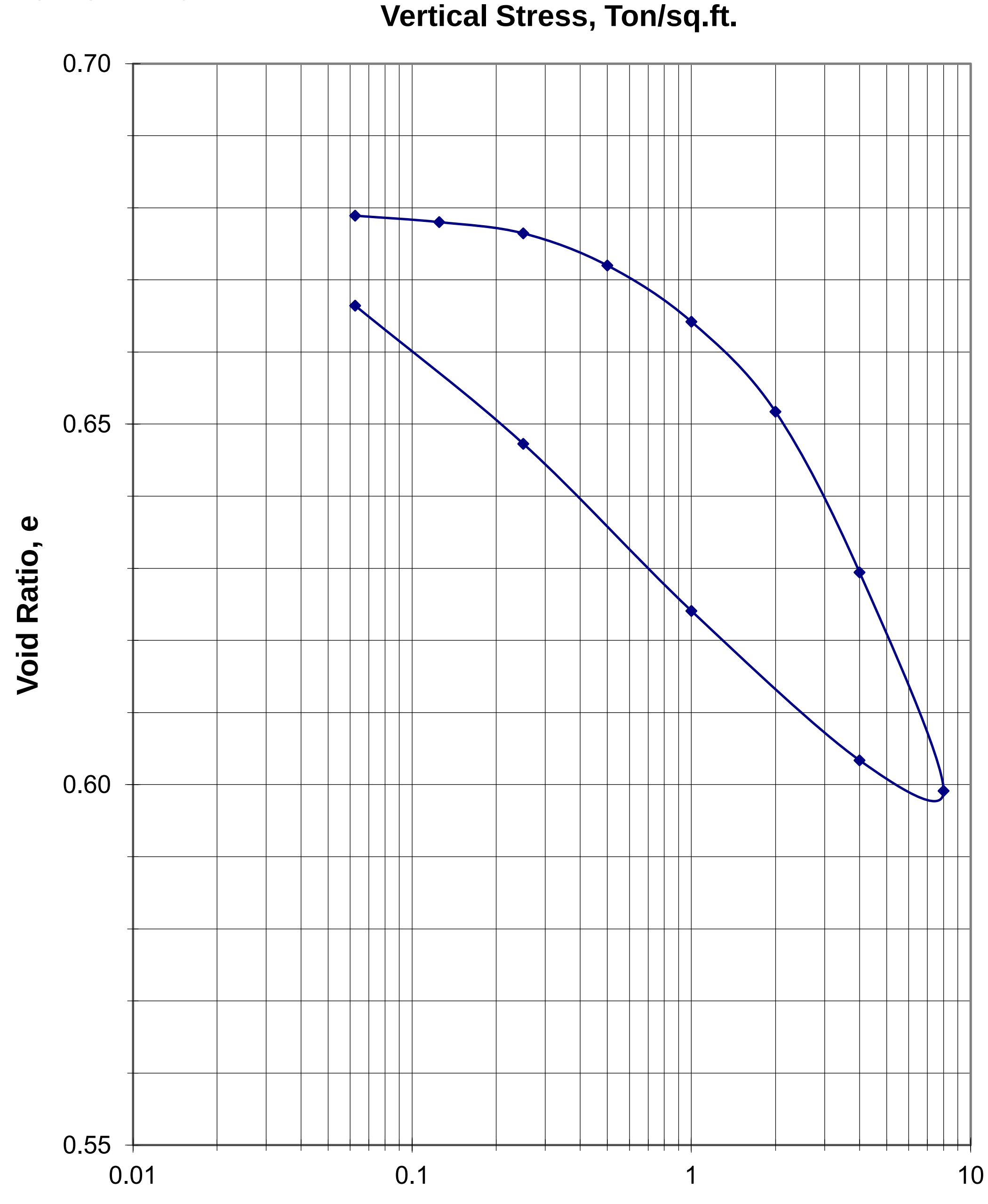
Unit Dry Weight: 99.9 pcf
Initial Water Content: 21.3%
Final Water Content: 27.1%
Liquid Limit: 44
Plastic Limit: 19
Minus #200: 97%



CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 23
Depth: 7-7.5 ft
Description: Brown and tan silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 99.9 pcf
Initial Water Content: 21.3%
Final Water Content: 27.1%
Liquid Limit: 44
Plastic Limit: 19
Minus #200: 97%



APPENDIX D

Final Report:

Site-Specific Ground Motion Response Analysis Results for the I-40 Replacement Bridge over the White River (AHTD Job BB0610)

for:

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May 27, 2014

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EXECUTIVE SUMMARY

Site-specific seismic ground motion response analyses using a weighted combination of equivalent linear (EQL) and nonlinear (NL) methods were used to develop median ground surface spectral acceleration (Sa) estimates for both sides the I-40 replacement bridge over the White River. The final results (i.e., delineated design acceleration response spectra) are summarized below in Figures 3-5 and Table 1. All site-specific ground motion response analyses were conducted in accordance with Section 3.4.3.2 in the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition.

Shear wave velocity (Vs) profiles were determined on both sides of the White River (hereafter referred to as the West and East sides) using a combination of active-source and ambient-wavefield surface wave methods. The use of active-source methods (i.e., the MASW method with both sledgehammer and vibroseis truck sources) allowed for resolution of near-surface layering details, while the use of 2D ambient-wavefield methods (i.e., microtremor array measurements, or MAM) allowed for deep subsurface profiling. The median Vs profile developed for the **West side of the river resulted in an average Vs30m/Vs100ft value of 171 m/s (560 ft/sec); a Site Class E** designation according to Table 3.4.2.1-1 in AASHTO (2011). The median Vs profile developed for the **East side of the river resulted in an average Vs30m/Vs100ft value of 207 m/s (680 ft/sec); a Site Class D** designation. The Vs profiles developed for the West and East side of the river are provided in Figures E1 and E2, respectively. They are discussed in greater detail in the main body of this report.

The Vs profiles on both sides of the river extend to depths exceeding 400 m (1300 ft), where material with an average shear wave velocity greater than 760 m/s (2500 ft/sec) (i.e., Site Class B rock) was encountered. The uncertainty in Vs was accounted for in the site response analyses by adjusting the derived Vs profiles by +/- 15%, and depth to "engineering bedrock", where ground motions were input, by +/- 20%. Uncertainty in the dynamic soil properties assigned to each soil and rock layer was accounted for by using the median and +/- one standard deviation normalized modulus reduction (G/Gmax) and damping (D) curves proposed by Darendeli (2001). Sets of these curves were strength corrected at high shear strains for compatibility with the measured soil shear strengths in the near-surface soil layers.

NL and EQL site response analyses were performed using the 1D site response program DEEPSOIL (http://deepsoil.cee.illinois.edu/). Four ground motions from the NUREG database were spectrally matched to the Site Class B Uniform Hazard Spectrum and used as the input ground motions for this study. The log-normal median surface response resulting from propagating these four input ground motions through each distinct set of Vs and dynamic soil properties was weighted and combined via a logic tree approach in order to determine an overall/"fully weighted" surface response for the West and East sides of the river, respectively. These fully weighted results were used to determine spectral ratios between the median input ground motions and median surface ground motions, which were then multiplied by the Site Class B generic/general acceleration response spectrum in order to determine the median site-specific response spectrum on each side of the river.

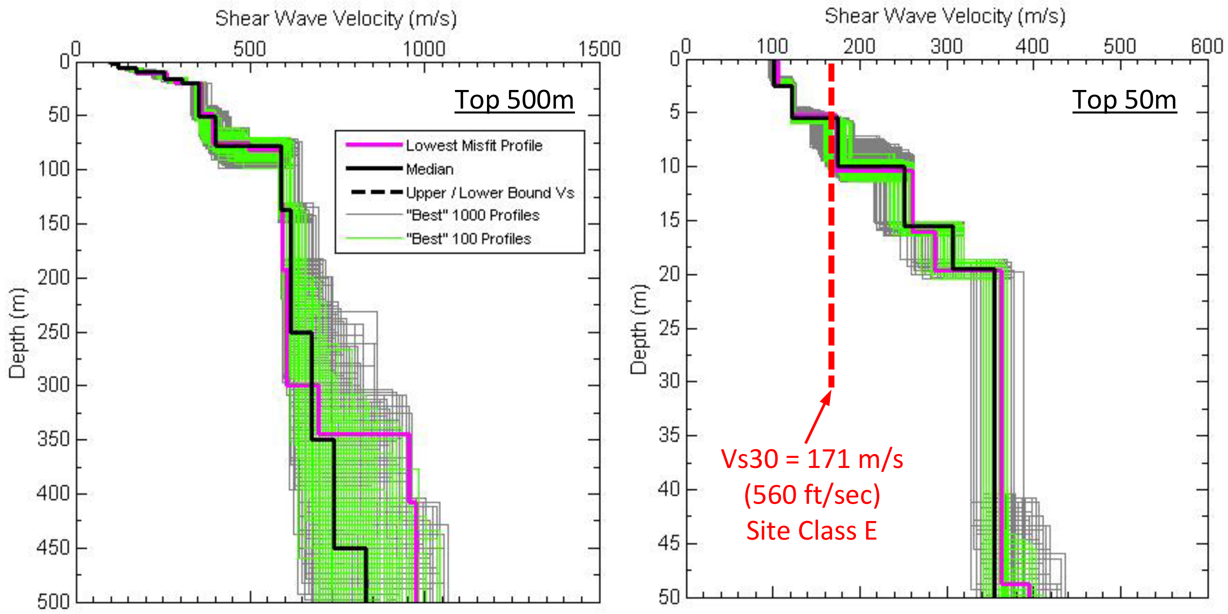

Figure E1. Shear wave velocity (Vs) profiles for the <u>West side</u> of the I-40 White River Bridge. The "best" 1000, "best" 100, and "best"/lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 171 m/s (560 ft/sec), resulting in a Site Class E seismic site classification.

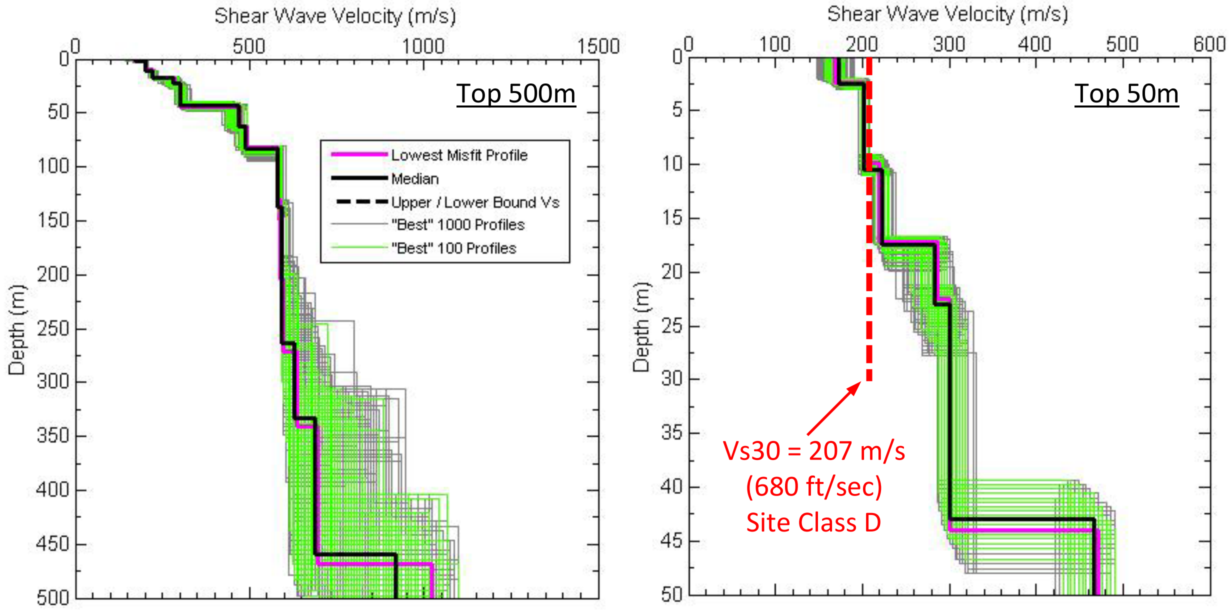

Figure E2. Shear wave velocity (Vs) profiles for the <u>East side</u> of the I-40 White River Bridge. The "best" 1000, "best" 100, and "best"/lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 207 m/s (680 ft/sec), resulting in a Site Class D seismic site classification.

The final results from the site-specific ground motion response analyses are detailed in Figures E3 and E4 for the West and East side of the river, respectively. As noted above, the West side of the river was found to classify as Site Class E in the vicinity where Vs testing was conducted (refer to Figure 1, below, for location). Therefore, the Site Class E spectrum based on the general AASHTO (2011) procedure (Section 3.4.1) is shown for reference in Figure E3. Also shown for reference is a spectrum that is equivalent to 2/3 of the Site Class E general spectrum, because according to Section 3.4.3 of AASHTO (2011), unless otherwise approved by the Owner, the response spectrum used in design shall not be lower than 2/3 of the general site-adjusted spectrum even if site-specific results indicate it is possible. The median site-specific response spectrum on the West side of the river is provided in Figure E3. This response spectrum is less than the allowable minimum of 2/3 Site Class E up to a period (T) of just over 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit, while still remaining below the general Site Class E spectrum, between periods of approximately 0.2 – 1.0 seconds, after which it falls slightly below the limit. The delineated site-specific design response spectrum, highlighted in Figure E3, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the West side of the river.

The East side of the river was found to classify as Site Class D in the vicinity where Vs testing was conducted (refer to Figure 1, below, for location). Therefore, the Site Class D spectrum based on the general AASHTO (2011) procedure is shown for reference in Figure E4, along with a spectrum equivalent to 2/3 of the Site Class D spectrum. The median site-specific response spectrum on the East side of the river is also provided in Figure E4. This response spectrum is generally less than the allowable minimum of 2/3 Site Class D up to a period (T) of approximately 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit between periods of approximately 0.2 – 3.0 seconds, and is essentially equal to the Site Class D general spectrum between periods of 0.7 – 1.0 seconds. The delineated site-specific design response spectrum, highlighted in Figure E4, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the East side of the river.

The site-specific ground motion response analysis results for both the West and East side of the White River are compared, along with their respective delineated design response spectra, in Figure E5. **Tabulated values of Sa, including PGA/As values for use in liquefaction analyses, for each delineated design response spectrum are also provided in Table E1. A corresponding moment magnitude (Mw) of 7.7 should be used in liquefaction analyses**. This is the modal magnitude at the bridge site for every available period in the 2002 USGS deaggregation tool. While the modal magnitude always stays fixed at Mw = 7.7, the mean magnitude consistently shifts upward from approximately Mw = 7.35 for PGA to Mw = 7.57 for the 2-second period data (maximum period available). Basically, this means the modal magnitude and the mean magnitude are expected to grow closer to one another for soft and/or deep soil sites with large natural periods. Since this site has an approximate natural period of 3 seconds, based on the deep Vs profiles shown above, the mean and modal magnitudes are expected to be very close to one another and approximately equal to Mw = 7.7.

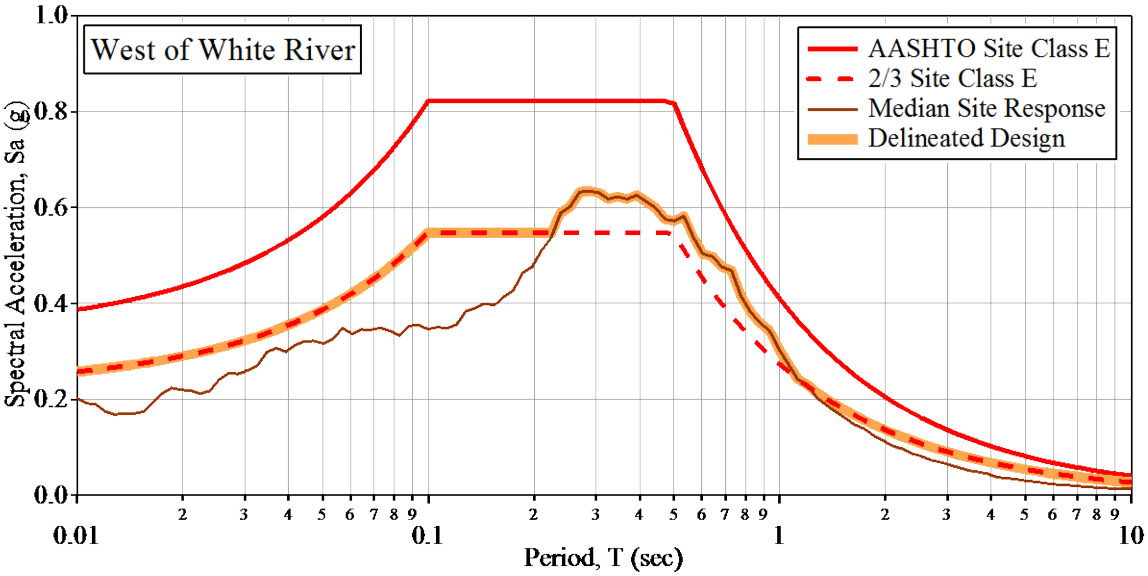

Figure E3. Site-specific ground motion response analysis results for the <u>West side</u> of the White River at I-40.

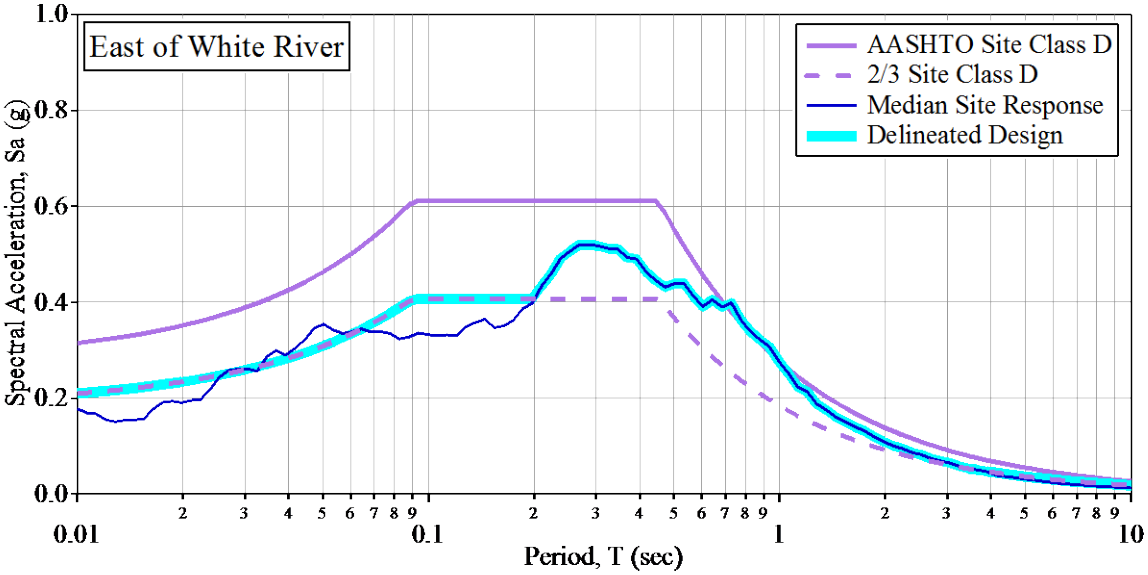

Figure E4. Site-specific ground motion response analysis results for the <u>East side</u> of the White River at I-40.

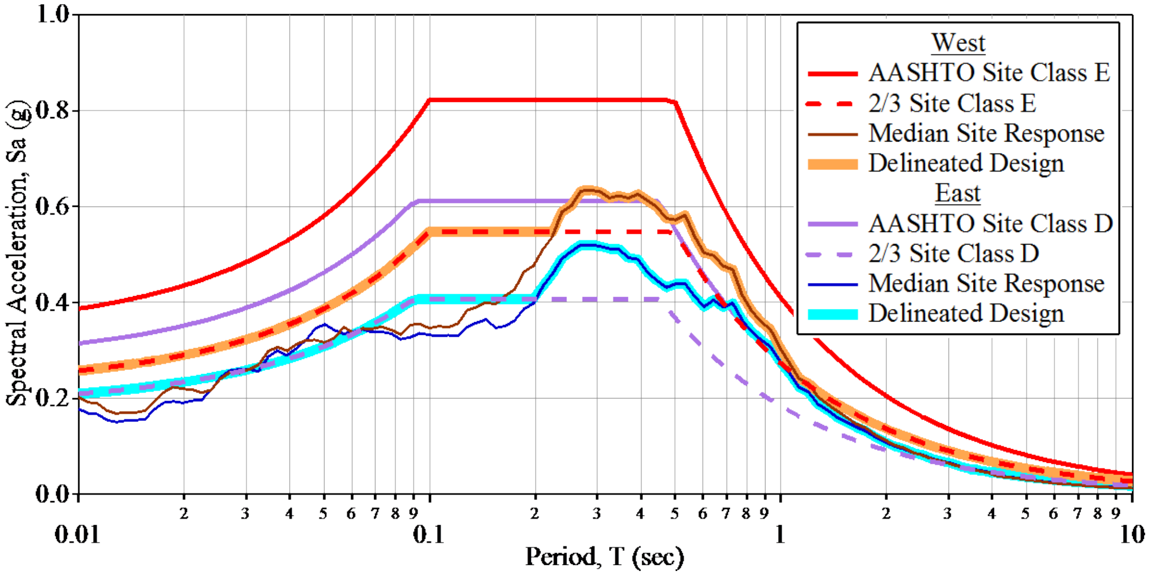

Figure E5. Comparison of site-specific ground motion response analysis results for the <u>West and East side</u> of the White River at I-40.

Table E1. Spectral acceleration (Sa) values for the delineated design response spectra obtained from site-specific ground motion response analyses for the West and East side of the White River at I-40.

West of White River	
Period (sec)	Delineated Design (g)
PGA	0.23
0.010	0.258
0.014	0.271
0.018	0.284
0.022	0.297
0.026	0.310
0.030	0.323
0.040	0.355
0.050	0.387
0.060	0.419
0.070	0.452
0.080	0.484
0.090	0.516
0.10	0.547
0.20	0.547
0.22	0.547
0.24	0.590
0.26	0.615
0.28	0.634
0.30	0.632
0.34	0.621
0.36	0.619
0.42	0.611
0.48	0.574
0.54	0.574
0.60	0.508
0.70	0.473
0.80	0.399
0.90	0.353
1.0	0.302
1.2	0.231
1.4	0.195
1.6	0.171
1.8	0.152
2.0	0.137
2.5	0.109
3.0	0.091
3.5	0.078
4.0	0.068
5.0	0.055
6.0	0.045
7.0	0.039
8.0	0.034
9.0	0.030
10.0	0.027

East of White River	
Period (sec)	Delineated Design (g)
PGA	0.17
0.010	0.210
0.014	0.220
0.018	0.230
0.022	0.239
0.026	0.249
0.030	0.259
0.040	0.284
0.050	0.309
0.060	0.333
0.070	0.358
0.080	0.383
0.090	0.404
0.10	0.408
0.20	0.413
0.22	0.453
0.24	0.494
0.26	0.512
0.28	0.520
0.30	0.518
0.34	0.511
0.36	0.500
0.42	0.461
0.48	0.433
0.54	0.435
0.60	0.394
0.70	0.393
0.80	0.351
0.90	0.316
1.0	0.275
1.2	0.213
1.4	0.169
1.6	0.144
1.8	0.126
2.0	0.108
2.5	0.083
3.0	0.067
3.5	0.053
4.0	0.046
5.0	0.037
6.0	0.031
7.0	0.026
8.0	0.023
9.0	0.020
10.0	0.018

INTRODUCTION

Site-specific seismic ground motion response analyses have been conducted to develop ground surface spectral acceleration (Sa) estimates for both sides the I-40 replacement bridge over the White River in Prairie County, Arkansas (AHTD Job BB0610). All site-specific ground motion response analyses were performed in accordance with Section 3.4.3.2 in the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition. Furthermore, this work has closely followed procedures that were developed for Mack Blackwell Rural Transportation Center (MBTC) project number MBTC 3032, "Site-Specific Seismic Ground Motion Analyses for Transportation Infrastructure in the New Madrid Seismic Zone". The report from that project, written by Cox et al. (2011), is available online at:

http://ww2.mackblackwell.org/web/research/ALL_RESEARCH_PROJECTS/3000s/3032/MBTC-3032FinalReport.pdf

The MBTC 3032 report includes thorough descriptions of the entire site-specific ground motion response analysis process. Herein, we primarily document critical details specific to understanding the analyses conducted for the White River bridge site, and refer the reader to MBTC 3032 for additional, more detailed information.

The approximate latitude and longitude coordinates for the proposed bridge site are provided in Table 1. An aerial photograph of the bridge site is provided in Figure 1.

SHEAR WAVE VELOCITY PROFILES

A combination of active-source and ambient-wavefield surface wave data were acquired on both the West and East sides of the river in order to develop deep shear wave velocity (Vs) profiles for use in the site response analyses. Active-source testing was conducted using the multi-channel analysis of surface waves method (MASW) with both a sledgehammer impact source and a vibroseis truck. Ambient-wavefield surface waves were recorded using 2-dimensional (2D) arrays of sensors; a technique referred to as microtremor array measurements (MAM). The locations of the MASW and MAM arrays are shown in Figure 1.

The following data was collected at the bridge site during field testing: (a) active-source (sledgehammer) stress wave records from 48, 4.5-Hz vertical geophones spaced at 2 m and collected using source offsets of 5, 10, 20 and 30 m, (b) active-source (vibroseis) records from 24, 4.5-Hz vertical geophones spaced at 2 m and collected using source offsets of 5, 10, 20 and 30 m, and (c) ambient-wavefield records from 10 broadband, 3-component seismometers placed in circular arrays with diameters of approximately 60, 200 and 400 m. Note that only one MAM array, roughly 200 m in diameter, was used on the West side of the river due to the difficulty of finding locations for receivers in the flooded timber.

Active-source sledgehammer and vibroseis data were analyzed using the frequency domain beamformer (FDBF) method (Zywicki 1999) coupled with the multiple source-offset technique for identifying near-field contamination and quantifying dispersion uncertainty. The ambient-wavefield MAM data were analyzed using both the 2D high resolution frequency-

Table 1. Site Coordinates for the I-40 replacement bridge over the White River (AHTD Job BB0610).

Latitude	34°50’22.62”	34.83967°
Longitude	-91°27’25.92”	-91.4572°

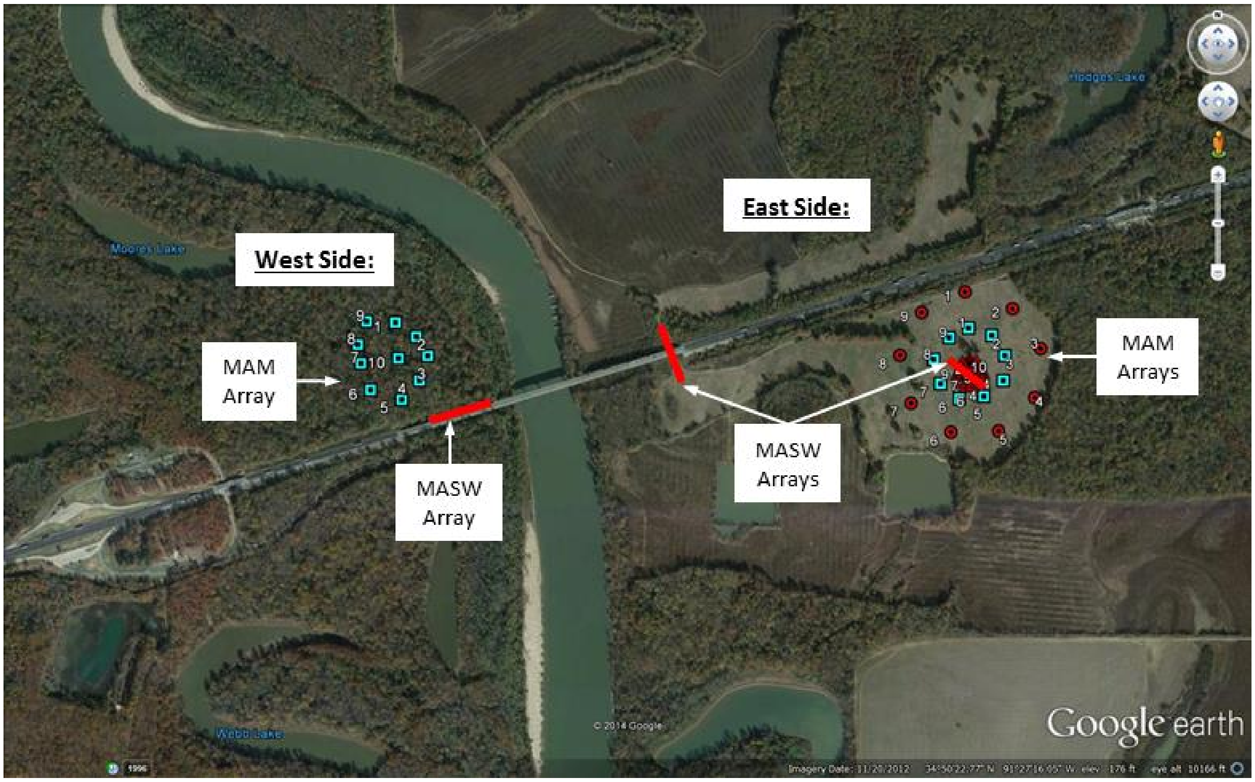

Figure 1. Current I-40 bridge over the White River (location 34.83967° N 91.4572° W). Also shown are the locations of the sensor arrays used for active-source multi-channel analysis of surface waves (MASW) testing and ambient-wavefield microtremor array measurements (MAM) on the West and East sides of the river.

wavenumber (HRFK) method and the Modified Spatial Autocorrelation (MSPAC) method programed in the Geopsy software package (http://www.geopsy.org). More detailed information about the procedures used to develop dispersion estimates from the active and ambient surface wave data may be found in Cox and Wood (2011) and Wood et al. (2014).

The MASW and MAM Rayleigh wave phase velocity dispersion data derived for the West and East sides of the river are compared in Figures 2 and 3, respectively. The agreement between the MASW and MAM data is excellent, particularly considering that the MASW and MAM arrays were not always co-located (refer to Figure 1). Furthermore, dispersion estimates were developed from approximately 1 – 20 Hz on both sides of the river, despite only having one MAM array on the West side. The dispersion data shown in Figures 2 and 3 were averaged in areas of overlap and resampled at a coarser frequency spacing in order to prepare the data for inversion. The resampled data is shown in Figures 4 and 5 for the West and East sides of the river, respectively.

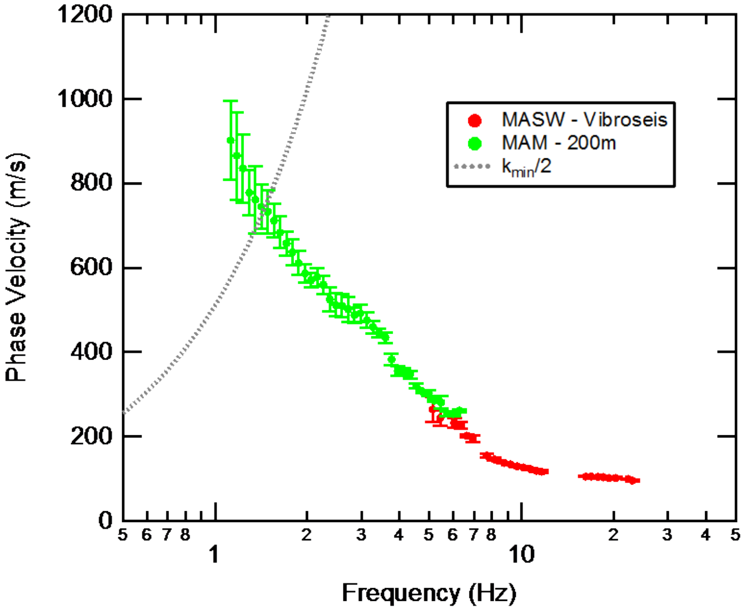

Figure 2. Comparison of MASW and MAM experimental Rayleigh wave phase velocity dispersion data on the West side of the I-40 White River Bridge.

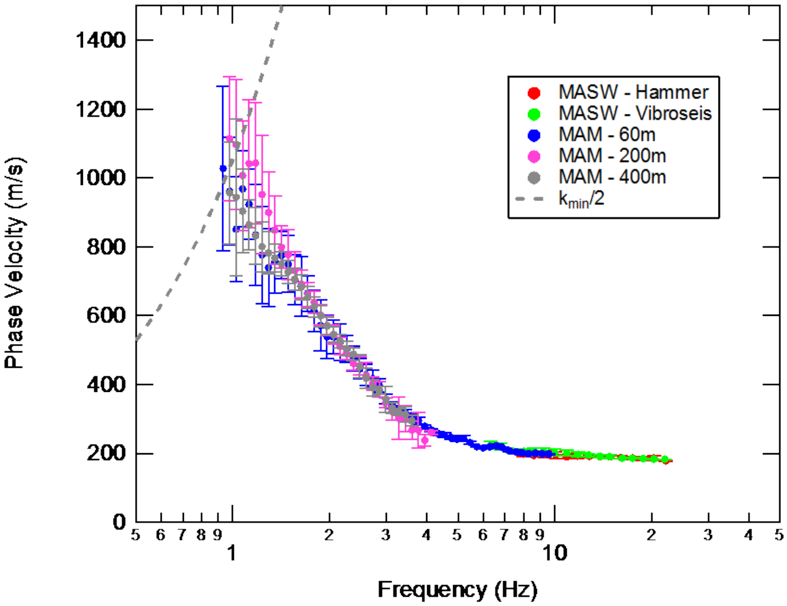

Figure 3. Comparison of MASW and MAM experimental Rayleigh wave phase velocity dispersion data on the East side of the I-40 White River Bridge.

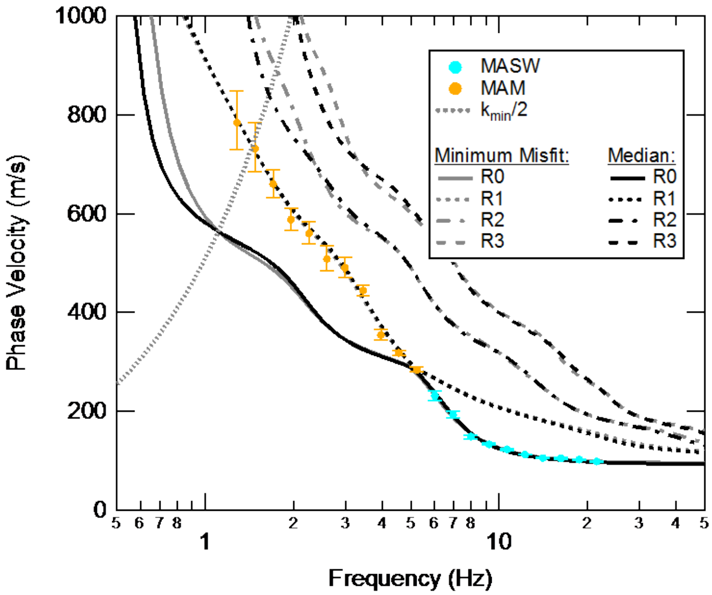

Figure 4. Combined and resampled experimental Rayleigh wave phase velocity dispersion data on the West side of the I-40 White River Bridge. Also shown are the theoretical dispersion curves for the minimum misfit and median shear wave velocity (Vs) profile resulting from the multi-modal surface wave inversion. Note that R0, R1, R2 and R3 correspond to the fundamental, 1st-higher, 2nd-higher and 3rd-higher theoretical modes of Rayleigh wave propagation for the inverted Vs profiles.

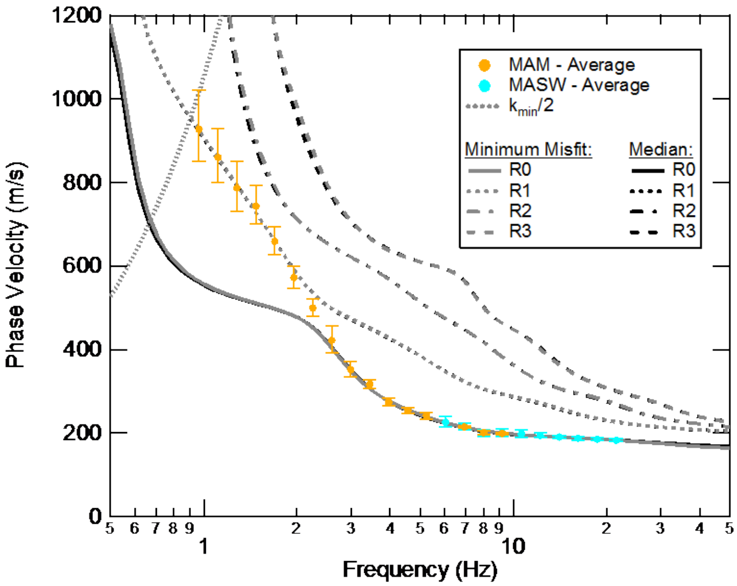

Figure 5. Combined and resampled experimental Rayleigh wave phase velocity dispersion data on the East side of the I-40 White River Bridge. Also shown are the theoretical dispersion curves for the minimum misfit and median shear wave velocity (Vs) profile resulting from the multi-modal surface wave inversion. Note that R0, R1, R2 and R3 correspond to the fundamental, 1st-higher, 2nd-higher and 3rd-higher theoretical modes of Rayleigh wave propagation for the inverted Vs profiles.

The dispersion data shown in Figures 4 and 5 were used to perform a multi-modal inversion in order to obtain a median shear wave velocity profile on each side of the river. Once again, the Geopsy software package was used to perform these inversions. During inversion, over 2 million layered soil profiles were considered using a neighborhood search algorithm. The shear wave velocities for the soil layers were initially estimated as a function of material type/geology and confining pressure, yet given the flexibility to vary within reasonable limits. The saturation depth was set at approximately 2-5 m (6-26 ft) deep (due to the relatively shallow water table near the river), thus constraining the P-wave velocity to 1500 m/s [5000 ft/s] (i.e., effectively fixing Poisson's ratio near 0.5) below this depth; however, at depths where the shear wave velocity exceeded approximately 750 m/s (2500 ft/s), the P-wave velocity constraint was relaxed and Poisson's ratio was allowed to vary between 0.25-0.35. The density of each layer was held constant at 2000 kg/m^3 (125 pcf) regardless of depth or suspected material type. The fundamental mode of Rayleigh wave propagation (R0) was considered, along with the first (R1), second (R2) and third (R3) higher modes. A misfit function between the experimental dispersion data and the theoretical dispersion data resulting from each layered soil profile was used to obtain the shear wave velocity profiles with the minimum misfit (i.e., best fit). The theoretical dispersion curves resulting from the layered profile with the minimum misfit are superimposed on the experimental data in Figures 4 and 5 for the West and East sides of the river, respectively. On the West side of the river, the experimental data was found to transition from R0 to R1 at approximately 5 Hz. On the East side of the river, the experimental data was found to transition from R0 to R1 at approximately 2 Hz. A very good fit to the experimental data was obtained in both cases.

The shear wave velocity profiles resulting from inversion of the dispersion data collected on the West and East sides of the river are shown in Figures 6 and 7, respectively. In both figures, the plot on the left displays the Vs profiles over the top 500 m (1600 ft), while the plot on the right displays the Vs profiles over the top 50 m (160 ft). The 1000 "best", 100 "best" and "best"/minimum misfit Vs profiles are all indicated, along with the median Vs profile of the 1000 "best" profiles. The median Vs profiles on both sides of the river were used as the basis for determining seismic site classification and as the starting profiles for incorporating Vs uncertainty into the site response analyses. The median Vs profile on the West side of the river (refer to Figure 6) resulted in a Vs30/Vs100 value of 171 m/s (560 ft/sec), yielding a Site Class E seismic site classification. The median Vs profile on the East side of the river (refer to Figure 7) resulted in a Vs30/Vs100 value of 207 m/s (680 ft/sec), yielding a Site Class D seismic site classification.

The median Vs profiles were used as the starting profiles for incorporating Vs uncertainty into the site response analyses. Ultimately, three different Vs profiles were used for the site response calculations on each side of the bridge. The three Vs profiles used for the West side of the river are shown in Figure 8, while the three Vs profiles used for the East side of the river are shown in Figure 9. These profiles were developed as follows: (1) The median Vs profiles were decreased and increased by 15% in order to obtain the lower- and upper-bound profiles, respectively; (2) The depth to "engineering bedrock" (i.e., the depth where Vs > 760 m/s, corresponding to Site Class B rock) was estimated between 350 – 450 m (1150-1500 ft) based on the "best" 1000 Vs profiles in Figures 6 and 7. This range corresponds well with estimates of the

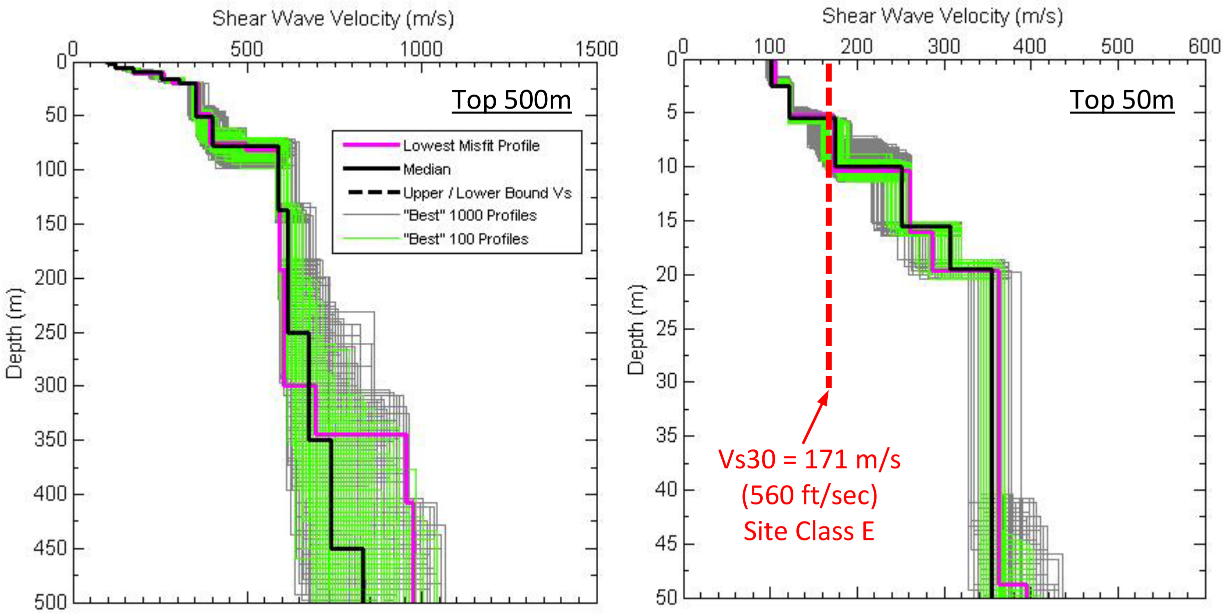

Figure 6. Shear wave velocity (Vs) profiles for the West side of the I-40 White River Bridge. The "best" 1000, "best" 100, and lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 171 m/s (560 ft/sec), resulting in a Site Class E seismic site classification.

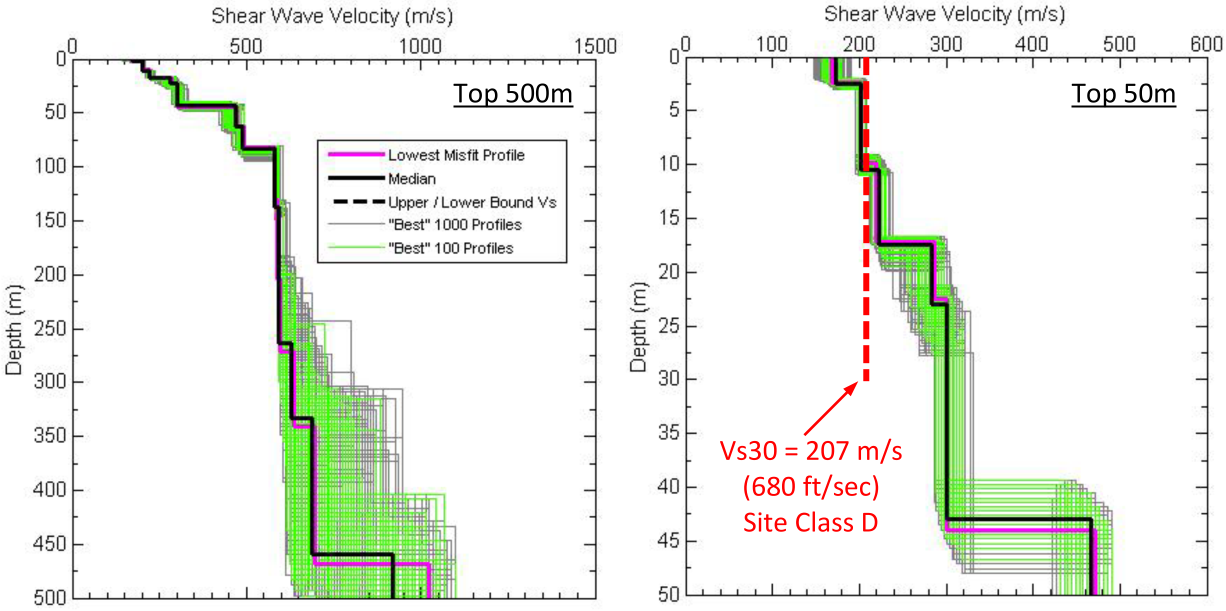

Figure 7. Shear wave velocity (Vs) profiles for the East side of the I-40 White River Bridge. The "best" 1000, "best" 100, and lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 207 m/s (680 ft/sec), resulting in a Site Class D seismic site classification.

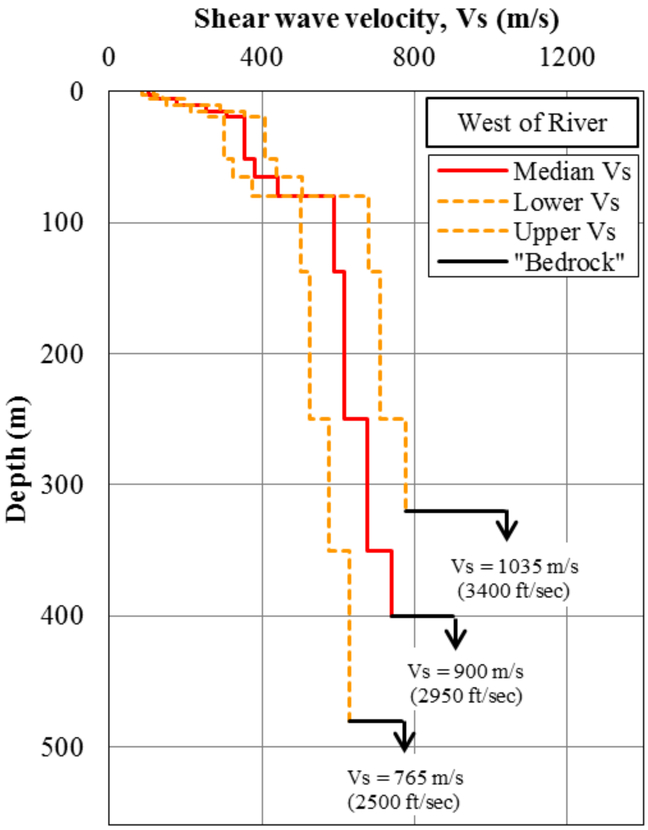

Figure 8. Median, upper- and lower-bound shear wave velocity (Vs) profiles used in the site response analyses conducted for the West side of the I-40 White River Bridge. The arrow locations indicate Vs values and depths assigned to "engineering bedrock", where ground motions were input for site response calculations.

depth to the top of the Mesozoic unit at this location (personal communication from Scott Ausbrooks of AGC and Carson Sloan of AHTD). Therefore, the depth to "engineering bedrock" was set at 400 m (1300 ft) for the median Vs profiles on each side of the river; (3) The depth to "engineering bedrock" was then increased and decreased by 20% in order to account for uncertainty in this depth, with the greater depth to bedrock being associated with the lower-bound Vs profile and the lesser depth to bedrock being associated with the upper-bound Vs profile; (4) An average Vs value of 900 m/s (2950 ft/s) was assigned to the "engineering bedrock", also based on the range of the "best" 1000 profiles on each side of the river. This Vs value was then decreased and increased by 15% in order to account for uncertainty, with the lesser bedrock Vs value associated with the lower-bound Vs profile and the greater bedrock Vs value associated with the upper-bound Vs profile. The depths to bedrock and their corresponding Vs values are indicated in Figures 8 and 9. These depths represent the locations where ground motions were input into the soil profiles for site response analyses.

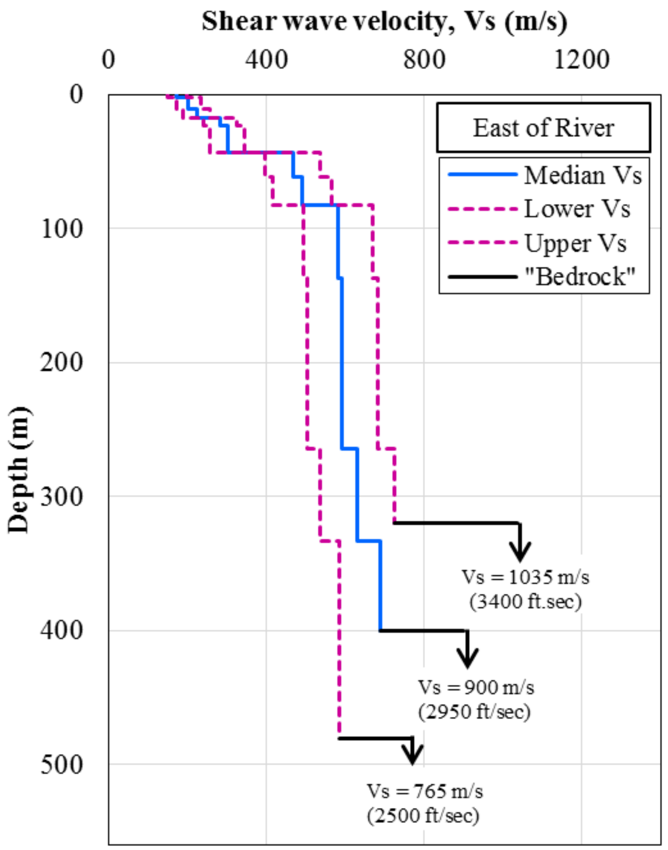

Figure 9. Median, upper- and lower-bound shear wave velocity (Vs) profiles used in the site response analyses conducted for the East side of the I-40 White River Bridge. The arrow locations indicate Vs values and depths assigned to "engineering bedrock", where ground motions were input for site response calculations.

DYNAMIC SOIL PROPERTIES

Dynamic soil properties (i.e., normalized shear modulus reduction [G/G_{max}] and material damping [D]) were developed for each soil layer using the empirical relationships published by Darendeli (2001). Three different sets of dynamic soil properties were utilized in order to account for uncertainty. The standard (i.e., "mean") G/G_{max} and D relationships of Darendeli (2001) were assigned to the median Vs profile, while the plus and minus one standard deviation relationships were assigned to the upper- and lower-bound Vs profiles, respectively. This same convention was followed in MBTC 3032 and the reader may consult that report for additional information.

The dynamic soil property relationships published by Darendeli (2001) are primarily influenced by mean effective confining pressure, soil plasticity index (PI), and overconsolidation ratio (OCR). A simplified soil profile was developed for each side of the bridge to aid in assigning these values to each Vs layer down to bedrock. The near-surface simplified soil profile was primarily derived from boring logs provided by Grubbs, Hoskyn, Barton & Wyatt, Inc., while deeper estimates were made based on geology. If limited information was available, the soil PI was assumed equal to zero and the OCR was assumed equal to one. Reasonable values of soil unit weight were used to estimate effective confining pressures. Figure 10 shows some of the standard G/G_{max} and D curves used for the East side of the river in conjunction with the median Vs profile. These curves were extracted from every 10th layer in the soil profile. In general, the G/G_{max} curves become more linear as the effective confining pressure increases. Thus, the upper-bound G/G_{max} curves in the figure represent soils at greater depths. Conversely, the lower-bound D curves represent soils at greater depths. These curves are primarily shown to provide a visualization that illustrates the range of dynamic soil properties used in the site response analyses with the median Vs profiles. A set of "softer" curves (i.e., minus one standard deviation G/G_{max} curves paired with plus one standard deviation D curves) were used with the lower-bound Vs profiles and a set of "stiffer" curves (i.e., plus one standard deviation G/G_{max} curves paired with minus one standard deviation D curves) were used with the upper-bound Vs profiles.

Implied shear strength corrections, as described in MBTC 3032, were conducted to adjust the G/G_{max} curves on both the West and East sides of the river to depths near 10 m (30 ft). The approximate layer boundaries and soil types for each of these layers are provided in Table 2, along with the target undrained shear strength (Su; for clay layers) or Mohr-Coulomb effective friction angle (ϕ'; for sand layers). These estimates are based on boring logs, discussions with Grubbs, Hoskyn, Barton & Wyatt, Inc., and engineering judgment. Target shear strengths were matched at approximately 3% shear strain.

SITE-SPECIFIC TARGET SPECTRUM

For site-specific ground motion response analyses, a target spectrum (a target for the selection and scaling of input ground motions) was developed to characterize the seismic hazard at the bottom of the soil profile. Herein, the 5% in 50-year uniform hazard spectrum (UHS) serves as that target. The return period for a 5% in 50-year hazard is 1,000 years, which is almost exactly equivalent to the return period for the 7% in 75-year hazard level specified by AASHTO (2011) guidelines. The Java Ground Motion Parameter Calculator (JGMPC) was used to develop a UHS from the 2002 national seismic hazard maps data at periods of 0 (PGA), 0.1, 0.2, 0.3, 0.5, 1.0, and 2.0 seconds. The target spectrum is illustrated in Figure 11. By default, the JGMPC computes hazards for Site Class B. Although the Vs structure varies between the East and West sides of the river, the "engineering bedrock" on both sides is indeed Site Class B. Thus, a single target spectrum was sufficient to describe the seismic hazard on both sides of the river. [Note that even AASHTO 2011 uses the 2002 data rather than the 2008 data; see notes in Figure 3.4.1-2a relative to discussion and references].

INPUT GROUND MOTION SELECTION

Deaggregation results for the site indicate that the seismic hazard is governed by approximately the same earthquake characteristics at each period: a modal magnitude of 7.7 at a distance of approximately 100 km (62 mi). These characteristics were used to select four input

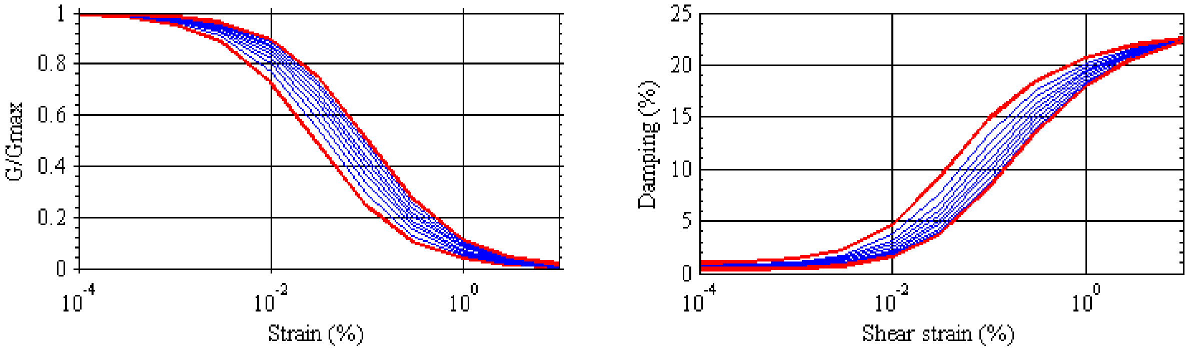

Figure 10. Dynamic soil properties assigned to every 10th layer in the median Vs profile on the East side of the I-40 White River Bridge.

Table 2. Target shear strengths used to perform shear strength corrections for the top three soil layers on both the West and East side of the I-40 White River Bridge.

	Depth	Assigned Soil Type	Target Shear Strength		
			Lower Vs	**Median Vs**	**Upper Vs**
West	Top 2.5 m (8.2 ft)	Clay	Su = 48 kPa	Su = 57 kPa	Su = 65 kPa
	2.5 - 5.5 m (8.2 - 18 ft)	Sand	φ' = 29°	φ' = 32°	φ' = 35°
	5.5 - 10 m (18 - 32.8 ft)	Dense Sand	φ' = 35°	φ' = 38°	φ' = 40°
East	Top 2.5 m (8.2 ft)	Clay	Su = 57 kPa	Su = 67 kPa	Su = 77 kPa
	2.5 - 10.5 m (8.2 - 34.4 ft)	Sand	φ' = 31°	φ' = 34°	φ' = 37°

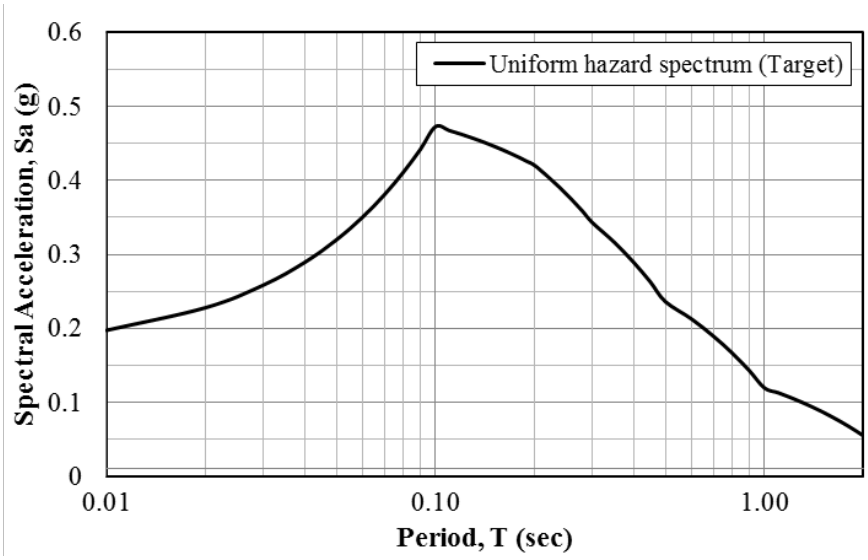

Figure 11. Uniform hazard spectrum (UHS)/target spectrum for the I-40 White River Bridge site developed using the data from the 2002 USGS maps for a 5% in 50-year probability of exceedance (equivalent to 7% in 75-year probability of exceedance).

ground motions from the NUREG database (see MBTC 3032 report for details). Each ground motion was scaled and then spectrally matched to the target spectrum for the White River Bridge site. Figure 12 illustrates the four spectrally matched input ground motions along with the target spectrum. Note that that ground motions match the target spectrum very well at periods greater than 0.1 seconds. The ground motions exceed the target spectrum at shorter periods because the NUREG ground motions were purposely modified to included greater short-period energy as a means to better characterize ground motions expected in the Central and Eastern U.S.

SITE RESPONSE METHODOLOGY

Six distinct sets of site response analyses were conducted for each side of the bridge in an effort to account for uncertainties associated with the input parameters, spatial variability, and methods of analysis. Three types of analyses were conducted to account for uncertainties associated with the soil profile. These include: (1) a softer-bound, defined by the softer set of dynamic soil properties and the lower-bound Vs profile; (2) an average, based on the mean dynamic soil properties and median Vs profile; and (3) a stiffer-bound, defined by the stiffer set of dynamic soil properties and the upper-bound Vs profile. Finally, all of the previous combinations were computed using both equivalent linear (EQL) and non-linear (NL) site response analysis methods. Each of these six distinct analyses were conducted using four scaled and spectrally matched input motions. The result from each type of analysis is described by the lognormal median of the four output (surface) response spectra. This is very similar to the "logic tree" approach that is described in detail in the MTBC 3032 report, except that the West and East sides of the bridge were analyzed separately. For clarity, the weighting factors used for this study are summarized in Figure 13.

RESULTS

All site response analyses were performed using the 1D site response program DEEPSOIL. The log-normal median surface response spectra resulting from six distinct site response analyses were weighted and combined using a logic tree approach in order to determine overall/"fully weighted" surface responses for the West and East sides of the river, respectively.

The output from DEEPSOIL is summarized in Figures 14 and 15 for the West and East sides of the bridge, respectively. The lognormal median response from the four input ground motions is plotted for each of the six distinct site response analyses described above (i.e., three different soil profiles with two types of analyses). The fully weighted response represents a single surface response spectrum obtained by applying the weighting factors from Figure 13 to each of the six lognormal median responses. For both sides of the bridge, ground motion amplification occurs at periods greater than approximately 0.2 sec. These fully weighted results were used to determine spectral ratios between surface ground motions and input ground motions, which were then multiplied by the generic Site Class B acceleration response spectrum (corresponding to the "engineering bedrock") in order to compute a site-specific response spectrum for each side of the river. The results are plotted along with the generic response

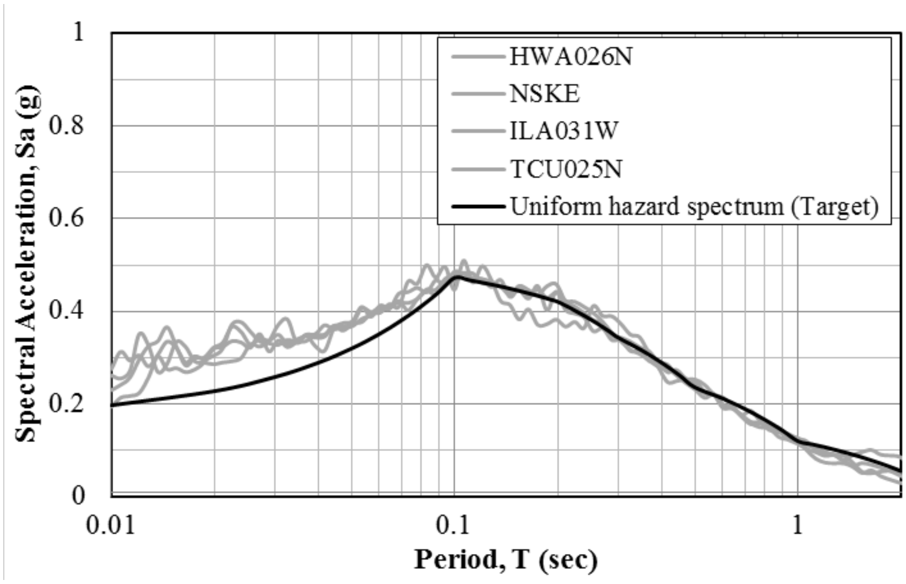

Figure 12. Uniform hazard spectrum (UHS)/target spectrum for the I-40 White River Bridge site with the four scaled and spectrally matched input ground motions selected for use in site response analyses.

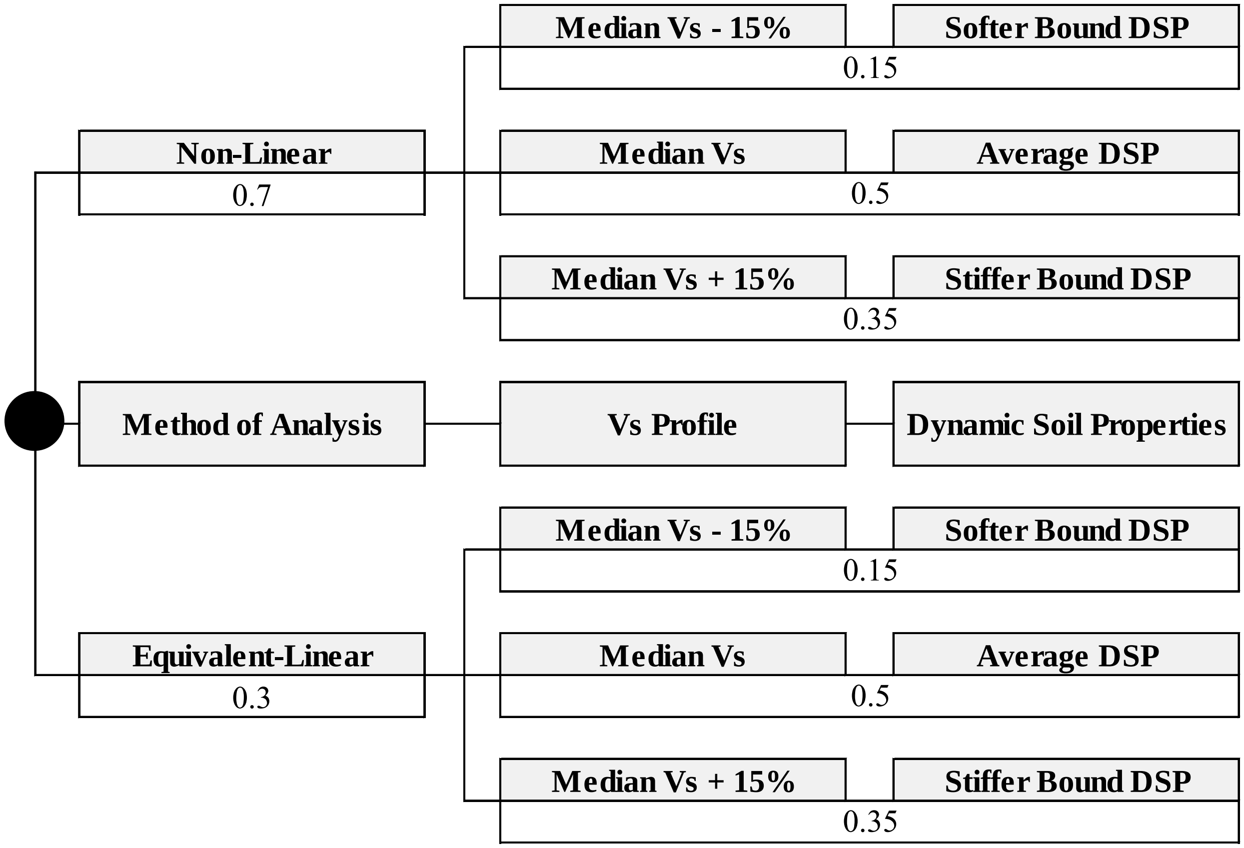

Figure 13. Weighting Factors Used to Combine Site Response Analysis Results that Utilized Different Methods of Analyses, Vs Profiles, and Dynamic Soil Properties (DSP) into a Single, "Fully Weighted" Surface Response Spectrum for Each Side of the Bridge.

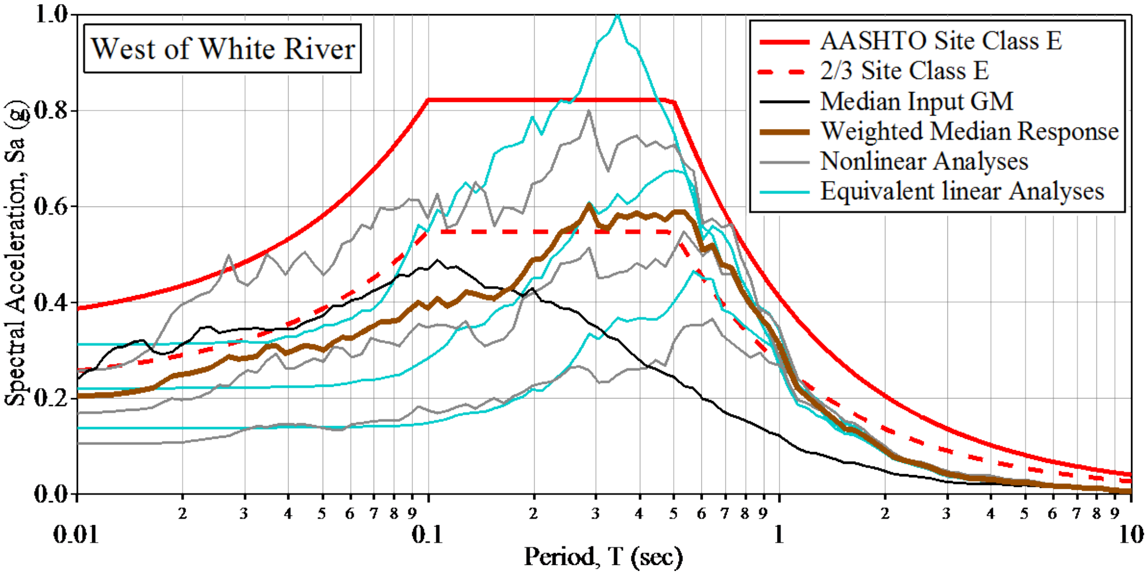

Figure 14. Log-normal median surface response from each distinct set of site response analyses, log-normal median of the input ground motions, generic Site Class E response spectrum, and the fully weighted median response spectra for the West side of the White River bridge over I-40.

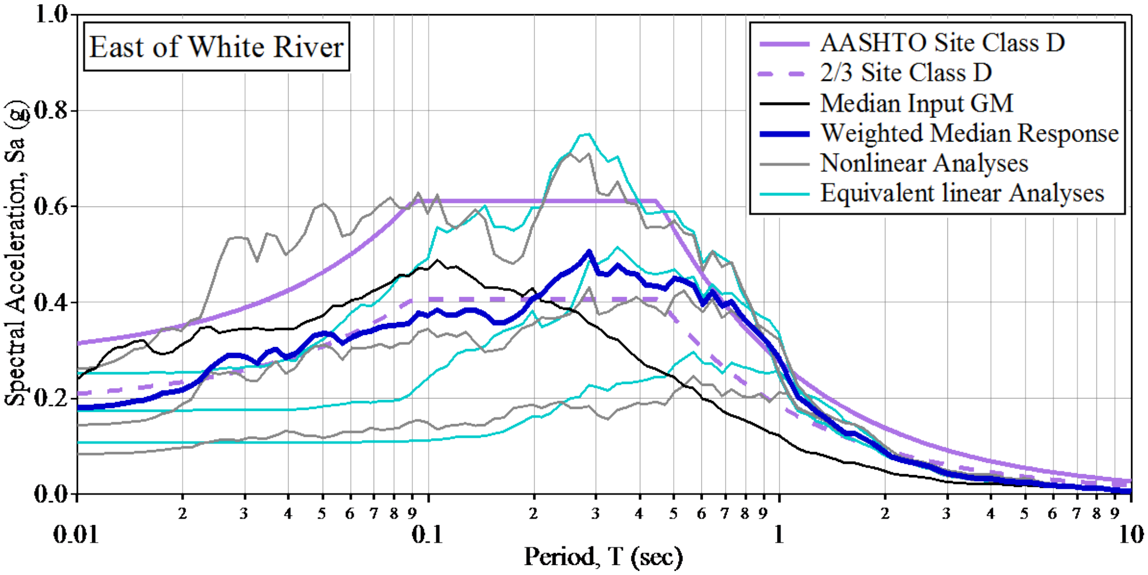

Figure 15. Log-normal median surface response from each distinct set of site response analyses, log-normal median of the input ground motions, generic Site Class D response spectrum, and the fully weighted median response spectra for the East side of the White River bridge over I-40.

spectrum that would be utilized in the absence of site-specific ground motion response analyses. This procedure is described more thoroughly in MBTC-3032.

The final results from the site-specific ground motion response analyses are detailed in Figures 16 and 17 for the West and East side of the river, respectively. As noted above, the West side of the river was found to classify as Site Class E in the vicinity where Vs testing was conducted. Therefore, the Site Class E spectrum based on the general AASHTO (2011) procedure (Section 3.4.1) is shown for reference in Figure 16. Also shown for reference is a spectrum that is equivalent to 2/3 of the Site Class E general spectrum, because according to Section 3.4.3 of AASHTO (2011), unless otherwise approved by the Owner, the response spectrum used in design shall not be lower than 2/3 of the general spectrum even if site-specific results indicate it is possible. The median site-specific response spectrum on the West side of the river is provided in Figure 16. This response spectrum is less than the allowable minimum of 2/3 Site Class E up to a period (T) of just over 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit, while still remaining below the general Site Class E spectrum, between periods of approximately 0.2 – 1.0 seconds, after which it falls slightly below the limit. The delineated site-specific design response spectrum, highlighted in Figure 16, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the West side of the river.

The East side of the river was found to classify as Site Class D in the vicinity where Vs testing was conducted. Therefore, the Site Class D spectrum based on the general AASHTO (2011) procedure is shown for reference in Figure 17 along with a spectrum equivalent to 2/3 of the Site Class D spectrum. The median site-specific response spectrum on the East side of the river is also provided in Figure 17. This response spectrum is generally less than the allowable minimum of 2/3 Site Class D up to a period (T) of approximately 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit between periods of approximately 0.2 – 3.0 seconds, and is essentially equal to the Site Class D general spectrum between periods of 0.7 – 1.0 seconds. The delineated site-specific design response spectrum, highlighted in Figure 17, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the East side of the river.

The site-specific ground motion response analysis results for both the West and East side of the White River are compared, along with their respective delineated design response spectra, in Figure 18. **Tabulated values of Sa, including PGA/As values for use in liquefaction analyses, for each delineated design response spectrum are also provided in Table 3. A corresponding moment magnitude (Mw) of 7.7 should be used in liquefaction analyses**. This is the modal magnitude at the bridge site for every available period in the 2002 USGS deaggregation tool. While the modal magnitude always stays fixed at Mw = 7.7, the mean magnitude consistently shifts upward from approximately Mw = 7.35 for PGA to Mw = 7.57 for the 2-second period data (maximum period available). Basically, this means the modal magnitude and the mean magnitude are expected to grow closer to one another for soft and/or deep soil sites with large natural periods. Since this site has an approximate natural period of 3 seconds, based on the deep Vs profiles shown above, the mean and modal magnitudes are expected to be very close to one another and approximately equal to Mw = 7.7.

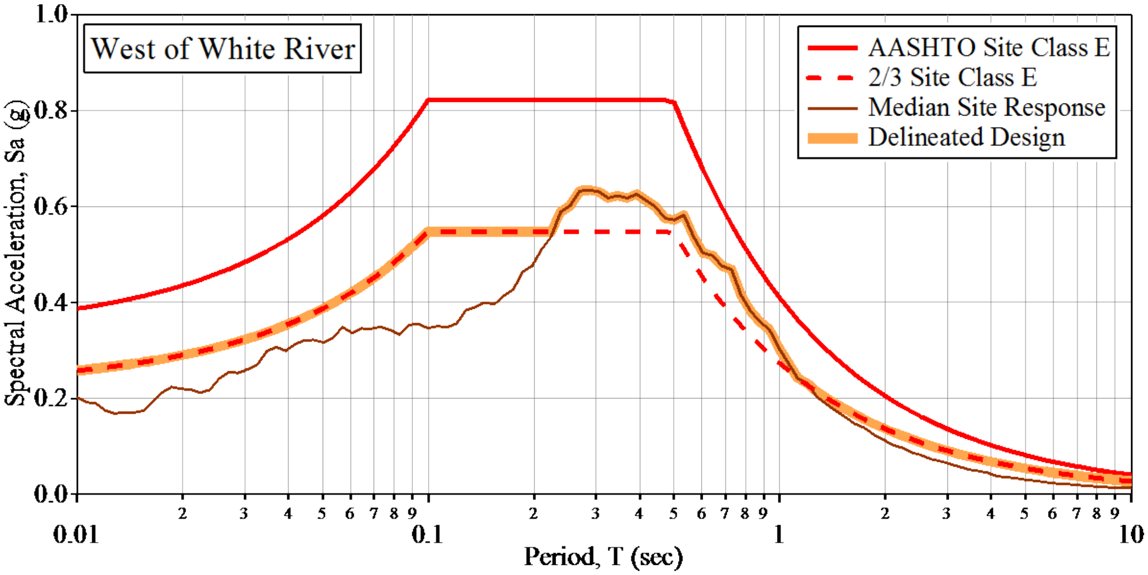

Figure 16. Site-specific ground motion response analysis results for the West side of the White River at I-40.

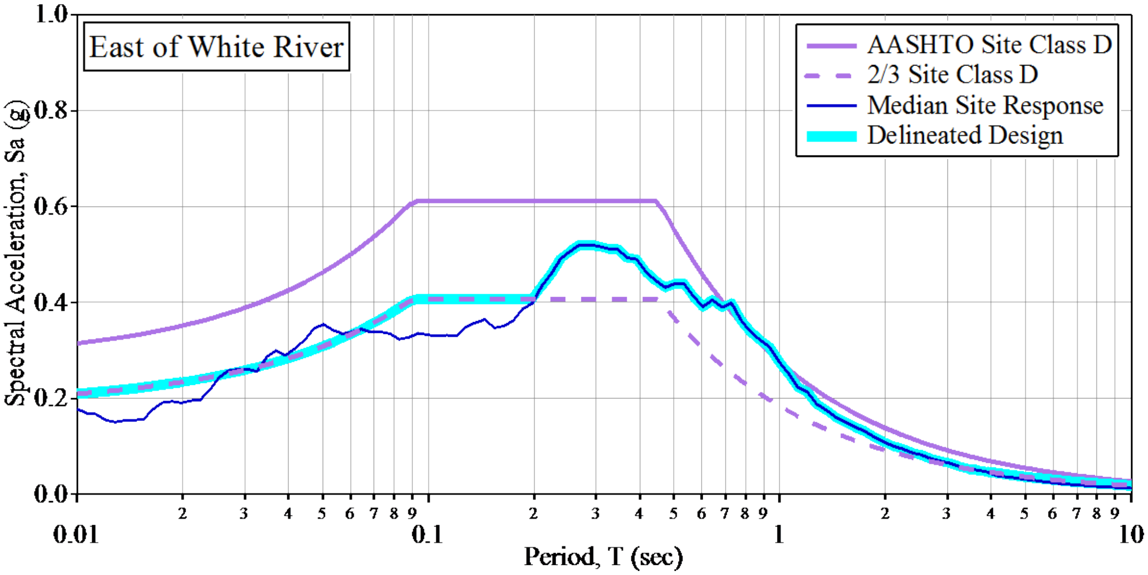

Figure 17. Site-specific ground motion response analysis results for the East side of the White River at I-40.

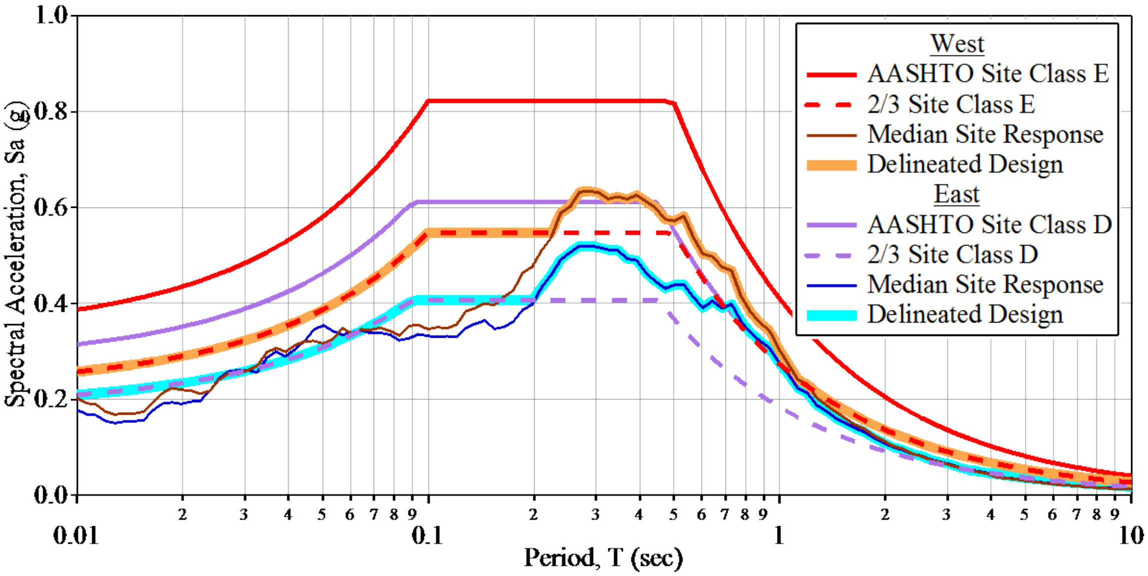

Figure 18. Comparison of site-specific ground motion response analysis results for the <u>West and East side</u> of the White River at I-40.

Table 3. Spectral acceleration (Sa) values for the delineated design response spectra obtained from site-specific ground motion response analyses for the West and East side of the White River at I-40.

West of White River	
Period (sec)	Delineated Design (g)
PGA	0.23
0.010	0.258
0.014	0.271
0.018	0.284
0.022	0.297
0.026	0.310
0.030	0.323
0.040	0.355
0.050	0.387
0.060	0.419
0.070	0.452
0.080	0.484
0.090	0.516
0.10	0.547
0.20	0.547
0.22	0.547
0.24	0.590
0.26	0.615
0.28	0.634
0.30	0.632
0.34	0.621
0.36	0.619
0.42	0.611
0.48	0.574
0.54	0.574
0.60	0.508
0.70	0.473
0.80	0.399
0.90	0.353
1.0	0.302
1.2	0.231
1.4	0.195
1.6	0.171
1.8	0.152
2.0	0.137
2.5	0.109
3.0	0.091
3.5	0.078
4.0	0.068
5.0	0.055
6.0	0.045
7.0	0.039
8.0	0.034
9.0	0.030
10.0	0.027

East of White River	
Period (sec)	Delineated Design (g)
PGA	0.17
0.010	0.210
0.014	0.220
0.018	0.230
0.022	0.239
0.026	0.249
0.030	0.259
0.040	0.284
0.050	0.309
0.060	0.333
0.070	0.358
0.080	0.383
0.090	0.404
0.10	0.408
0.20	0.413
0.22	0.453
0.24	0.494
0.26	0.512
0.28	0.520
0.30	0.518
0.34	0.511
0.36	0.500
0.42	0.461
0.48	0.433
0.54	0.435
0.60	0.394
0.70	0.393
0.80	0.351
0.90	0.316
1.0	0.275
1.2	0.213
1.4	0.169
1.6	0.144
1.8	0.126
2.0	0.108
2.5	0.083
3.0	0.067
3.5	0.053
4.0	0.046
5.0	0.037
6.0	0.031
7.0	0.026
8.0	0.023
9.0	0.020
10.0	0.018

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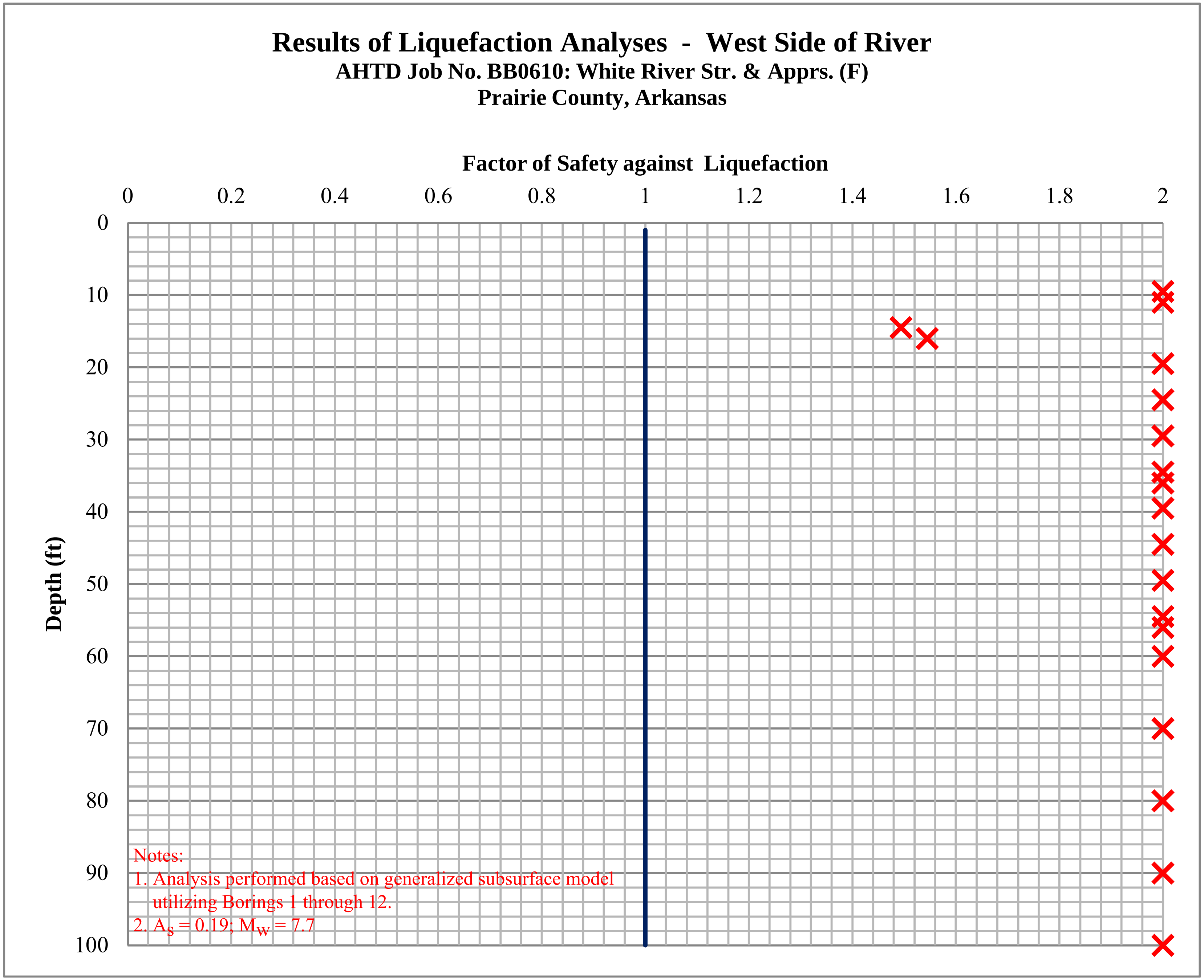
APPENDIX E



Results of Liquefaction Analyses - West Side of River

AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas

Notes:

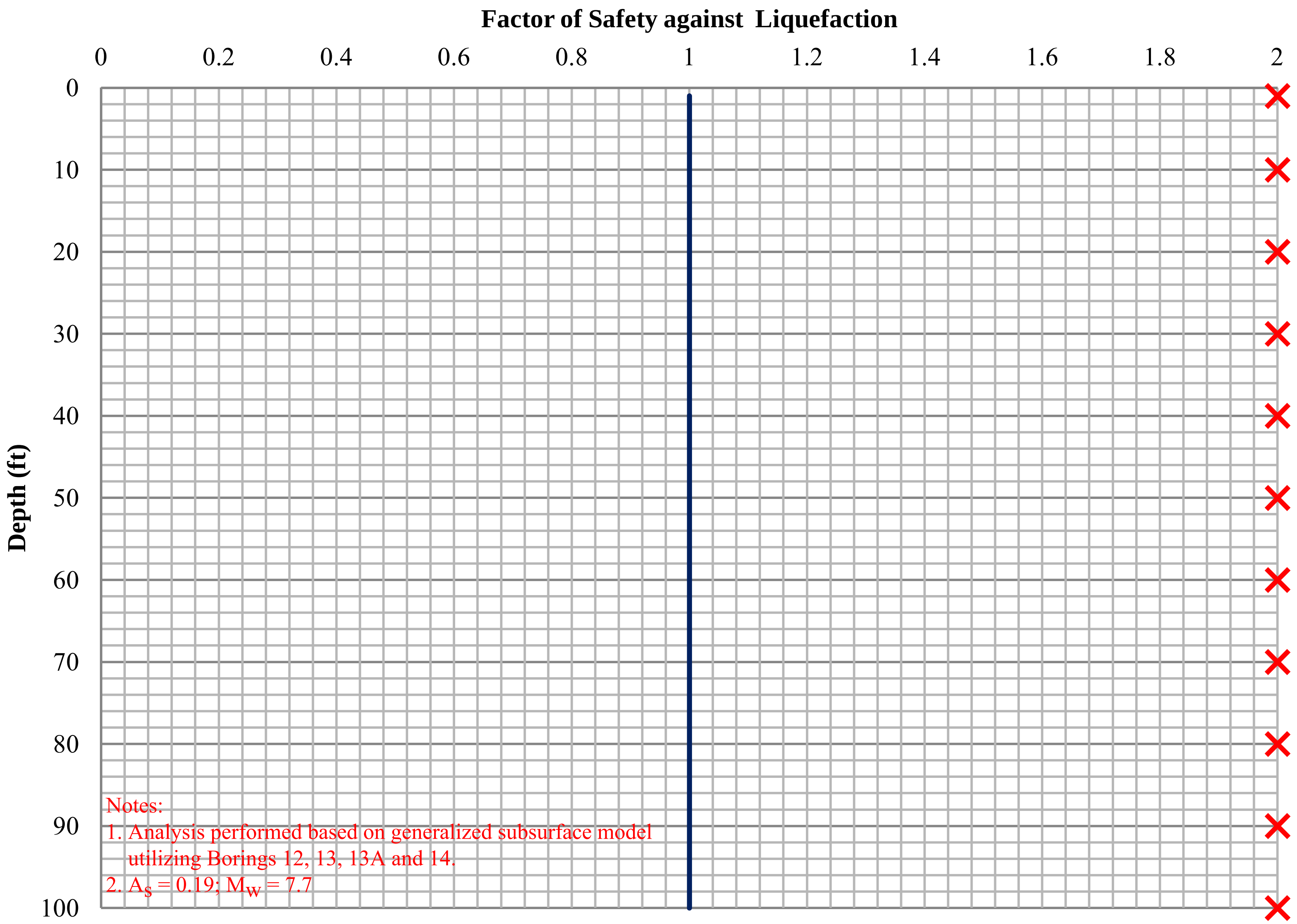
1. Analysis performed based on generalized subsurface model utilizing Borings 1 through 12.
2. $A_S = 0.19$; $M_W = 7.7$

GRUBBS, HOSKYN, BARTON & WYATT, INC.

Consulting Engineers

Results of Liquefaction Analyses - River Channel

AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas



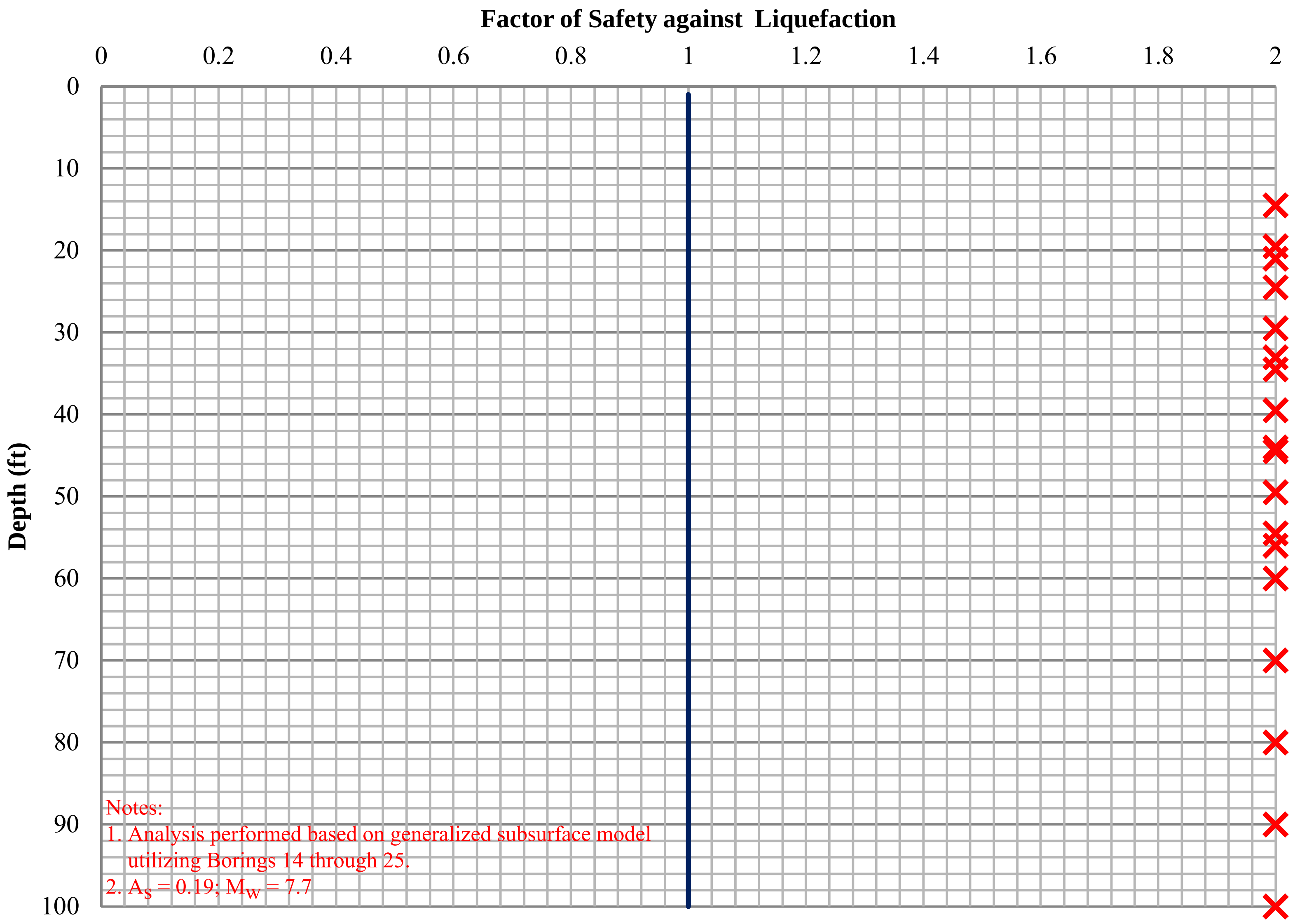
Notes:
1. Analysis performed based on generalized subsurface model utilizing Borings 12, 13, 13A and 14.
2. $A_s = 0.19$; $M_w = 7.7$

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

Results of Liquefaction Analyses - East Side of River

AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas

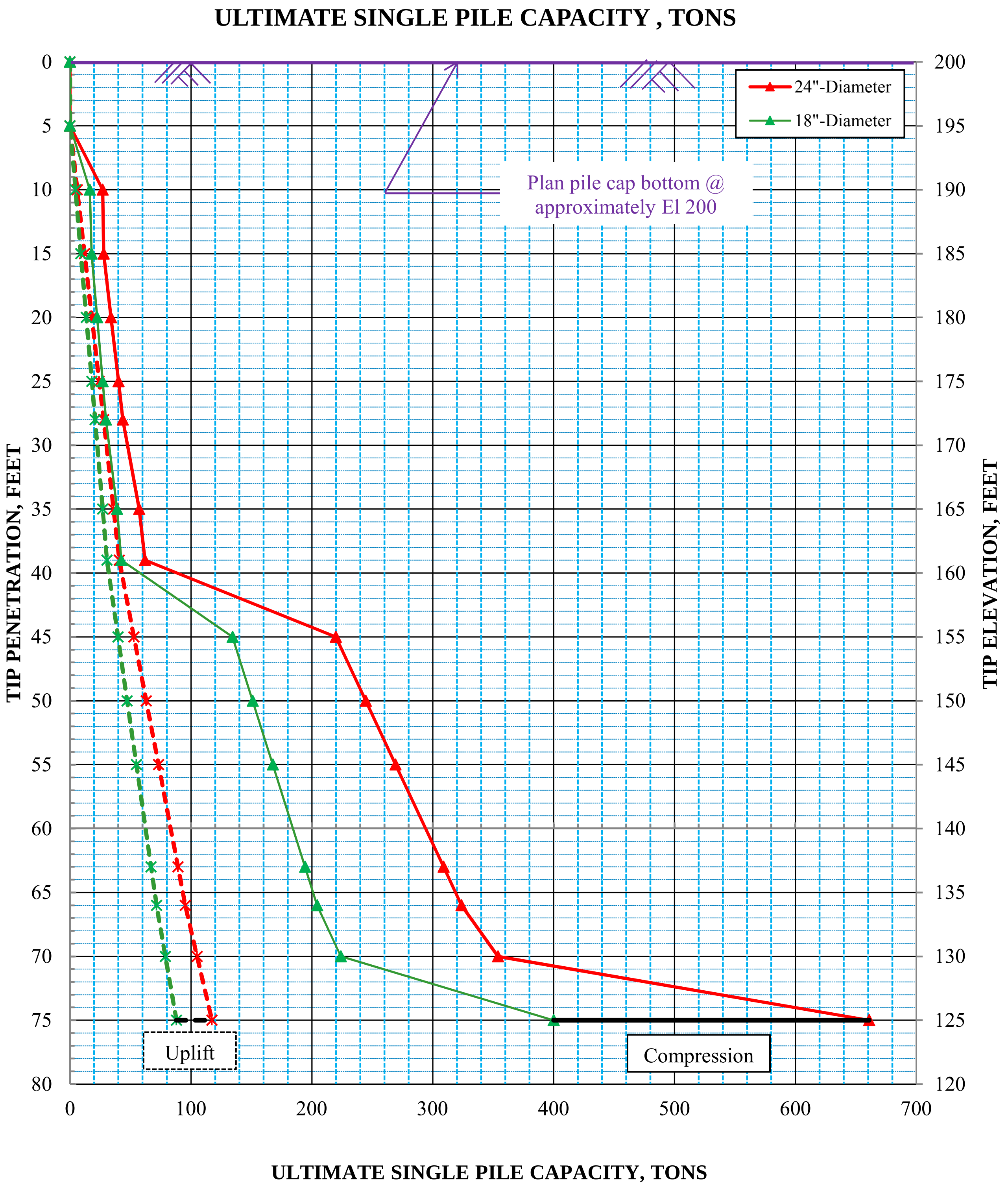


Notes:

1. Analysis performed based on generalized subsurface model utilizing Borings 14 through 25.
2. $A_s = 0.19$; $M_W = 7.7$

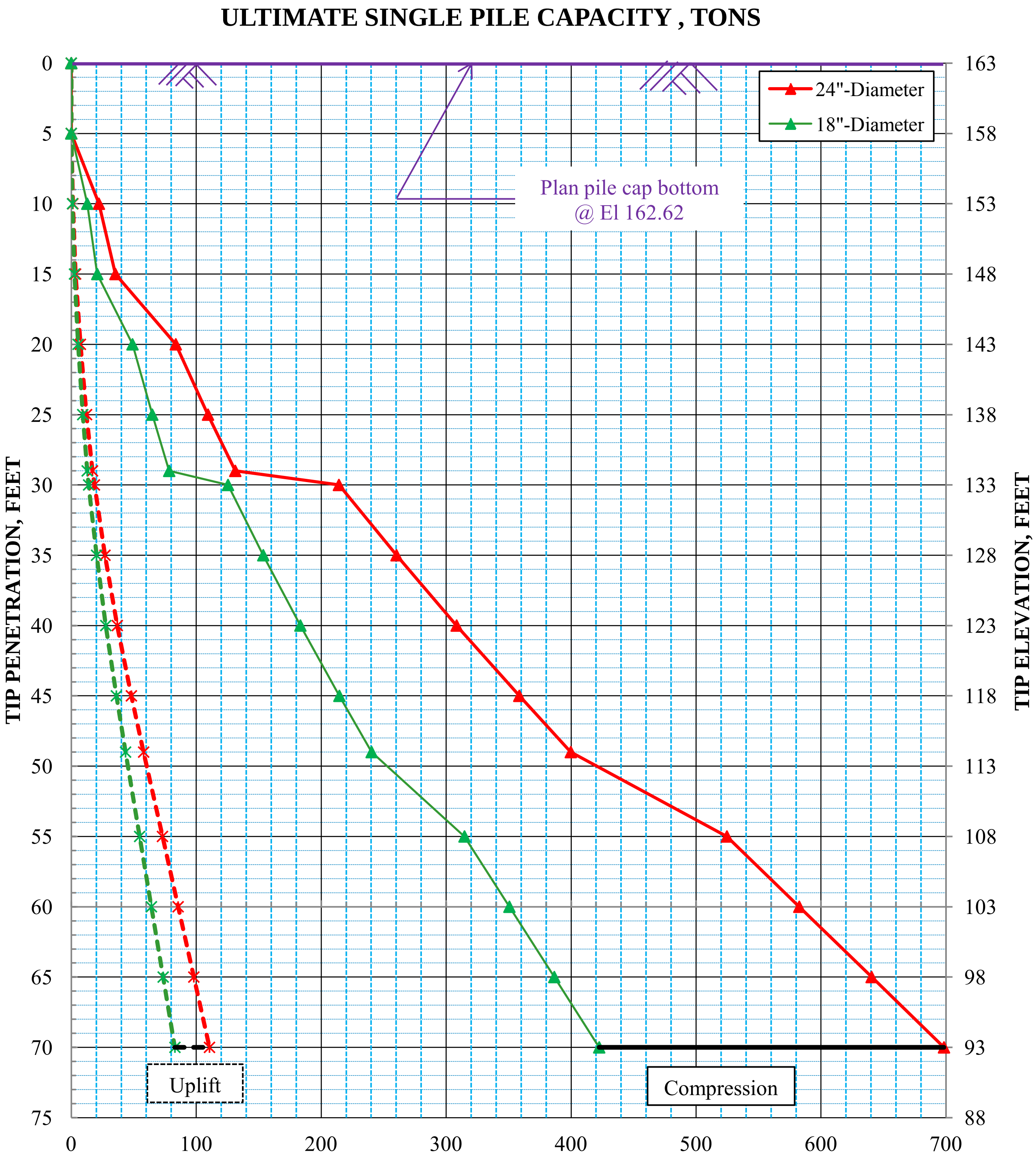
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers

APPENDIX F



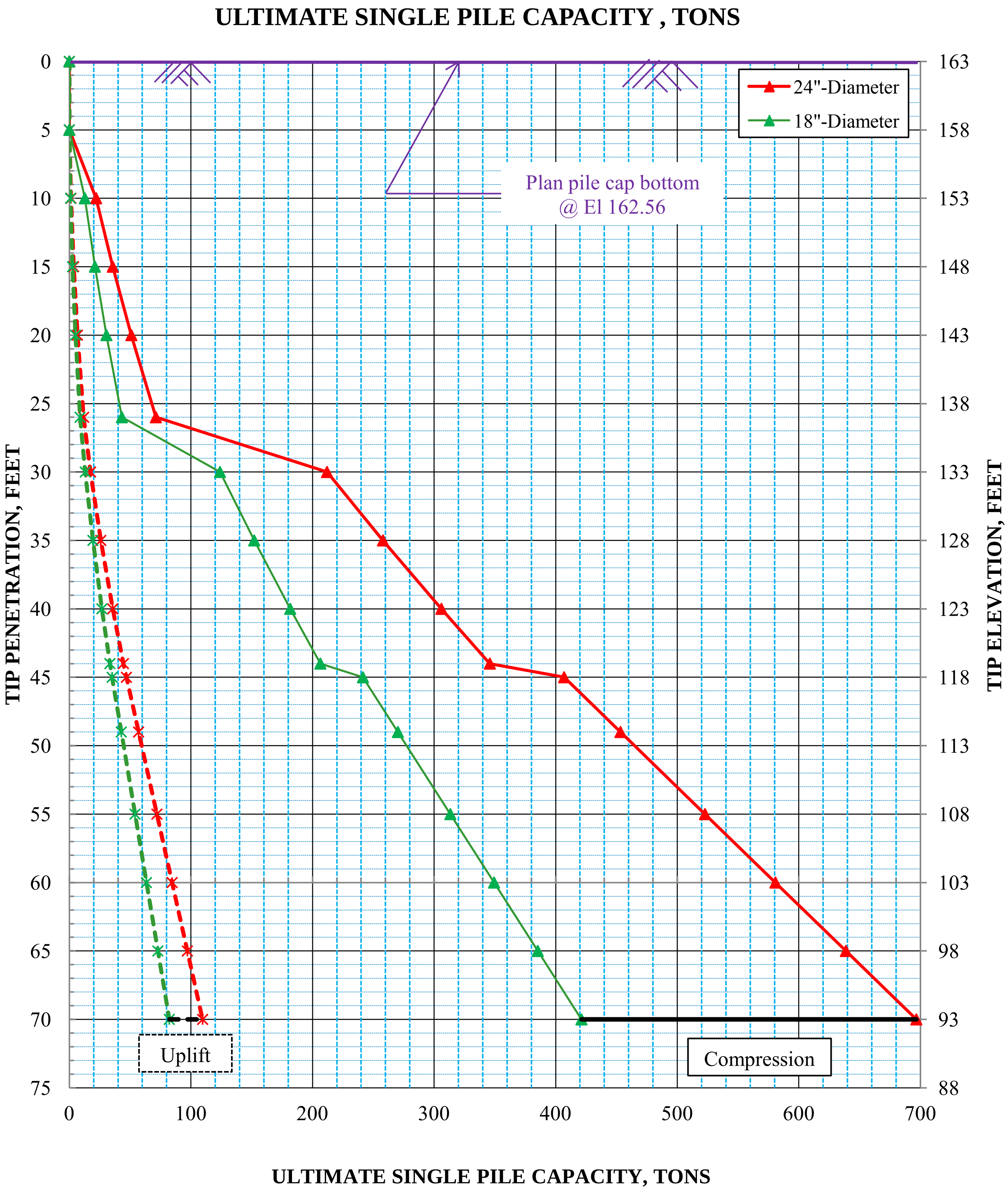
Driven Steel Shells
Bent 1 (West Abutment)
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be driven to plan tip elevation.



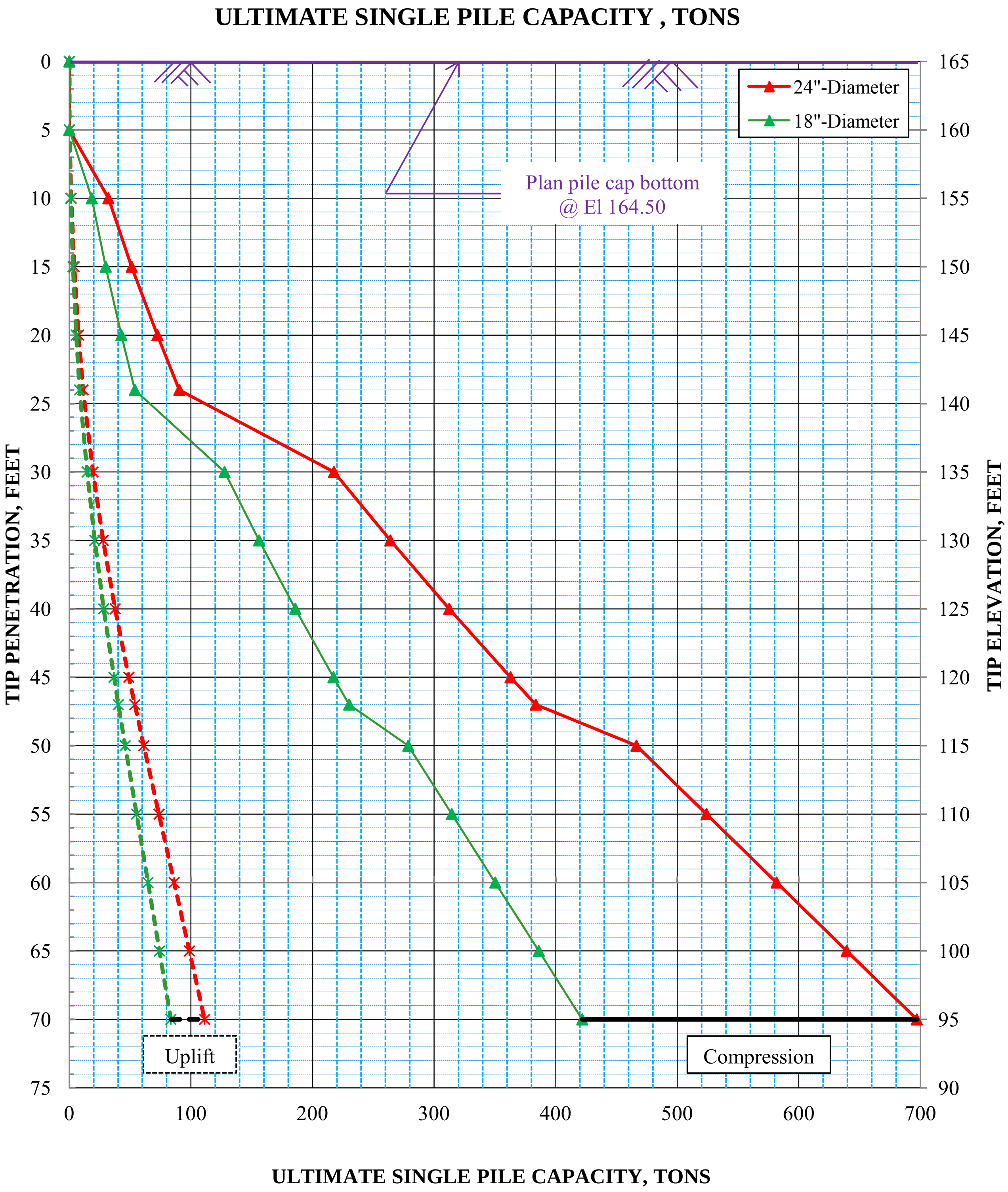
Driven Steel Shells
Bent 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



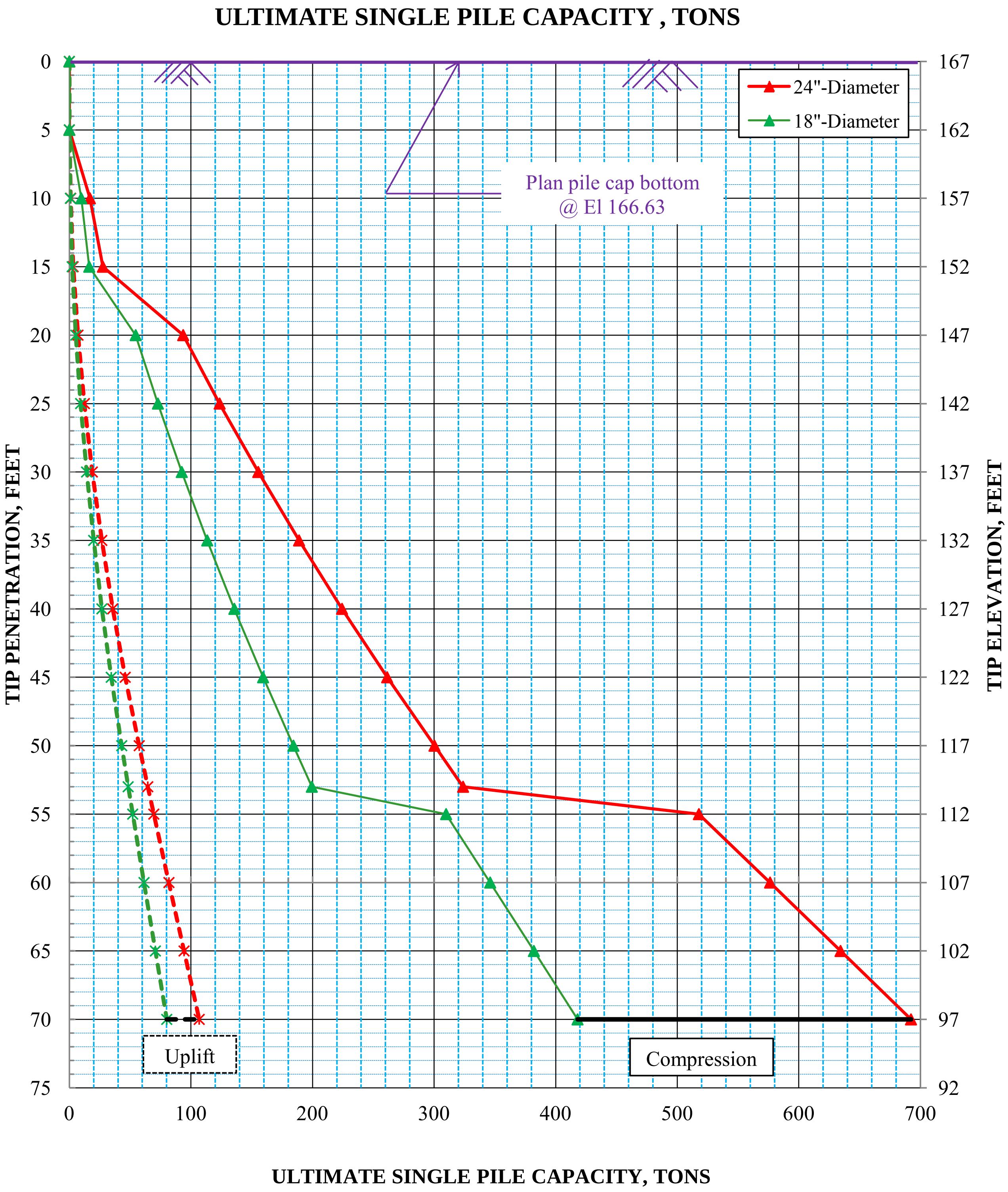
Driven Steel Shells
Bent 3
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



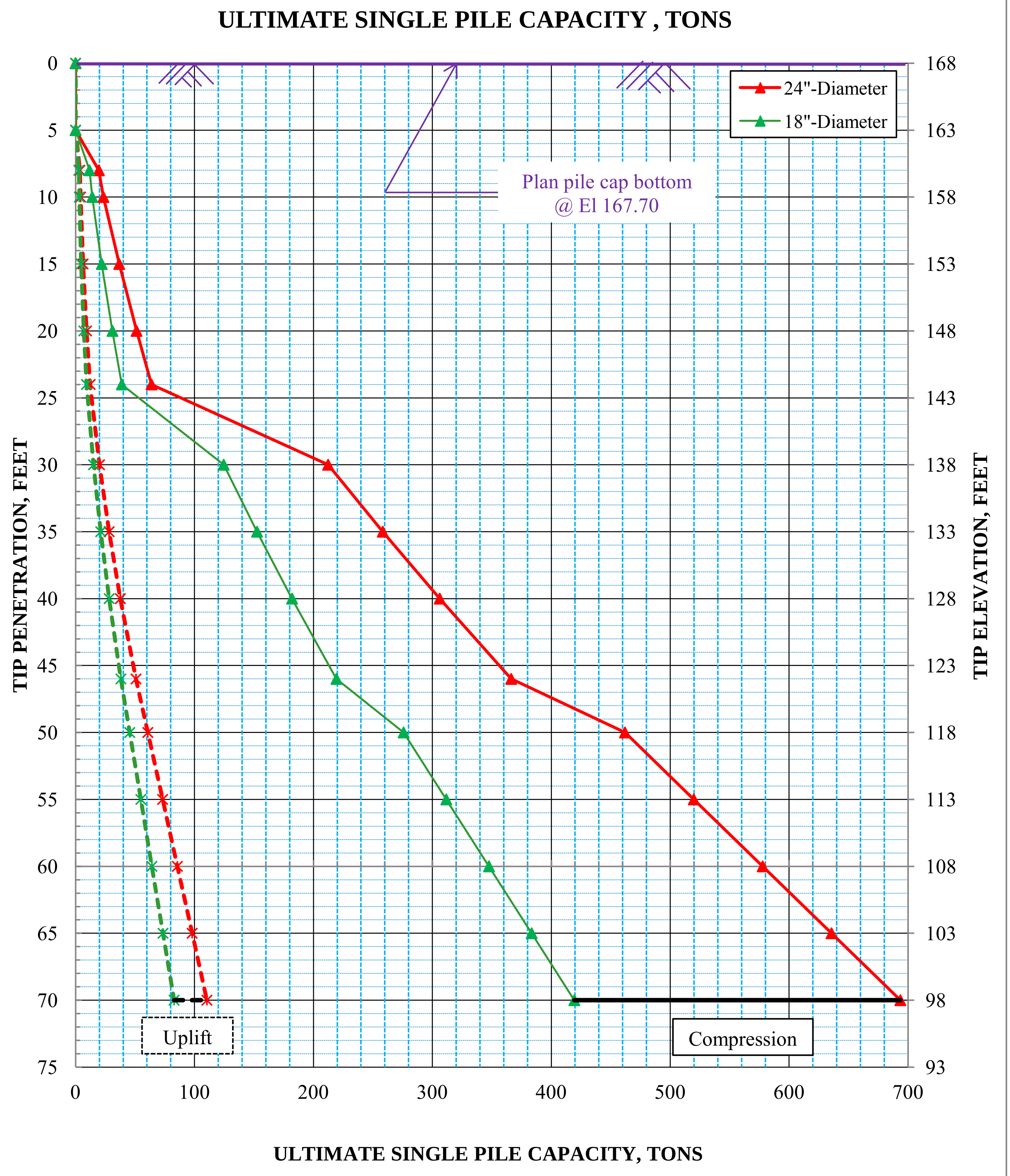
Driven Steel Shells
Bent 4
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



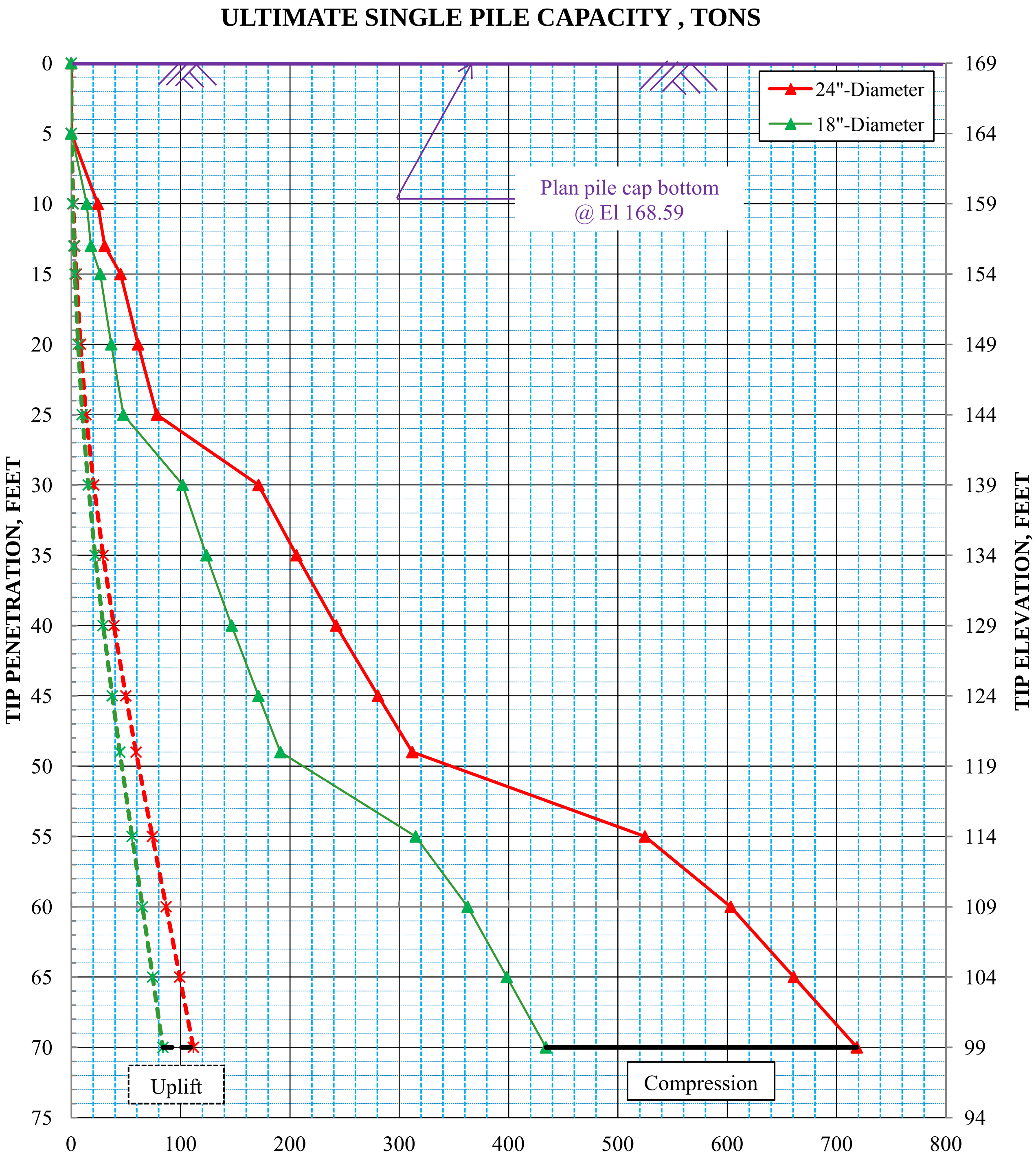
Driven Steel Shells
Bent 5
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 6
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

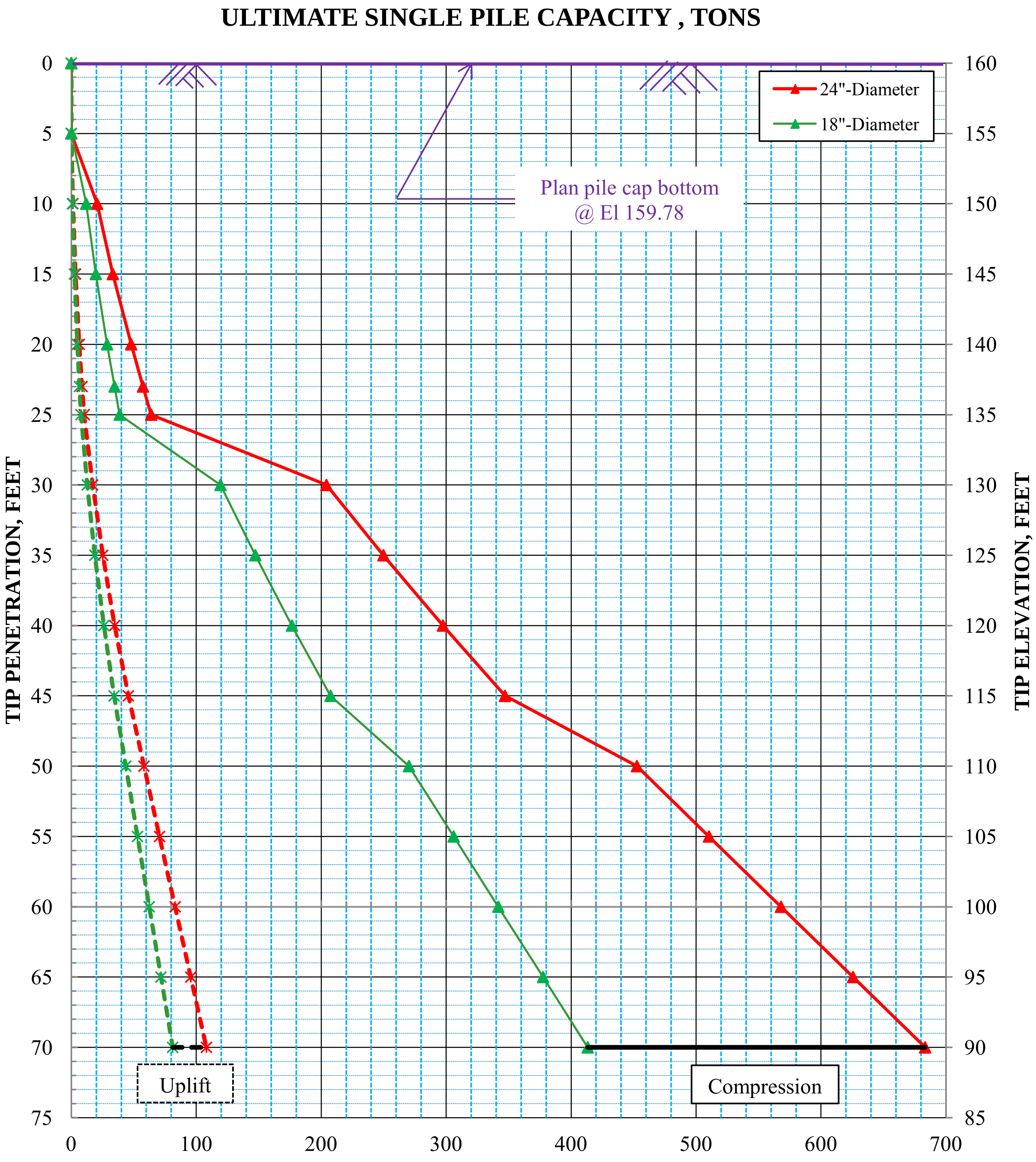
Note: Piles assumed to be jetted to 5 ft above tip elevation.



ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Bent 7
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

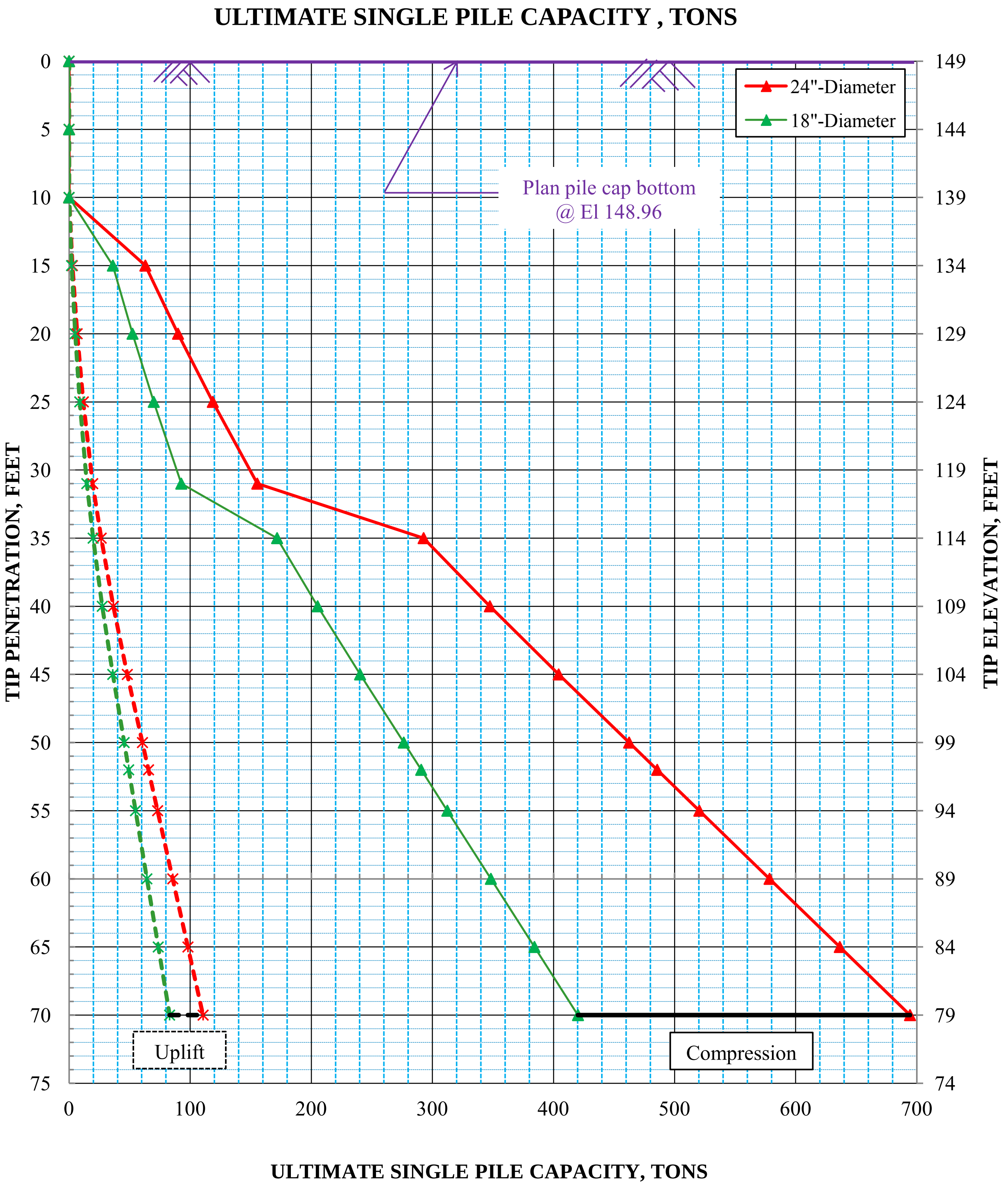
Note: Piles assumed to be jetted to 5 ft above tip elevation.



ULTIMATE SINGLE PILE CAPACITY, TONS

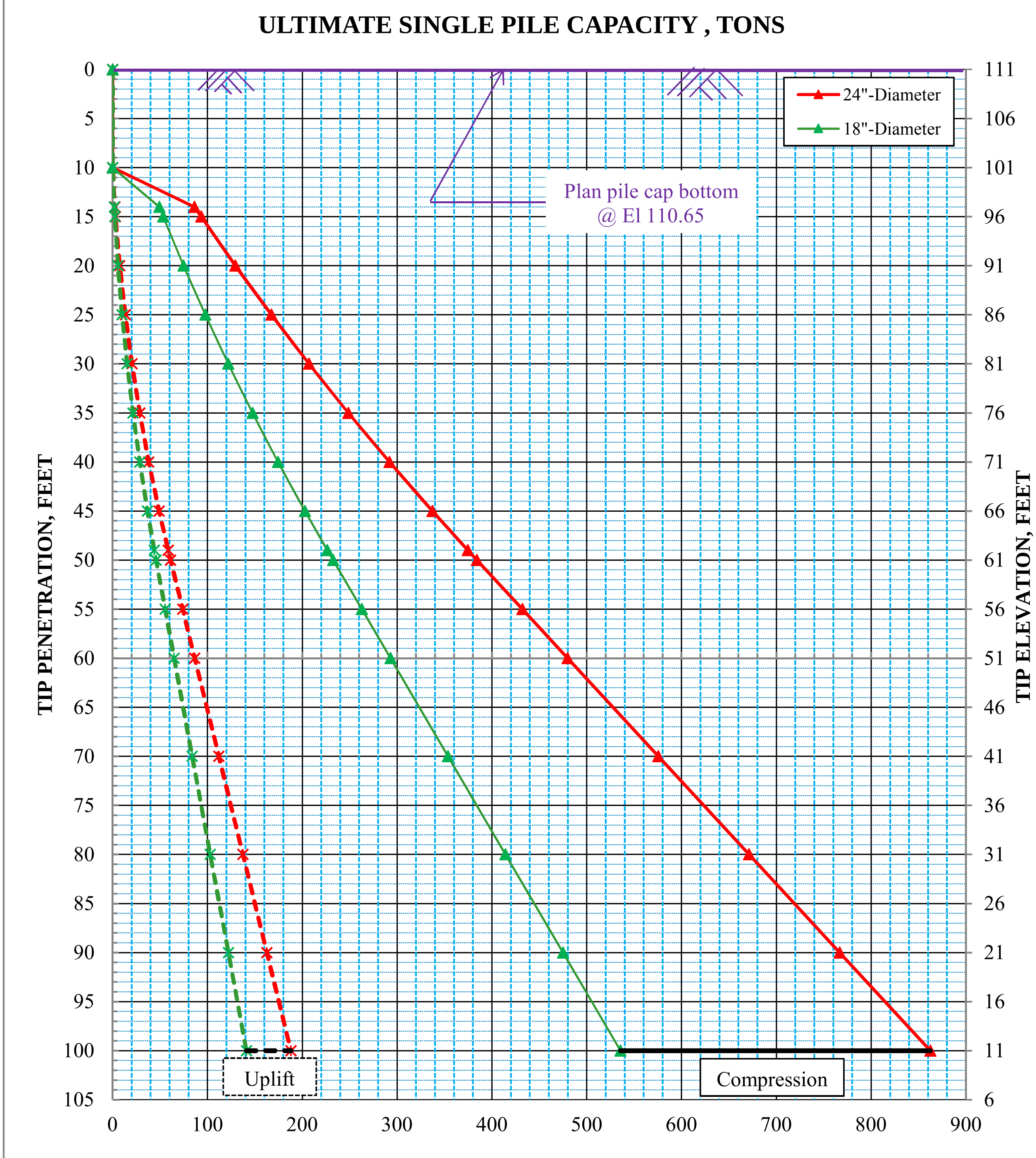
Driven Steel Shells
Bent 8
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 9
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

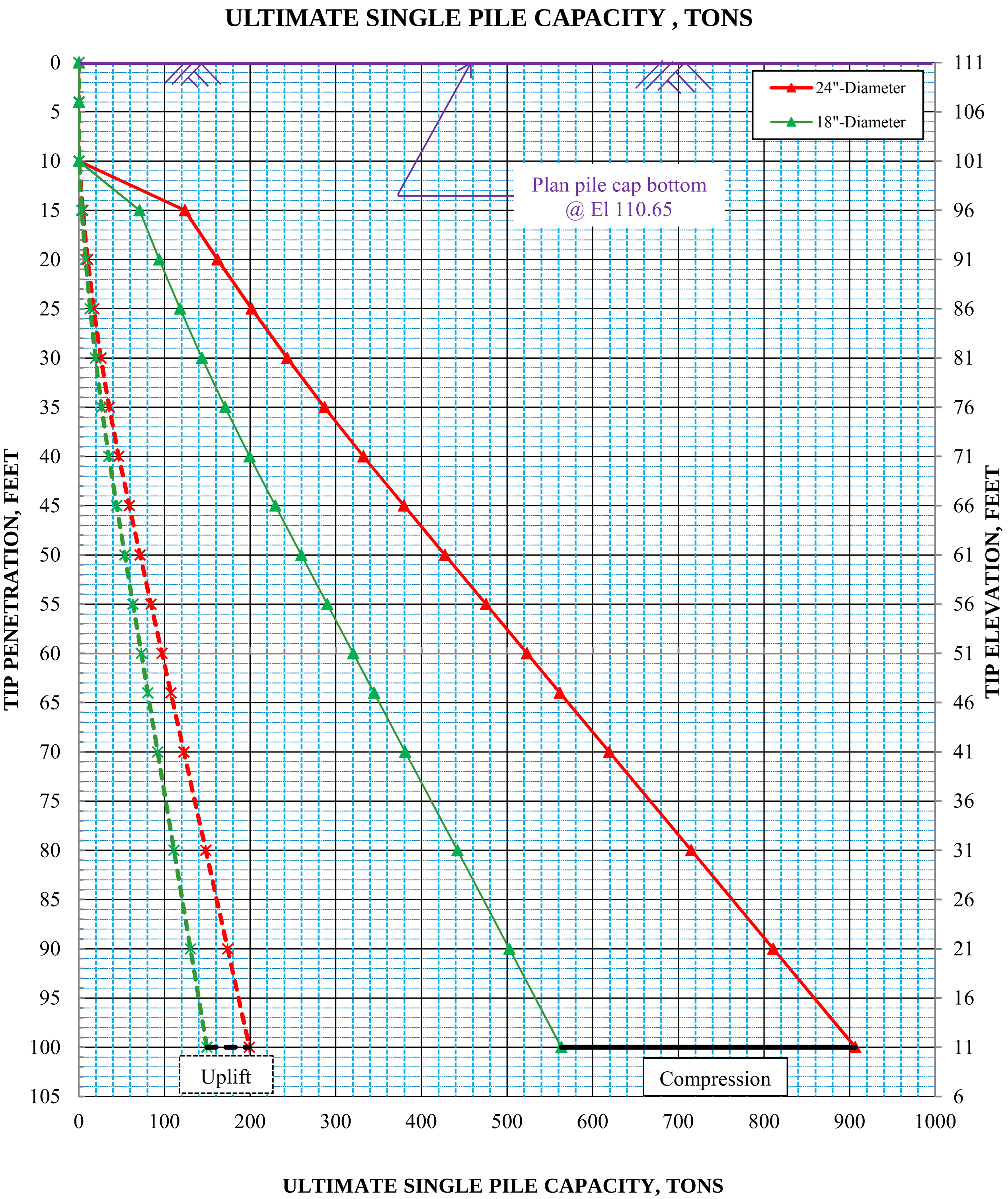


ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Pier No. 1
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

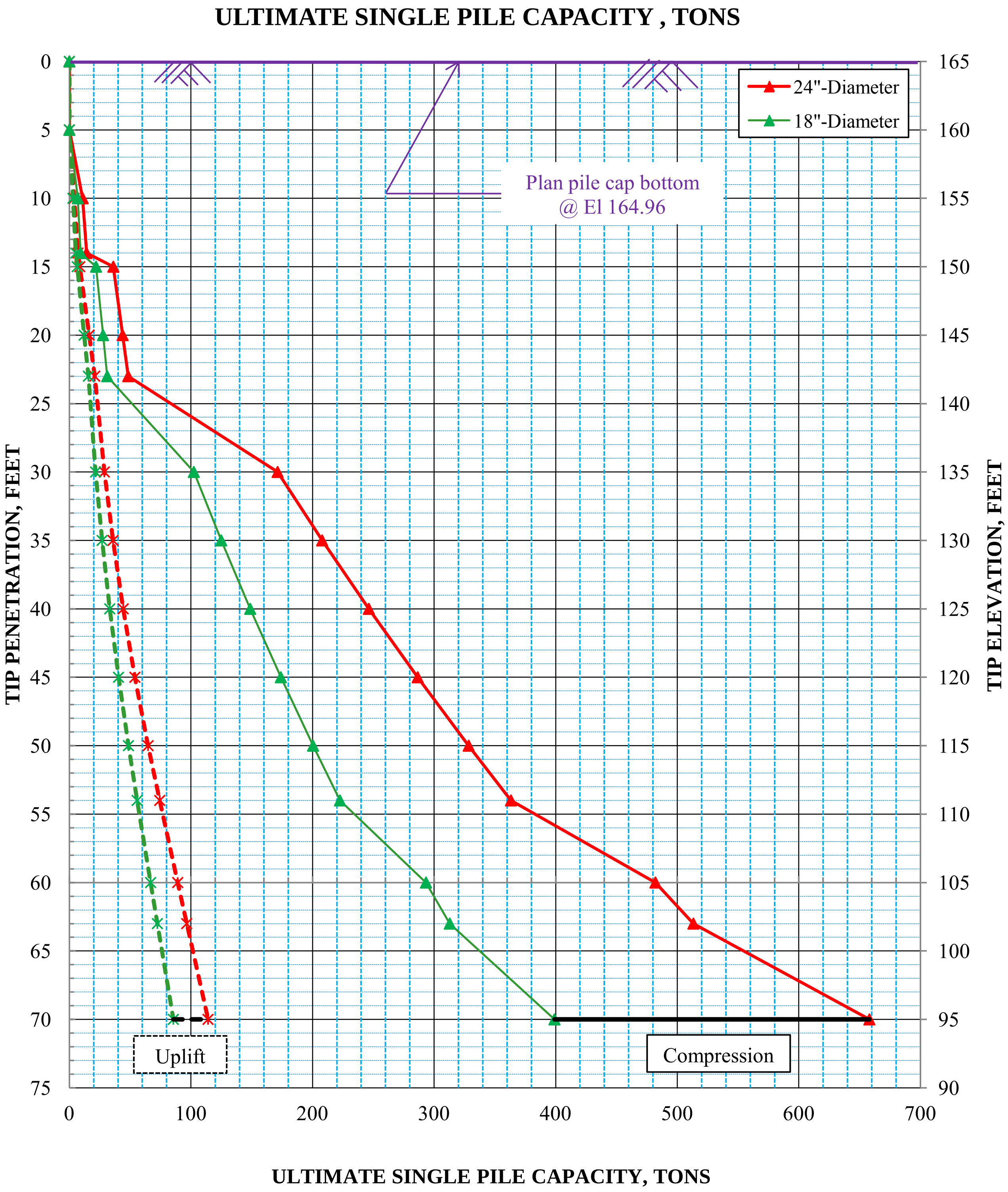
Note: Piles assumed to be jetted to 5 ft above tip elevation.

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



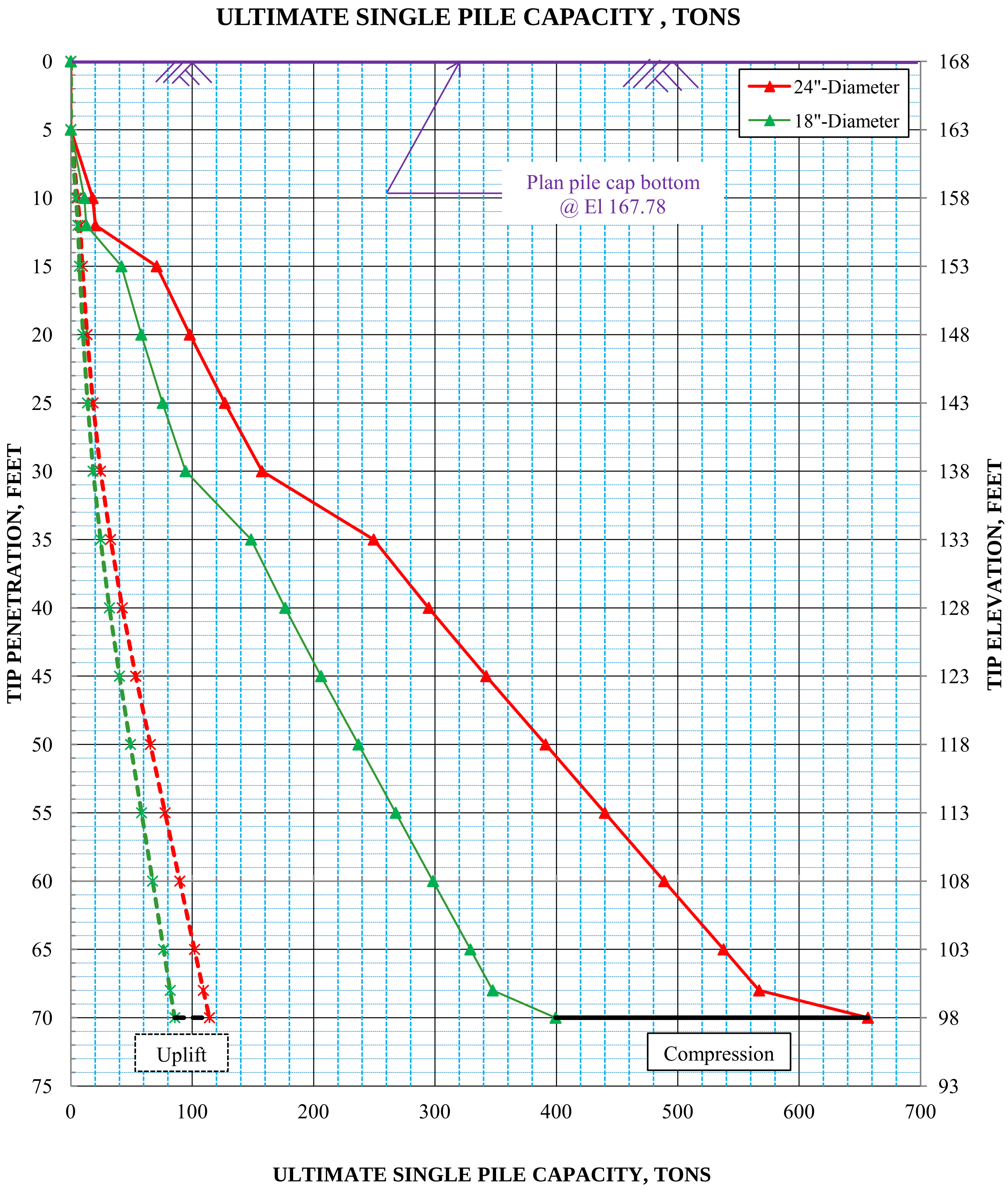
Driven Steel Shells
Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



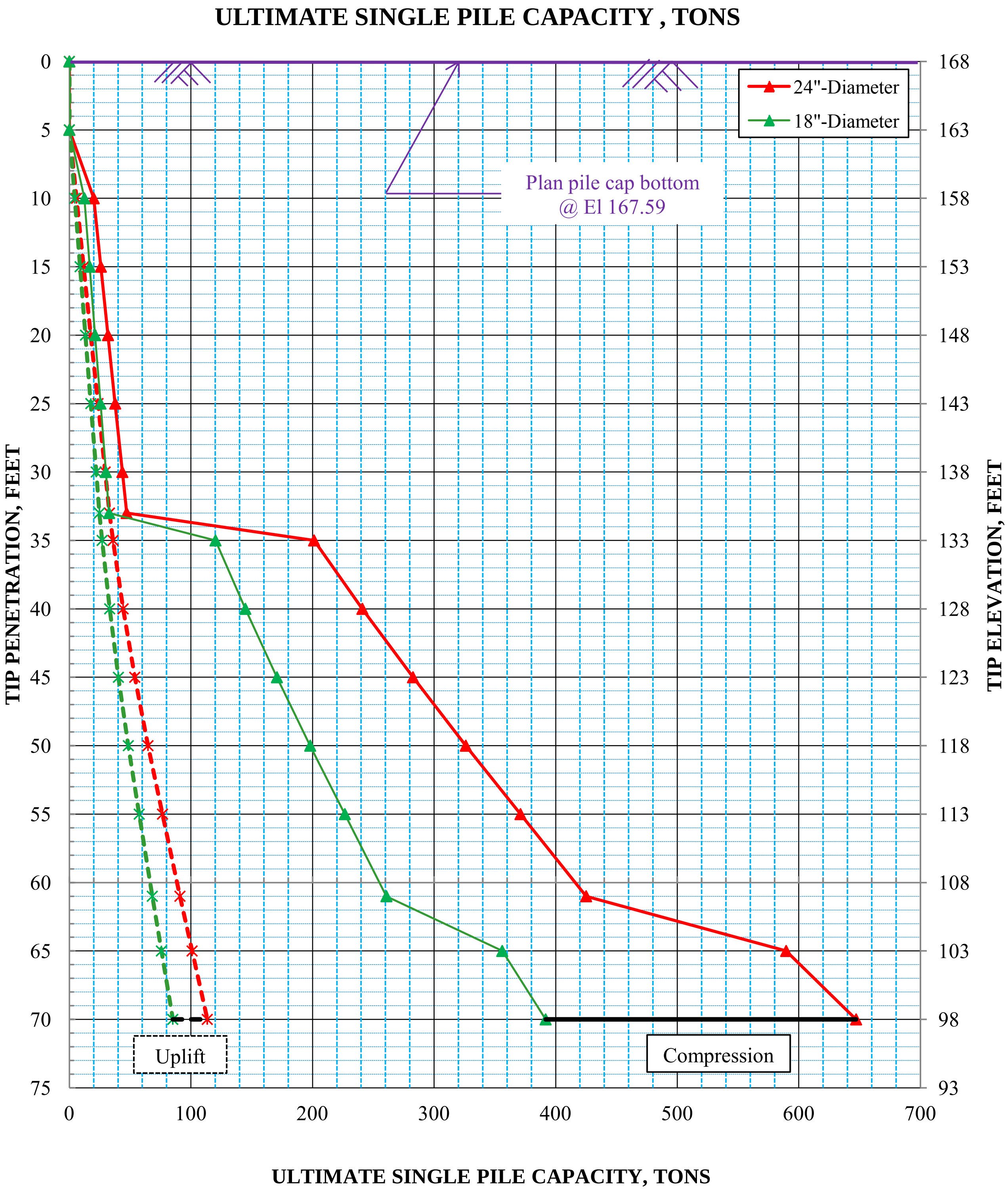
Driven Steel Shells
Bent 10
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



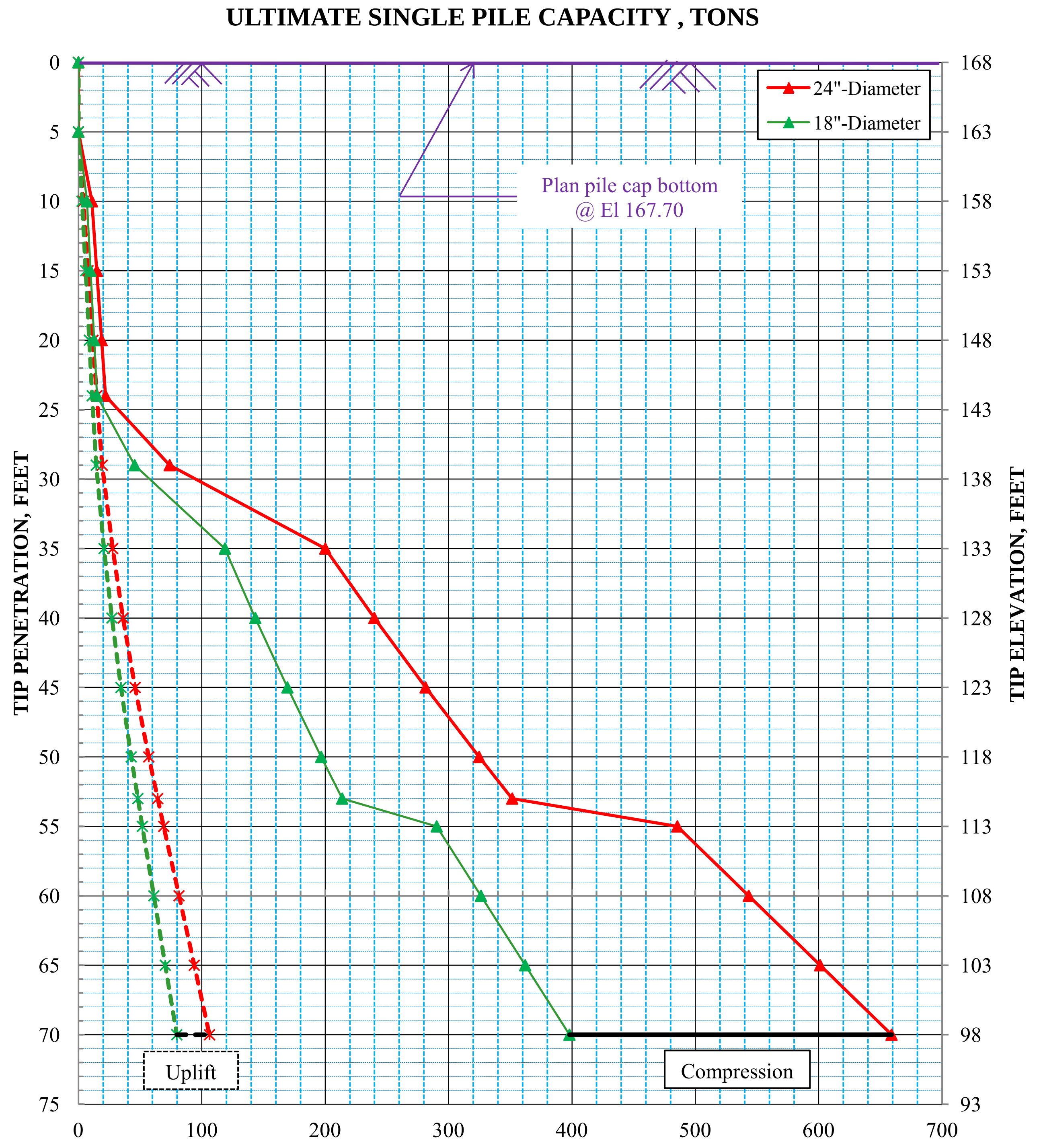
Driven Steel Shells
Bent 11
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 12
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

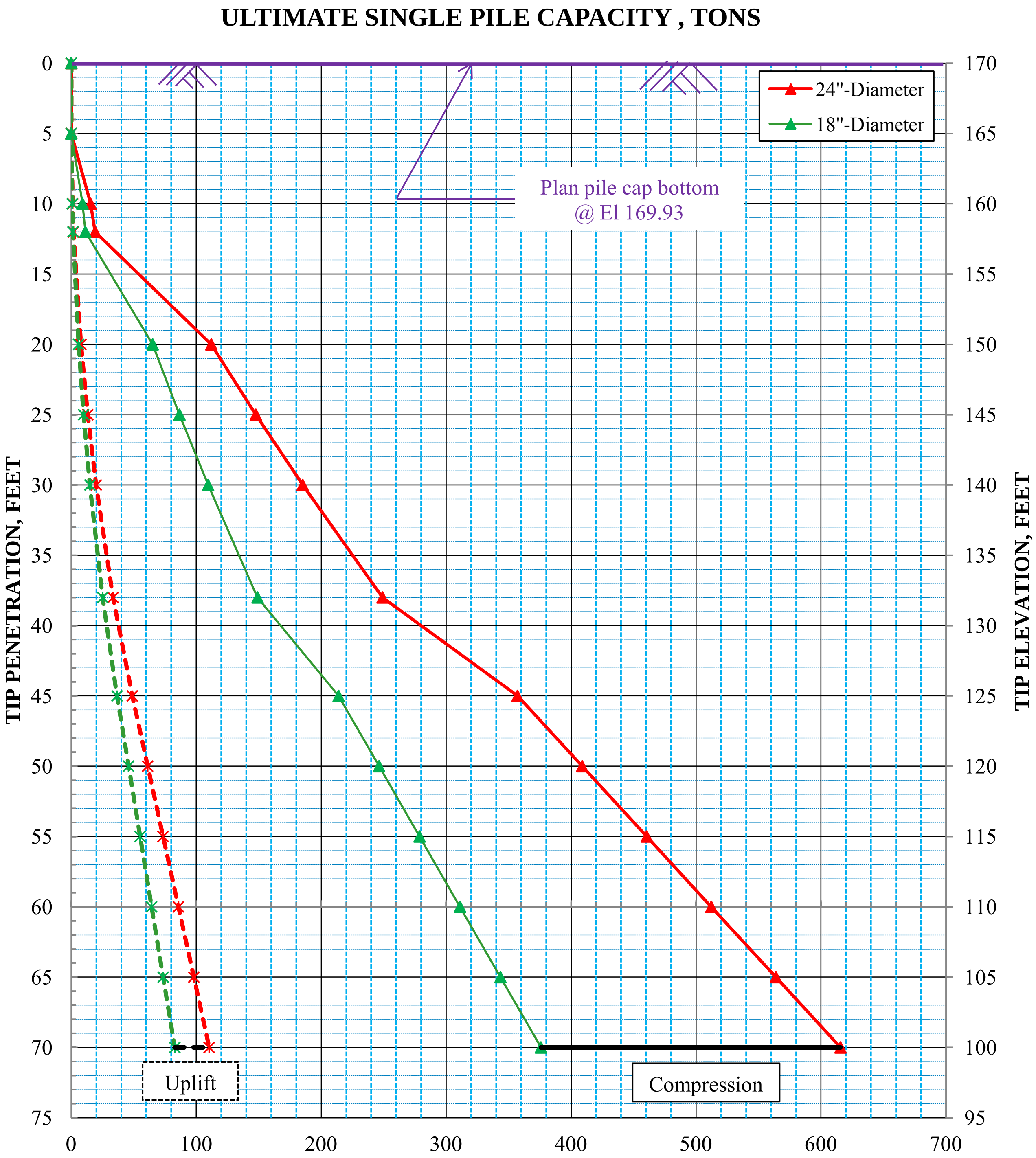
Note: Piles assumed to be jetted to 5 ft above tip elevation.



ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Bent 13
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

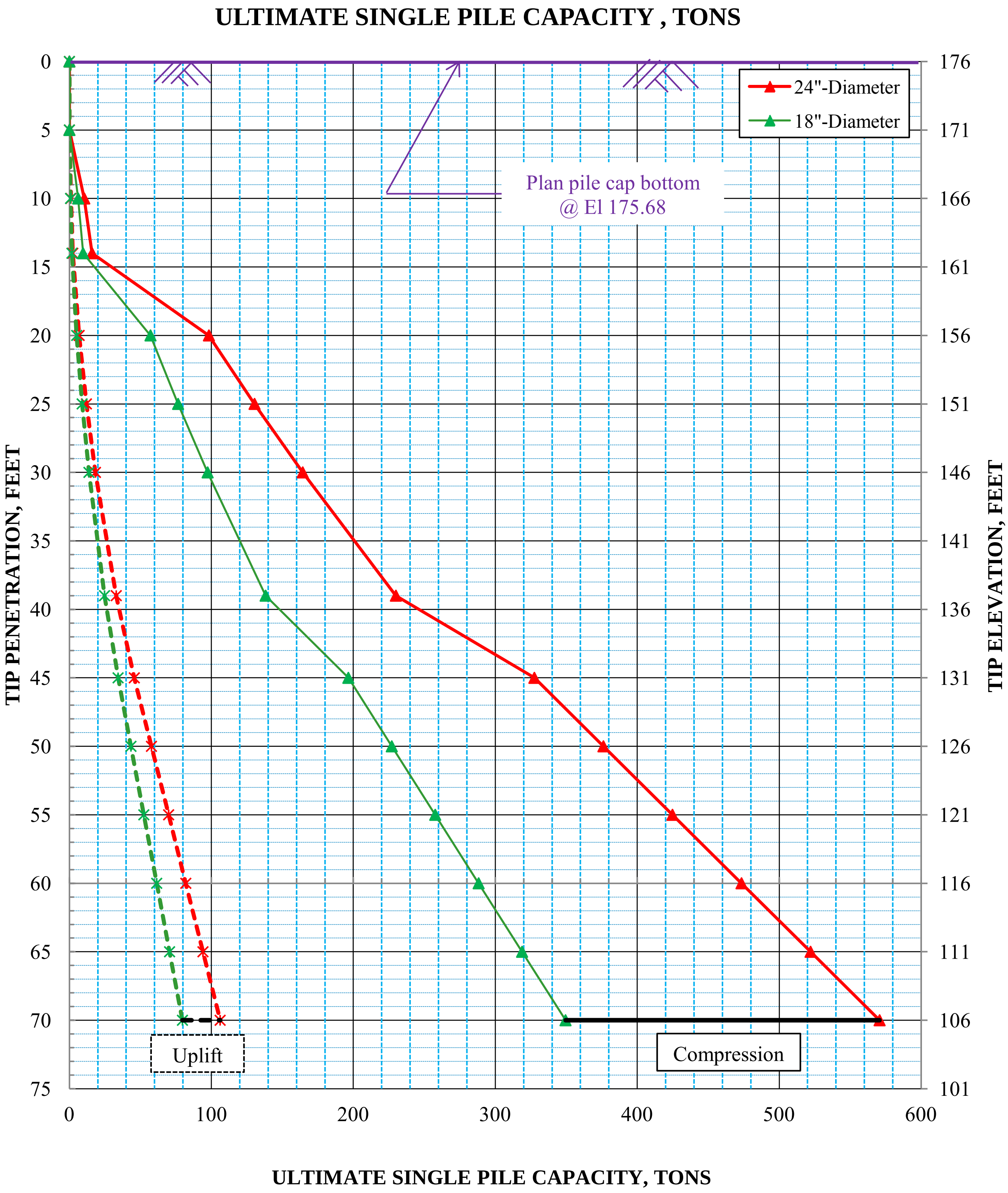
Note: Piles assumed to be jetted to 5 ft above tip elevation.



ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Bent 14
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

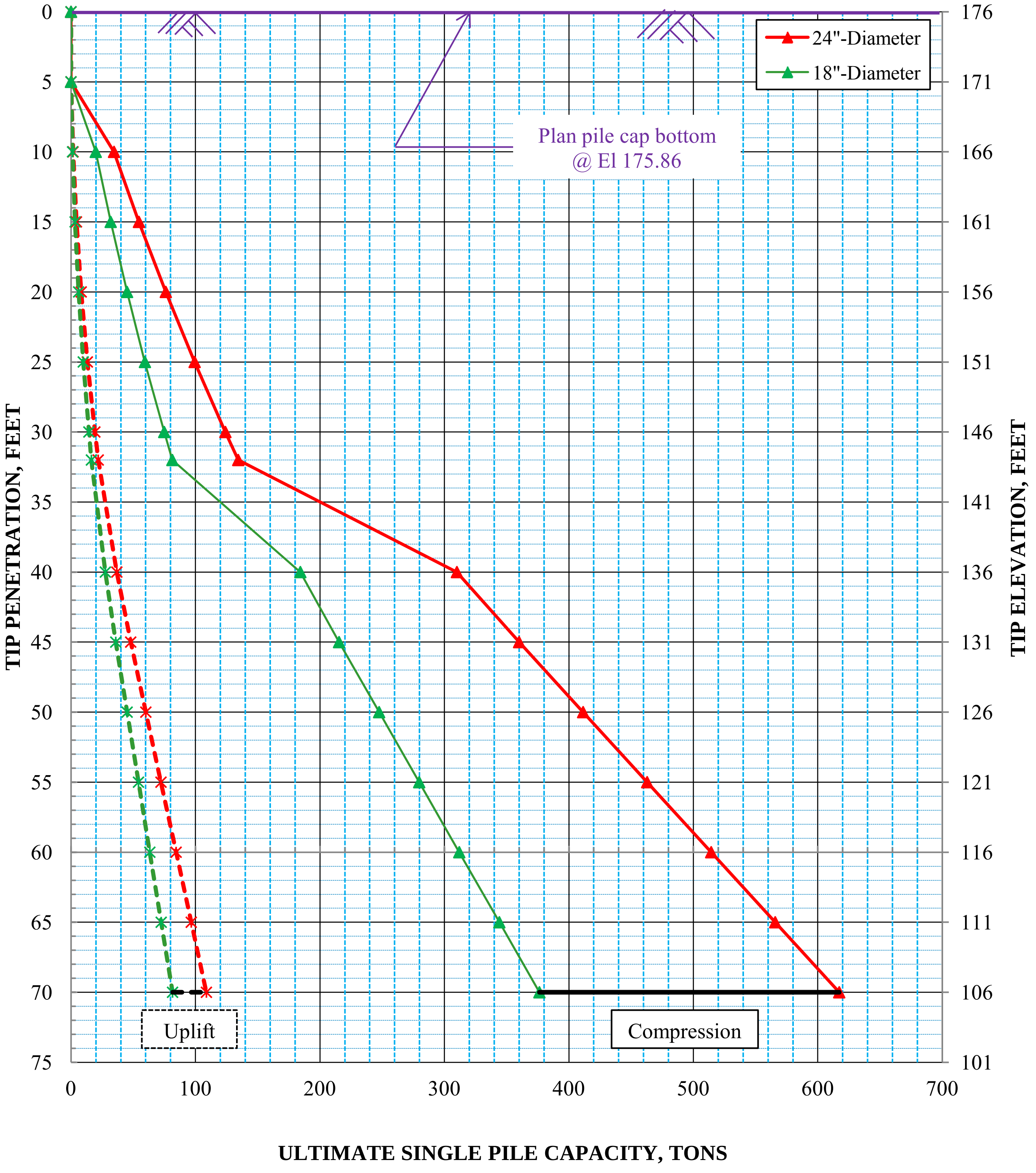
Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 15
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

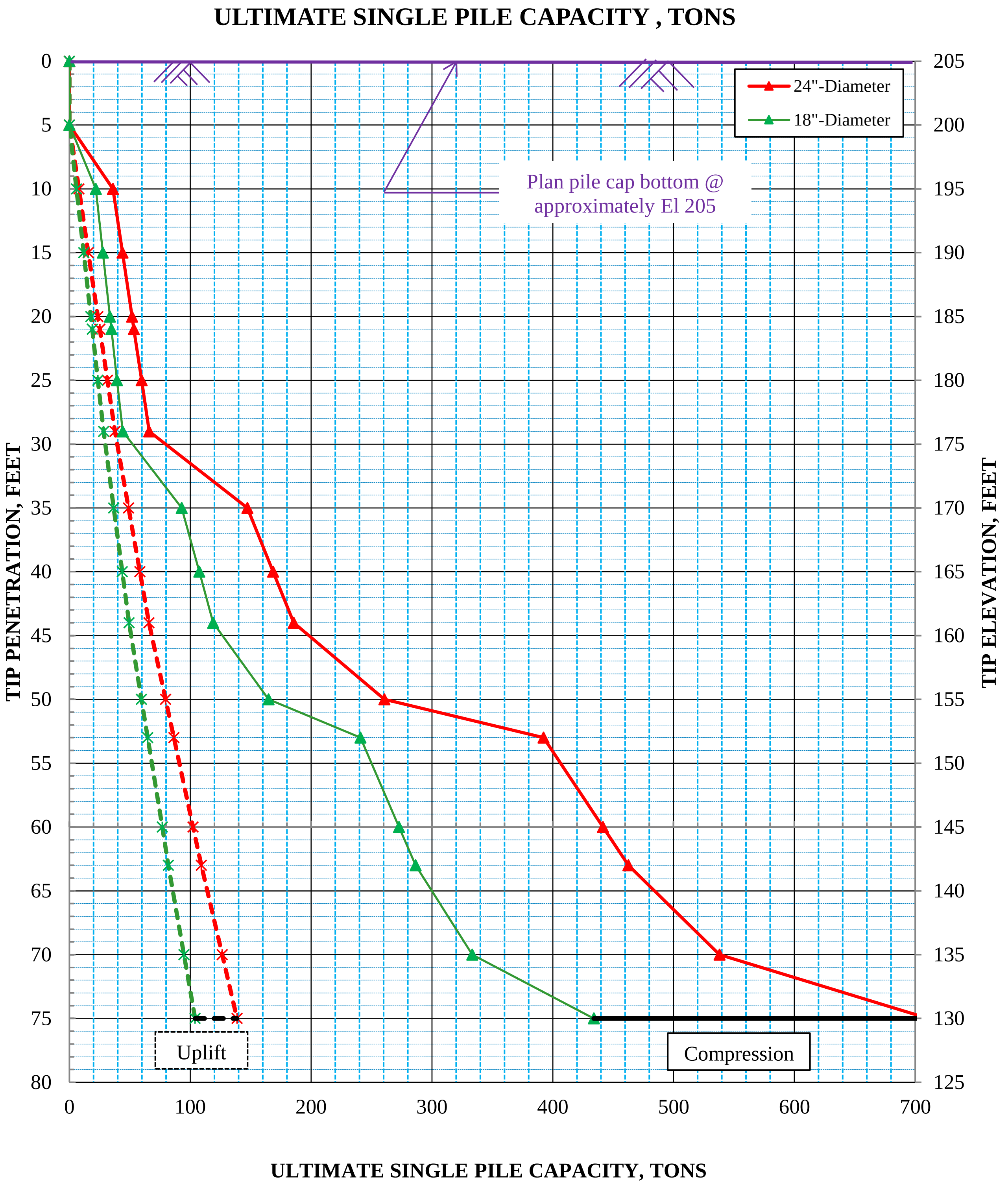
Note: Piles assumed to be jetted to 5 ft above tip elevation.

ULTIMATE SINGLE PILE CAPACITY , TONS



Driven Steel Shells
Bent 16
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

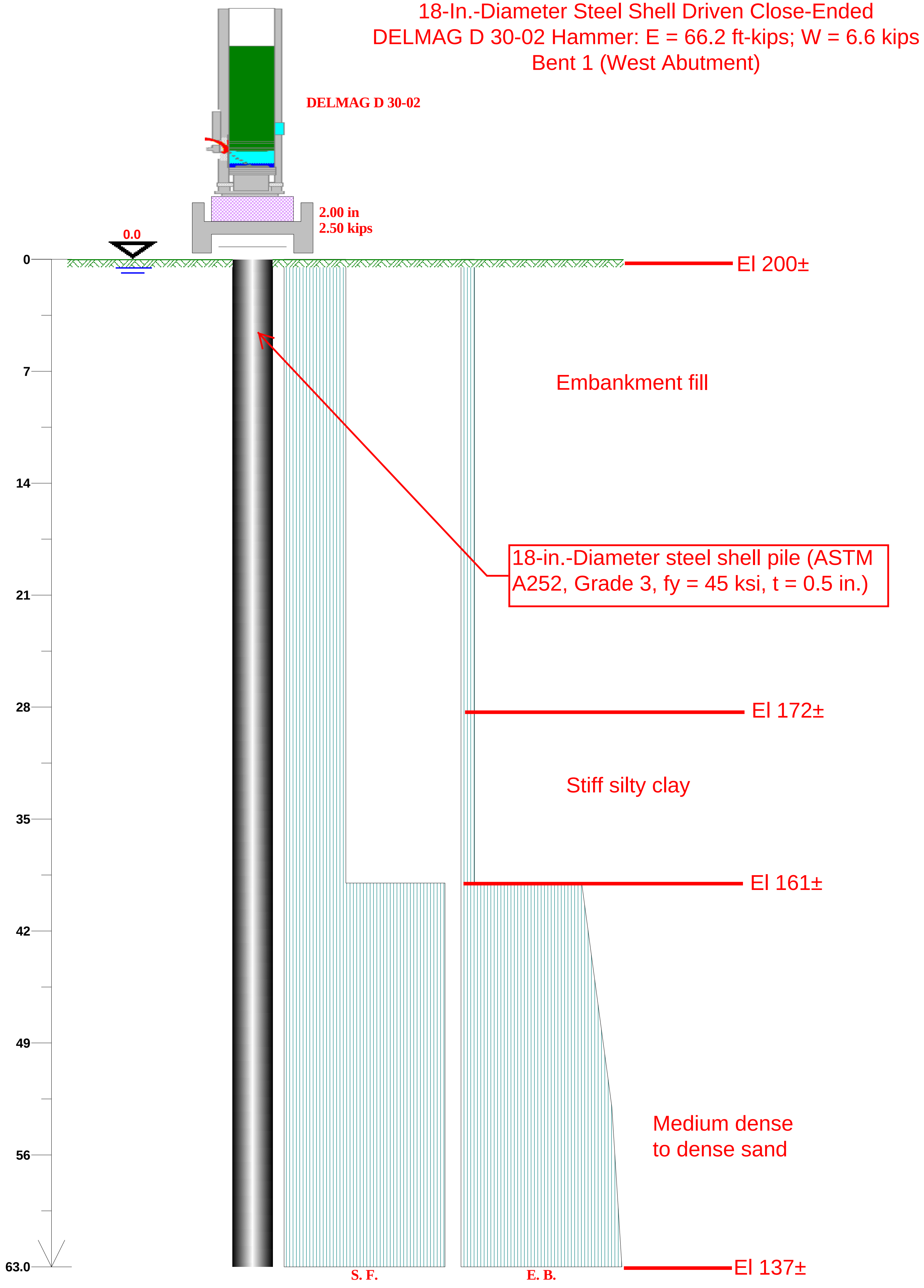


Driven Steel Shells
Bent 17 (East Abutment)
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

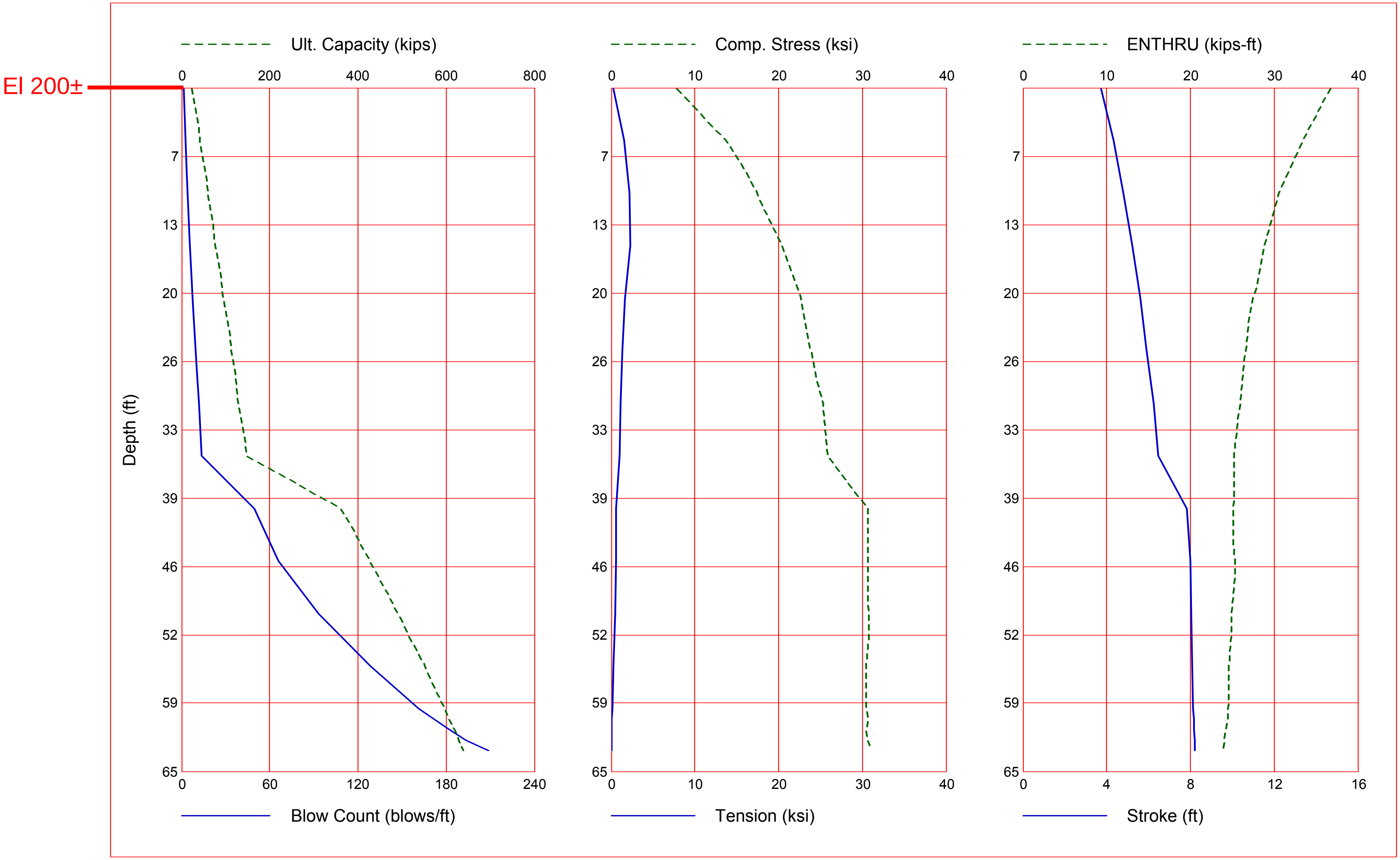
Note: Piles assumed to be driven to plan tip elevation.

APPENDIX G

Model for Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1 (West Abutment)



Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1

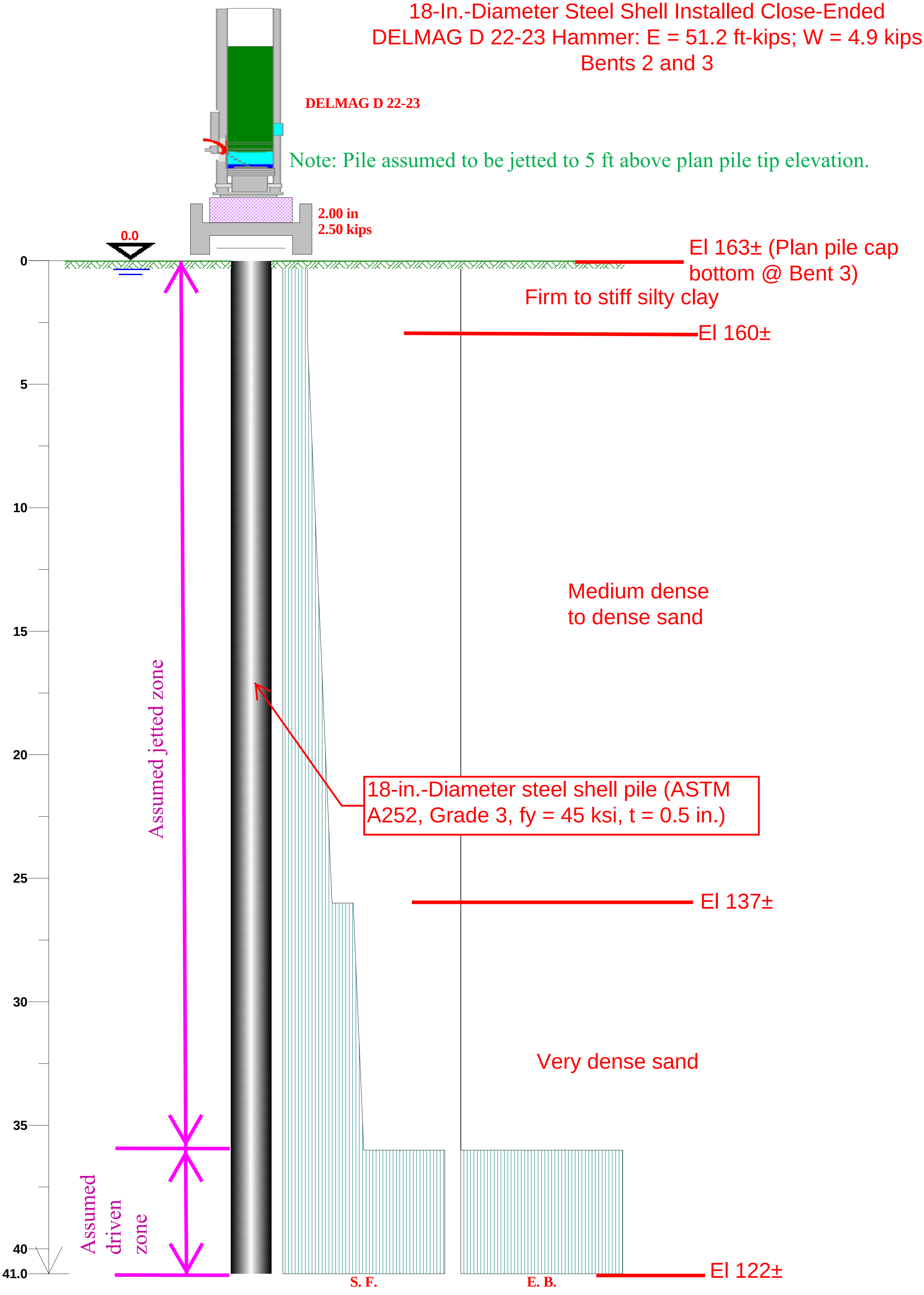
Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 29 2015
13-017_Bent 1_18 in. GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 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	24.2	0.4	23.8	1.3	7.834	-0.253	3.73	36.7
5.0	41.4	17.6	23.8	2.5	13.725	-1.502	4.31	33.4
10.0	59.1	35.2	23.8	4.0	17.456	-2.172	4.80	30.6
15.0	76.7	52.9	23.8	5.7	20.475	-2.247	5.23	28.8
20.0	94.3	70.5	23.8	7.5	22.696	-1.617	5.59	27.5
25.0	111.9	88.1	23.8	9.5	23.884	-1.295	5.91	26.6
30.0	129.6	105.7	23.8	11.8	25.323	-1.070	6.24	26.0
35.0	147.2	123.4	23.8	13.9	25.888	-1.039	6.46	25.2
40.0	362.1	146.6	215.4	49.7	30.623	-0.561	7.85	25.1
45.0	426.8	192.5	234.3	66.1	30.643	-0.569	8.02	25.3
50.0	491.5	238.3	253.2	93.0	30.826	-0.533	8.04	24.9
55.0	552.2	284.1	268.1	128.7	30.426	-0.289	8.10	24.6
59.0	595.9	320.8	275.2	161.5	30.495	-0.145	8.14	24.5
60.0	606.9	329.9	276.9	171.4	30.662	-0.085	8.18	24.5
61.0	617.8	339.1	278.7	182.1	30.430	0.000	8.19	24.3
62.0	628.7	348.3	280.5	193.4	30.656	0.000	8.22	24.1
63.0	639.7	357.4	282.2	208.7	31.065	0.000	8.22	23.8

Total Continuous Driving Time 70.00 minutes; Total Number of Blows 2954

Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1

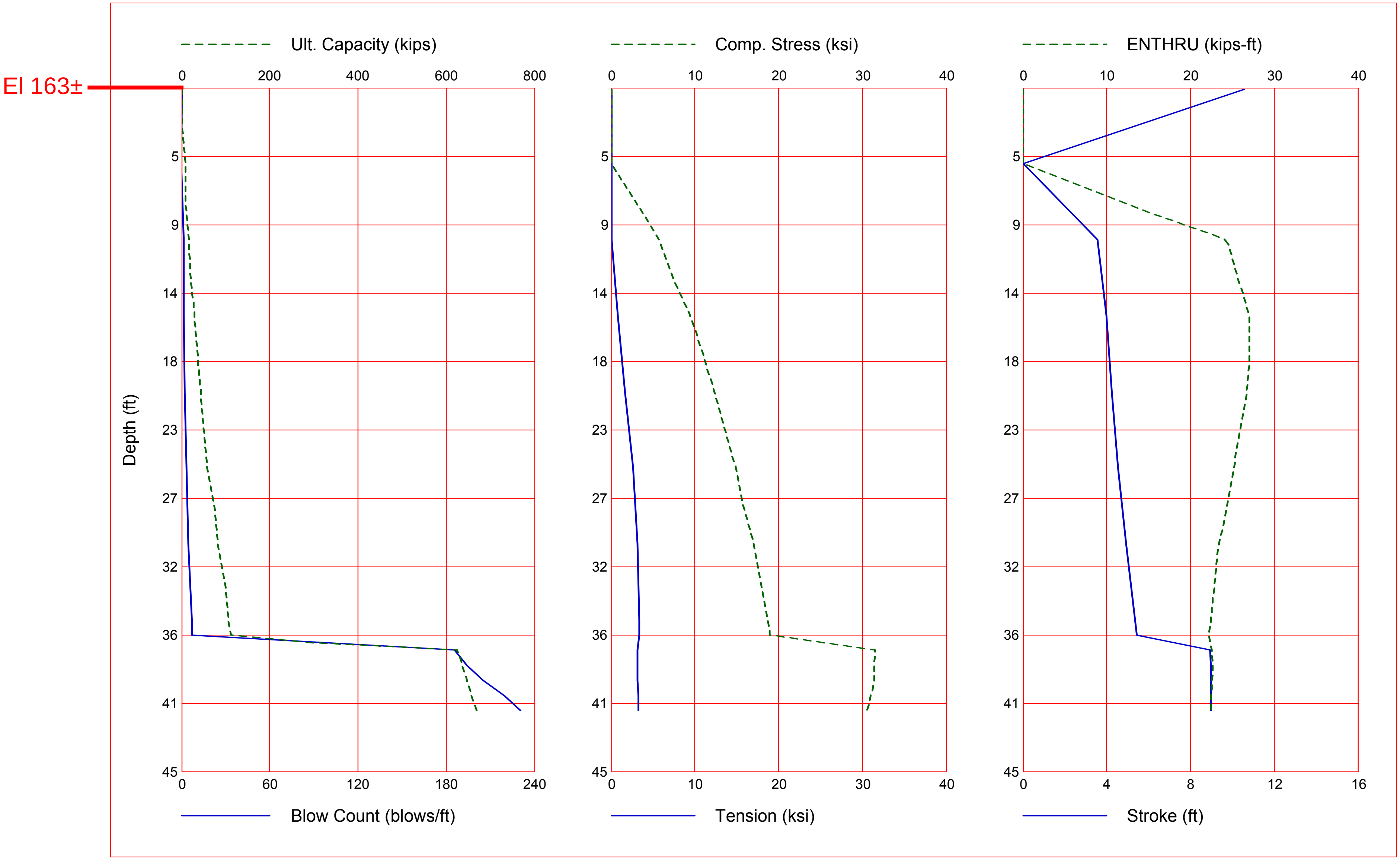


Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 3_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3

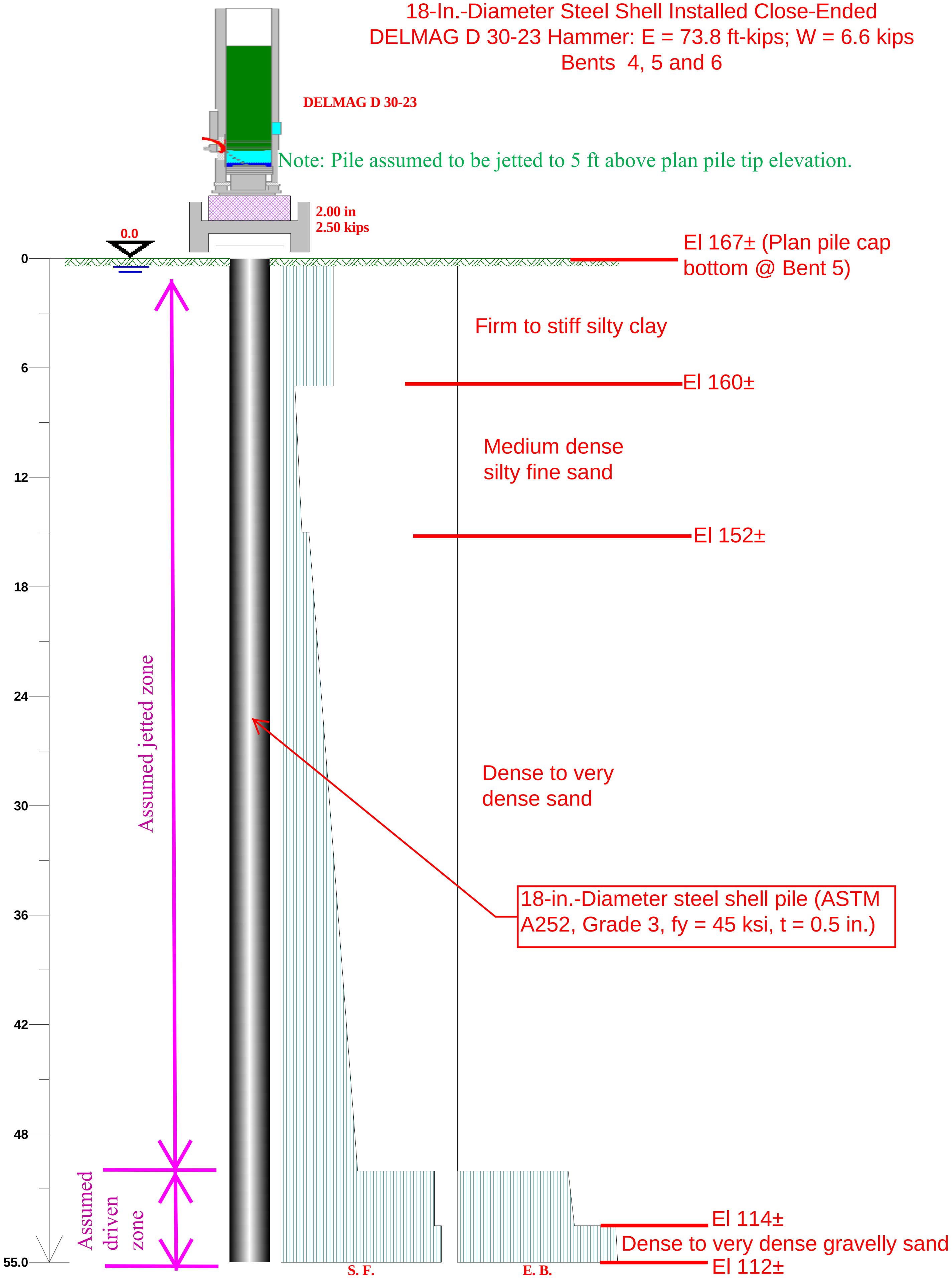
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	10.56	0.0
5.0	8.4	8.4	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	18.2	18.2	0.0	1.5	5.694	0.000	3.58	24.4
15.0	29.8	29.8	0.0	1.8	9.442	-0.757	3.97	27.0
20.0	43.2	43.2	0.0	2.3	12.366	-1.602	4.25	26.7
25.0	58.4	58.4	0.0	3.2	14.879	-2.656	4.56	25.2
30.0	81.1	81.1	0.0	4.8	17.009	-3.102	4.92	23.4
35.0	106.8	106.8	0.0	6.7	18.712	-3.349	5.35	22.4
36.0	112.2	112.2	0.0	7.1	18.951	-3.332	5.42	22.2
37.0	625.7	123.0	502.7	185.4	31.557	-3.135	8.95	22.6
38.0	636.5	133.8	502.7	193.9	31.456	-3.163	8.96	22.7
39.0	647.4	144.6	502.7	204.9	31.362	-3.186	8.97	22.6
40.0	658.2	155.5	502.7	219.6	31.019	-3.208	8.96	22.4
41.0	669.0	166.3	502.7	230.5	30.594	-3.220	8.96	22.4

Total Continuous Driving Time 25.00 minutes; Total Number of Blows 1014

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3

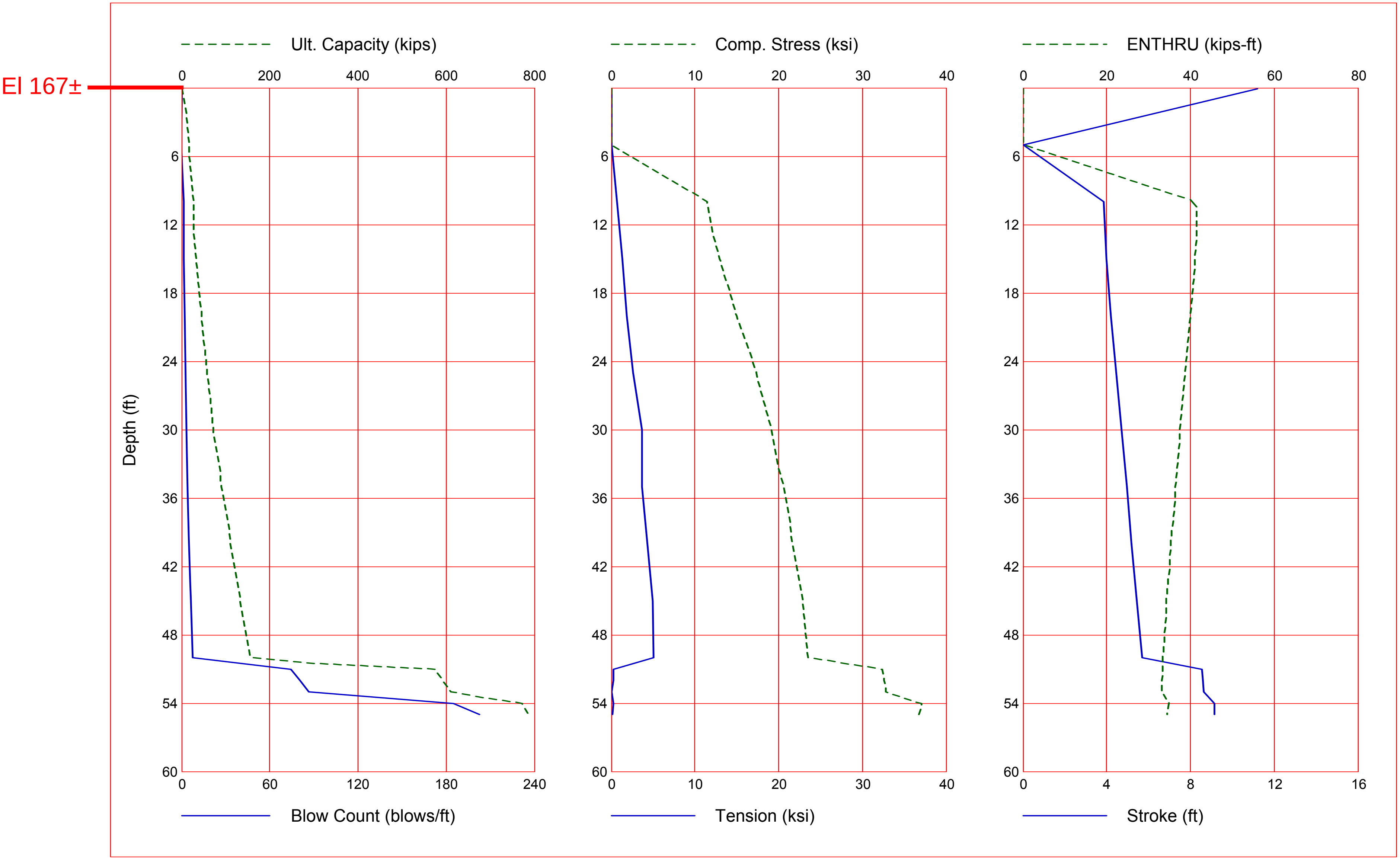
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 5_18 in.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 29 2015
13-017_Bent 5_18 in. GRLWEAP Version 2010

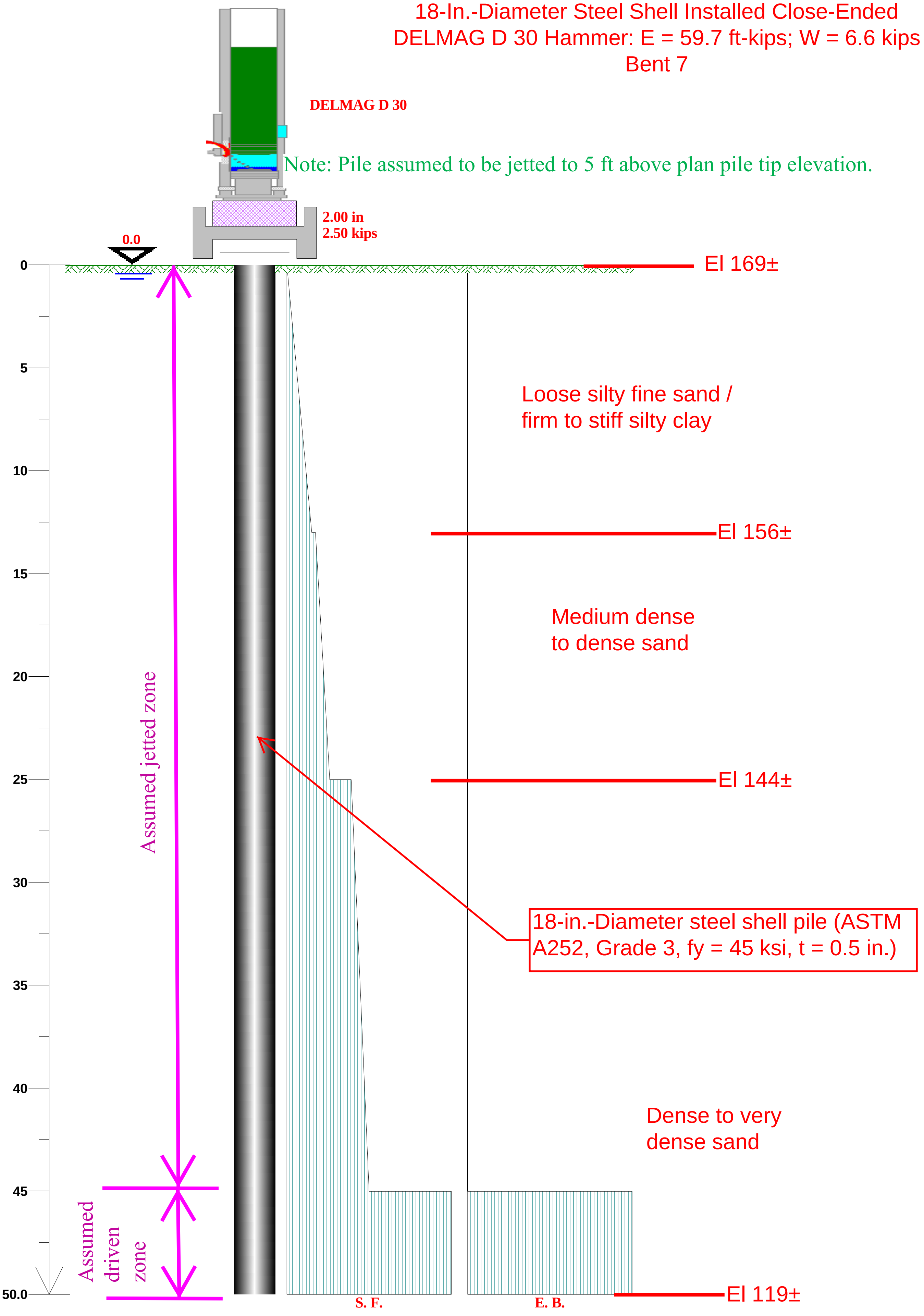
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.4	0.4	0.0	0.0	0.000	0.000	11.18	0.0
5.0	17.6	17.6	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	27.8	27.8	0.0	1.6	11.504	-0.665	3.88	41.5
15.0	34.1	34.1	0.0	1.8	12.936	-1.376	3.99	41.1
20.0	44.6	44.6	0.0	2.2	14.998	-1.879	4.20	40.0
25.0	57.6	57.6	0.0	2.7	17.336	-2.593	4.46	38.8
30.0	72.8	72.8	0.0	3.4	19.134	-3.645	4.70	37.5
35.0	90.5	90.5	0.0	4.1	20.688	-3.670	4.95	36.4
40.0	110.4	110.4	0.0	5.1	21.647	-4.373	5.20	35.3
45.0	132.8	132.8	0.0	6.1	22.886	-4.907	5.45	34.3
50.0	157.4	157.4	0.0	7.5	23.546	-5.080	5.71	33.4
51.0	574.1	167.8	406.3	74.2	32.323	-0.286	8.54	33.3
52.0	592.1	178.1	413.9	80.3	32.561	-0.243	8.59	33.2
53.0	610.0	188.5	421.6	86.3	32.849	-0.069	8.63	33.1
54.0	772.5	199.3	573.3	184.9	37.076	-0.313	9.14	34.9
55.0	786.9	210.1	576.8	202.6	36.761	-0.172	9.14	34.4

Total Continuous Driving Time 16.00 minutes; Total Number of Blows 685

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7



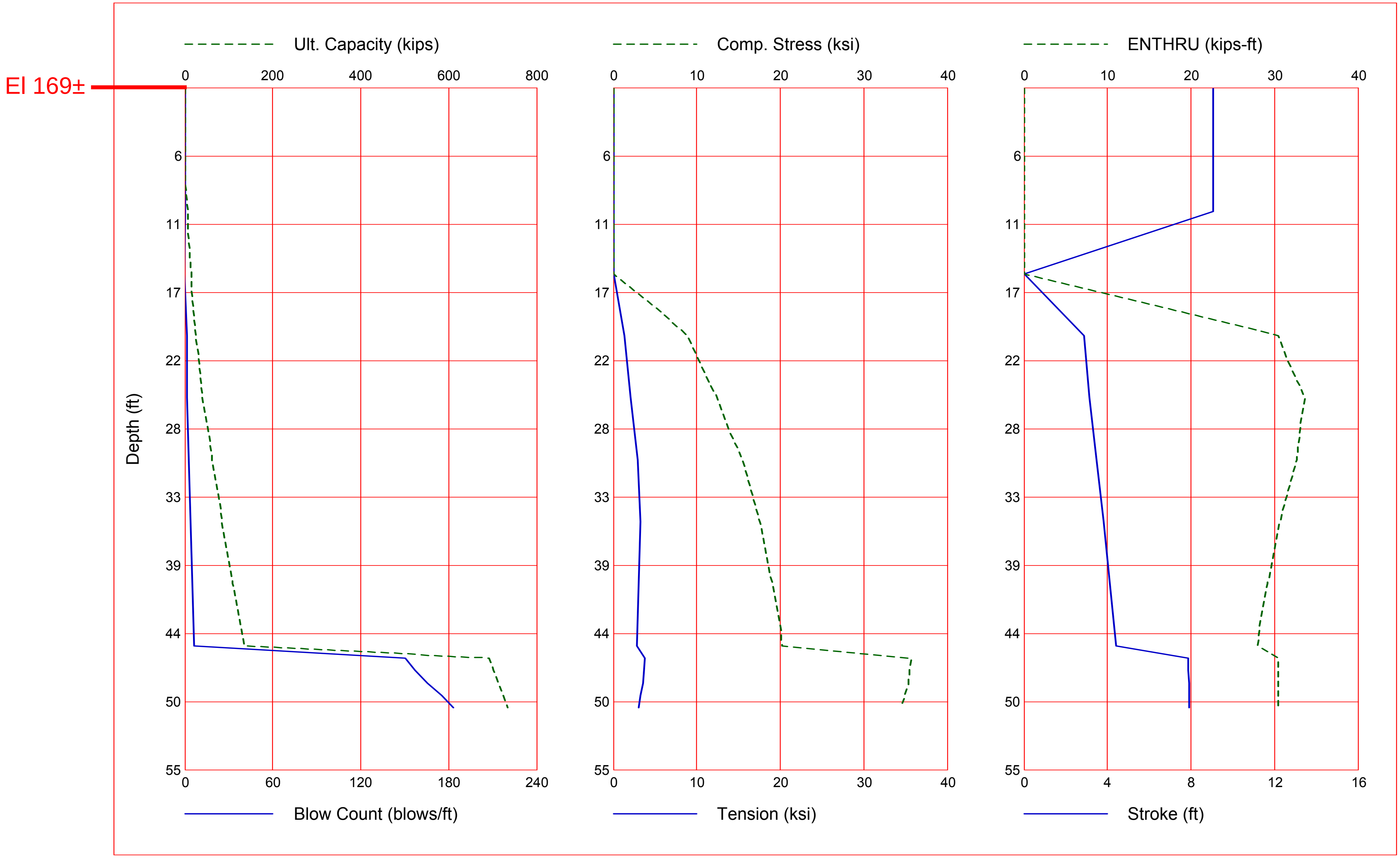
Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

18-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.5 in.)

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 7_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 7_18 in.

Jan 23 2015
GRLWEAP Version 2010

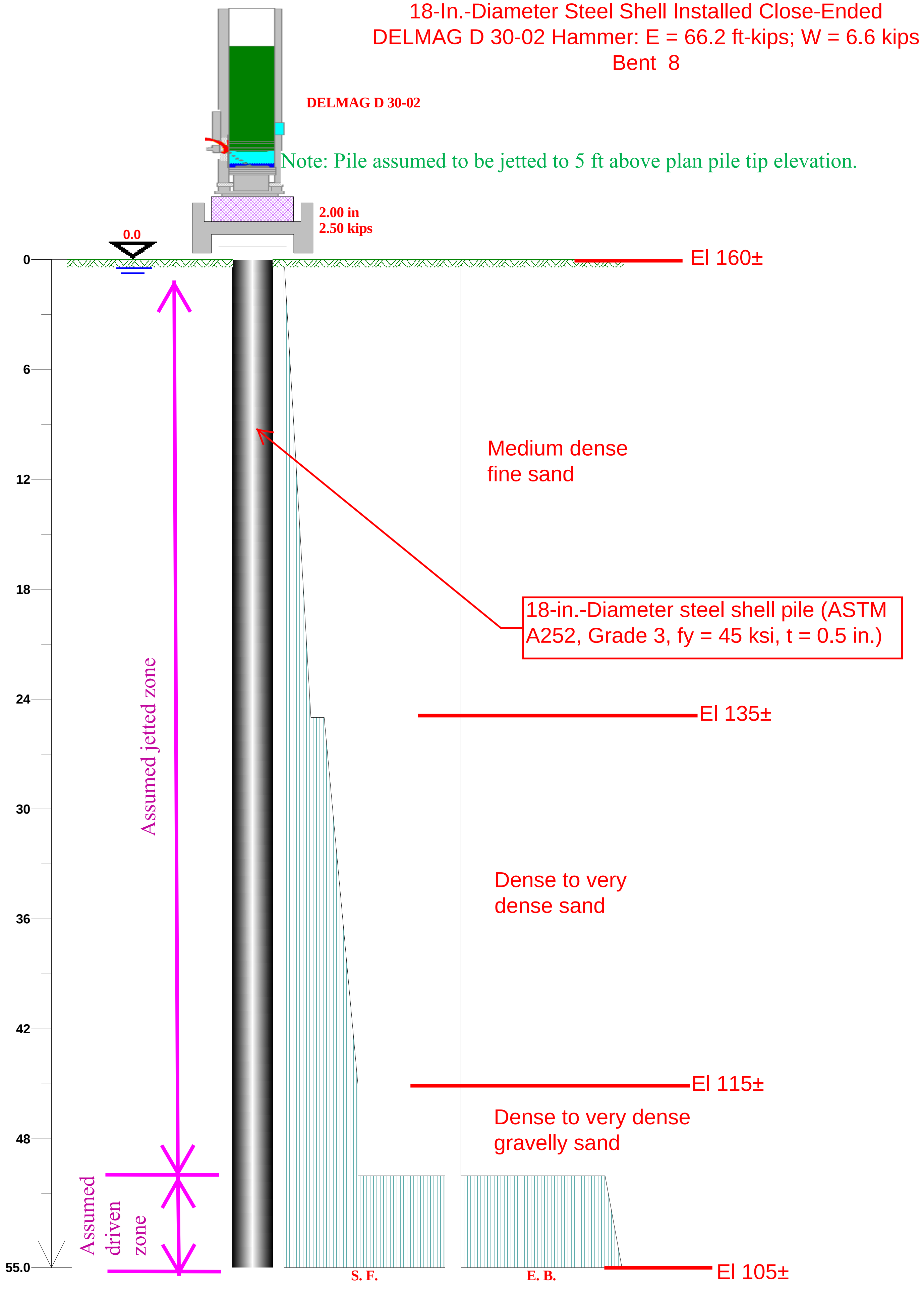
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	9.05	0.0
5.0	1.6	1.6	0.0	0.0	0.000	0.000	9.05	0.0
10.0	6.3	6.3	0.0	0.0	0.000	0.000	9.05	0.0
15.0	14.6	14.6	0.0	-1.0	0.000	0.000	0.00	0.0
20.0	25.8	25.8	0.0	1.4	8.874	-1.325	2.88	30.4
25.0	38.9	38.9	0.0	1.6	12.456	-2.090	3.15	33.6
30.0	60.8	60.8	0.0	2.5	15.531	-2.936	3.48	32.7
35.0	84.1	84.1	0.0	3.7	17.544	-3.215	3.81	30.7
40.0	109.0	109.0	0.0	5.1	19.014	-3.055	4.11	29.2
45.0	135.2	135.2	0.0	6.6	20.251	-2.856	4.41	28.0
46.0	691.1	146.1	545.0	150.4	35.628	-3.823	7.88	30.4
47.0	701.9	156.9	545.0	157.2	35.478	-3.723	7.88	30.5
48.0	712.7	167.7	545.0	165.7	35.333	-3.526	7.90	30.5
49.0	723.5	178.5	545.0	175.1	34.931	-3.305	7.90	30.4
50.0	734.3	189.3	545.0	183.4	34.453	-3.082	7.90	30.5

Total Continuous Driving Time 19.00 minutes; Total Number of Blows 830

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7

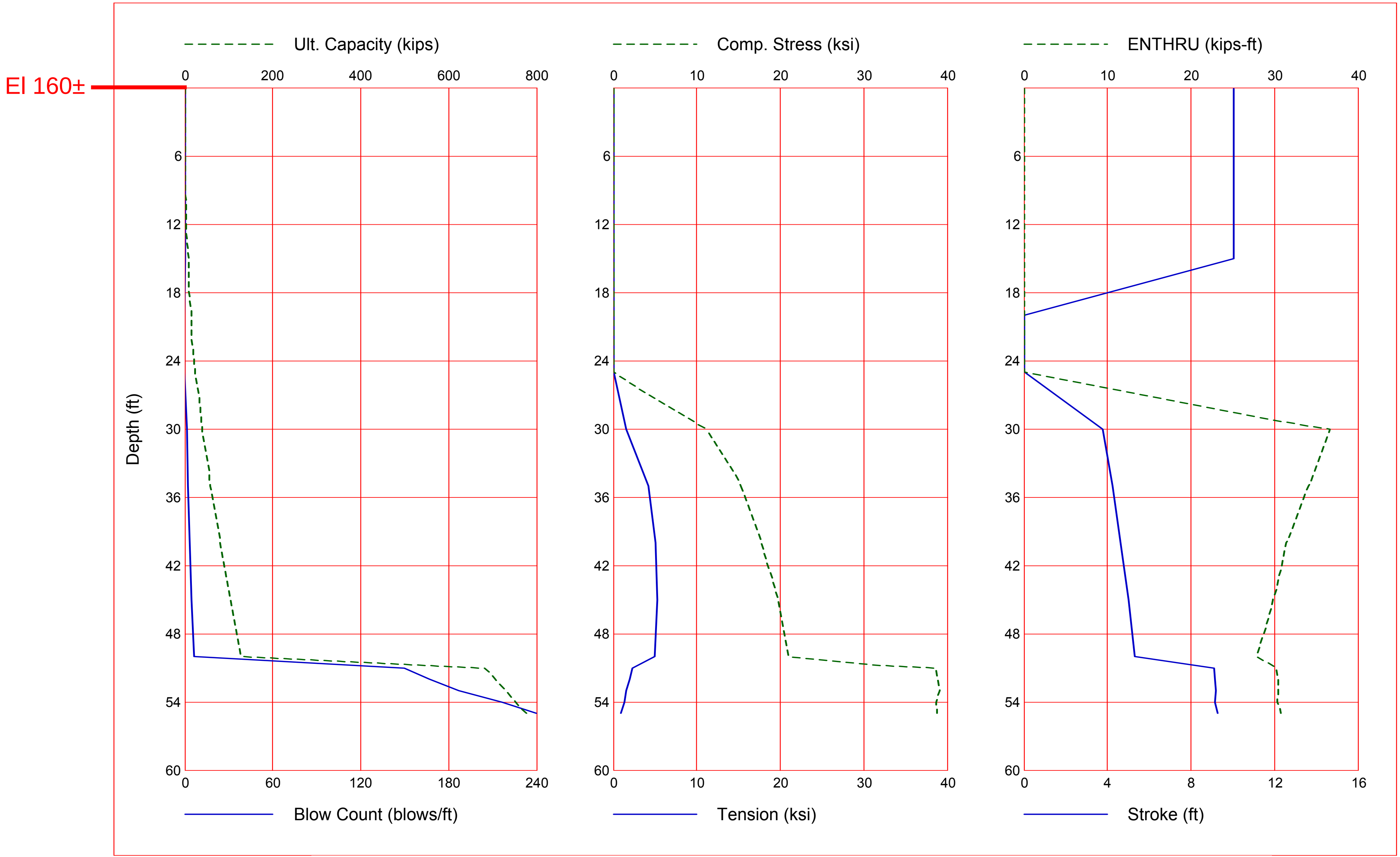
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 8_18 In.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 29 2015
13-017_Bent 8_18 In. GRLWEAP Version 2010

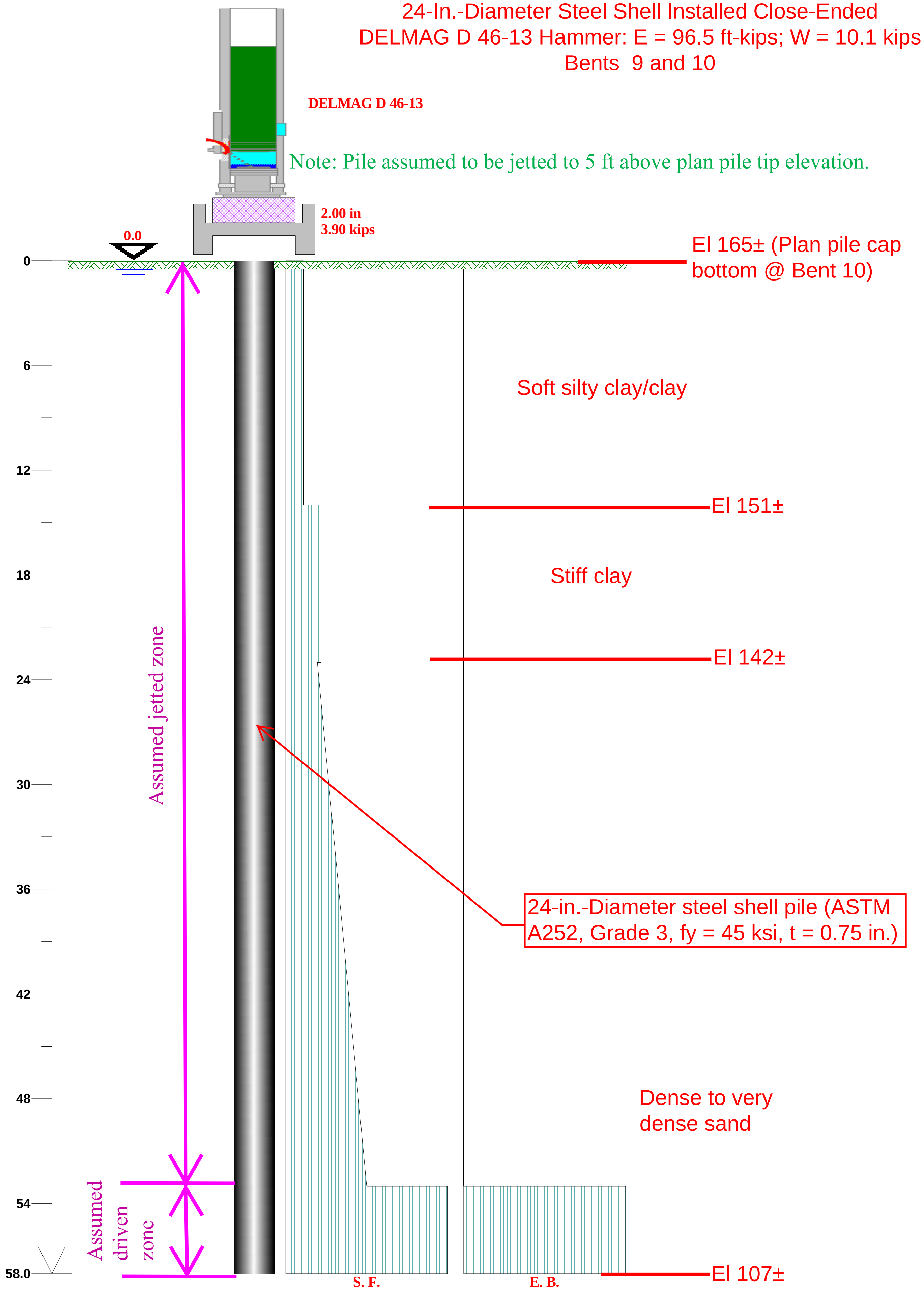
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	10.03	0.0
5.0	0.9	0.9	0.0	0.0	0.000	0.000	10.03	0.0
10.0	3.8	3.8	0.0	0.0	0.000	0.000	10.03	0.0
15.0	8.5	8.5	0.0	0.0	0.000	0.000	10.03	0.0
20.0	15.0	15.0	0.0	-1.0	0.000	0.000	0.00	0.0
25.0	23.5	23.5	0.0	-1.0	0.000	0.000	0.00	0.0
30.0	39.1	39.1	0.0	1.4	11.107	-1.556	3.78	36.6
35.0	57.6	57.6	0.0	2.2	15.285	-4.178	4.24	34.1
40.0	79.0	79.0	0.0	3.3	17.808	-5.035	4.61	31.4
45.0	103.4	103.4	0.0	4.7	19.736	-5.293	5.02	29.8
50.0	129.2	129.2	0.0	6.3	20.979	-4.979	5.32	27.9
51.0	682.0	140.5	541.5	149.9	38.661	-2.340	9.11	30.2
52.0	705.7	151.8	553.9	166.9	38.860	-1.976	9.16	30.4
53.0	729.3	163.1	566.2	186.7	39.048	-1.561	9.20	30.5
54.0	752.9	174.4	578.6	216.5	38.636	-1.370	9.16	30.3
55.0	776.6	185.7	590.9	239.6	38.723	-0.944	9.28	30.8

Total Continuous Driving Time 23.00 minutes; Total Number of Blows 917

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8

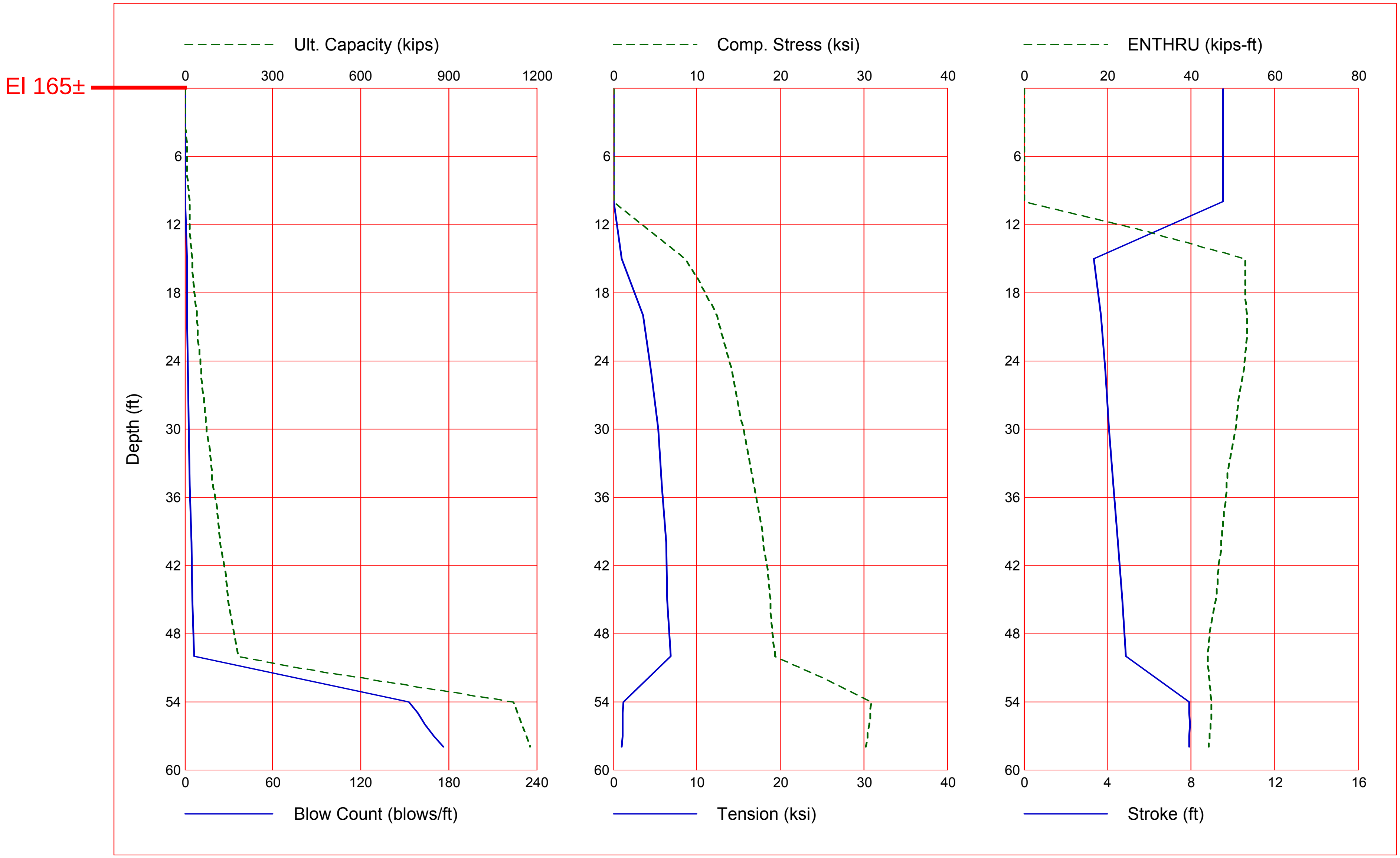
Model for Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 10_24 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 10_24 in.

Jan 23 2015
GRLWEAP Version 2010

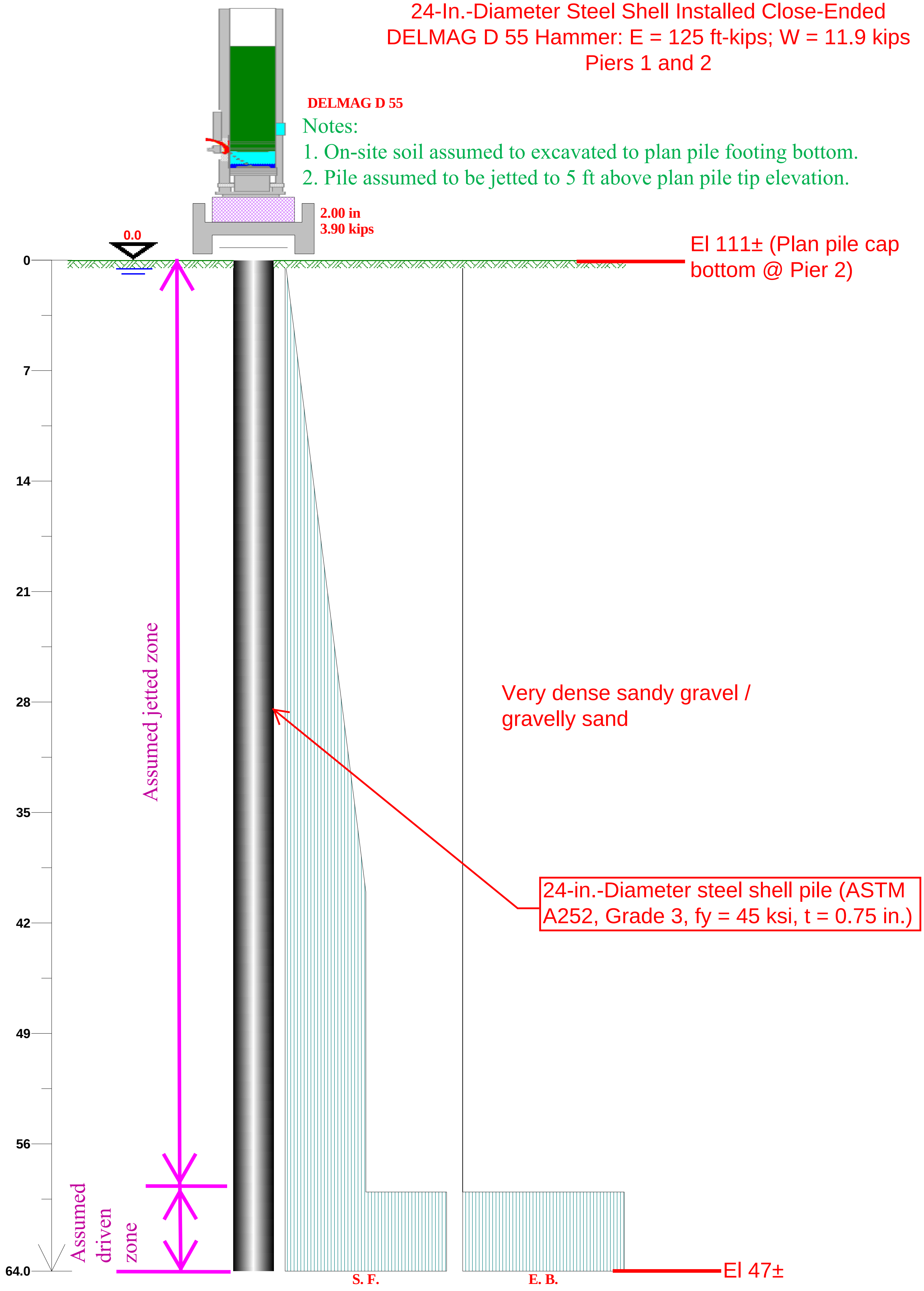
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	9.52	0.0
5.0	7.9	7.9	0.0	0.0	0.000	0.000	9.52	0.0
10.0	15.8	15.8	0.0	0.0	0.000	0.000	9.52	0.0
15.0	25.2	25.2	0.0	1.4	8.624	-1.045	3.34	53.0
20.0	41.0	41.0	0.0	1.8	12.445	-3.533	3.68	53.4
25.0	56.4	56.4	0.0	2.2	14.281	-4.554	3.91	52.5
30.0	73.8	73.8	0.0	2.7	15.649	-5.347	4.09	50.6
35.0	95.0	95.0	0.0	3.5	16.870	-5.858	4.27	48.5
40.0	119.8	119.8	0.0	4.3	17.973	-6.312	4.50	47.2
45.0	148.3	148.3	0.0	5.3	18.841	-6.498	4.71	45.9
50.0	180.5	180.5	0.0	6.5	19.388	-6.865	4.87	44.1
54.0	1120.1	216.1	904.0	153.0	30.883	-1.190	7.92	45.0
55.0	1134.6	230.6	904.0	159.0	30.767	-1.166	7.93	44.8
56.0	1149.1	245.1	904.0	163.6	30.703	-1.122	7.94	44.7
57.0	1163.6	259.6	904.0	169.8	30.470	-1.122	7.93	44.5
58.0	1178.1	274.1	904.0	176.7	30.207	-1.053	7.93	44.3

Total Continuous Driving Time 25.00 minutes; Total Number of Blows 1098

Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10

Model for Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips
Piers 1 and 2



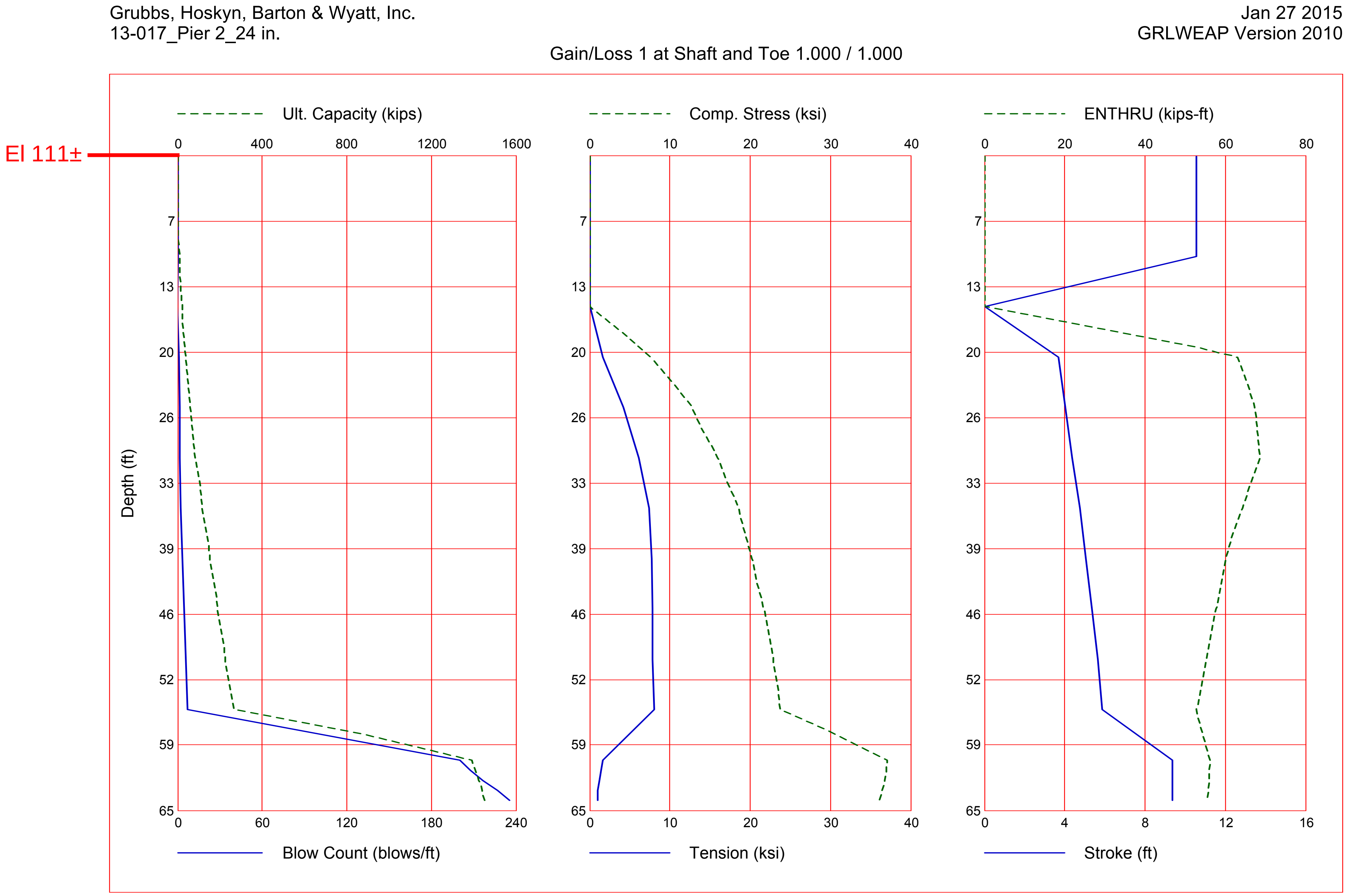
Notes:
1. On-site soil assumed to excavated to plan pile footing bottom.
2. Pile assumed to be jetted to 5 ft above plan pile tip elevation.

El 111± (Plan pile cap bottom @ Pier 2)

Very dense sandy gravel / gravelly sand

24-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.75 in.)

El 47±



Results of Driveability Analysis

24-In.-Diameter Steel Shell Installed Close-Ended

DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips

Bents 9 and 10 and Piers 1 and 2

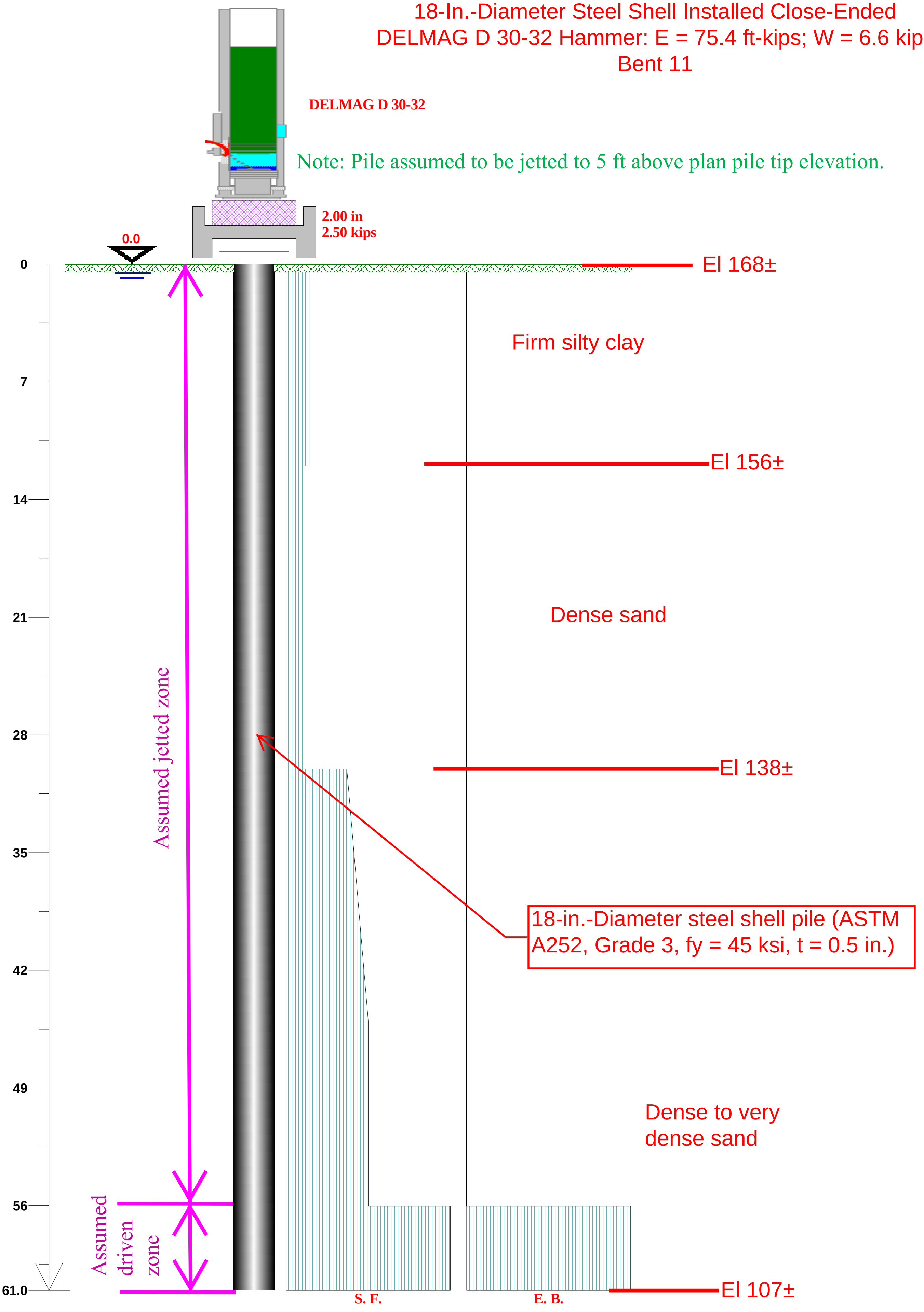
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	10.54	0.0
5.0	2.4	2.4	0.0	0.0	0.000	0.000	10.54	0.0
10.0	9.4	9.4	0.0	0.0	0.000	0.000	10.54	0.0
15.0	21.3	21.3	0.0	-1.0	0.000	0.000	0.00	0.0
20.0	37.8	37.8	0.0	1.1	7.624	-1.690	3.71	63.0
25.0	59.1	59.1	0.0	1.3	12.736	-4.235	4.02	67.2
30.0	85.0	85.0	0.0	1.6	15.973	-6.082	4.36	68.5
35.0	115.8	115.8	0.0	2.3	18.580	-7.433	4.74	64.3
40.0	151.2	151.2	0.0	3.3	20.172	-7.698	5.04	60.0
45.0	189.0	189.0	0.0	4.4	21.679	-7.833	5.36	57.6
50.0	226.8	226.8	0.0	5.6	22.928	-7.833	5.66	55.4
55.0	264.6	264.6	0.0	6.8	23.764	-8.002	5.87	52.9
60.0	1392.9	310.0	1082.9	200.4	37.103	-1.614	9.38	56.2
61.0	1408.0	325.1	1082.9	207.6	36.982	-1.391	9.38	56.1
62.0	1423.1	340.2	1082.9	216.4	36.746	-1.228	9.37	55.9
63.0	1438.2	355.3	1082.9	226.8	36.473	-1.040	9.36	55.7
64.0	1453.4	370.4	1082.9	235.6	36.117	-0.989	9.36	55.6

Total Continuous Driving Time 38.00 minutes; Total Number of Blows 1502

Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips
Bents 9 and 10 and Piers 1 and 2

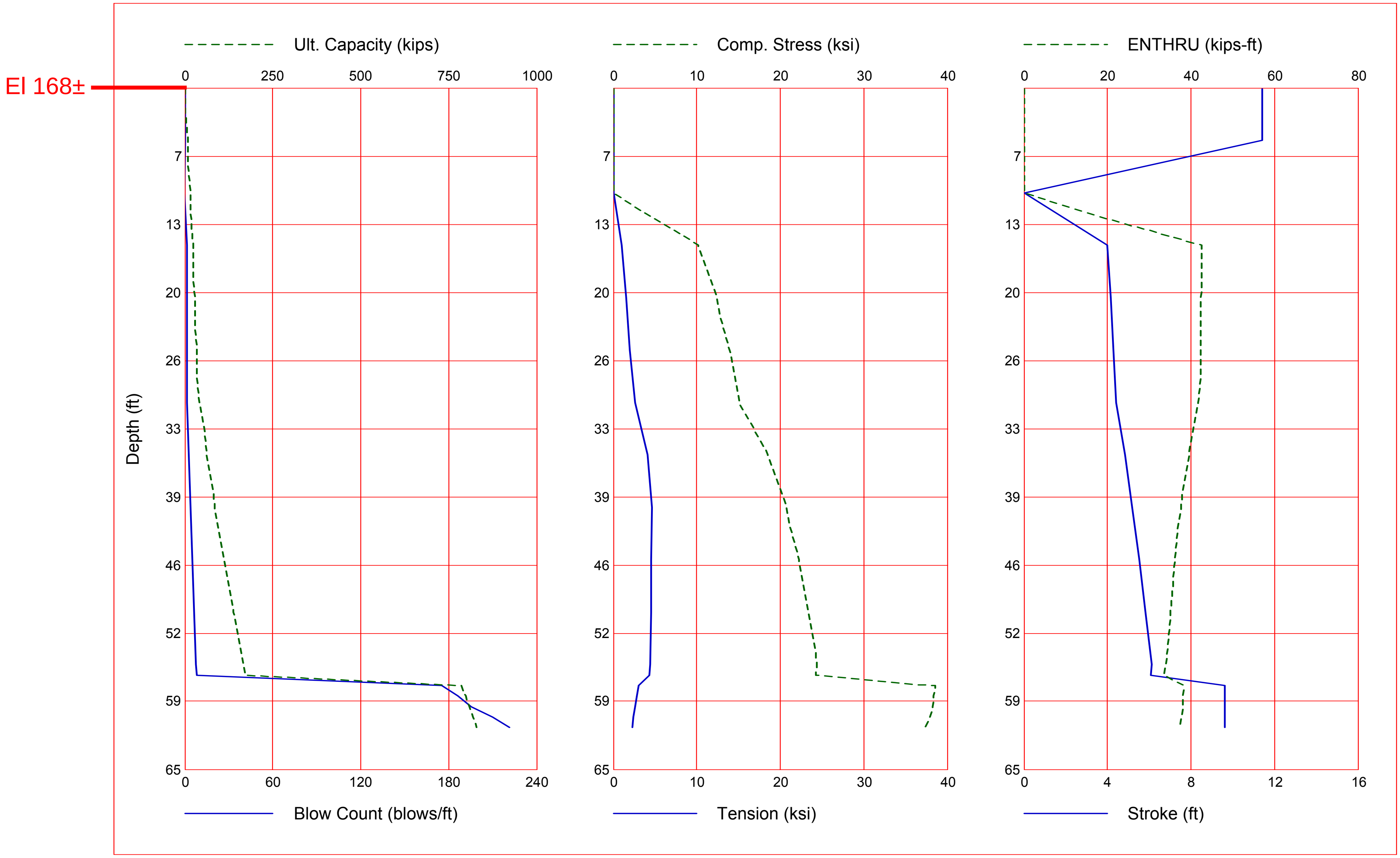
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 11_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11

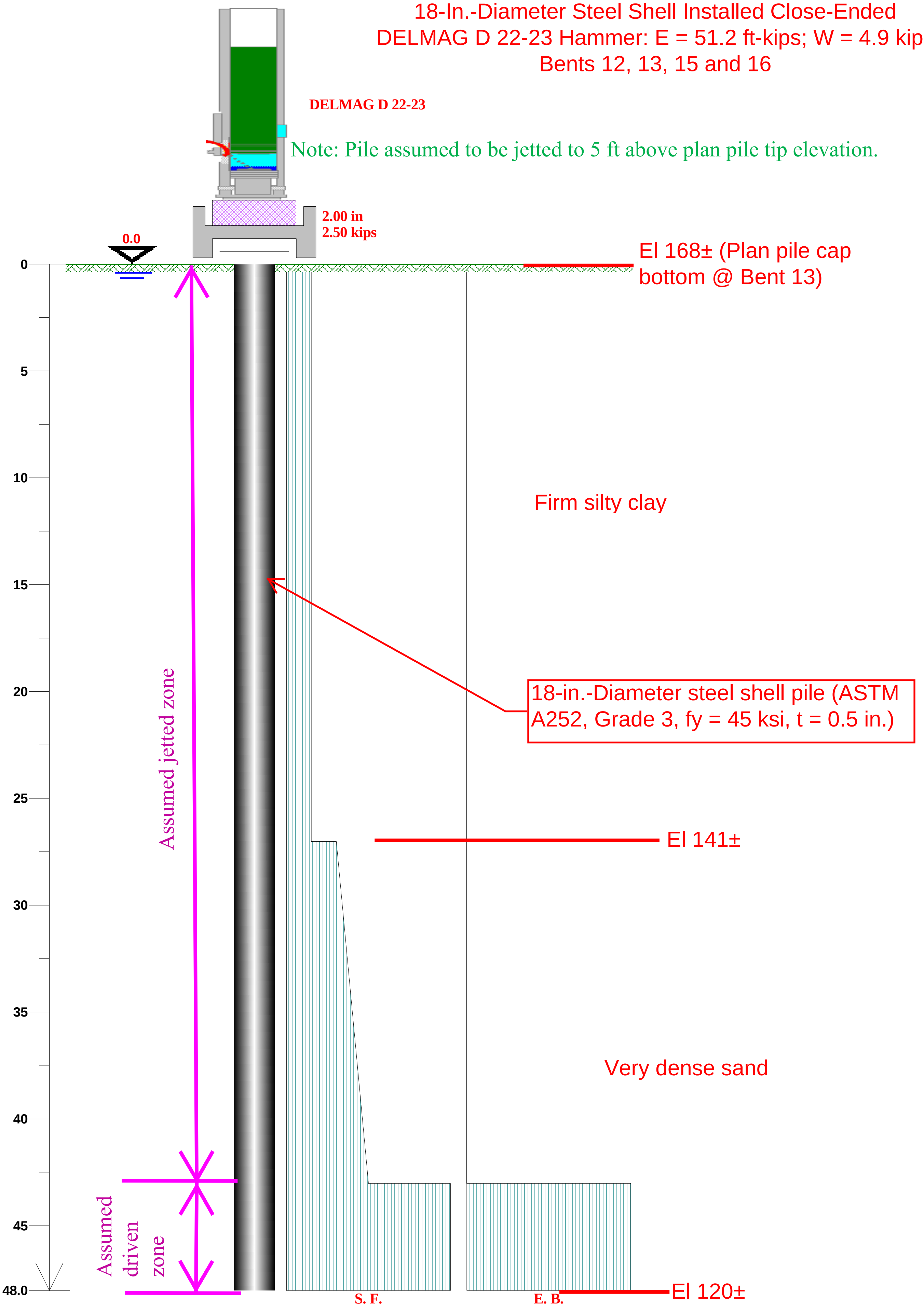
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	11.43	0.0
5.0	8.2	8.2	0.0	0.0	0.000	0.000	11.43	0.0
10.0	16.5	16.5	0.0	-1.0	0.000	0.000	0.00	0.0
15.0	23.3	23.3	0.0	1.4	10.210	-1.019	4.00	42.5
20.0	29.1	29.1	0.0	1.5	12.455	-1.549	4.18	42.3
25.0	35.0	35.0	0.0	1.6	13.895	-1.945	4.30	42.3
30.0	40.9	40.9	0.0	1.8	15.067	-2.619	4.42	41.8
35.0	62.0	62.0	0.0	2.7	18.532	-4.119	4.83	39.6
40.0	85.5	85.5	0.0	3.7	20.787	-4.626	5.19	37.7
45.0	111.4	111.4	0.0	4.9	22.231	-4.506	5.52	36.2
50.0	138.4	138.4	0.0	6.1	23.400	-4.562	5.82	35.1
55.0	165.4	165.4	0.0	7.6	24.346	-4.397	6.11	34.1
56.0	170.8	170.8	0.0	8.0	24.235	-4.269	6.10	33.6
57.0	786.7	181.7	605.0	175.2	38.574	-3.019	9.64	38.3
58.0	797.5	192.5	605.0	186.5	38.344	-2.791	9.63	38.1
59.0	808.3	203.3	605.0	195.6	38.238	-2.630	9.63	38.0
60.0	819.1	214.1	605.0	210.0	37.882	-2.401	9.62	37.6
61.0	829.9	224.9	605.0	221.7	37.439	-2.260	9.61	37.5

Total Continuous Driving Time 26.00 minutes; Total Number of Blows 1028

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16

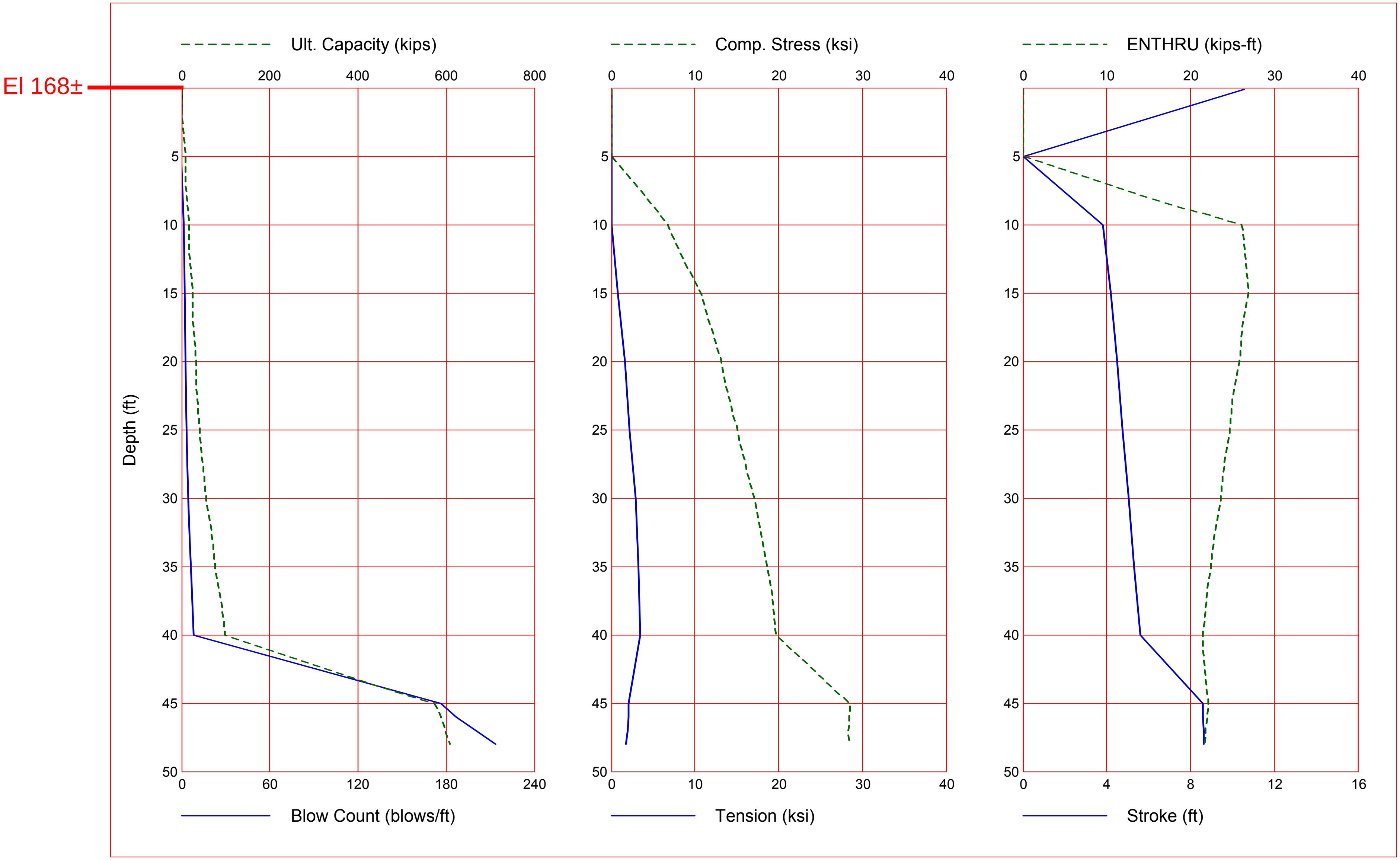


Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 13_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16

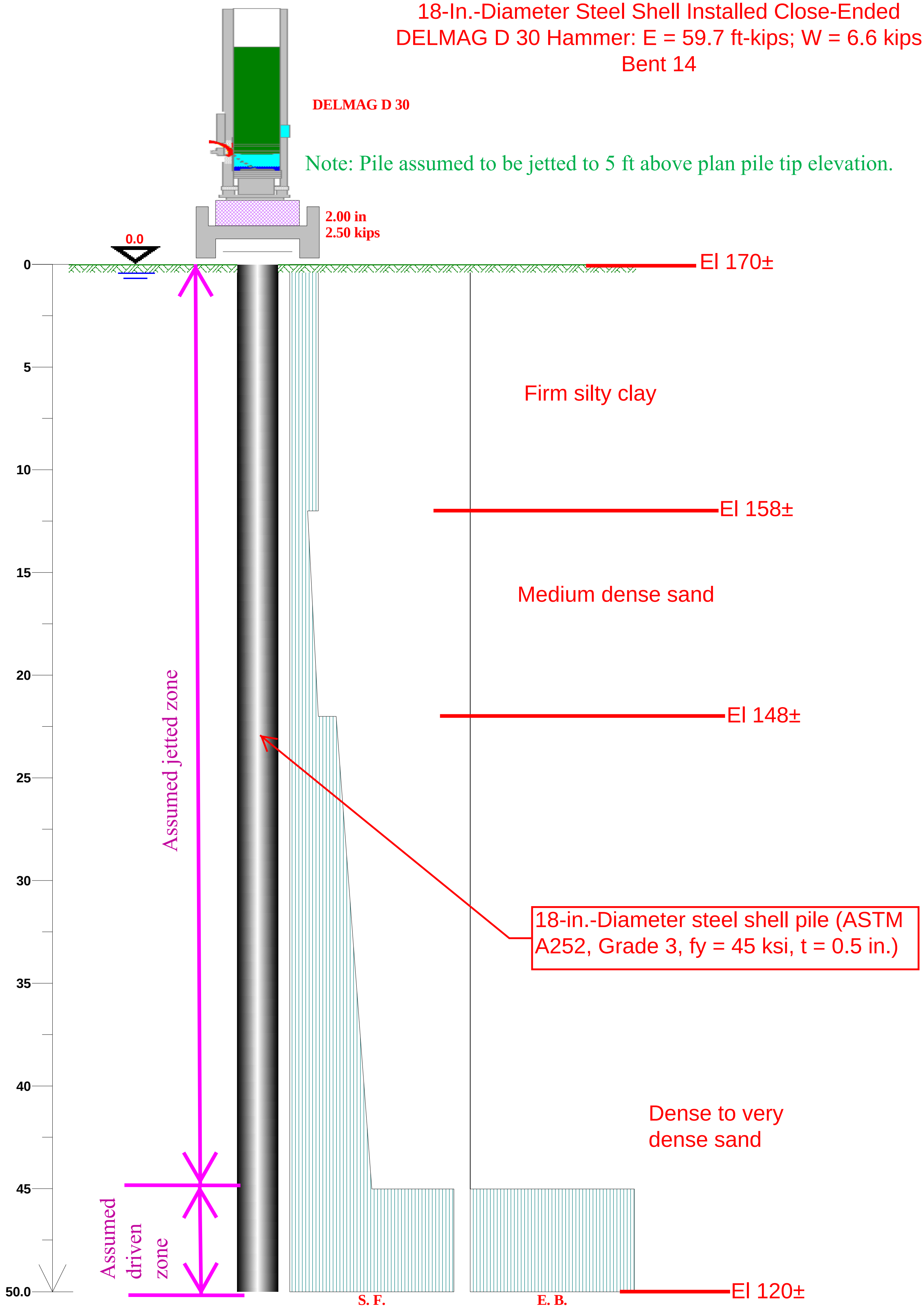
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	10.56	0.0
5.0	8.2	8.2	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	16.4	16.4	0.0	1.7	6.797	0.000	3.82	26.3
15.0	24.7	24.7	0.0	2.1	10.734	-0.771	4.21	26.9
20.0	32.9	32.9	0.0	2.8	13.146	-1.629	4.51	25.9
25.0	41.1	41.1	0.0	3.6	15.053	-2.239	4.77	24.7
30.0	54.9	54.9	0.0	4.7	17.106	-2.913	5.04	23.6
35.0	75.0	75.0	0.0	6.2	18.565	-3.214	5.33	22.5
40.0	98.4	98.4	0.0	8.0	19.702	-3.446	5.62	21.5
45.0	576.6	135.6	441.0	176.4	28.521	-2.075	8.59	22.1
46.0	587.4	146.4	441.0	186.8	28.431	-2.075	8.60	22.0
47.0	598.2	157.2	441.0	200.3	28.356	-1.957	8.62	21.8
48.0	609.0	168.0	441.0	213.6	28.385	-1.812	8.62	21.7

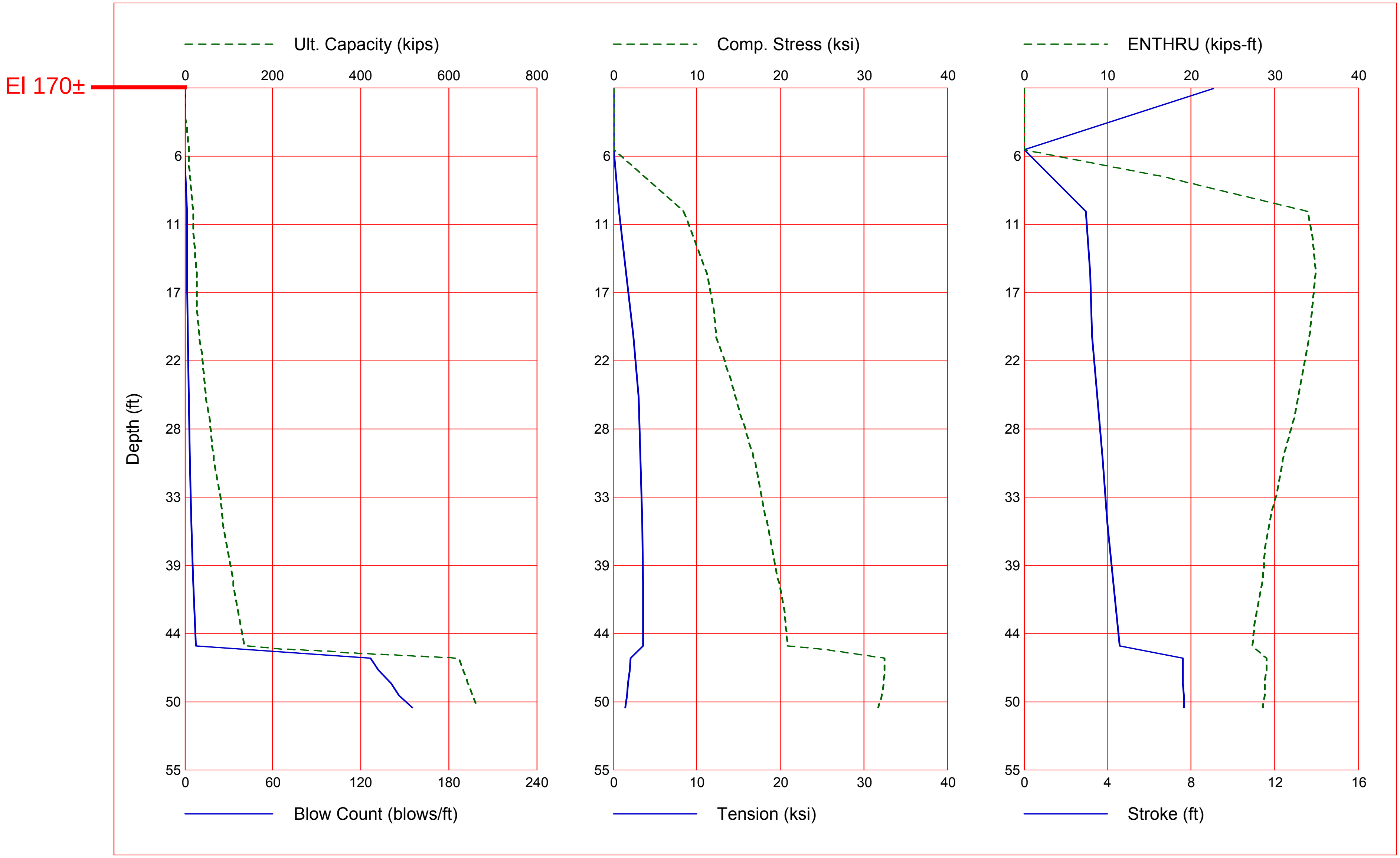
Total Continuous Driving Time 28.00 minutes; Total Number of Blows 1168

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 14



Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 14

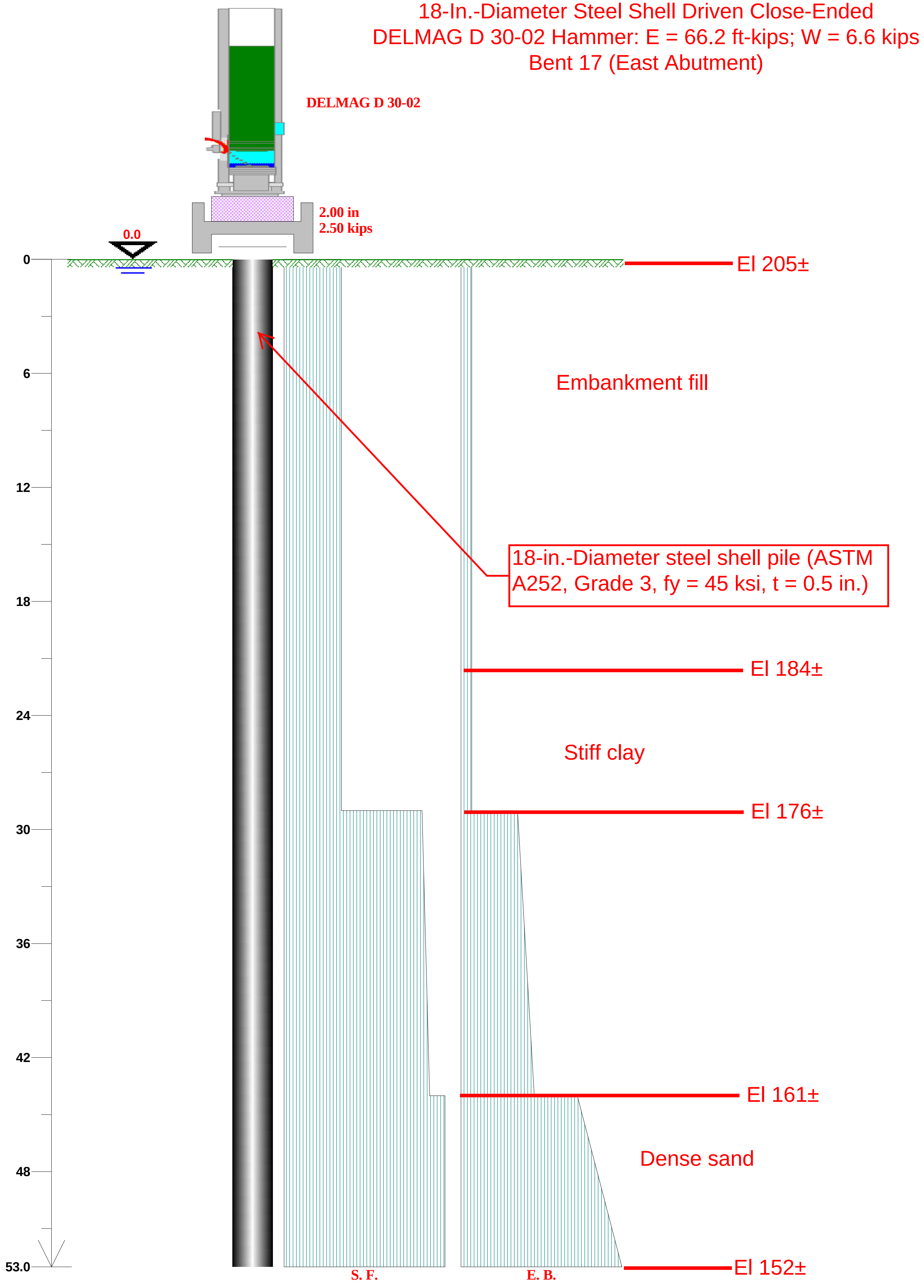
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	9.05	0.0
5.0	9.4	9.4	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	18.8	18.8	0.0	1.5	8.502	-0.644	2.96	34.1
15.0	26.4	26.4	0.0	1.7	11.242	-1.552	3.16	34.9
20.0	34.2	34.2	0.0	2.0	12.353	-2.388	3.26	34.2
25.0	47.5	47.5	0.0	2.6	14.700	-3.021	3.52	32.8
30.0	65.5	65.5	0.0	3.5	16.959	-3.247	3.76	31.0
35.0	86.2	86.2	0.0	4.7	18.403	-3.439	4.00	29.4
40.0	109.4	109.4	0.0	6.0	19.897	-3.583	4.30	28.5
45.0	135.1	135.1	0.0	7.6	20.810	-3.594	4.57	27.4
46.0	622.2	145.9	476.2	126.9	32.521	-2.071	7.60	29.1
47.0	633.0	156.7	476.2	132.4	32.463	-1.936	7.62	29.1
48.0	643.8	167.6	476.2	140.4	32.390	-1.796	7.63	28.9
49.0	654.6	178.4	476.2	146.1	32.128	-1.692	7.64	28.9
50.0	665.4	189.2	476.2	155.1	31.766	-1.437	7.65	28.6

Total Continuous Driving Time 17.00 minutes; Total Number of Blows 756

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bents 14

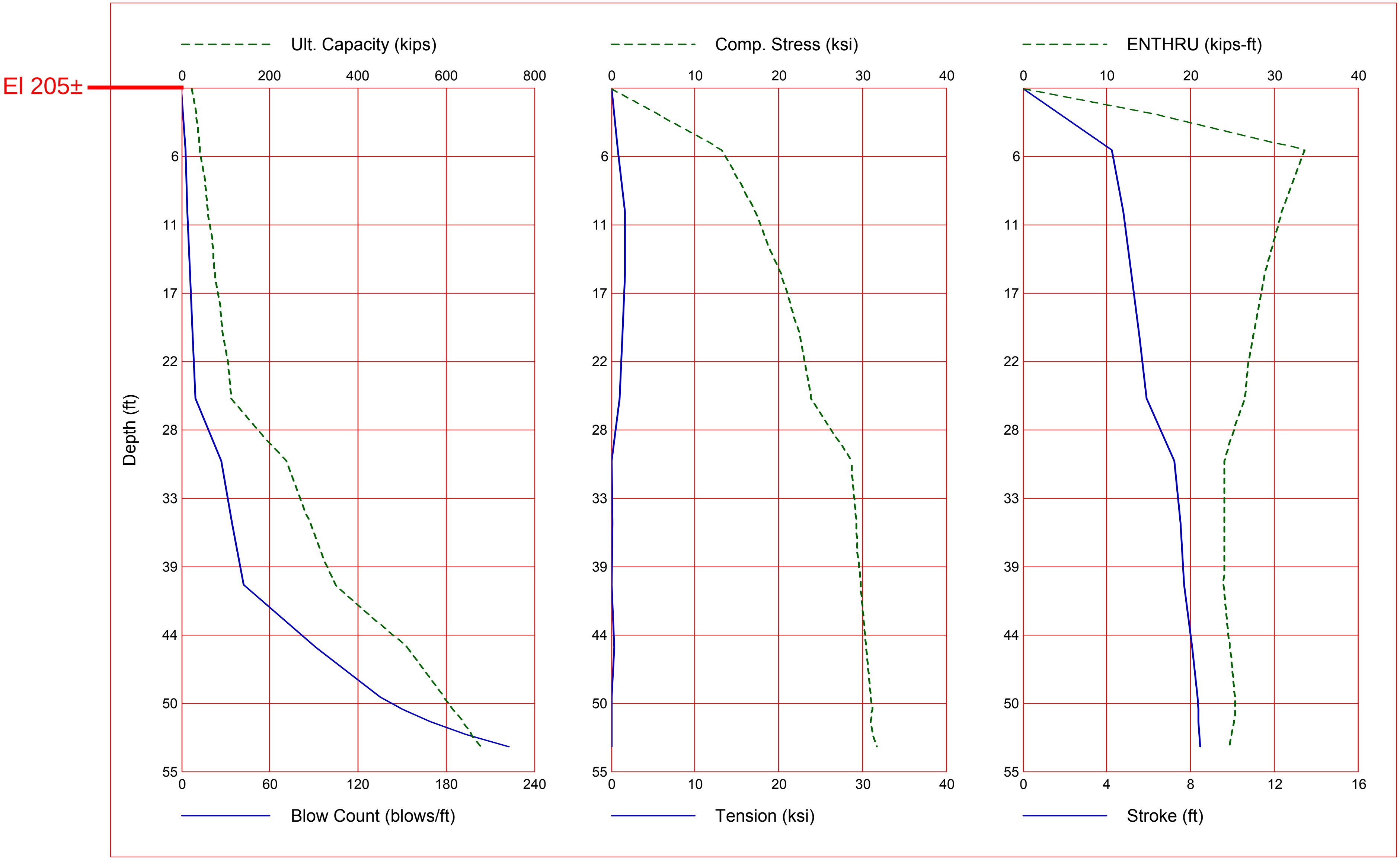
Model for Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17 (East Abutment)



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 17_18 In.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17

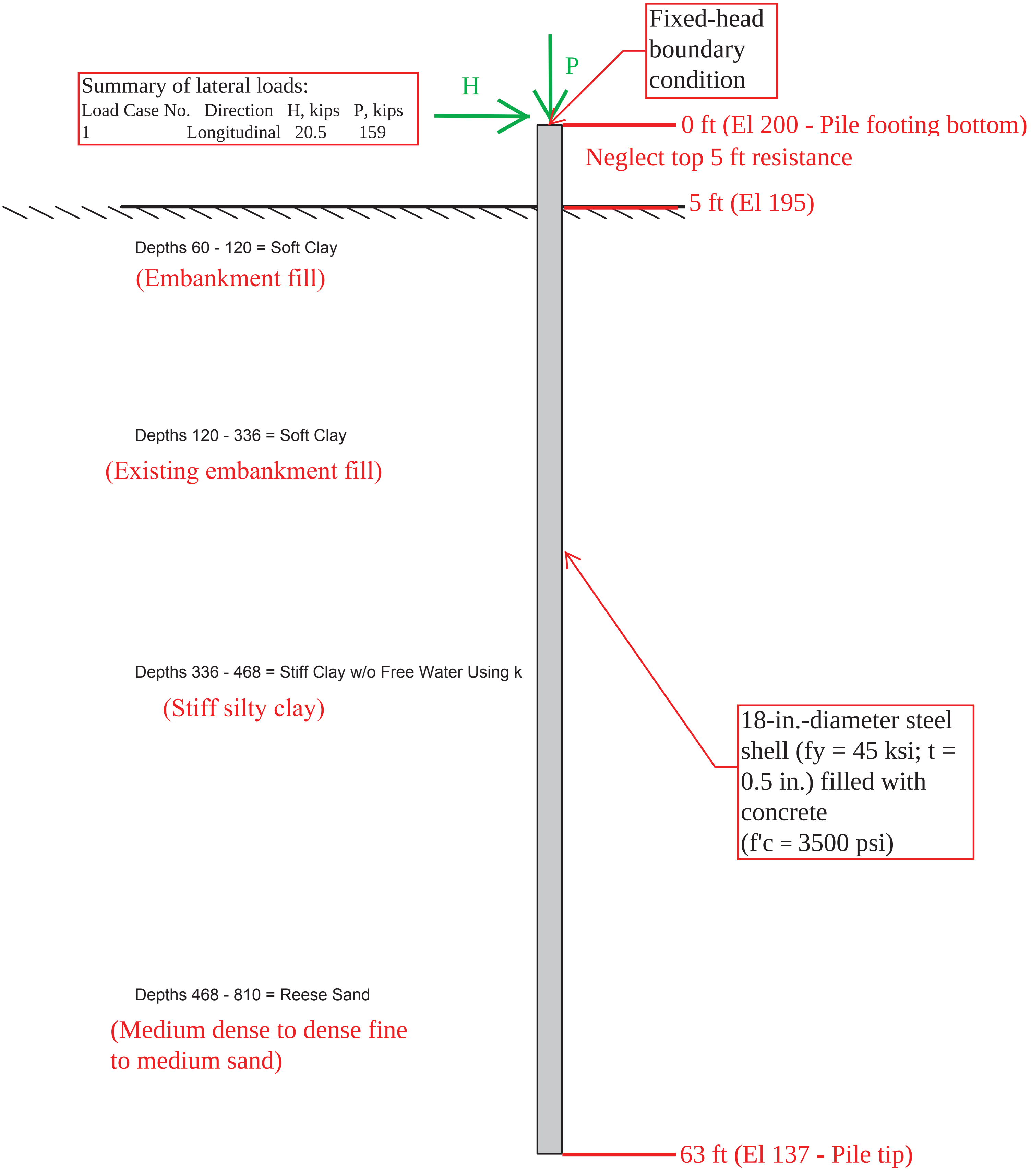
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	24.2	0.4	23.8	-1.0	0.000	0.000	0.00	0.0
5.0	41.4	17.6	23.8	2.5	13.192	-0.818	4.23	33.6
10.0	59.1	35.2	23.8	4.1	17.272	-1.613	4.78	30.9
15.0	76.7	52.9	23.8	5.8	20.335	-1.644	5.20	28.9
20.0	94.3	70.5	23.8	7.6	22.550	-1.284	5.57	27.5
25.0	111.9	88.1	23.8	9.6	23.794	-1.017	5.90	26.5
30.0	238.4	110.7	127.7	27.3	28.710	0.000	7.23	24.1
35.0	293.6	153.5	140.1	34.4	29.254	-0.155	7.53	24.0
40.0	349.6	197.2	152.4	42.2	29.855	0.000	7.70	23.9
45.0	511.0	242.5	268.5	91.5	30.405	-0.360	8.08	24.7
49.0	594.4	282.0	312.4	135.0	30.962	0.000	8.33	25.3
50.0	615.2	291.9	323.4	150.2	31.185	0.000	8.38	25.3
51.0	636.1	301.7	334.4	169.4	31.039	0.000	8.40	25.2
52.0	656.9	311.6	345.3	193.7	31.233	0.000	8.42	24.9
53.0	677.8	321.5	356.3	222.7	31.687	0.000	8.45	24.7

Total Continuous Driving Time 49.00 minutes; Total Number of Blows 2040

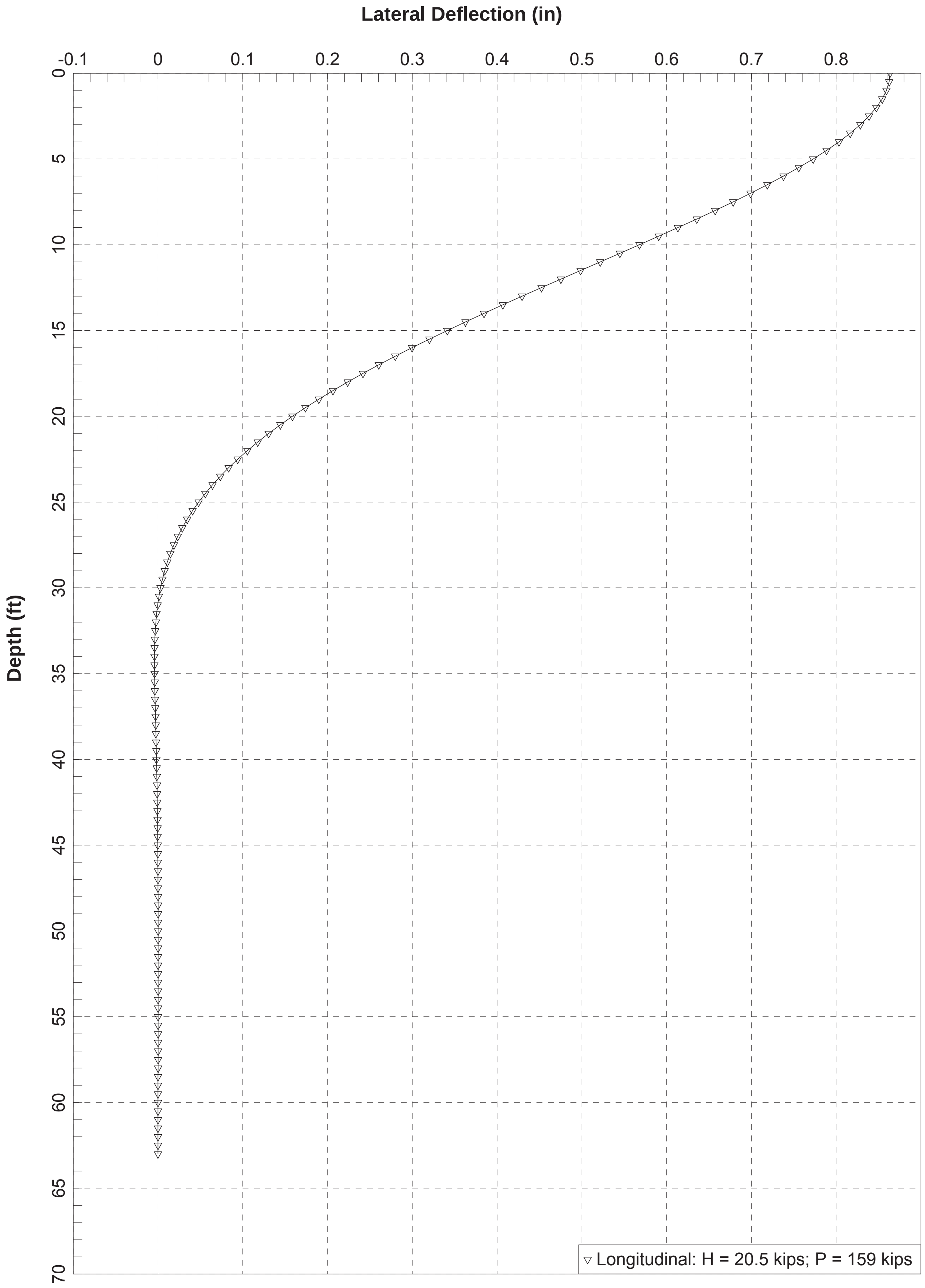
Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17

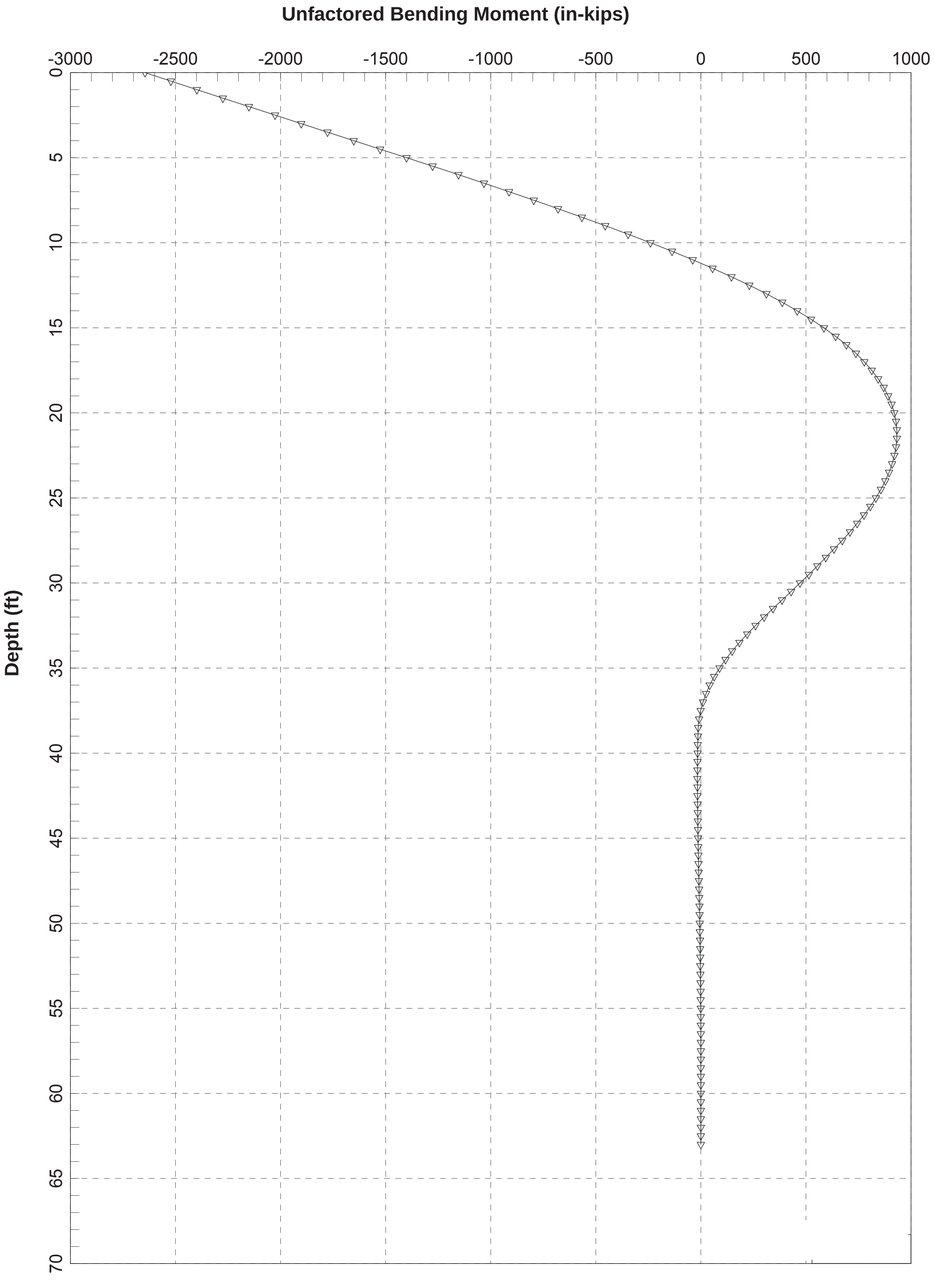
APPENDIX H

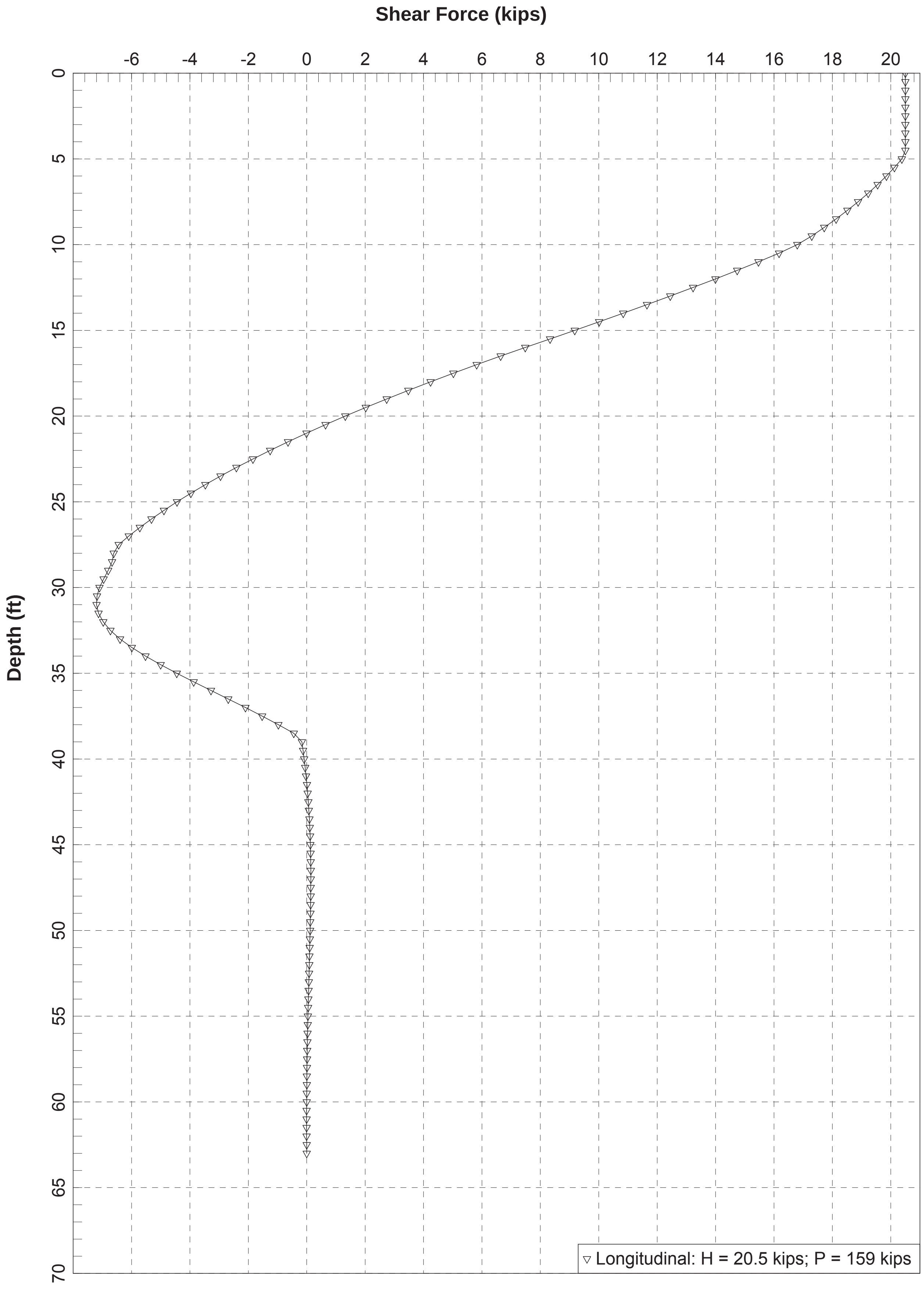


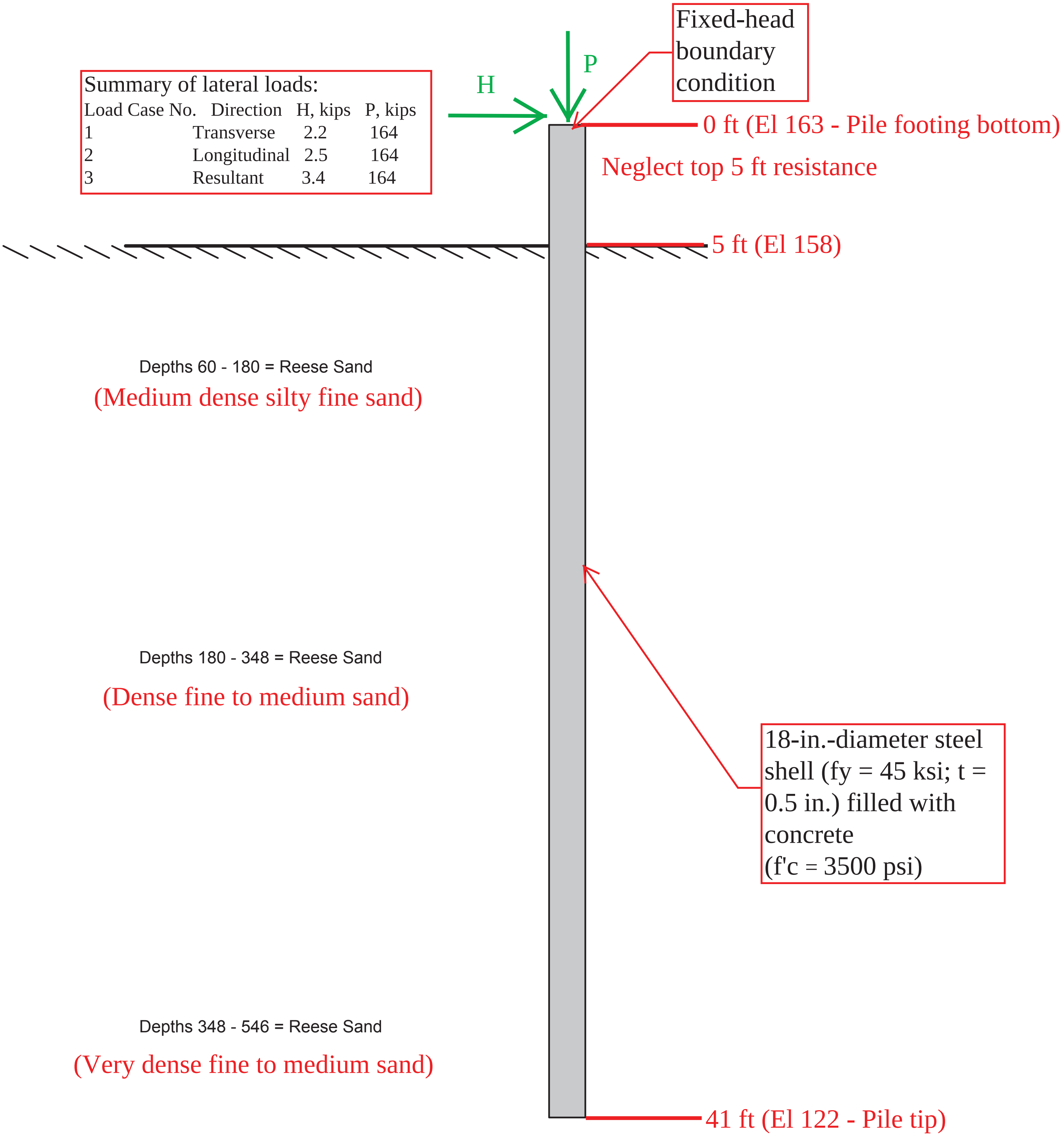
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 1 (West Abutment)
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Longitudinal: H = 20.5 kips; P = 159 kips


Unfactored Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70


Shear Force (kips)
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
20
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Longitudinal: H = 20.5 kips; P = 159 kips

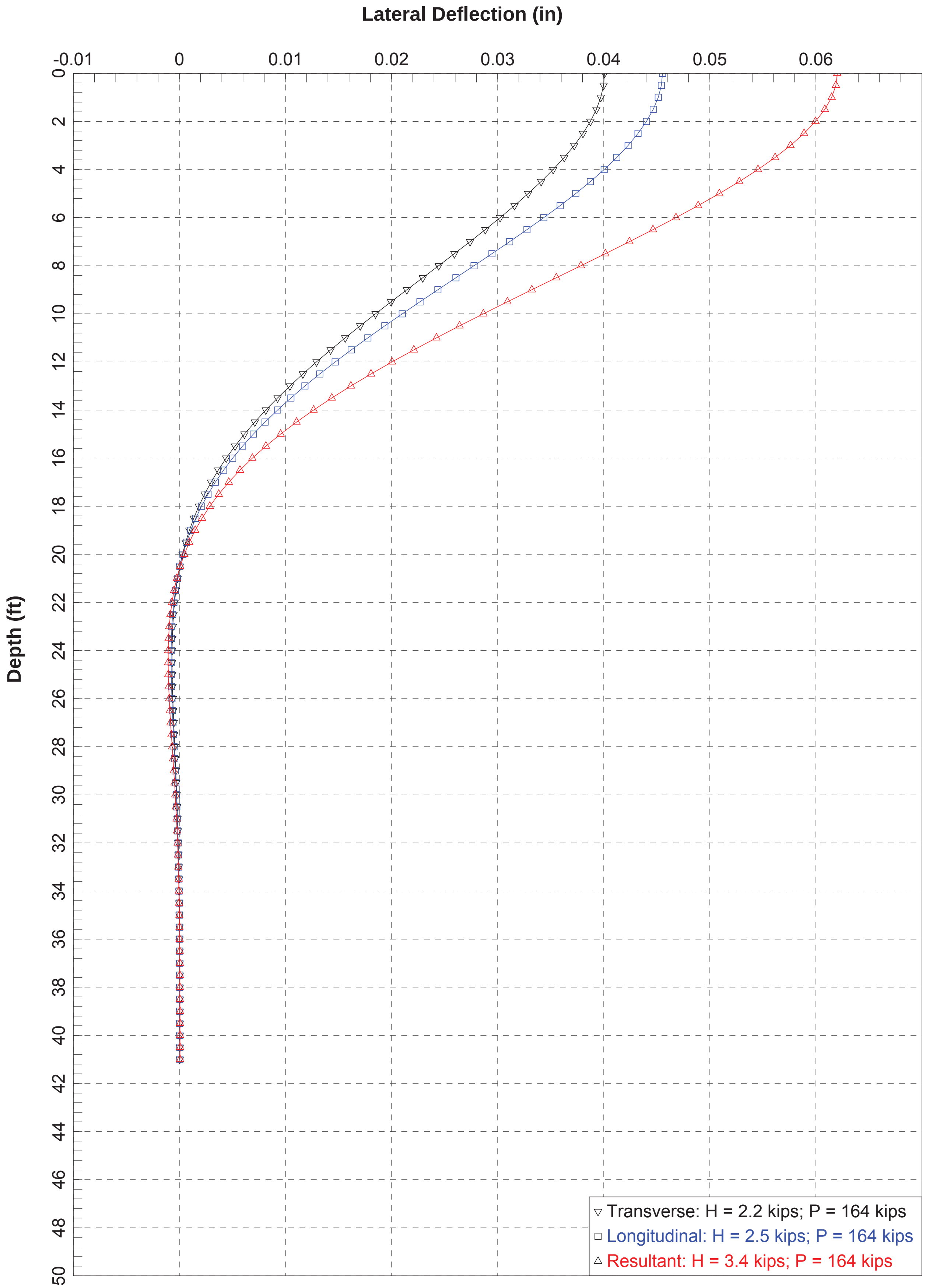


Summary of lateral loads:

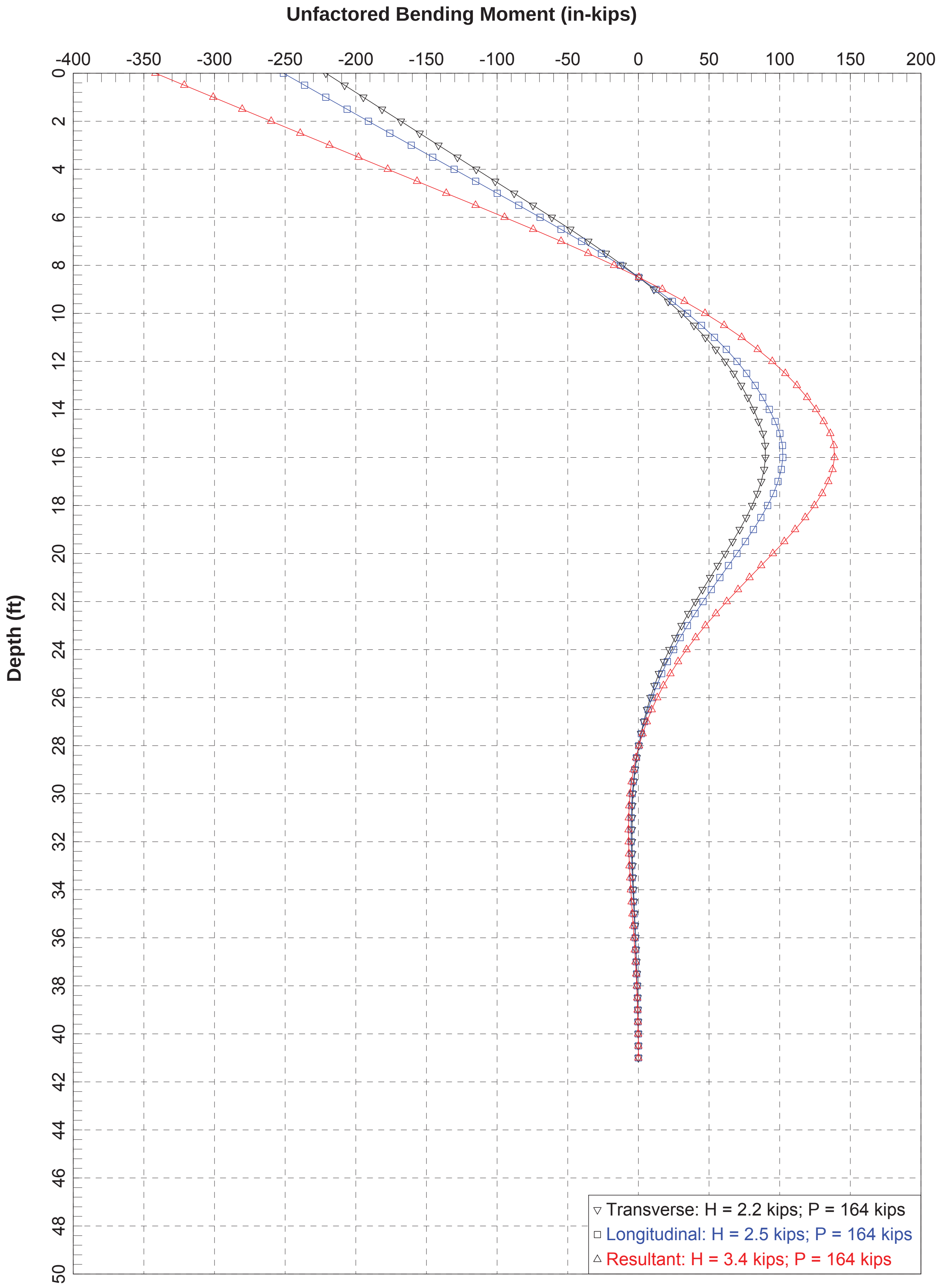
Load Case No.	Direction	H, kips	P, kips
1	Transverse	2.2	164
2	Longitudinal	2.5	164
3	Resultant	3.4	164

Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 2
I-40 over White River

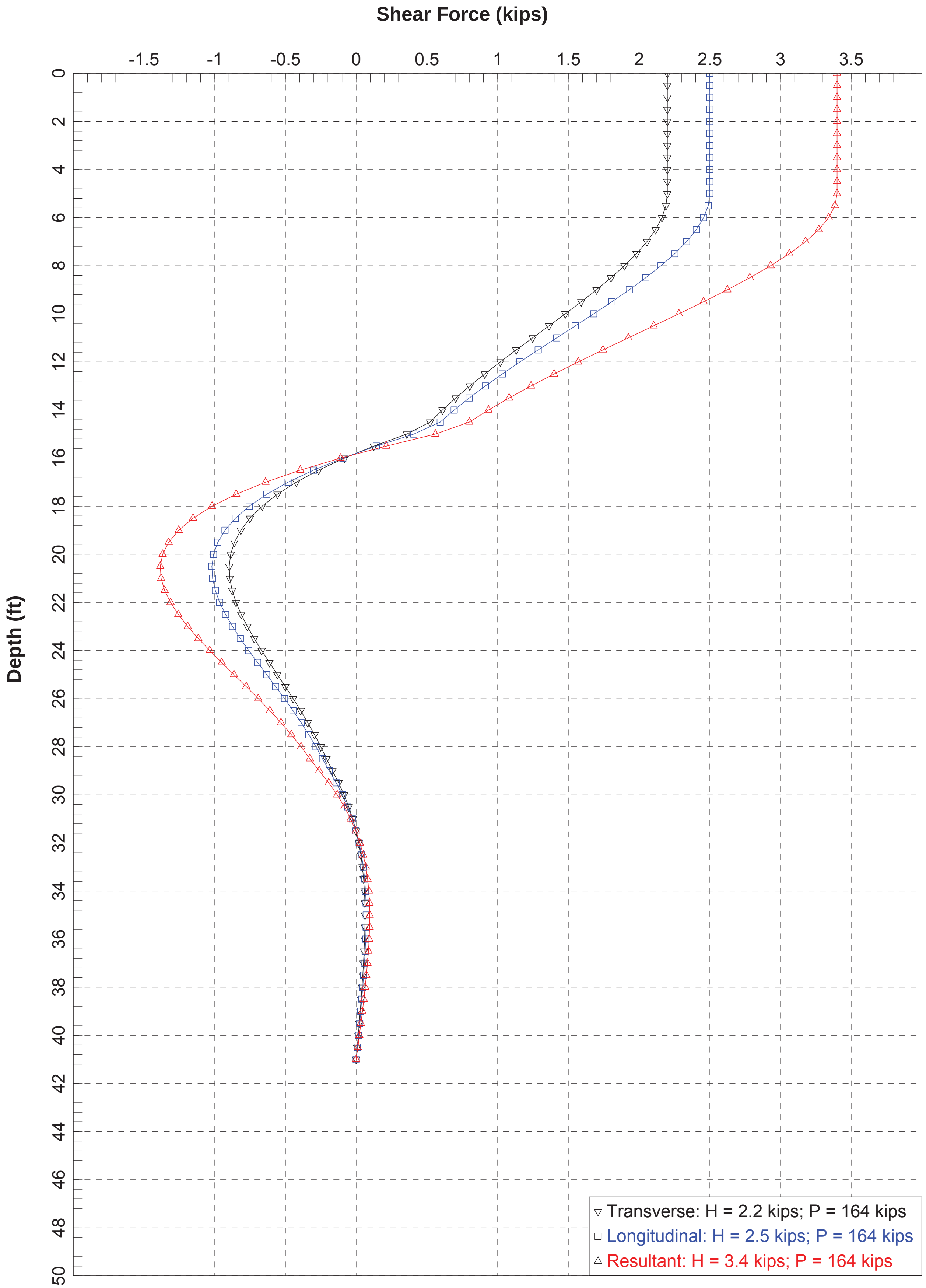
LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.

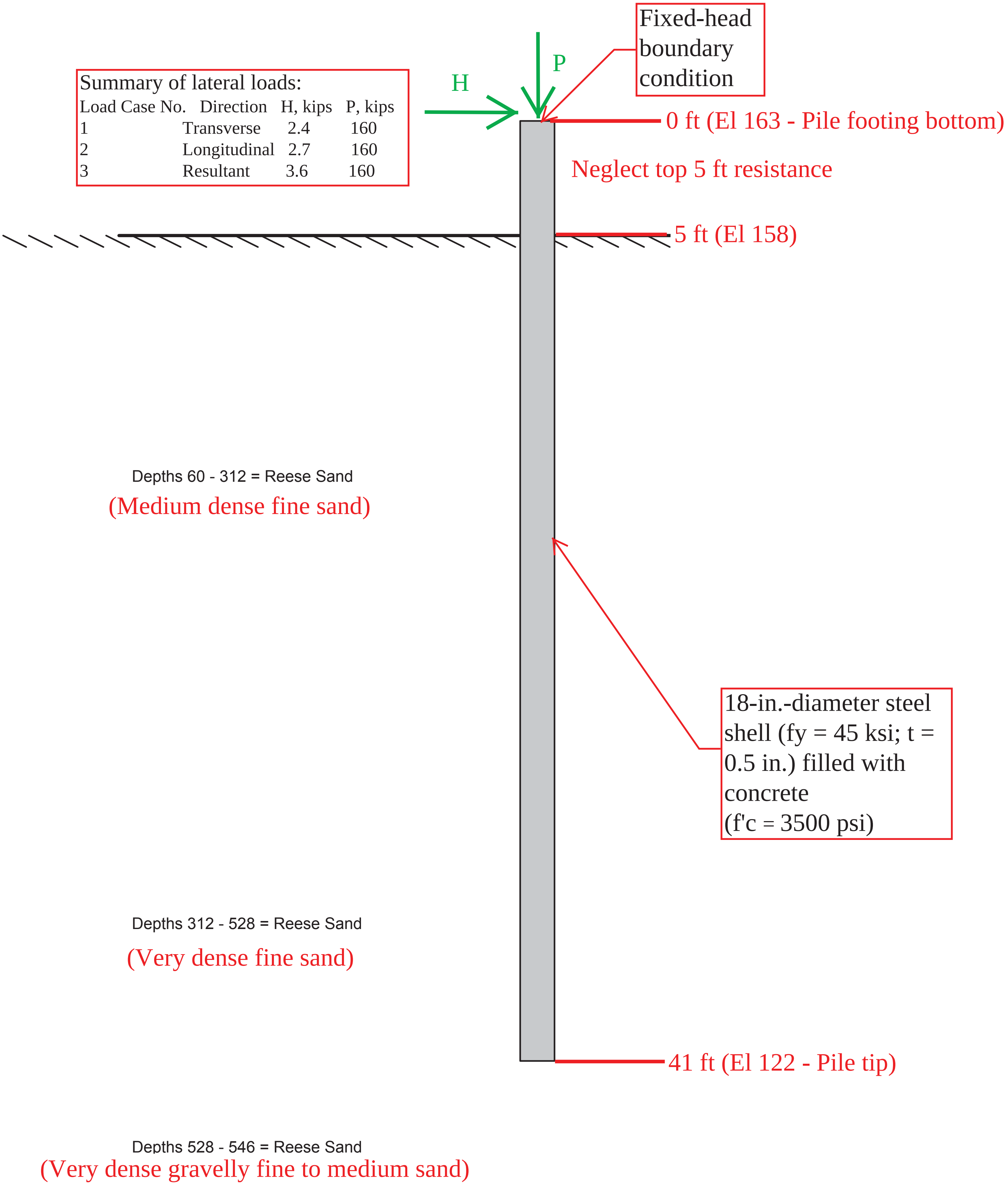


Lateral Deflection (in)
-0.01
0
0.01
0.02
0.03
0.04
0.05
0.06
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips



Unfactored Bending Moment (in-kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips


Shear Force (kips)
-1.5
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips

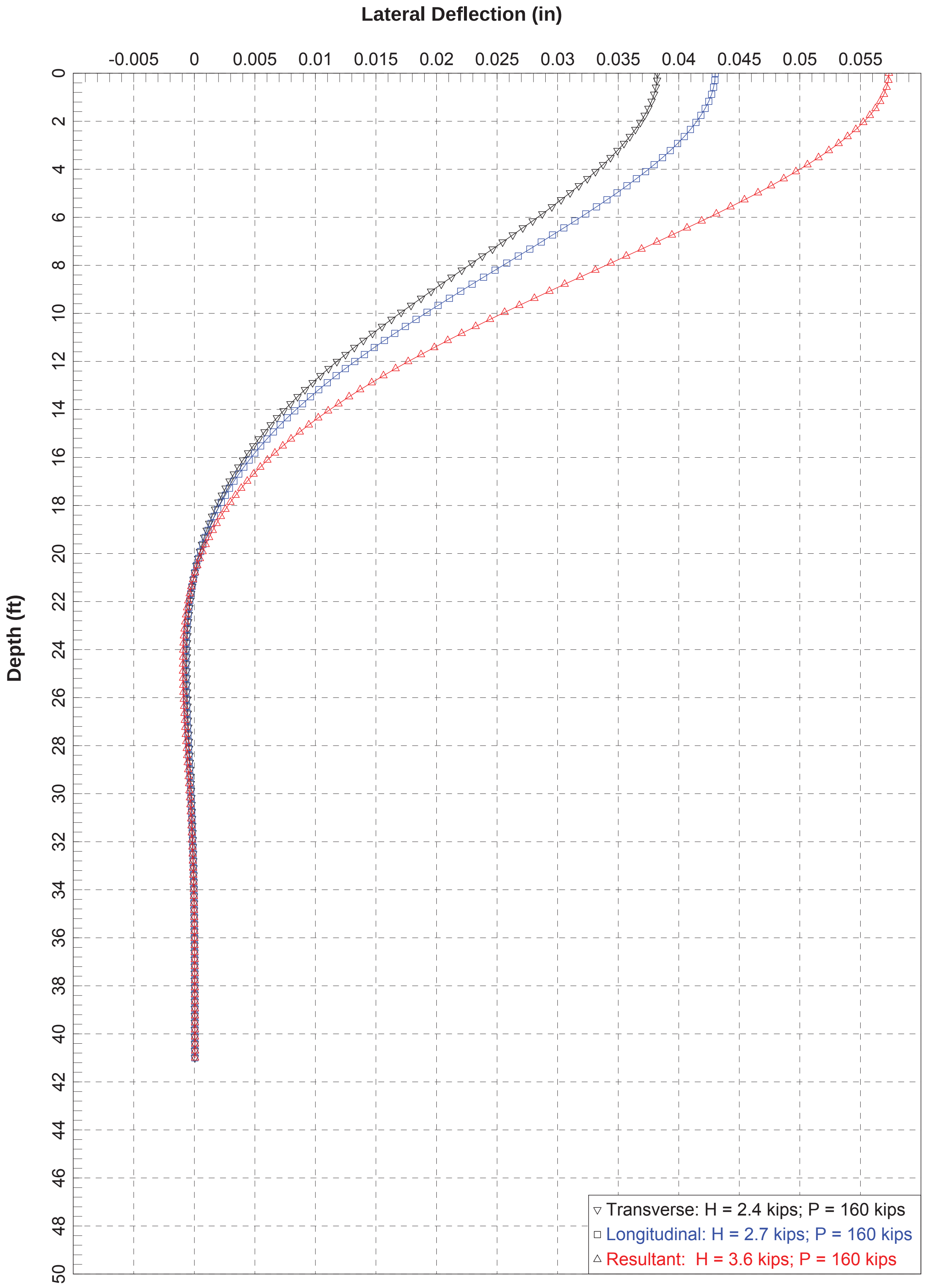


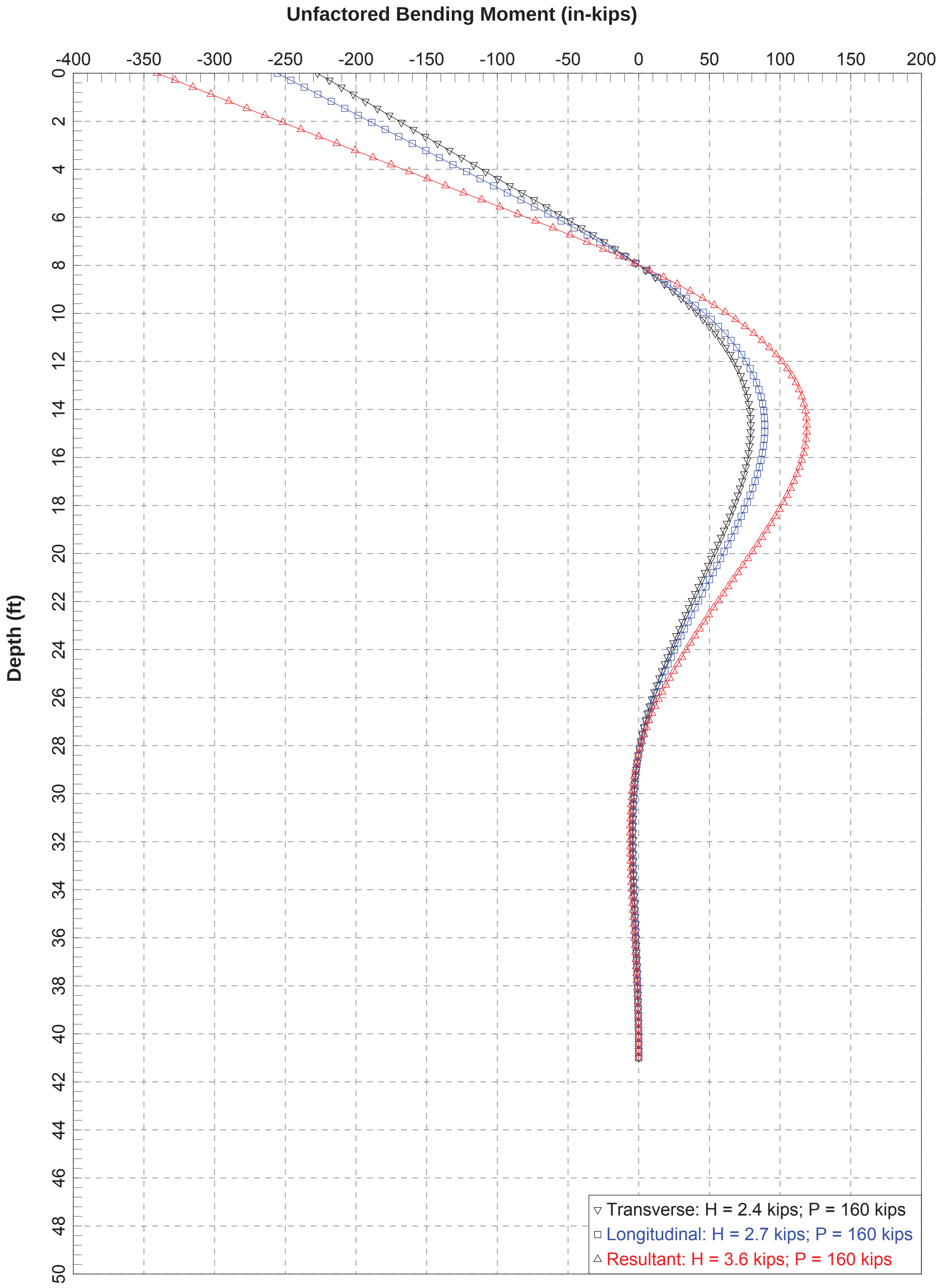
Summary of lateral loads:

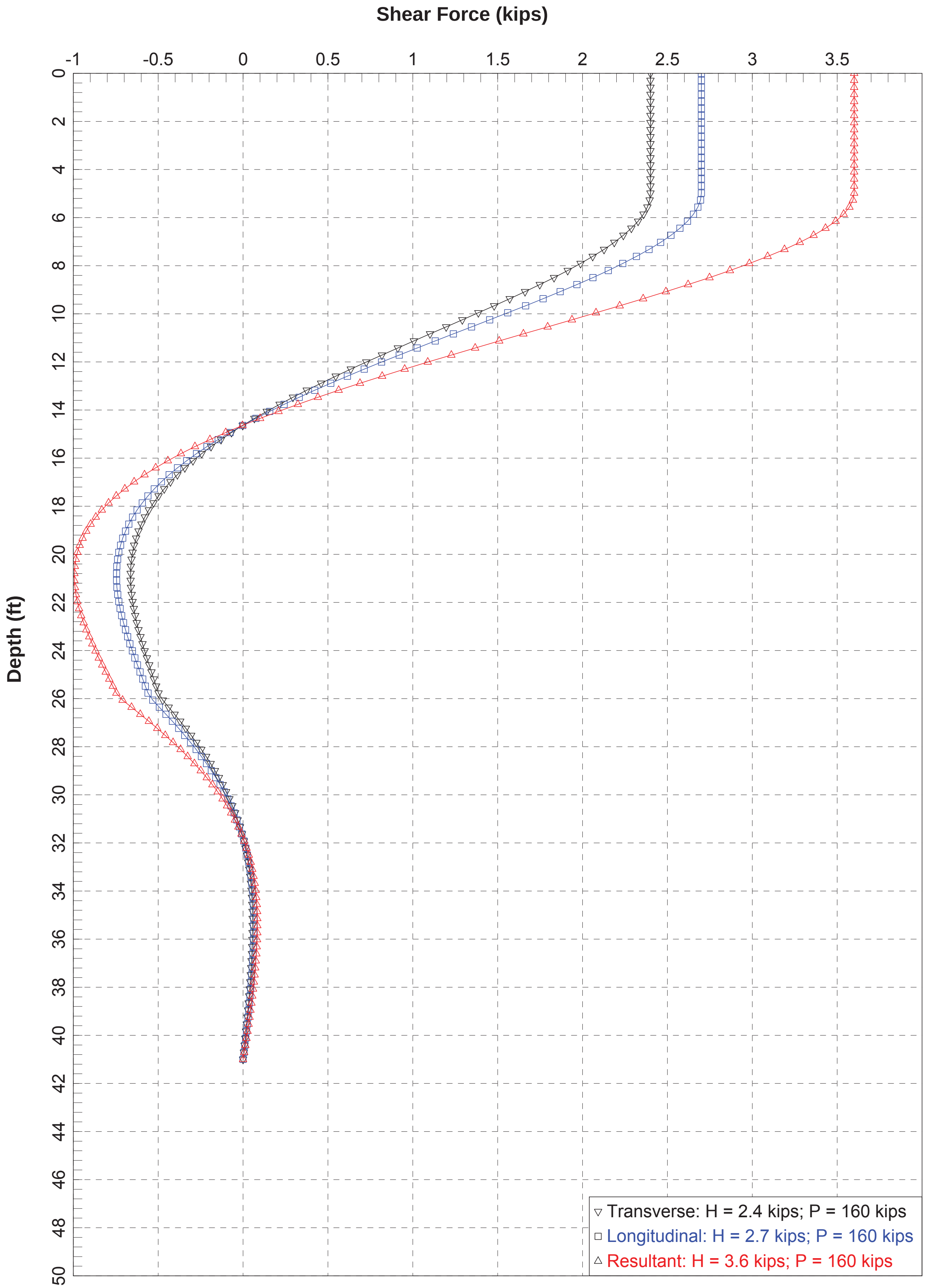
Load Case No.	Direction	H, kips	P, kips
1	Transverse	2.4	160
2	Longitudinal	2.7	160
3	Resultant	3.6	160

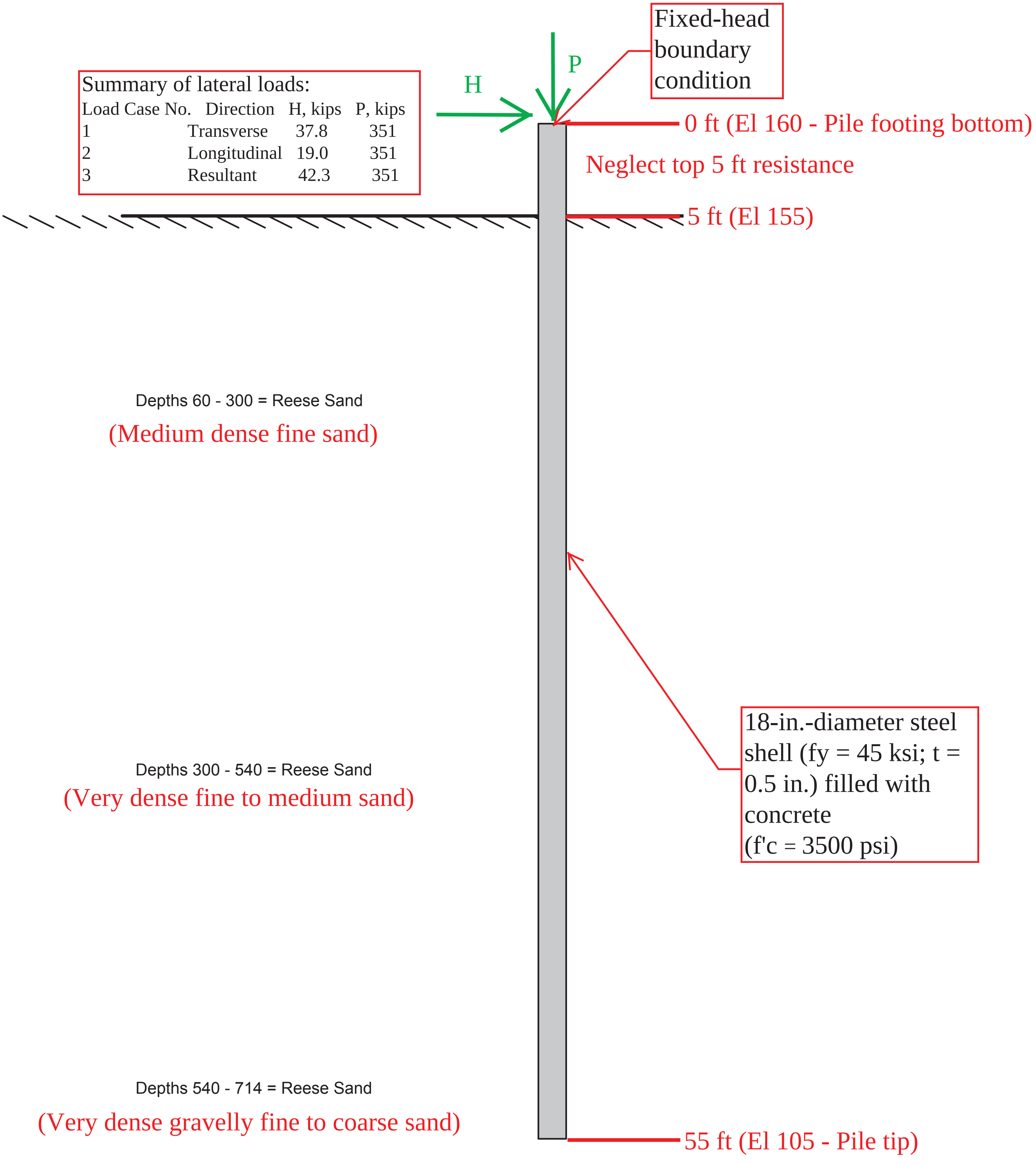
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 3
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05
0.055
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips


Unfactored Bending Moment (in-kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips


Shear Force (kips)
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips

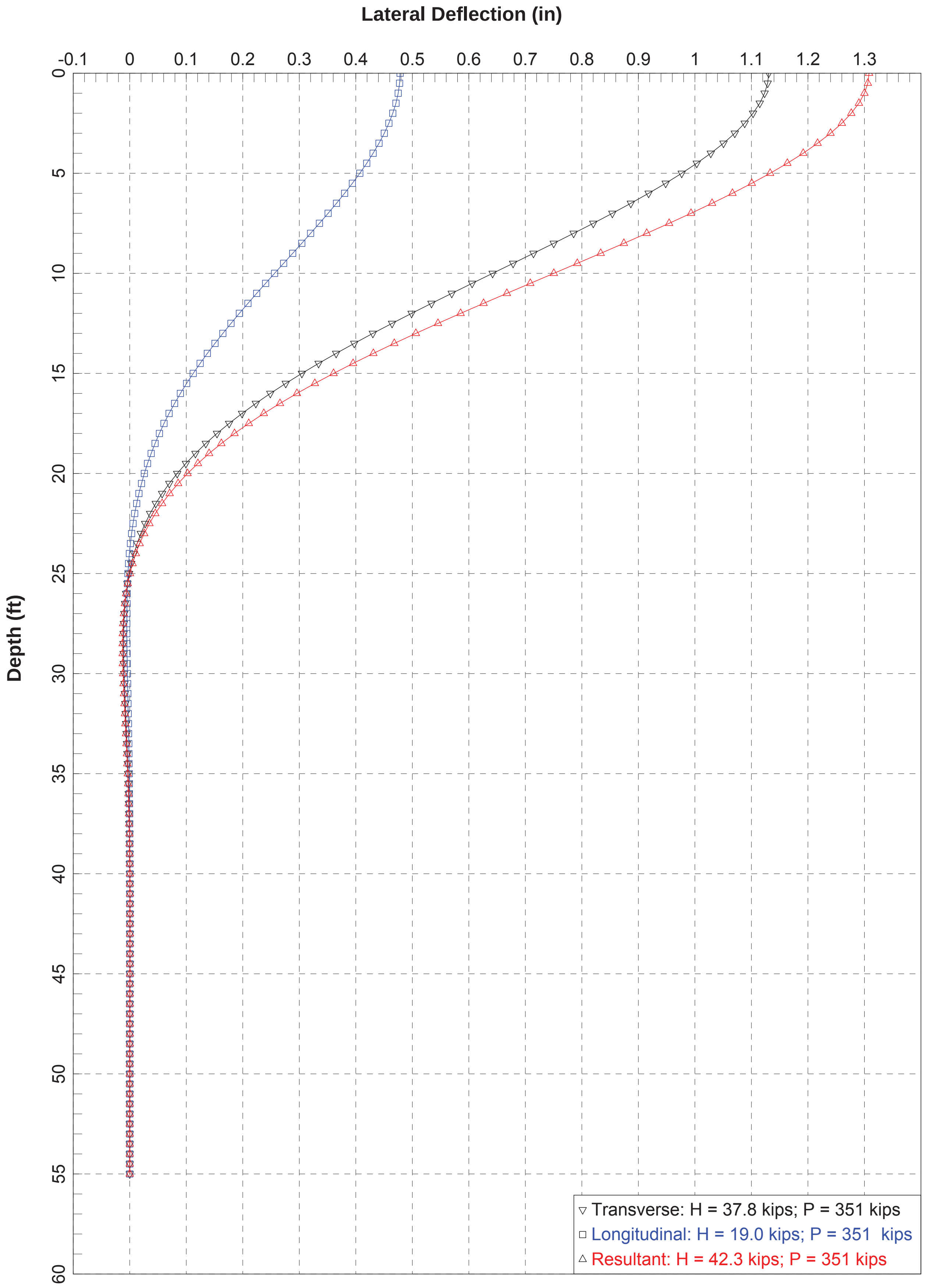


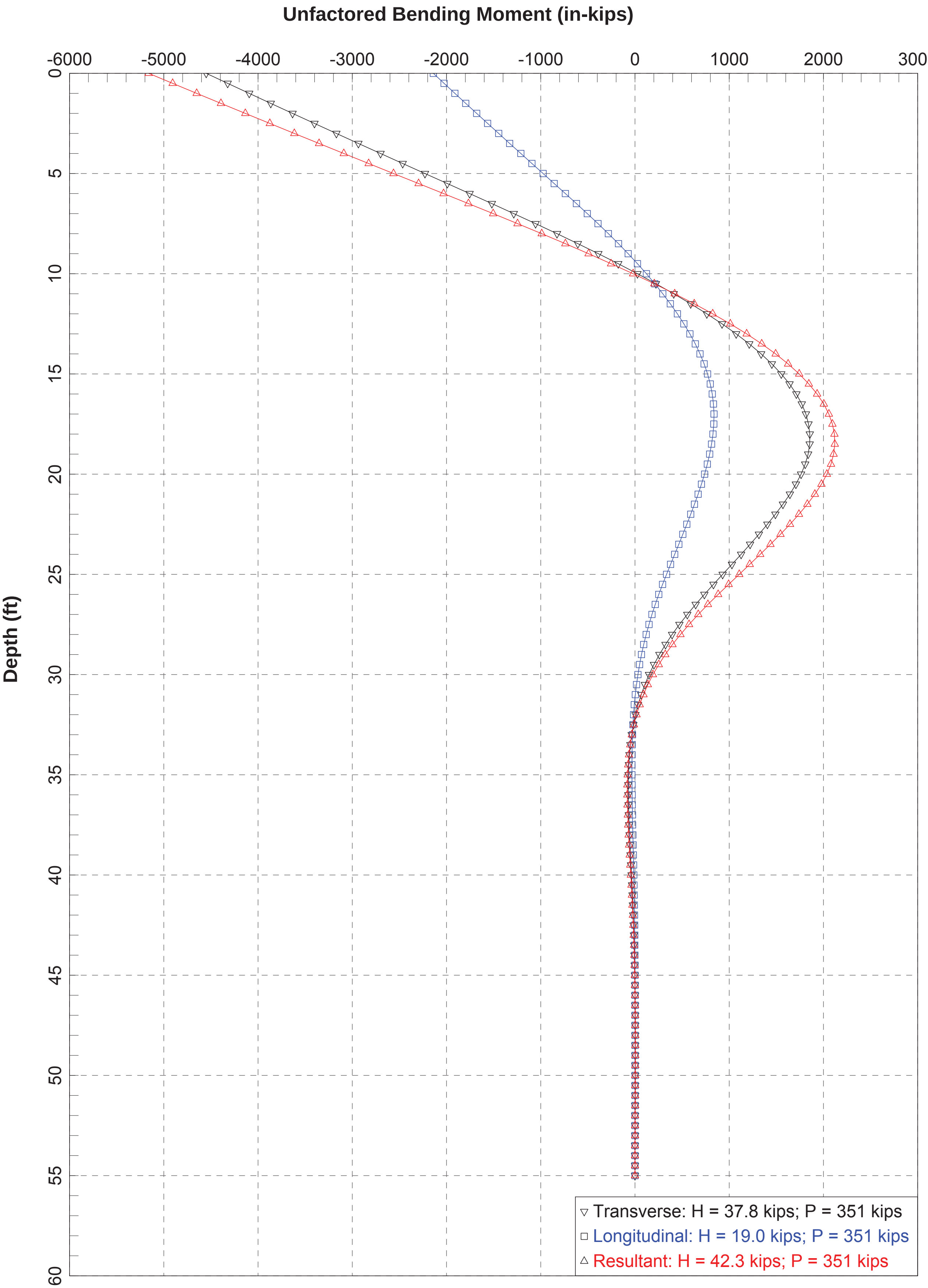
Summary of lateral loads:

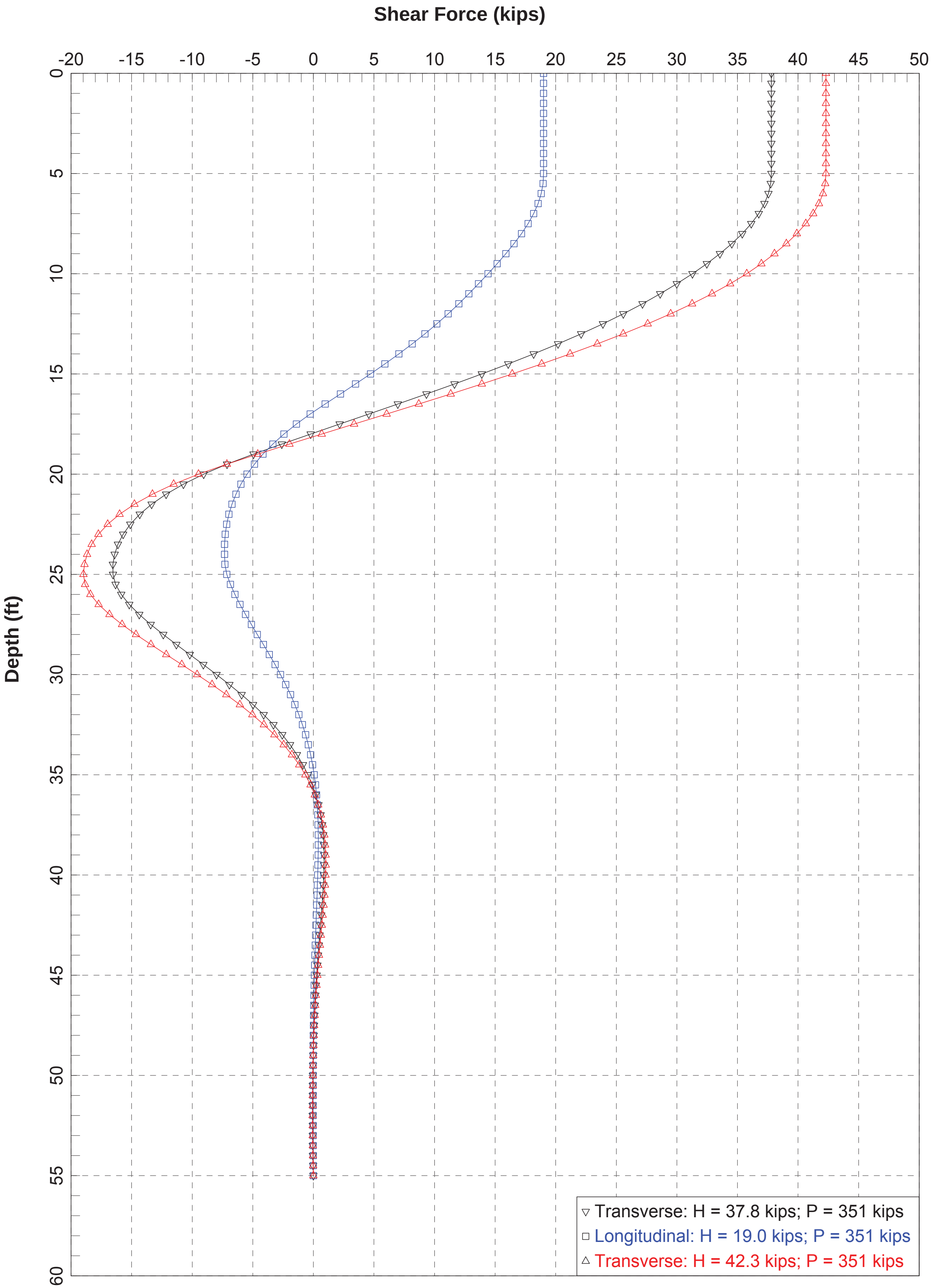
Load Case No.	Direction	H, kips	P, kips
1	Transverse	37.8	351
2	Longitudinal	19.0	351
3	Resultant	42.3	351

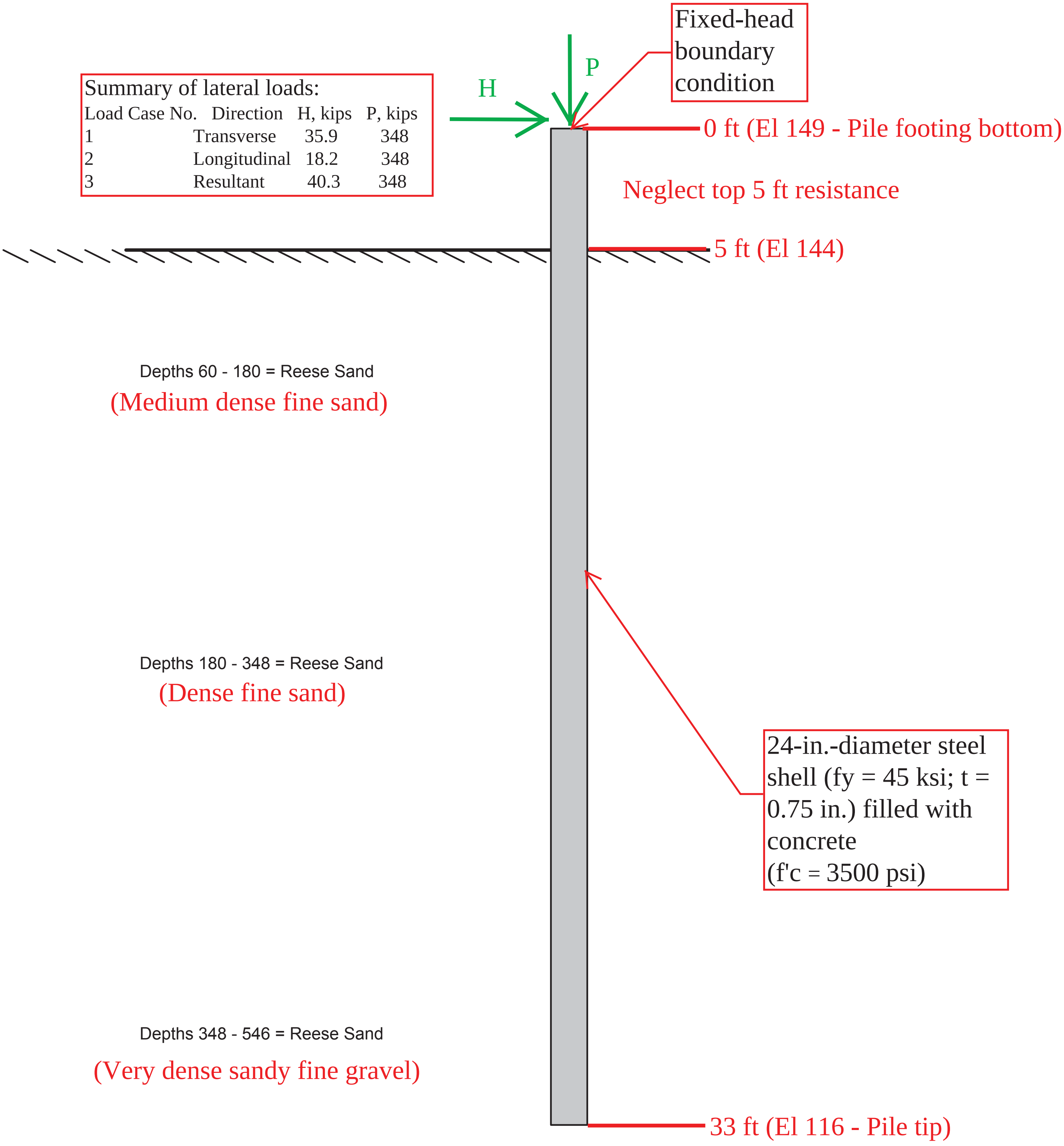
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 8
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Resultant: H = 42.3 kips; P = 351 kips


Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Resultant: H = 42.3 kips; P = 351 kips

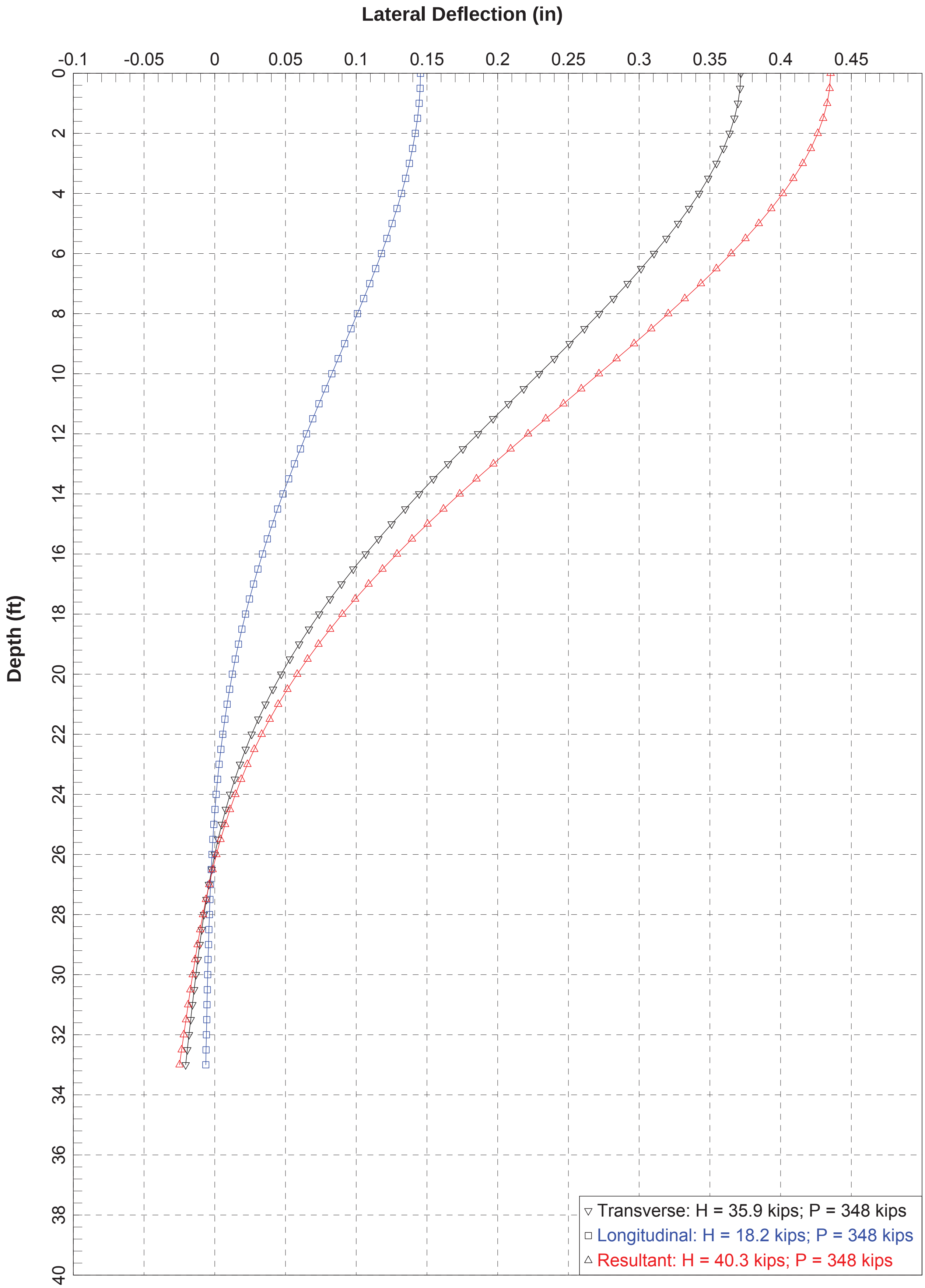

Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Transverse: H = 42.3 kips; P = 351 kips

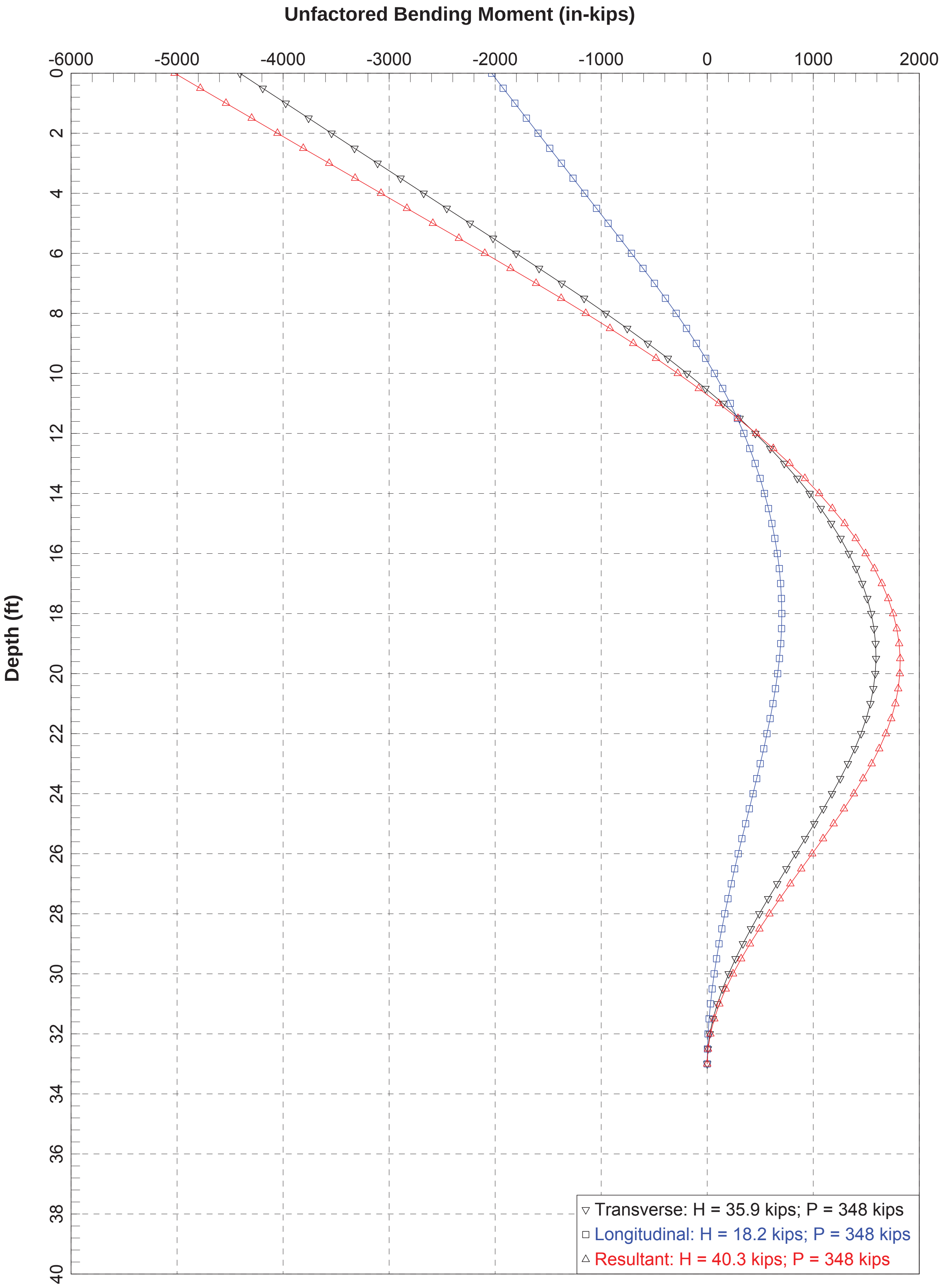

Summary of lateral loads:

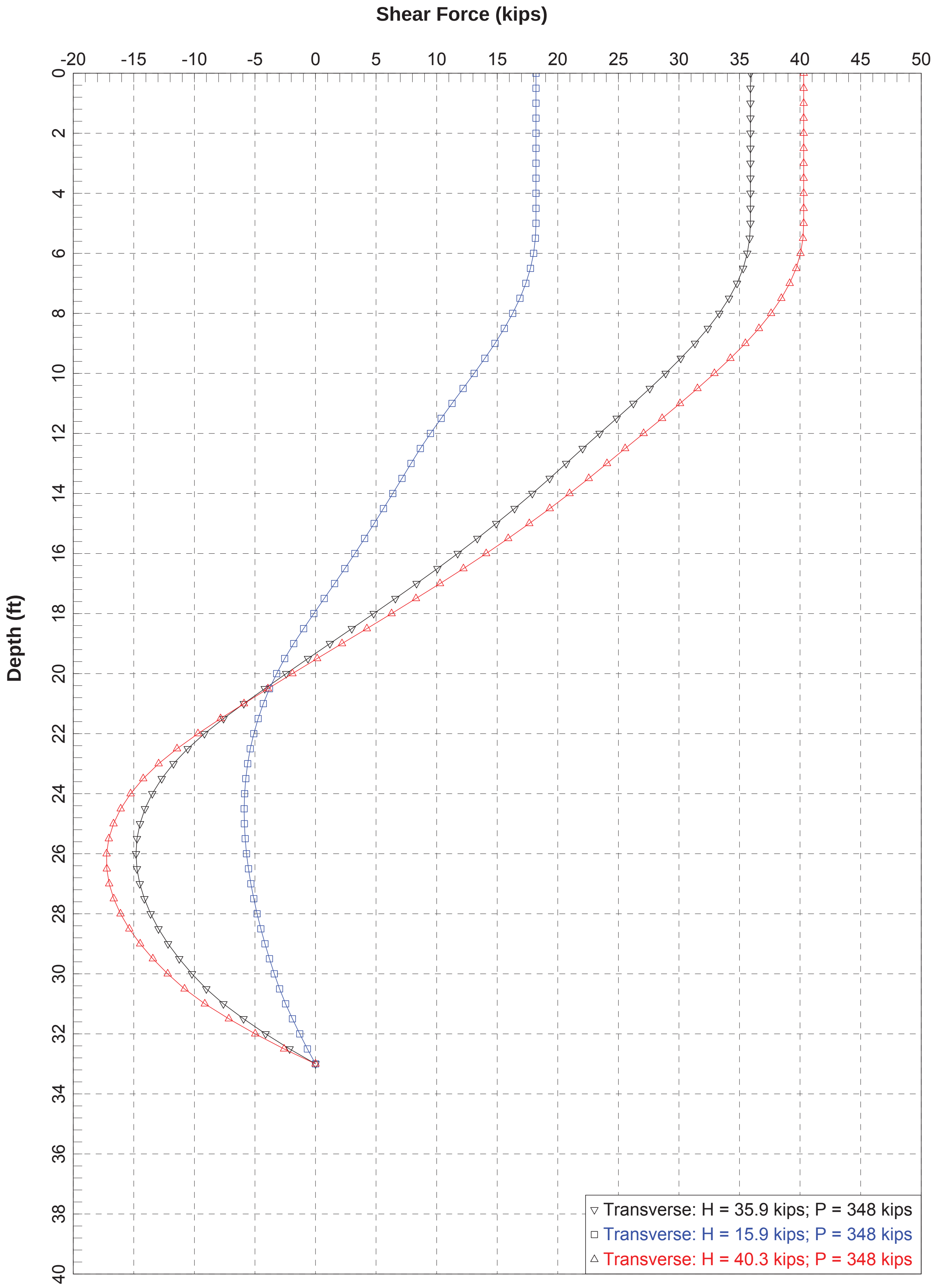
Load Case No.	Direction	H, kips	P, kips
1	Transverse	35.9	348
2	Longitudinal	18.2	348
3	Resultant	40.3	348

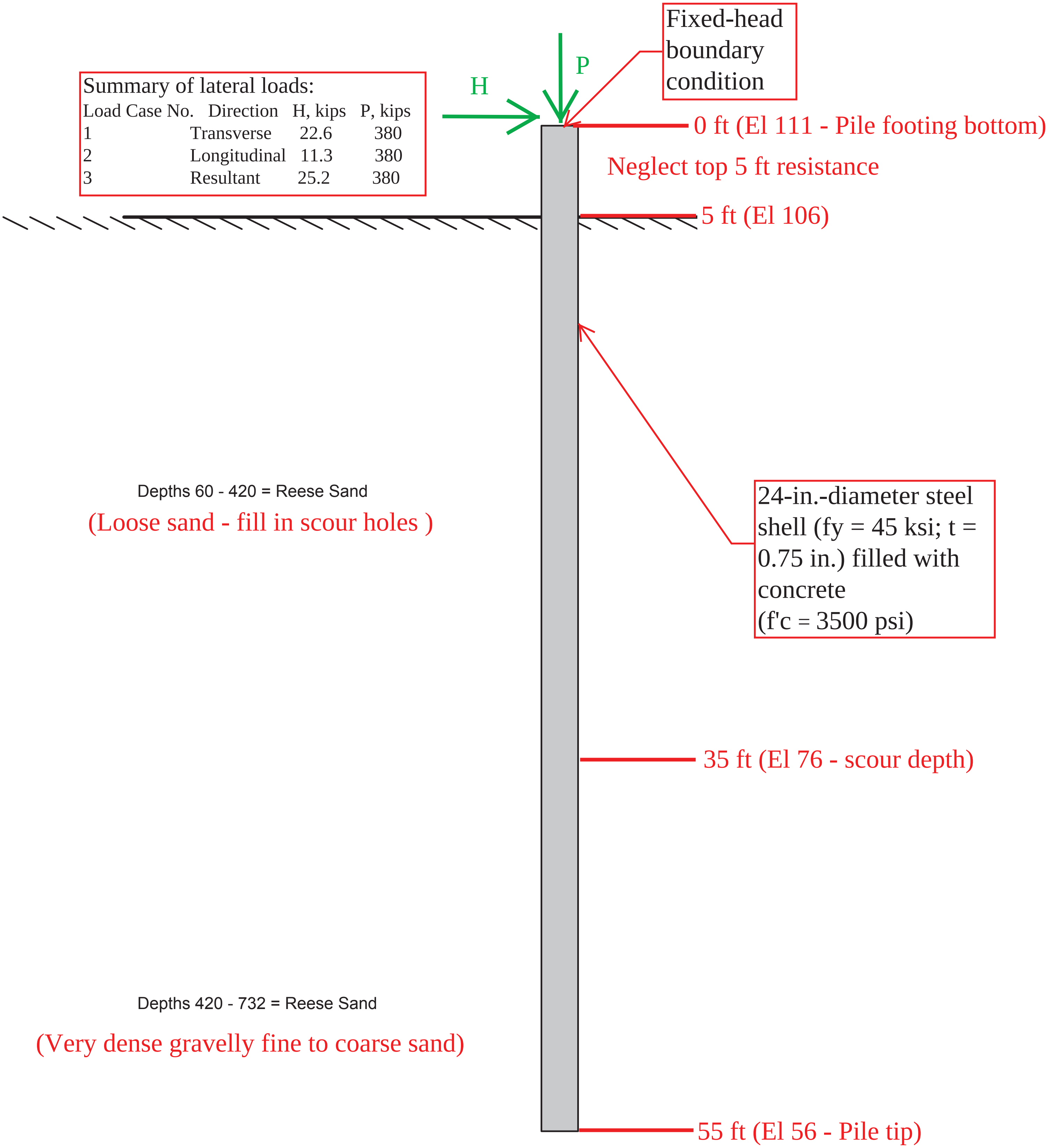
Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 9
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Longitudinal: H = 18.2 kips; P = 348 kips
△ Resultant: H = 40.3 kips; P = 348 kips


Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Longitudinal: H = 18.2 kips; P = 348 kips
△ Resultant: H = 40.3 kips; P = 348 kips


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Transverse: H = 15.9 kips; P = 348 kips
△ Transverse: H = 40.3 kips; P = 348 kips

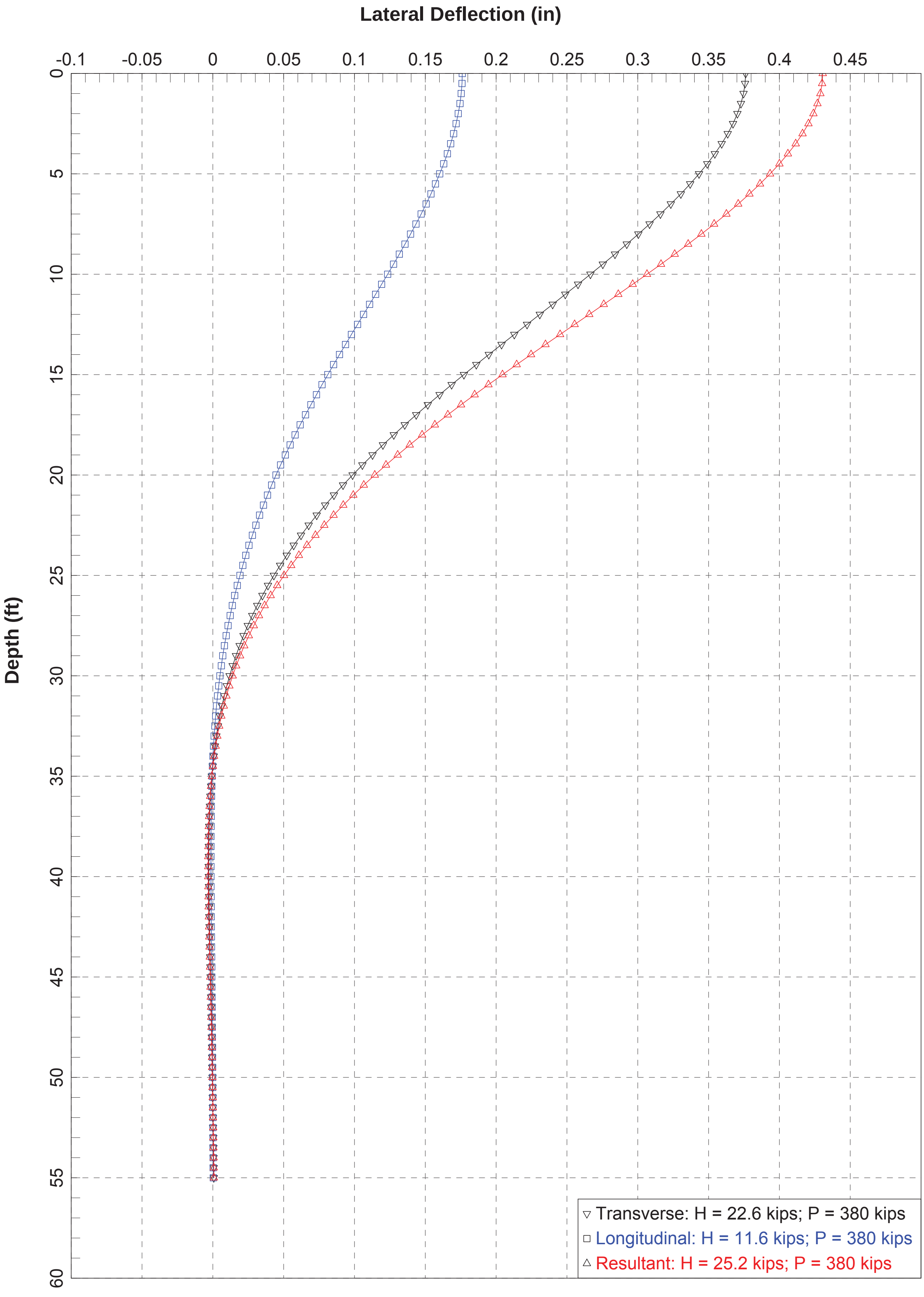


Summary of lateral loads:

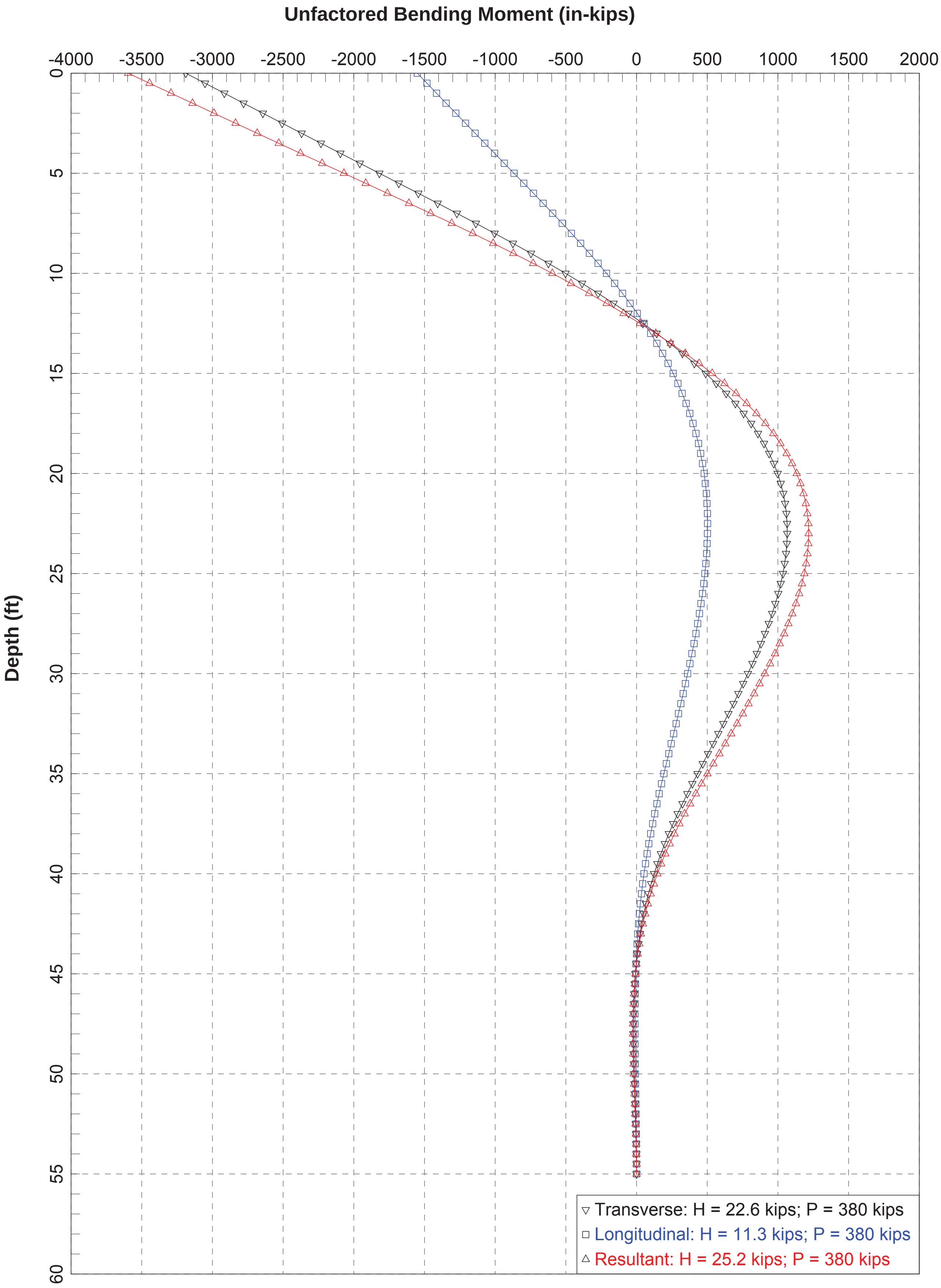
Load Case No.	Direction	H, kips	P, kips
1	Transverse	22.6	380
2	Longitudinal	11.3	380
3	Resultant	25.2	380

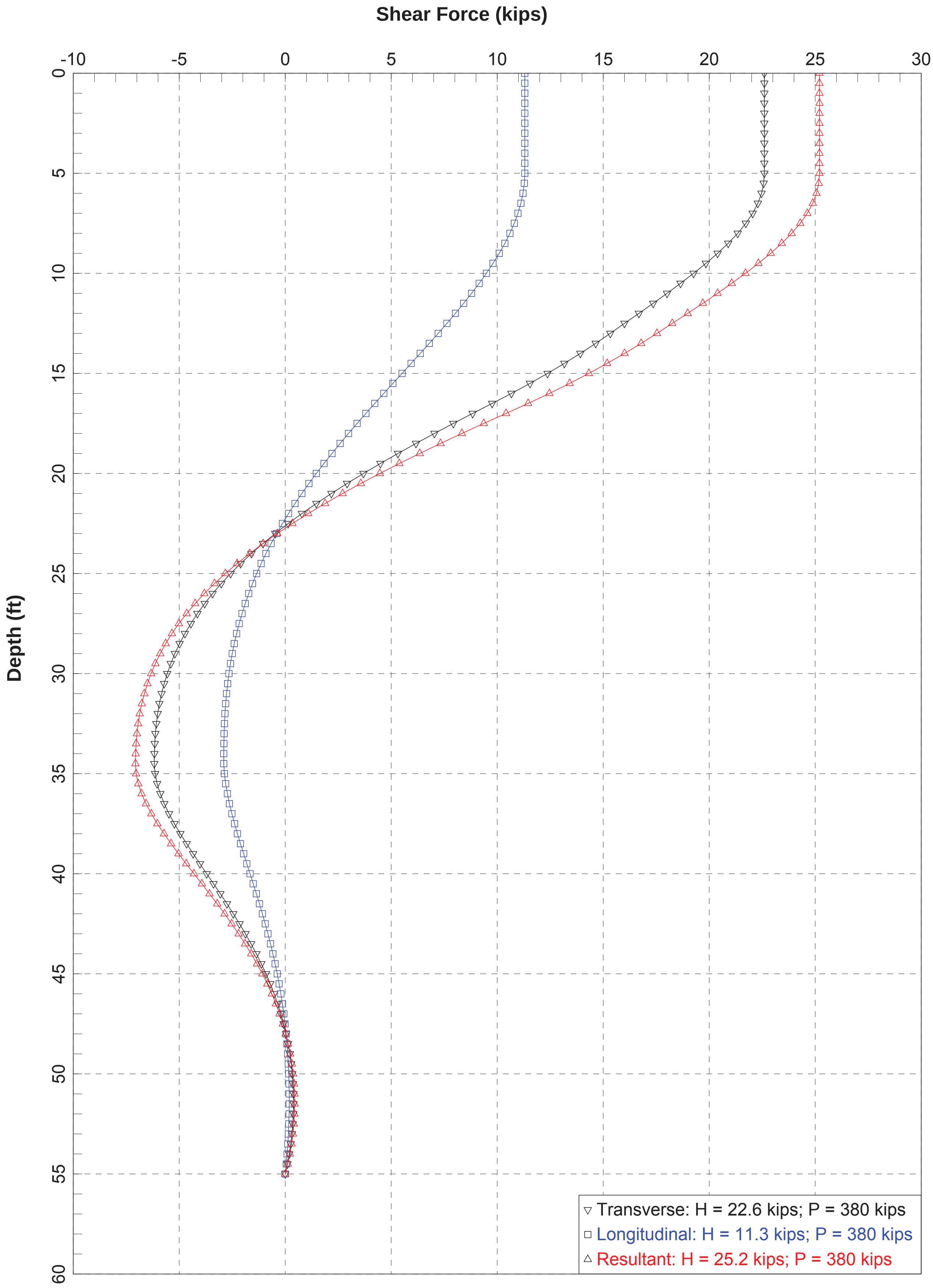
Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Pier 1
I-40 over White River

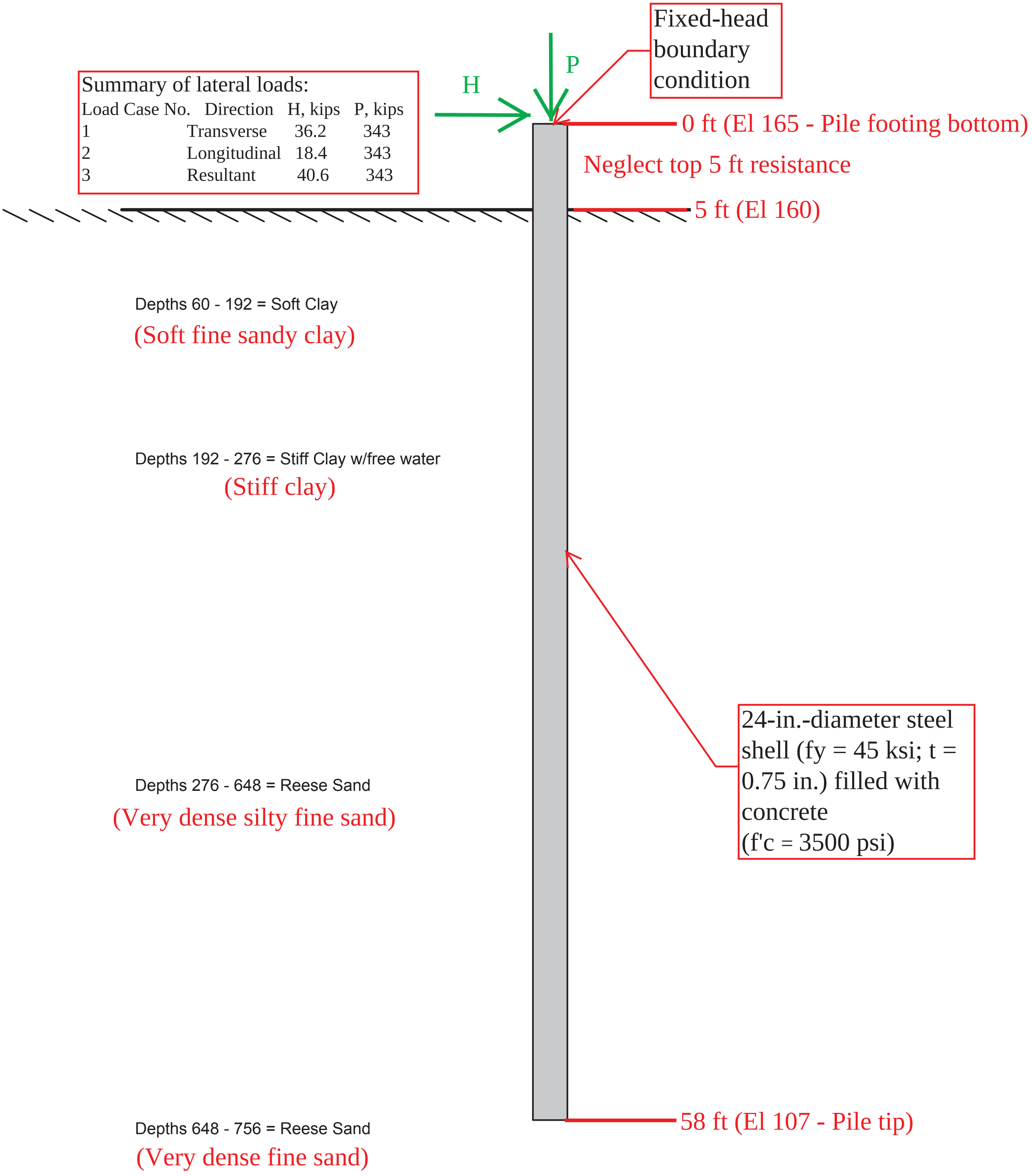
LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.



Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.6 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips


Unfactored Bending Moment (in-kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.3 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips


Shear Force (kips)
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.3 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips

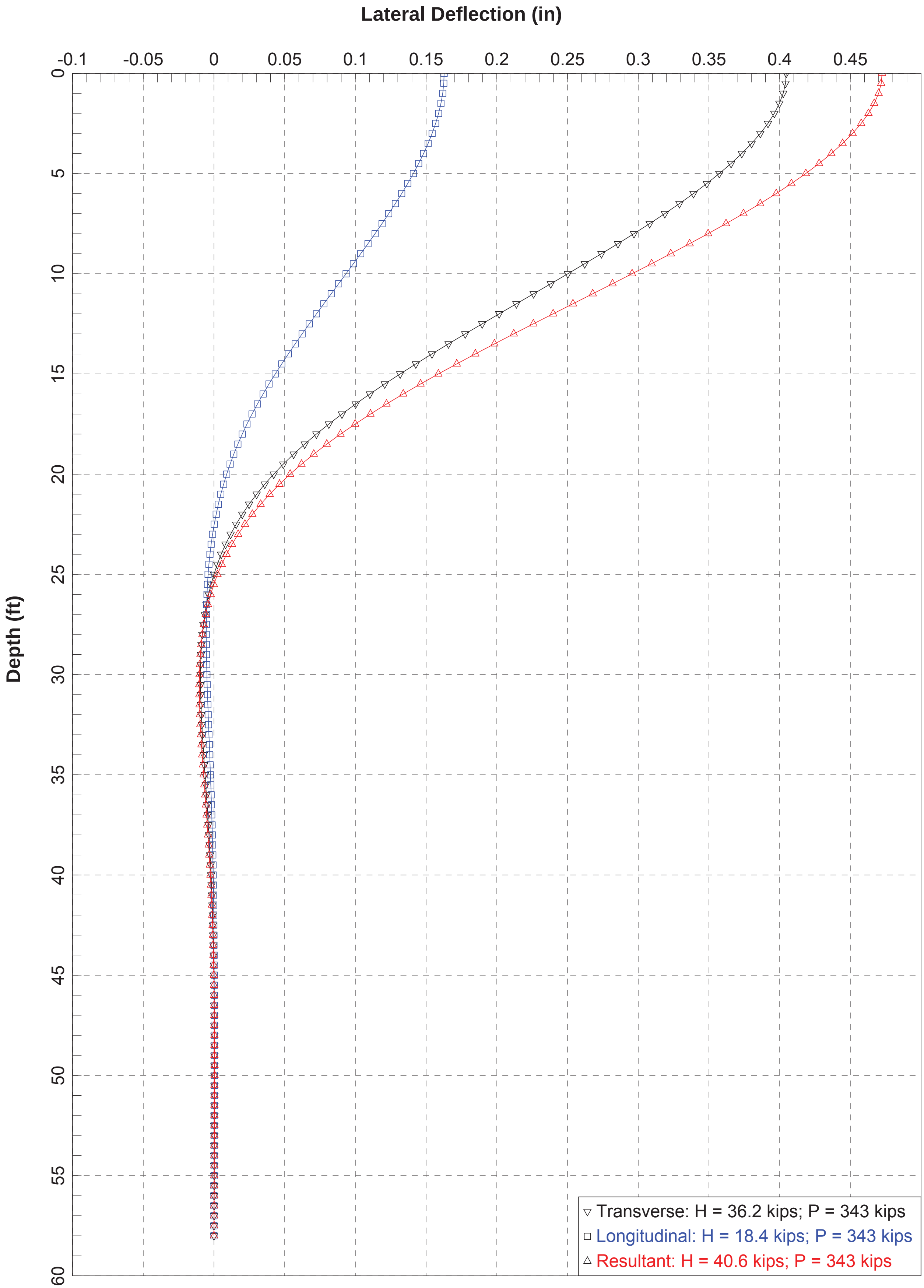


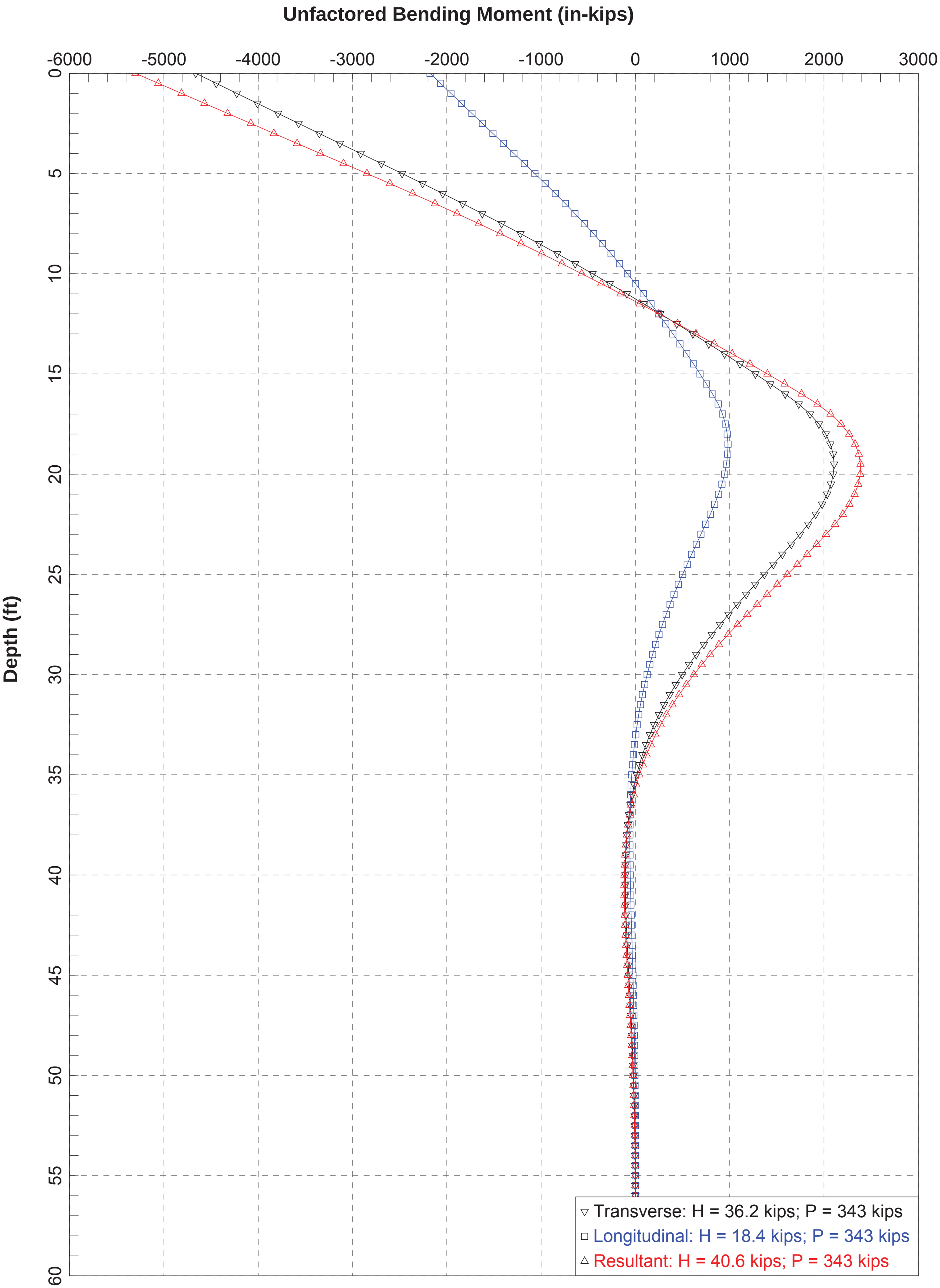
Summary of lateral loads:

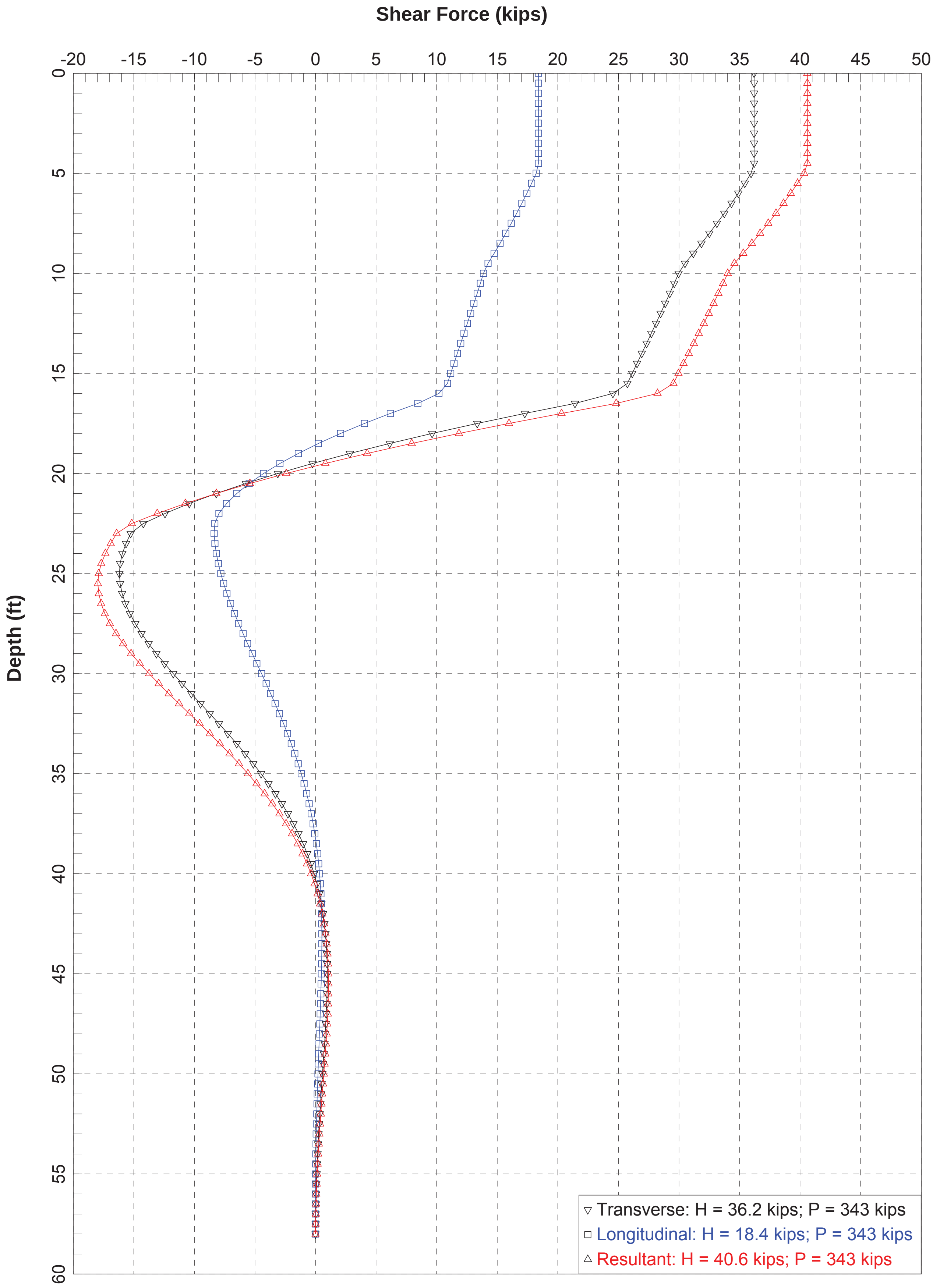
Load Case No.	Direction	H, kips	P, kips
1	Transverse	36.2	343
2	Longitudinal	18.4	343
3	Resultant	40.6	343

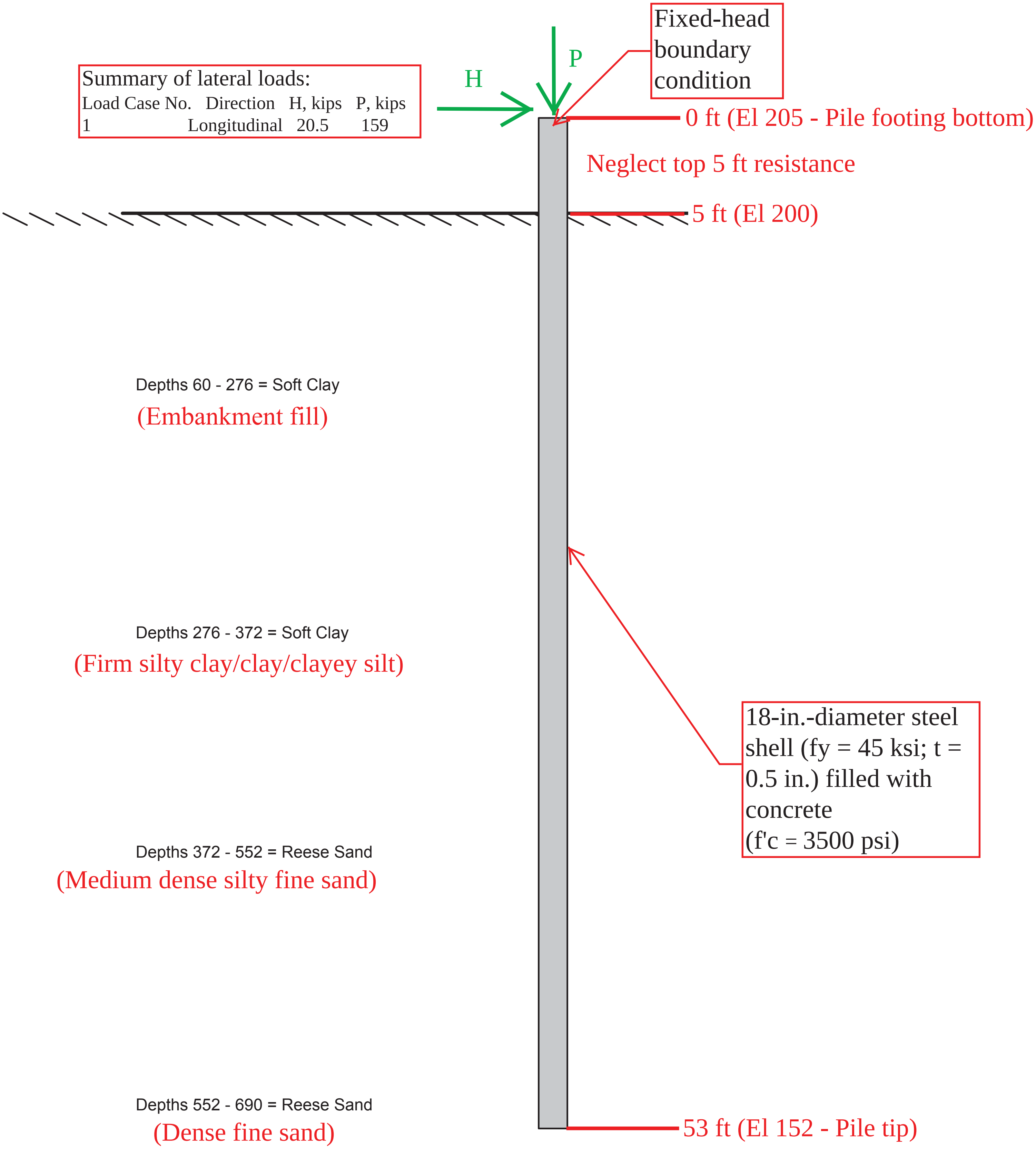
Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 10
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips

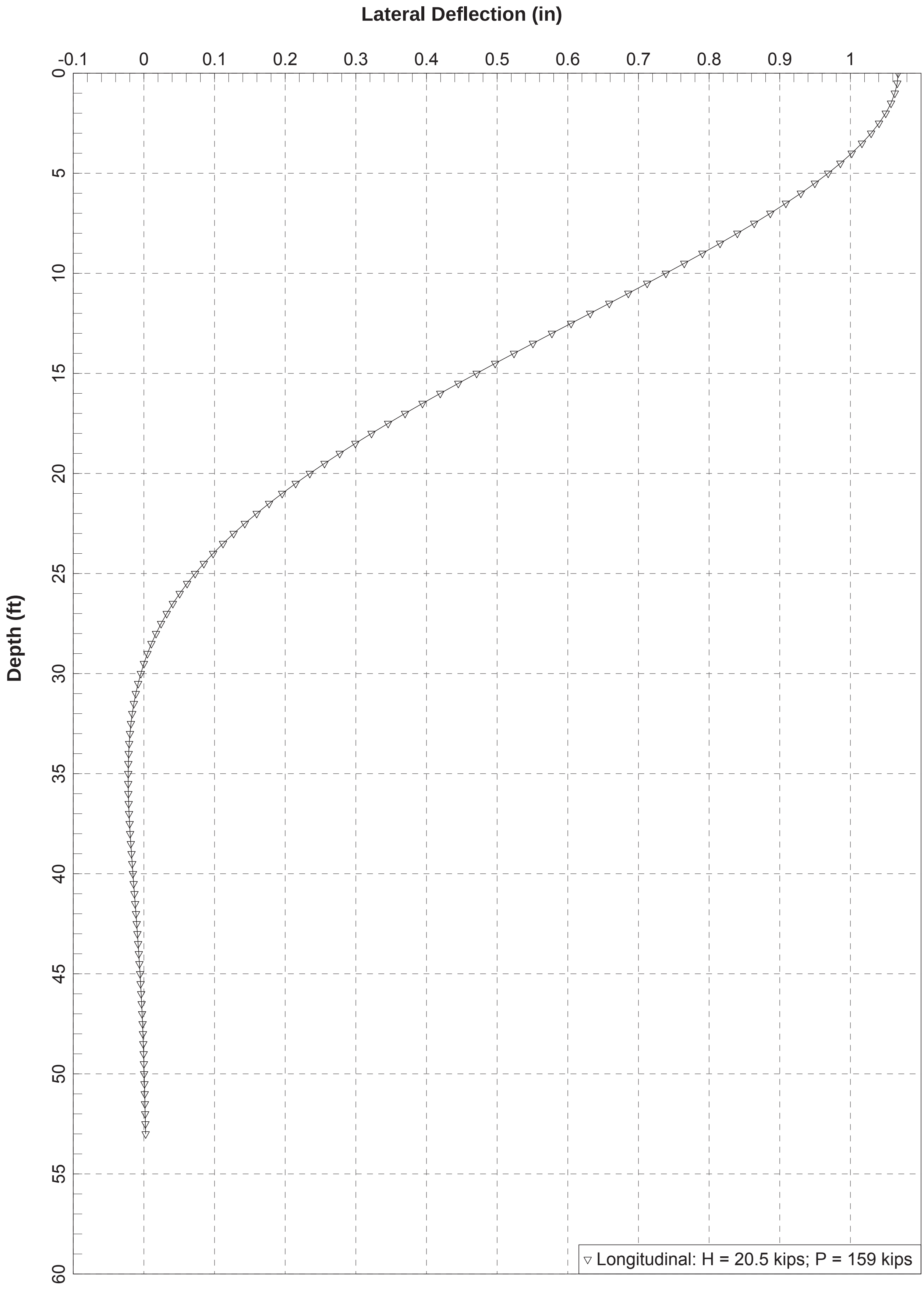

Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips

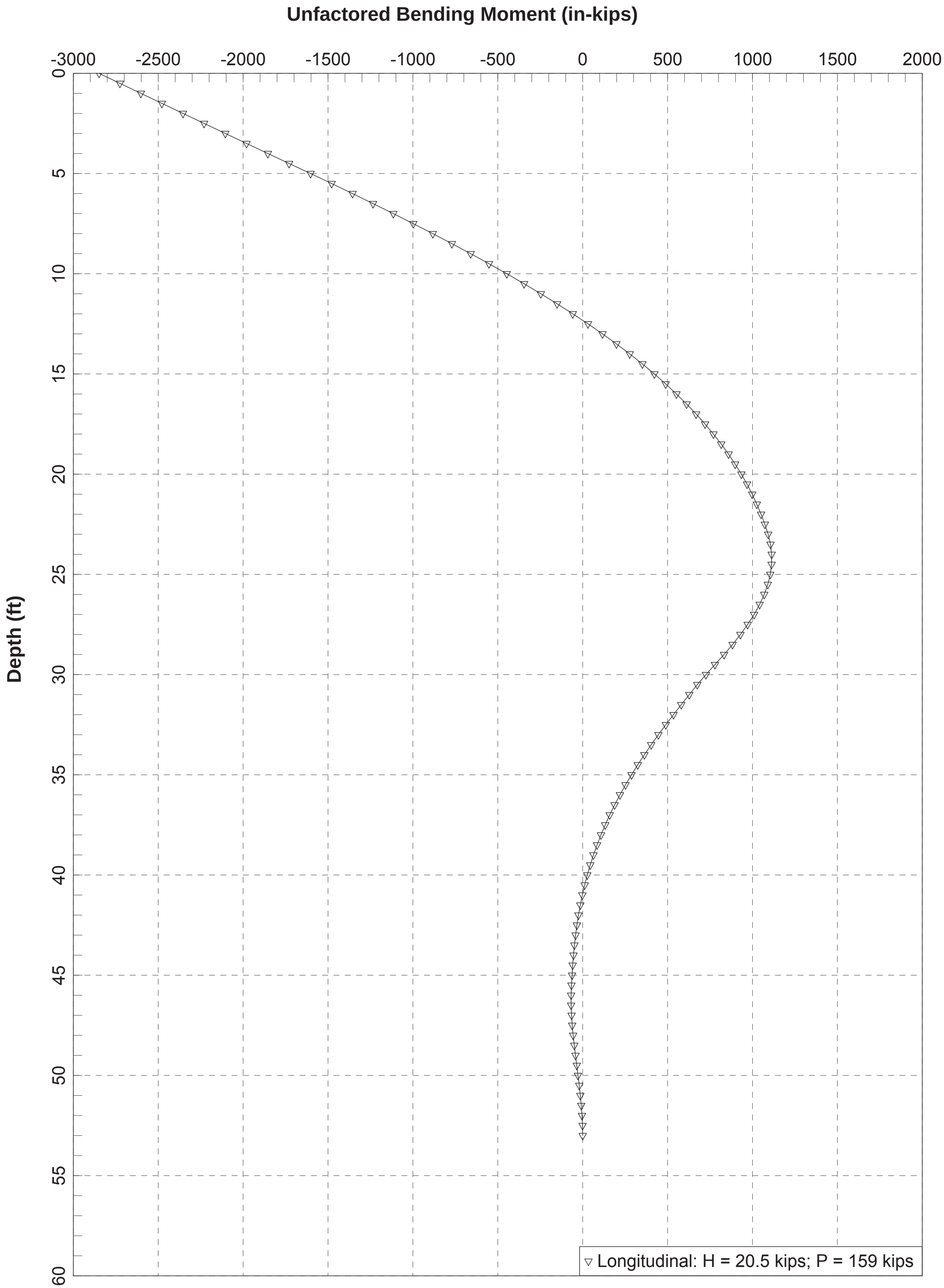

Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips

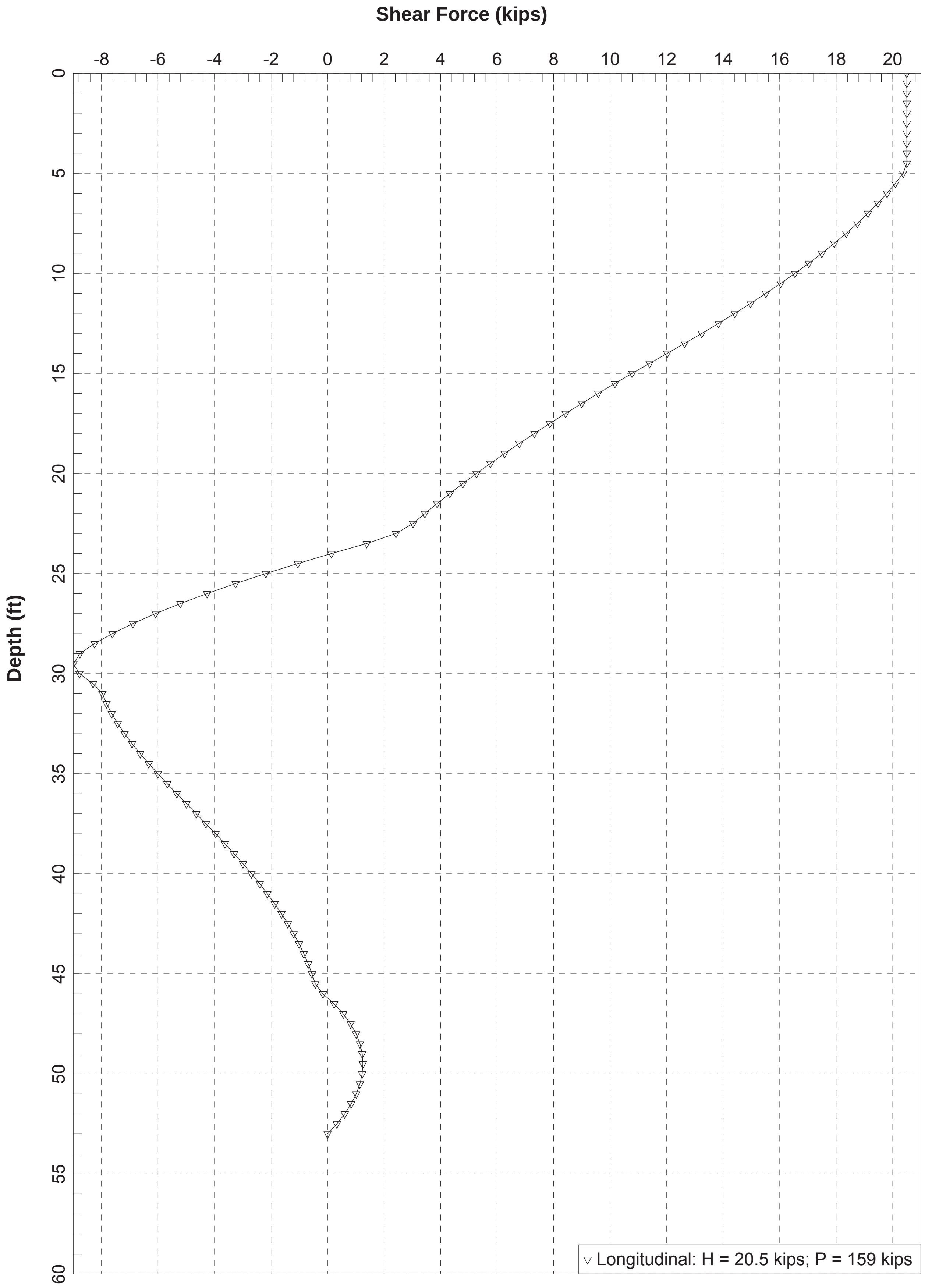


Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 17 (East Abutment)
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips


Unfactored Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips

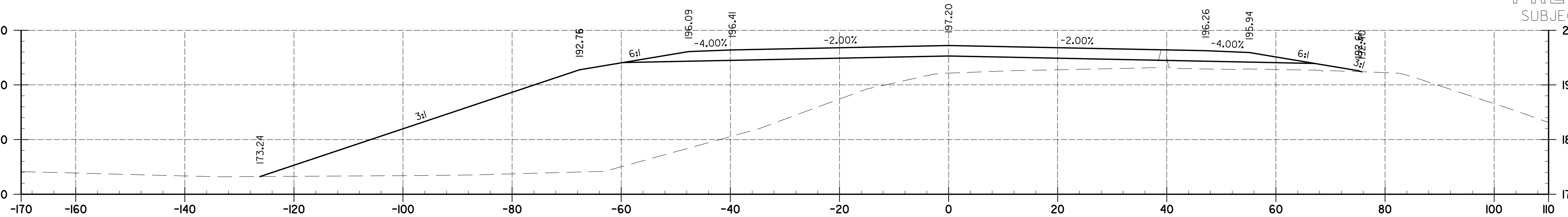

Shear Force (kips)
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
20
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips

APPENDIX I

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.		BB0610		–

②

PRELIMINARY
SUBJECT TO REVISION



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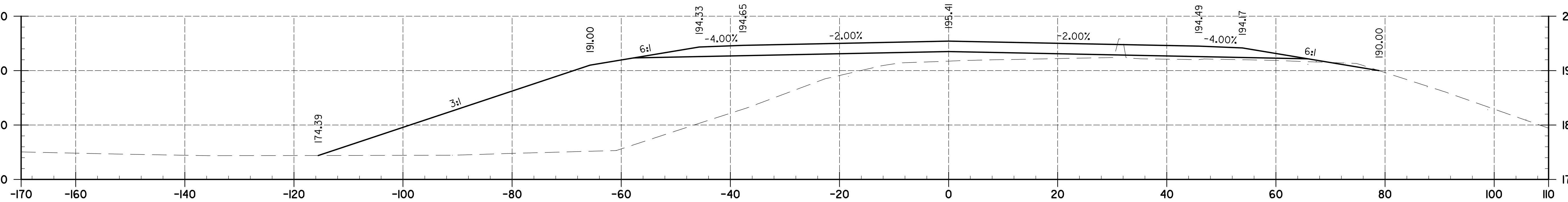
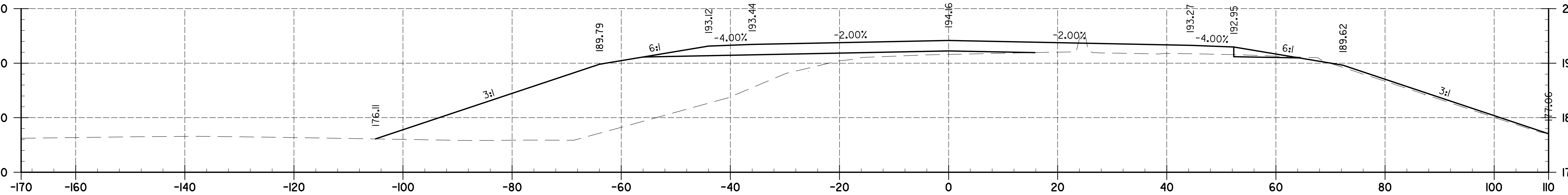


1877+00



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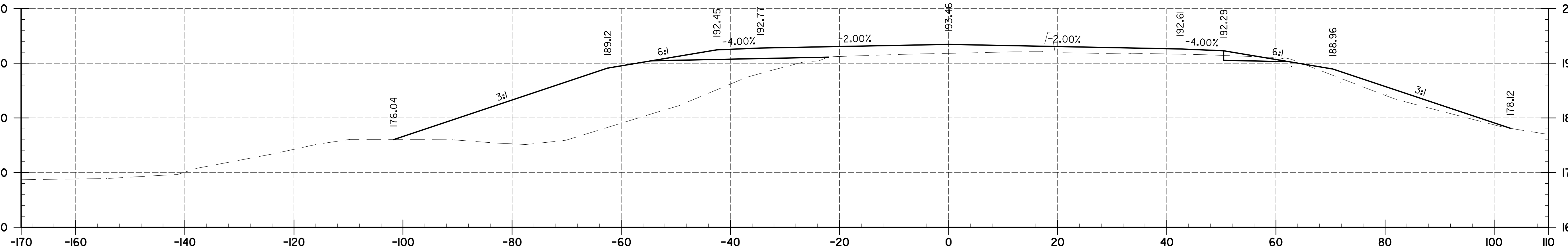

1875+00

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...\WLXM2620-Cross-Section1.dgn

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4/25/2014

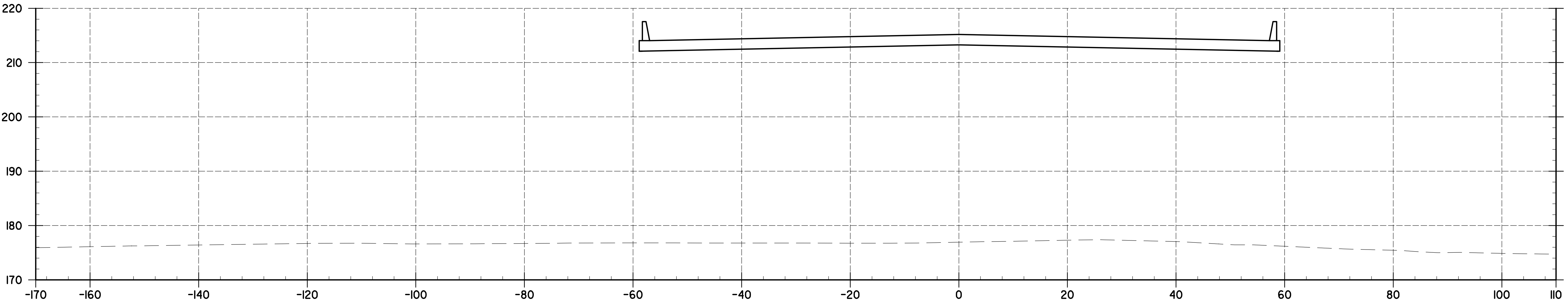
JACOBS

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED

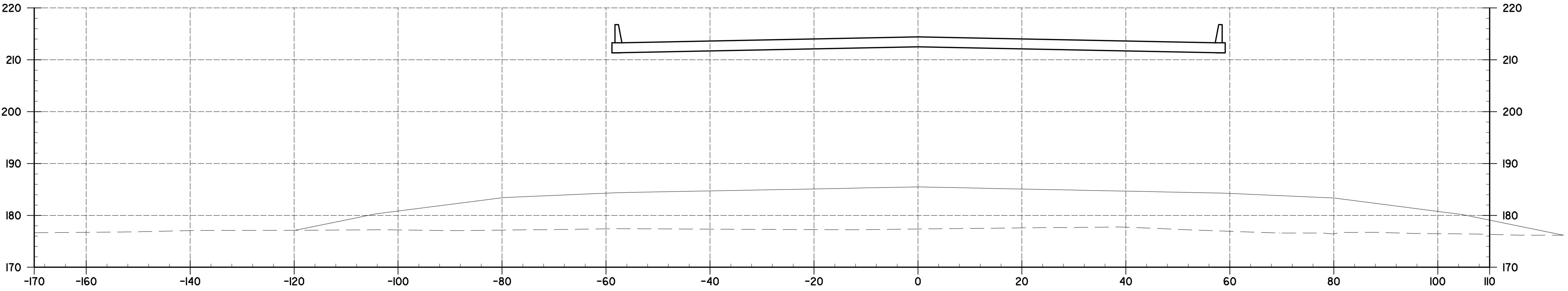
FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
6	ARK.			
JOB NO.		BB0610		_

②

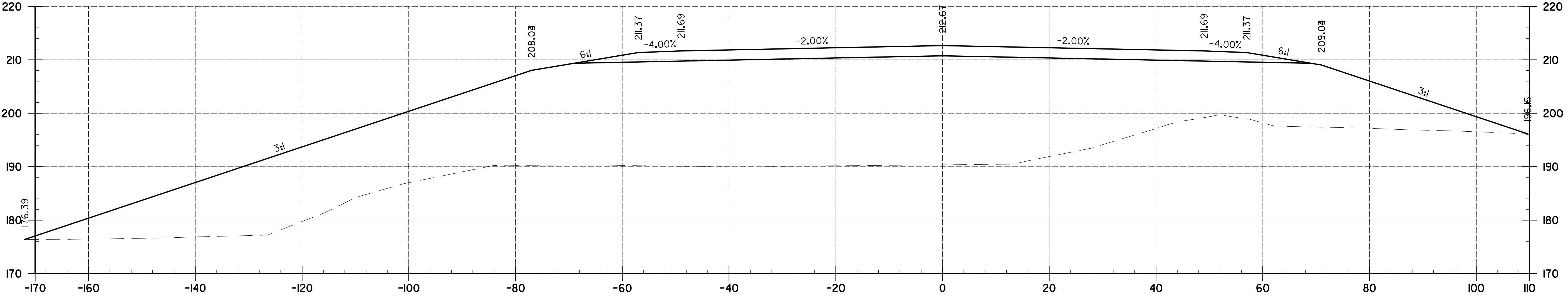
PRELIMINARY
SUBJECT TO REVISION



1884+25.60 - TOE OF FILL SLOPE



1884+00



1883+41.92 - BEGIN BRIDGE

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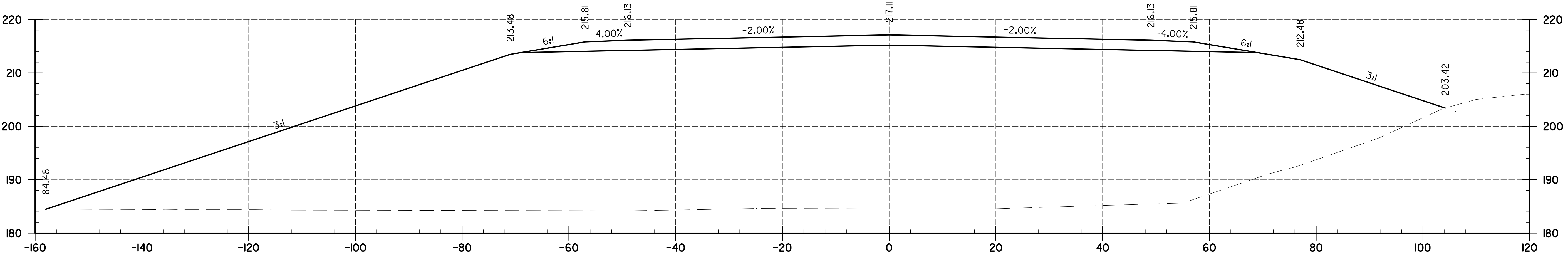
4/25/2014

JACOBS

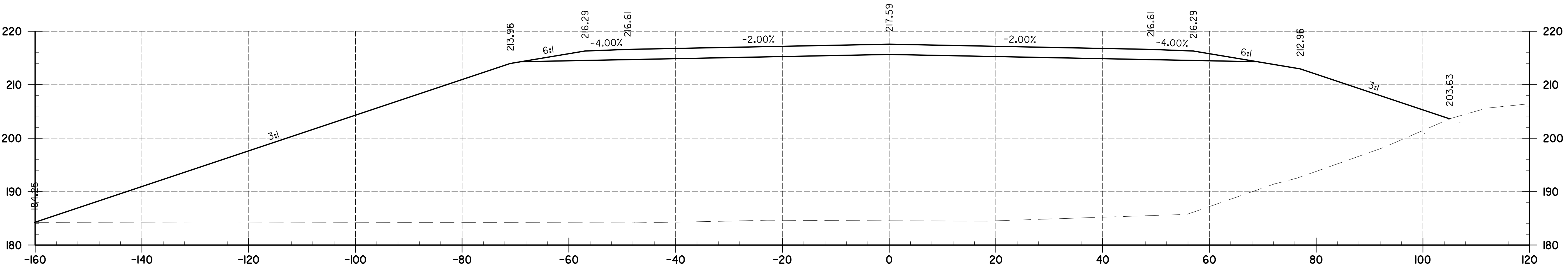
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610		_

②

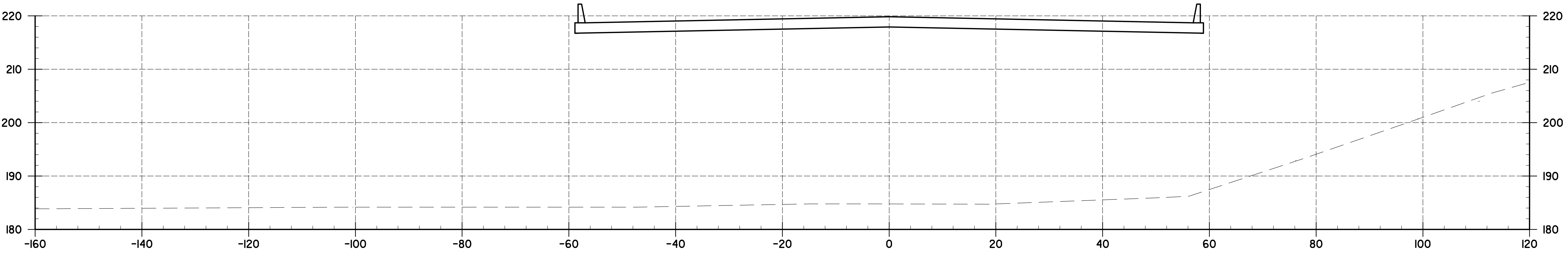
PRELIMINARY
SUBJECT TO REVISION



1912+00



1911+84.08 - END BRIDGE



1911+09.10 - TOE OF FILL SLOPE

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4/25/2014

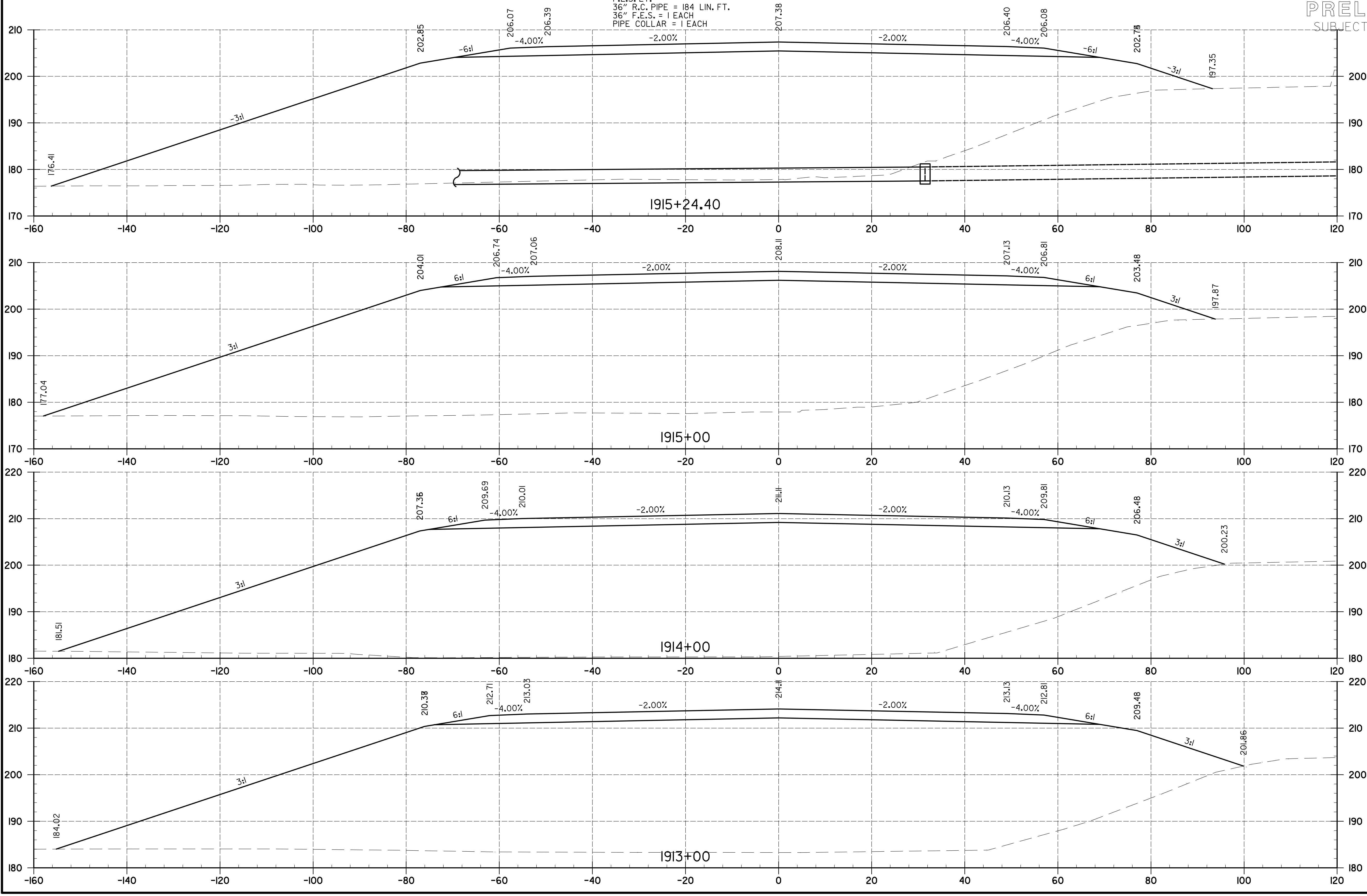
JACOBS

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.		BB0610		–

②

PRELIMINARY
SUBJECT TO REVISION

STA. 1915+24.4 IN PLACE
36"x171' R.C. PIPE CULVERT
WITH HDWLS. LT. & RT.
REMOVE HDWL. LT. AND EXTEND
R.C. PIPE 180' LT. ON 10°50' LT. FWD. SKEW
(CLASS III)(TYPE 3 BEDDING) WITH
F.E.S. LT.
36" R.C. PIPE = 184 LIN. FT.
36" F.E.S. = 1 EACH
PIPE COLLAR = 1 EACH



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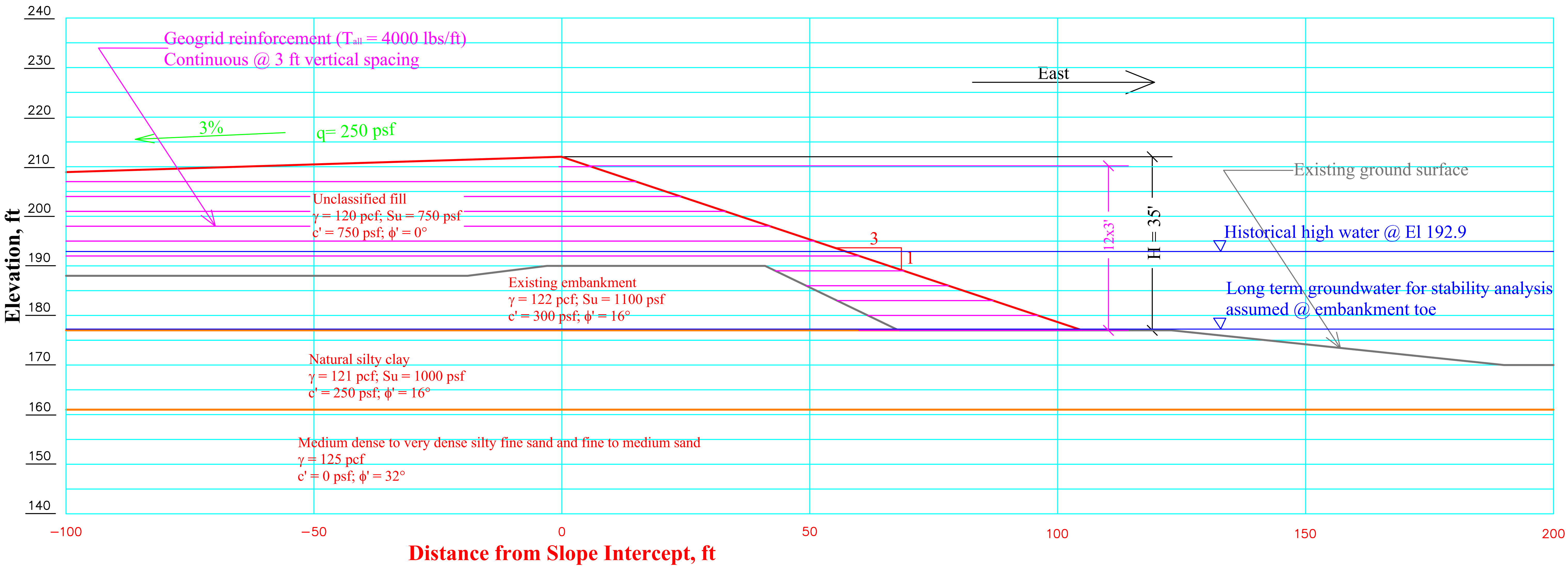
4/25/2014

JACOBS

APPENDIX J



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

Section and Material Parameters for Stability Analysis
Reinforced End Slope @ West Abutment
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

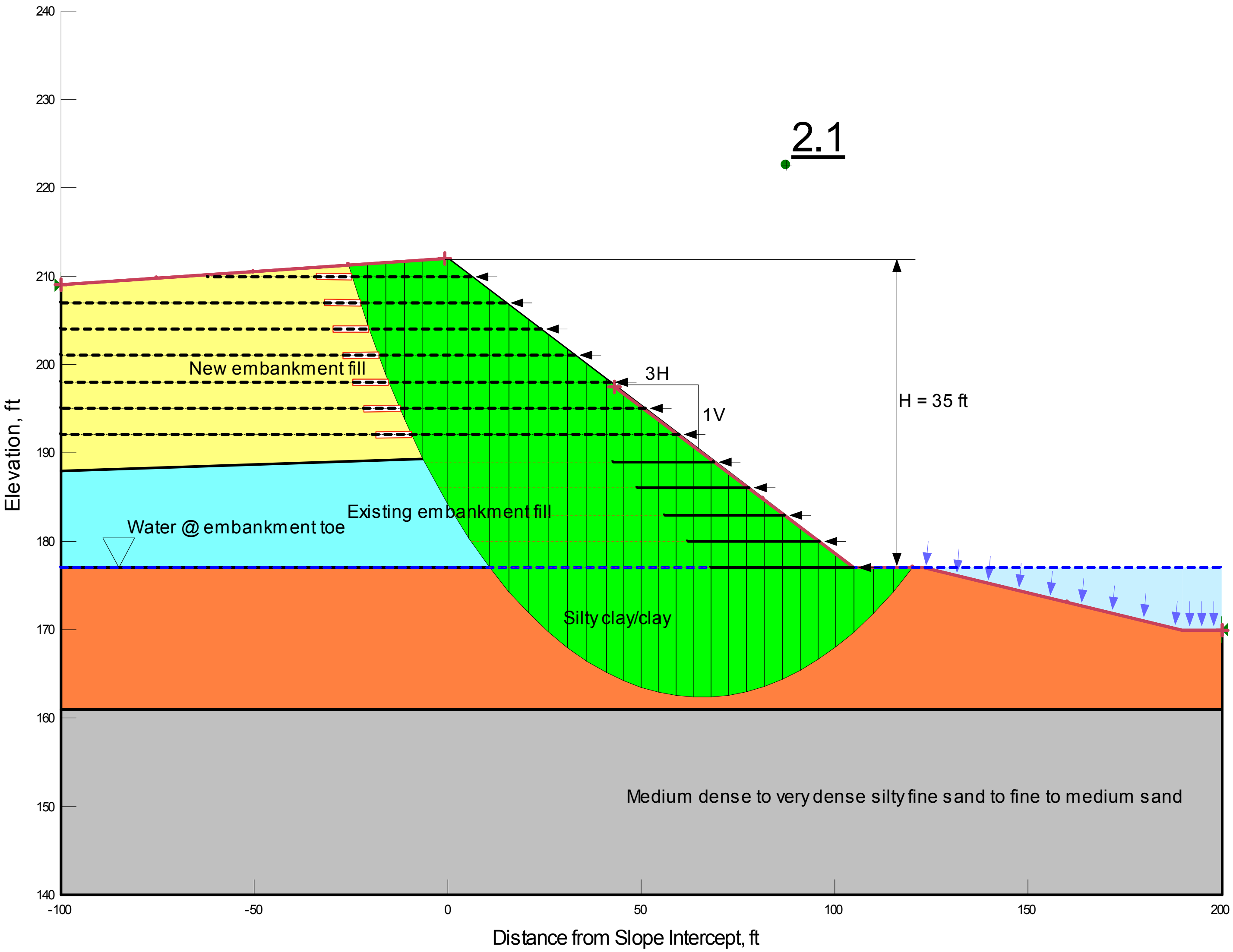
May 8, 2014

Plate

Stability Analysis Results – West Abutment
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

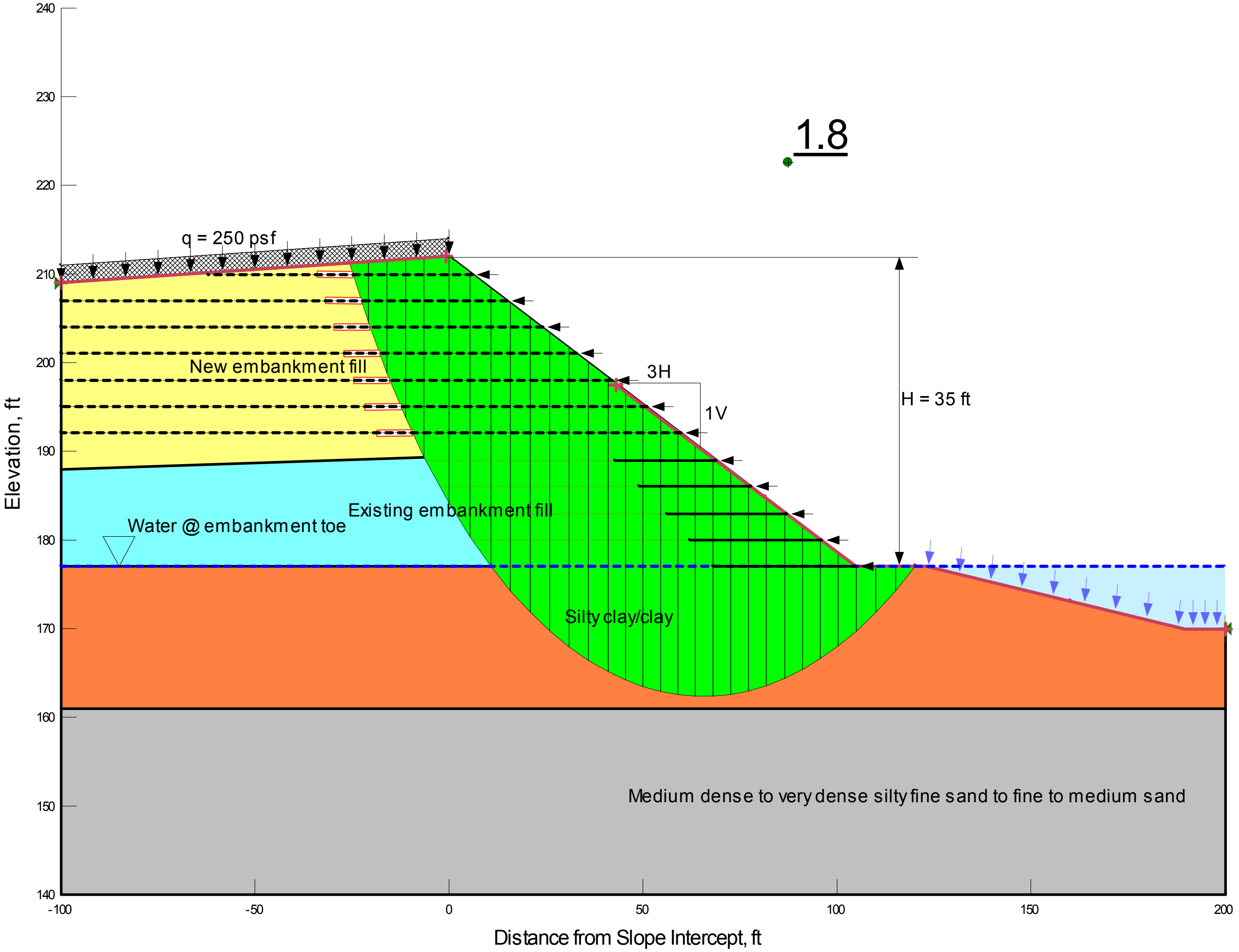
Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	Analysis long term groundwater @ embankment toe	2.1
Long Term	Analysis long term groundwater @ embankment toe	1.8
Long Term	Historical high water @ El 192.9	1.9
Seismic (k_h = 0.5A_S = 0.10)*	Analysis long term groundwater @ embankment toe	1.3
Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

* <u>Design and Construction of Mechanistically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II</u>, Publication No. FHWA-NHI-10-025, FHWA, November 2009, Page 8-10.



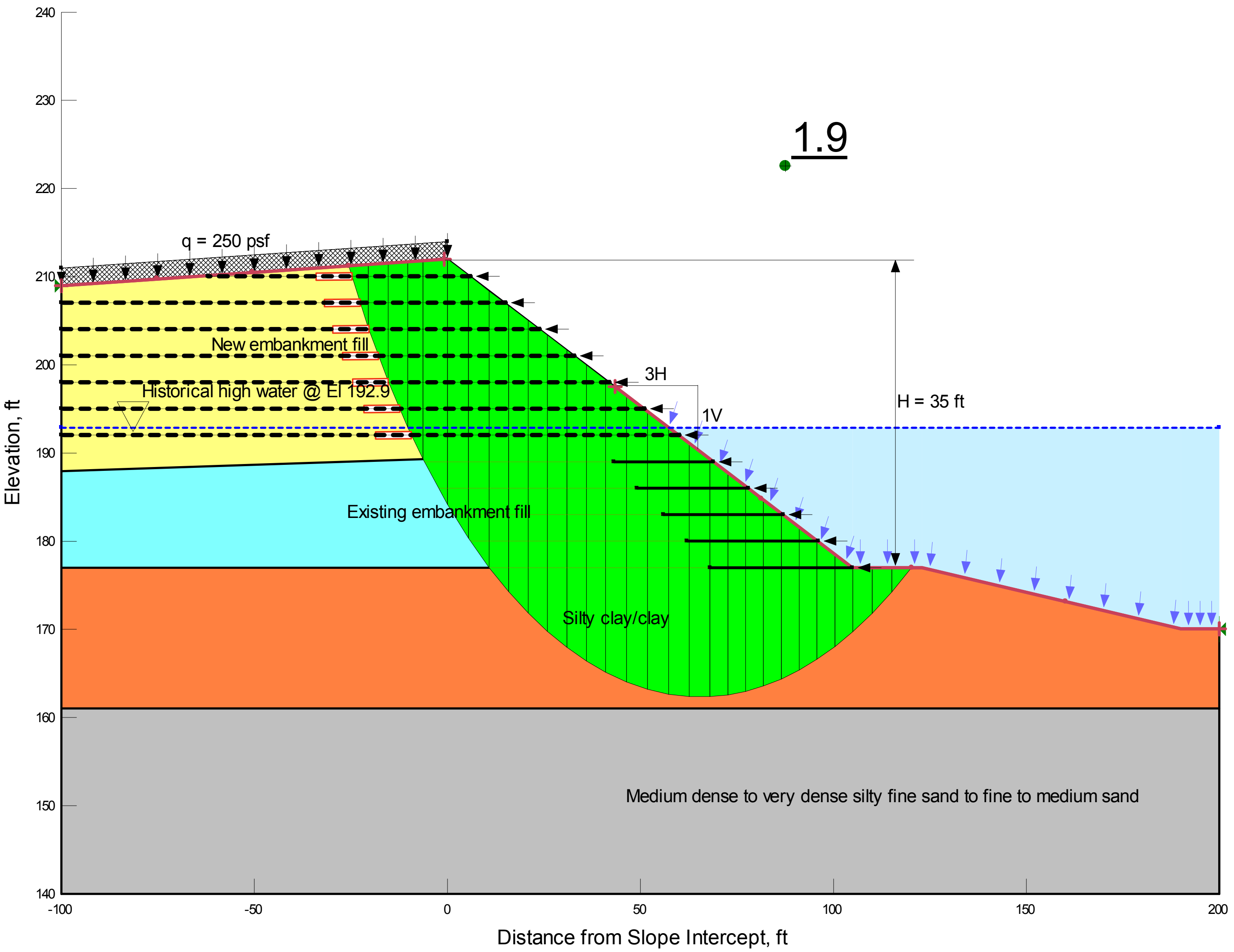
Results of Stability Analyses – End of Construction Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



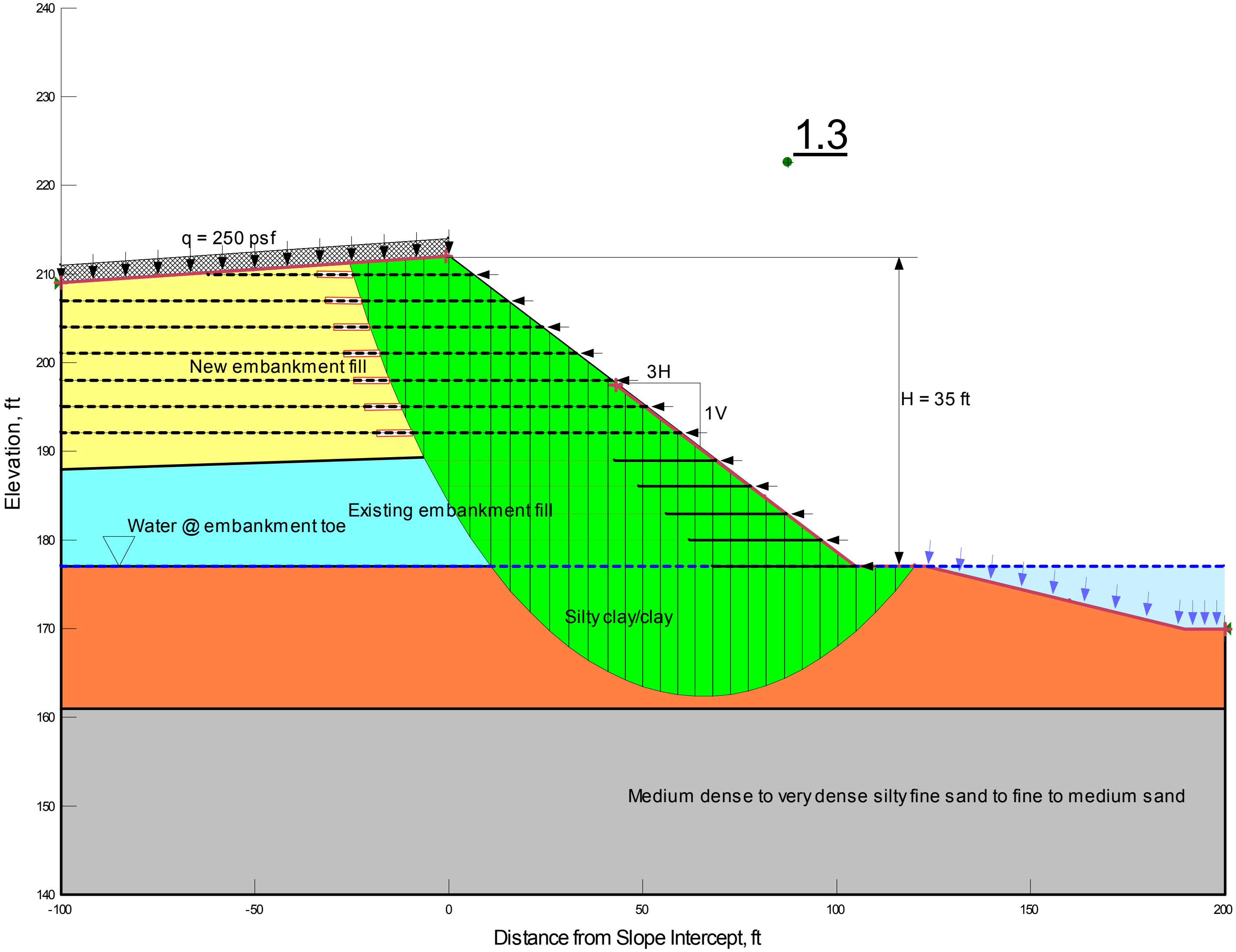
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



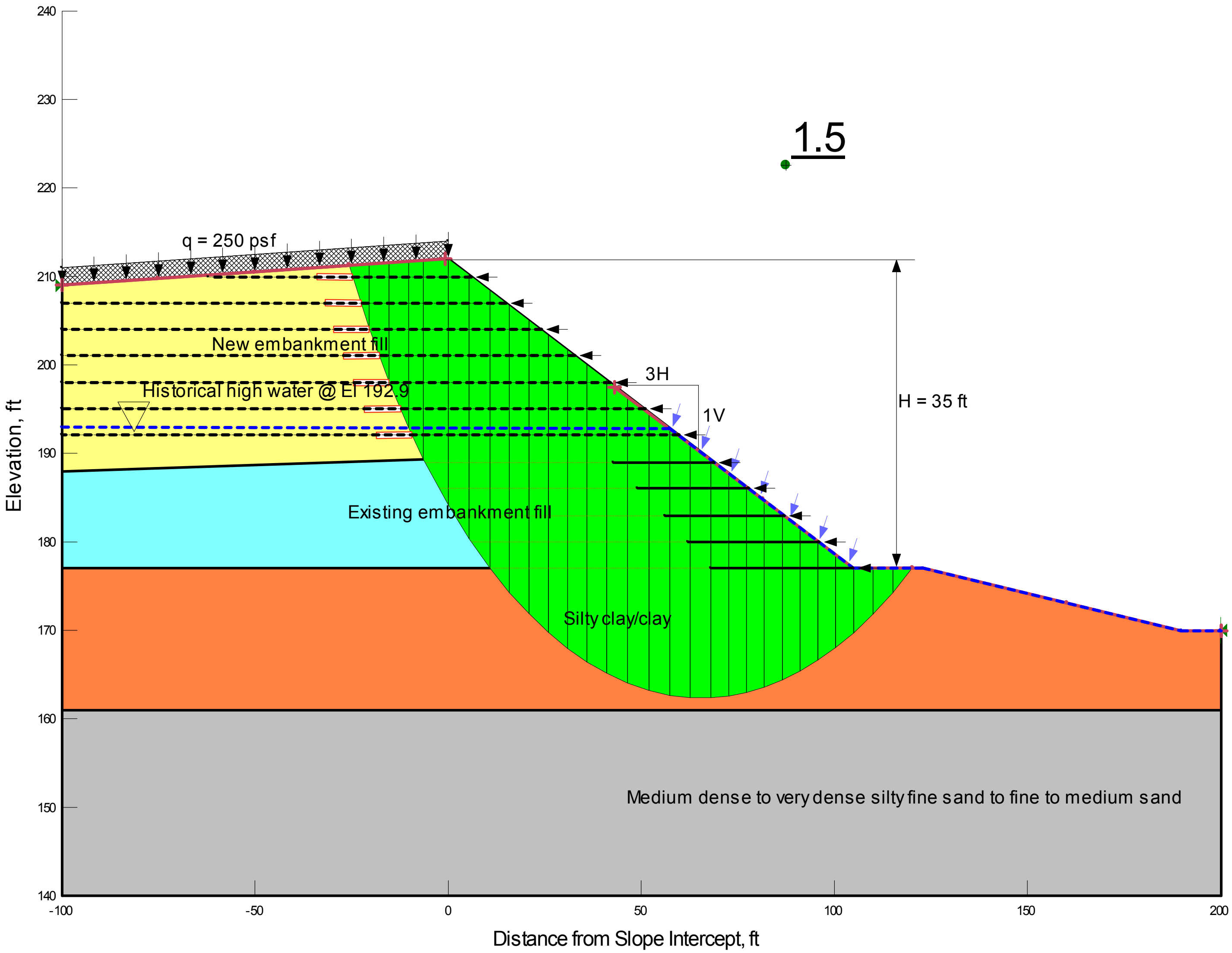
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

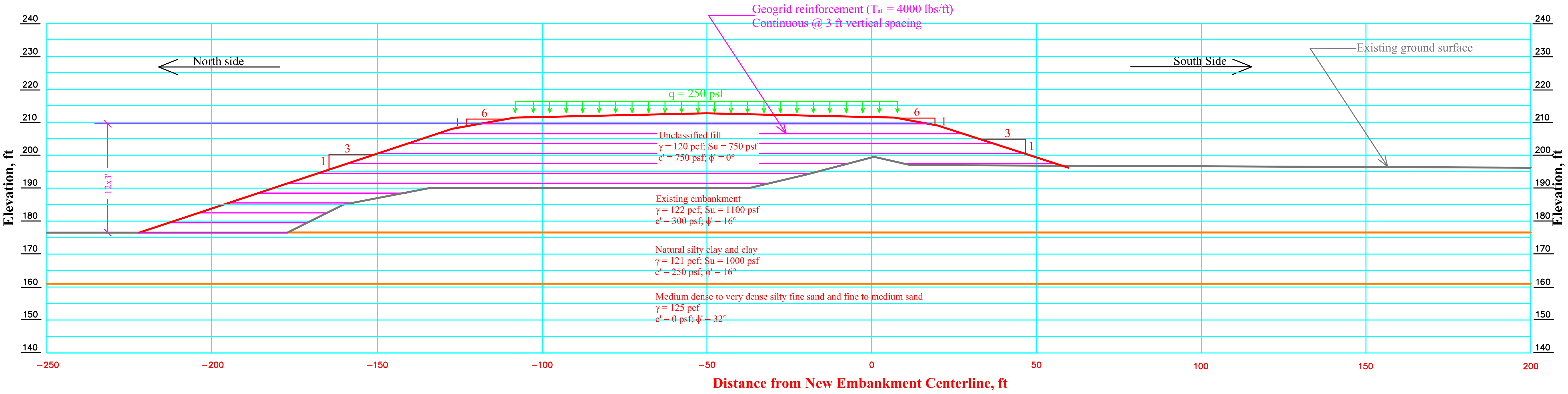
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX K



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

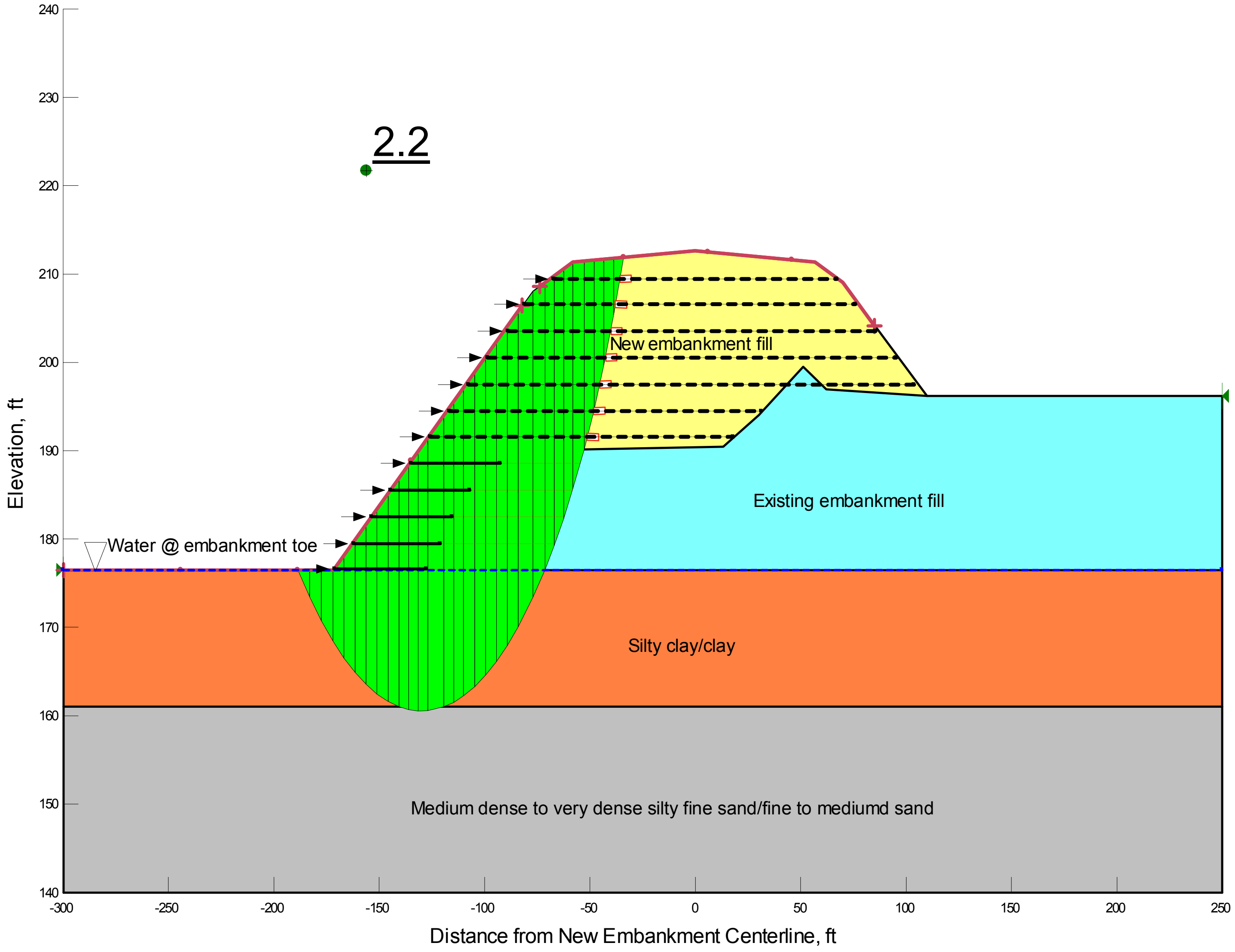
Section and Material Parameters for Stability Analysis
Reinforced Side Slopes @ West Bridge Abutment - Sta 1883+41.92
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	**Scale: As Shown**
May 4, 2014	**Plate**

Stability Analysis Results
Reinforced Side Slopes @ West Abutment - Sta 1883+41.92
AHTD JOB BB0610:White River Str. & Apprs. (F)

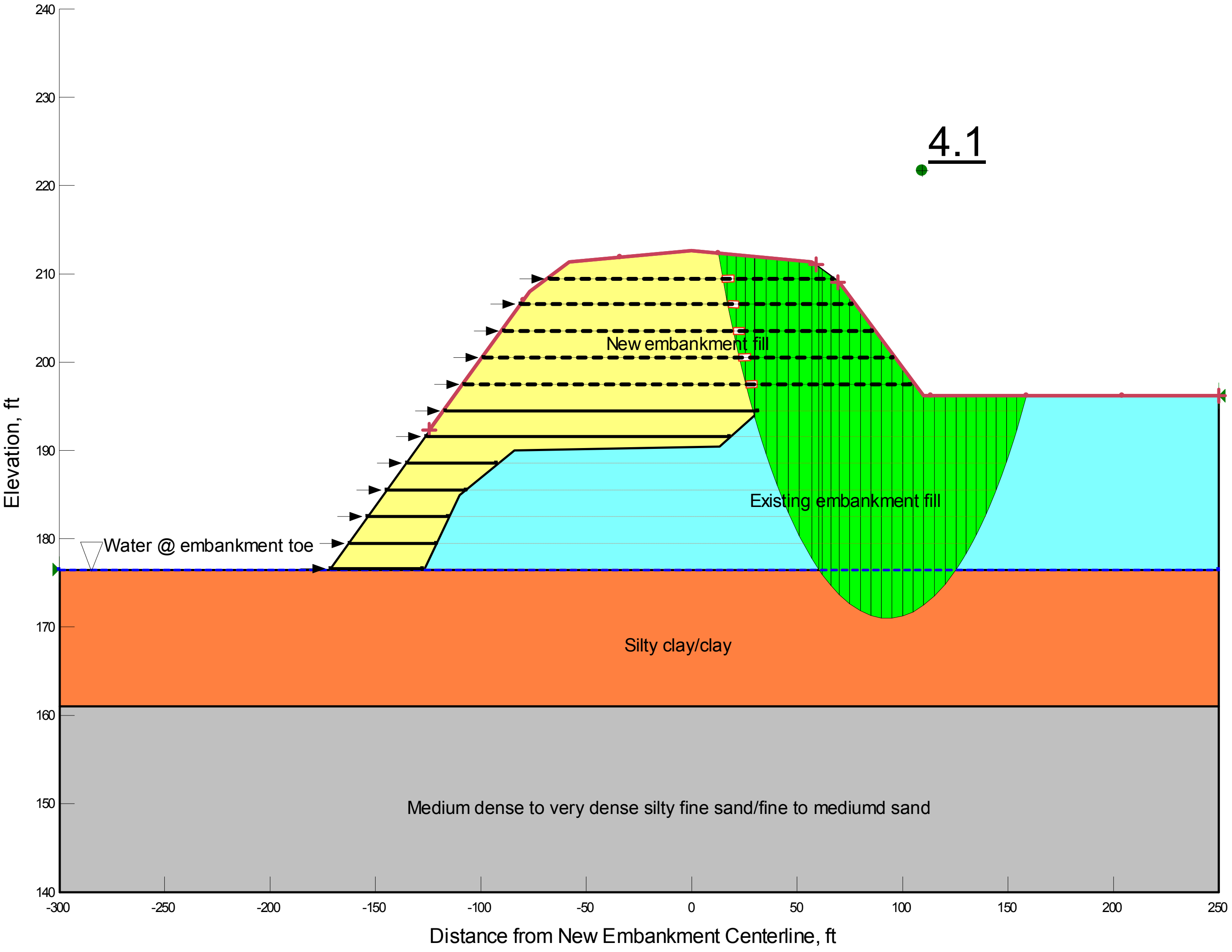
Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.2
South			4.1
North	Long Term	Analysis long term groundwater @ embankment toe	1.9
South			2.0
North		Historical high water @ El 192.9	1.9
South			2.0
North	Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.4
South			1.5
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

* 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.



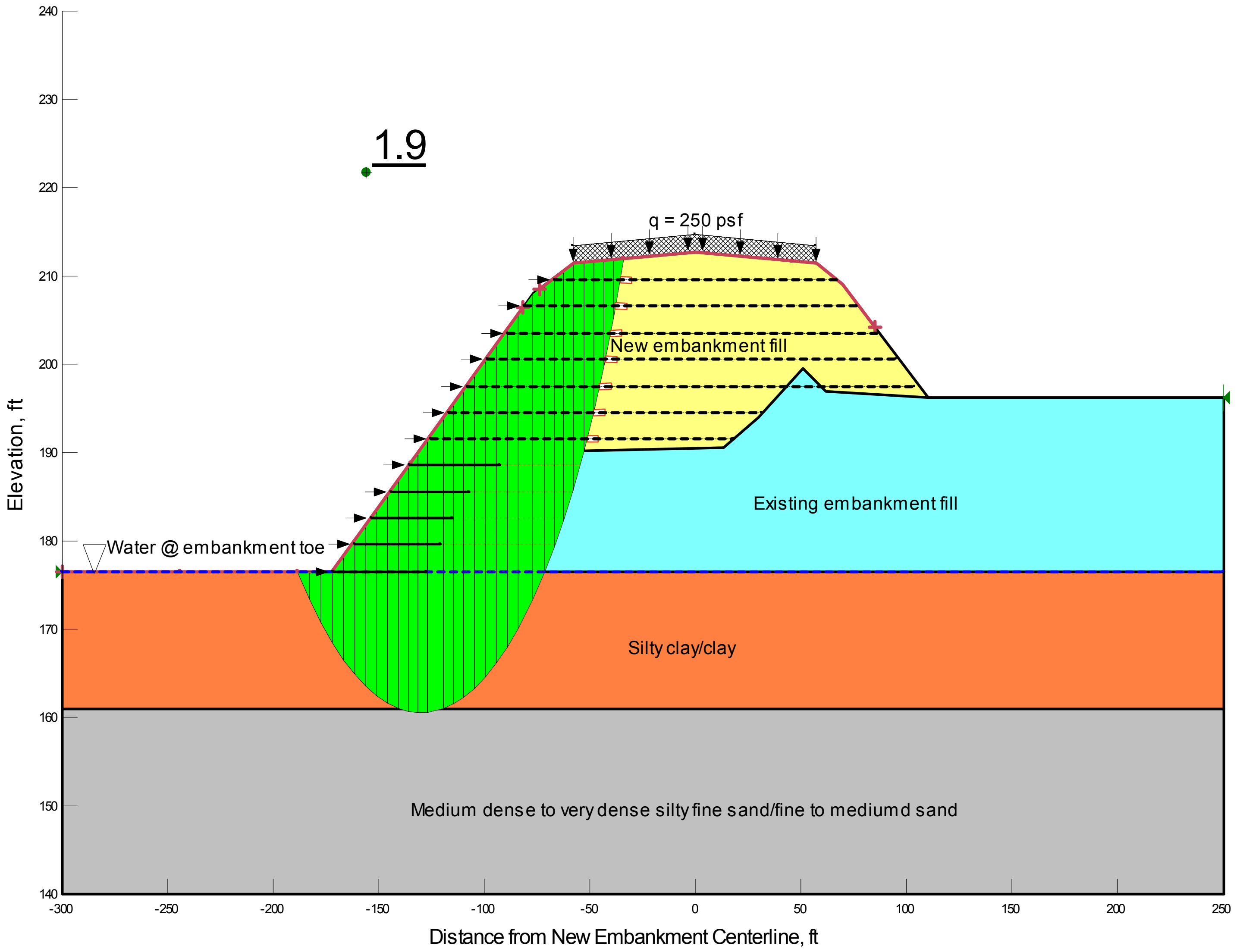
Results of Stability Analyses – End of Construction Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



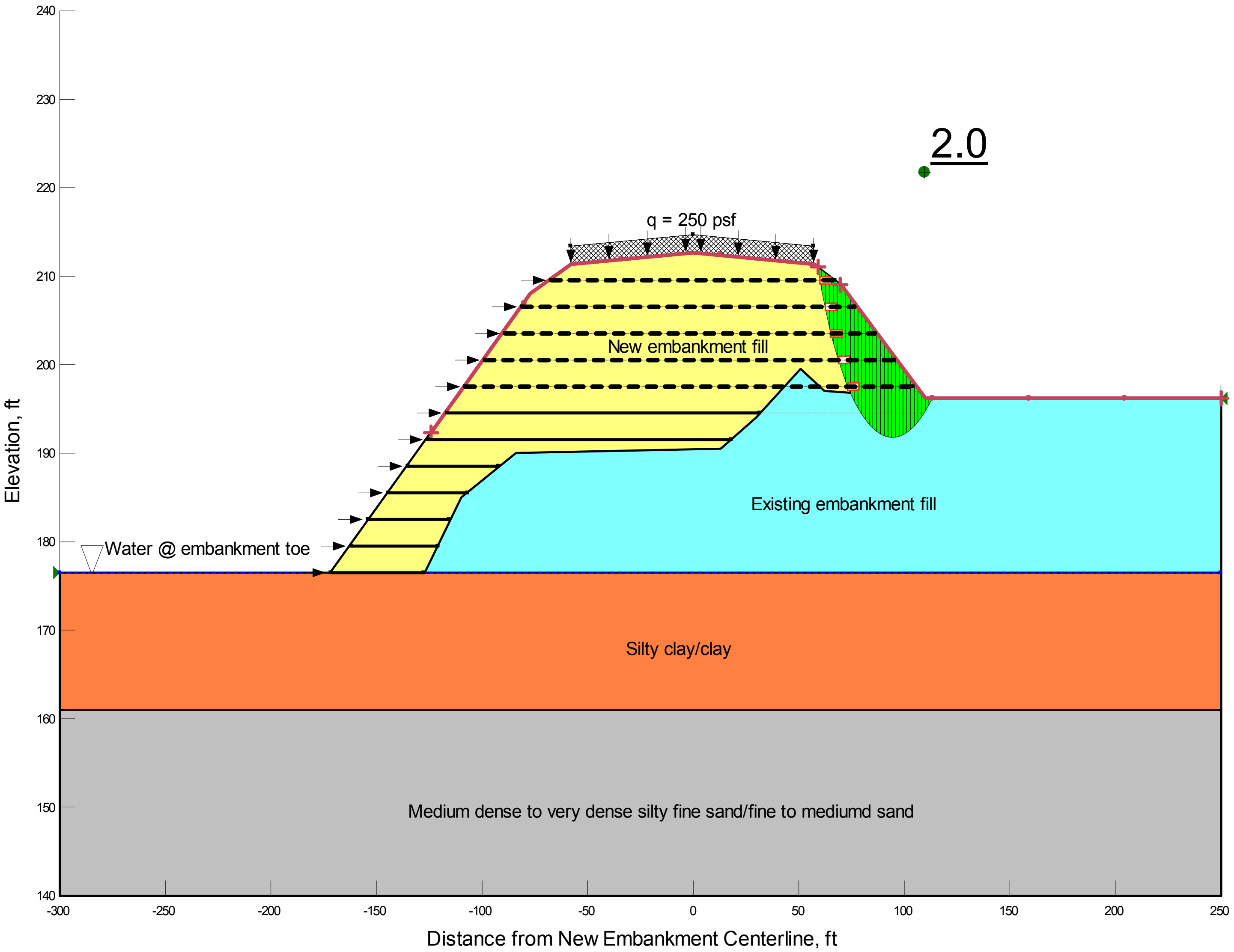
Results of Stability Analyses – End of Construction Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



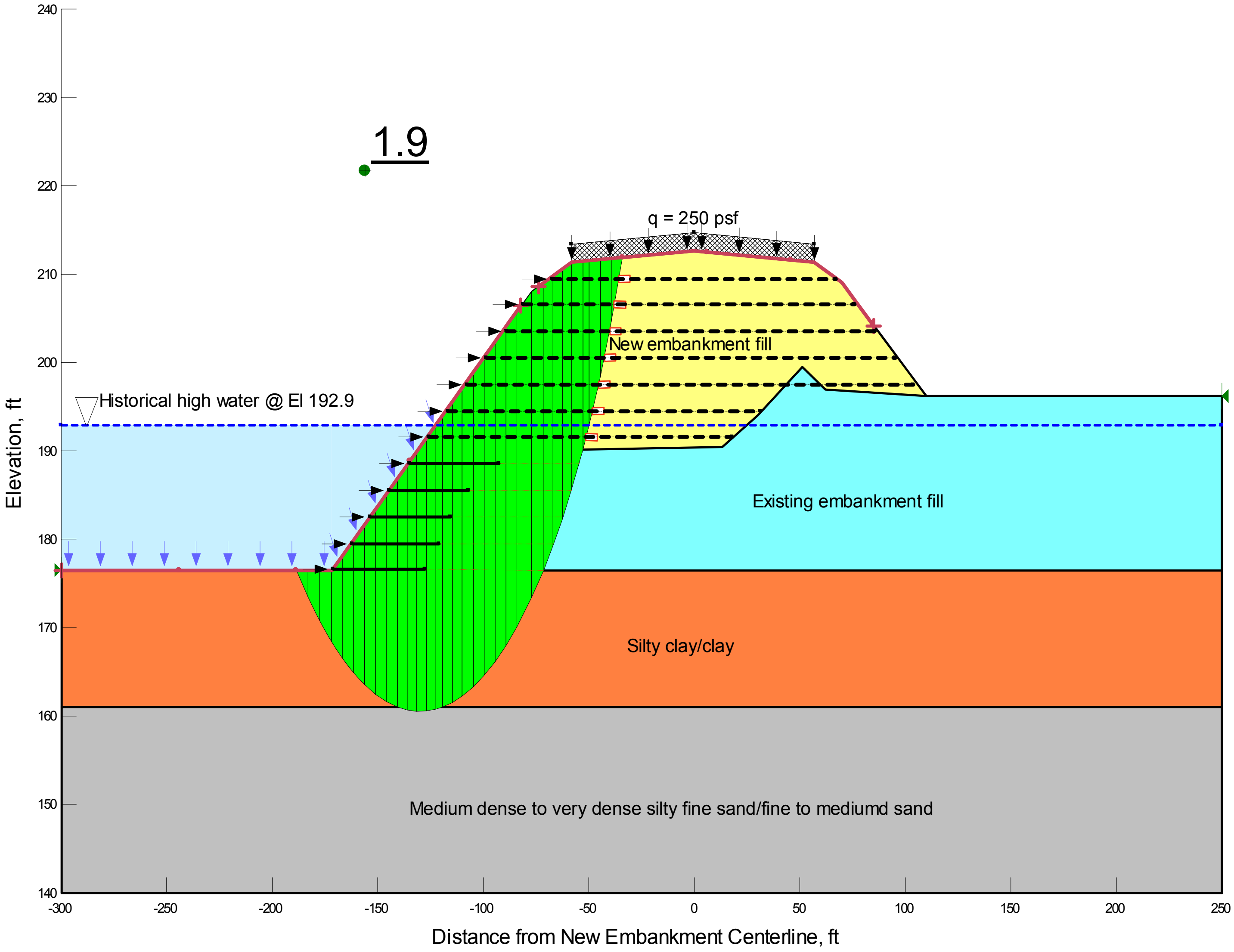
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



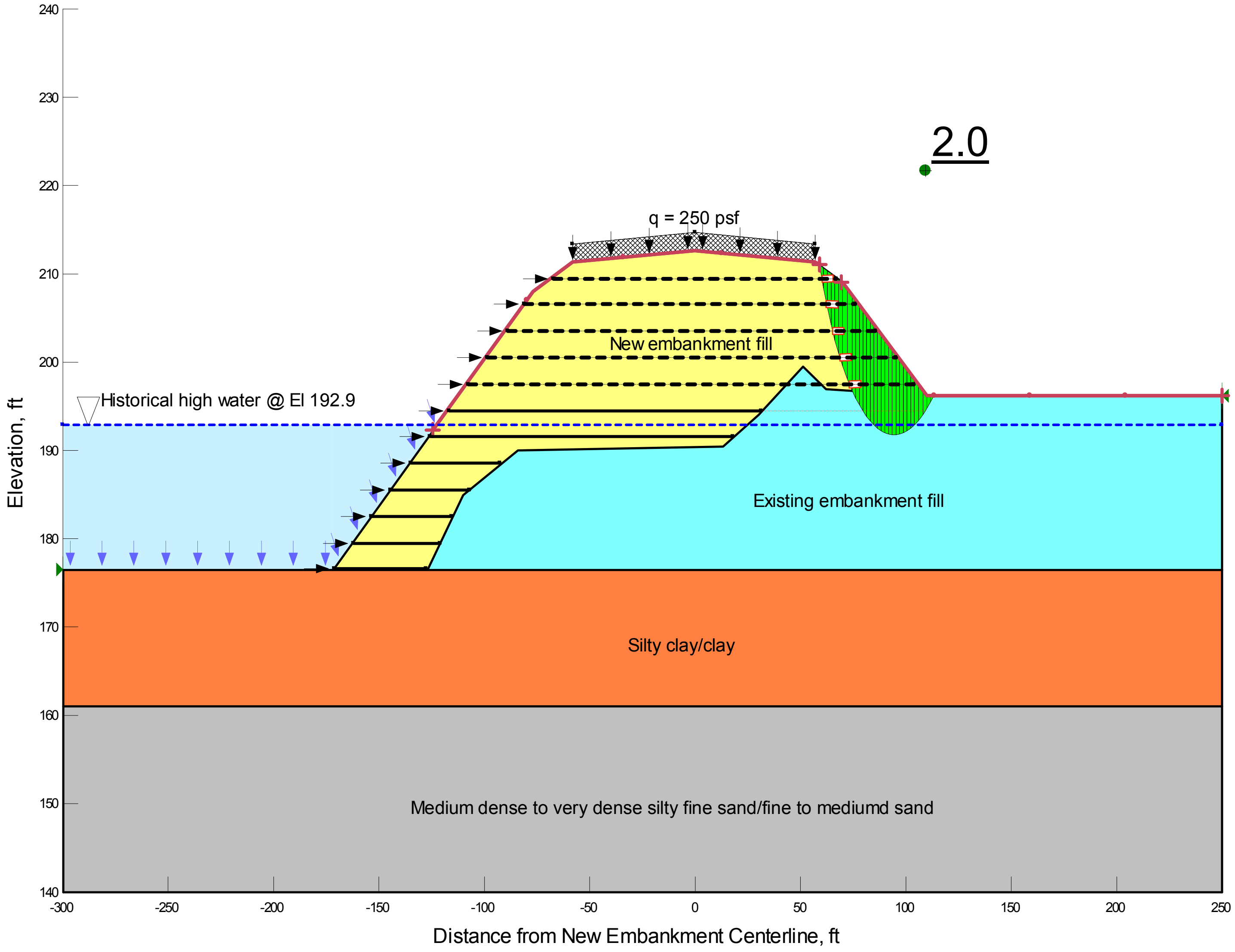
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



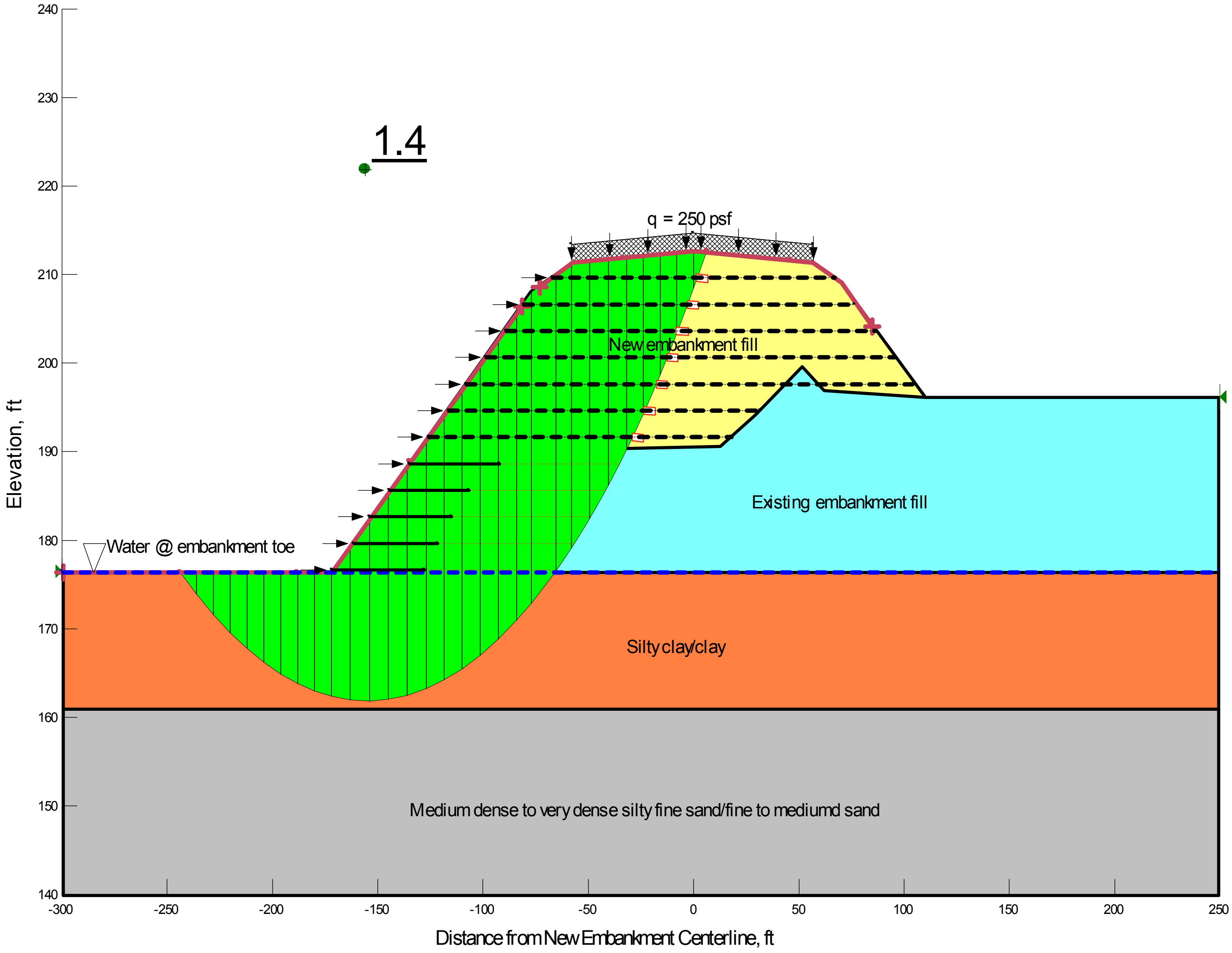
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



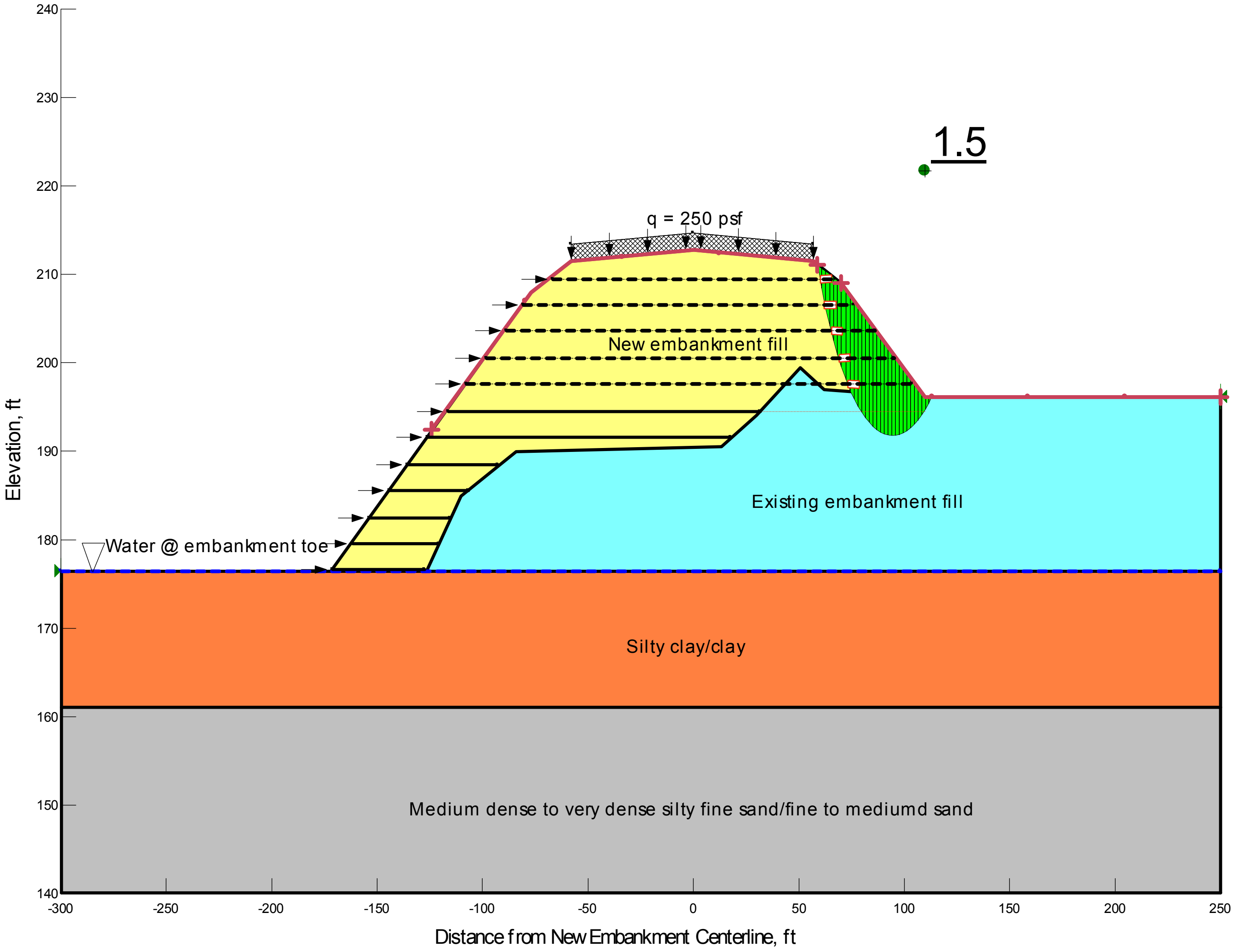
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



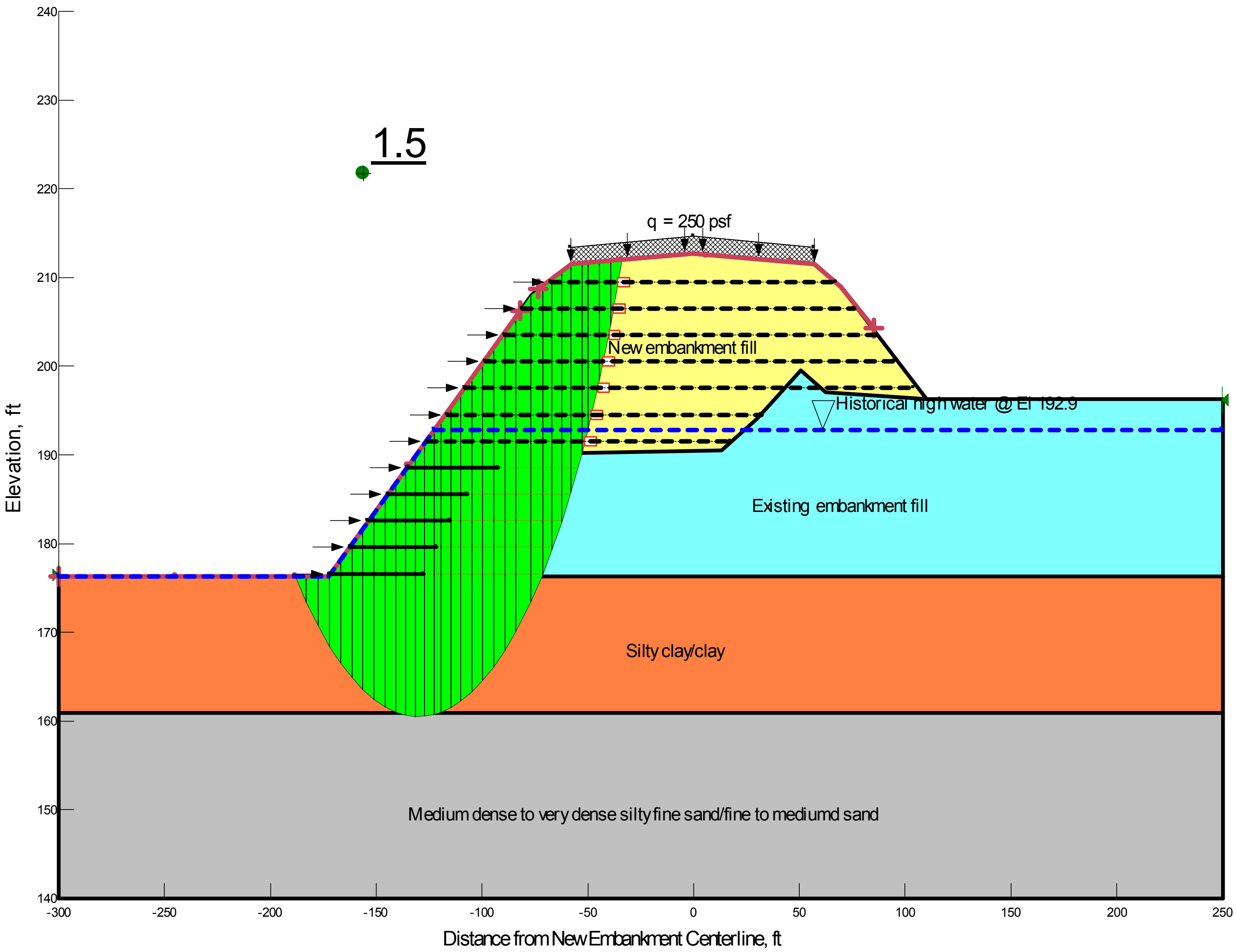
Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

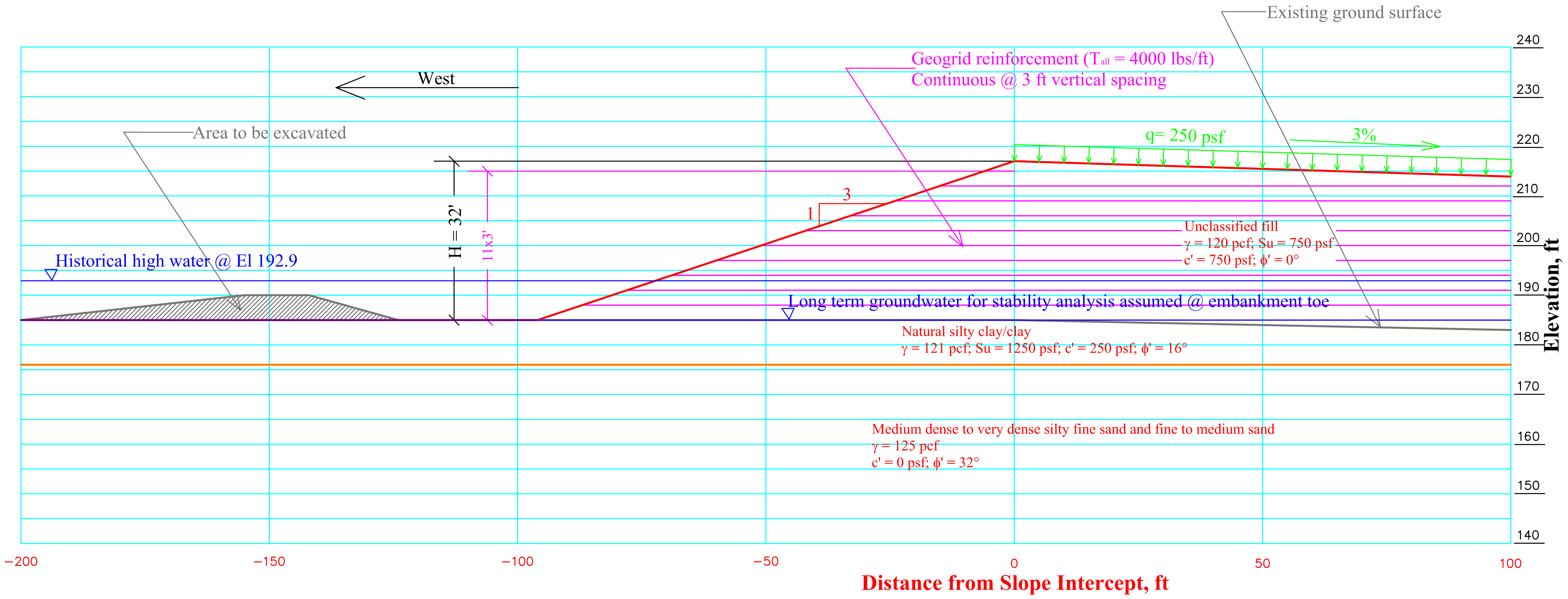
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX L



10 ft 0 10 20 ft

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

Section and Material Parameters for Stability Analysis
Reinforced End Slope @ East Abutment
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

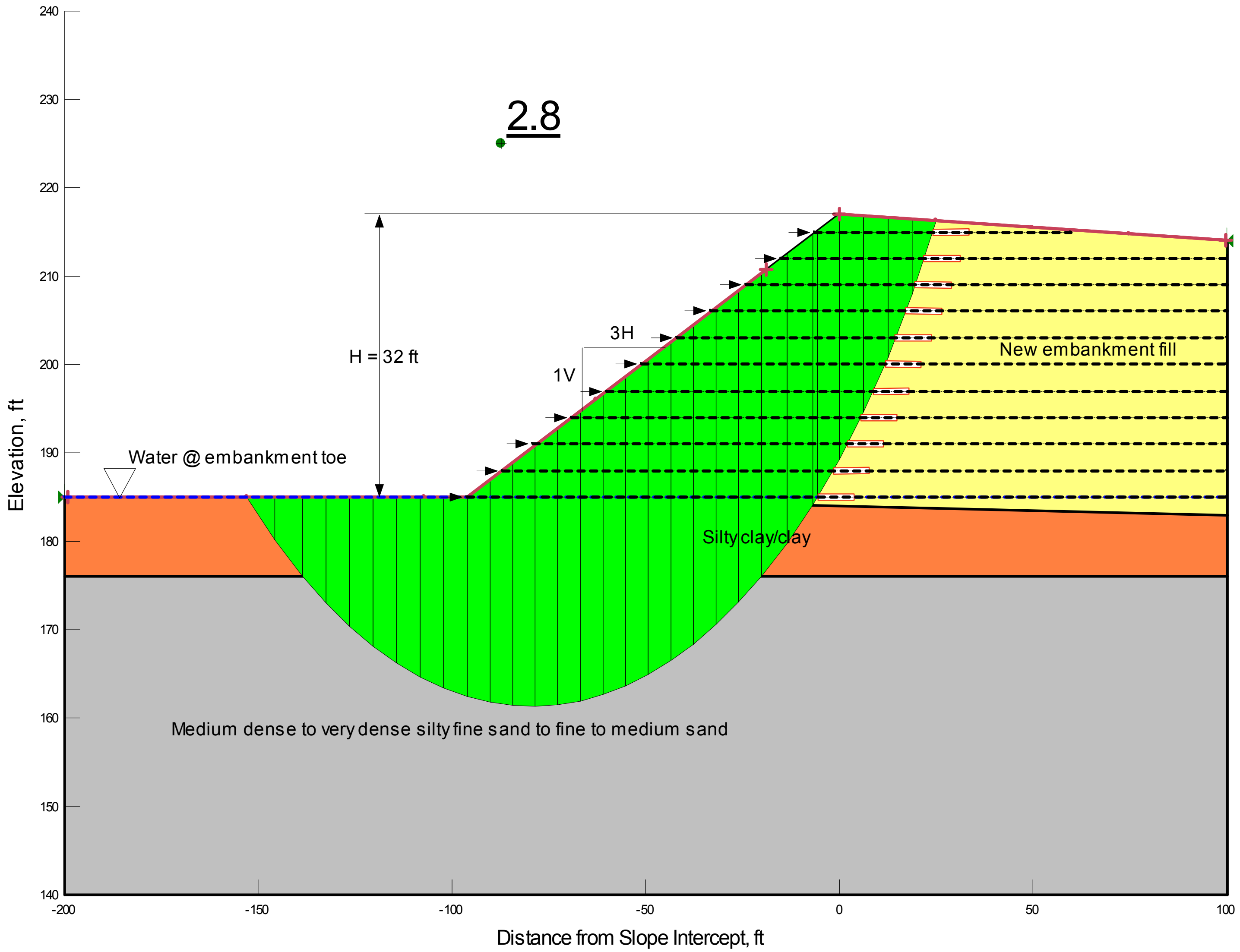
May 8, 2014

Plate

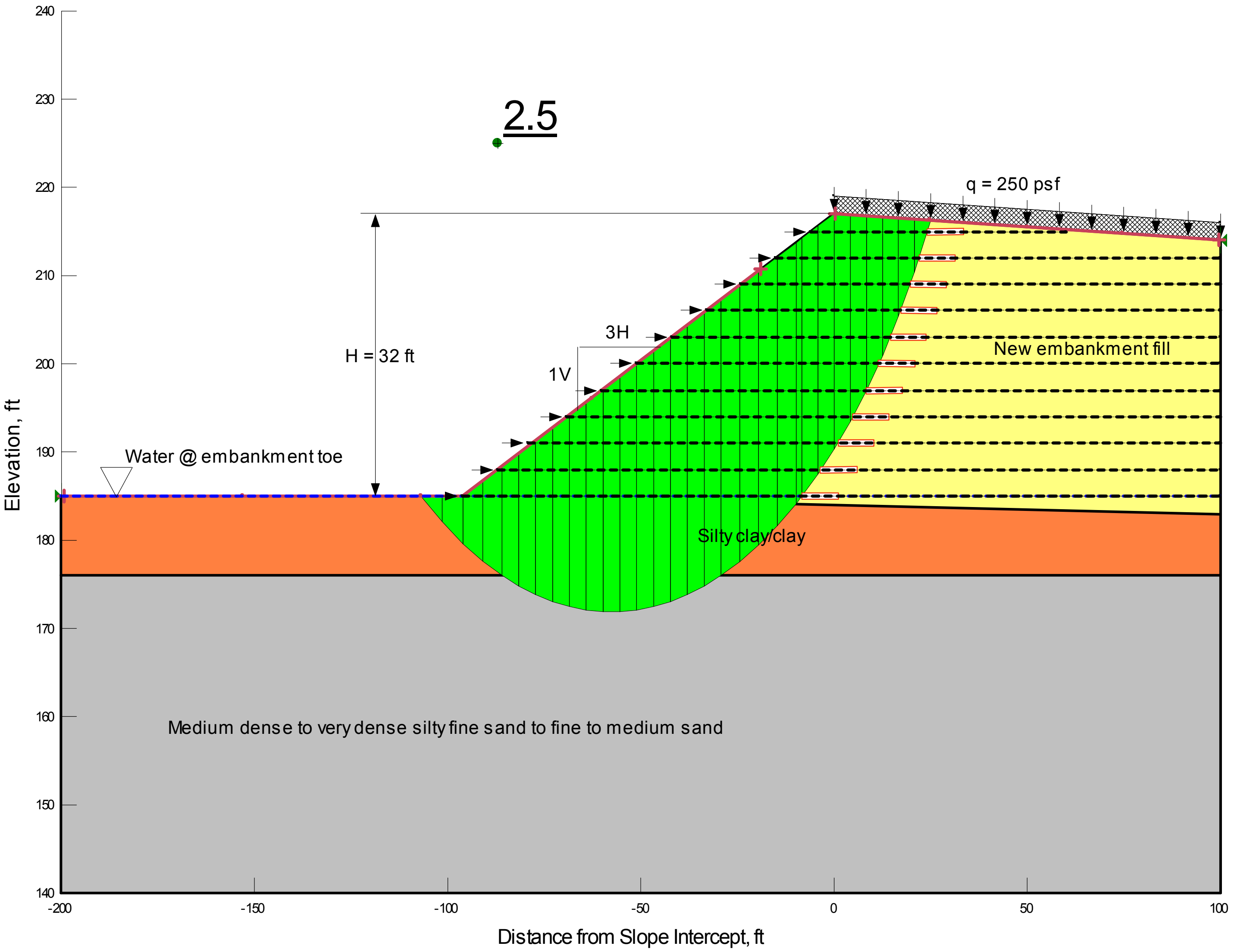
Stability Analysis Results – East Abutment
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Analysis long term groundwater @ embankment toe	2.8
Long Term	Analysis long term groundwater @ embankment toe	2.5
Long Term	Historical high water @ El 192.9	2.4
Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.8
Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.3

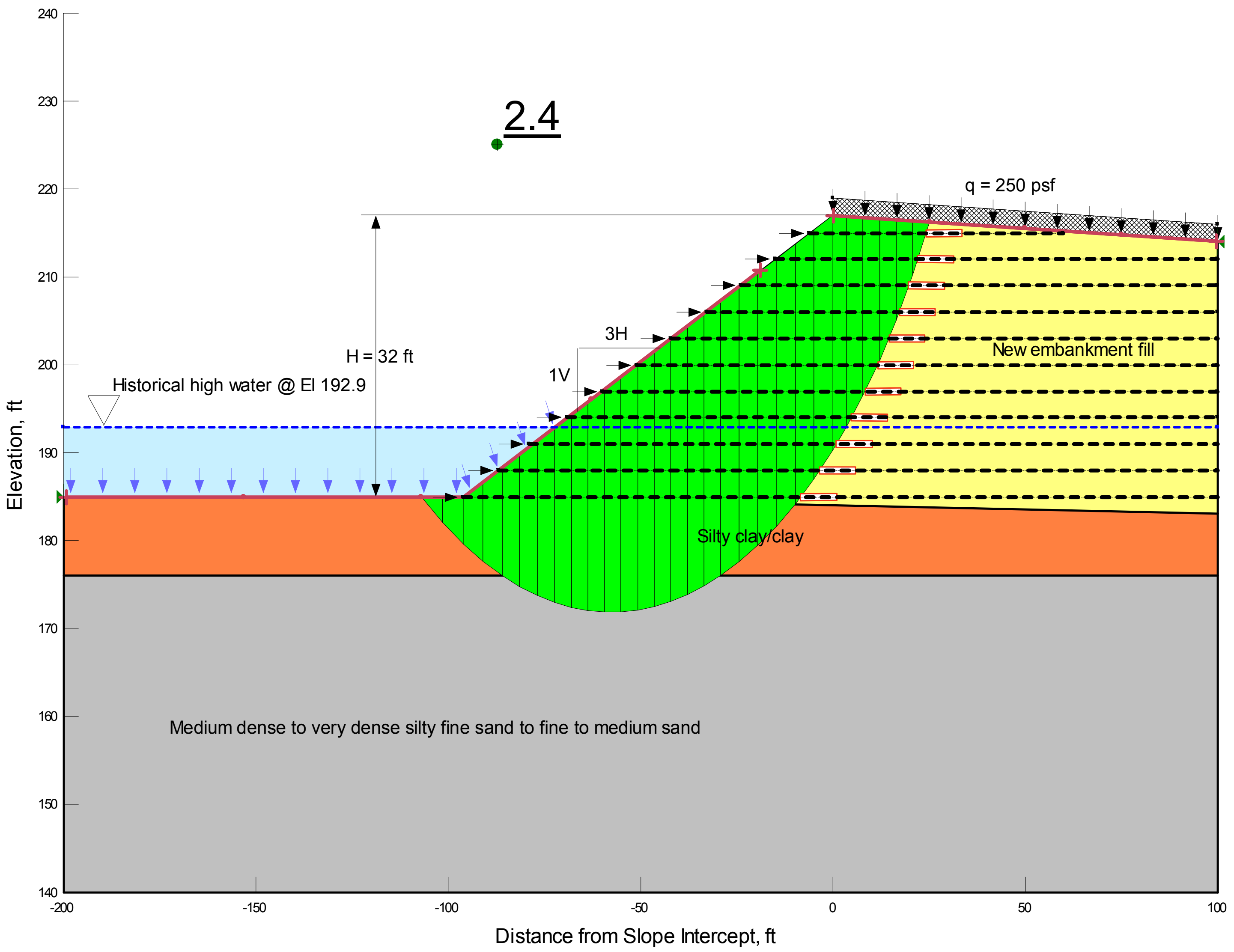
* Design and Construction of Mechanistically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II, Publication No. FHWA-NHI-10-025, FHWA, November 2009, Page 8-10.



Results of Stability Analyses – End of Construction Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

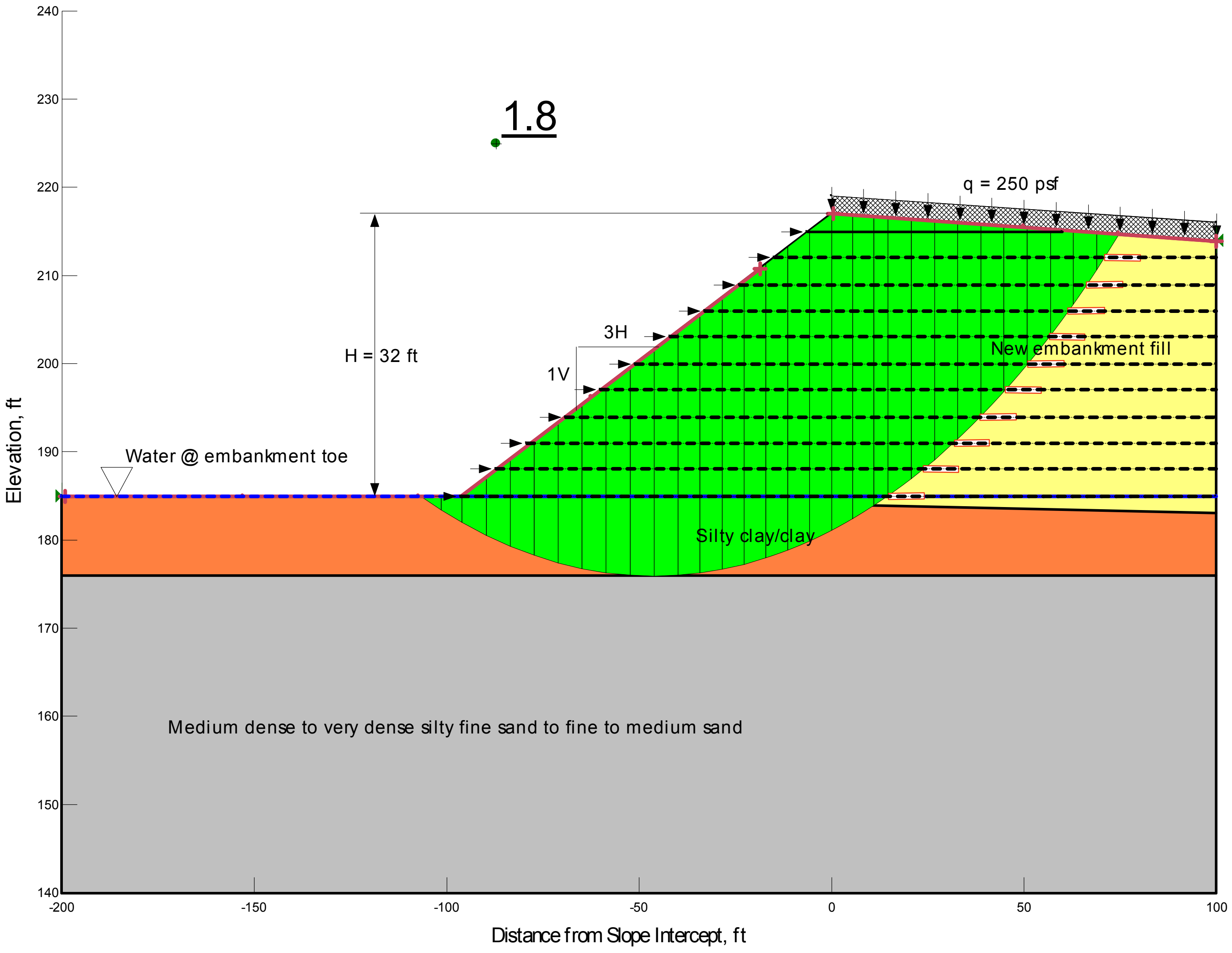


Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



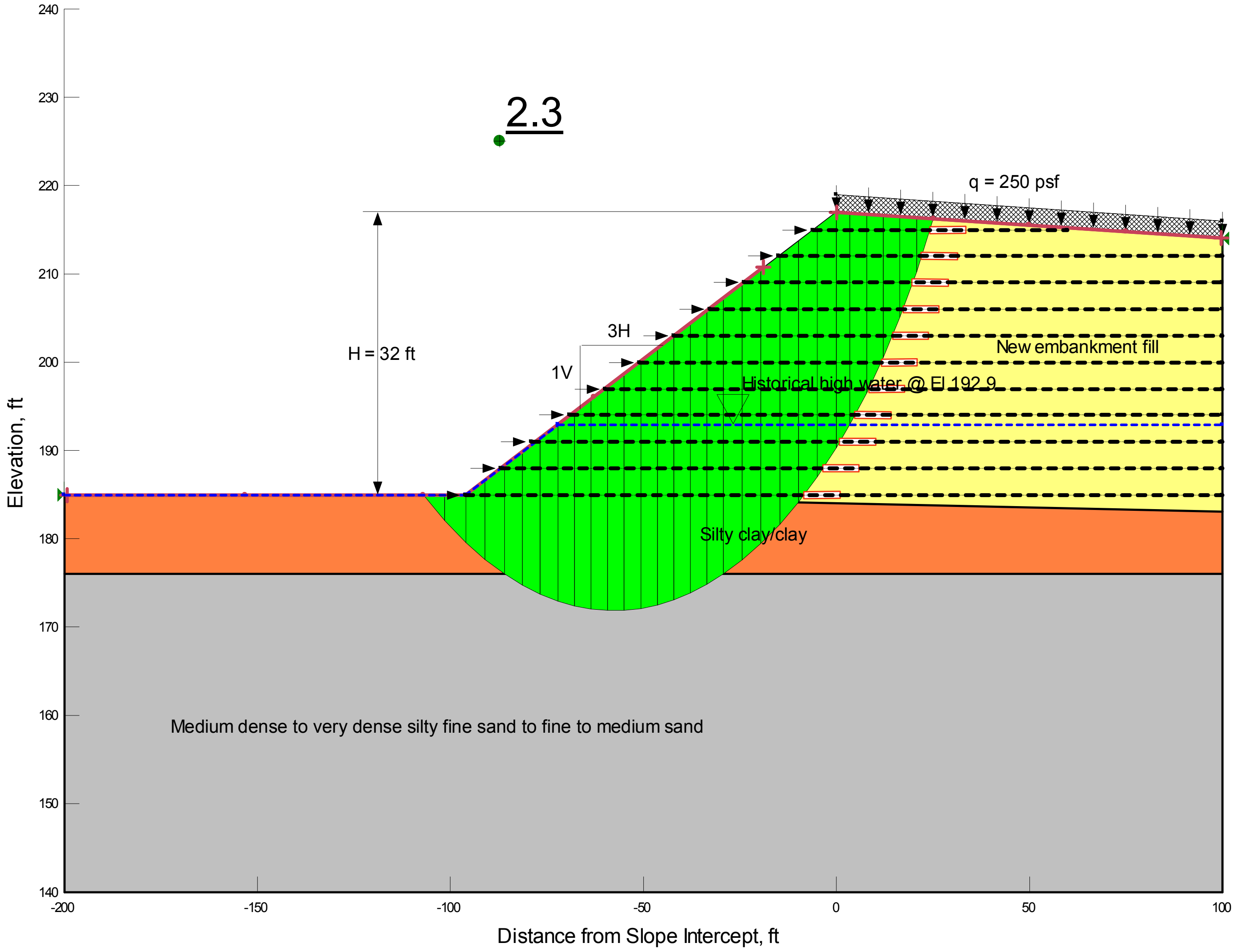
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

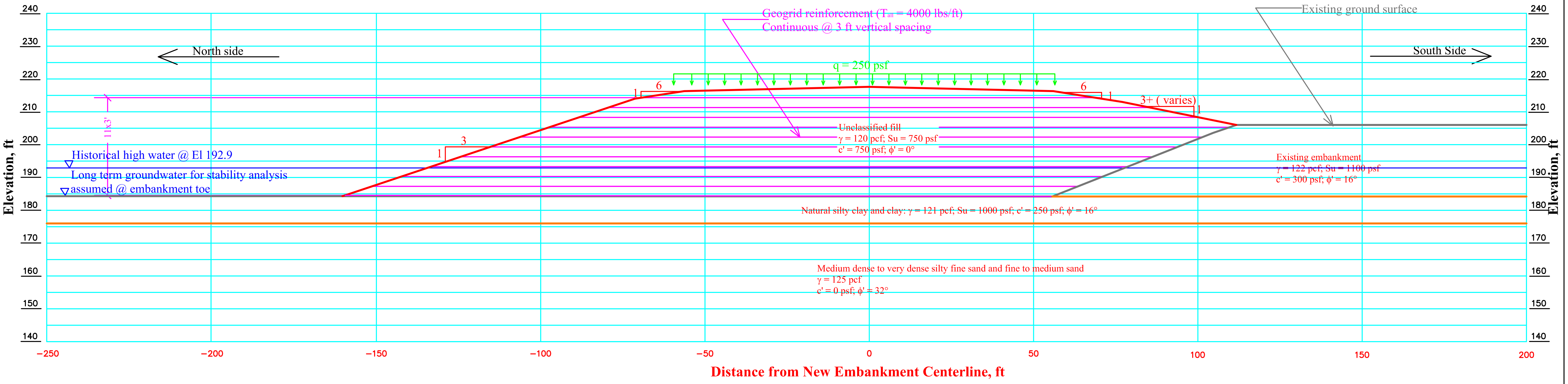
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX M



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

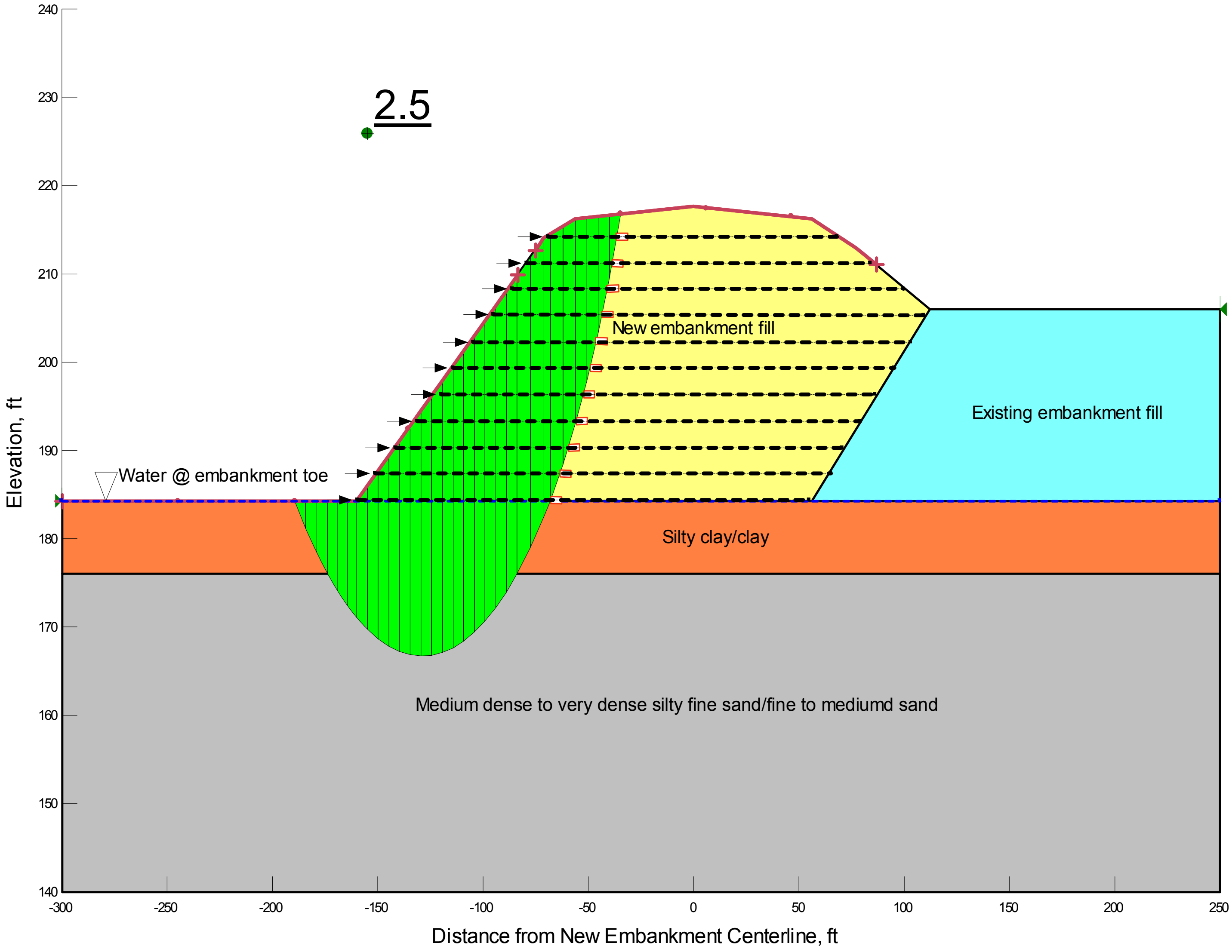
Section and Material Parameters for Stability Analysis
Reinforced Side Slopes @ East Bridge Abutment - Sta 1911+84.08
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	Scale: As Shown
May 7, 2014	Plate

Stability Analysis Results
Reinforced Side Slopes @ East Abutment - Sta 1911+84.08
AHTD JOB BB0610:White River Str. & Apprs. (F)

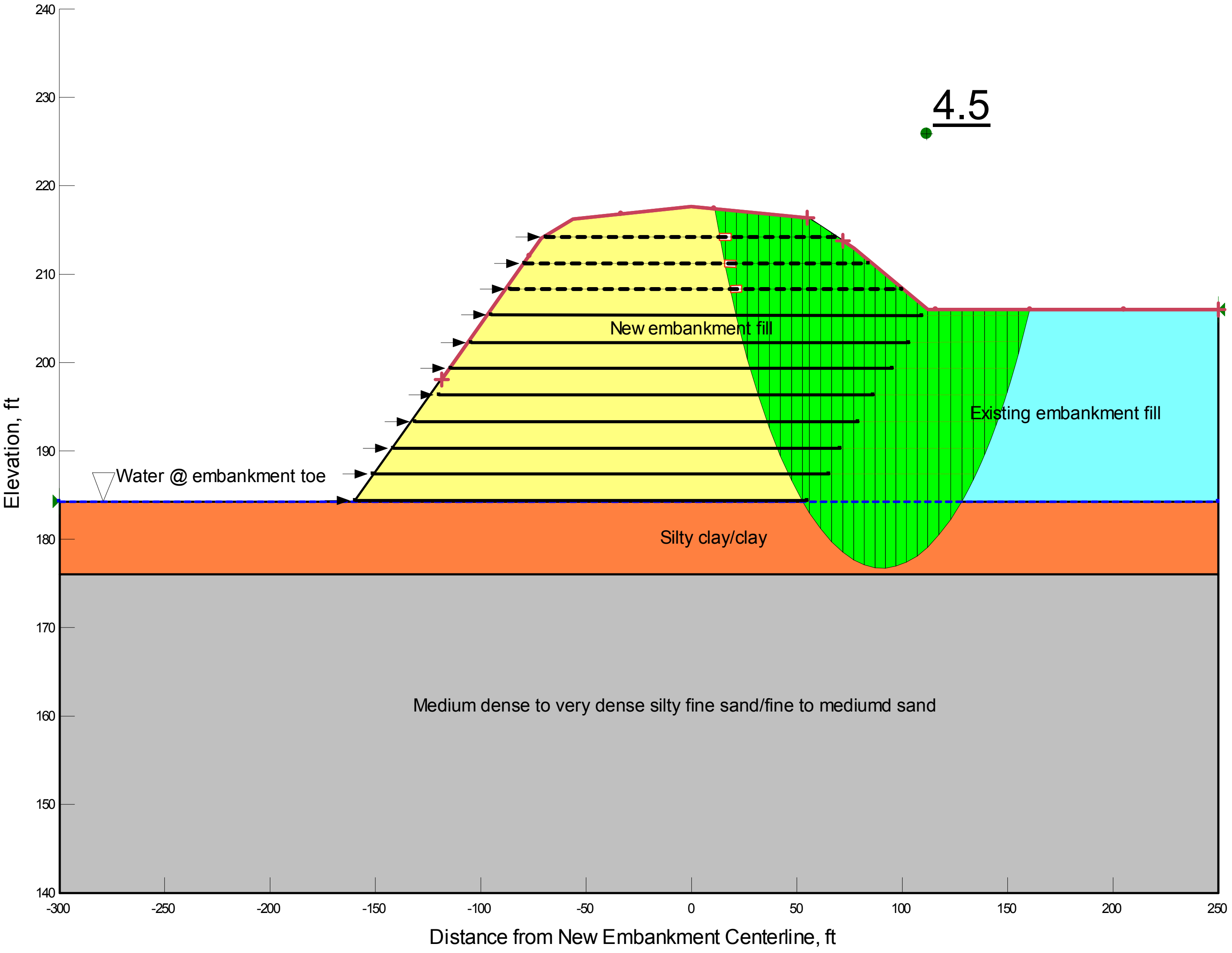
Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.5
South			4.5
North	Long Term	Analysis long term groundwater @ embankment toe	2.3
South			3.4
North		Historical high water @ El 192.9	2.3
South			3.4
North	Seismic ($k_h = 0.5A_S =$ 0.10)*	Analysis long term groundwater @ embankment toe	1.7
South			1.9
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.1

* 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.



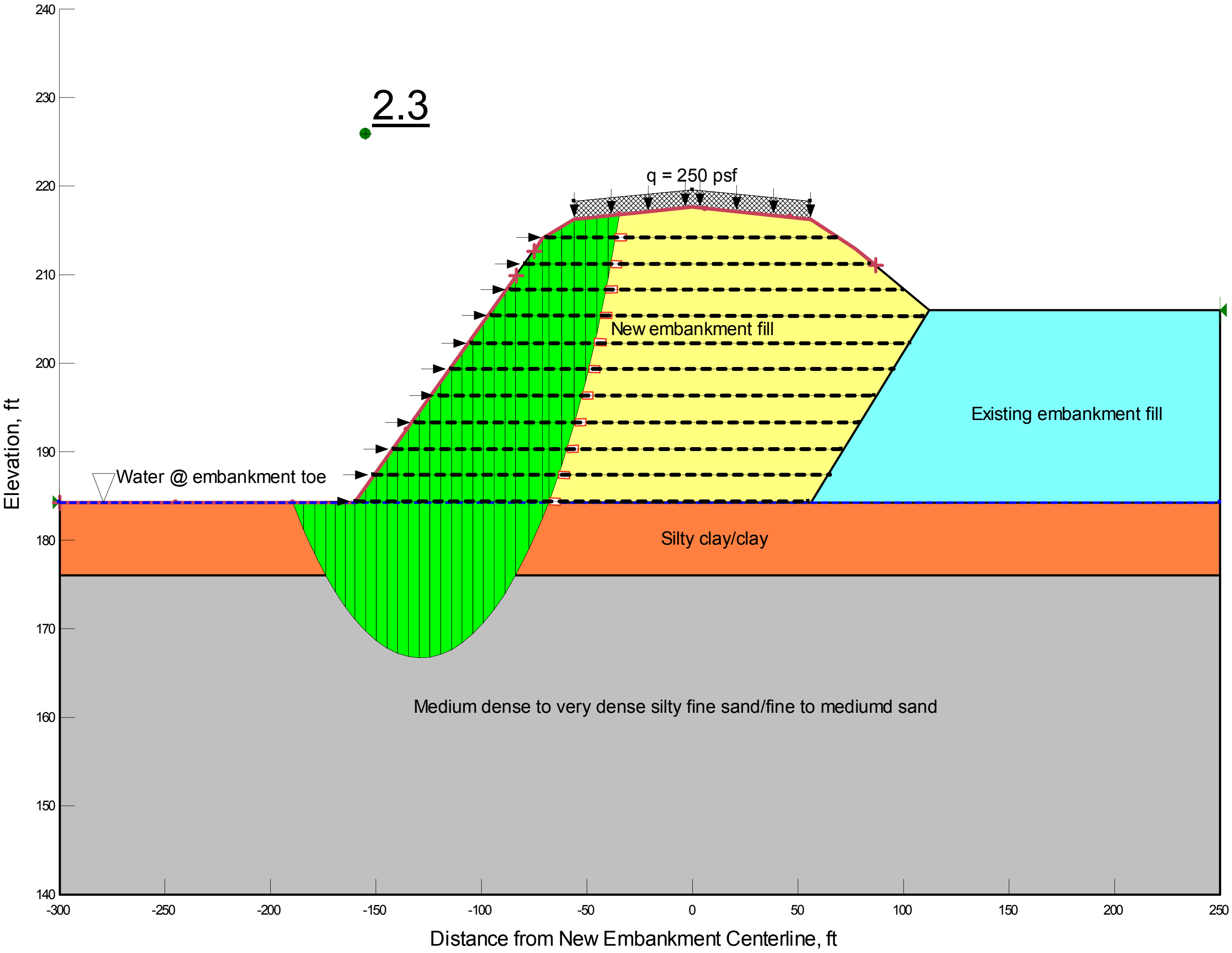
Results of Stability Analyses – End of Construction Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



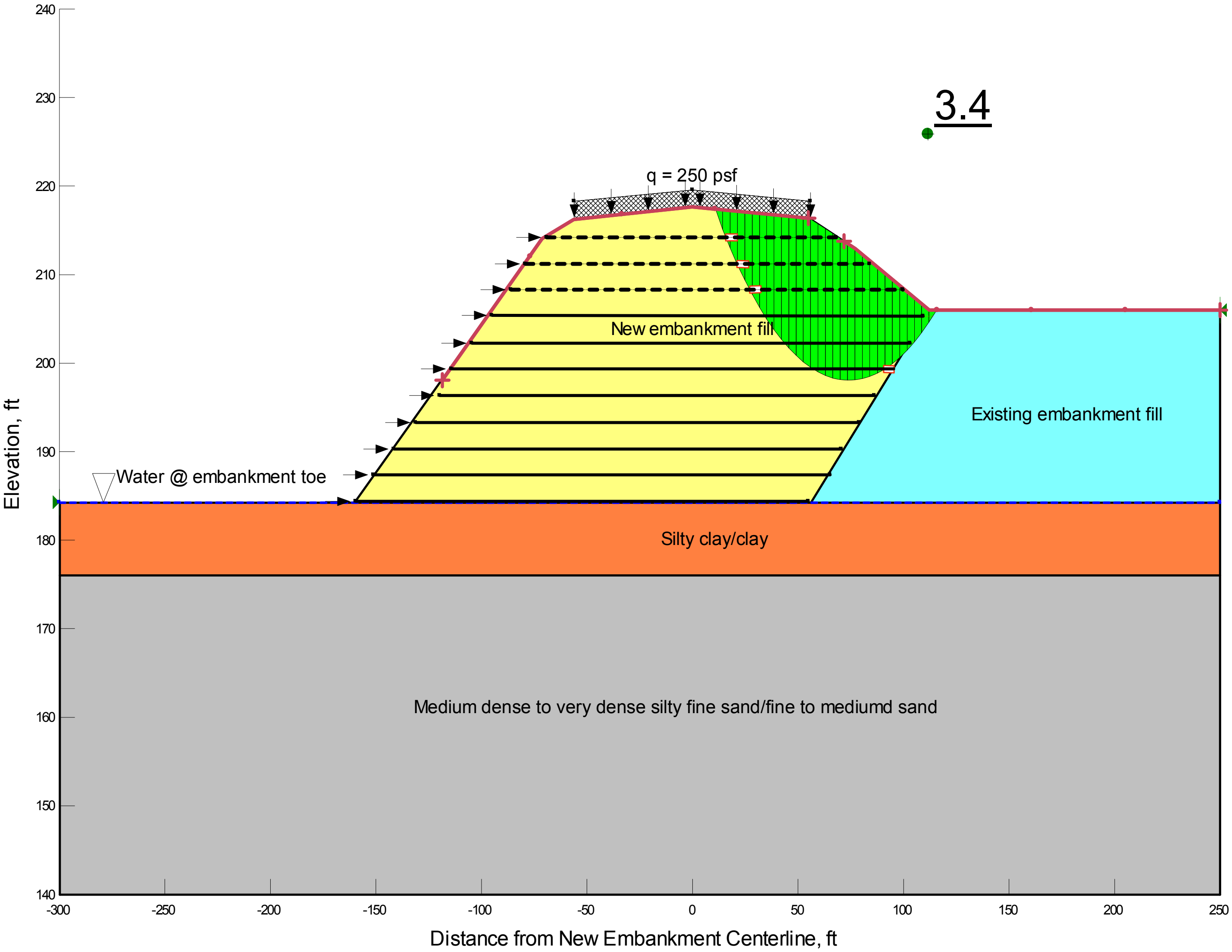
Results of Stability Analyses – End of Construction Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



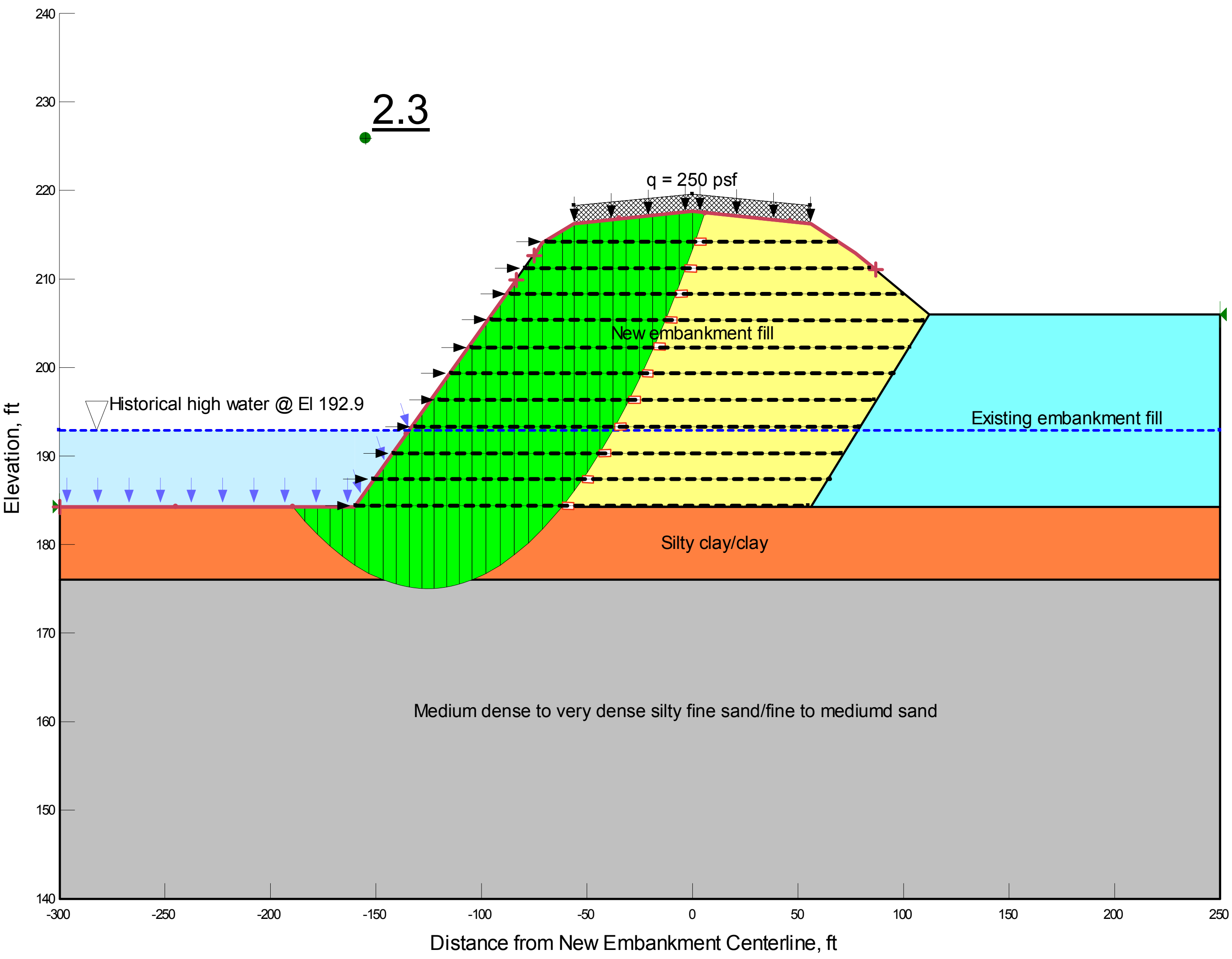
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



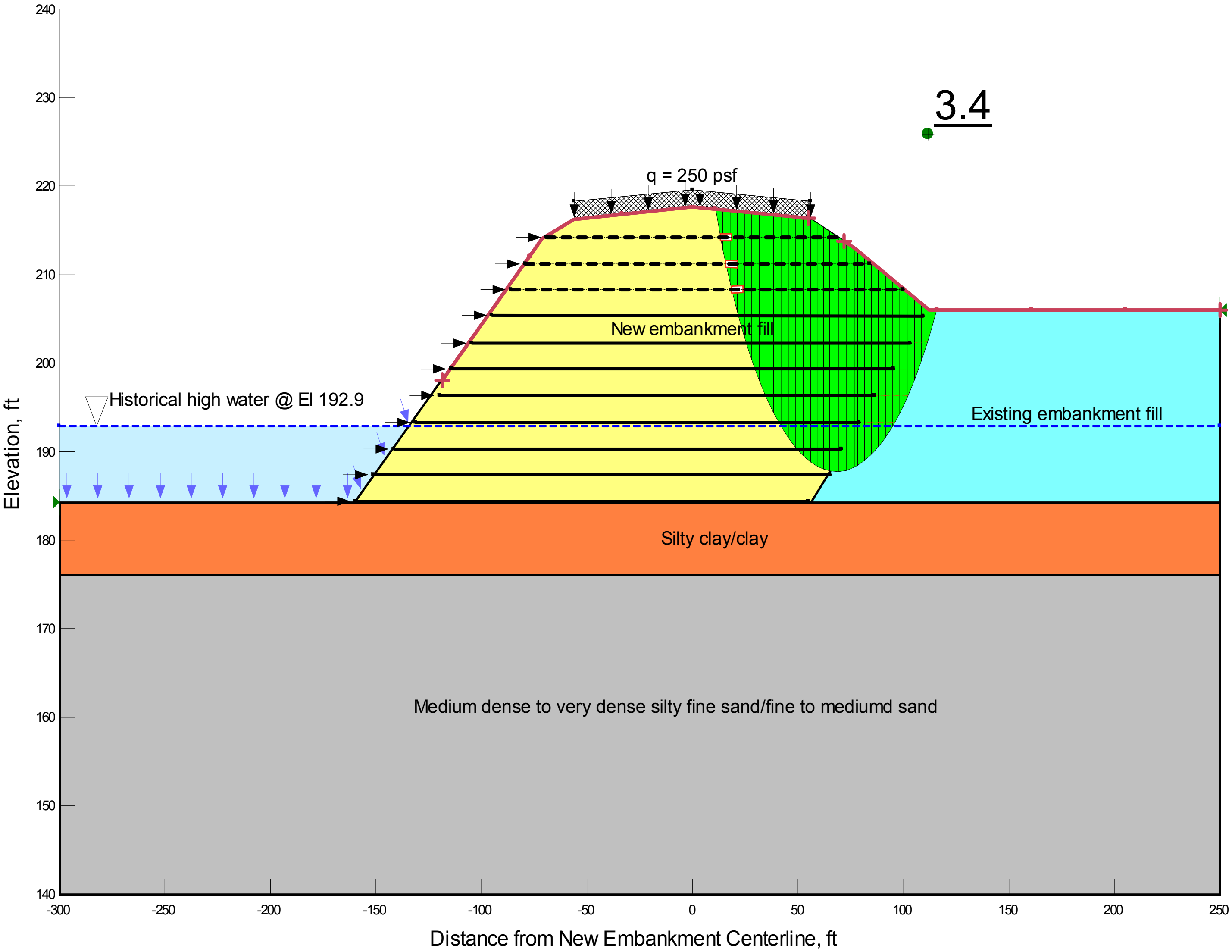
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



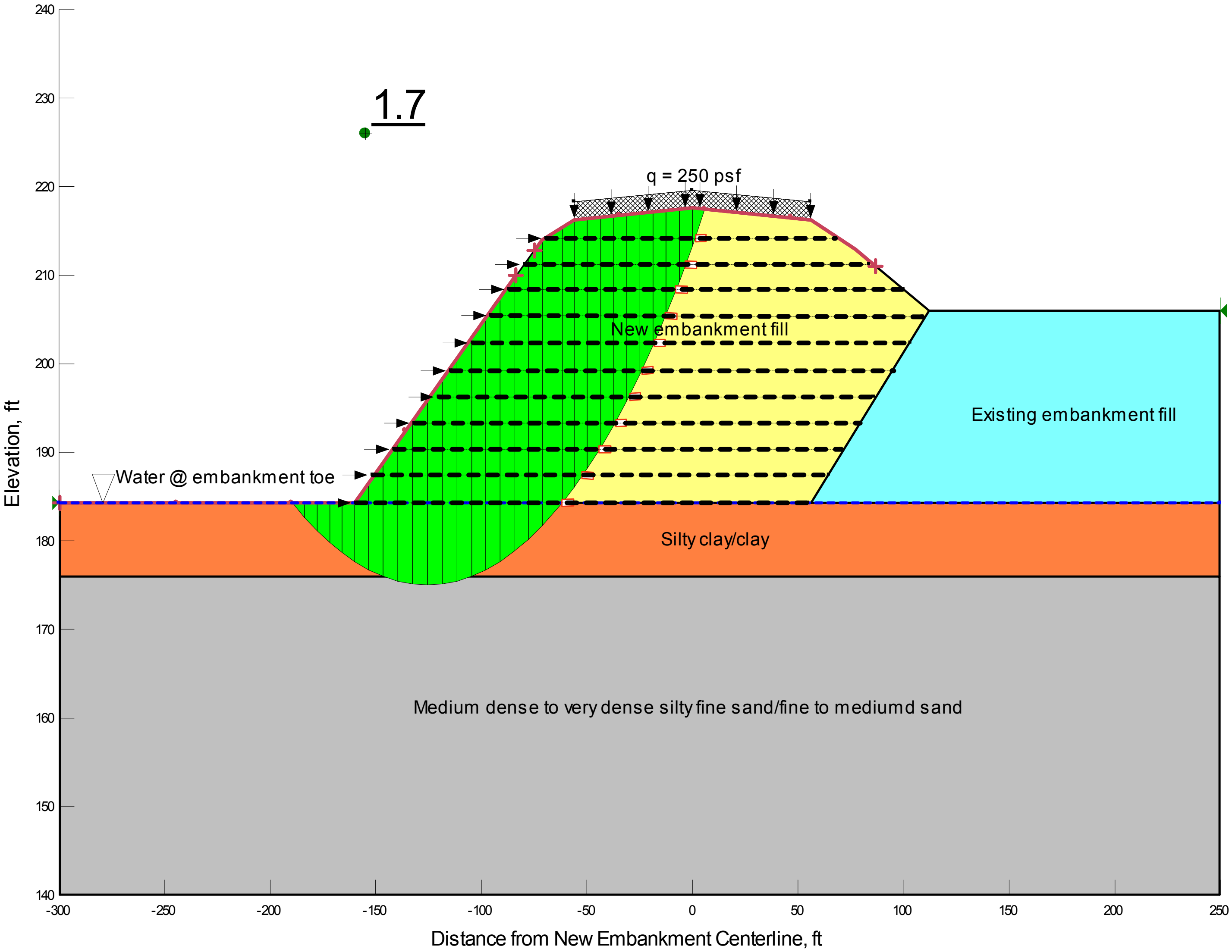
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



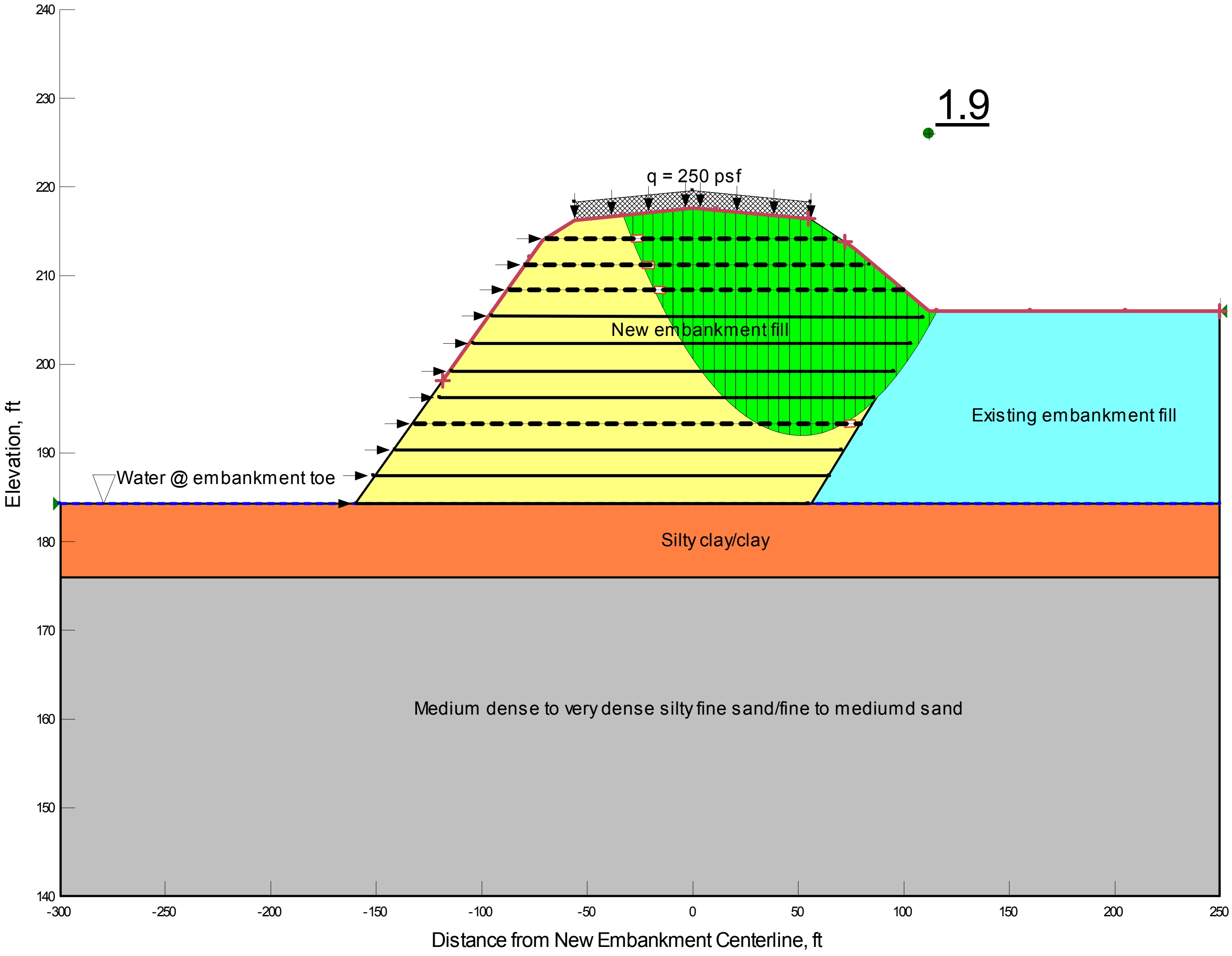
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



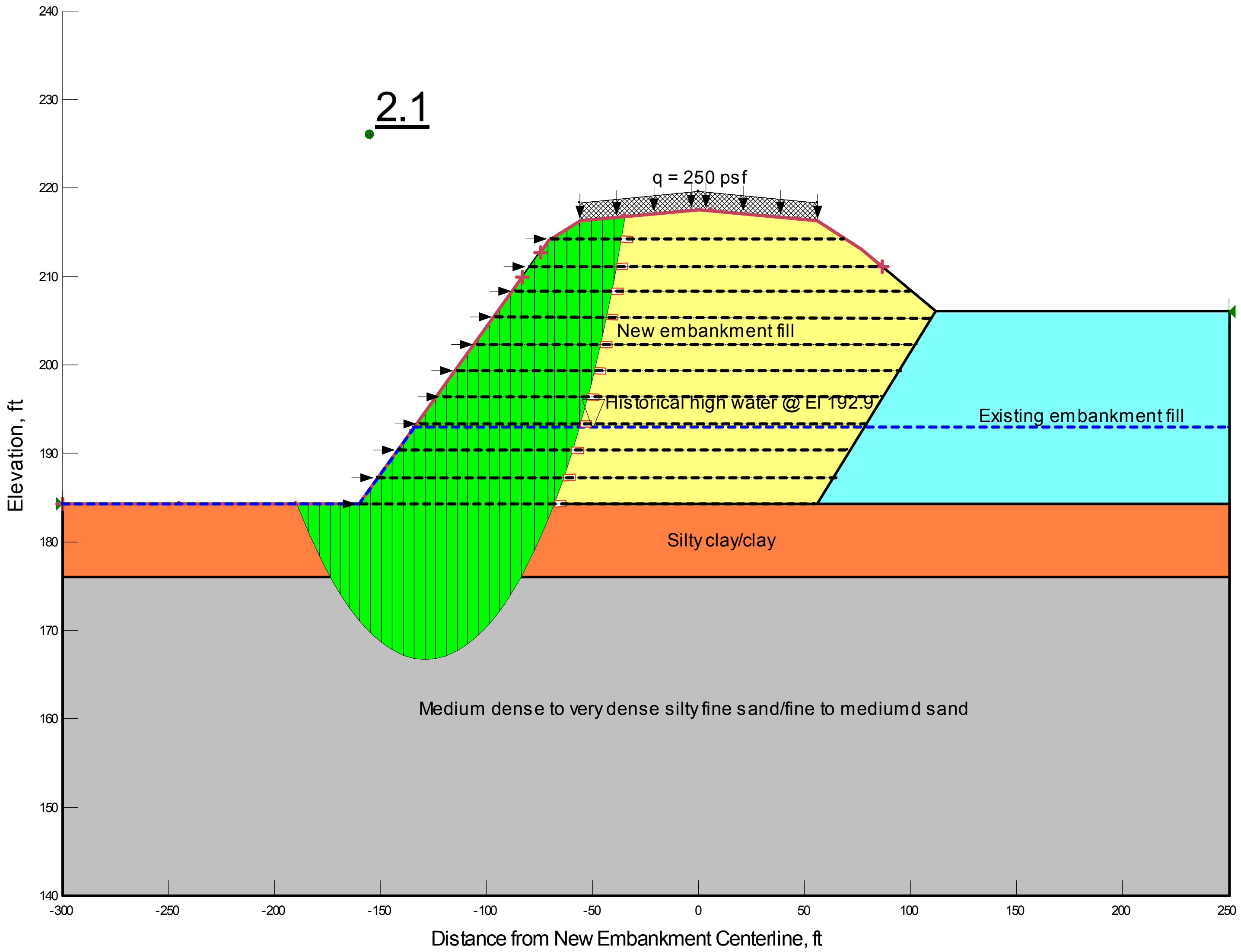
Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

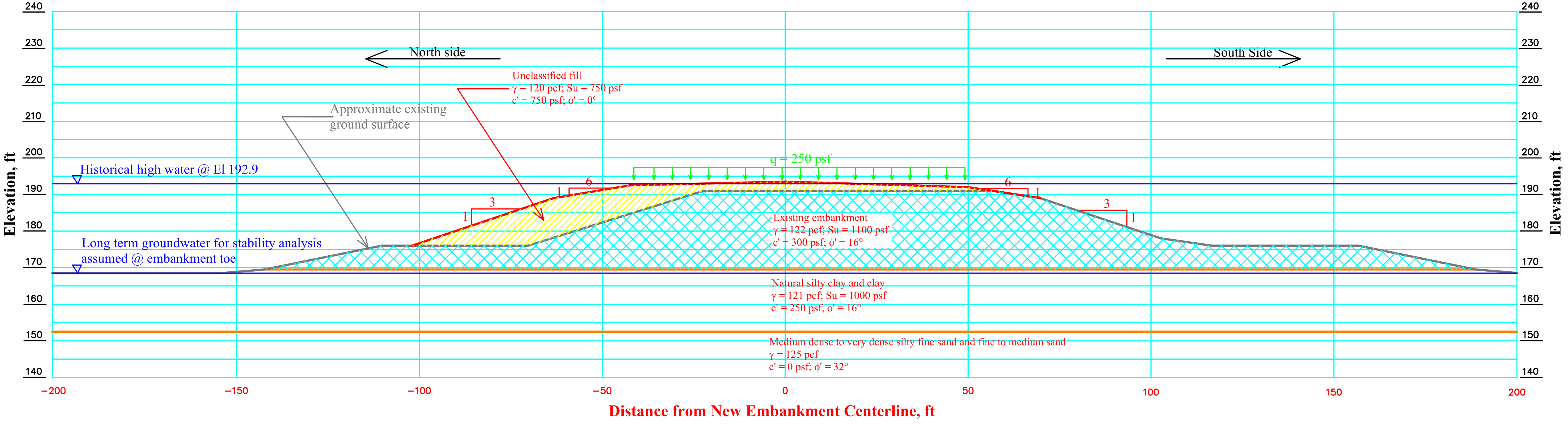
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX N



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

Section and Material Parameters for Stability Analysis
Unreinforced Side Slopes @ Sta 1875+00
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

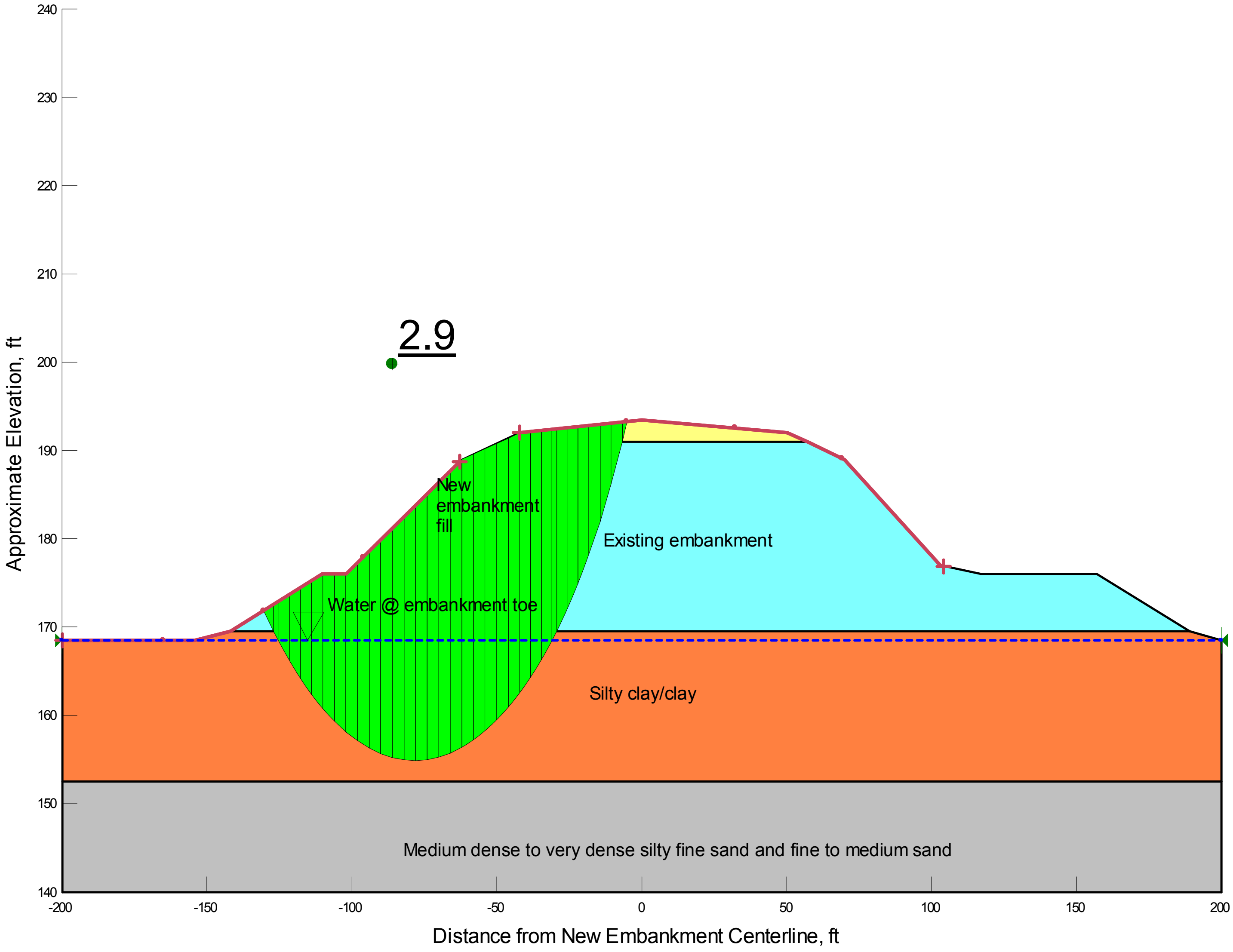
Scale: As Shown

May 2, 2014

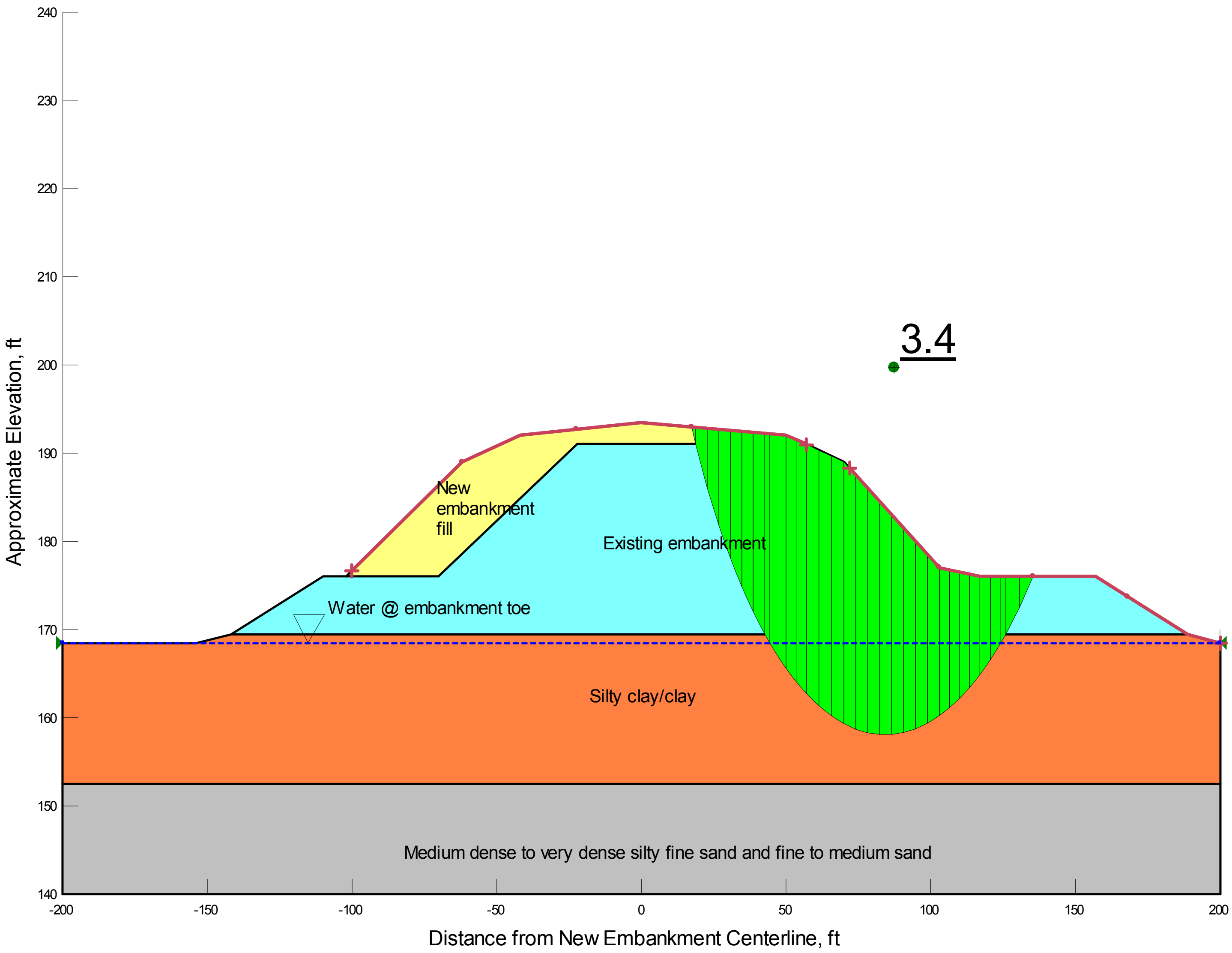
Plate

Stability Analysis Results
Unreinforced Side Slopes @ Sta 1875+00
AHTD JOB BB0610:White River Str. & Apprs. (F)

Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.9
South			3.4
North	Long Term	Analysis long term groundwater @ embankment toe	2.2
South			2.4
North		Historical high water @ El 192.9	3.0
South			3.2
North	Seismic ($k_h = 1.0A_S = 0.19$)	Analysis long term groundwater @ embankment toe	1.1
South			1.2
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.6
South			1.7

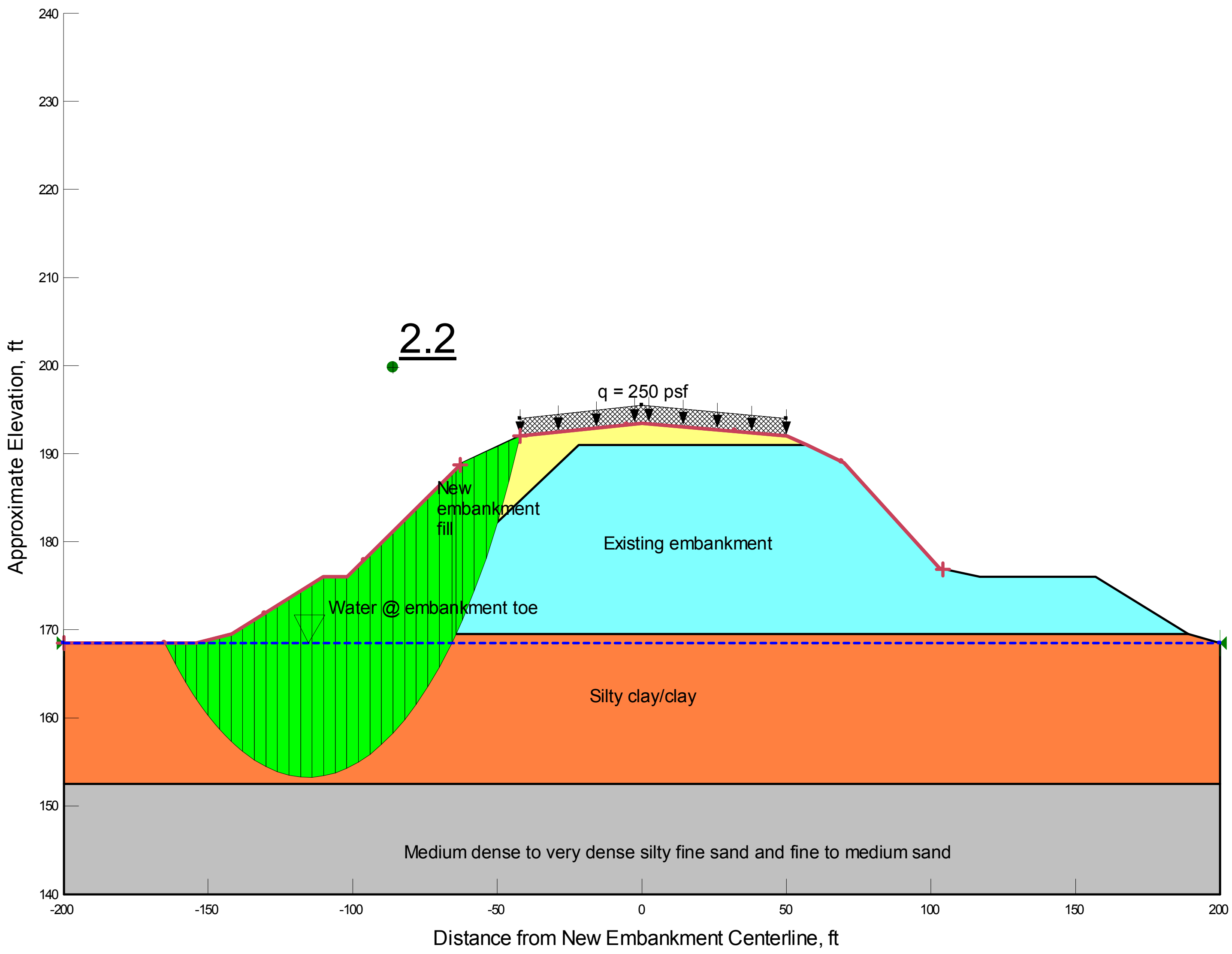


Results of Stability Analyses – End of Construction Condition
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

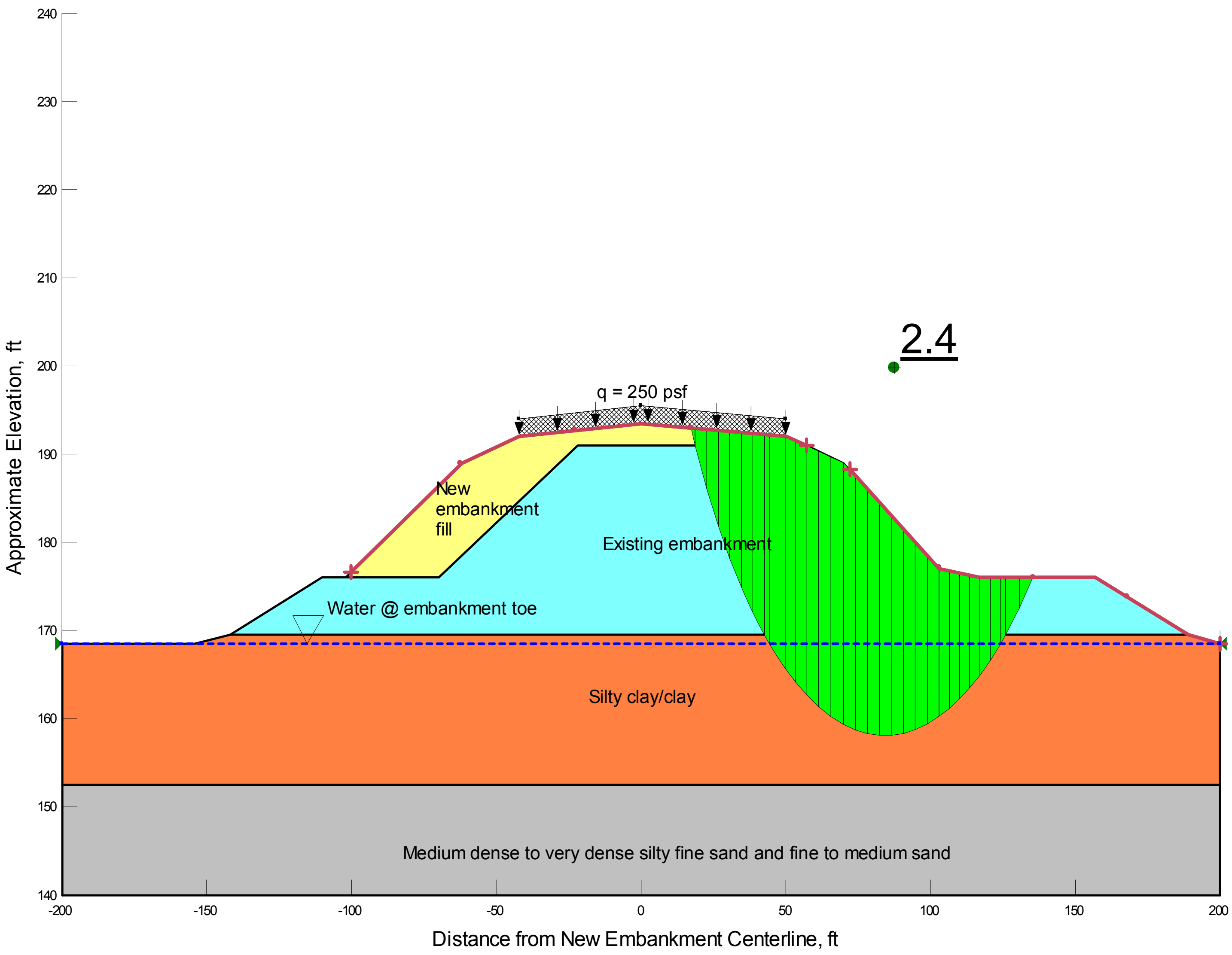


Results of Stability Analyses – End of Construction Condition
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

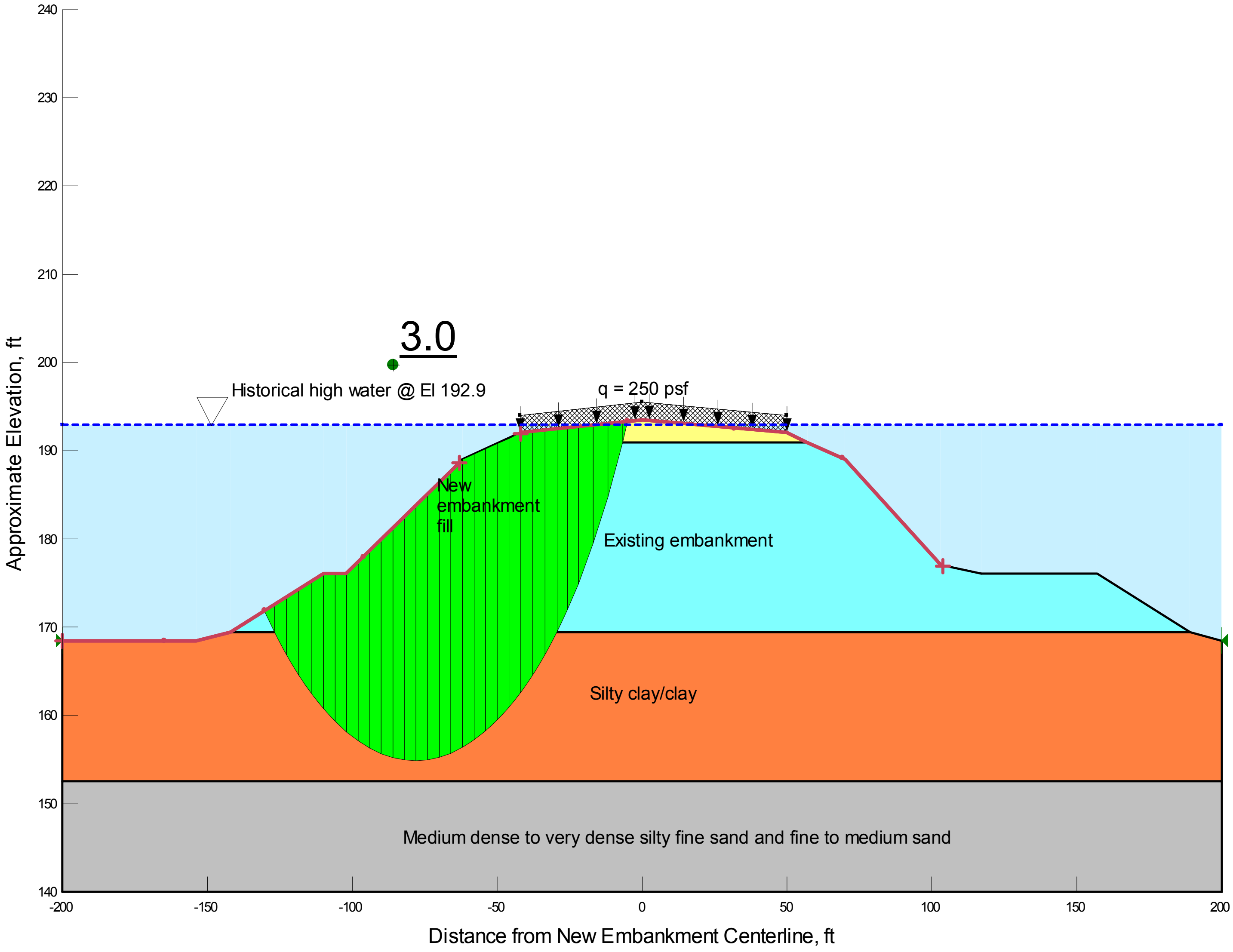


Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



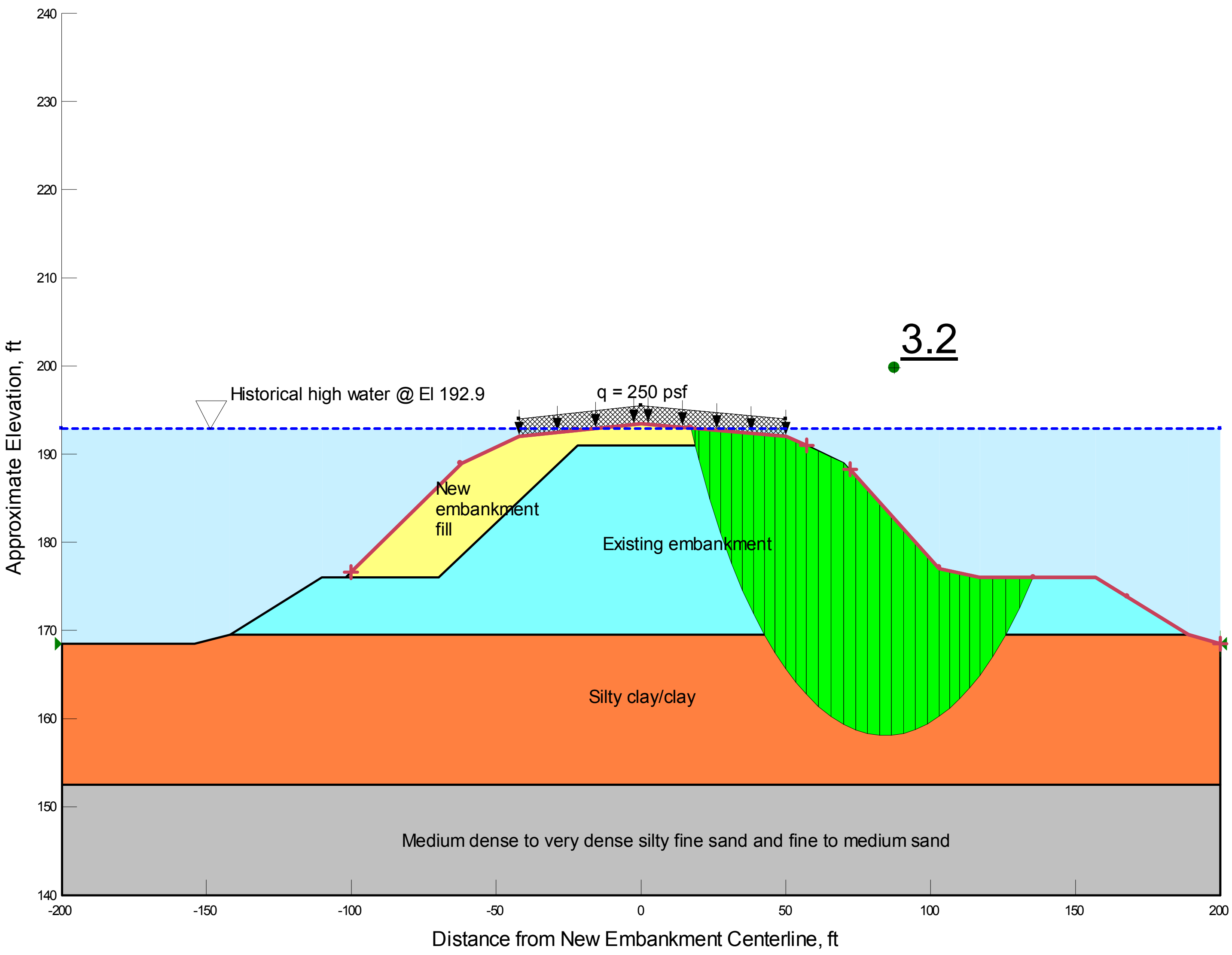
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



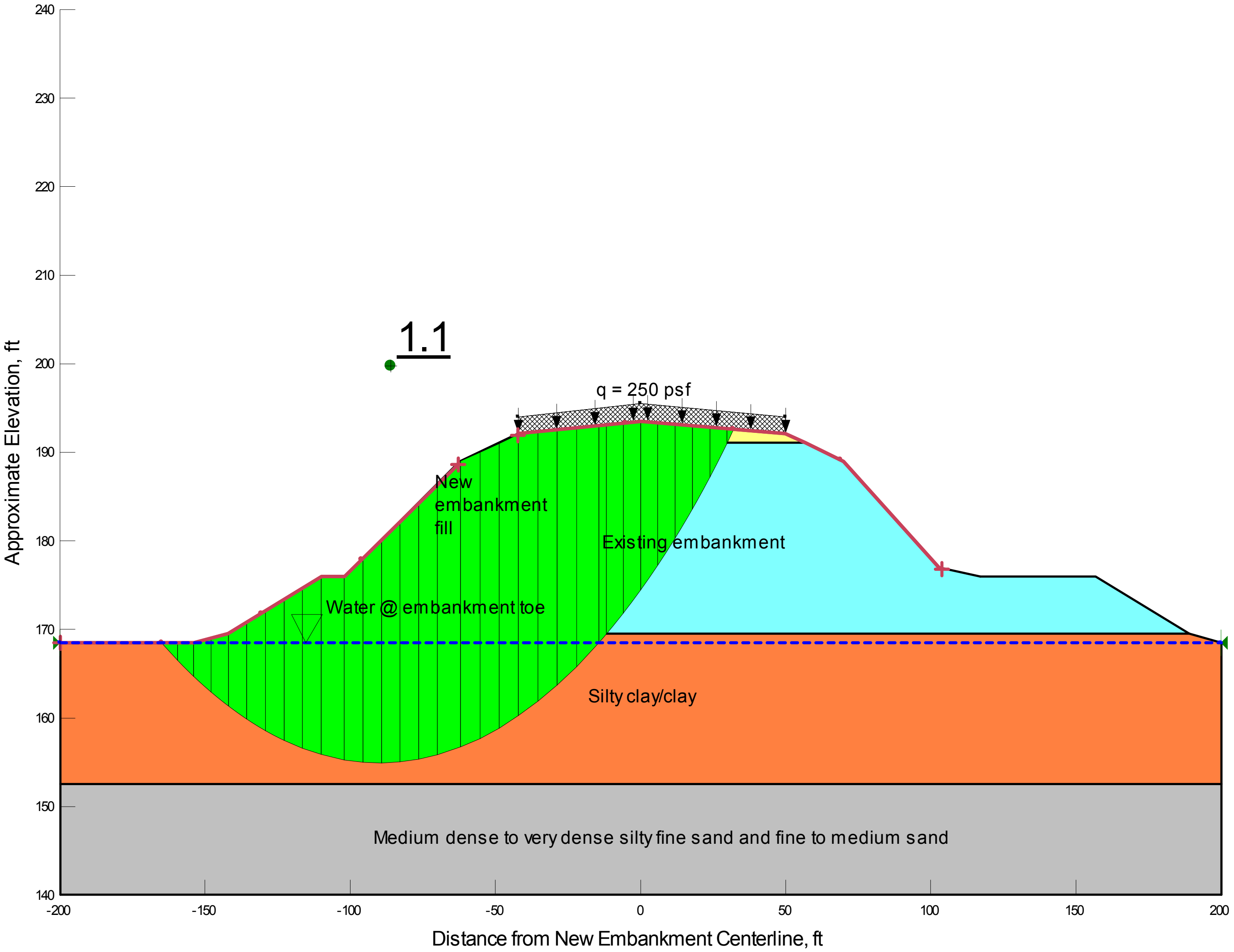
Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1875+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



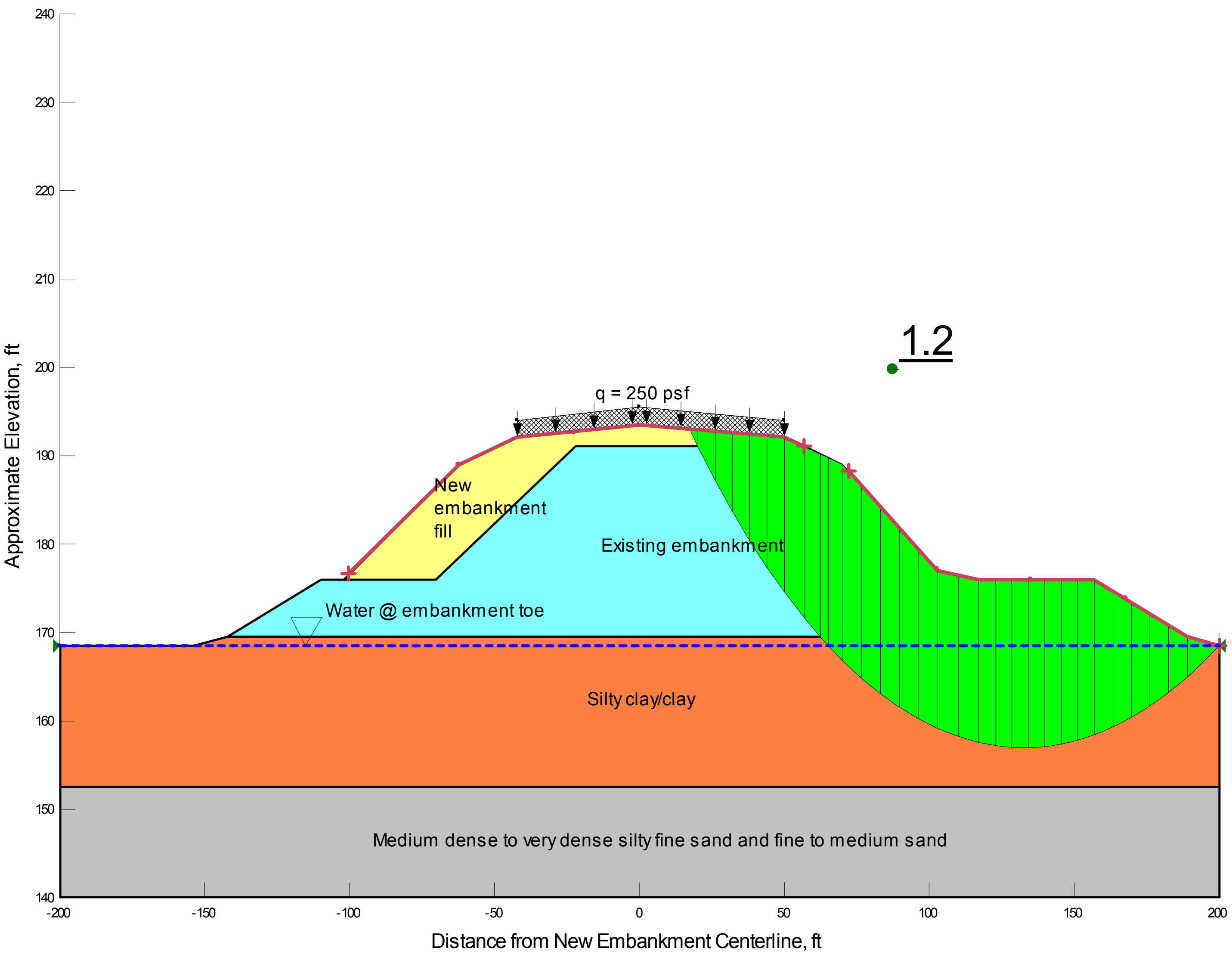
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1875+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



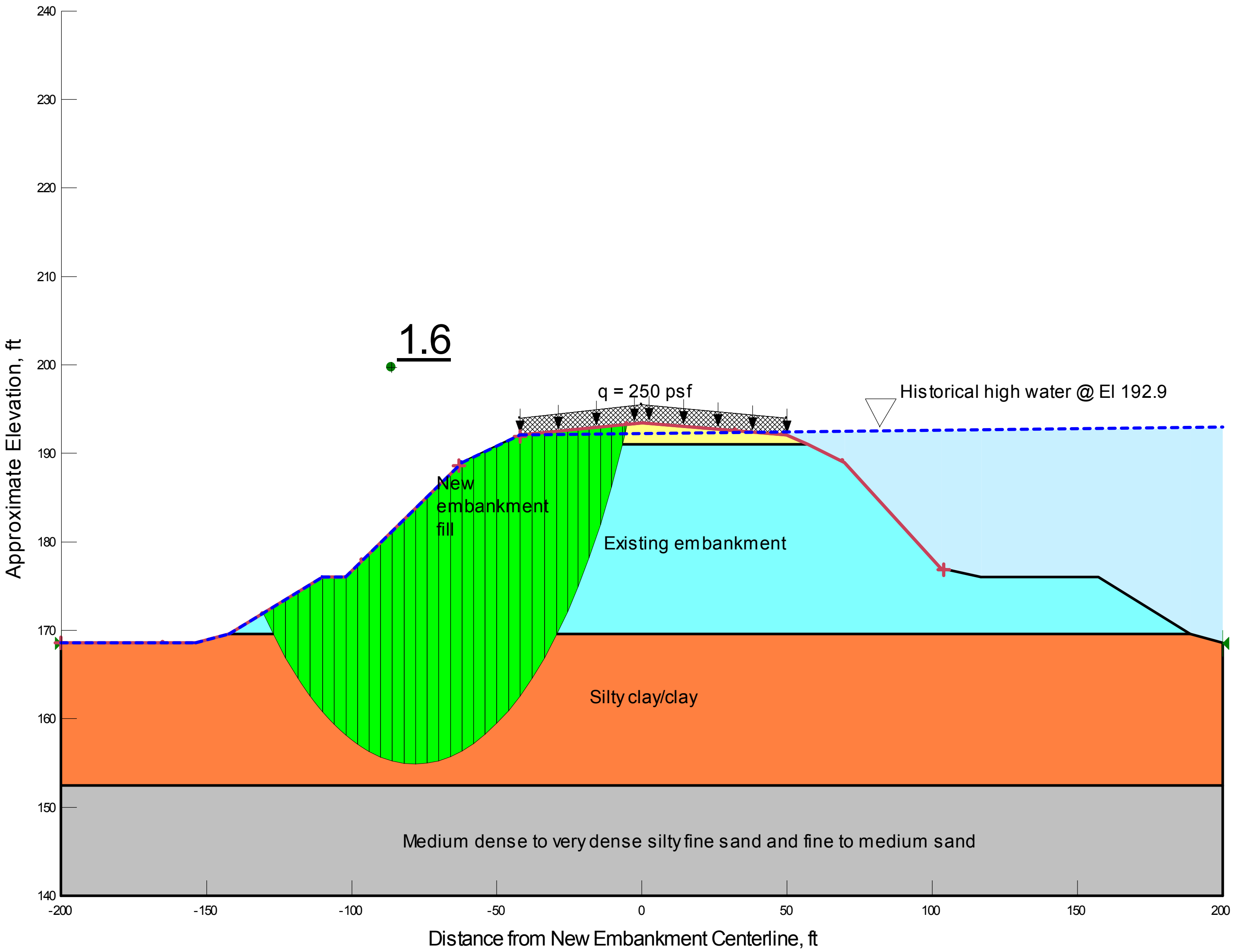
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



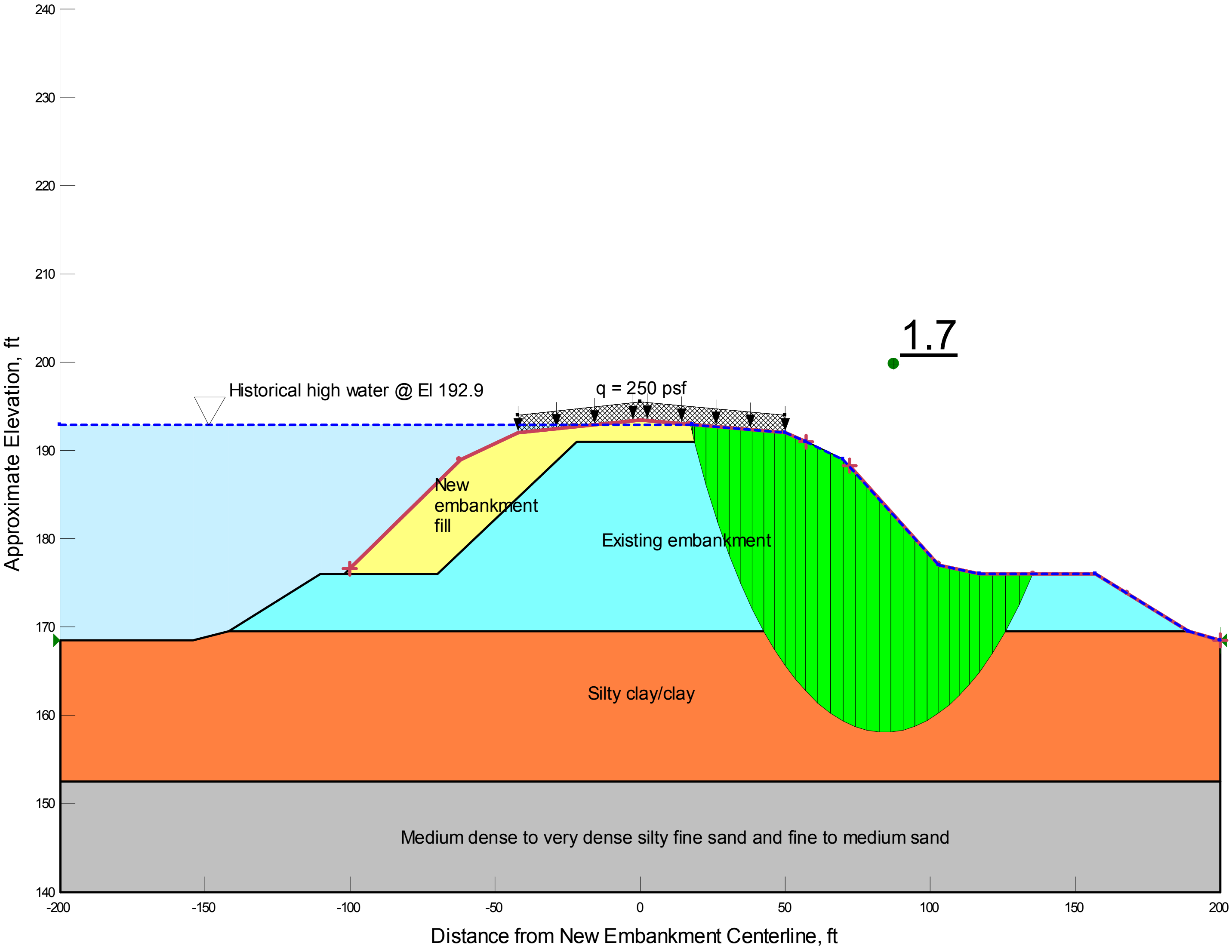
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced North Side Slope @ Sta 1875+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

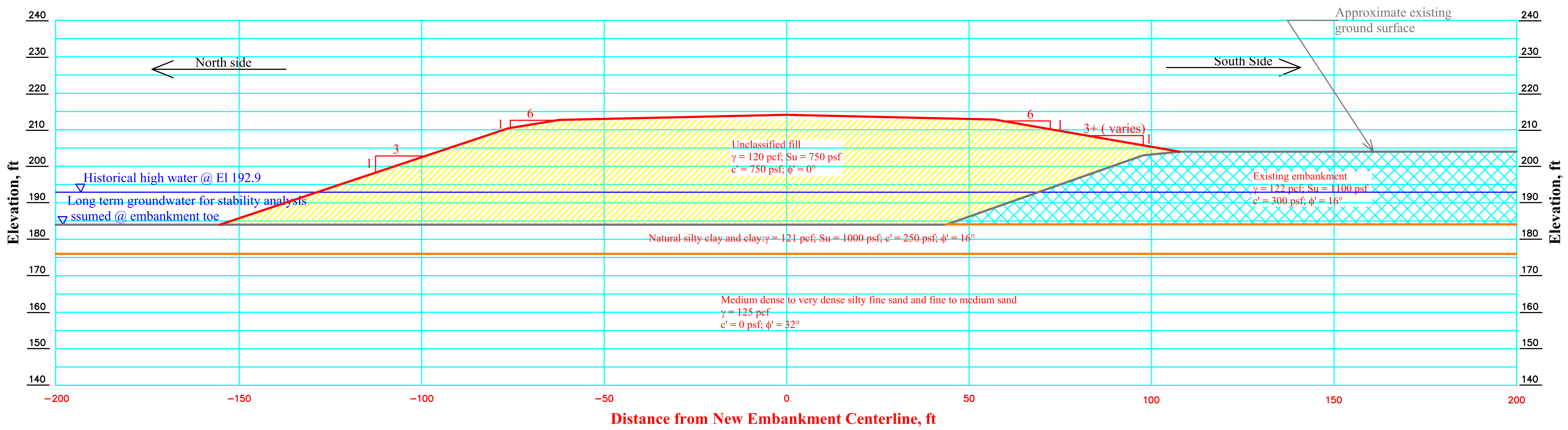
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced South Side Slope @ Sta 1875+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX O



15 ft 0 15 30 ft

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

Section and Material Parameters for Stability Analysis
Unreinforced Side Slopes @ Sta 1913+00
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

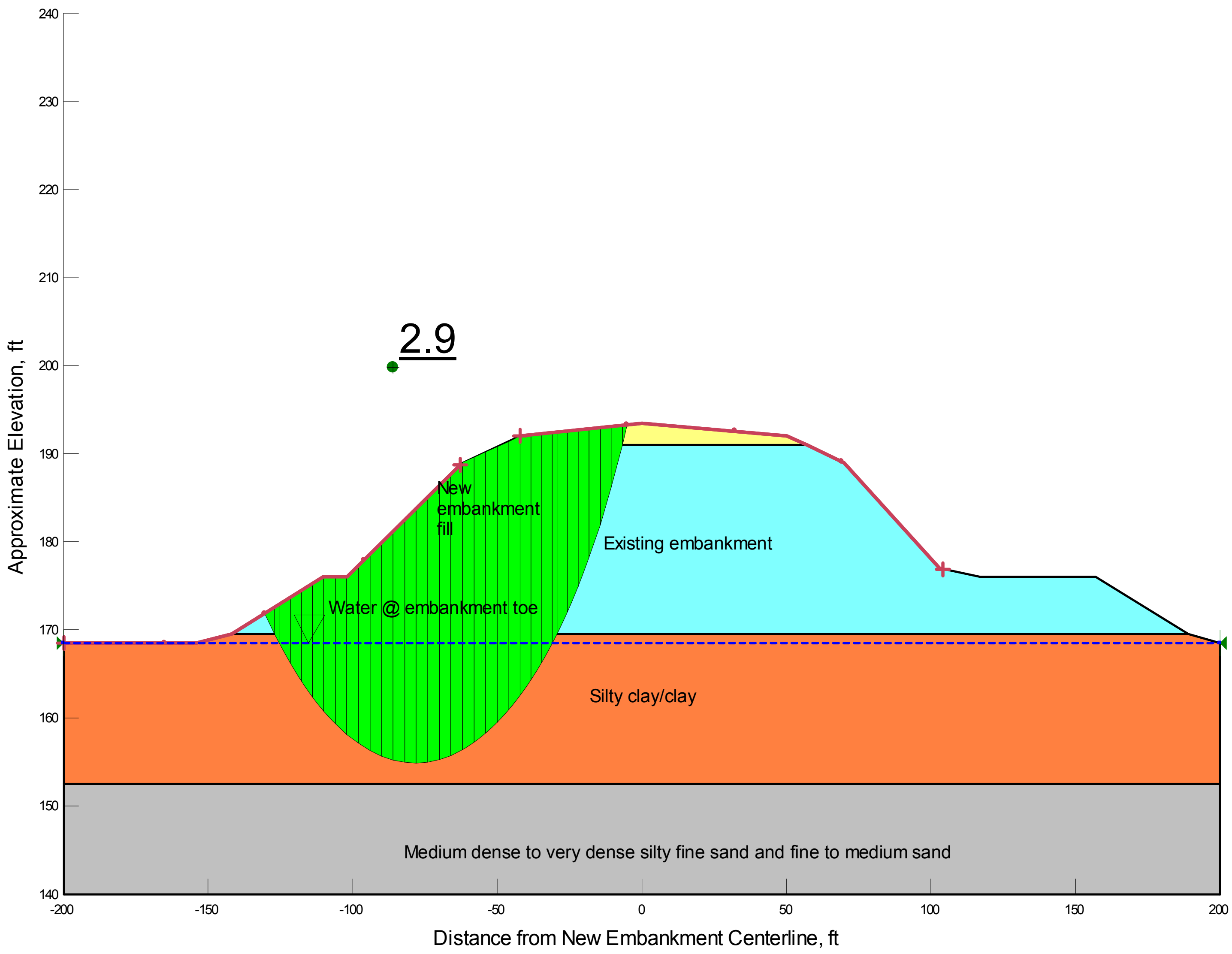
Scale: As Shown

May 5, 2014

Plate

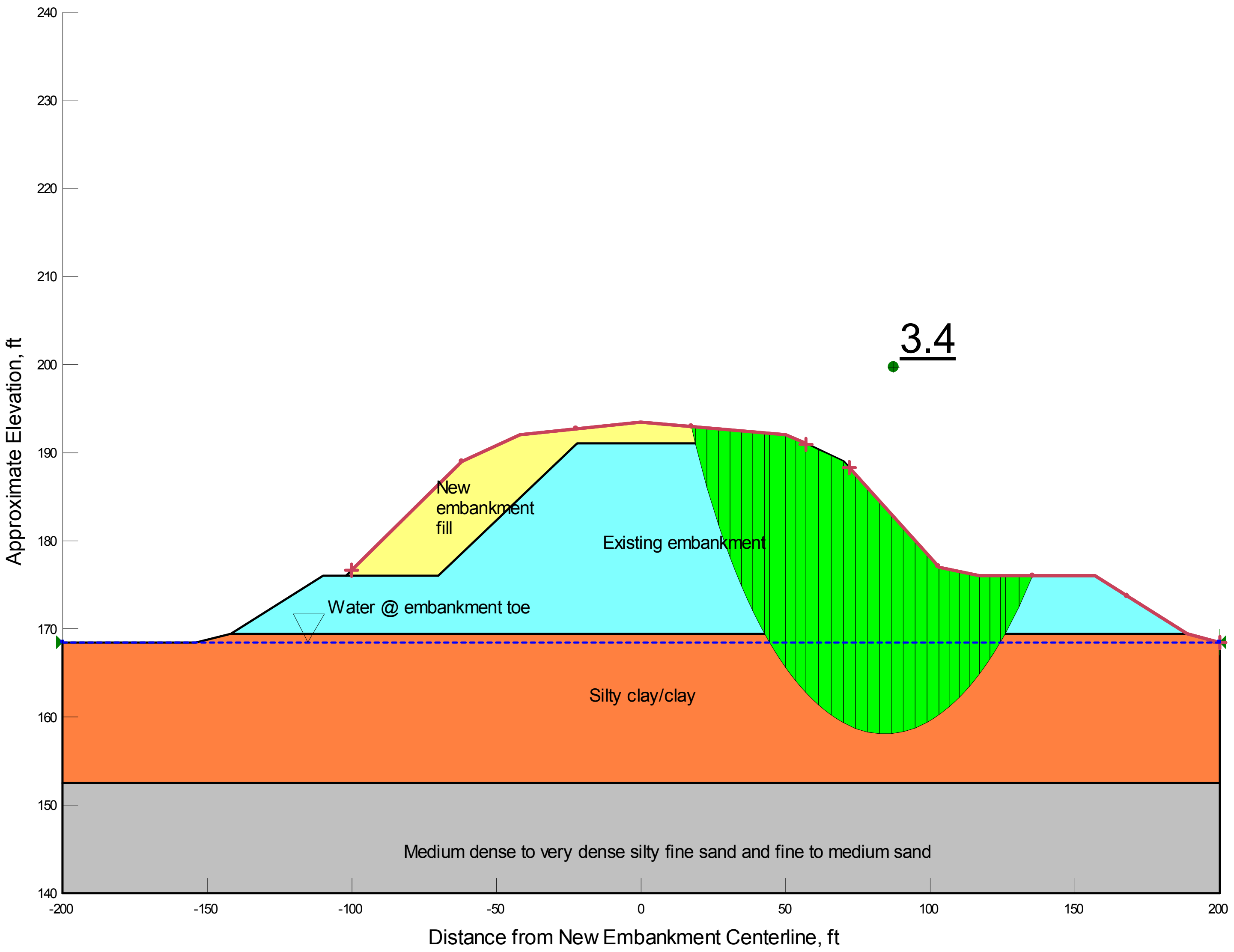
Stability Analysis Results
Unreinforced Side Slopes @ Sta 1913+00
AHTD JOB BB0610:White River Str. & Apprs. (F)

Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.9
South			3.4
North	Long Term	Analysis long term groundwater @ embankment toe	1.8
South			4.3
North		Historical high water @ El 192.9	1.8
South			3.9
North	Seismic ($k_h = 1.0A_S = 0.19$)	Analysis long term groundwater @ embankment toe	1.1
South			1.6
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.6



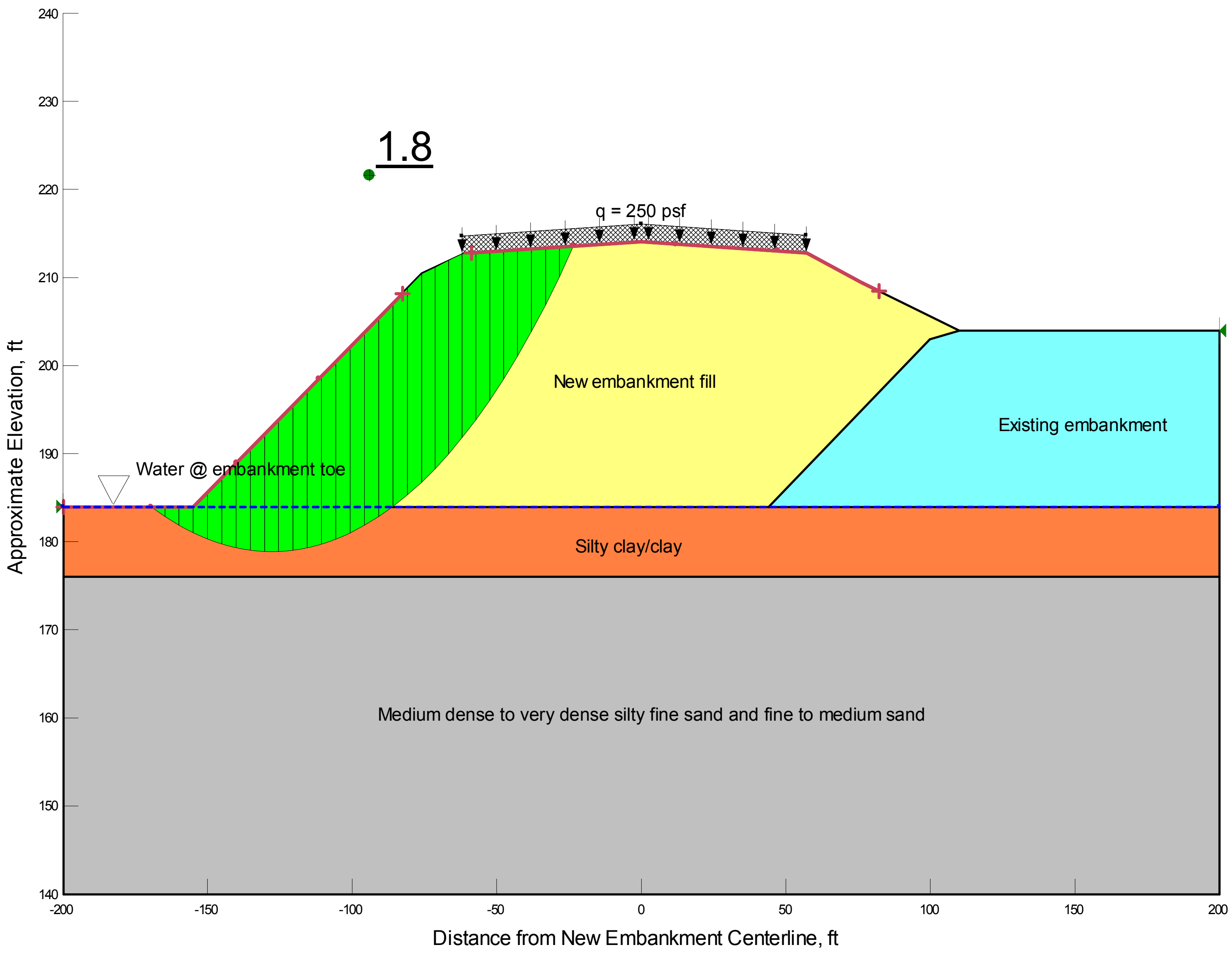
Results of Stability Analyses – End of Construction Condition
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



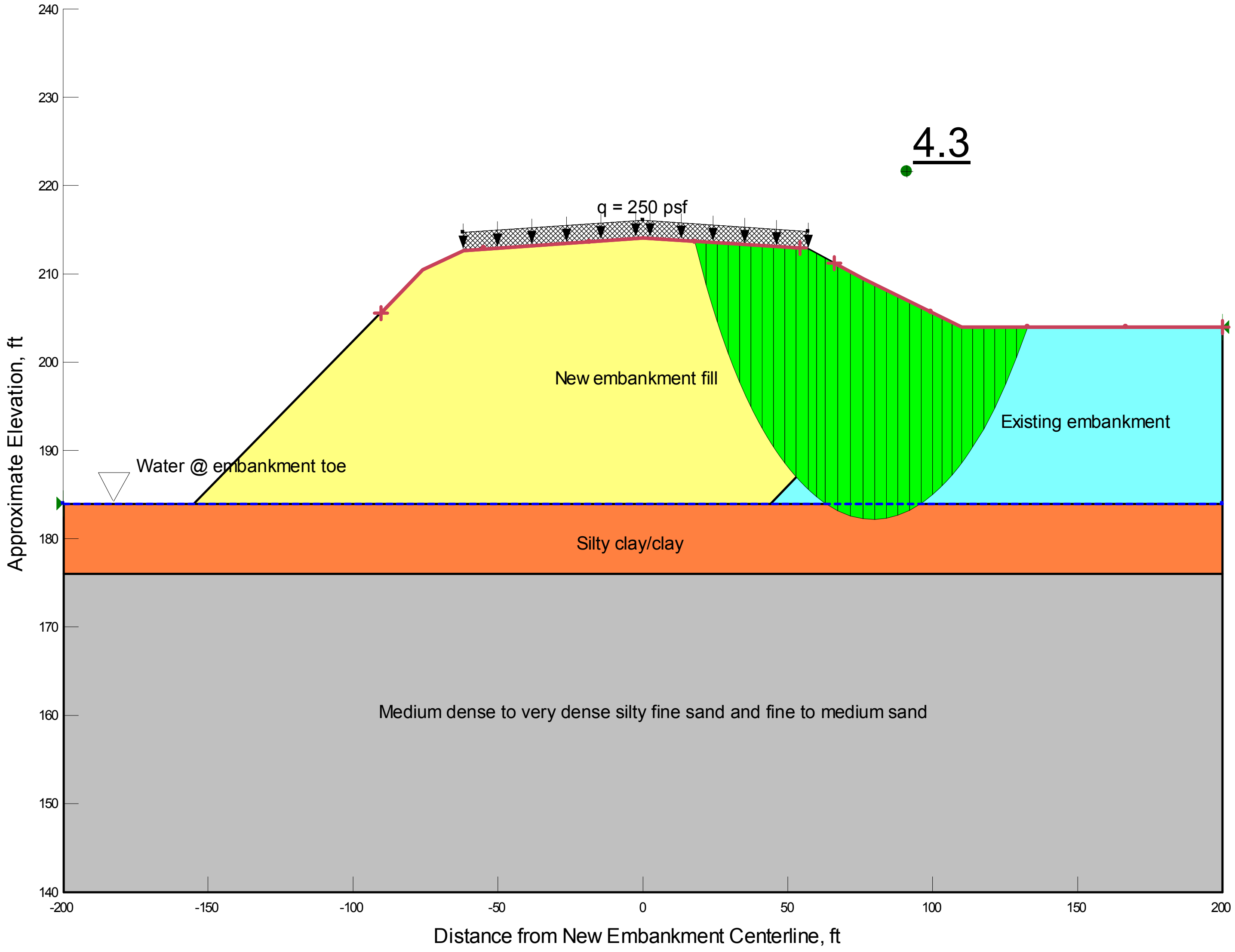
Results of Stability Analyses – End of Construction Condition
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



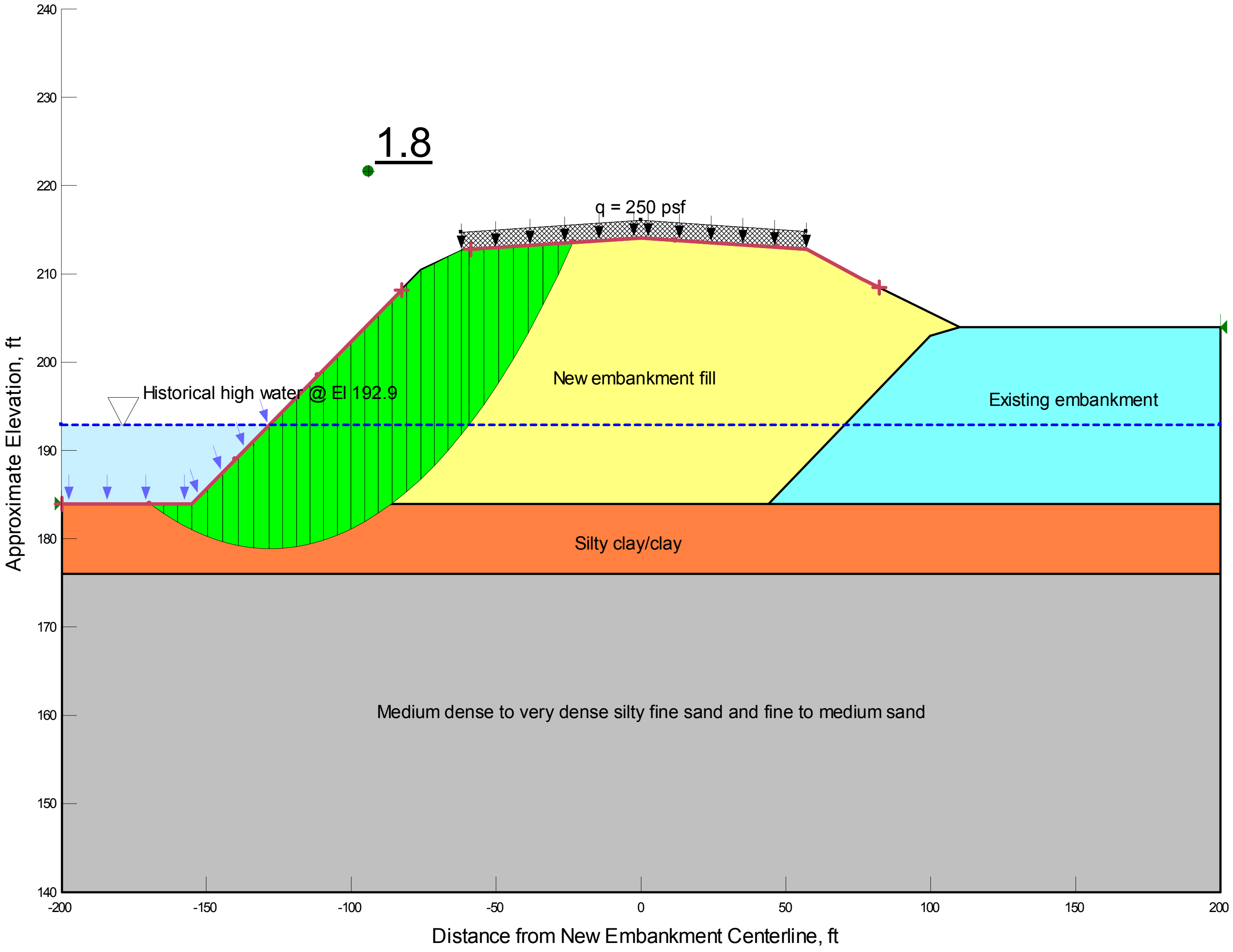
Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



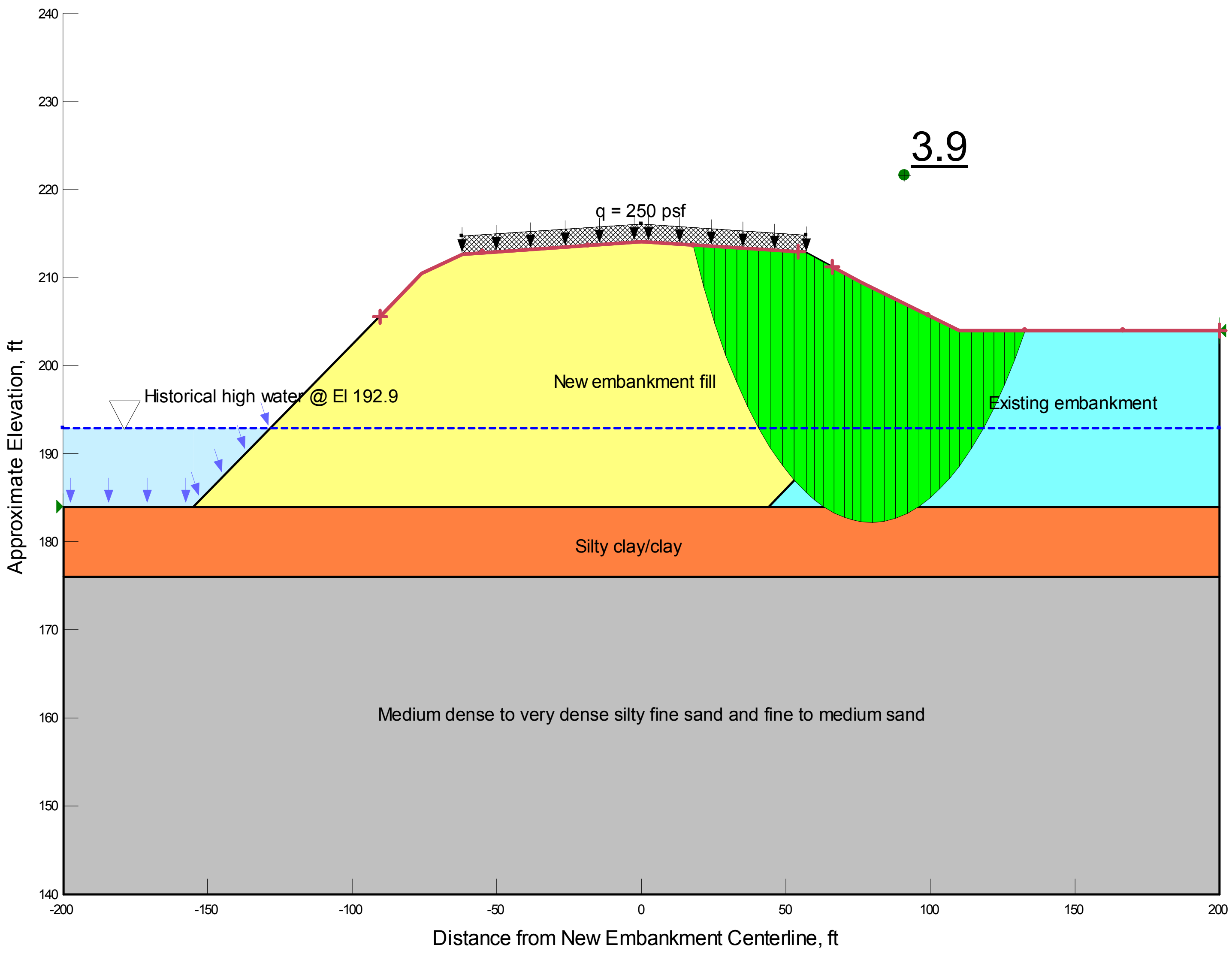
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



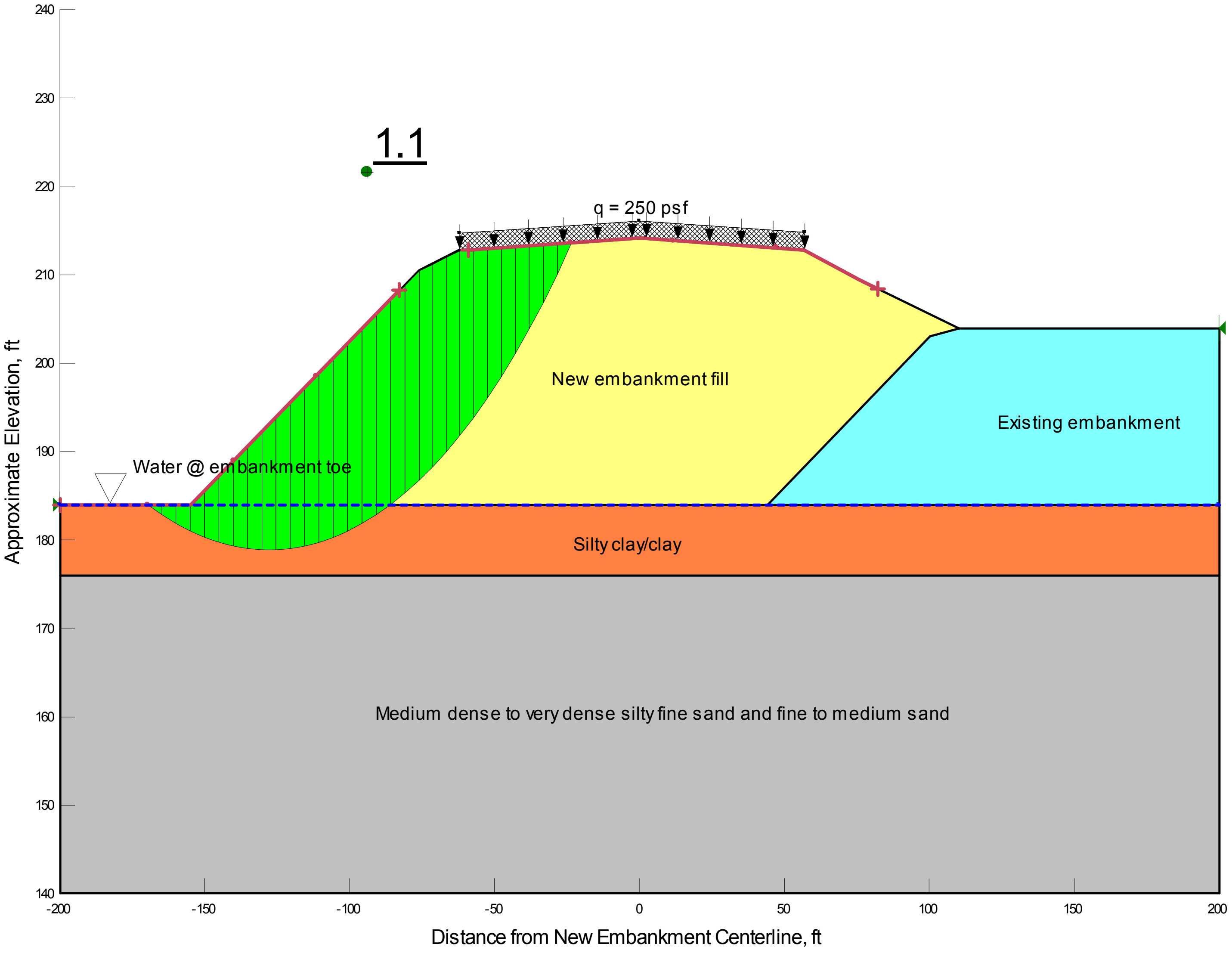
Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1913+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



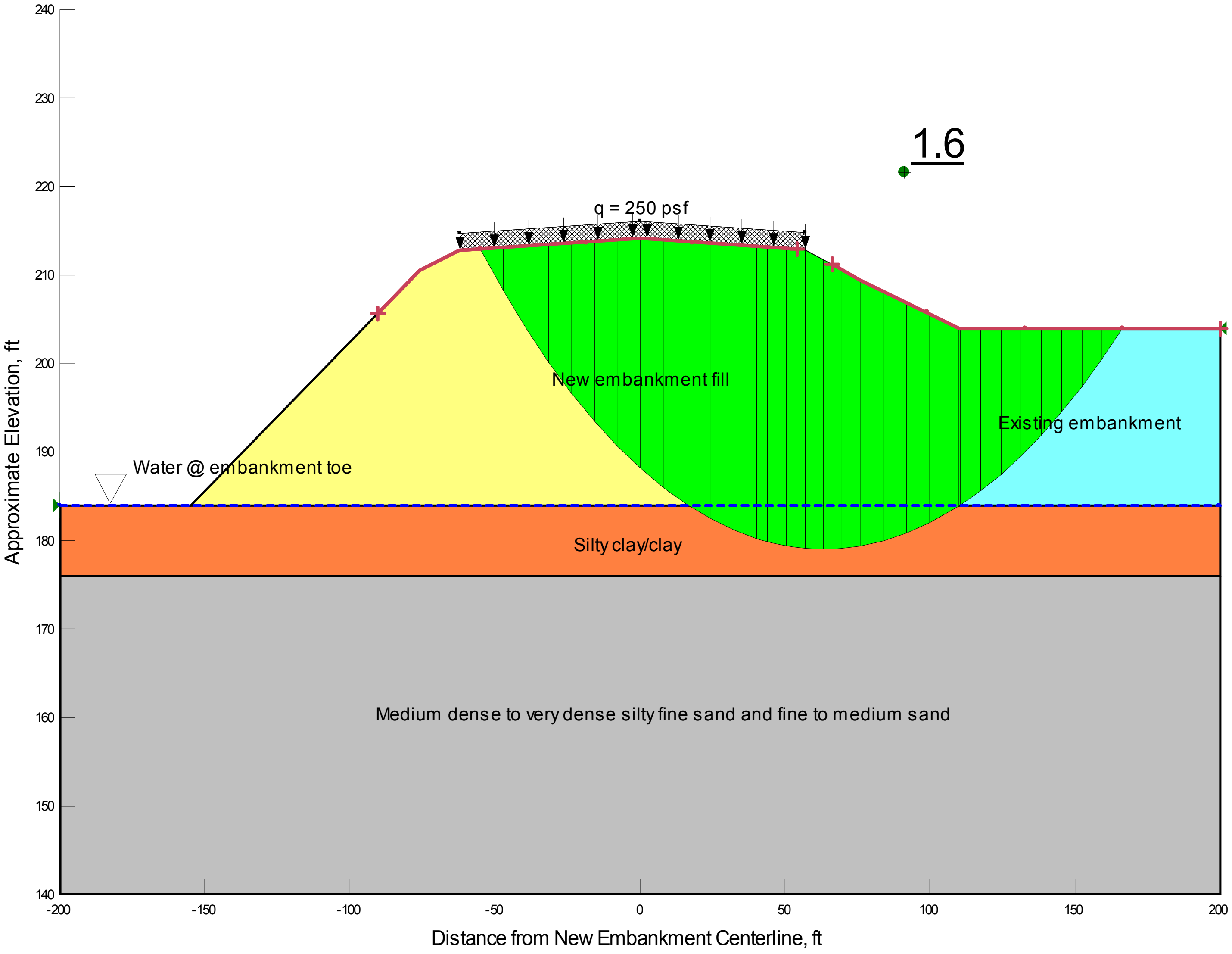
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1913+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

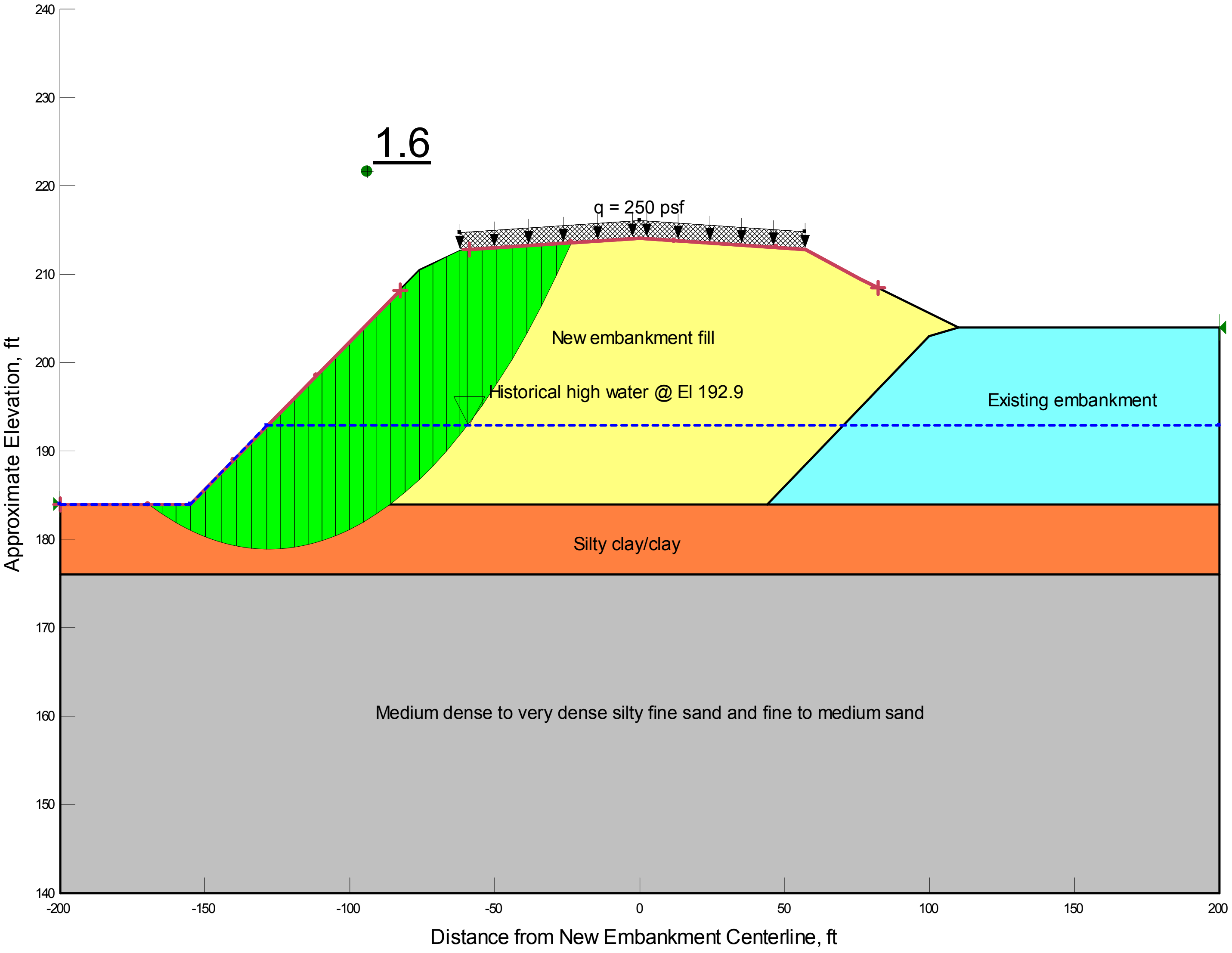


Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



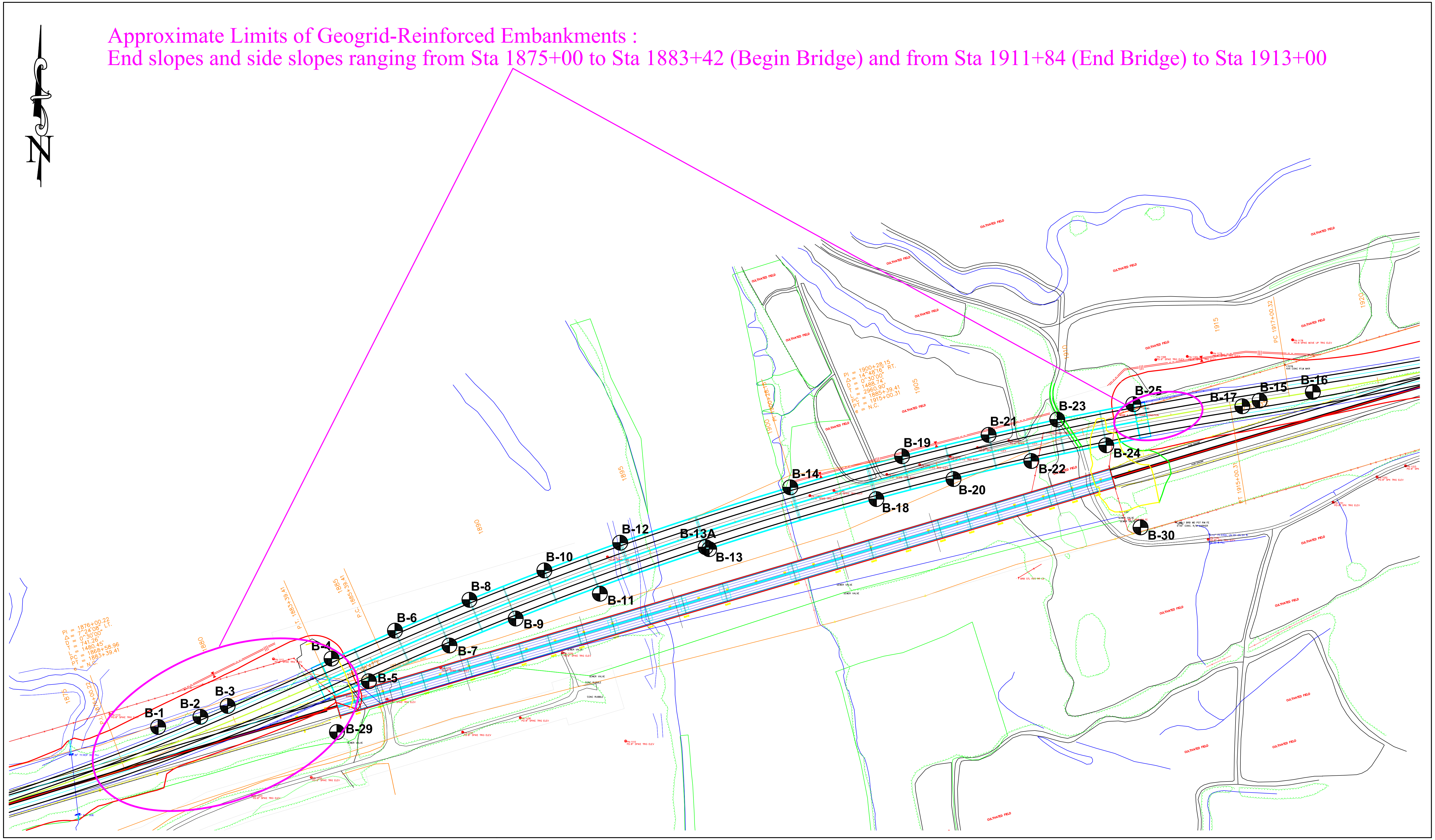
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced North Side Slope @ Sta 1913+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX P


Approximate Limits of Geogrid-Reinforced Embankments :
End slopes and side slopes ranging from Sta 1875+00 to Sta 1883+42 (Begin Bridge) and from Sta 1911+84 (End Bridge) to Sta 1913+00

Note: Plan provided by Jacobs.

150 ft 0 150 300 ft

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

Recommended Limits of Geogrid-Reinforced Embankments
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

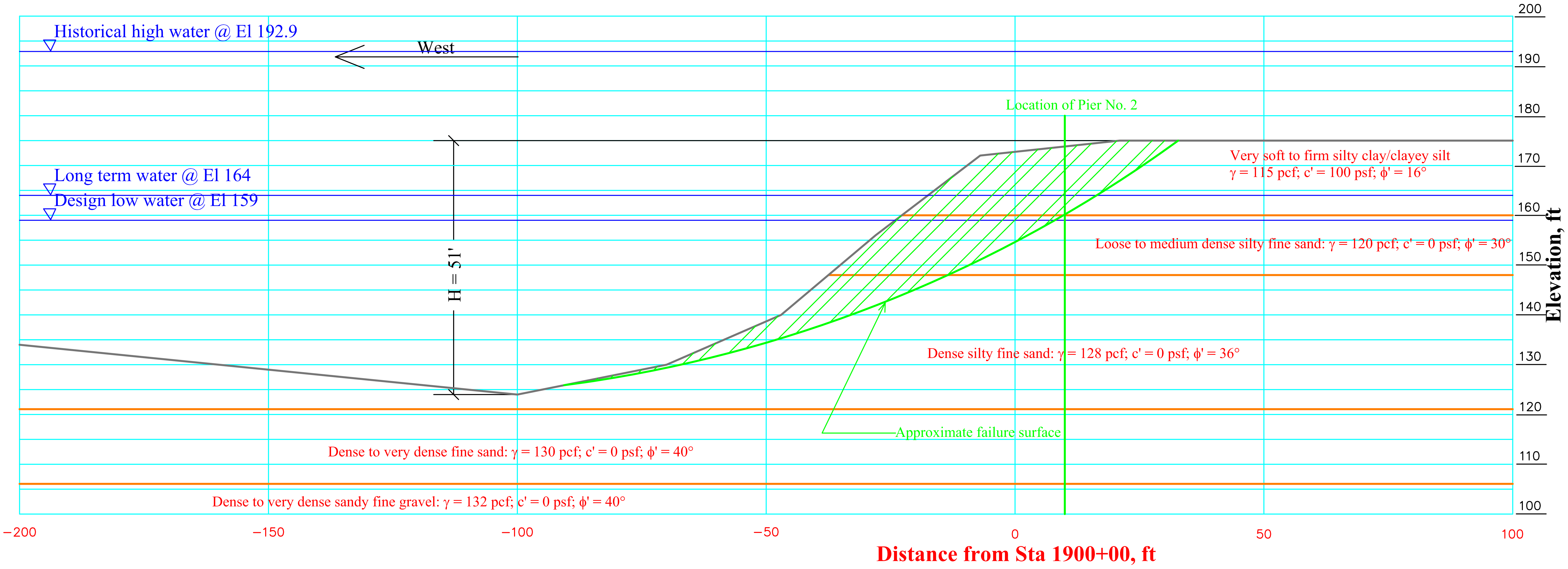
Job No.: 13-017

Scale: As Shown

April 29, 2014

Plate 2a

APPENDIX Q



10 ft 0 10 20 ft

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

Section and Material Parameters for Stability Analysis
Existing East River Bank near Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

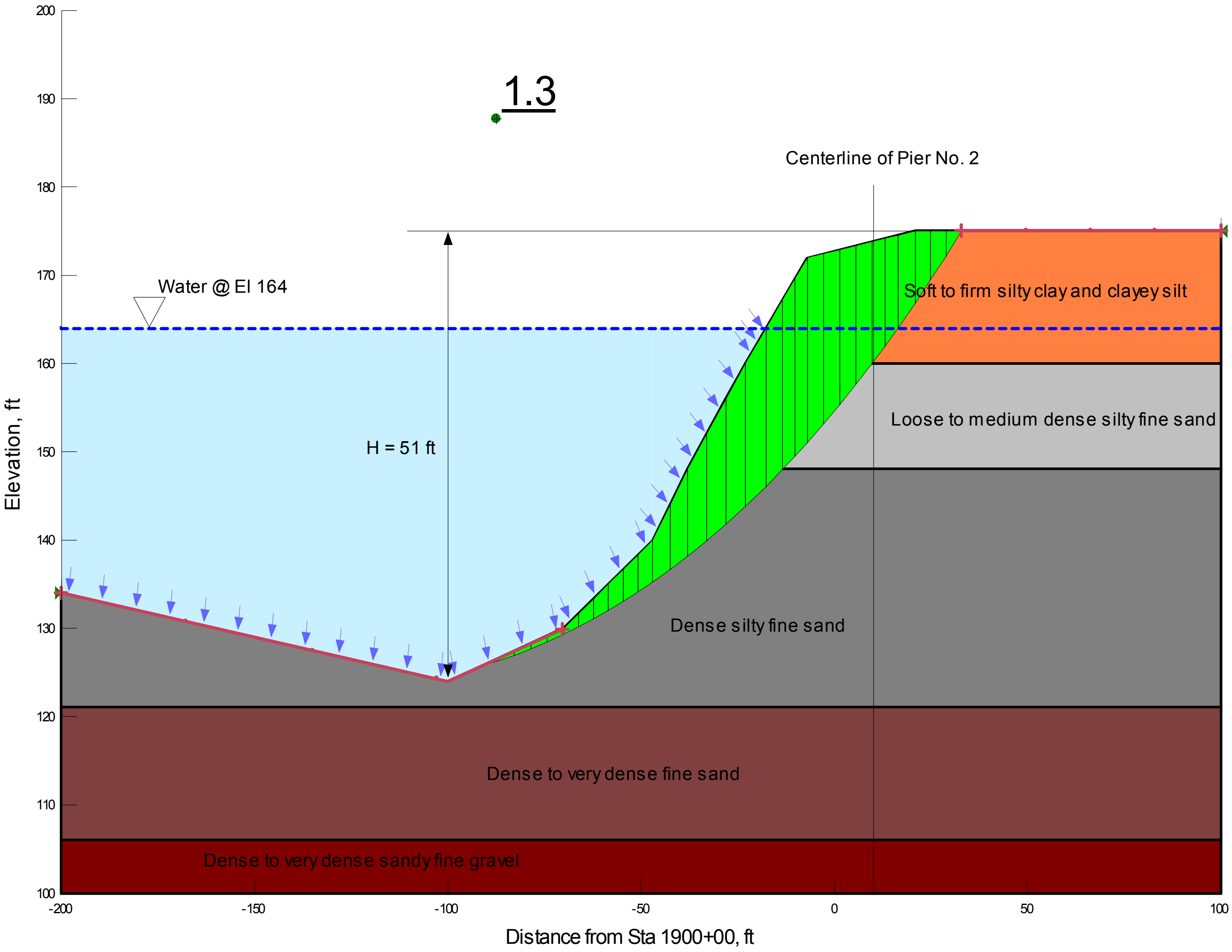
Scale: As Shown

May 8, 2014

Plate

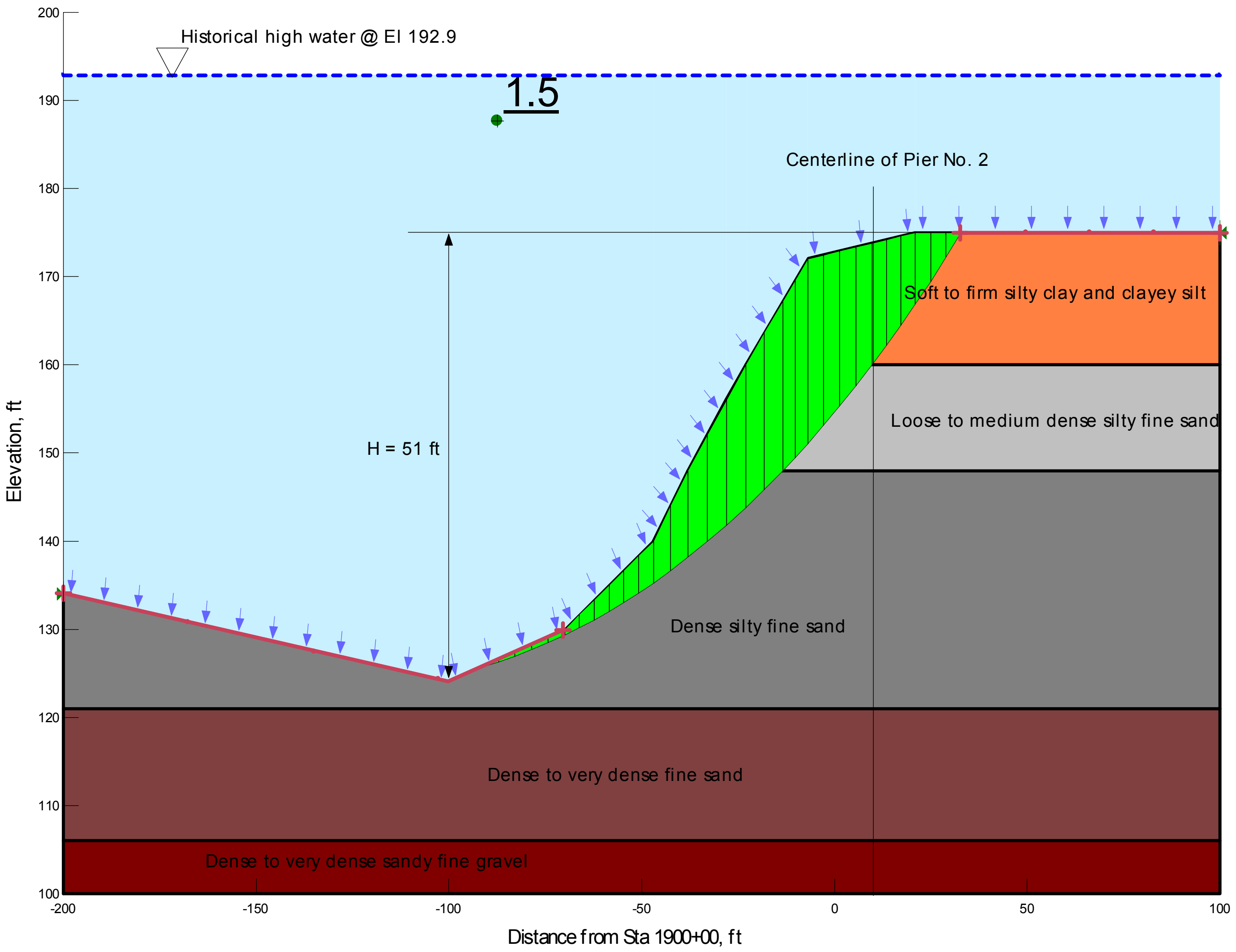
Stability Analysis Results – East Abutment
Existing East River Bank near Pier No. 2 – H = 51 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
Long Term	Water @ El 164	1.3
Long Term	Historical high water @ El 192.9	1.5
Seismic (k_h = 1.0A_S = 0.19)*	Water @ El 164	0.7
Rapid Drawdown	Drawdown to El 159 (design low water)	0.8



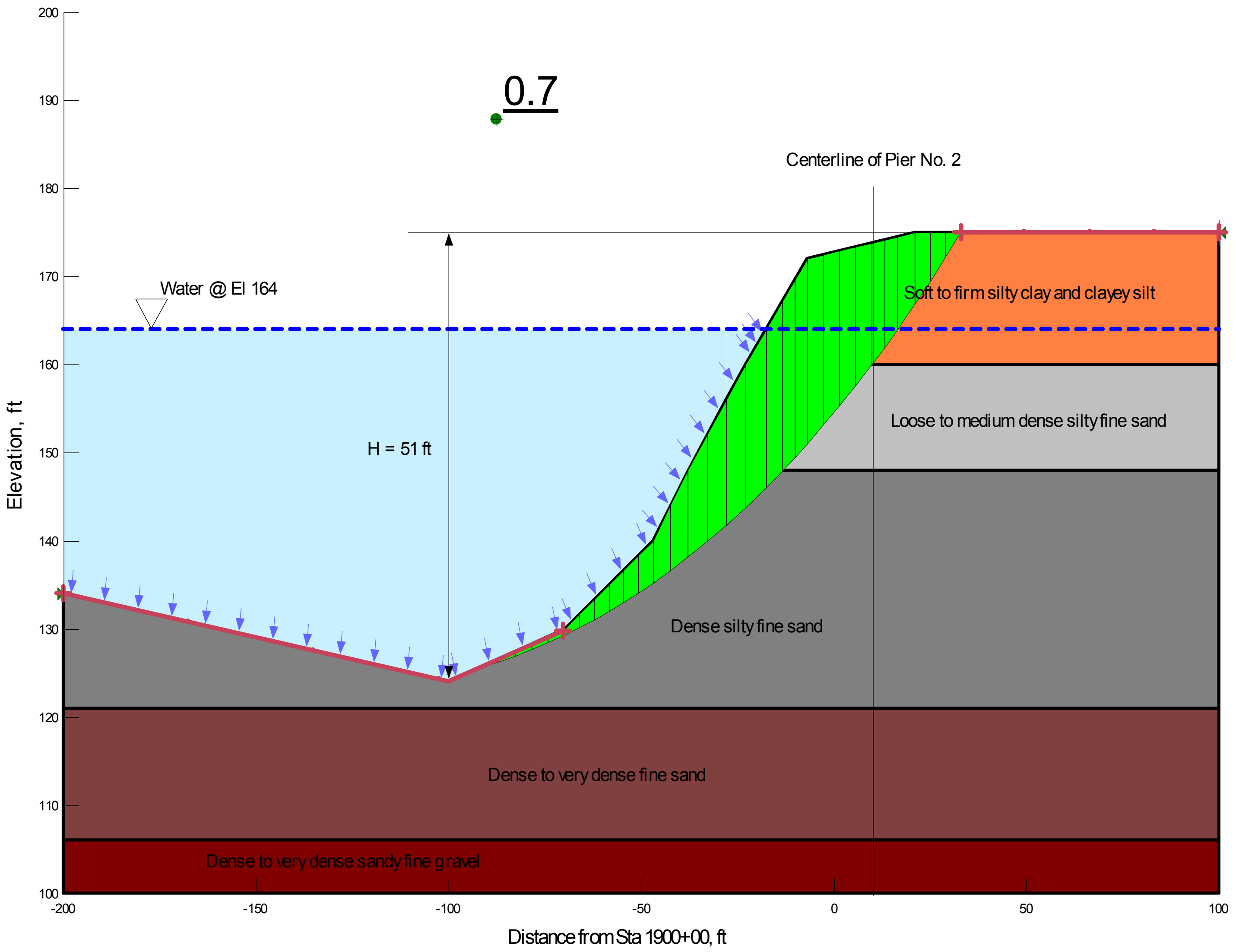
Results of Stability Analyses – Long Term Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



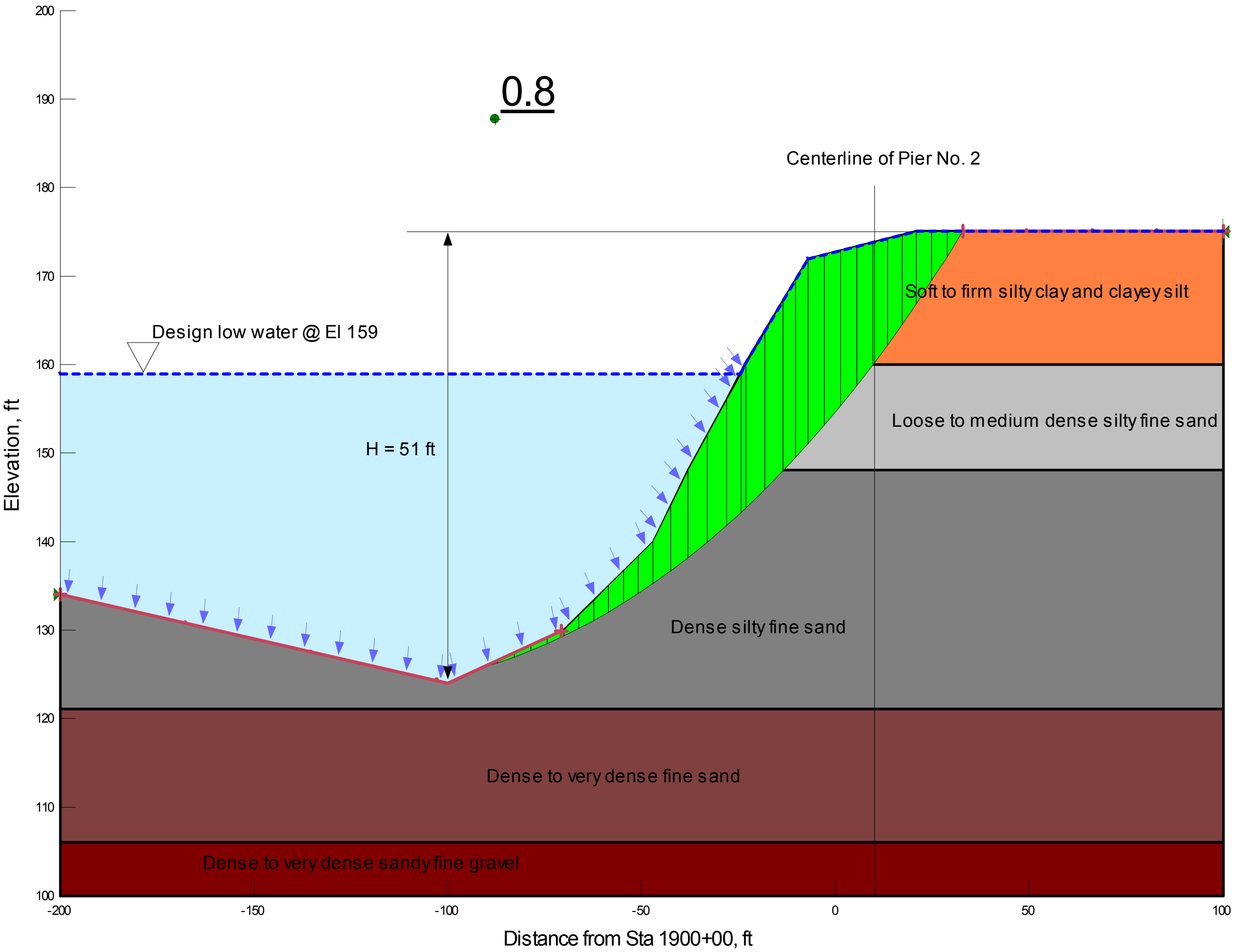
Results of Stability Analyses – Long Term Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

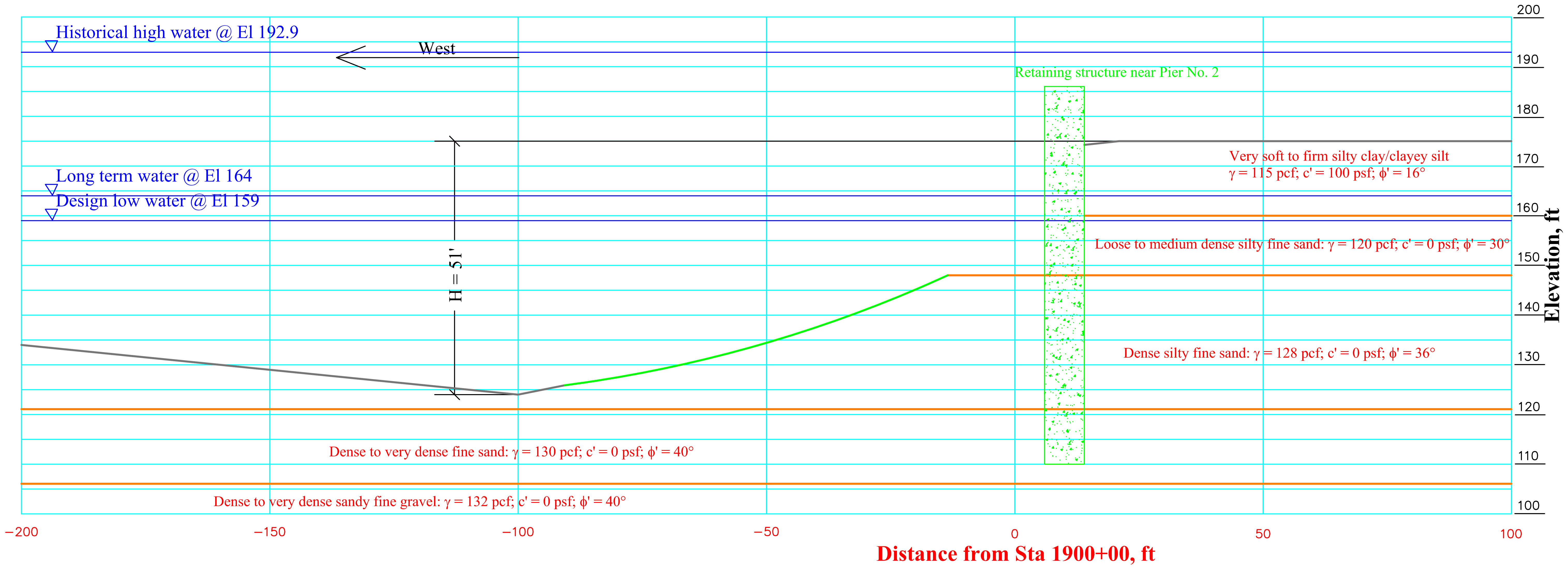


Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Existing East River Bank near Pier No. 2 – H = 51 ft
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Rapid Drawdown Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Drawdown to El 159 (Design Low Water)
AHTD JOB BB0610:White River Str. & Apprs. (F)



10 ft 0 10 20 ft

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

Section and Material Parameters for Stability Analysis
Retaining Structure near Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

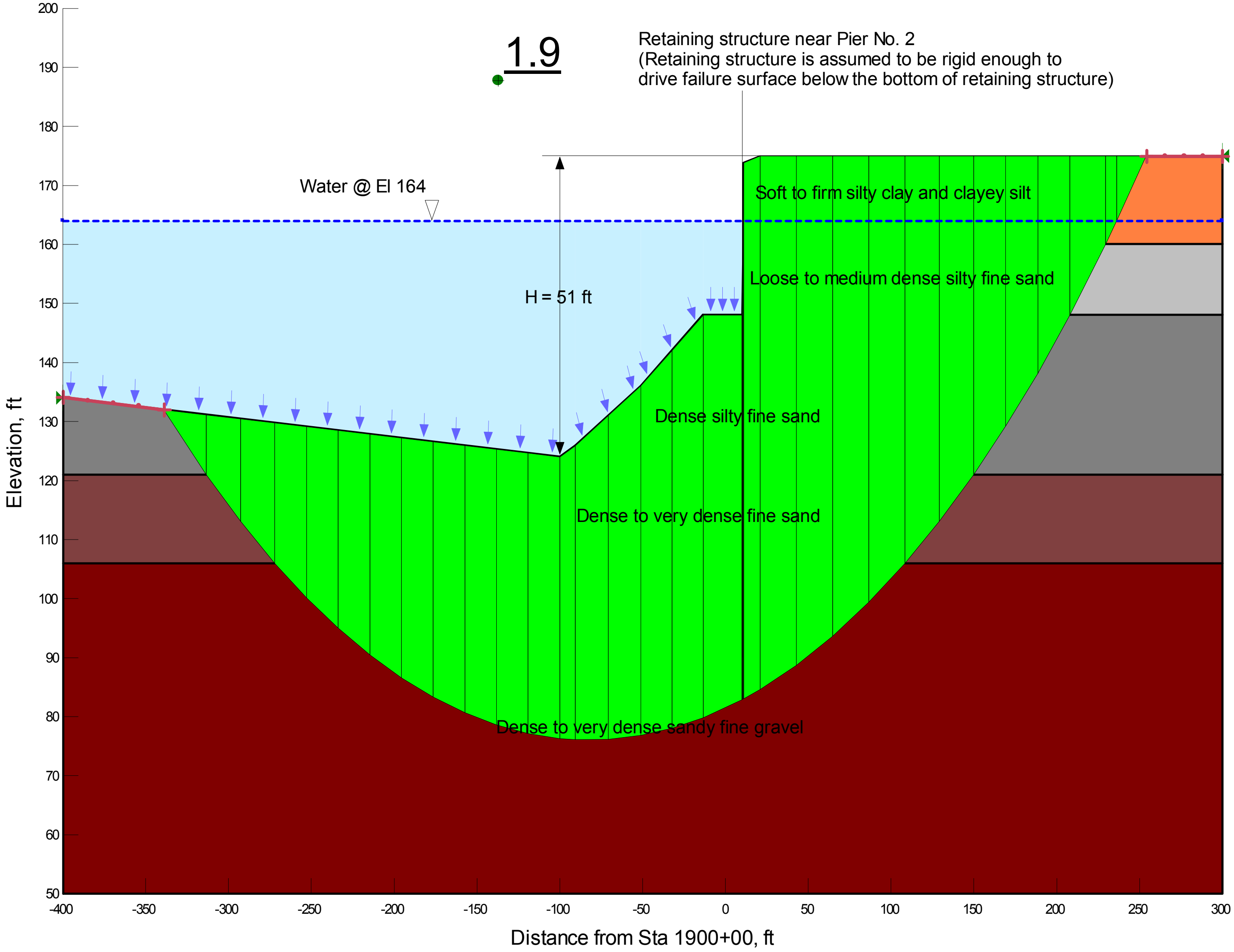
Scale: As Shown

September 18, 2014

Plate

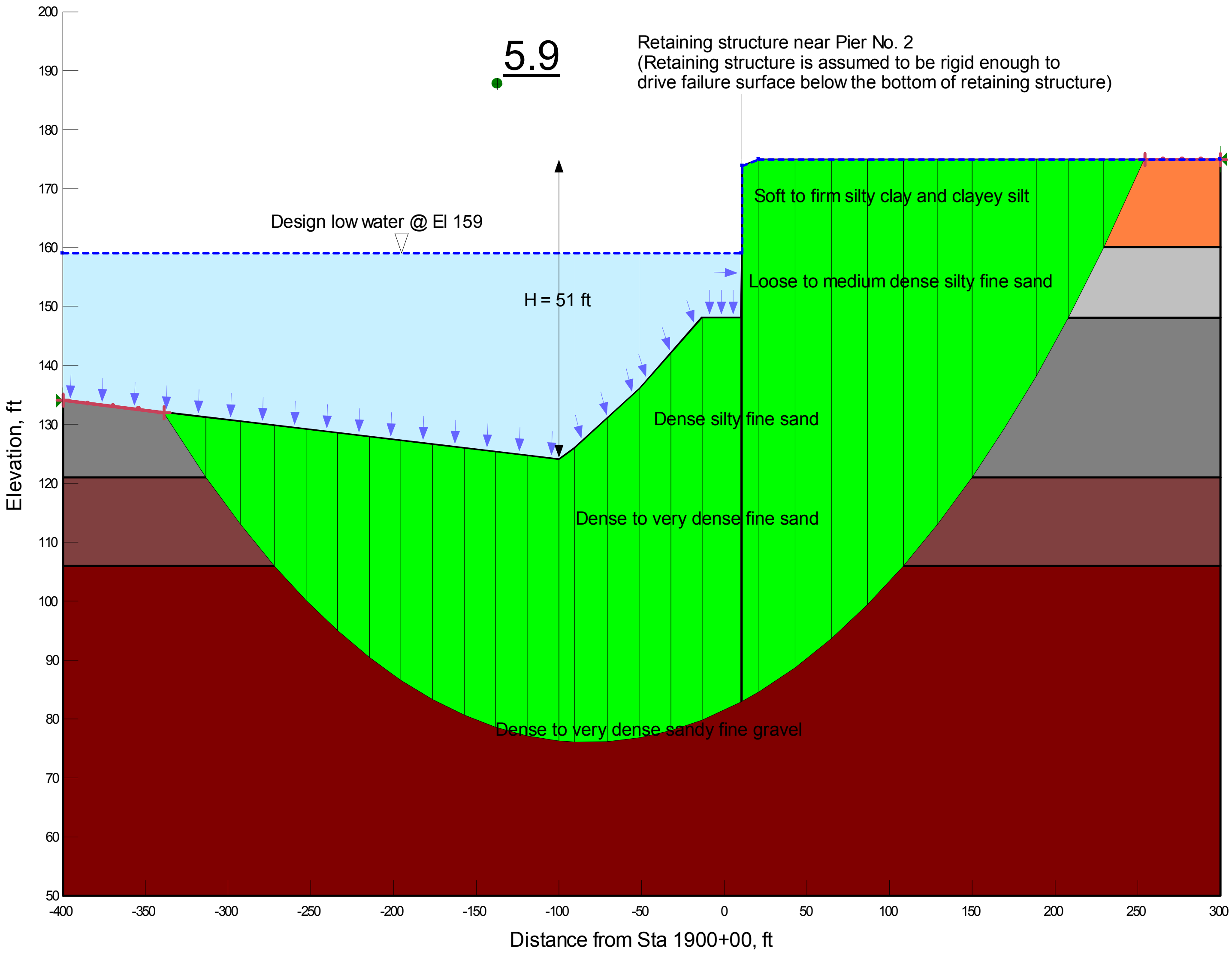
Stability Analysis Results
East River Bank Remedied by Retaining Structure
AHTD Job BB0610: White River Str. & Apprs. (F)

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
Seismic ($k_h = 1.0A_S = 0.19$)*	Water @ El 164	1.9
Rapid Drawdown	Drawdown to El 159 (design low water)	5.9



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.23$)
East River Bank Remedied by Retaining Structure
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

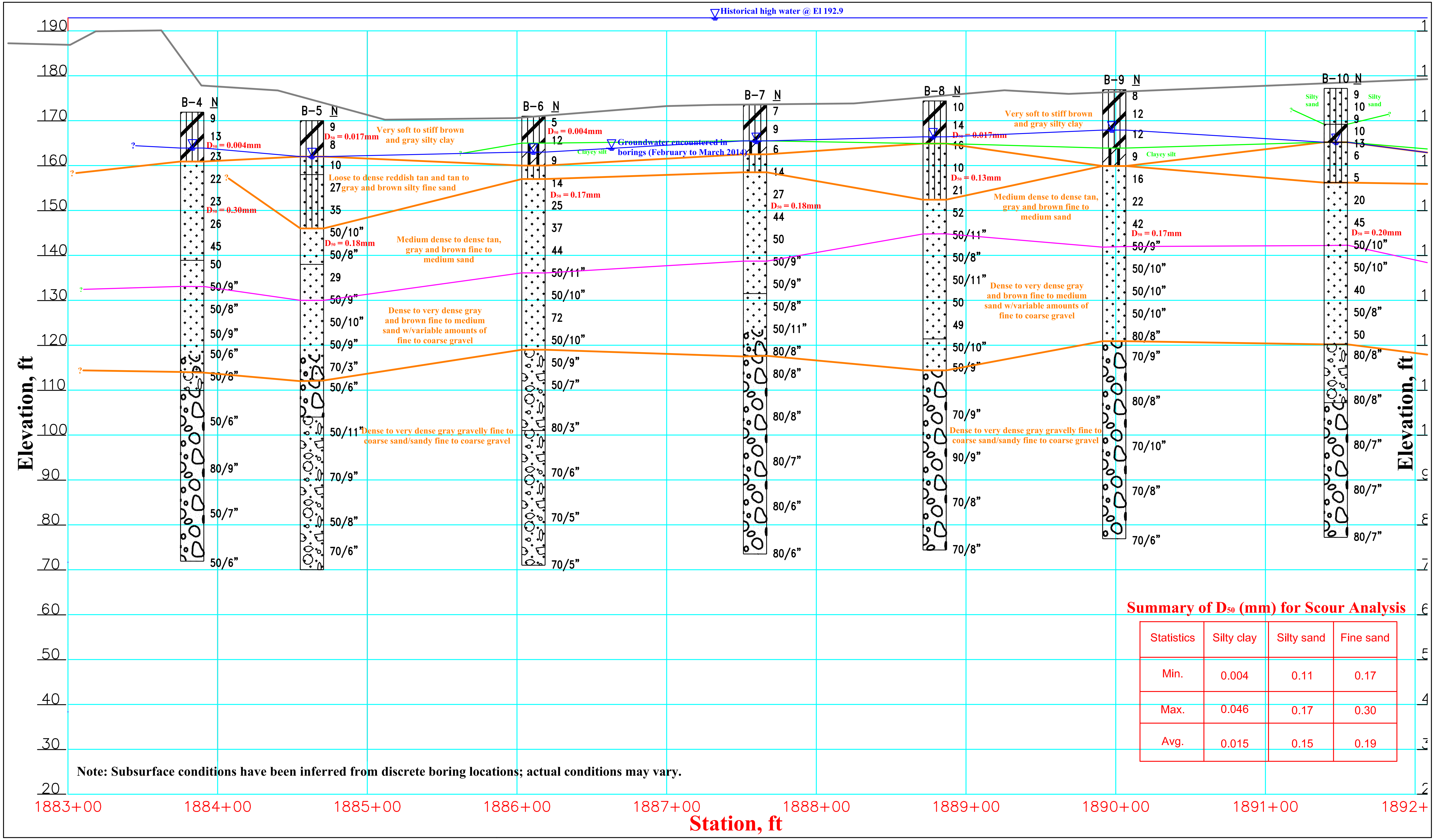
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
East River Bank Remedied by Retaining Structure
Drawdown to El 159 (Design Low Water)
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX R



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Note: Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.

Horizontal Scale: 30 ft 0 30 60 ft

Vertical Scale: 10 ft 0 10 20 ft

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

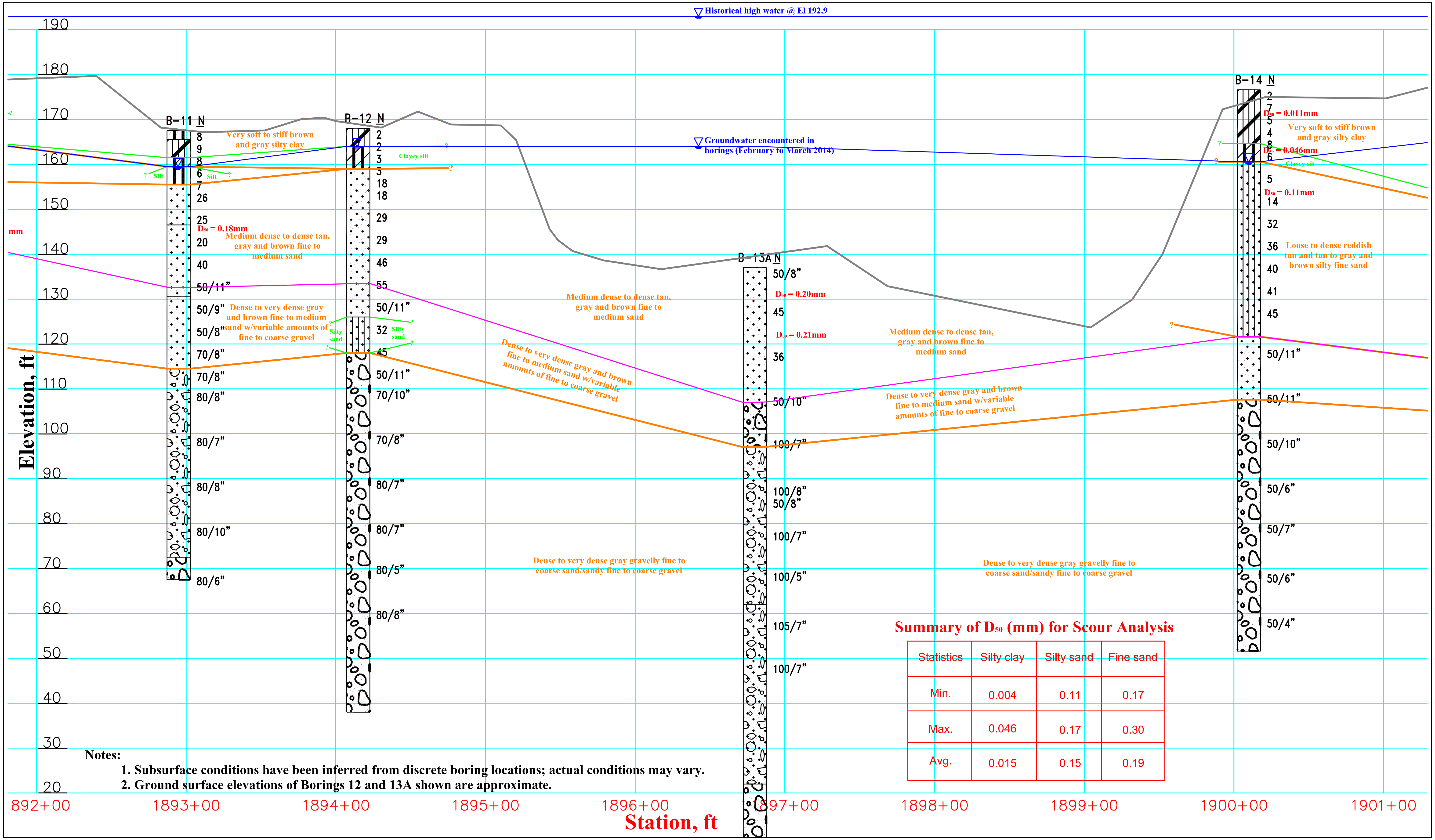
D_{50} for Scour Analysis - Page 1 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

April 8, 2014

Plate



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Notes:

1. Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.
2. Ground surface elevations of Borings 12 and 13A shown are approximate.

Horizontal Scale: 30 ft 0 30 60 ft

Vertical Scale: 10 ft 0 10 20 ft

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

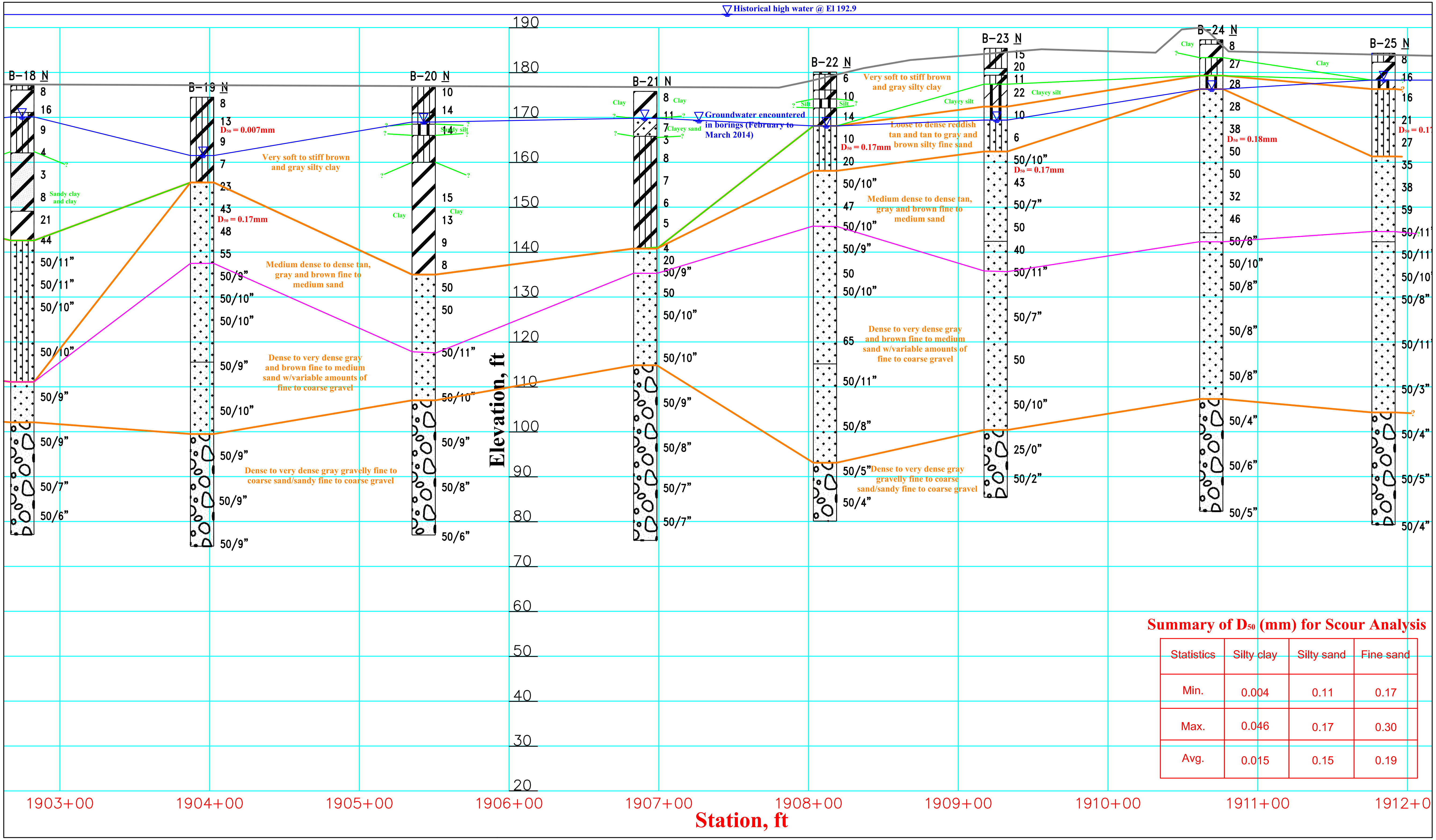
D_{50} for Scour Analysis - Page 2 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

April 8, 2014

Plate



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

Grubbs, Hoskyn,
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Consulting Engineers

D_{50} for Scour Analysis - Page 3 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

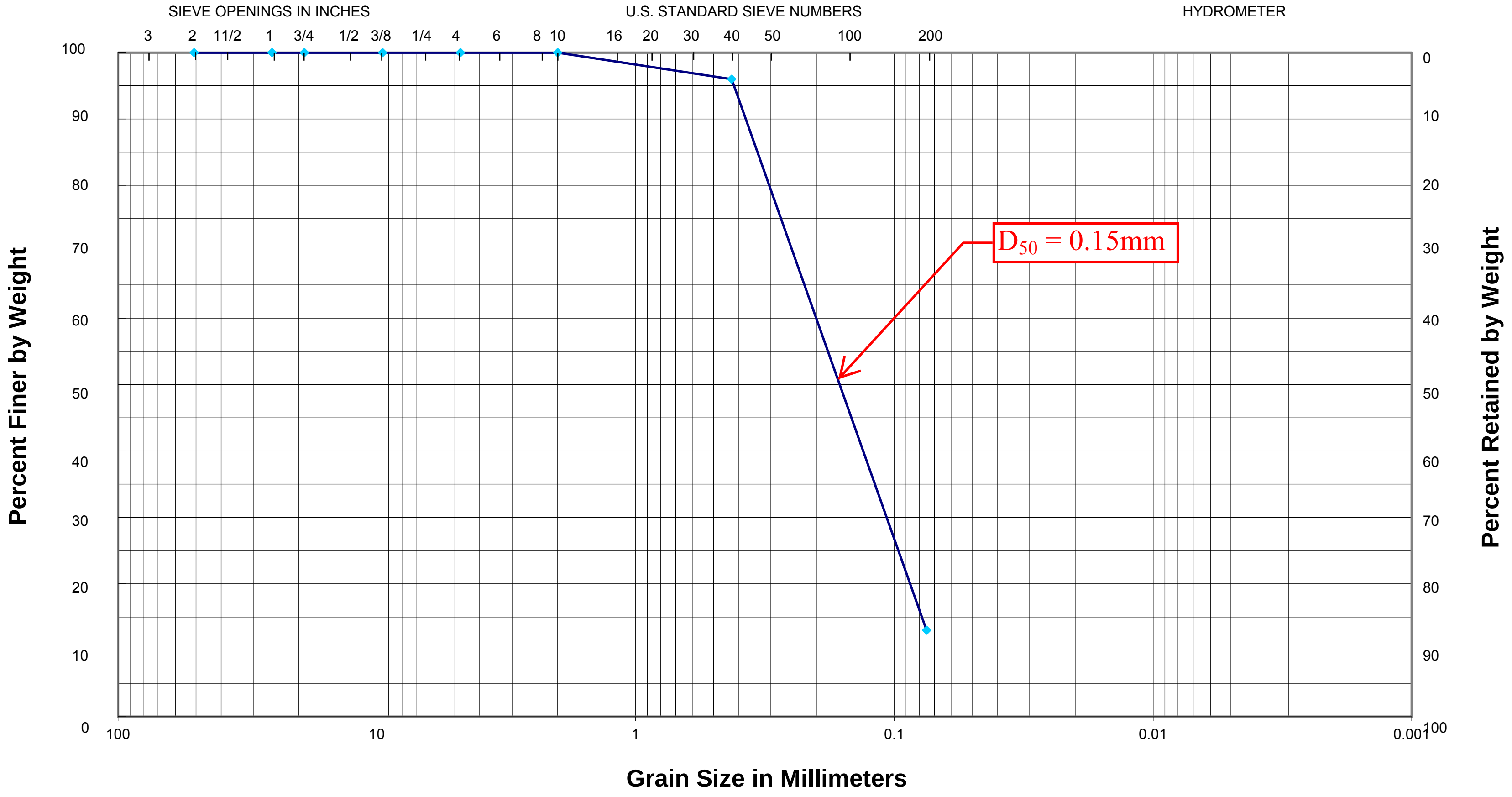
Scale: As Shown

April 8, 2014

Plate

13-017

GRAIN SIZE CURVE



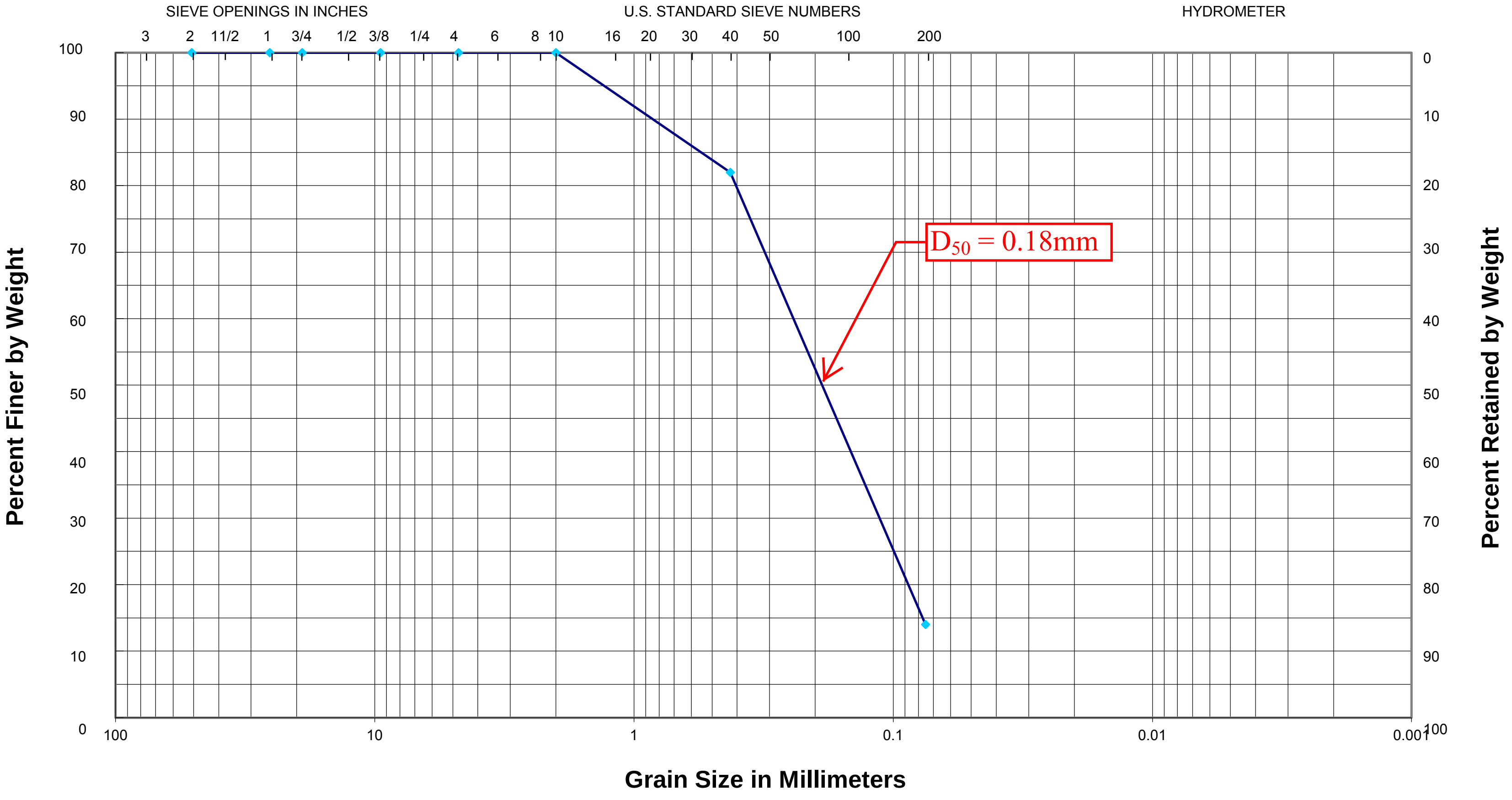
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



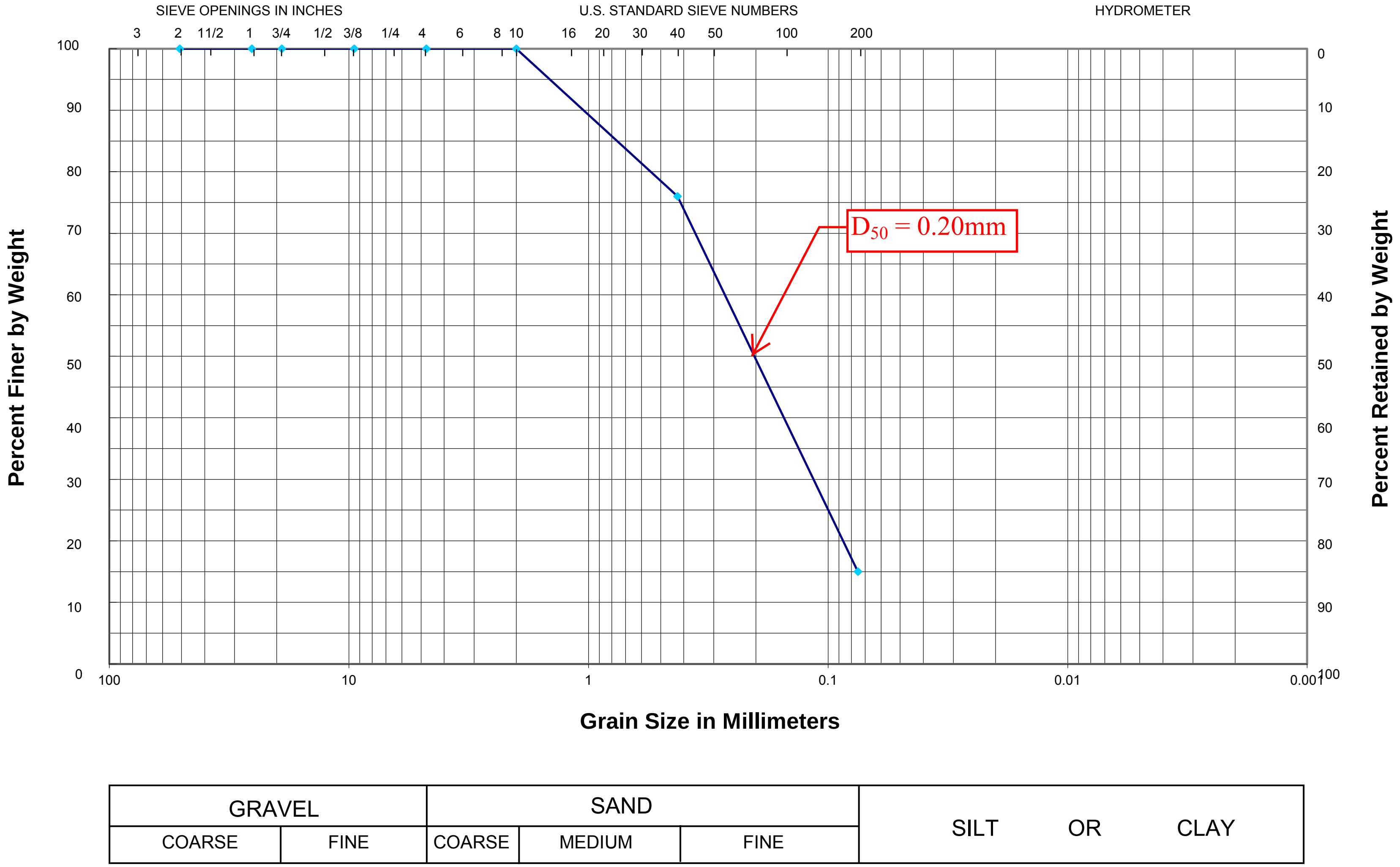
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 2, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

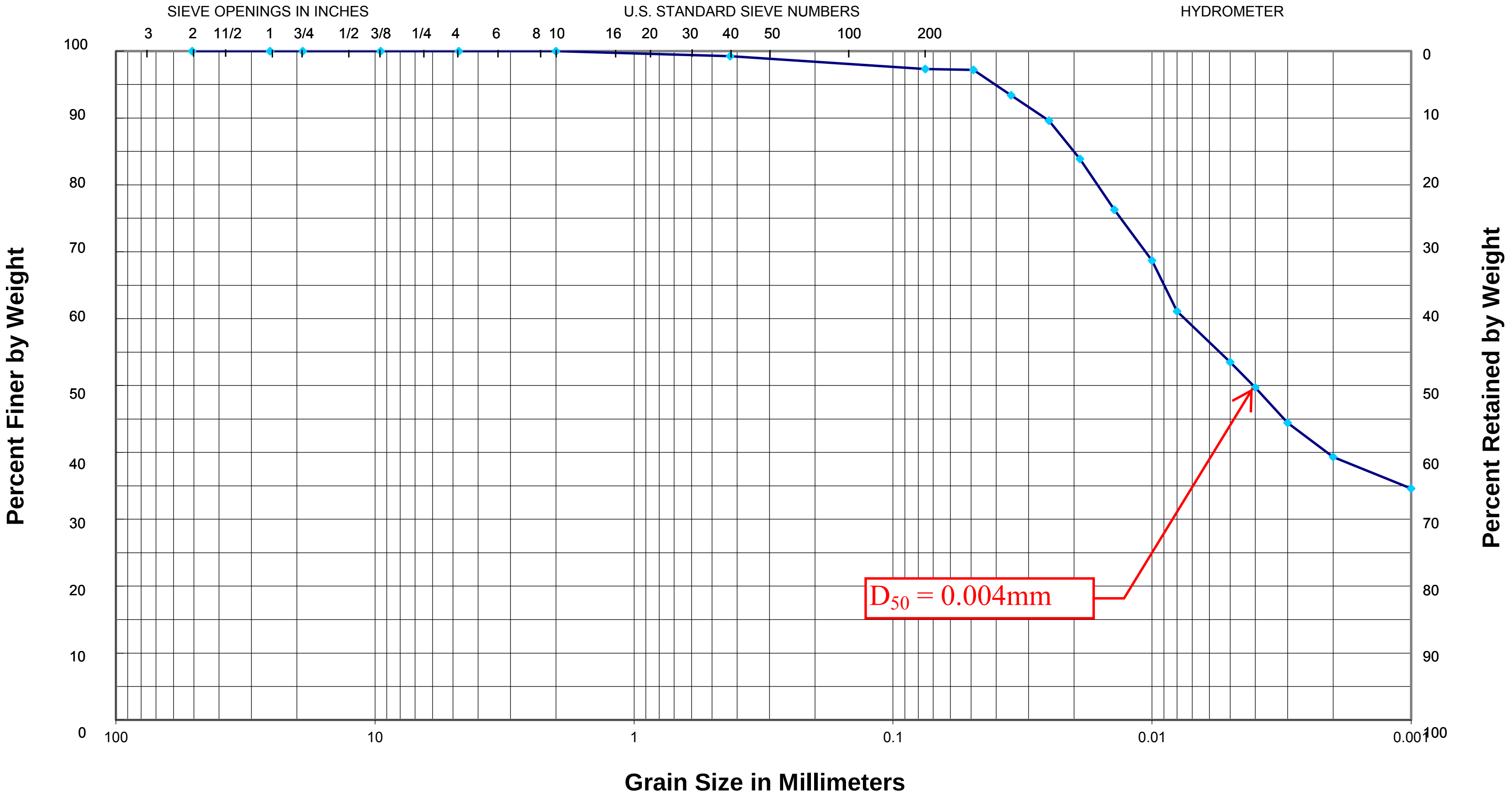
GRAIN SIZE CURVE



Sample: Boring 3, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

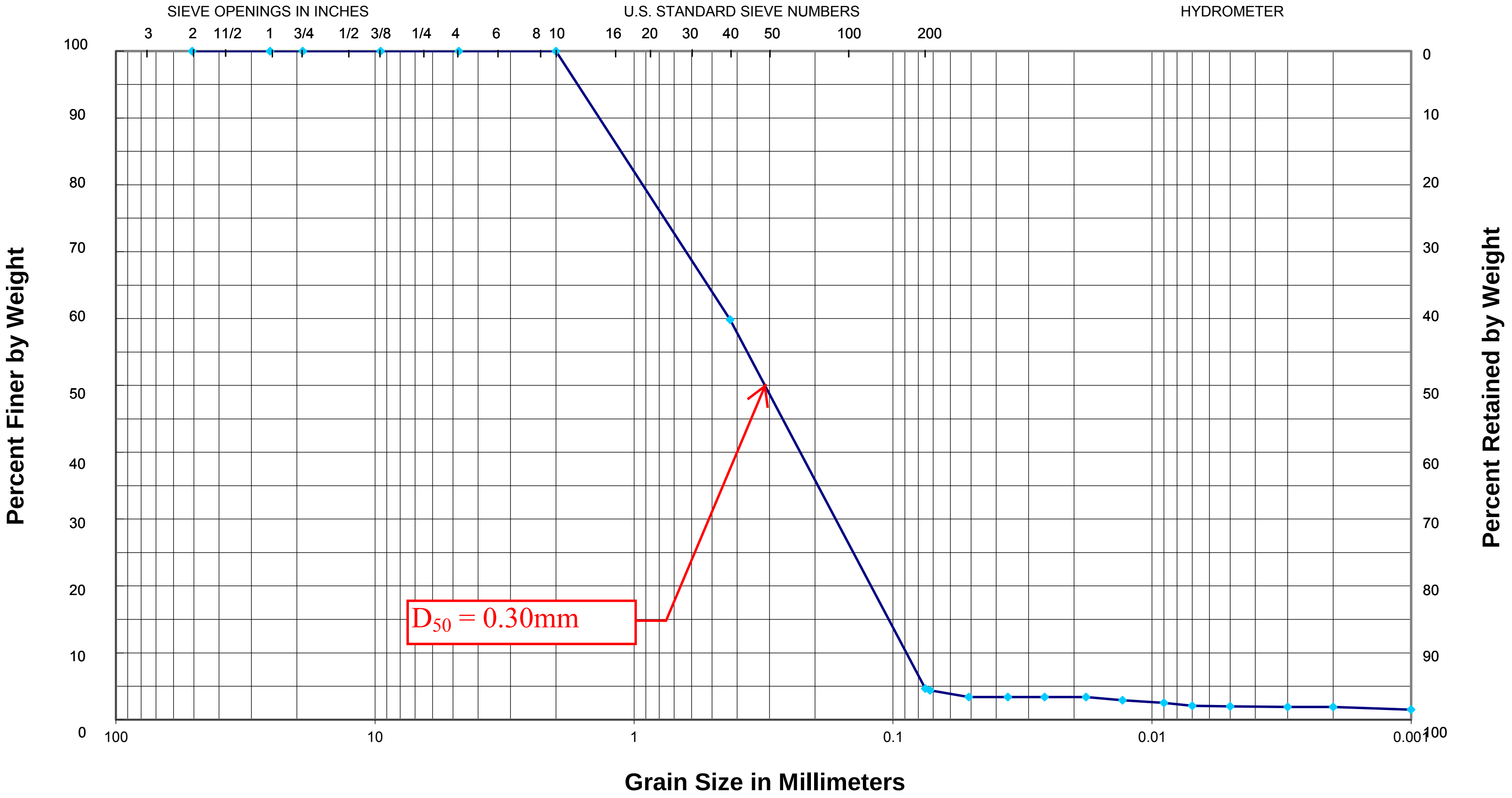
Sample: Boring 4, 7-7.5 ft
Properties: G_s = 2.700; LL = 46, PL = 19, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

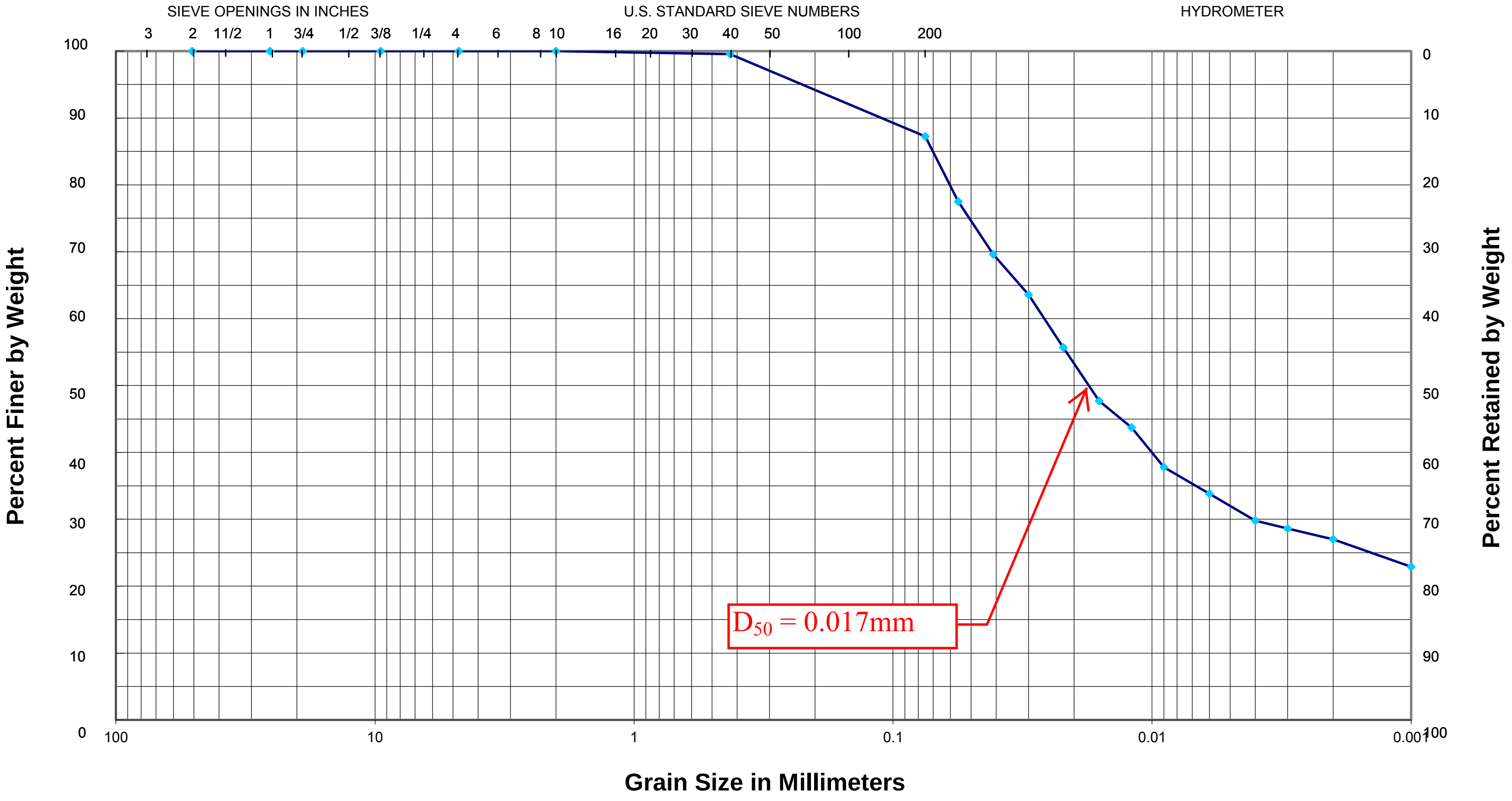
Sample: Boring 4, 14-15 ft
Properties: G_s = 2.653; Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



D_{50} = 0.017mm

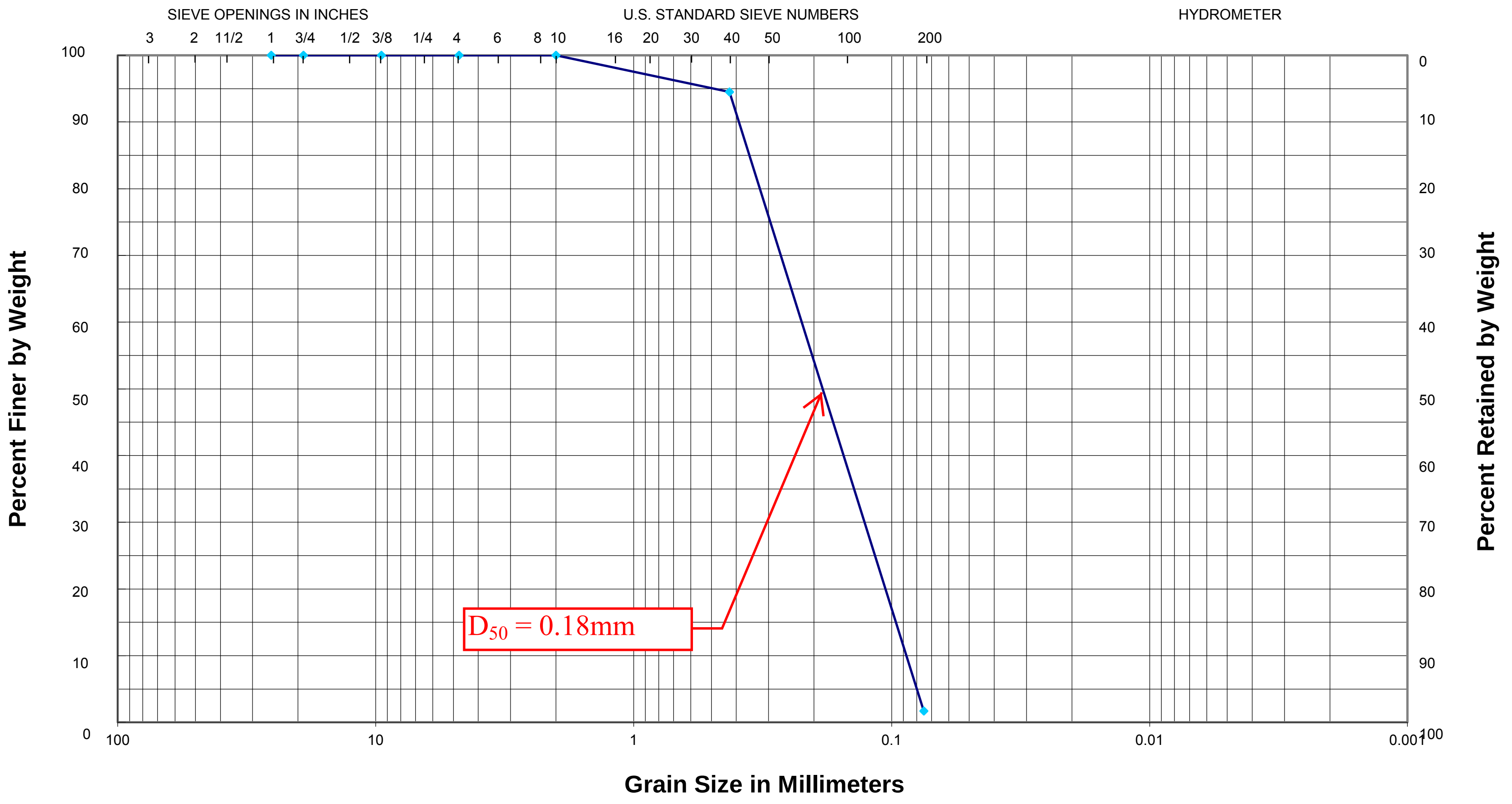
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 5, 3-3.5 ft
Properties: G_s = 2.637; LL = 33, PL = 17, PI = 16

Description: Dark gray and brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

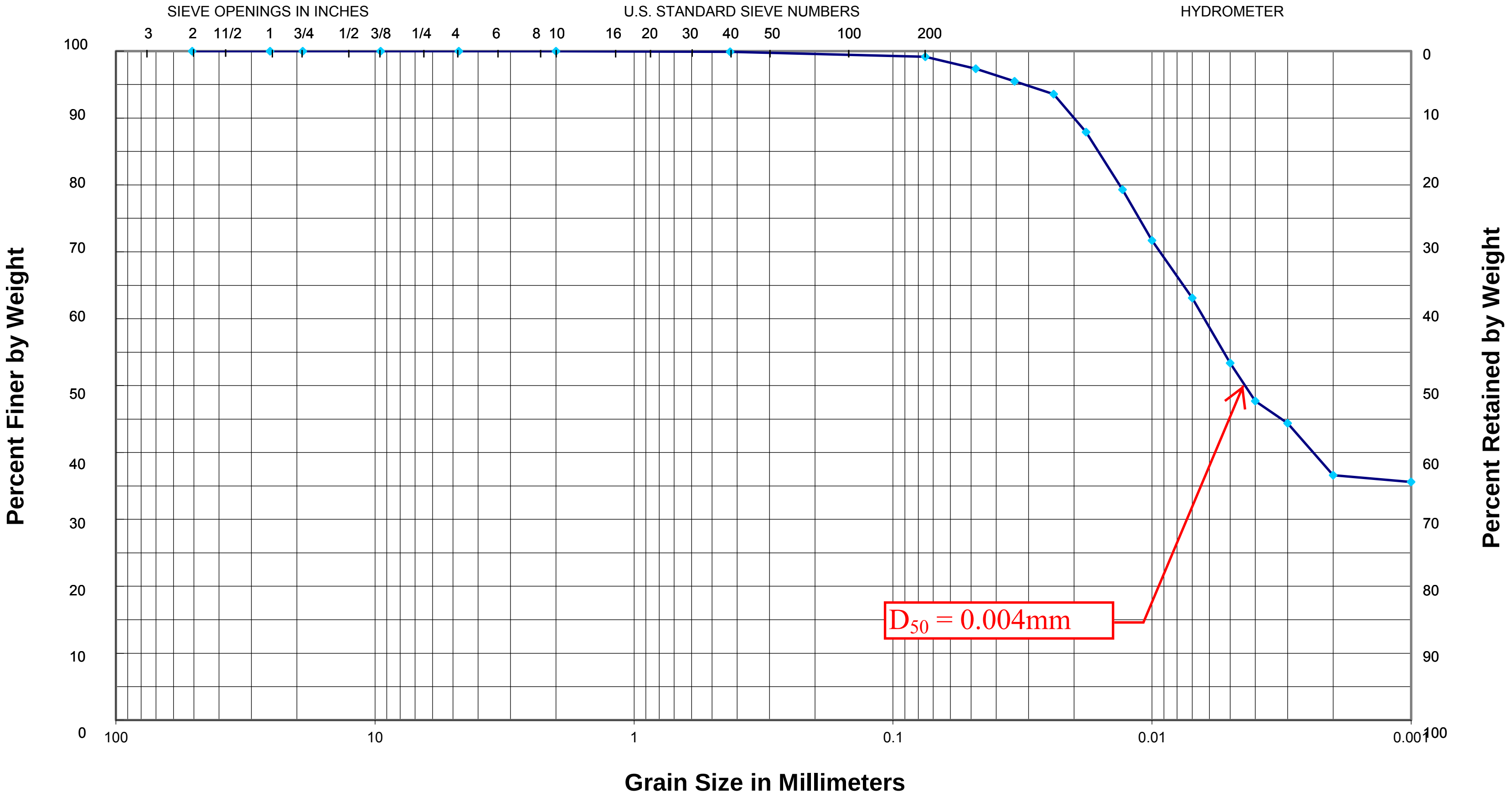
Sample: Boring 5, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray and tan fine sand w/trace medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



D_{50} = 0.004mm

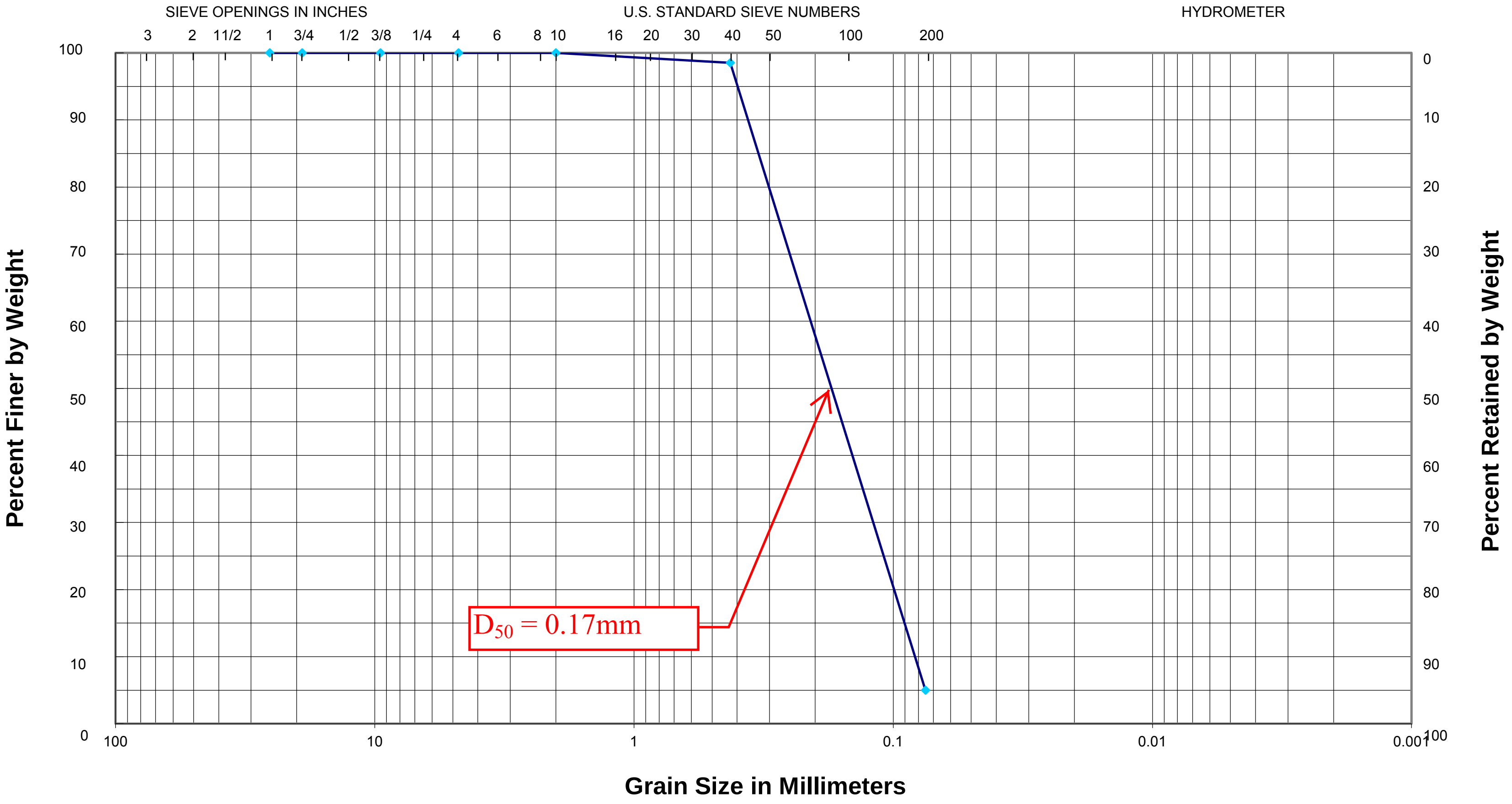
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 6, 3-3.5 ft
Properties: G_s = 2.707; LL = 48, PL = 21, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE



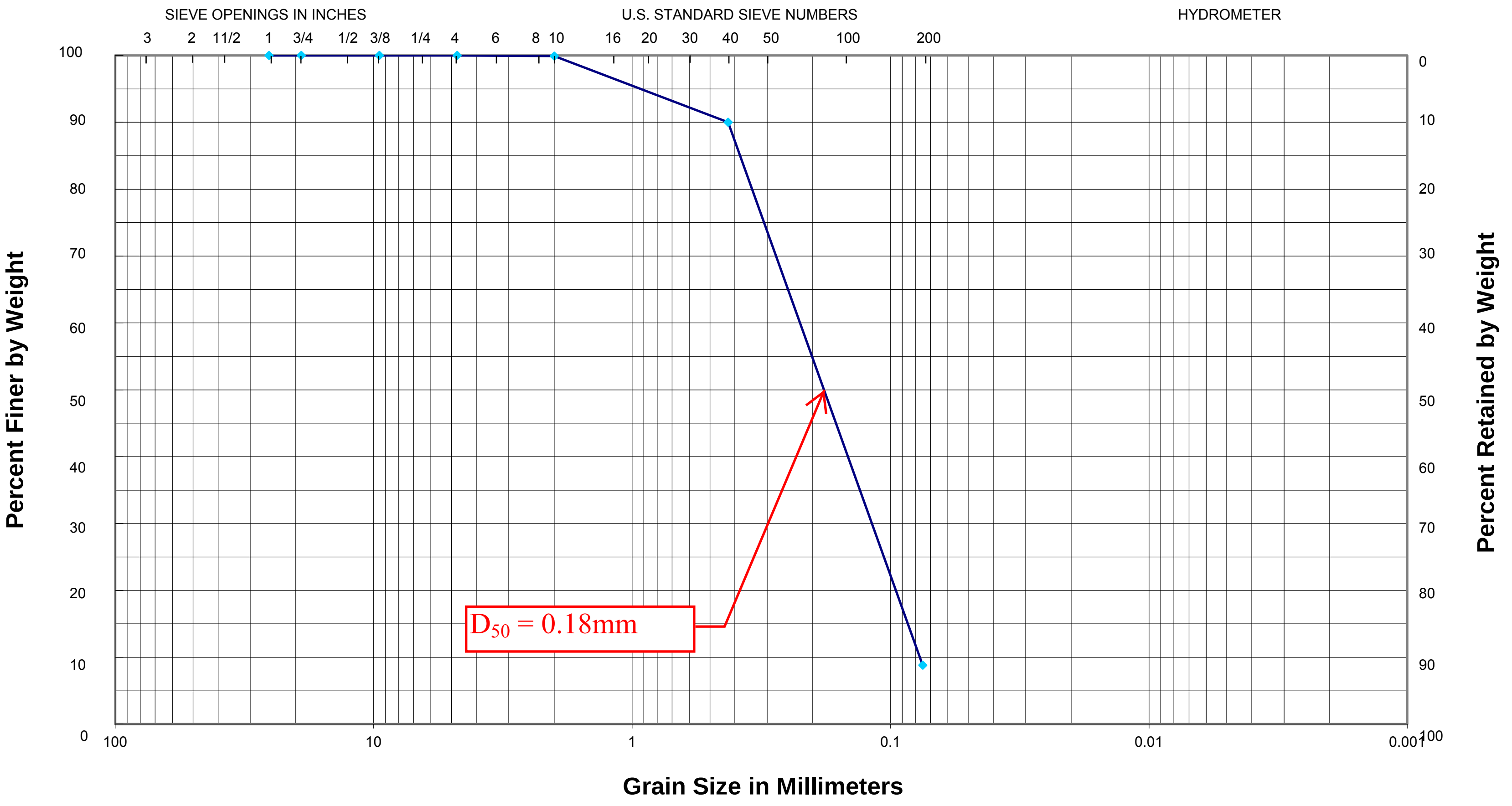
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 6, 19-20 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



D_{50} = 0.18mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

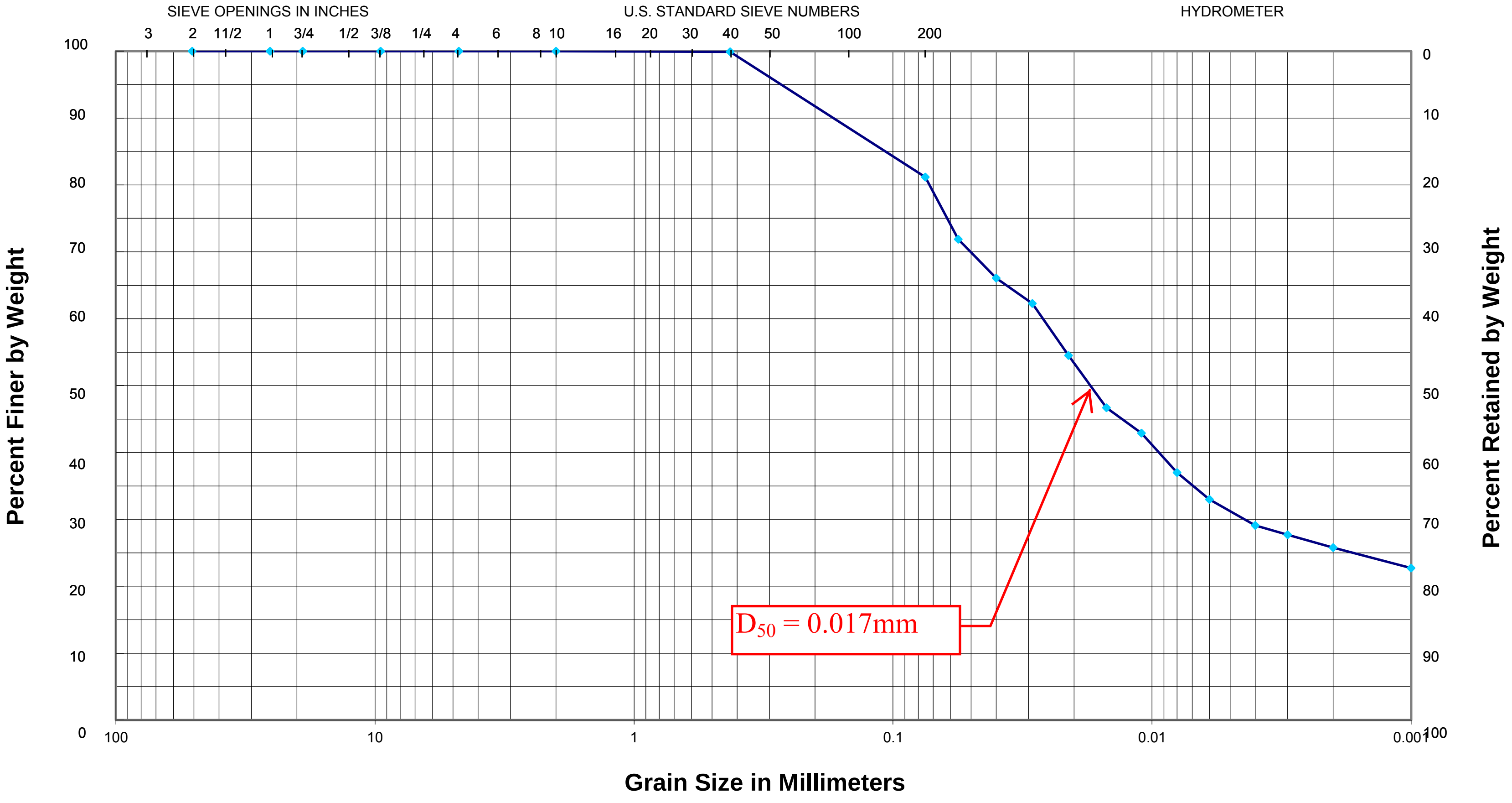
Sample: Boring 7, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



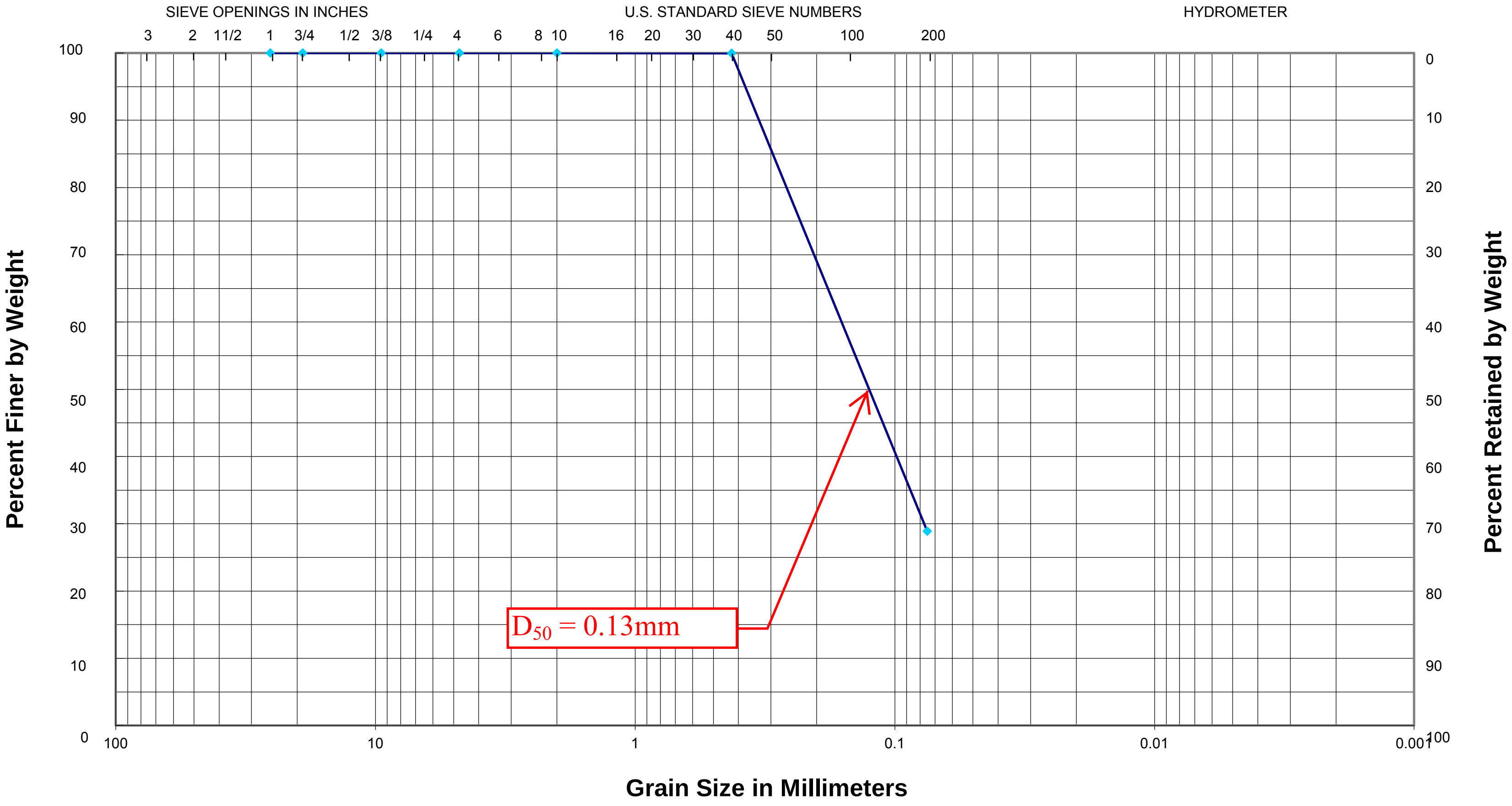
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 8, 7-7.5 ft
Properties: G_s = 2.697; LL = 37, PL = 15, PI = 12

Description: Gray and brown silty clay, sandy
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



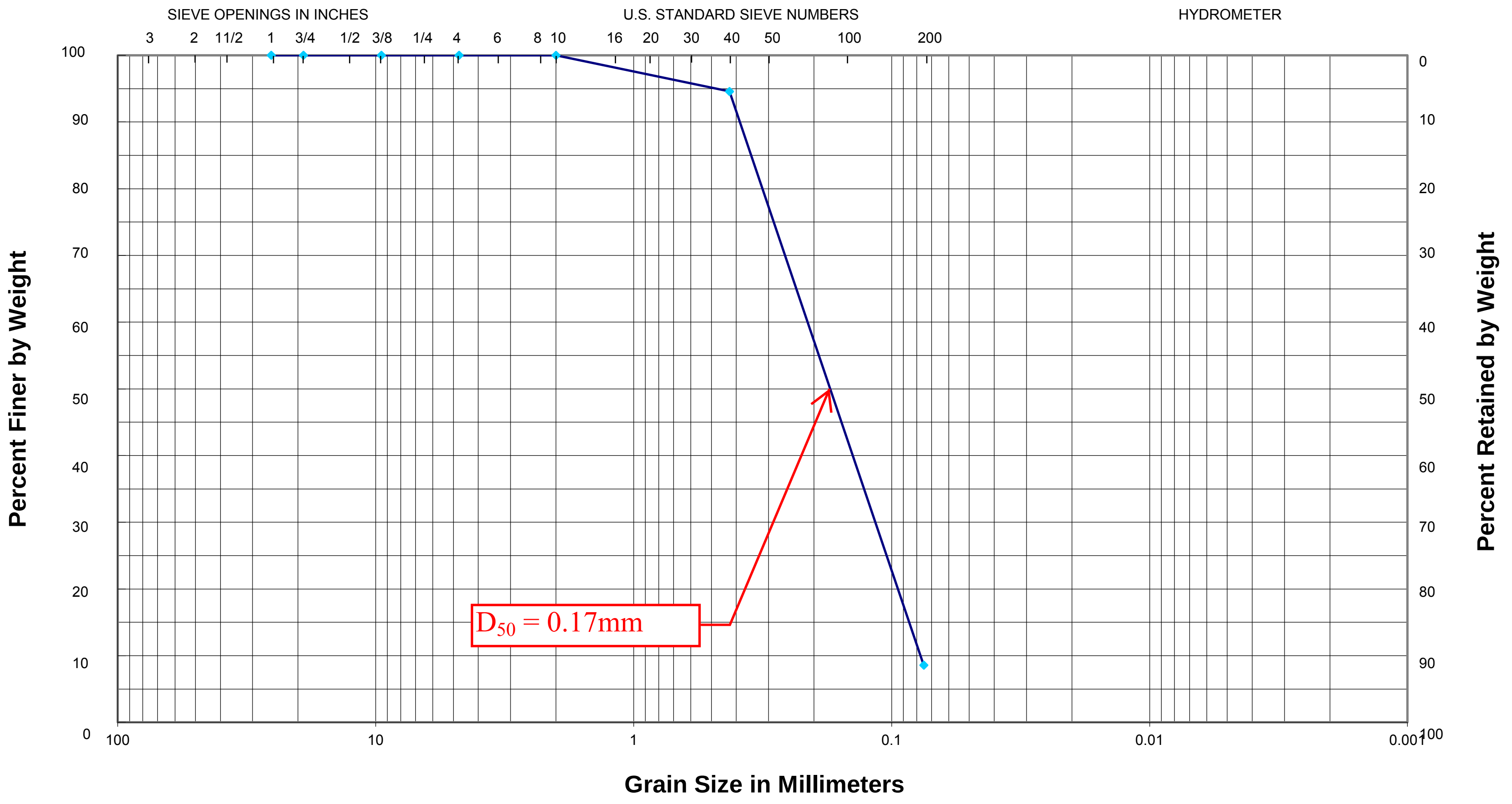
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



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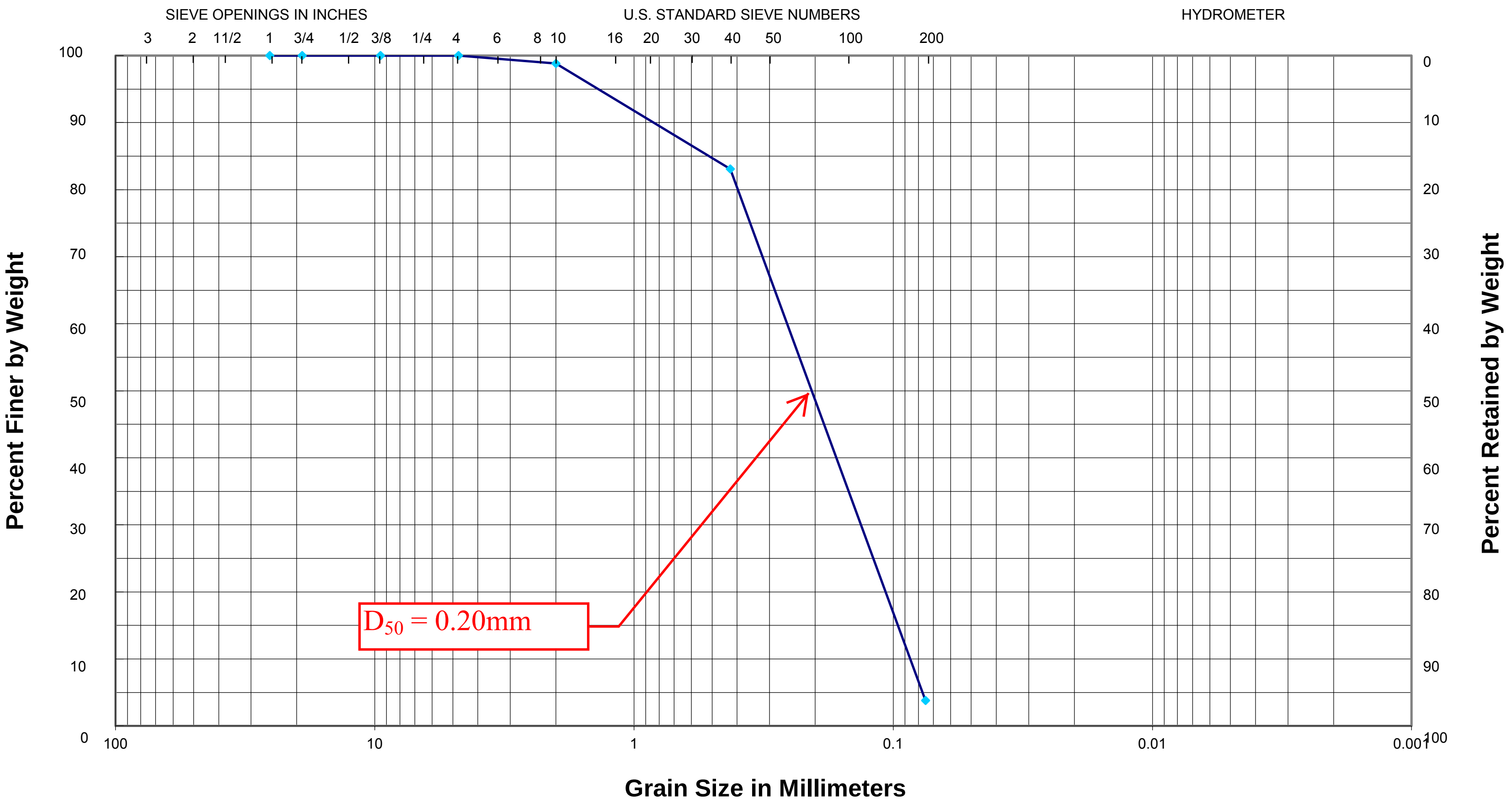
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Sample: Boring 9, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



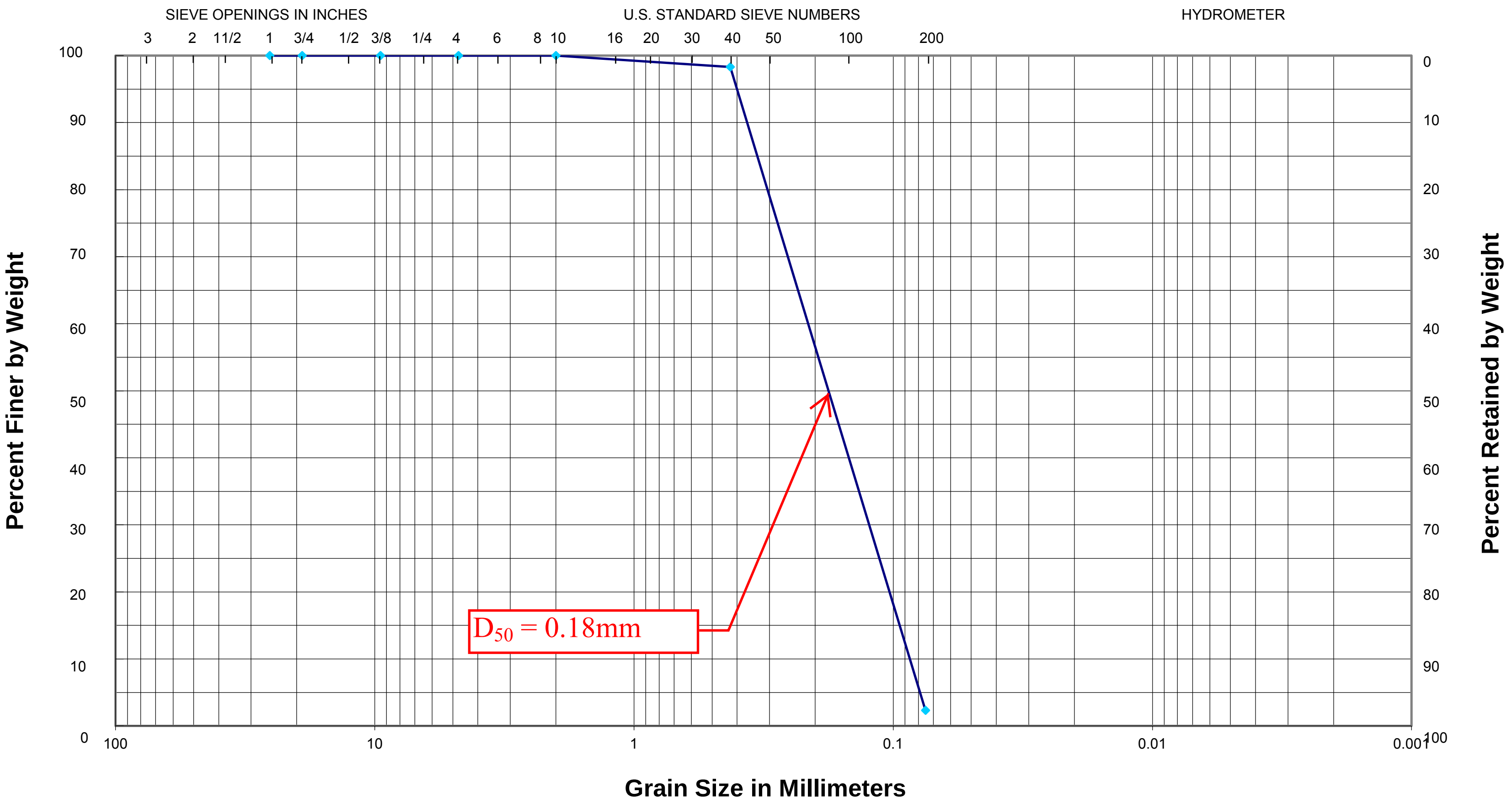
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 10, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



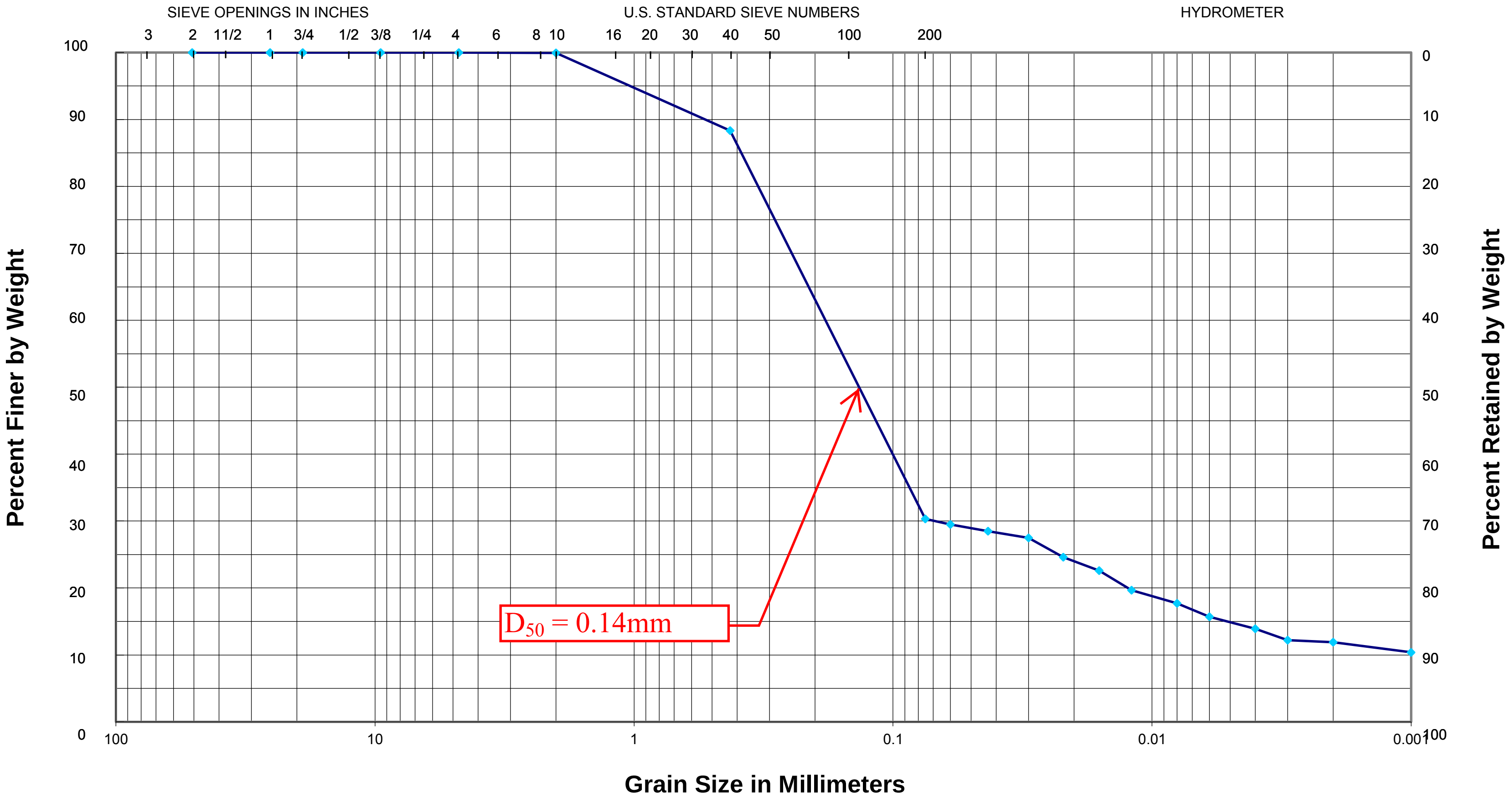
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 11, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



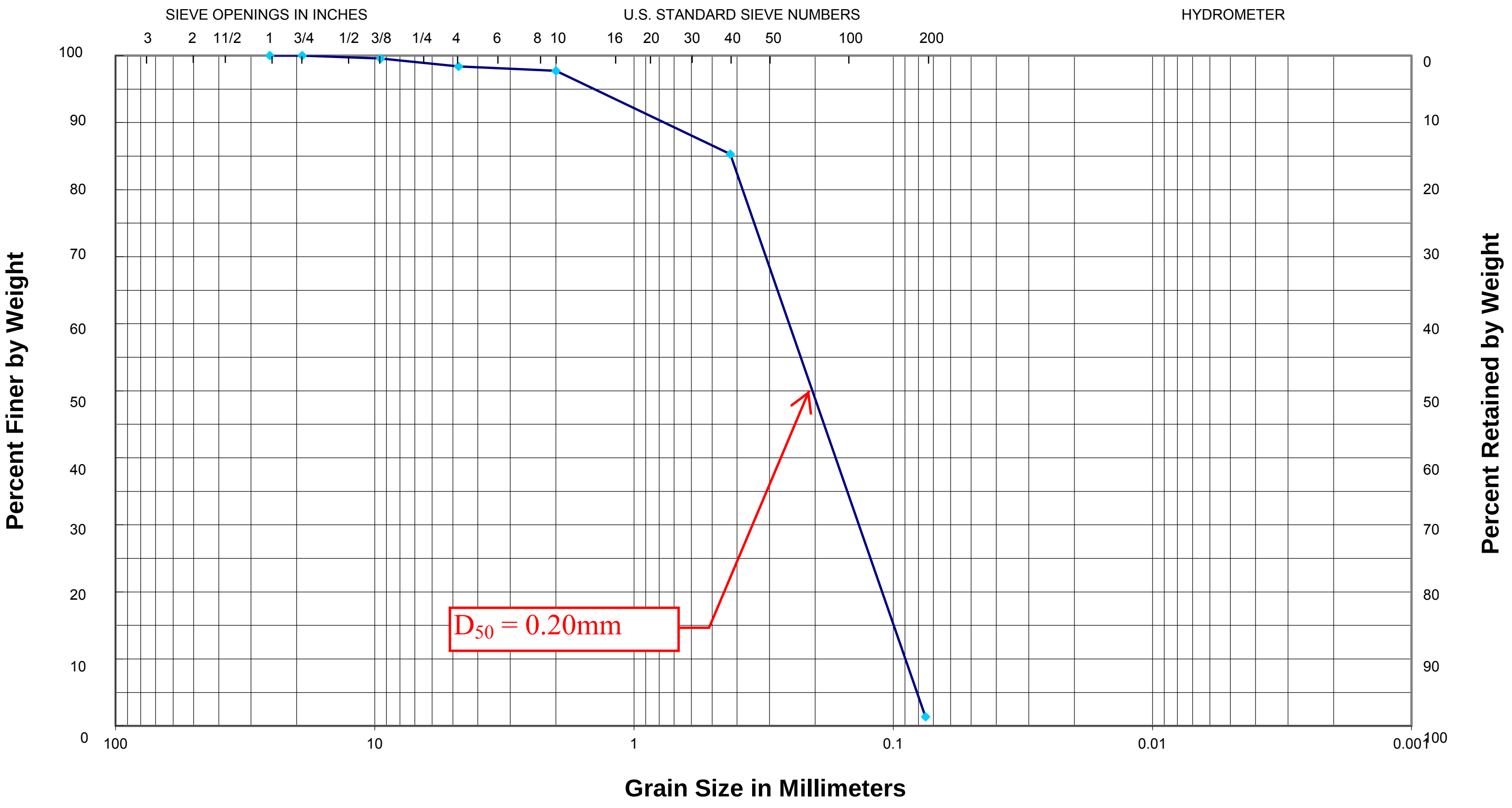
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13, 6.5-7.5 ft
Properties: G_s = 2.661; Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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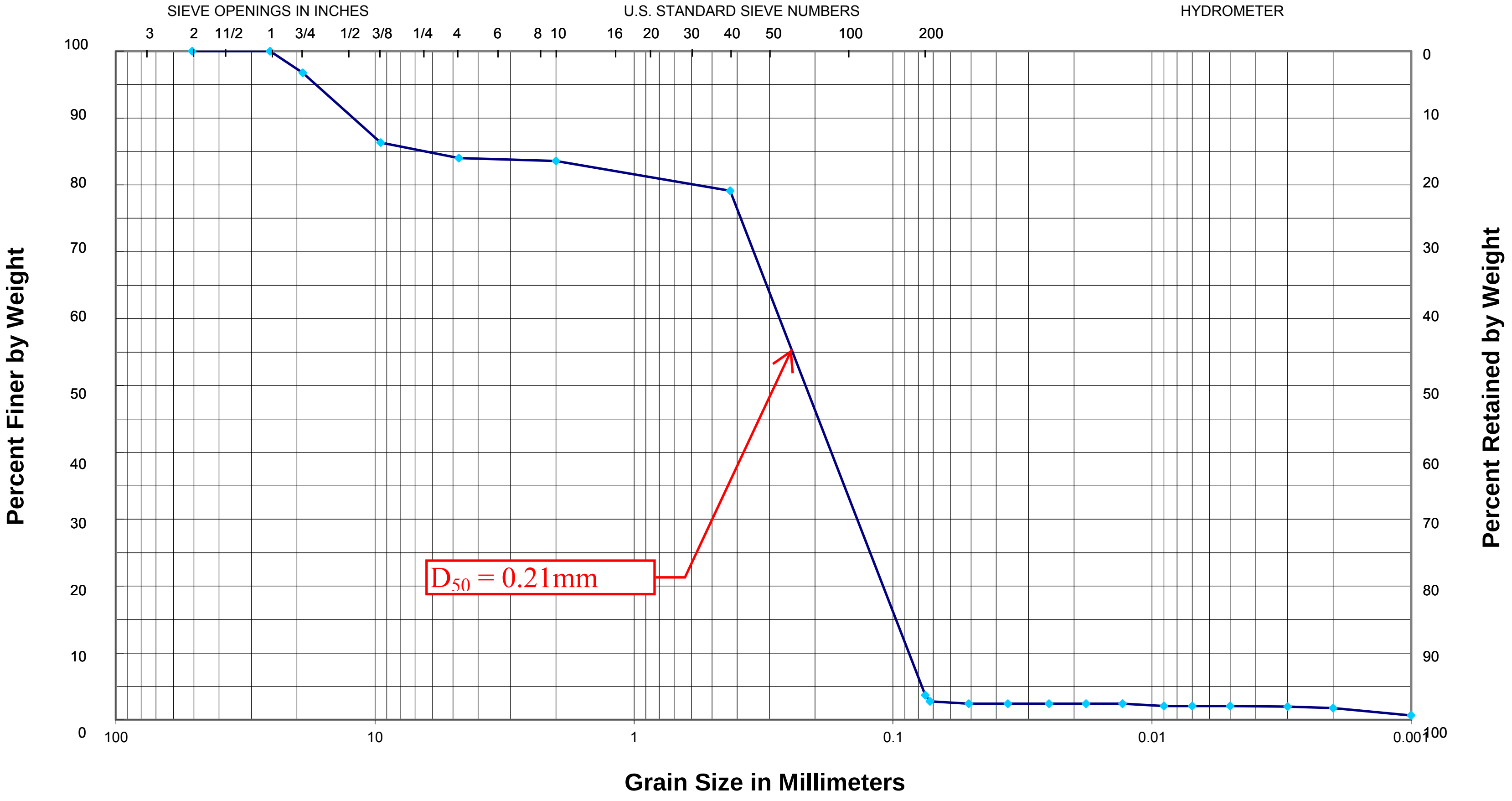
Barton & Wyatt, INC.
CONSULTING ENGINEERS



D_{50} = 0.21mm

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<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

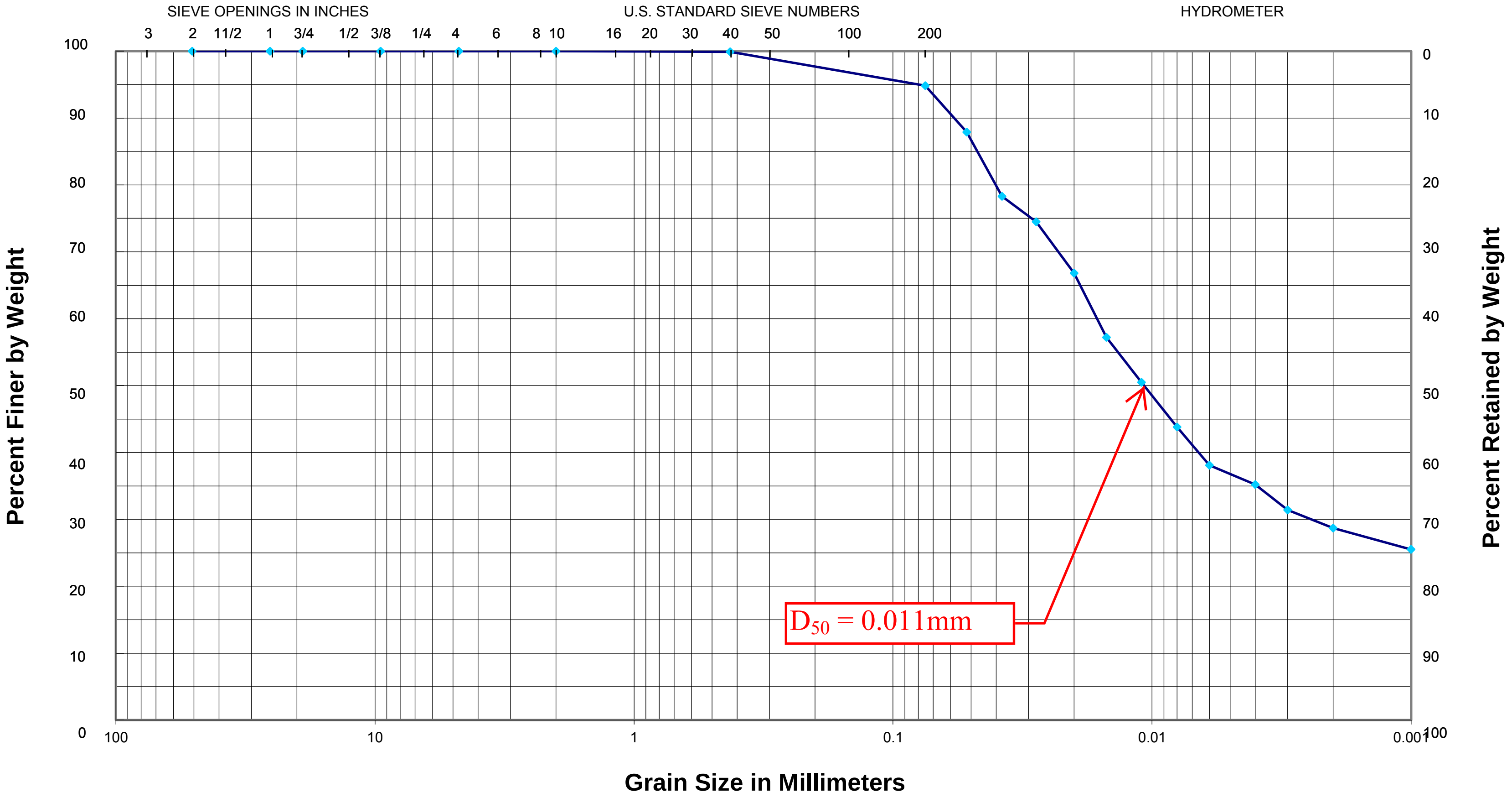
Sample: Boring 13A, 19-20 ft
Properties: G_s = 2.654; Non-plastic

Description: Brown and tan fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

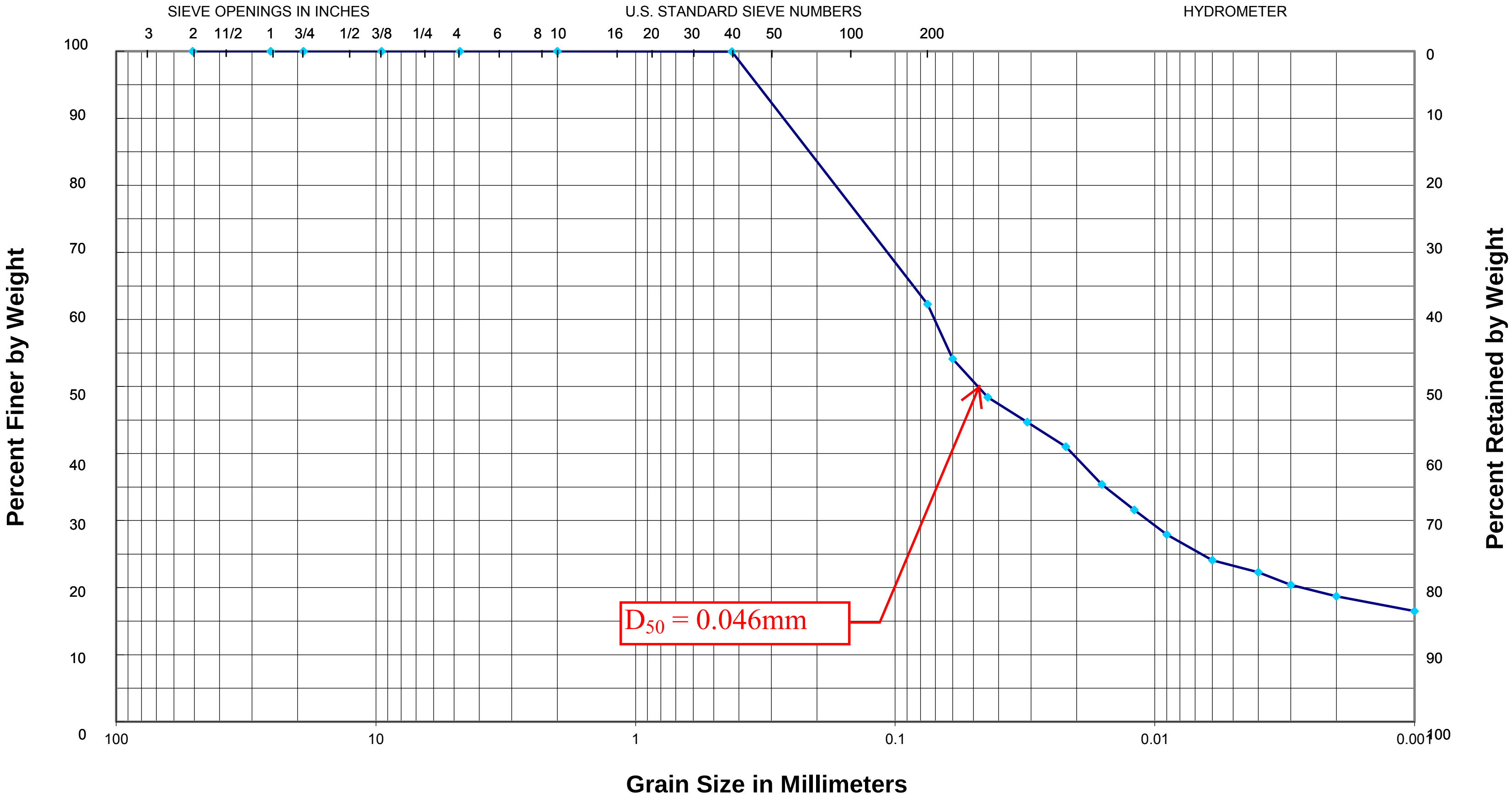
Sample: Boring 14, 4.5-5.5 ft
Properties: G_s = 2.669; LL = 34, PL = 19, PI = 15

Description: Brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

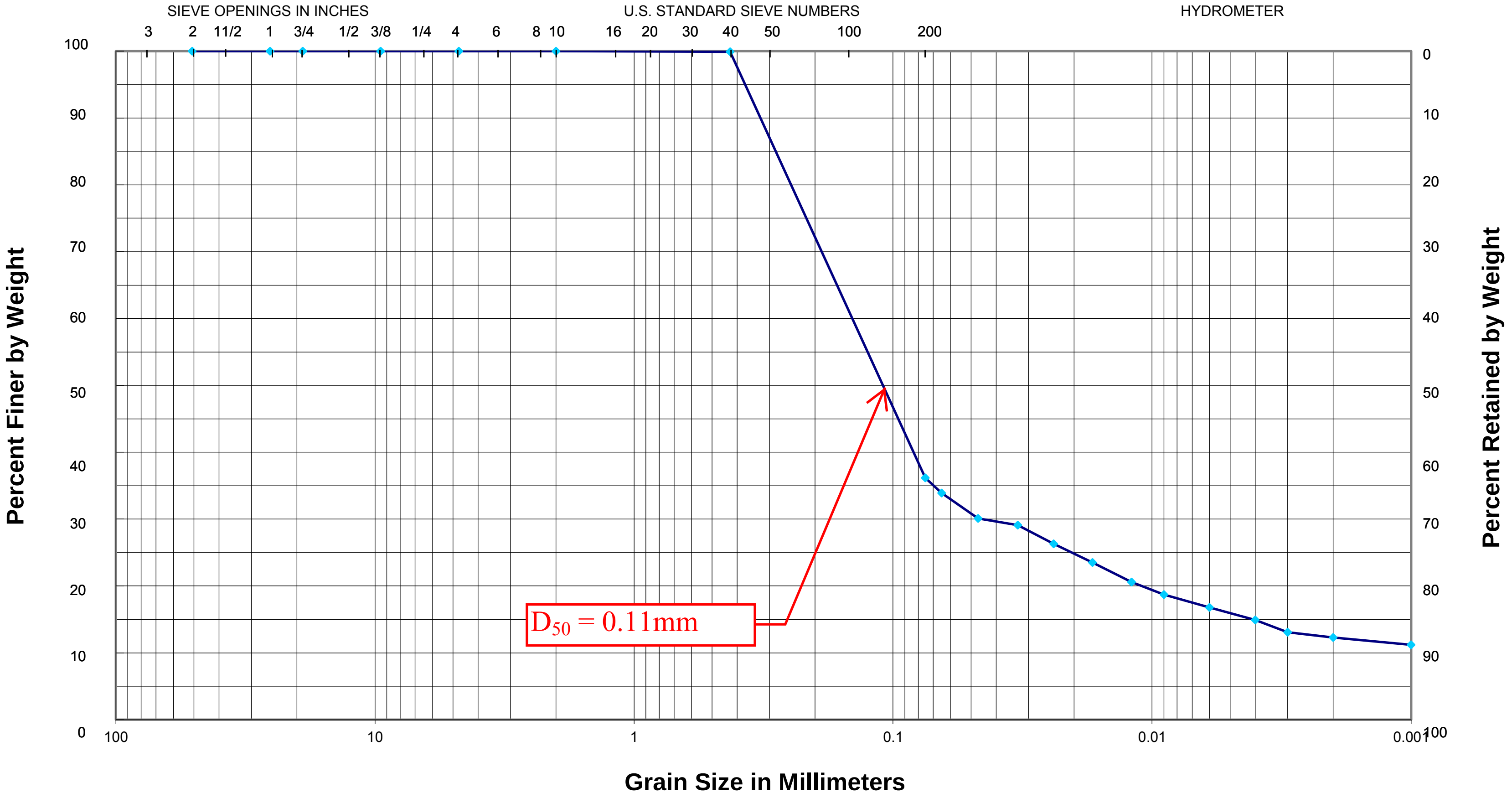
Sample: Boring 14, 14-15 ft
Properties: G_s = 2.668; LL = 22, PL = 17, PI = 5

Description: Gray clayey silt
Classification: USCS = CL-ML; AASHTO = A-4

13-017

GRAIN SIZE CURVE

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



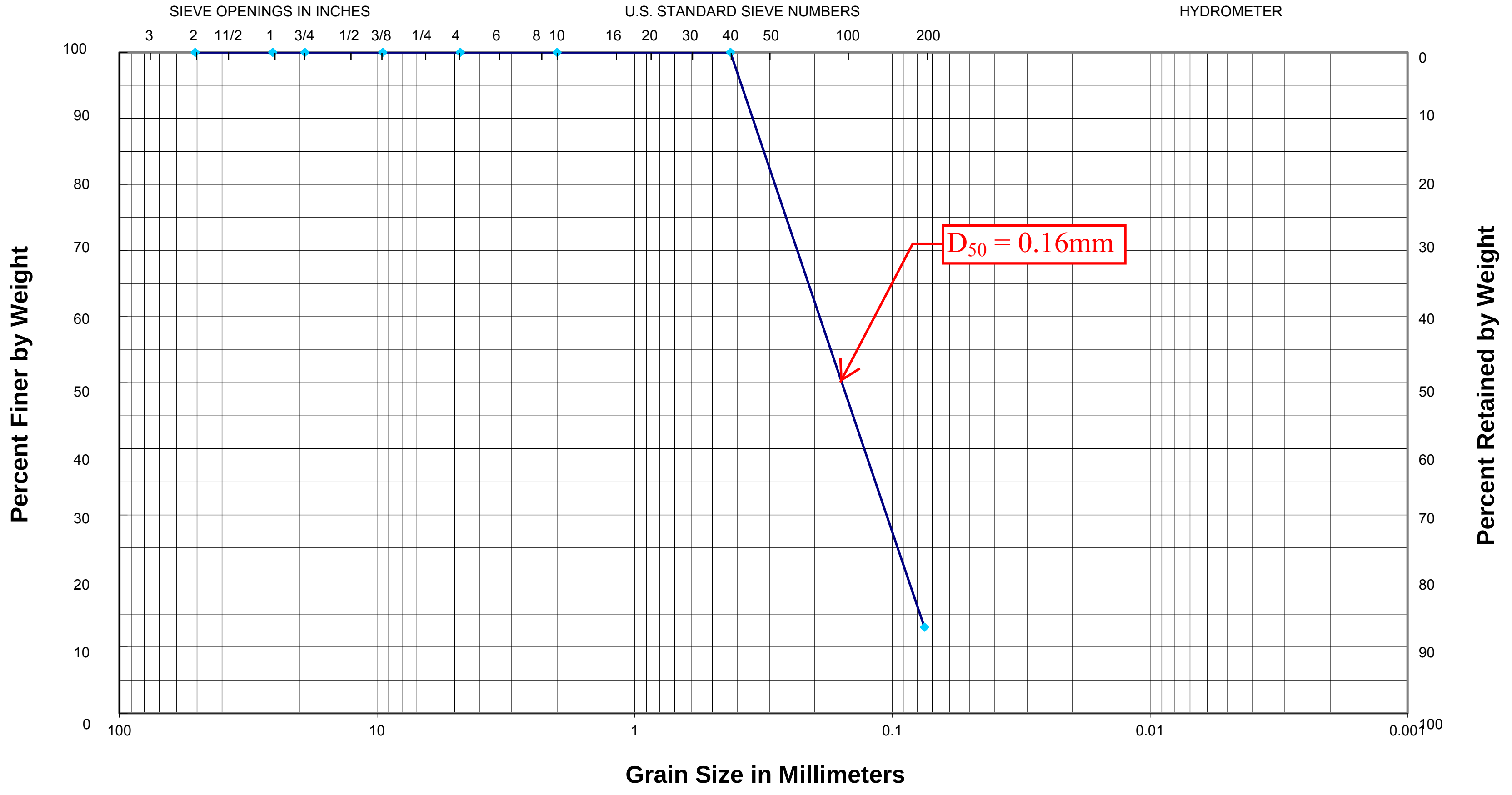
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 14, 24-24.5 ft
Properties: G_s = 2.668; Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-4

13-017

GRAIN SIZE CURVE



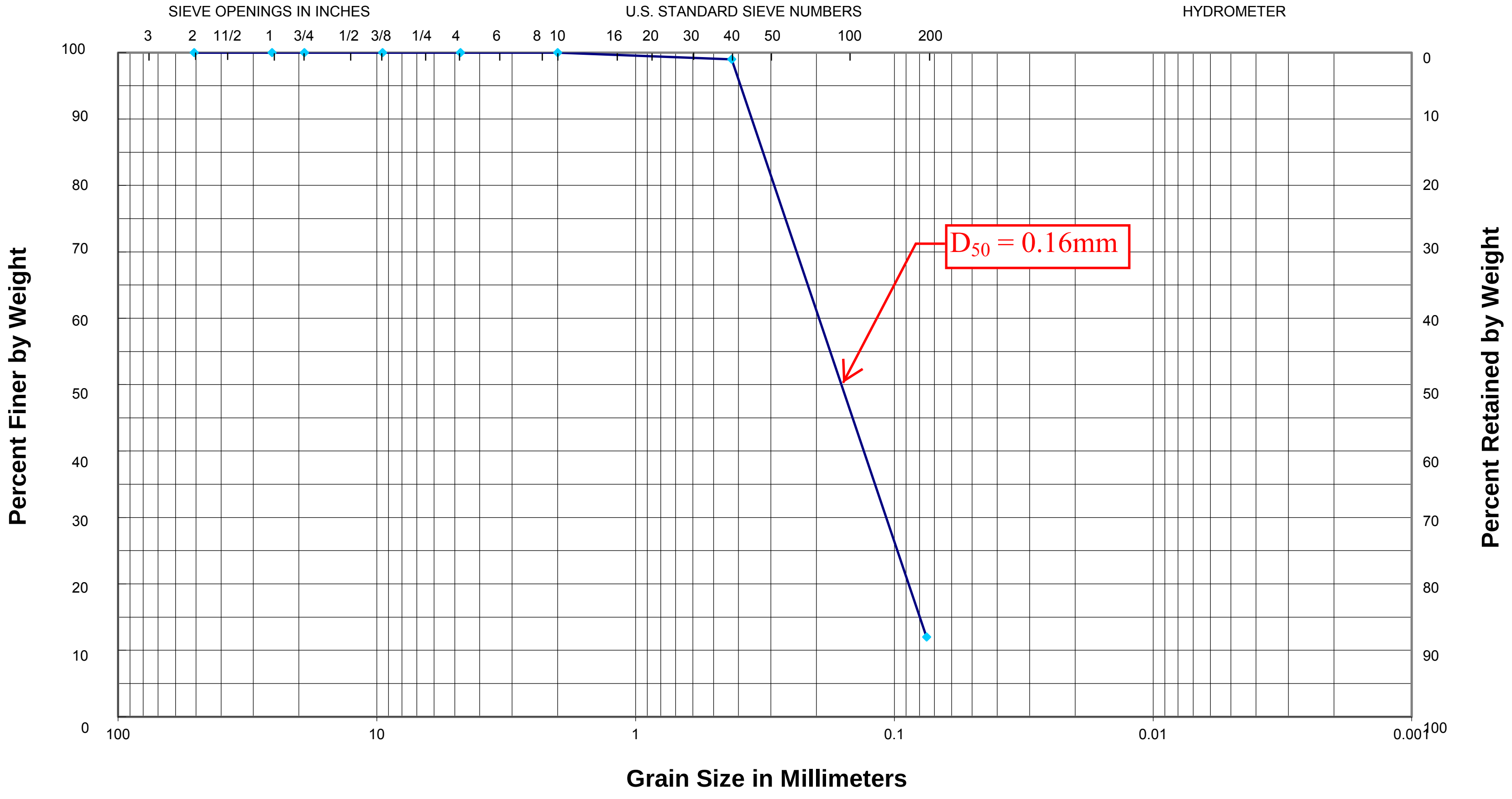
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 6.5-7.5 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



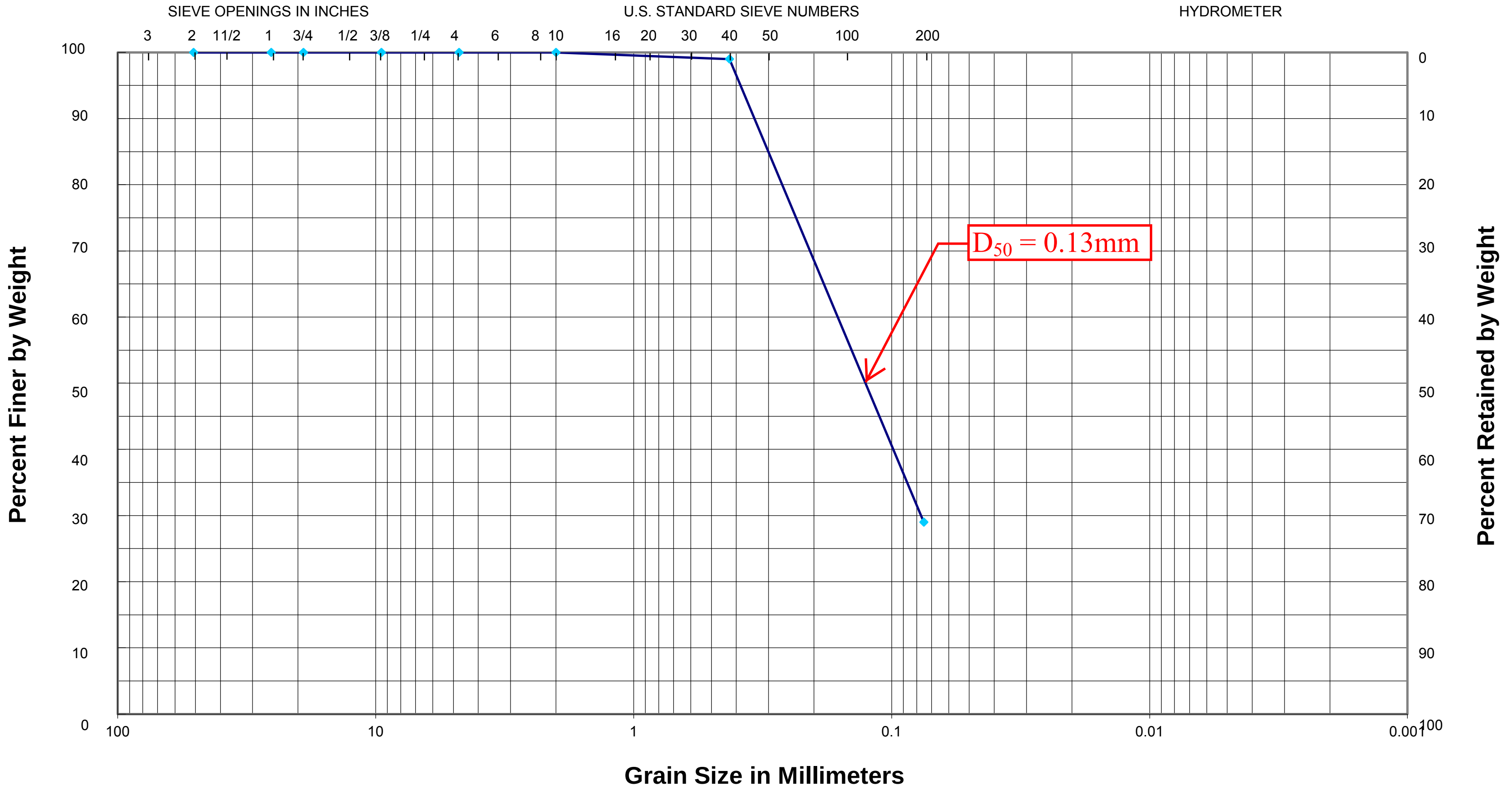
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 24-25 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



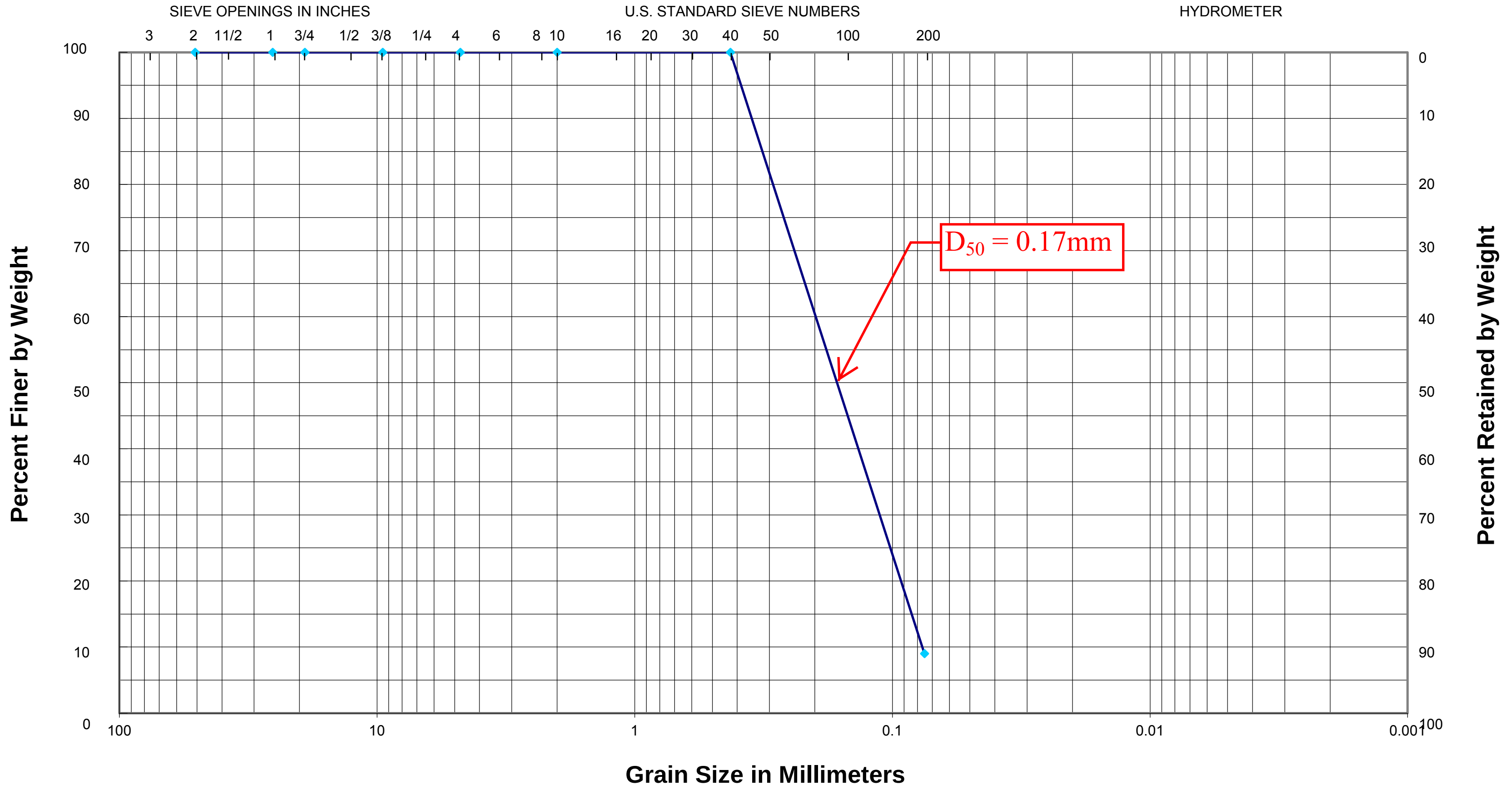
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



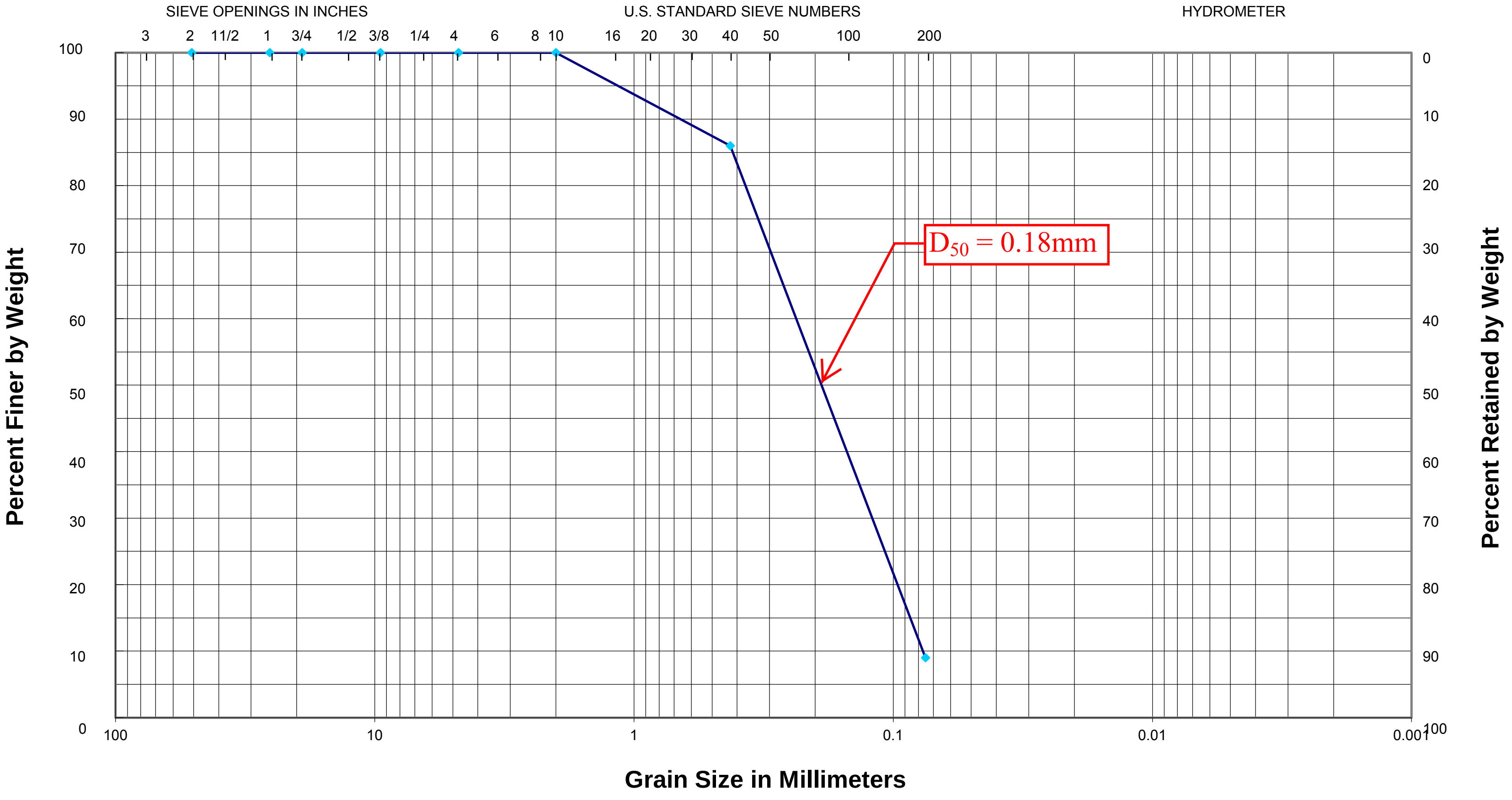
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

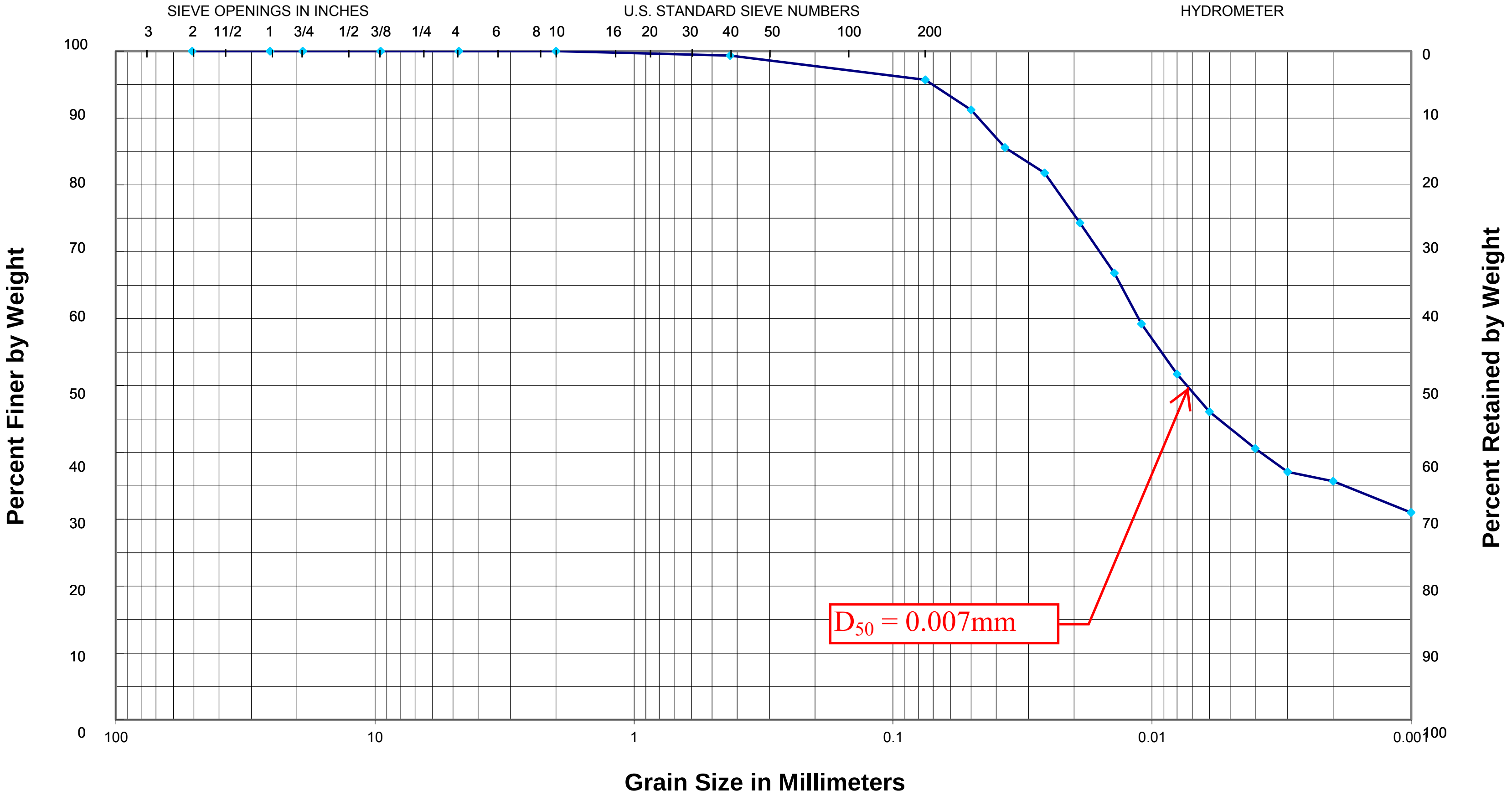
Sample: Boring 17, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

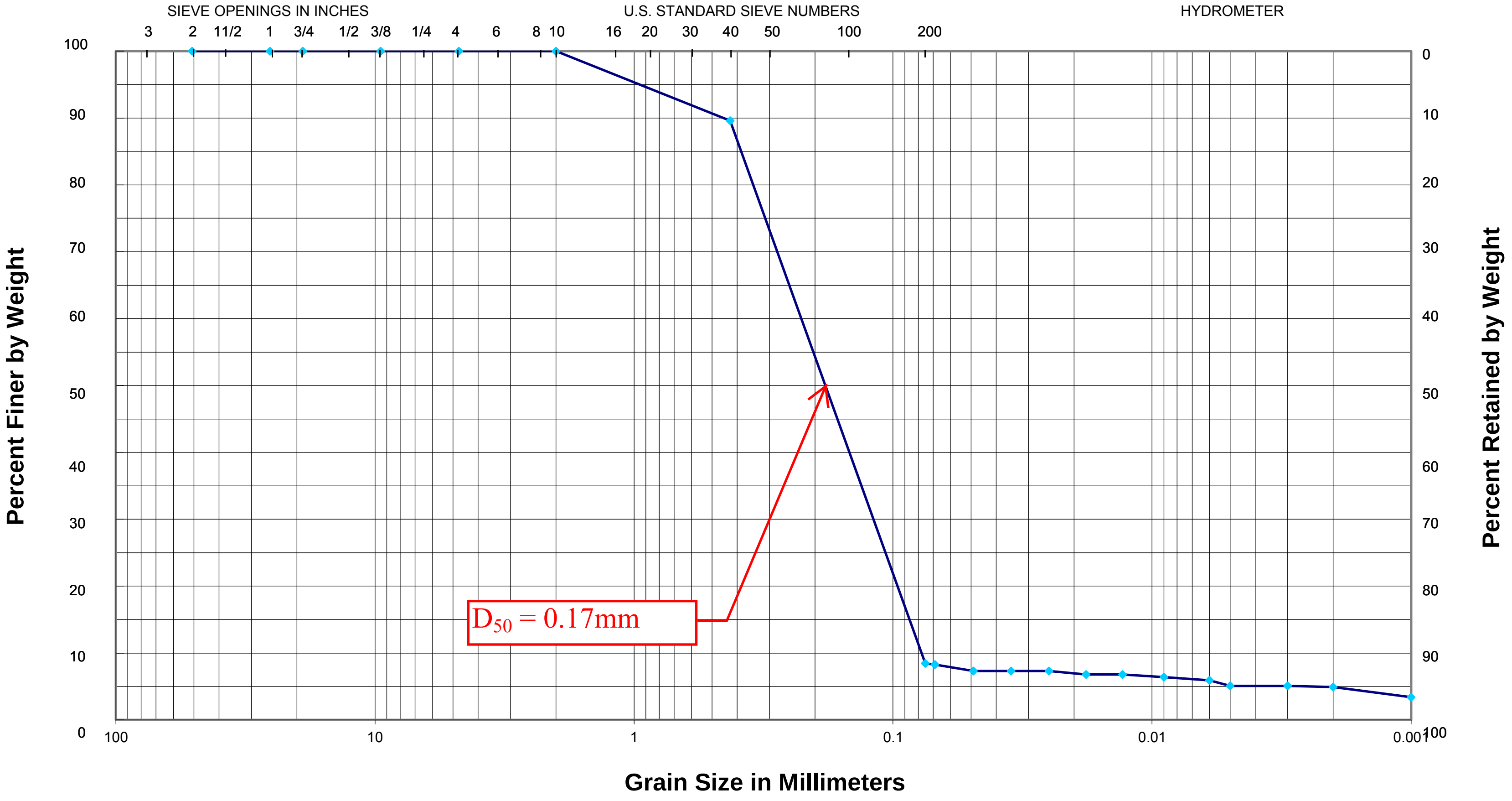
Sample: Boring 19, 6.5-7 ft
Properties: G_s = 2.696; LL = 40, PL = 18, PI = 22

Description: Gray and brown silty clay w/ferrous stains and nodules
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



D_{50} = 0.17mm

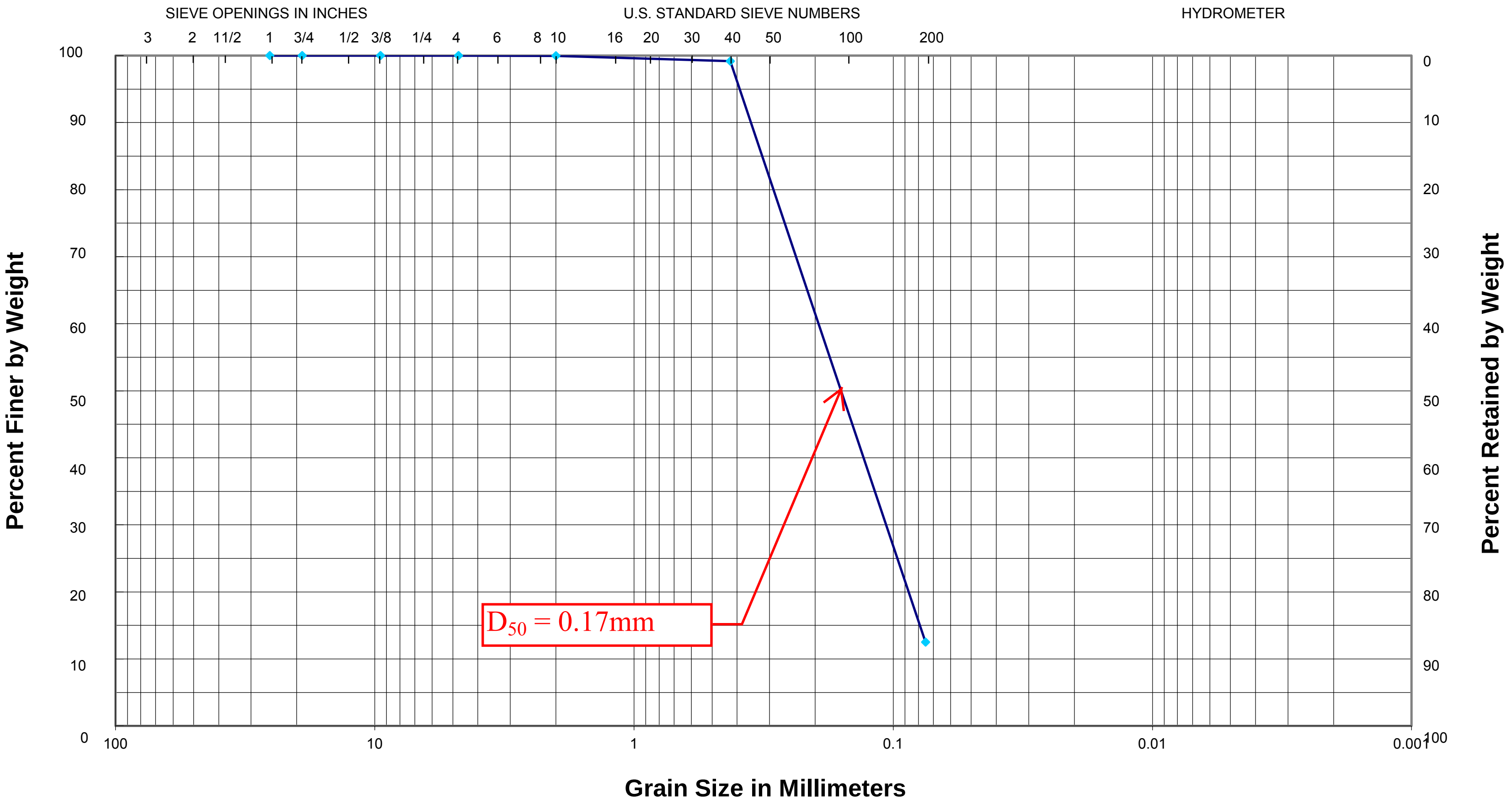
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 19, 24-25 ft
Properties: G_s = 2.678; Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = sp-sm; AASHTO = A-3

13-017

GRAIN SIZE CURVE



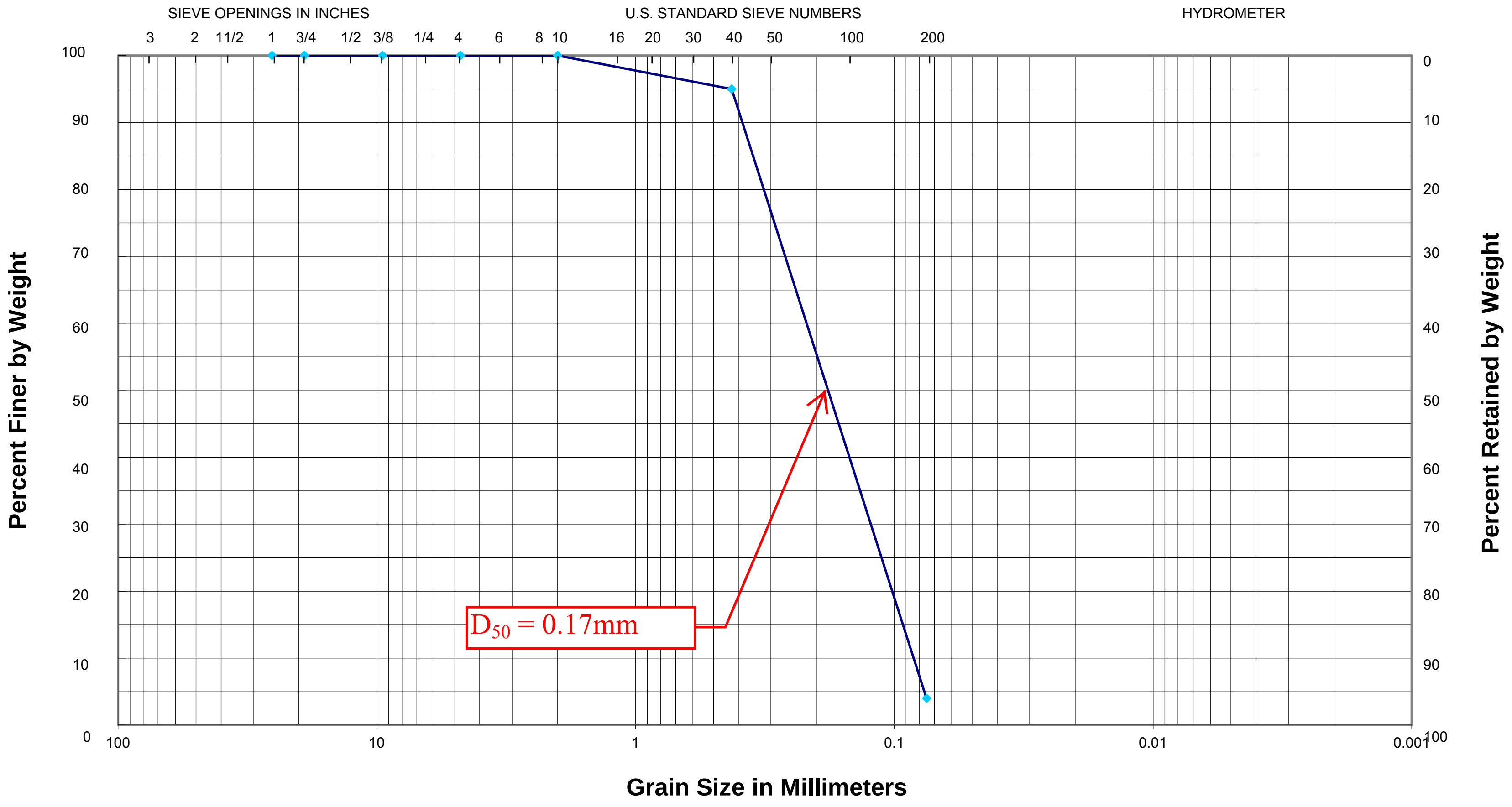
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



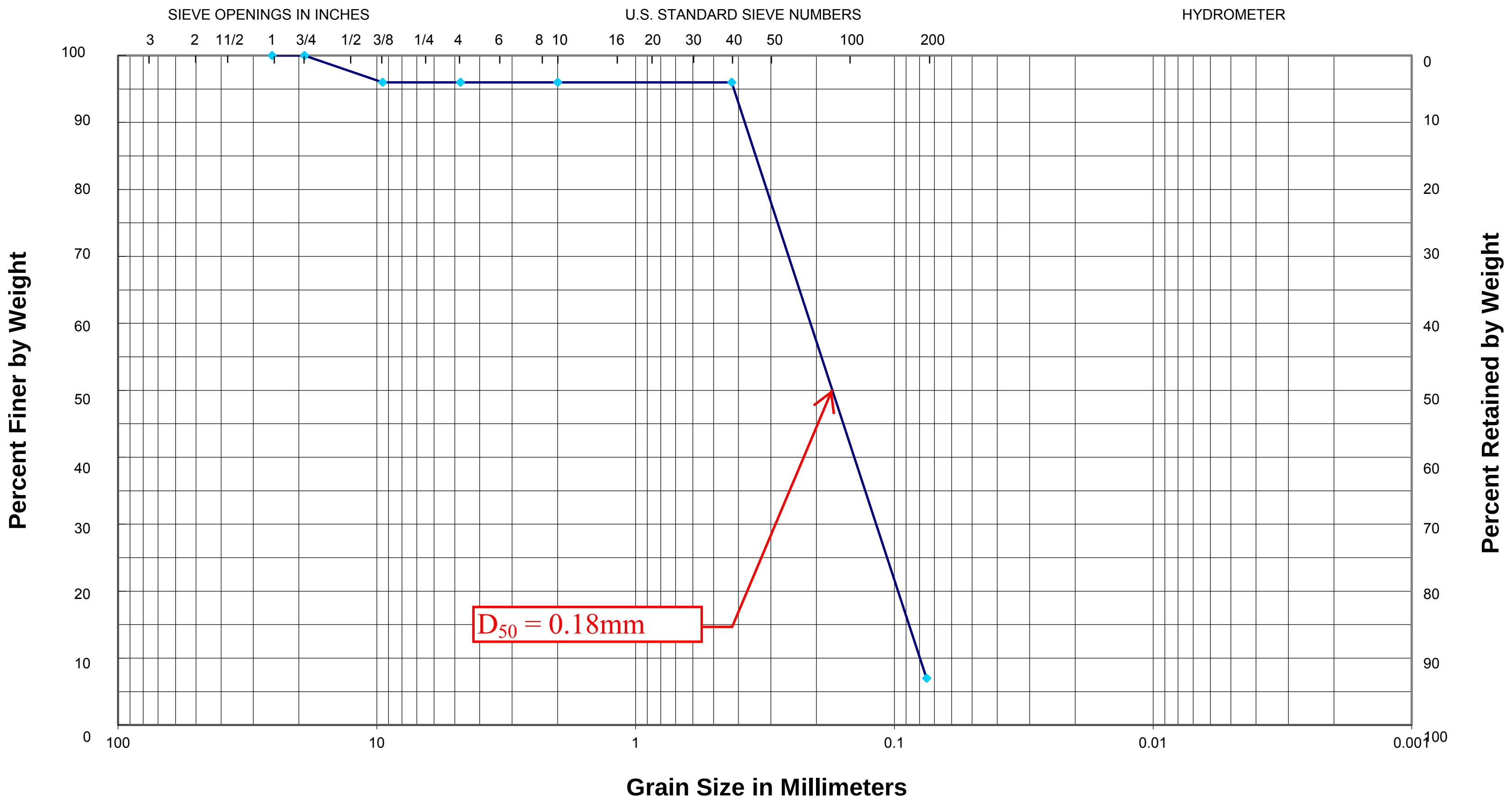
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 24-25 ft
Atterberg Limits: Non-plastic

Description: Grayish brown fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



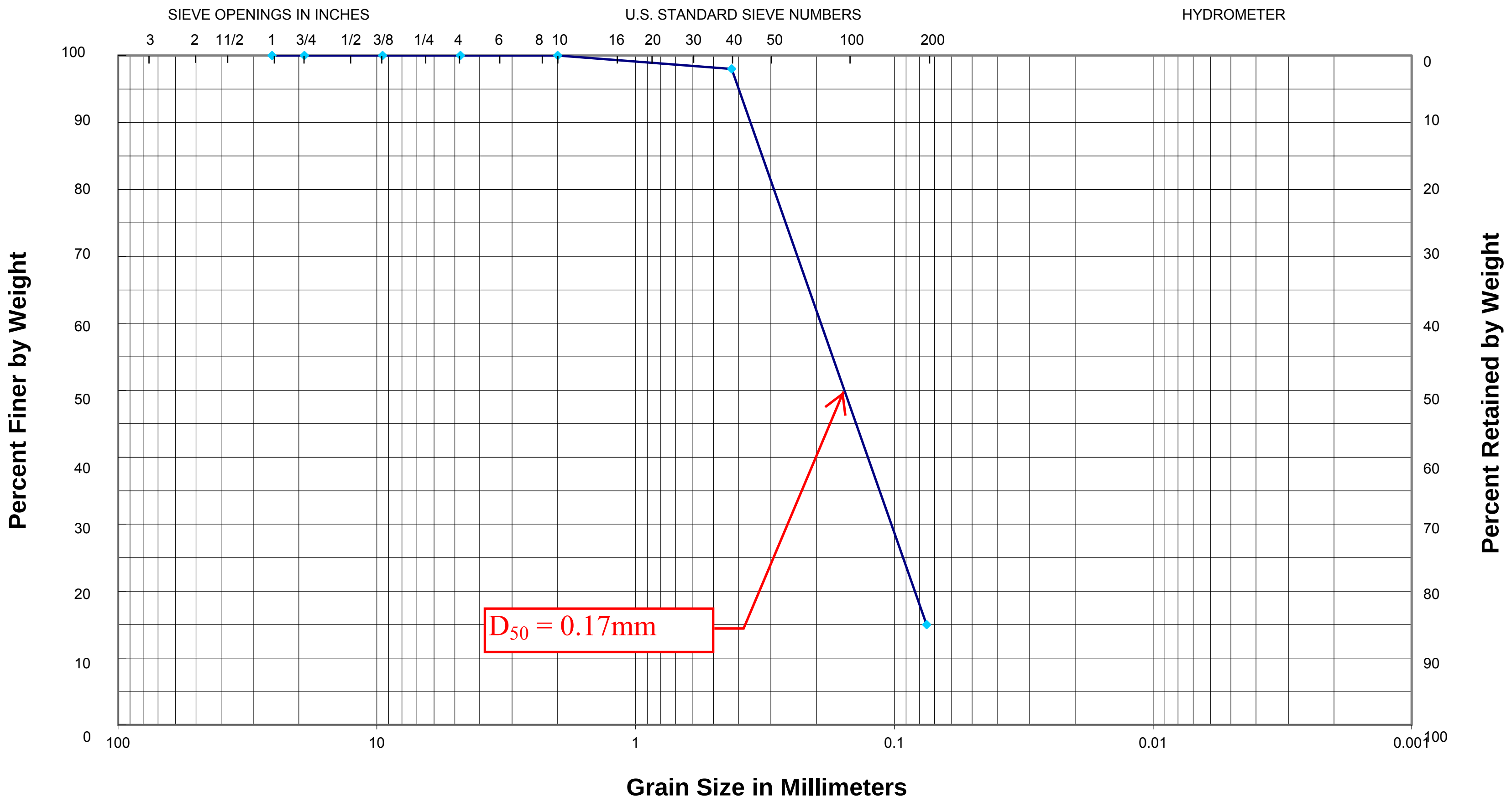
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 25, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

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

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

September 8, 2016
Job No. 13-017

Jacobs Engineering Group Inc.
10816 Executive Center Drive
Suite 300
Little Rock, Arkansas 72211

Attn: Mr. Mark A. Asher, P.E.

REF: RESULTS OF SUPPLEMENTAL BORINGS
AHTD JOB BB0610: WHITE RIVER STR. & APPRS. (F)
I-40 over WHITE RIVER - PRAIRIE COUNTY, ARKANSAS

Introduction

This submittal provides the final results of the supplemental borings performed for AHTD Job BB0610, White River Str. & Apprs. (F), in Prairie County, Arkansas. This project phase consisted of obtaining supplemental information on subsurface conditions in support of supplemental design related to the drilled shaft foundation alternate. Specifically, supplemental borings were performed to provide additional data on subsurface conditions in the vicinity of Piers 1 and 2. These supplemental services were verbally authorized by Jacobs on June 30, 2016.

Subsurface Exploration

Subsurface conditions at selected locations near the navigation channel, i.e., Piers 1 and 2, were investigated by drilling three (3) sample borings to 157- to 240-ft depth. The scope of work initially consisted of drilling two (2) borings. However, when casing was lost in one (1) boring, that boring had to be abandoned and a third boring was added to obtain subsurface data to the plan 200-ft exploration depth.

Two (2) borings (Borings 13B and 13C) were drilled within the footprint of Pier 1 and one (1) boring (Boring 14A) was drilled within the footprint of Pier 2. The approximate boring locations are shown on the Plan of Borings, Plate 1 of Attachment 1. The subsurface conditions encountered in the borings, and the results of field and laboratory tests, are shown on the boring logs, Plates 2 through 12 of Attachment 1. The approximate ground surface elevation, as inferred from the available topographic information, is also shown on each boring log. It must be recognized that the inferred elevations shown are approximate and actual elevations may vary. The approximate centerline station and offset of the boring locations are also noted on the logs. A key to the terms and symbols used on the logs is presented in Attachment 1 as Plate 13.

The borings were drilled with a truck-mounted Mobile B-53 rotary-drilling rig using a combination of dry-auger and rotatory-wash drilling techniques. The borings drilled at Pier 1, Borings 13B and 13C, were drilled from a segmental barge. The boring at Pier 2, Boring 14A, was drilled from a location on land. 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 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 a portion thereof, is defined as the Standard Penetration Number (N). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column. Selected soil samples were also obtained using a tube sampler.

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

Laboratory Testing

Laboratory tests consisting of natural water content determinations and classification tests were performed on selected representative soil samples to determine pertinent physical and engineering characteristics of the subsurface soils. Soil shear strength was estimated in the field using hand penetrometer and SPT results. In addition, laboratory strength testing included one (1) unconsolidated-undrained triaxial compression (AASHTO T 296) test. Undrained shear strength (cohesion) determined from the results of the compression test is plotted at the appropriate depth, in tons per sq ft, as an open triangle. Dry unit weight and natural water content were also determined as a part of the unconsolidated-undrained test.

A total of 45 natural water content determinations (AASHTO T 265) were performed to develop information on *in-situ* water content. Water contents are plotted on the logs as a solid circle in accordance with the scale and symbols shown in the legend located in the upper-right corner of the logs.

To verify field classification and to evaluate soil plasticity, 18 liquid and plastic limit (Atterberg limits, AASHTO T 89 and T 90) determinations and 34 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 percentage of soil passing through the No. 200 Sieve is noted in the "- No. 200 %" column on the appropriate boring log. Classification test results, as well as soil classification by the Unified Soil Classification System, are summarized in Attachment 2.

Closing

The results of the borings provided herein were obtained from Borings 13B, 13C and 14A in July and August 2016. Soil conditions elsewhere may vary from the subsurface conditions encountered in these borings. The results of these borings do not constitute a warranty or guarantee of subsurface conditions or foundation conditions, neither expressed nor implied. Differing subsurface conditions could be encountered during drilled shaft construction.

The following illustrations are attached and complete this submittal.

Attachment 1	Plan of Borings and Boring Logs
Attachment 2	Laboratory Test Results

* * * *

We appreciate the opportunity to be of continued service to you and the Arkansas State Highway and Transportation Department during this supplemental phase of the project. Should you have any questions regarding this information, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON & WYATT, INC.

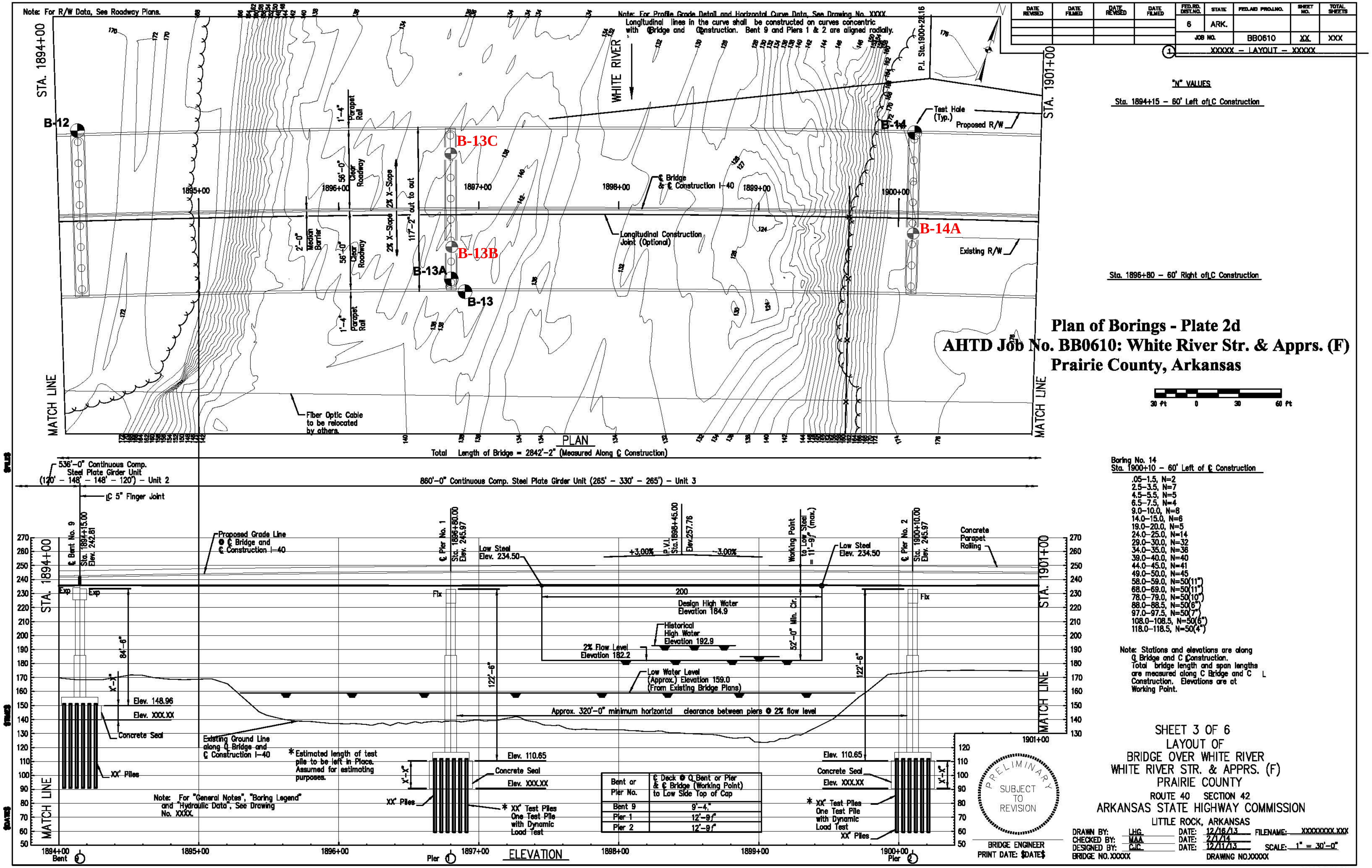
Mark E. Wyatt, P.E.
President

No. 7791
MARK E. WYATT

BJD/MEW:jw

Copies submitted: Jacobs Engineering Group Inc.
Attn: Mr. Mark A. Asher, P.E. (2+email)
Attn: Mr. Chris Criswell, P.E. (1-email)

ATTACHMENT 1



Note: Layout provided by Jacobs.

◐ Supplemental Boring Location

PLAN OF SUPPLEMENTAL BORINGS BB0610

PLATE 1

13-017

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Consulting Engineers

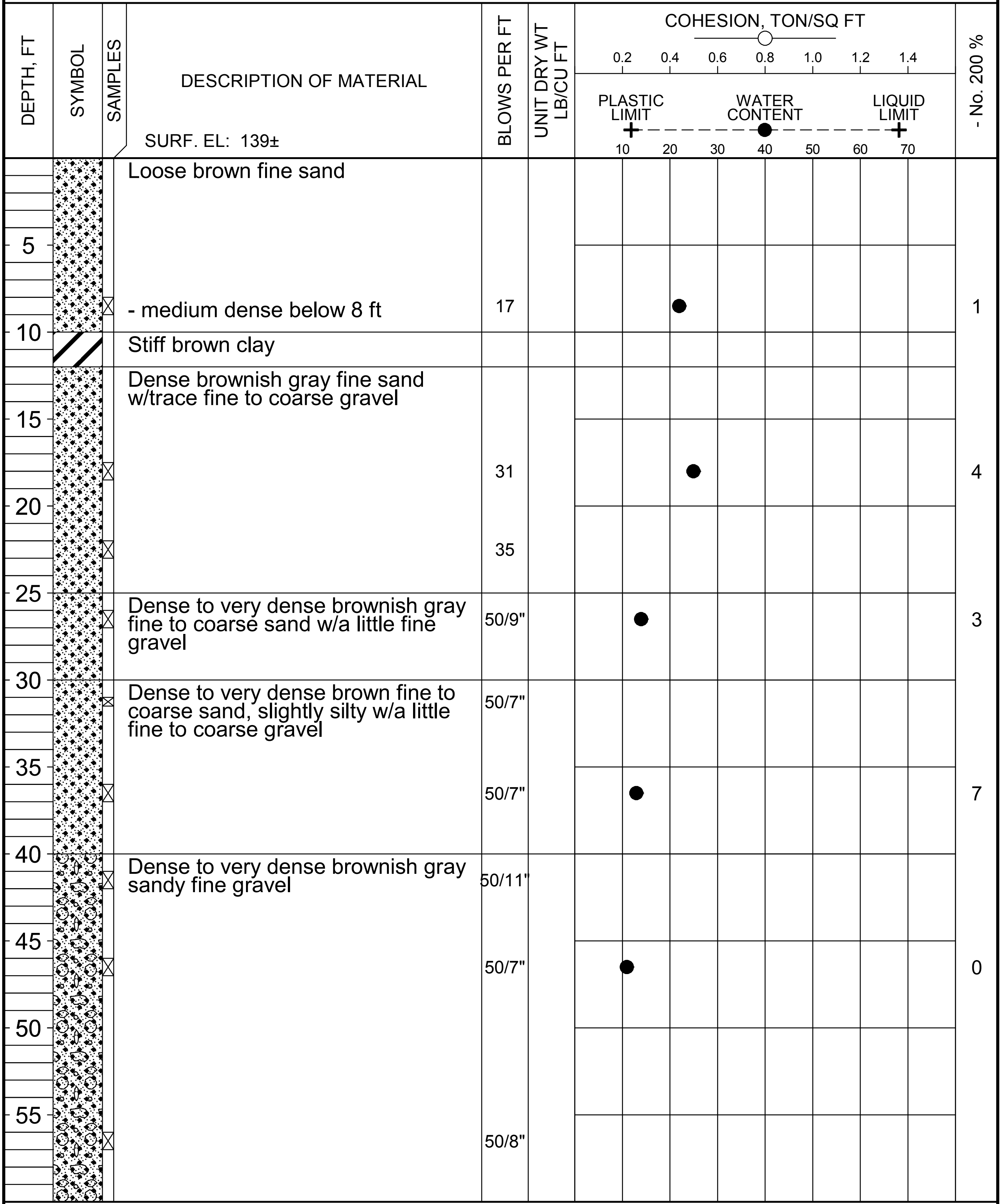
LOG OF BORING NO. 13B

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 25 ft Rt - Pier 1

SURF. EL: 139±



DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
Loose brown fine sand		
- medium dense below 8 ft	17	1
Stiff brown clay		
Dense brownish gray fine sand w/trace fine to coarse gravel	31	4
	35	
Dense to very dense brownish gray fine to coarse sand w/a little fine gravel	50/9"	3
Dense to very dense brown fine to coarse sand, slightly silty w/a little fine to coarse gravel	50/7"	
	50/7"	7
Dense to very dense brownish gray sandy fine gravel	50/11"	
	50/7"	0
	50/8"	

COMPLETION DEPTH: 157.0 ft
DATE: 7-14-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/14/2016

13-017

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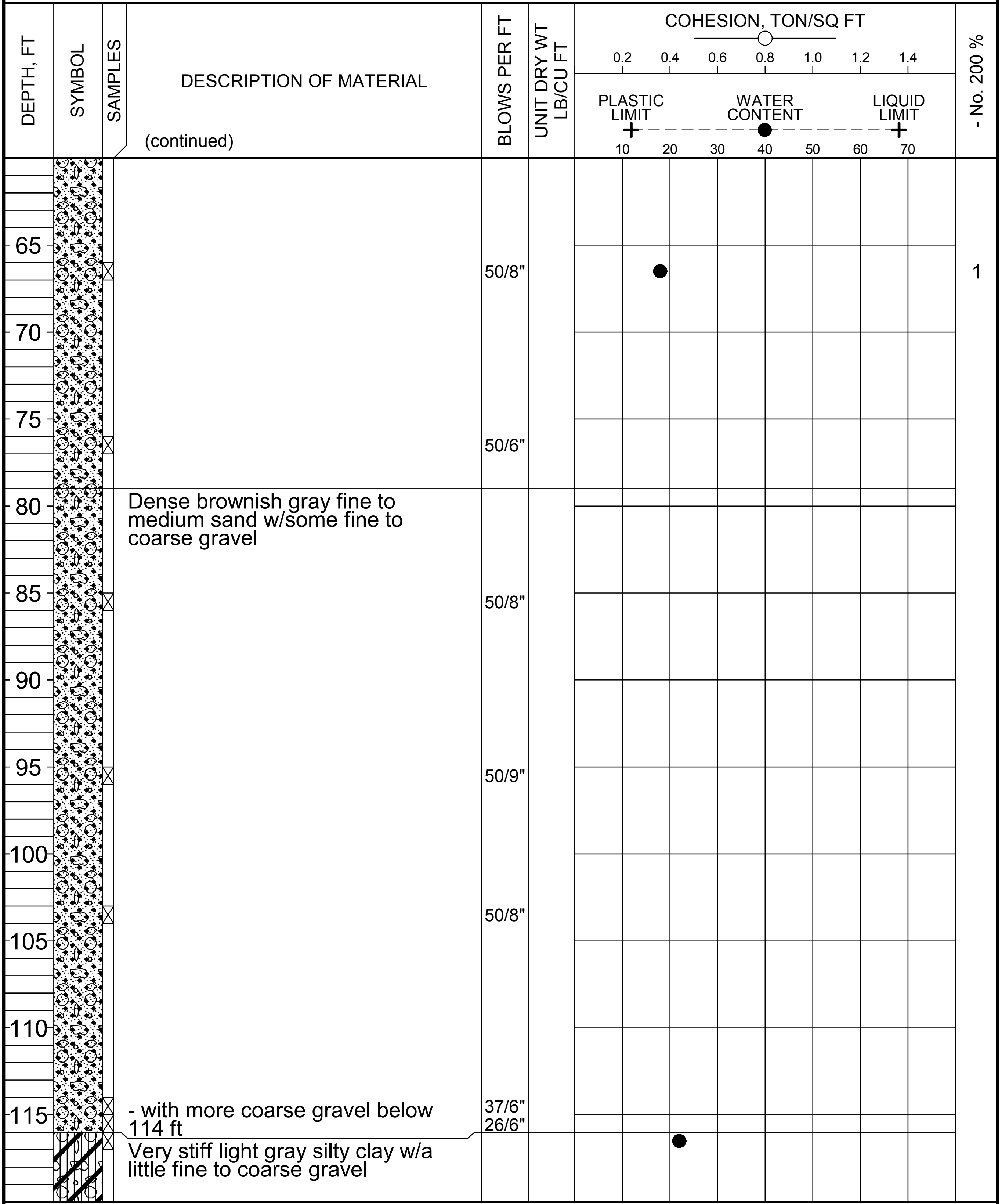
LOG OF BORING NO. 13B

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 25 ft Rt - Pier 1



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
66		50/8"		1
76		50/6"		
79	Dense brownish gray fine to medium sand w/some fine to coarse gravel			
85		50/8"		
95		50/9"		
103		50/8"		
114	- with more coarse gravel below 114 ft	37/6"		
115		26/6"		
116	Very stiff light gray silty clay w/a little fine to coarse gravel			

COMPLETION DEPTH: 157.0 ft
DATE: 7-14-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/14/2016

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

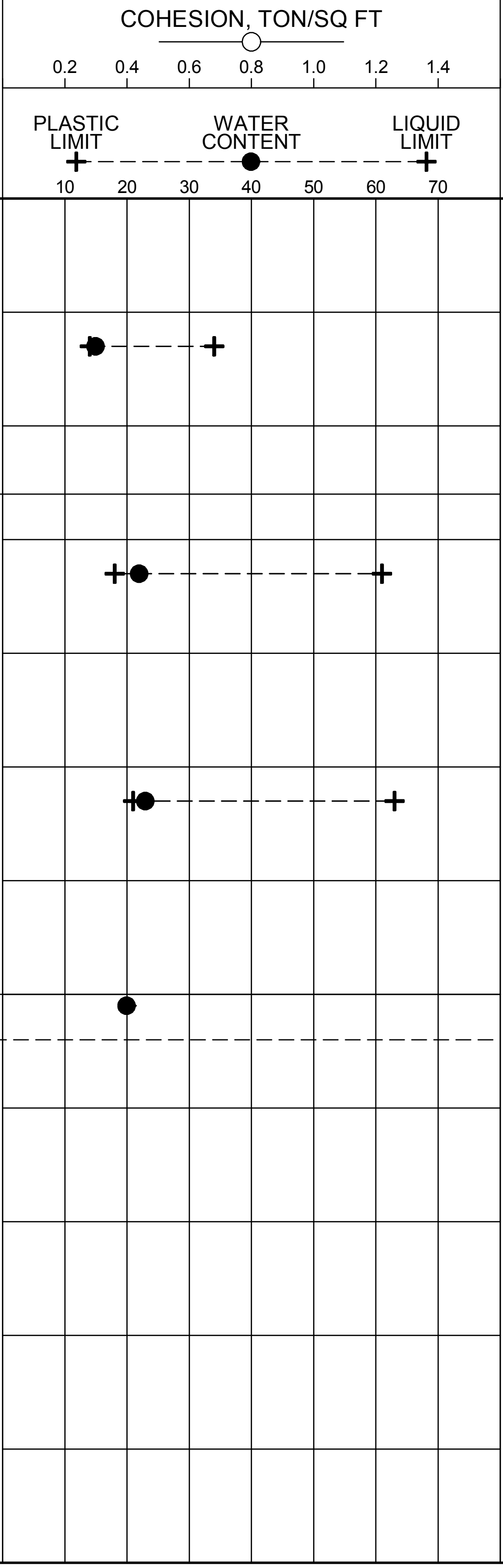
LOG OF BORING NO. 13B

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 25 ft Rt - Pier 1

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
125 130		☒	- with occasional interbedded fine to coarse gravel seams and layers below 120 ft	50/7"		67
135 140 145 150		☒ ☒	Hard brownish gray clay, slightly blocky w/some fine to coarse gravel and occasional lignite inclusions	50/8" 50/11"		53 98
155		☒	Hard greenish gray silty clay w/fine sand pockets and inclusions	50/4"		



NOTE 1: Water depth 28 ft on 7/11/16.
NOTE 2: Water depth 29 ft on 7/12/16.
NOTE 3: Set 70 ft casing, ± 41 ft below mudline.
NOTE 4: Lost casing at 157 ft.

Depth scale: 125, 130, 135, 140, 145, 150, 155, 160, 165, 170, 175

COMPLETION DEPTH: 157.0 ft
DATE: 7-14-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/14/2016

Form 108-6(74) Job No. 13-017

13-017

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L O G O F B O R I N G N O. 13C

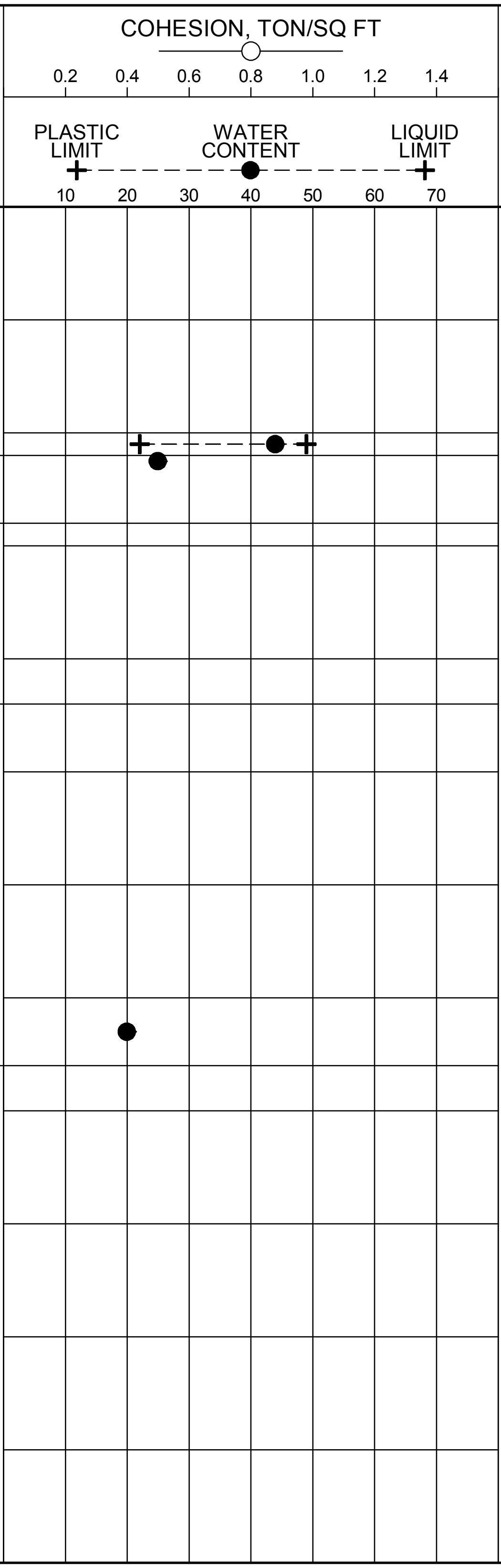
I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 40 ft Lt - Pier 1

SURF. EL: 138±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very soft to soft gray silty clay, wet			
5				
10	- stiff below 9 ft	12		99
	Medium dense brown silty fine sand			31
15	Dense to very dense brownish gray fine to medium sand, slightly silty w/trace fine gravel	50/11"		
20		50/8"		5
	Dense to very dense brownish gray fine sand, slightly silty w/a trace fine gravel			
25		48		
	- fine to medium sand, slightly silty below 28 ft			
30		50/7"		8
35	- silty fine sand below 35 ft	50/7"		15
	Dense brownish gray fine to coarse sand and fine gravel			
40		50/7"		1
45		50/6"		
50				
55		50/7"		



COMPLETION DEPTH: 215.5 ft
DATE: 7-29-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/22/2016

LGBNEW 13-017.GPJ 8-16-16

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Consulting Engineers

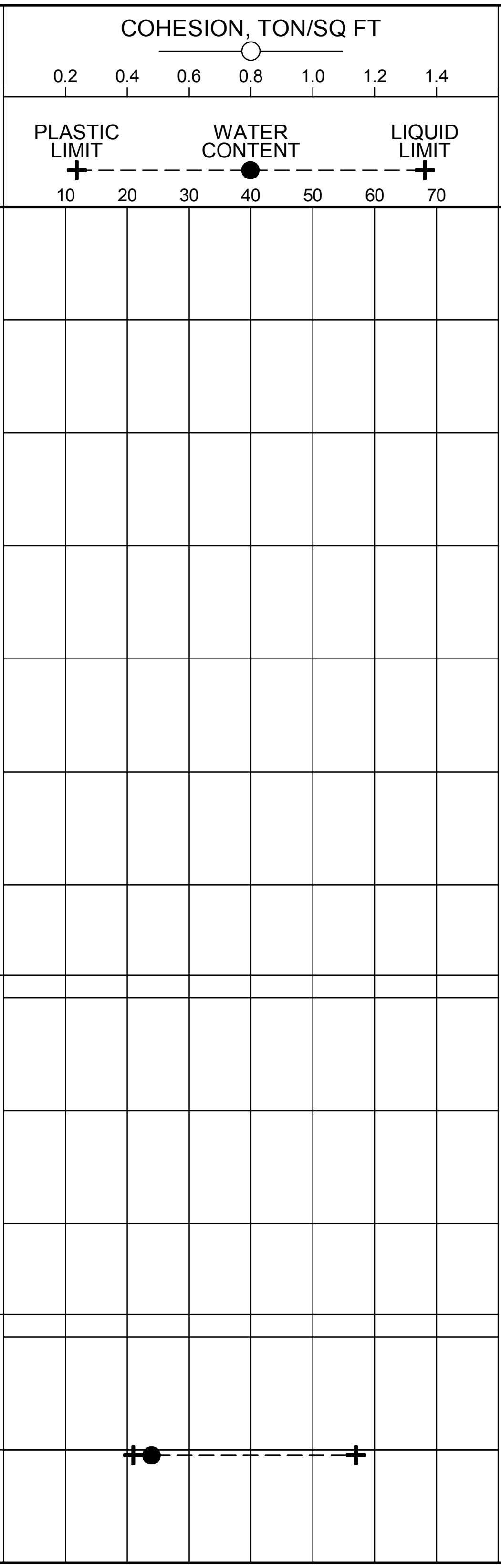
LOG OF BORING NO. 13C

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 40 ft Lt - Pier 1

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
65	- fine sand layer from 64 - 66 ft	50/9"		
75		50/6"		
85		50/6"		
95	Dense to very dense gray fine to coarse sand w/some fine gravel	50/6"		4
110	Very stiff to hard gray silty clay			
115	Hard brownish gray clay, slightly blocky	50/9"		98



COMPLETION DEPTH: 215.5 ft
DATE: 7-29-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/22/2016

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

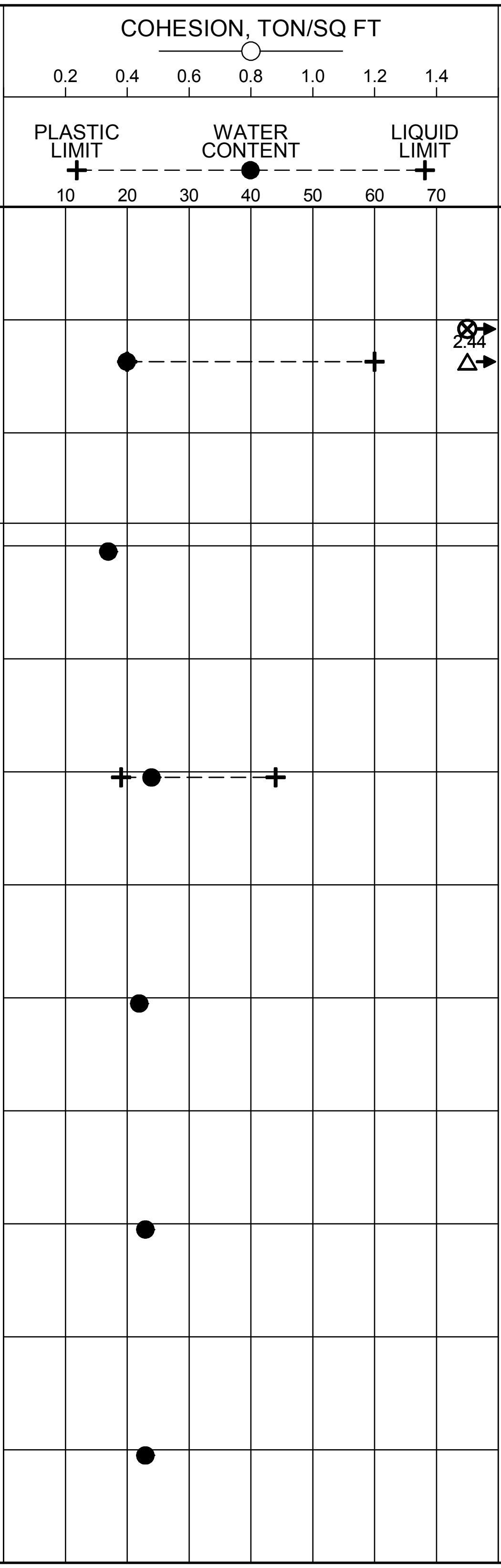
LOG OF BORING NO. 13C

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 40 ft Lt - Pier 1

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
122	Hard light gray clay			
127			109	100
134	Hard greenish gray silty clay, blocky w/silty fine sand partings and occasional lignite inclusions			
135		50/7"		
145		50/7"		80
155	- with some clayey fine sand seams and lignite inclusions below 155 ft	50/6"		
165	- with more fine sand and more lignite content below 165 ft	50/5"		
175		50/6"		



COMPLETION DEPTH: 215.5 ft
DATE: 7-29-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/22/2016

Form 108-6(74) Job No. 13-017

13-017

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

LOG OF BORING NO. 13C

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 40 ft Lt - Pier 1

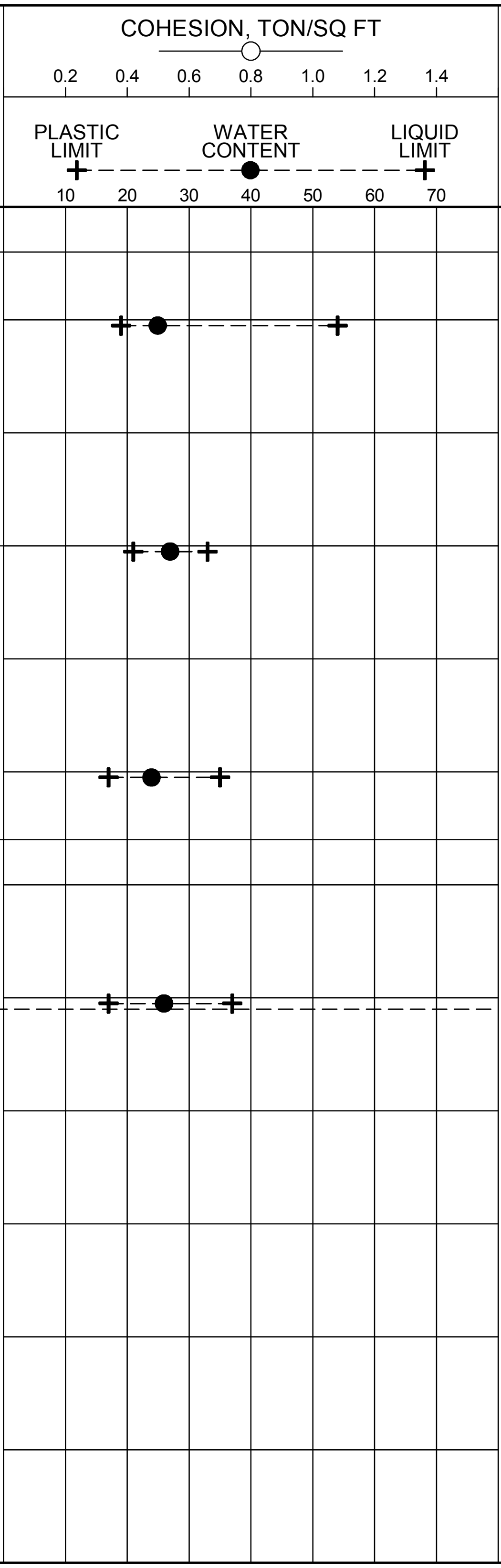
(continued)

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Hard light grayish green clay			
185		50/4"		98
195	Hard greenish gray fine sandy clay w/some clay seams and lignite inclusions	50/6"		57
200	- with interbedded fine sand partings and clay seams below 200 ft			
205		50/5"		60
	Hard greenish gray silty clay, blocky w/fine sand partings and occasional clayey fine sand seams			
215		50/6"		78

NOTE 1: Water at 29 ft on 7/19/16 - 7/22/16.

NOTE 2: Set 140 ft of casing as of 7/28/16.

NOTE 3: Water at 28 ft on 7/28/16, 27.5 ft on 7/29/16.



COMPLETION DEPTH: 215.5 ft
DATE: 7-29-16

DEPTH TO WATER
IN BORING: NA

DATE: 7/22/2016

LGBNEW 13-017.GPJ 8-16-16

Form 108-6(74) Job No. 13-017

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

LOG OF BORING NO. 14A

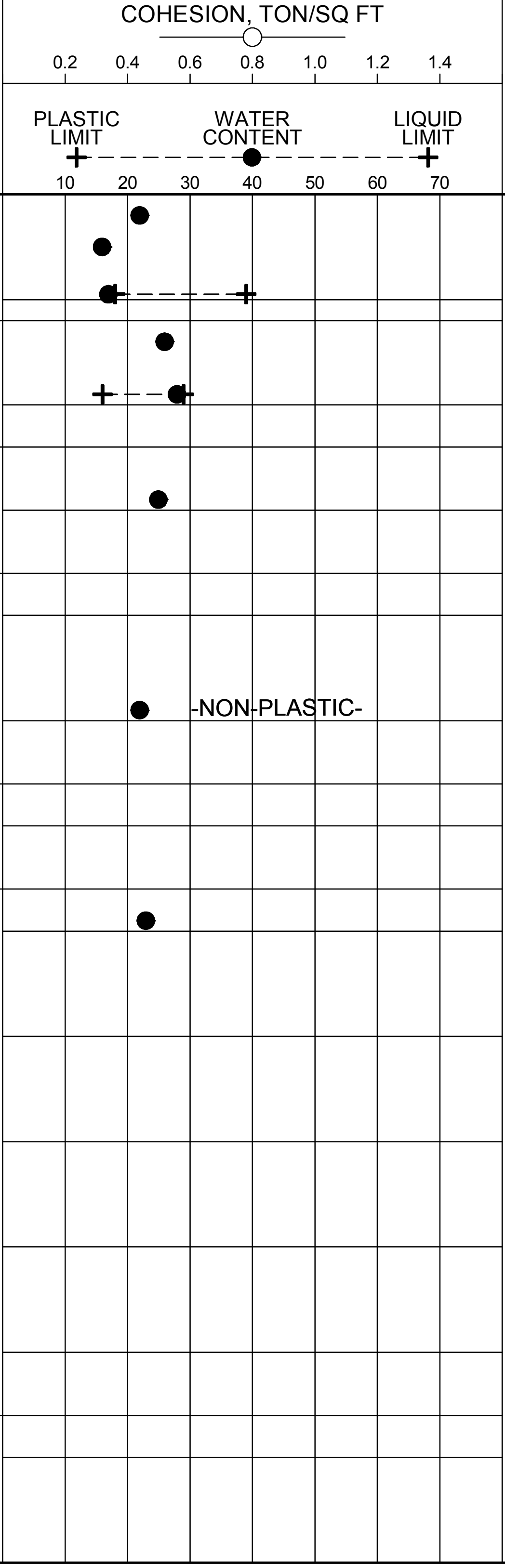
I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Sta 1900+10, 10 ft Rt - Pier 2

SURF. EL: 174±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very soft to soft brown and gray silty clay, slightly sandy w/silt partings, organic inclusions and organic stains, damp	4		
		11		
5	- stiff, dry below 2 ft	10		96
	Firm brown fine sandy clay	7		
10	- very soft below 8.5 ft	3		79
	- water at 12 ft			
15	Loose grayish brown silty fine sand w/ferrous stains	5		32
20	Very loose to loose gray silty fine sand w/interbedded silty clay seams and layers	4		
25	- medium dense below 23 ft	12		27
30	Loose gray silty fine sand	6		
35	Medium dense grayish brown fine sand, slightly silty	25		7
40		26		
45		23		
50	- dense to very dense below 48 ft	50/10"		
55				
60	Dense to very dense gray fine to coarse sand, slightly silty w/fine to coarse gravel and occasional organic inclusions	50/11"		



COMPLETION DEPTH: 240.0 ft
DATE: 8-9-16

DEPTH TO WATER
IN BORING: 12 ft

DATE: 8/4/2016

LGBNEW 13-017.GPJ 8-16-16

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

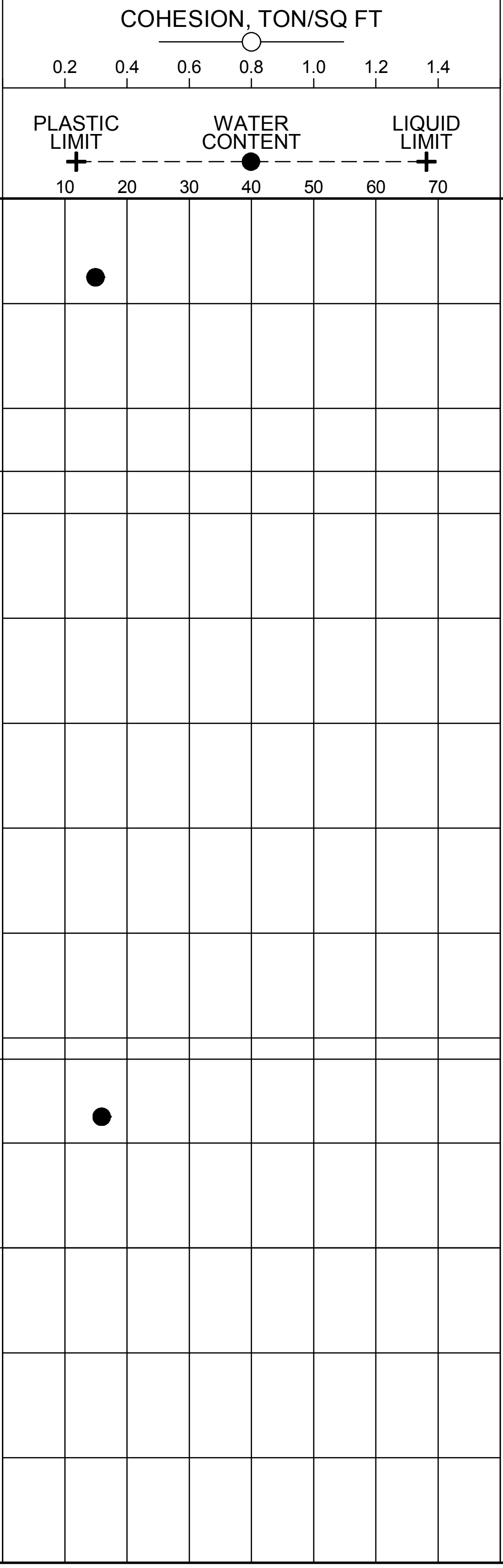
LOG OF BORING NO. 14A

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Sta 1900+10, 10 ft Rt - Pier 2

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
70		50/6"		7
80	Dense to very dense gray fine to coarse sand w/some fine to coarse gravel	50/7"		
90		50/4"		
100		50/7"		
110	Dense to very dense brownish gray fine to medium sand w/a little fine to coarse gravel	50/5"		3
120	Dense to very dense gray fine to coarse sand w/some fine to coarse gravel	50/4"		
125		50/4"		



COMPLETION DEPTH: 240.0 ft
DATE: 8-9-16

DEPTH TO WATER
IN BORING: 12 ft

DATE: 8/4/2016

Form 108-6(74) Job No. 13-017

13-017

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

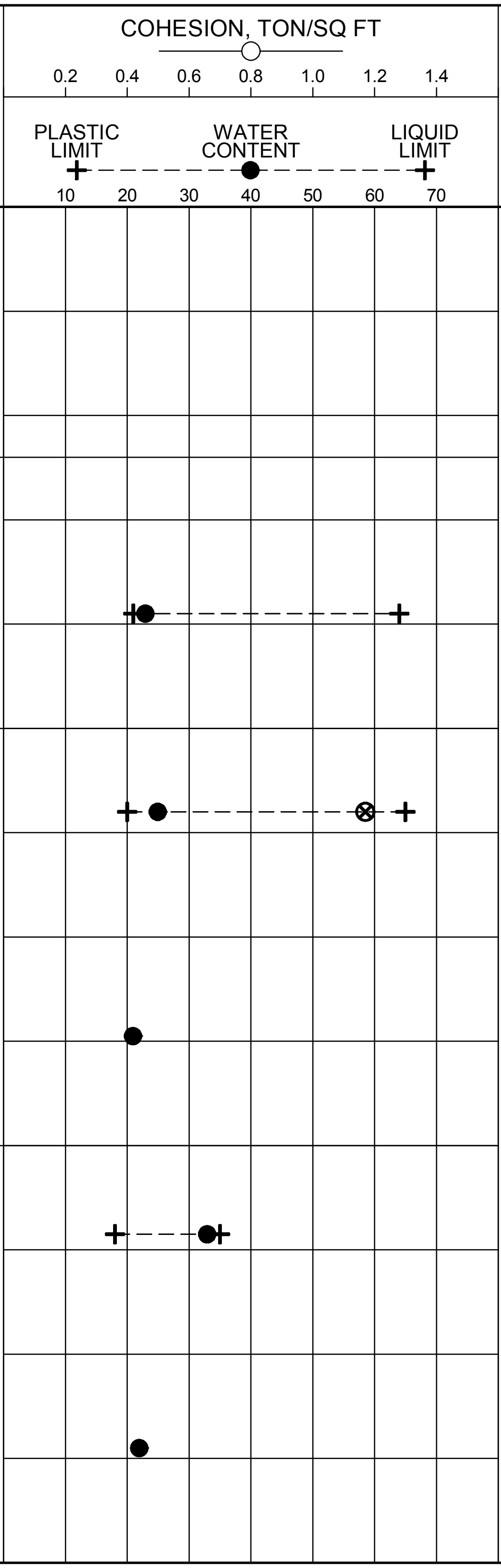
LOG OF BORING NO. 14A

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Sta 1900+10, 10 ft Rt - Pier 2

DEPTH, FT	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
135					
	☒		50/4"		
140					
		Hard brownish gray clay, blocky			
145					
150	☒		50/8"		100
155		Hard light gray clay, blocky (NOTE: Sample Disturbed)			
160	■			104	53
165					
170	☒		50/6"		
175		Hard brownish gray fine sandy clay w/fine sand partings and pockets and organic stains			
180	☒		50/5"		64
185					
190	☒		50/7"		



COMPLETION DEPTH: 240.0 ft
DATE: 8-9-16

DEPTH TO WATER
IN BORING: 12 ft

DATE: 8/4/2016

LGBNEW 13-017.GPJ 8-16-16

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

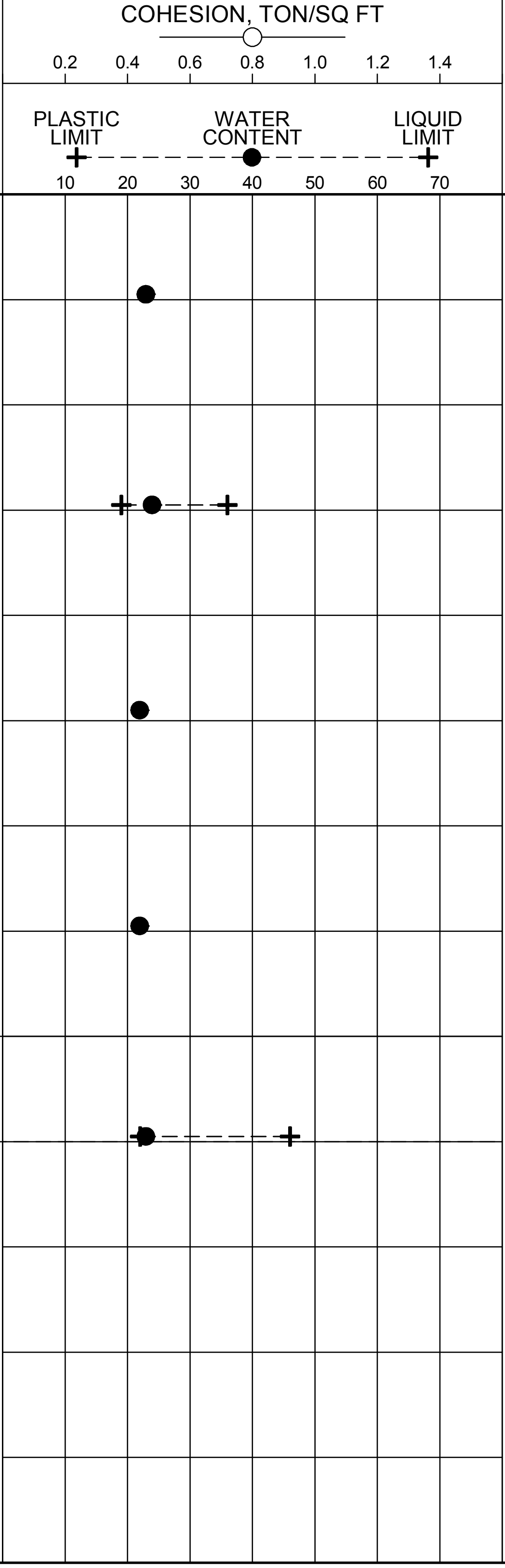
LOG OF BORING NO. 14A

I-40 over White River - AHTD Job BB0610
Prairie County, Arkansas

TYPE: Auger to 30 ft /Wash

LOCATION: Sta 1900+10, 10 ft Rt - Pier 2

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
			- with lignite inclusions below 195 ft			
200		☒		50/6"		
205			- with decreasing fine sand seams and pockets below 205 ft			
210		☒		50/5"		81
215			- with more lignite inclusions below 215 ft			
220		☒		50/8"		
225						
230		☒		50/4"		
235			Medium dense greenish gray silty clay w/fine sand partings			
240		☒	NOTE: Set 30 ft casing.	50/6"		98
245						
250						
255						



COMPLETION DEPTH: 240.0 ft
DATE: 8-9-16

DEPTH TO WATER
IN BORING: 12 ft

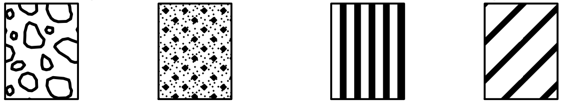
DATE: 8/4/2016

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

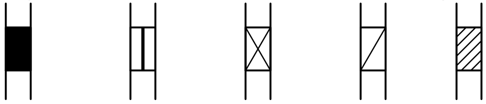
(SHOWN IN SYMBOLS COLUMN)



Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



TERMS DESCRIBING CONSISTENCY OR CONDITION

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

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

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

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

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

TERMS CHARACTERIZING SOIL STRUCTURE

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

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

KEY 8-16-16

ATTACHMENT 2

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job BB0610
LOCATION: Prairie County, Arkansas
JOB NUMBER: 13-017

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		
13B	8-9	22	---	---	---	---	---	---	---	---	---	1	SP	A-3
13B	17.5-18.5	25	---	---	---	100	100	100	100	100	95	4	SP	A-3
13B	26.5-27.3	14	---	---	---	100	94	88	80	74	22	3	SW	A-1-b
13B	36.5-37	13	---	---	---	100	100	94	83	68	38	7	SP-SM	A-1-b
13B	46.5-47.5	11	---	---	---	100	100	64	44	13	1	0	GP	A-1-a
13B	66.5-67.5	18	---	---	---	100	97	83	36	10	3	1	GP	A-1-a
13B	126-126.5	15	34	14	20	100	84	71	71	70	69	67	CL	A-6
13B	136-137	22	61	18	43	100	85	65	61	58	55	53	CH	A-7-6
13B	146.5-147	23	63	21	42	---	---	---	---	---	---	98	CH	A-7-6

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job BB0610
LOCATION: Prairie County, Arkansas
JOB NUMBER: 13-017

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		
13C	10-11	44	49	22	27	---	---	---	---	---	---	99	CL	A-7-6
13C	11-11.5	25	---	---	---	---	---	---	---	---	---	31	SM	A-2-4
13C	20-20.7	25	---	---	---	100	94	84	60	32	19	5	SP-SM	A-1-b
13C	30.5-31	19	---	---	---	100	89	83	74	67	57	8	SP-SM	A-1-b
13C	35.5-36	20	---	---	---	100	100	96	92	90	85	15	SM	A-2-4
13C	40.5-41	18	---	---	---	100	100	81	63	40	7	1	SP	A-1-a
13C	95-95.5	19	---	---	---	100	96	83	71	59	32	4	SP	A-1-b
13C	115-115.7	24	57	21	36	---	---	---	---	---	---	98	CH	A-7-6
13C	125-125.5	19	60	20	40	---	---	---	---	---	---	100	CH	A-7-6
13C	145.5-146	24	44	19	25	---	---	---	---	---	---	80	CL	A-7-6
13C	185-185.5	25	54	19	35	---	---	---	---	---	---	98	CH	A-7-6
13C	195-195.5	27	33	21	12	---	---	---	---	---	---	57	CL	A-6
13C	205-205.5	24	35	17	18	---	---	---	---	---	---	60	CL	A-6
13C	215-215.5	26	37	17	20	---	---	---	---	---	---	78	CL	A-6

SUMMARY of CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job BB0610
LOCATION: Prairie County, Arkansas
JOB NUMBER: 13-017

Boring No.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			Percent Passing No. 200, %	UNIFIED CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX			
14A	4.5-5.5	17	39	18	21	96	CL	A-6
14A	9-10	28	29	16	13	76	CL	A-6
14A	14-15	25	--	--	--	32	SM	A-2-4
14A	24-25	22	NP	NP	NP	27	SM	A-2-4
14A	34-35	23	--	--	--	7	SP-SM	A-2-4
14A	68.5-69	15	--	--	--	7	SP-SM	A-2-4
14A	108.5-109	16	--	--	--	3	SP	A-2-4
14A	149-149.7	23	64	21	43	100	CH	A-7-6
14A	178.5-179	23	35	18	17	64	CL	A-6
14A	209.5-209.9	24	36	19	17	81	CL	A-6
14A	239.5-240	23	46	22	24	98	CL	A-7-6

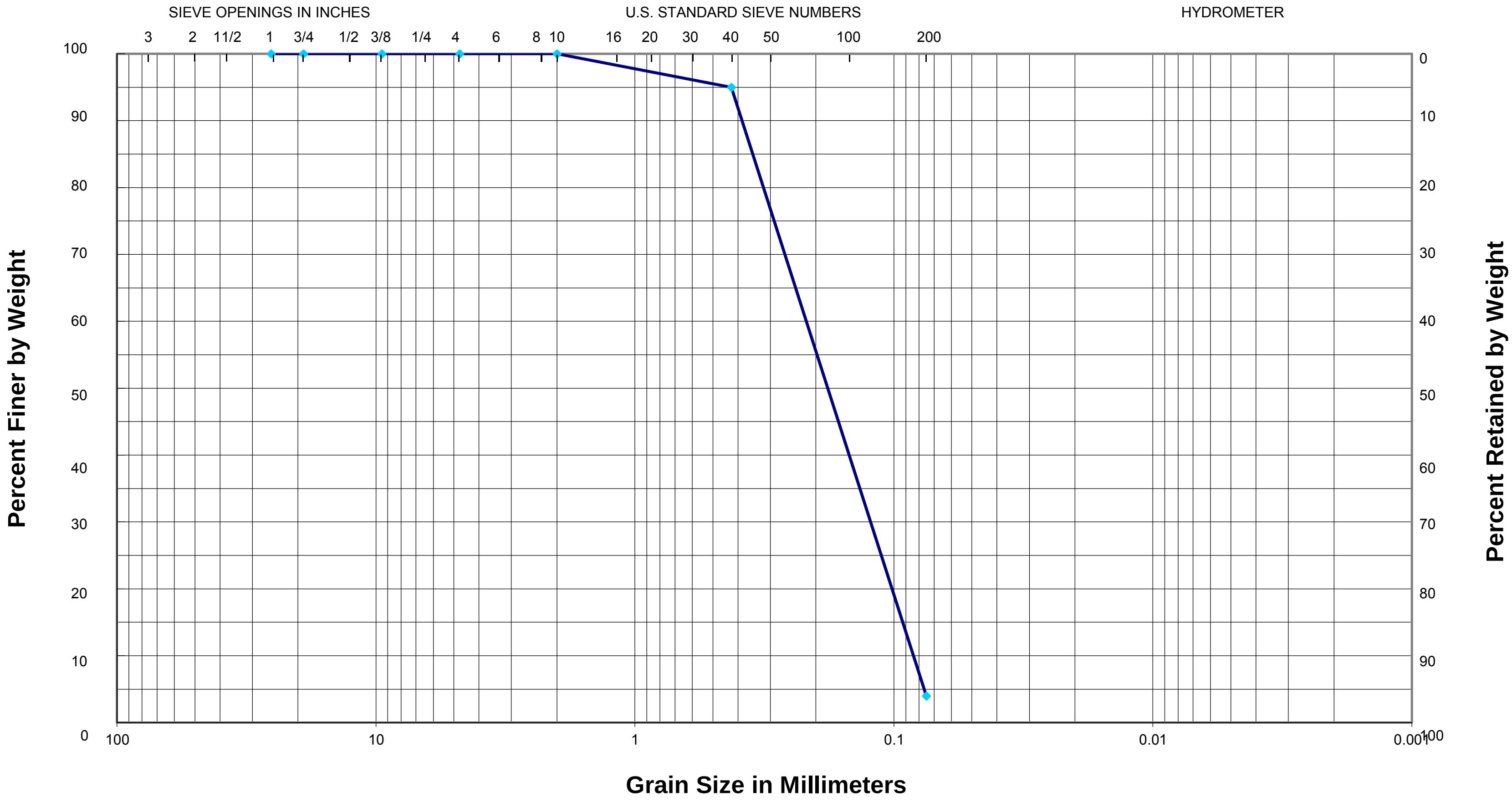
Grubbs, Hoskyn,
Barton & Wyatt, Inc.
CONSULTING ENGINEERS

PLATE

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

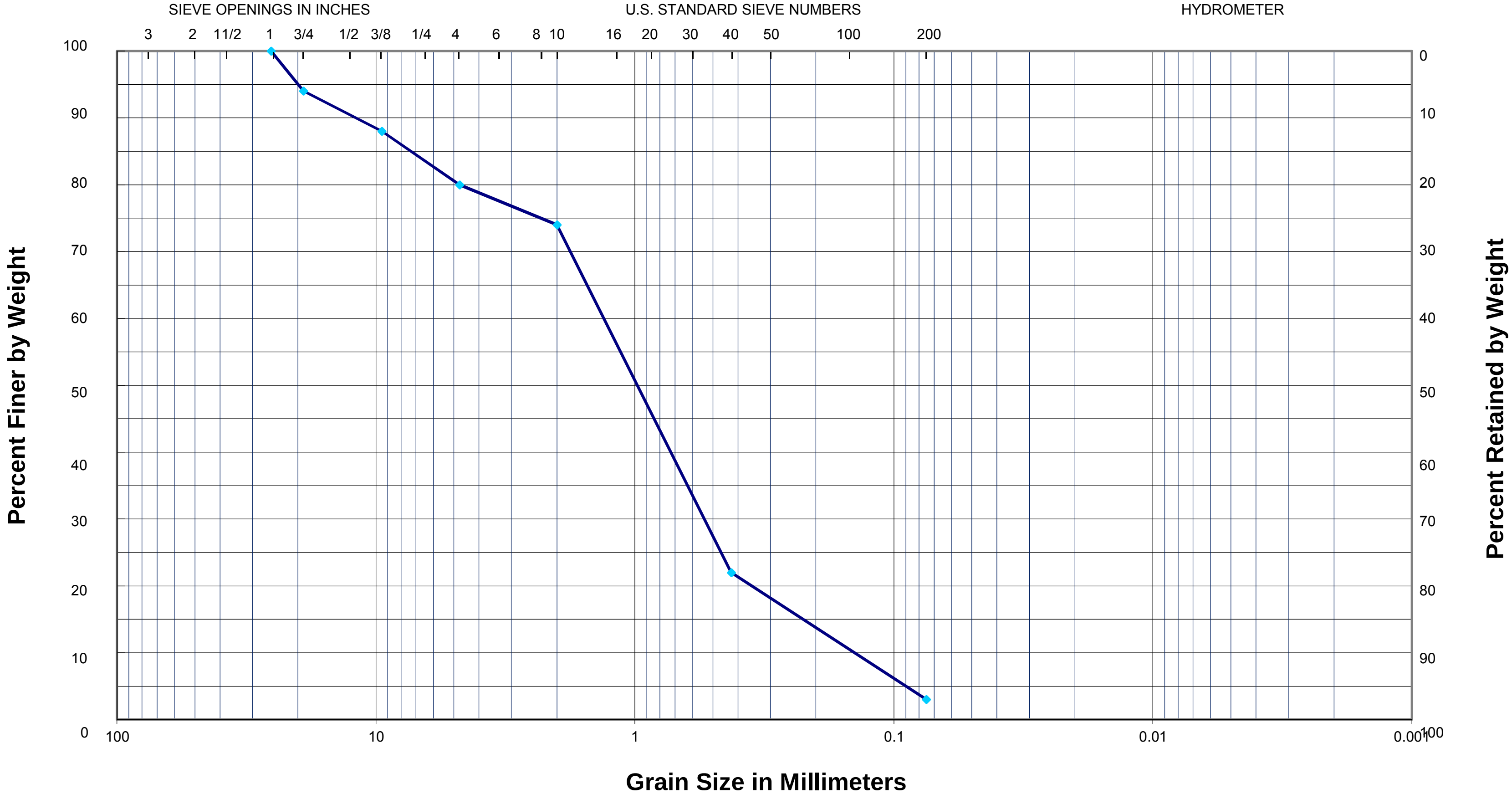
Sample: Boring 13B 17.5 - 18.5 ft
Description: Brownish gray fine SAND

USCS = SP AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

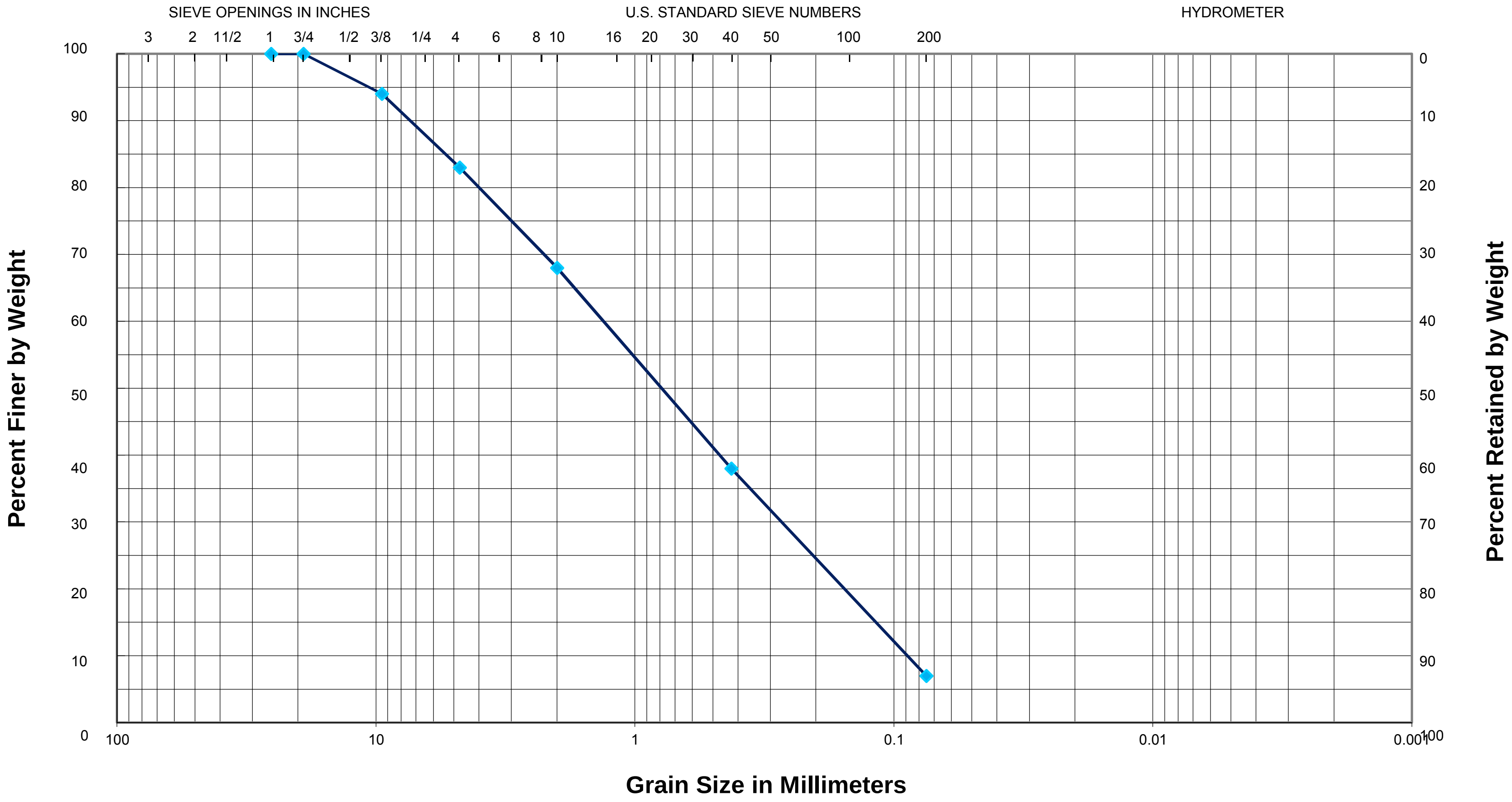
Sample: Boring 13B 26.5 - 27.3 ft
Description: Brownish gray fine to coarse SAND
with a little fine gravel

USCS = SW AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

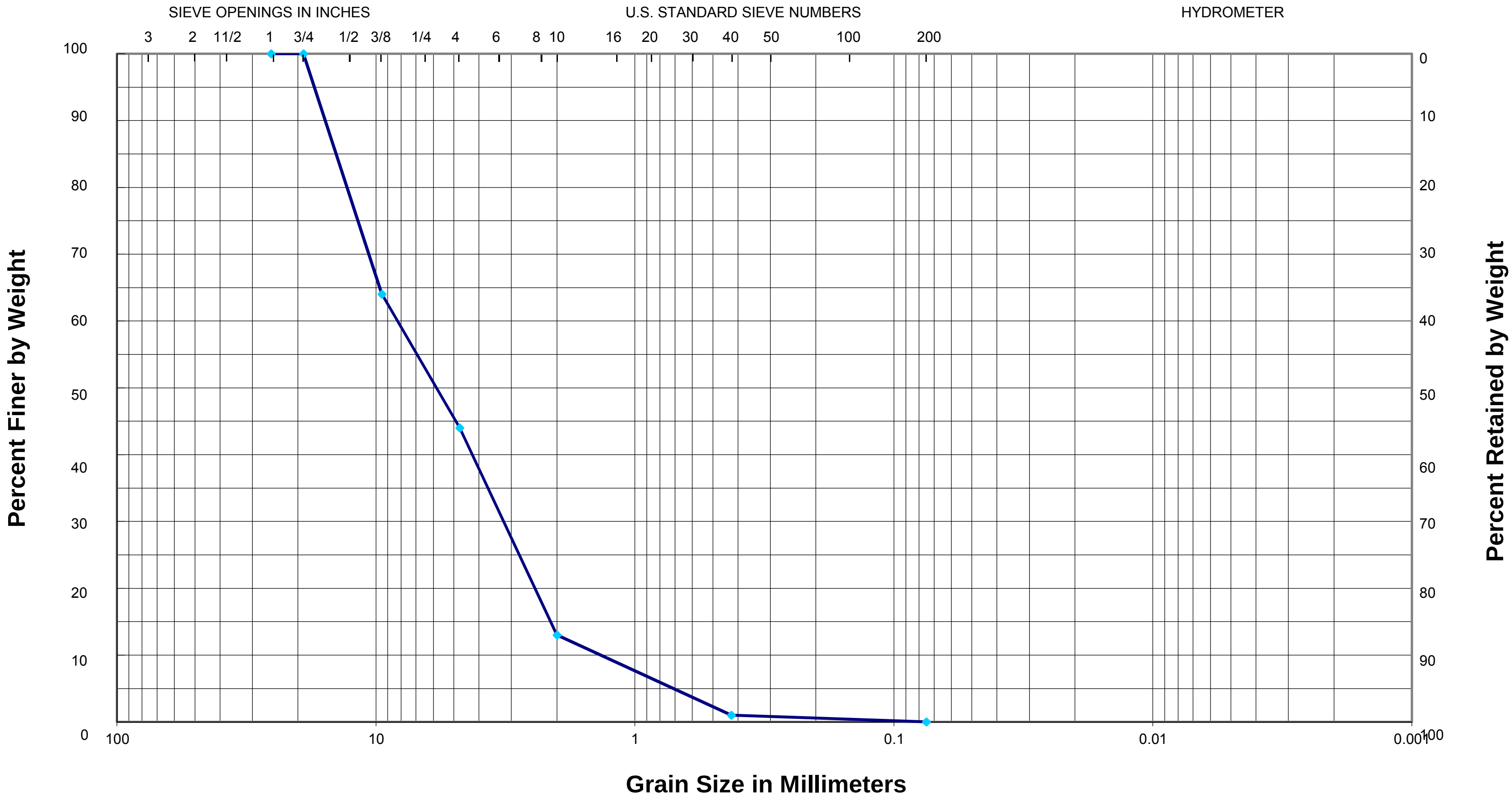
Sample: Boring 13B 36.5 - 37.1 ft
Description: Brown fine to coarse SAND, slightly silty
with a little fine gravel

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

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

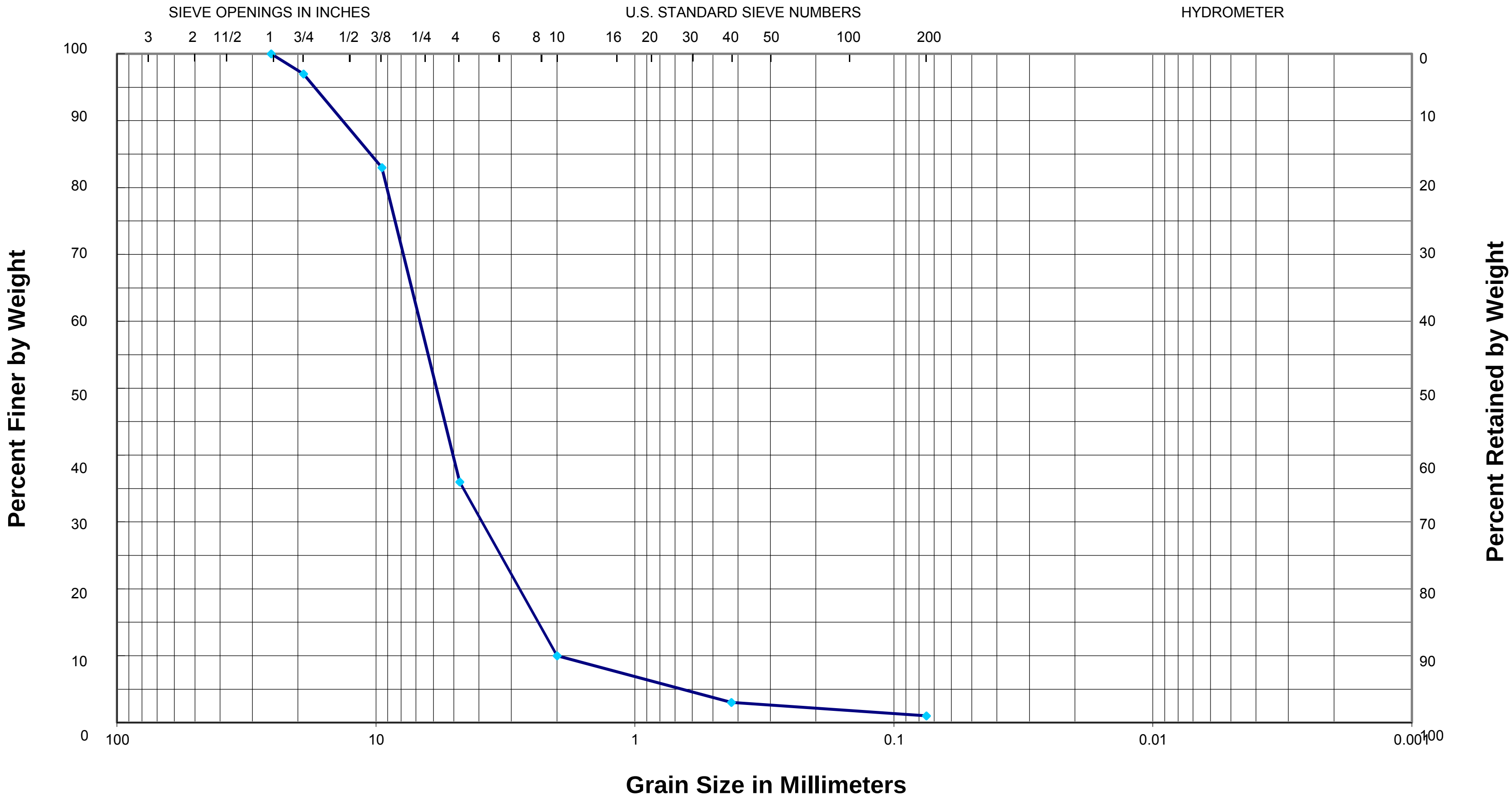
Sample: Boring 13B 46.5 - 47.5 ft
Description: Brown sandy fine GRAVEL

USCS = GP **AASHTO = A-1-a**

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

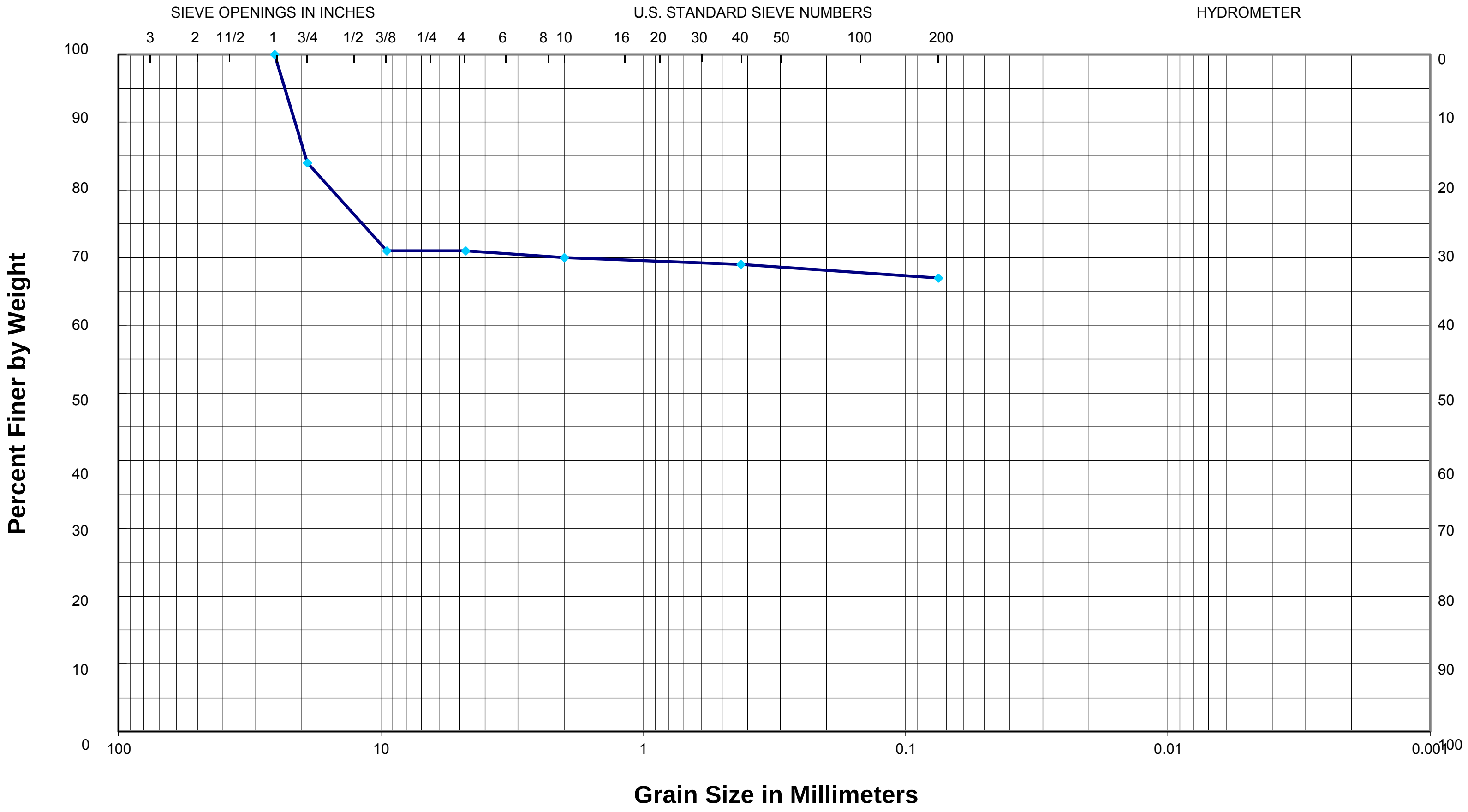
Sample: Boring 13B 66.5 - 67.5 ft
Description: Brownish gray sandy fine GRAVEL

USCS = GP **AASHTO = A-1-a**

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13B 126 - 126.5 ft; LL = 34, PL = 14, PI = 20
Description: Light gray silty CLAY
with a little fine to coarse gravel

USCS = CL AASHTO = A-6

13-017

GRAIN SIZE CURVE

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

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

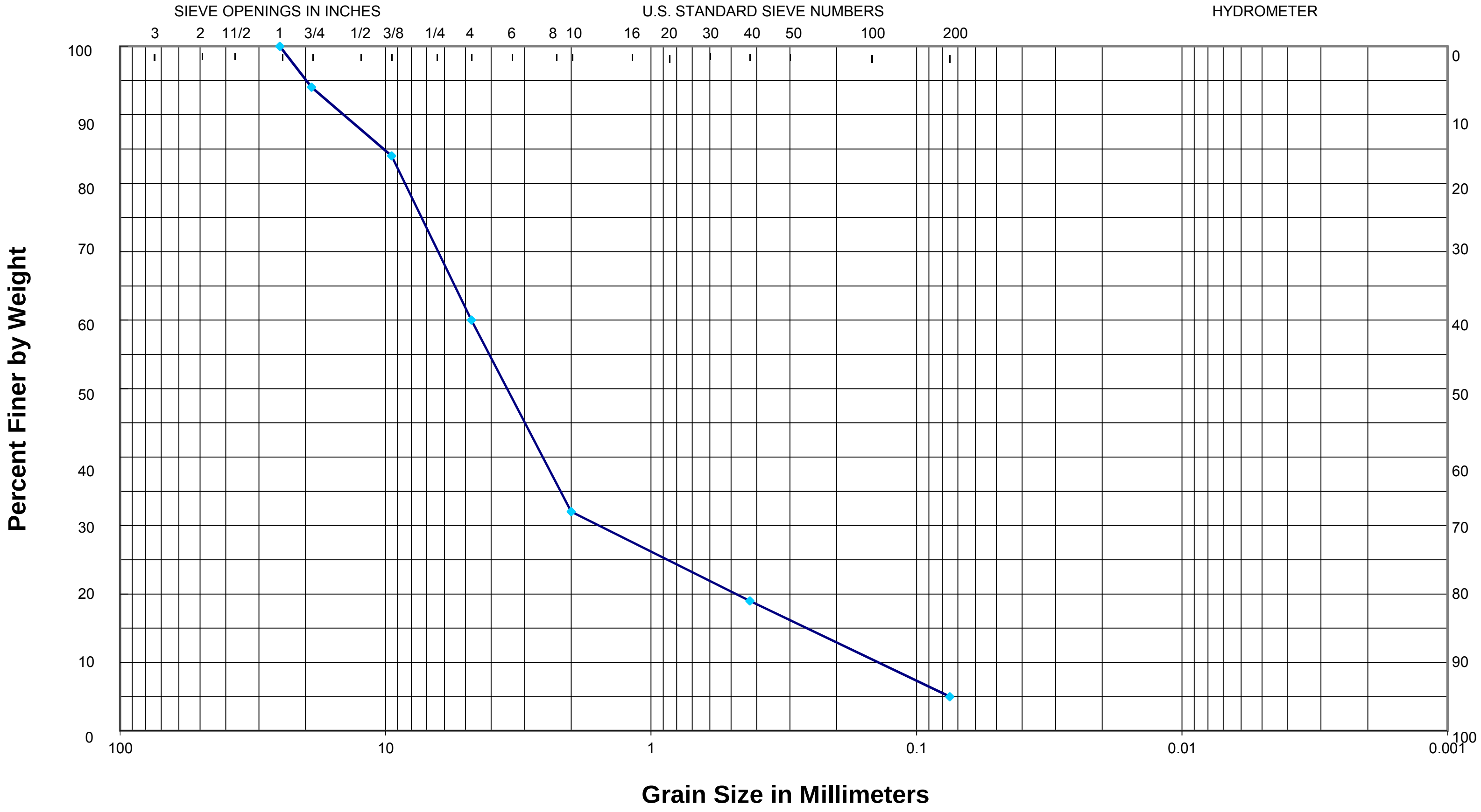
Sample: Boring 13B 136 - 137 ft; LL = 61, PL = 18, PI = 43
Description: Brownish gray CLAY,
with some fine to coarse gravel

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

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT and CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13C, 20-20.7 ft
Description: Brownish gray fine-medium SAND, slightly silty w/ trace fine gravel

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

13-017

GRAIN SIZE CURVE

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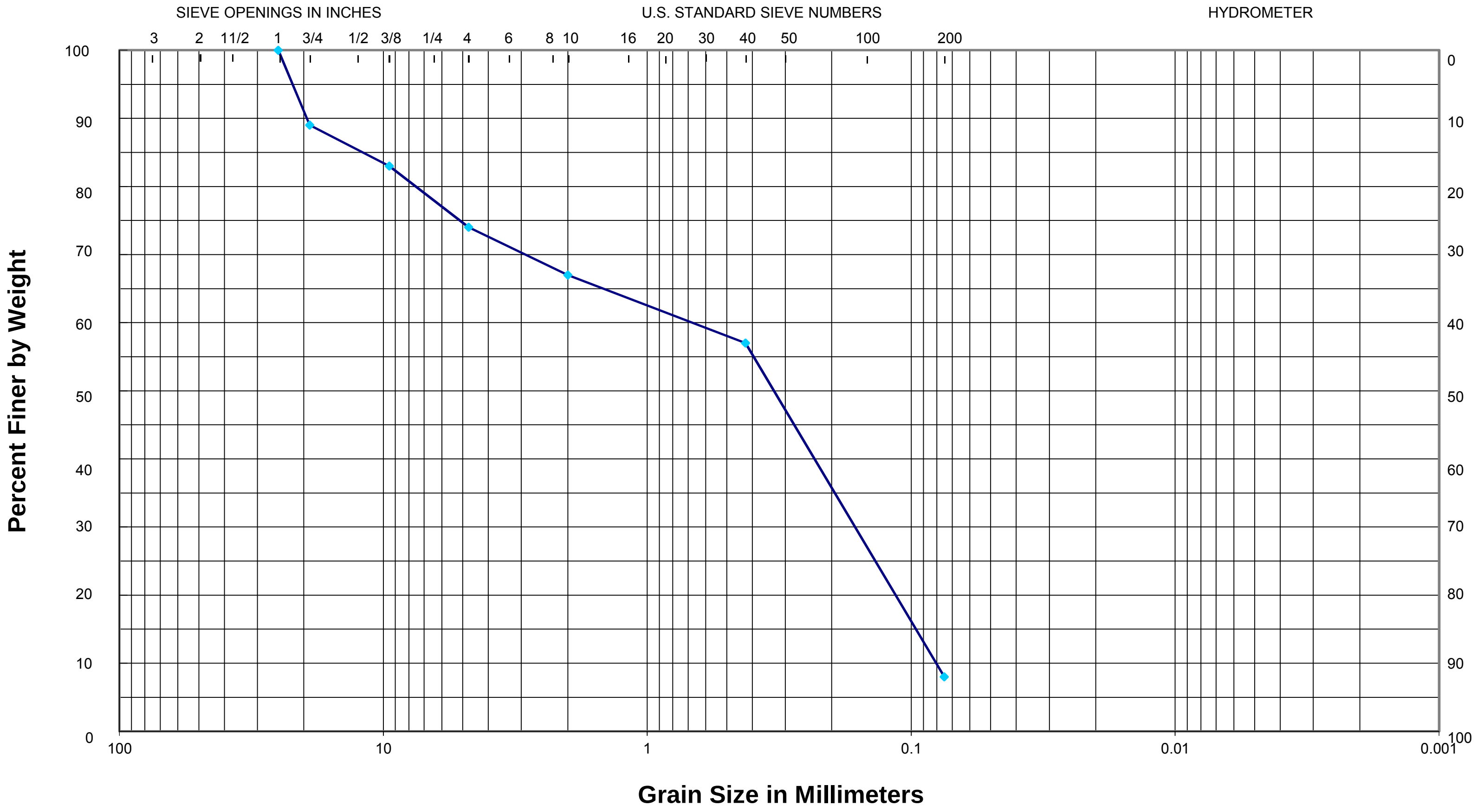
Barton & Wyatt, INC.
CONSULTING ENGINEERS



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

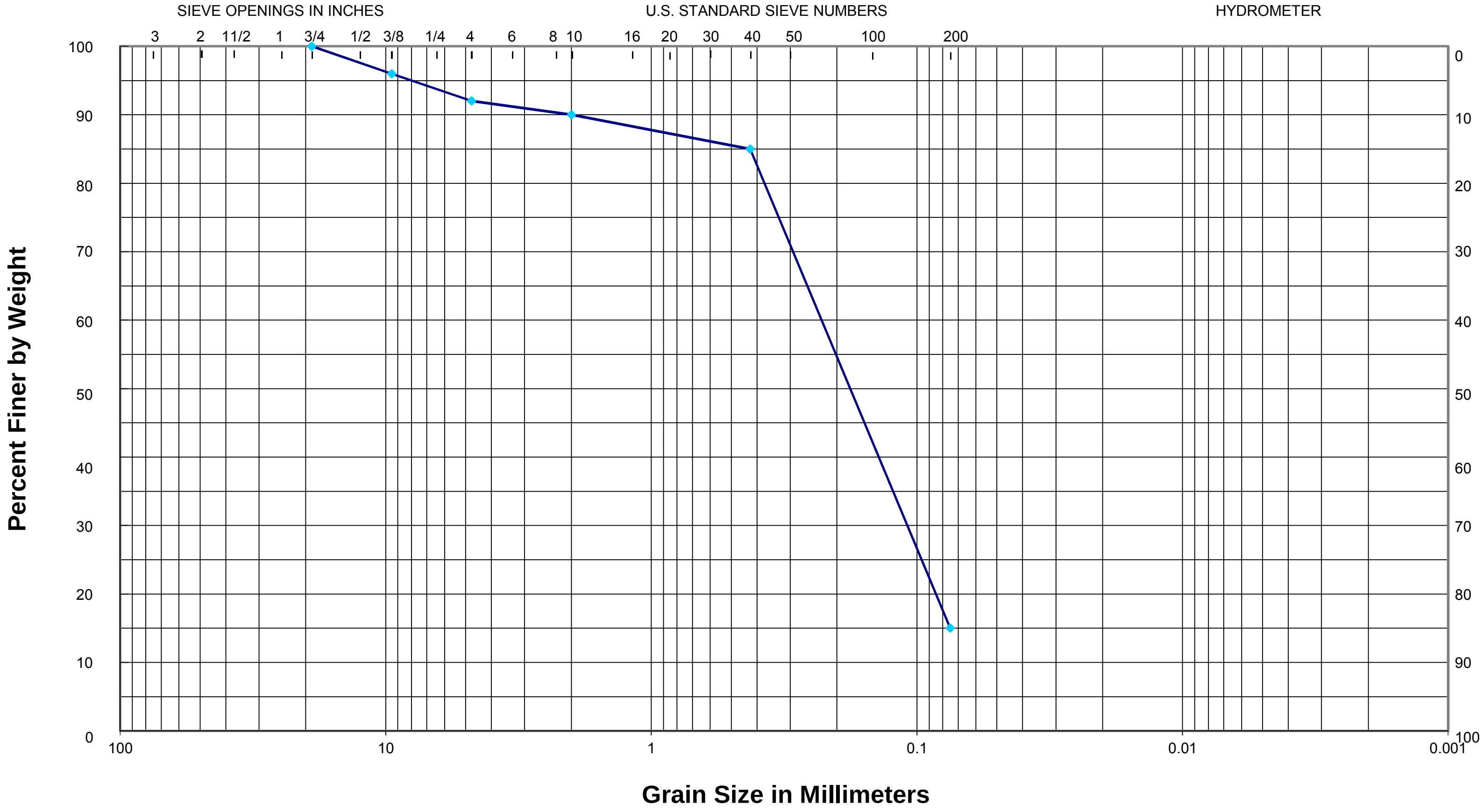
Sample: Boring 13C, 30.5-31 ft
Description: Brownish gray fine-medium SAND, slightly silty w/ trace fine gravel

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

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

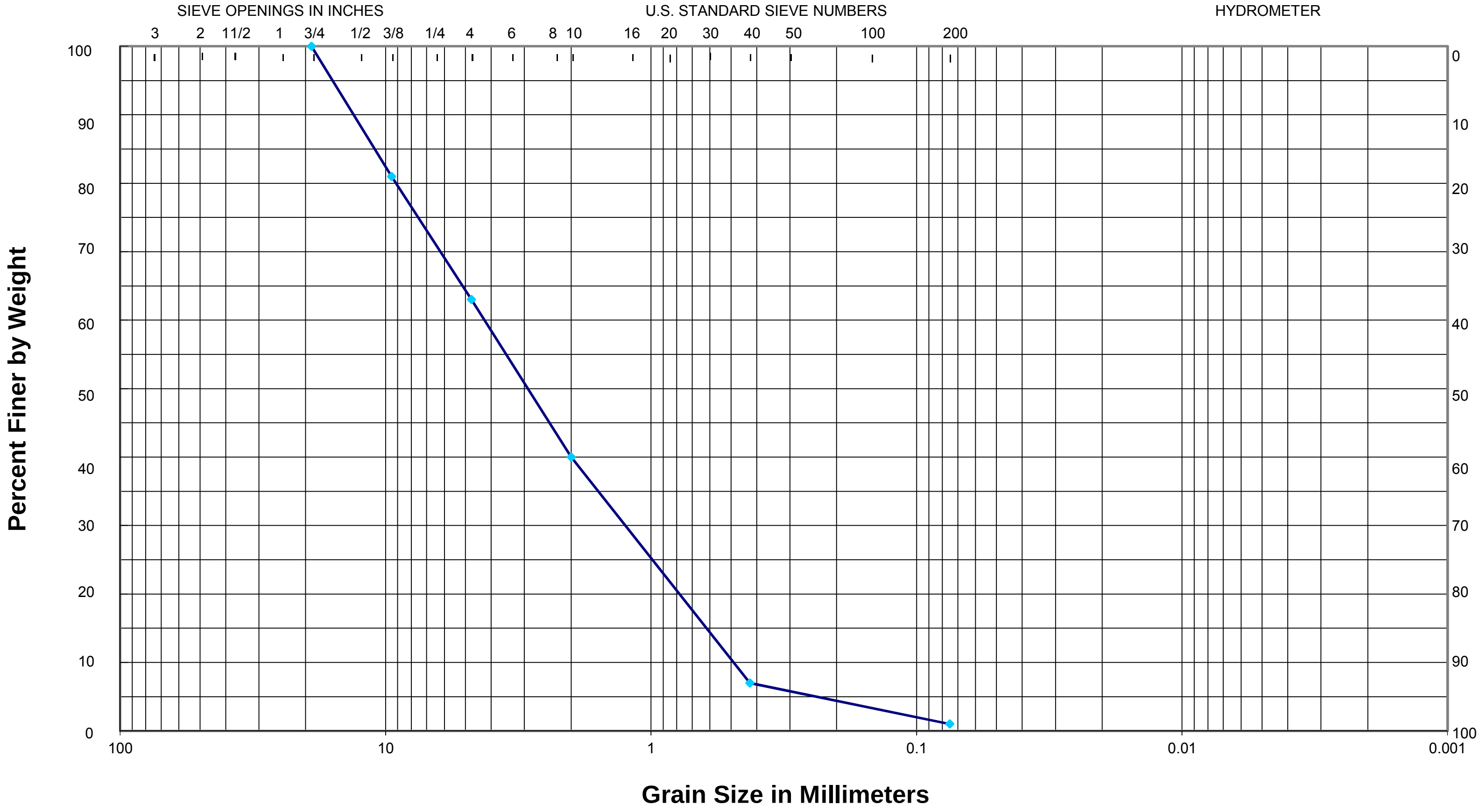
Sample: Boring 13C, 35.5-36 ft
Description: Brownish gray silty fine SAND w/ trace fine gravel)

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

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT and CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13C, 40.5-41 ft
Description: Brownish gray fine-coarse SAND and fine gravel

USCS = SP AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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

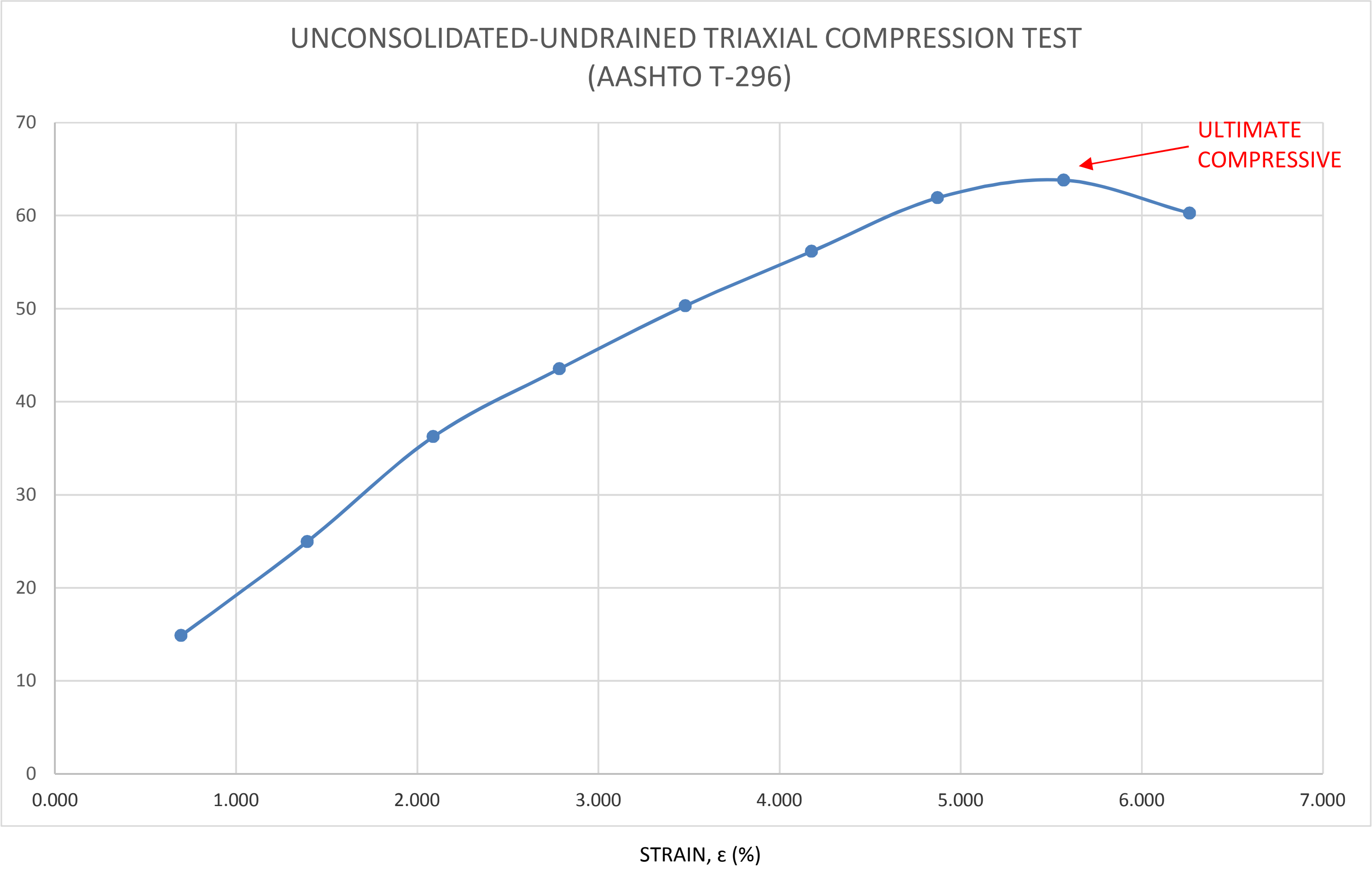
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13C, 95-95.5 ft
Description: Gray fine-coarse SAND w/ some fine gravel

USCS = SP AASHTO = A-1-b

RESULTS of UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

PROJECT: I-40 over White River - AHTD Job BB0610
LOCATION: Prairie County, Arkansas
JOB NUMBER: 13-017



Boring:	13C
Depth, ft:	125-125.5
Description:	Light gray CLAY
Ultimate Compressive Strength, psi:	64
Undrained shear strength, psi:	32
Undrained shear strength, psf:	4608

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

PLATE

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

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

January 30, 2015
Job No. 13-017

Jacobs Engineering Group Inc.
10816 Executive Center Drive
Suite 300
Little Rock, Arkansas 72211

Attn: Mr. Mark A. Asher, P.E.

GEOTECHNICAL INVESTIGATION
AHTD JOB BB0610: WHITE RIVER STR. & APPRS. (F)
I-40 over WHITE RIVER
PRAIRIE COUNTY, ARKANSAS

INTRODUCTION

Provided herein are the final results of the geotechnical investigation performed for the replacement bridge and associated approaches for the Interstate-40 (I-40) over the White River project planned in Prairie County, Arkansas. This project has been designated as White River Str. & Apprs. (F) project (AHTD Job No. BB0610). These services were authorized by the Indefinite Delivery Subconsulting Agreement No. WLXM2600-I11-0004 between Jacobs Engineering Group Inc. (Jacobs/the Engineer) and Grubbs, Hoskyn, Barton & Wyatt, Inc. (GHBW/the Geotechnical Engineer) and our subsequent discussions with Jacobs and the Arkansas State Highway and Transportation Department (AHTD/the Department). Interim results and recommendations have been provided throughout the course of this study.

The replacement bridge will be constructed on the upstream side (north) of the existing bridge structure (AHTD Bridge No. 3713). The replacement bridge will have five (5) continuous composite steel plate girder units and a total of 17 bents (Bents 1 through 17) and two (2) piers (Piers 1 and 2 between Bents 9 and 10) at the river navigation channel. The total length of the bridge will be about 2842 feet. Steel shells are expected to be utilized to support the foundation loads at each bent/pier.

The project also includes reconstruction of the approaches to the replacement bridge. Fill embankments are planned at the bridge abutments. We understand that the existing embankments will be incorporated into the new approach embankments. A 3-horizontal to 1-vertical (3H:1V) configuration is planned for the end slopes and the side slopes of the new embankment. The outer faces of the end slopes will be armored by dumped riprap for erosion protection. Maximum embankment height will be on the order of 35 ft at the west bridge end and 32 ft at the east bridge end.

The purposes of this study phase were to explore subsurface conditions at the replacement bridge location and to develop recommendations to guide design and construction of foundations and earthwork. These purposes have been achieved by a multi-phased study that included the following:

- Drilling soil sample borings to evaluate subsurface conditions and to obtain samples for laboratory testing;
- Conducting a site-specific ground motion response analysis to assist in evaluating seismic site class, determining design spectral acceleration coefficients, and developing conclusions regarding seismic performance zone / seismic design category;
- Performing laboratory tests to establish pertinent engineering properties of the foundation and subgrade strata; and
- Analyzing field and laboratory data and incorporating the results of site-specific analysis to develop recommendations for seismic site class, liquefaction potential, foundation design, embankment configurations, and construction considerations.

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

SUBSURFACE EXPLORATION

Subsurface conditions in the new bridge and approach embankment alignment were evaluated by drilling a total of 26 sample borings (Borings 1 through 25 and Boring 13A) to 70- to 150-ft depth. Boring 13 was advanced at the planned location to 78-ft depth and was abandoned due to borehole caving. Consequently, Boring 13A was performed at an adjacent offset location to 150-ft depth. The subsurface investigation program also included two (2) sample borings (Borings 29 and 30) performed in the existing bridge abutments to 30- to 35-ft depth to evaluate the existing embankment fill.

The site vicinity is shown on Plate 1. The approximate locations of the borings are shown on the overall Plan of Borings, Plate 2a. The locations of the structural borings (Borings 4 through 14, Boring 13A, and Borings 18 through 25) are also shown specifically on the Plans of Structure

Borings which were developed utilizing bridge layout drawings. These are attached as Plates 2b through 2f.

The subsurface conditions encountered in the borings, and the results of the field and laboratory tests, are shown on the boring logs, Plates 3 through 58. The approximate centerline station and offset of the boring locations are noted on the logs. Surveyed ground surface elevations, as provided by Jacobs, are also indicated on the logs. Where the surveyed ground surface elevation was not available, the approximate ground surface elevation has been inferred from the available topographic information. It must be noted that the inferred ground surface elevations shown are approximate and actual surface or channel bottom elevations may vary. A key to the terms and symbols used on the logs is presented as Plate 59. A summary of the subsurface exploration program is provided in Appendix A. A subsurface profile is provided in Appendix B.

Most borings were drilled with a truck-mounted Mobile B-53 rotary-drilling rig using a combination of dry-auger and rotary wash drilling methods. Borings 29 and 30 were drilled with a truck-mounted SIMCO 2400 rotary-drilling rig using dry-auger drilling procedures. Soil samples were typically obtained using a 2-in.-diameter split-barrel sampler driven into the strata by blows of a 140-lb safety hammer dropped 30 in. in accordance with Standard Penetration Test (SPT) procedures. The number of blows required to drive the standard split-barrel sampler the final 12 in. of an 18-in. total drive, or portion thereof, is defined as the Standard Penetration Number (N). Recorded N-values are shown on the boring logs in the "Blows Per Ft" column.

Selected undisturbed samples of cohesive soils were obtained using a 3-in.-diameter thin-walled tube (Shelby Tube) hydraulically advanced into the soil. Undrained shear strength of cohesive soils obtained using tubes were estimated in the field using a calibrated hand penetrometer. Estimated shear strength values are plotted on the log forms, in tons per sq ft, as circles enclosing an “x”.

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

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

LABORATORY TESTING

To evaluate pertinent physical and engineering characteristics of the foundation and subgrade soils encountered in the borings, laboratory tests consisting of natural water content determinations, classification tests, shear strength measurements, and consolidation tests were performed on selected representative samples. The laboratory testing program included the following.

- Soil water content (AASHTO T-265)
- Liquid limit, plastic limit, and plasticity index (AASHTO T-89 and T-90)
- Grain size analyses (AASHTO T-88)
- Specific gravity of soils (AASHTO T-100)
- Unconfined compressive strength of soils (AASHTO T-208)
- Consolidated-drained direct shear tests (AASHTO T-236)
- Unconfined undrained triaxial compressive strength of soils (AASHTO T-296)
- One-dimensional consolidation tests (AASHTO T-216)

Water Content Determinations

A total of 226 natural water content determinations were performed to complete soil water content profiles for each boring. Water content results are plotted on the log forms in accordance with the scale and symbols shown in the legend located in the upper-right corner of the logs.

Classification Tests

To verify field classifications and to evaluate soil plasticity, 56 Atterberg (liquid and plastic) limit determinations and 145 sieve analyses, including eleven (11) hydrometer tests were performed on selected representative soil samples. The Atterberg limits of the soil samples are plotted on the logs as plus signs connected with a dashed line. The percentage by weight of soil passing the No. 200 sieve is noted in the "- No. 200%" column on the far right side of the log forms. In addition, specific gravity was measured for use in each hydrometer analysis of particle size distribution. A summary of laboratory test results and classification by the Unified Soil Classification System and AASHTO classification is presented in Appendix C. Grain-size distribution curves are also included in Appendix C.

Soil Strength Measurements

Soil shear strength was estimated in the field using hand penetrometer and/or SPT results. Laboratory soil strength testing included six (6) consolidated-drained direct shear tests performed on representative undisturbed samples of cohesive soils and a remolded, cohesive soil sample of silty fine sand. The results of the direct shear tests are provided graphically in Appendix C.

In addition, twelve (12) unconsolidated-undrained triaxial compression tests and one (1) unconfined compression test were performed on undisturbed samples to measure undrained shear strength. Undrained shear strength (cohesion) determined from the results of the compression tests are plotted on the logs at the appropriate depth, in tons per sq ft, as an open triangle or an open circle for the triaxial compressive strength and unconfined compressive strength, respectively. Unit dry weight and natural water content were also determined as a part of each shear strength test. The unit dry weight, in lb per cubic ft is noted in the "UNIT DRY WT" column on the log forms.

Consolidation Tests

To evaluate consolidation characteristics of the foundation soils for use in determining the settlement potential, three (3) one-dimensional standard consolidation tests were performed on three (3) undisturbed silty clay samples. In these tests, a representative sample was placed in a consolidometer and loaded incrementally to an appropriate pressure. The sample was subsequently unloaded in increments and the rebound measured. The results of the consolidation tests are presented in Appendix C. Dry unit weight was also determined as part of these tests and is noted in the "UNIT DRY WT" column on the boring logs.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The replacement bridge is planned on the north side of the existing I-40 bridge over the White River. The existing I-40 bridge (Bridge No. 03713) over the White River is located in Prairie County, Arkansas (approximate GPS coordinates: 34.839616, -91.457113), about 60 miles east of Little Rock, Arkansas. We understand that the existing bridge structure will be demolished. However, the approach embankments will be incorporated into the new bridge alignment.

The White River at the plan replacement bridge location flows from the north to the south. The river has a wide flood plain with a channel width on the order of 500 ft at the normal pool elevation (El 164). Average water depth in the water channel was estimated on the order of 30 ft at the time of field study (March 2014). The reported historical high water is at El 192.9, about 29 ft higher than the normal pool elevation. The project alignment is in a flat, low-lying flood plain with poor surface drainage. This area is generally a fallow ground with a combination of tall grass, bushes, and small to medium trees.

Site Geology

The site is in the flood plain of the White River. Recent (Quaternary) Alluvium in the area is typically underlain by Quaternary Terrace Deposits. The alluvial deposits are comprised of variable sand, silt, gravel and clay units, and mixtures of any or all of these clastic materials. The granular units are often water bearing. The Terrace deposits are comprised of a complex sequence of unconsolidated gravel, sand, silt and clay. Individual Terrace deposits are often lenticular and discontinuous. The Alluvium is comprised of recent stream-deposited alluvial sediments which include gravel, sand, silt, clay and mixtures of all components. The thickness of the Terrace and Alluvial deposits is variable. Depth of bedrock in this area is estimated to exceed 1300 feet.

Seismic Conditions

Seismic Site Class. A site-specific ground motion response analysis has been performed for this project by the University of Texas at Austin[1]. The results of the site-specific analysis are provided in Appendix D. The site-specific analysis included measurement of shear wave velocity (V_s) on both sides of the White River utilizing a combination of active-source and ambient-wavefield surface wave methods. The active-source method (i.e., Multiple-Channel Analysis of Surface Wave or MASW method) was used to generate a more detailed shear wave velocity profile of the near-surface soils. The ambient-wavefield method (i.e., microtremor array measurements or MAM method) was used to measure shear wave velocity of the deeper subsurface soils. The shear wave velocity profiling on both sides of the White River extended to at least 1300-ft depth where a measured shear wave velocity in excess of 2500 ft per sec (i.e., Seismic Site Class B rock) was determined.

---

[1] Site-Specific Ground Motion Response Analysis Results for the I-40 Replacement Bridge over the White River, Final Report; Brady R. Cox et al.; The University of Texas; May 27, 2014.

Based on the results of the site-specific analysis, an average V_s value of 560 ft/sec was calculated for the upper 100 ft of the subsurface conditions on the west side of the White River. This V_s value of 560 ft per sec corresponds to a Site Seismic Class E (soft soil profile) with respect to the criteria of the 2012 AASHTO LRFD Bridge Design Specifications[2] and those of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design[3].

On the east side of the river, an average V_s value of 680 ft per sec was measured in the upper 100 ft, indicating a Site Seismic Class D (stiff soil profile). It should be noted that these average shear wave velocity values were determined for the top 100 ft below the existing ground surface (December 2013).

The combined results of soil shear wave velocity (V_s) measurements for the project alignment on both the west and east sides of the river channel indicate an overall average V_s value of 620 ft per second. These data indicate a Site Seismic Class D (stiff soil profile).

Furthermore, it must be noted the replacement bridge over the White River is relatively long (approximately 2840 ft) with multiple bents/spans. For this type of bridge, AASHTO LRFD seismic design guides suggest that the average shear wave velocity should be determined for the top 100 ft of subsurface soils below the "depth to motion (Z_{DTM})". For deep foundations comprised of driven piles with a pile cap/footing, AASHTO guidelines suggest that the depth to motion should be defined at the pile cap bottom. We understand that the pile cap bottom for the replacement bridge is typically planned at a depth of 6 to 7 ft or deeper below the existing ground surface. For the bent/piers (Bent 9 and Piers 1 and 2) in or near the river channel, pile cap bottom is planned at deeper depth up to 80 ft deep below the existing grade. Based on the shear wave velocity profile data developed in the site-specific seismic study and a conservatively assumed Z_{DTM} of 6 ft, an average V_s value of 670 ft per sec has been determined. This shear wave velocity value also corresponds to a Seismic Site Class D.

Based on the results of the borings performed for the geotechnical investigation on this project, an average Standard Penetration Test (SPT) N-value of 32 blows per ft on the west side of the river channel and 28 blows per ft was calculated for the top 100 ft on the east side of the river channel. These SPT N-values correspond to a Seismic Site Class D (stiff soil profile) in accordance with the criteria of AASHTO LRFD seismic bridge design guides.

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[2] AASHTO LRFD Bridge Design Specifications, AASHTO, 2012.

[3] AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition; AASHTO; 2011.

In light of the relatively uniform subsurface conditions revealed by the borings and the average results of the shear wave velocity measurements, it is our recommendation that a **Seismic Site Class D (stiff soil profile)** be utilized for design in accordance with the criteria of AASHTO LRFD seismic bridge design guides.

Seismic Performance Zone / Seismic Design Category. Based on the bridge location and utilizing the code-based procedure of the AASHTO LRFD seismic bridge design guides, the mapped 1.0-sec period spectral acceleration coefficient (S_1) for a Seismic Site Class B is 0.119. This mapped S_1 value is based on a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years). The site coefficient (F_v) for S_1 adjusted for Seismic Site Class D is 2.32. Accordingly, the calculated design 1.0-sec period spectral acceleration coefficient (S_{D1}) is 0.28 for the replacement bridge site in accordance with the code-based procedure.

The site-specific ground motion response analysis has determined an S_{D1} value of 0.302 on the west side of the bridge alignment and an S_{D1} value of 0.275 on the east side. Consequently, a design 1.0-sec period spectral acceleration coefficient **(S_{D1}) of 0.29**, i.e., average value of 0.302 and 0.275, is considered appropriate for the bridge alignment. This is the larger of two-thirds (2/3) of the value determined based on AASHTO LRFD code-based procedure (S_{D1} of 0.19) or the value determined utilizing the site-specific procedure (average S_{D1} of 0.29).

For an S_{D1} value of 0.29, Table 3.10.6-1 of the 2012 AASHTO LRFD Bridge Design Specifications indicates that a **Seismic Performance Zone 2** is fitting for the bridge site. Per Table 3.5-1 of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design, a **Seismic Design Category B** is considered appropriate for the replacement bridge site.

Design Peak Ground Acceleration. The code-based procedure of the AASHTO LRFD seismic bridge design guides indicates 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 0.196. For a Seismic Site Class D, the Site Coefficient for the PGA, F_{PGA} is determined to be 1.41. Consequently, a design PGA (A_s) value of 0.28 is considered appropriate in accordance with code-based procedure.

The site-specific analysis has determined an average A_s value of 0.19. Consequently, a design PGA **(A_s) value of 0.19**, which is the larger of two-thirds (2/3) of the value determined based on AASHTO LRFD code-based procedure (A_s value of 0.19) and the value determined utilizing the site-specific procedure (average A_s value of 0.19), is considered to be appropriate.

Design Earthquake Moment Magnitude. Based on the United States Geology Survey (USGS) 2002 interactive deaggregations data[4], an earthquake moment magnitude (M_w) value of 7.7 is determined to be fitting for the replacement bridge site. The M_w value of 7.7 is also recommended in the final report of the site-specific analysis. Consequently, an earthquake moment magnitude of 7.7 is considered to be suitable for seismic analysis at the project site.

Results of Liquefaction Analyses. Liquefaction analyses have been performed to evaluate the liquefaction potential of the foundation soils. These analyses were performed utilizing a Microsoft Excel© spreadsheet which is developed based on the methodology and procedures proposed by Idriss and Boulanger[5] in 2008. The spreadsheet was provided by the Department. An earthquake Moment Magnitude (M_w) value of 7.7 and a design PGA (A_S) value of 0.19 were utilized in the liquefaction analyses. Three (3) generalized subsurface models have been utilized for these analyses:

- A generalized subsurface model based on the results of the borings drilled on the west land side of the river (Borings 1 through 12);
- A generalized subsurface model based on the results of the borings performed in the river channel (Borings 13 and 13A); and
- A generalized subsurface model based on the results of the borings advanced on the east side of the river (Borings 14 through 25).

The results of liquefaction analyses are presented in Appendix E as plots of calculated factors of safety against liquefaction versus depth. The calculated factor of safety against liquefaction was determined as the ratio of cyclic shear stress required to cause liquefaction (soil strength) to cyclic shear stress induced by an earthquake (earthquake loading). The liquefaction analysis results indicate that for a design earthquake with a 7 percent chance of exceedance in 75 years (or mean return period of approximately 1000 years), the liquefaction potential is generally low for the I-40 over White River replacement bridge site.

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[4] http://geohazards.usgs.gov/deaggint/2002/

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

Subsurface Conditions

Based on the results of the borings, the subsurface conditions are relatively uniform and consistent. The stratigraphy may be summarized into six (6) general strata as follows:

Stratum I: Existing bridge end embankments are on-site fill (see Borings 29 and 30). The existing embankments are comprised primarily of firm to very stiff tan and gray to reddish brown silty clay and clay. The embankment fill also includes localized layers of firm to stiff reddish brown clayey silt and medium dense tan silty fine sand. The predominant clay and silty clay embankment fill has moderate to high plasticity. The subordinate clayey silt and silty fine sand are of low-plasticity or are non-plastic. The embankment fill is relatively compact with overall moderate shear strength. The fill encountered in the borings extends to about 24- to 26-ft depth (approximately El 181 to El 174). Fill content, depth, and compaction will vary with location.

Stratum II: The surface and near-surface soils in the new bridge alignment are generally very soft to stiff brown and gray silty clay with subordinate units of clay and clayey silt. There are also minor, localized and discontinuous units of silty fine sand and clayey fine sand. The predominant cohesive surface and near-surface soils typically extend to about 8- to 13-ft depth. Locally, this stratum is deeper and extends to about 35- to 42-ft depth (see Borings 18, 19, and 21). The natural silty clay and clay also have moderate to high plasticity. These moderate- to high-plasticity soils classify as A-6 to A-7-6 by the AASHTO soil classification system and exhibit poor to very poor pavement subgrade support characteristics. The low-plasticity clayey silt is generally classified as A-4 by the AASHTO system. However, this soil is highly moisture-sensitive and potentially unstable. SPT N-values in this stratum range from 2 to 44 blows per ft (average 10 blows per ft), indicating variable and overall low to moderate shear strength and moderate to low compressibility.

Stratum III: Loose to very dense tan and reddish tan to gray and brown silty fine sand is below the predominantly cohesive soils of Stratum II. Thickness of the non-plastic silty fine sand stratum encountered in the borings varies widely, ranging from absence to about 39 ft and averaging about 8 feet. The Stratum III silty fine sand exhibits variable and typically increasing low to high relative density with depth. SPT N-values range from 5 blows per ft to in excess of 50 blows per ft and average 25 blows per feet. The silty fine sand extends to variable depths of 7 to 66 feet below existing grades.

Stratum IVa: The silty fine sand is underlain below about 7- to 66-ft depth by medium dense to dense tan, gray, and brown fine sand and fine to medium sand. The fine sand and fine to medium sand exhibit moderate to high relative density and low compressibility with field SPT-N values averaging 38 blows per foot. Outside the river channel, this stratum extends to about 22- to 66-ft depth (approximately El 158 to El 111) and an average depth of 40 ft (average El 136). In the river channel, the medium dense to dense fine sand and fine to medium sand extend to approximately El 107.

Stratum IVb: The Stratum IVa fine to medium sand grades to dense to very dense gray and brown fine to coarse sand with variable amounts fine to coarse gravel (Stratum IVb). The Stratum IVb fine to medium sand exhibits high relative density and low compressibility. SPT-N values in the Stratum IVb sand units are typically greater than 50 blows per foot.

Stratum V: The basal stratum encountered within the exploration depths of the borings is dense to very dense gray gravelly fine to coarse sand and sandy fine to coarse gravel. The basal gravelly sand/sandy gravel exhibits high relative density with SPT-N values greater than 50 blows per foot.

Generalized Subsurface Profile. To aid in visualizing the subsurface conditions in the replacement bridge alignment, A Generalized Subsurface Profile is developed utilizing the results of the borings drilled for the bridge foundations (Borings 4 through 12, 13A, and 14 through 25). As noted, this profile is included in Appendix B. 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 subsurface conditions, variations from the stratigraphy illustrated by the profiles 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 encountered in the borings at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014. Groundwater levels will vary with seasonal precipitation, surface infiltration, and stream levels of the White River. Seasonal seeps and springs are likely to be present.

Significant Conditions

The significant site and subsurface conditions considered pertinent to design and construction of the replacement bridge over the White River project are summarized below:

- The existing bridge structure to be removed with existing approach embankments to be incorporated into the new embankment.
- The relatively wide, well-defined White River channel with a channel width of about 500 ft and an average water depth of about 30 ft in February to March 2014.
- The generally low-lying and flat terrain of the flood plain at the White River banks.
- The generally low liquefaction potential.

- The primarily cohesive existing embankment fill (Stratum I) comprised of clay, silty clay, and clayey silt and exhibiting overall fair compaction and moderate shear strength.
- The cohesive surface and near-surface soils in the new bridge alignment, comprised of very soft to stiff silty clay, clay, and clayey silt (Stratum II), extending to about 8- to 13-ft depth or deeper, and exhibiting overall low shear strength.
- The predominance of granular soils (Strata III, IVa, IVb, and V) present below the Stratum II cohesive soils and extending in excess of the maximum 150 ft exploration depth of the borings.
- The generally increasing relative density and decreasing compressibility of the granular soils (Strata III, IVa, IVb, and V) with depth.
- The dense to very dense fine to medium sand (Stratum IVb) underlain by very dense gravelly fine to coarse sand/sandy fine to coarse gravel (Stratum V), present below about El 158 to El 107 (average El 136) and exhibiting high to very high relative density.
- The groundwater encountered in the borings at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014.

The relationships of these factors to design and construction of the bridge and approaches have been considered in developing the conclusions and recommendations discussed in the following report sections.

ANALYSES and RECOMMENDATIONS

Foundation Design

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

In light of the predominance of granular subsurface soils revealed by the results of the borings, the anticipated heavy bridge foundation loads, and our understanding of the project, deep foundations comprised of steel shell piling are recommended to support the bridge foundation loads.

Piling

Axial Pile Capacities. We recommend that the foundation loads of the I-40 Bridge over the White River be supported on a deep foundation system comprised of steel shell piling. We understand that 18-in.-diameter and 24-in.-diameter steel shells are planned. Ultimate single pile capacity curves for the planned 18-in.-diameter and 24-in.-diameter steel shell piles are included in Appendix F for each bridge bent and pier. Plan pile cap bottom elevation is also shown on the ultimate pile capacities curves. Capacities for alternative pile types and sizes can be provided, if desired. We also understand that both the 18-in.- and 24-in.-diameter steel shell piles will be equipped with closure plates. All the steel shell piles will be filled with concrete after installation.

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 dated December 16, 2013. Normal pool was assumed in developing these axial pile capacity curves.

Based on the results of the borings, jetting or a combination of pre-boring and jetting is anticipated to be required at the interior bents and piers to facilitate penetration of the closed-ended steel shell piles into the dense to very dense sand units. Pre-boring is expected to be more effective in the surface and near-surface cohesive soils (Strata I and II). Jetting will be effective in the cohesionless soil strata (Strata III through V) but will be less effective in cohesive soils. The ultimate axial pile capacity curves of the interior bents and piers included in Appendix F have been developed based on the assumption that the steel shells will be installed using jetting. We recommend that the final 5 ft of pile penetration be achieved by driving. A closure plate is recommended for the steel shell piles as per AHTD criteria.

The ultimate axial capacities shown in Appendix F are based on single, isolated foundations. Piles spaced closer than six (6) pile diameters may develop lower individual capacity due to group effects. We understand that the 18-in.-diameter steel shell piles will be spaced 54 to 72 in. (4.5 to 6 ft) on center. This spacing is greater than two-and-half (2.5) diameters of the 18-in.-diameter piles. The 24-in.-diameter piles at the bents for the main bridge spans (Bents 9 and 10 and Piers 1 and 2) will be spaced 72 in. (6 ft) on center, also greater than 2.5 pile diameters. In light of the predominance of granular foundation soils and the pile spacing greater than 2.5 pile diameters, the reduction in axial compressive pile capacity due to pile grouping interaction is considered negligible. As per AASHTO LRFD Bridge Design Specifications, a group efficiency factor (η) of 1.0 is considered to be fitting for both pile sizes. Our analyses indicate that the factored uplift resistance of a pile group as a block exceeds the sum of the factored individual pile uplift resistance. Consequently, we recommend individual pile uplift resistance be utilized for foundation design.

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 AASHTO LRFD geotechnical design procedures, an effective resistance factor (φ_{stat}) of 0.45 is recommended for evaluation of factored compression capacity. For evaluation of factored uplift capacities, a resistance factor (φ_{up}) of 0.35 is recommended. These resistance factors are based on Strength Limit States. For Extreme Events Limit States such as earthquake loading, vessel collision, check flooding, etc. resistance factors of 1.0 and 0.8 are recommended for evaluating compression and uplift capacities, respectively. Post-construction settlement of piles installed 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 predominance of granular subsurface soils and the anticipated construction sequence as specified by AHTD Standard Specifications Section 805.02.

WEAP Driveability Analyses. To evaluate suitable driving equipment, driveability analyses have been performed for representative bents utilizing wave equation analysis of piles (WEAP) and the computer program GRLWEAP 2010[6]. In the driveability analyses, both the 18-in.- and 24-in.-diameter steel shell piles were assumed to be installed close-ended. Preliminary information regarding the steel shell pile foundations, as provided by the Engineer, is summarized in Table 1a below. The results of driveability analyses for representative bents are also indicated in this table.

Table 1a: Summary of Piling Foundations

Bent No.	Pile Diameter, in.	Plan Pile Cap El, ft	Plan Pile Tip El, ft	Plan Pile Length	Representative Bent Selected for Analysis
Bent 1 (W Abut)	18	200	137	63	Bent 1
Bent 2	18	163	122	41	Bent 3
Bent 3	18	163	122	41	

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[6] GRLWEAP 2010; Pile Dynamics, Inc.

Bent No.	**Pile Diameter, in.**	**Plan Pile Cap El, ft**	**Plan Pile Tip El, ft**	**Plan Pile Length**	**Representative Bent Selected for Analysis**
Bent 4	18	165	118	47	Bent 5
Bent 5	18	167	112	55	
Bent 6	18	168	118	50	
Bent 7	18	169	119	50	Bent 7
Bent 8	18	160	105	55	Bent 8
Bent 9	24	149	116	33	Bent 10
Bent 10	24	165	107	58	
Pier 1	24	111	56	55	Pier 2
Pier 2	24	111	47	64	
Bent 11	18	168	107	61	Bent 11
Bent 12	18	168	120	48	Bent 13
Bent 13	18	168	120	48	
Bent 15	18	176	132	44	
Bent 16	18	176	135	41	
Bent 14	18	170	120	50	Bent 14
Bent 17 (E Abut)	18	205	152	53	Bent 17

We understand that the steel shell piles will conform to ASTM A252, Grade 3 with a minimum yield strength (f_y) of 45 kips per sq inch. Wall thickness (t) of ½ in. is planned for the 18-in.-diameter steel shell piles while a wall thickness of ¾ in. is planned for the 24-in.-diameter steel shell piles. The steel shell will be filled with Class S concrete (AHTD Standard Specifications Sub-Section 802.04) with a minimum compressive strength (f'_c) of 3500 lbs per sq inch. These pile dimensions and properties have been assumed in the driveability analyses of the piling foundations.

DELMAG diesel hammers was utilized for the driveability analyses of both pile sizes. Hammer and pile cushion information was based on manufacturer-recommended values. Both the 18-in.- and 24-in-diameter piles are assumed to be installed close-ended. In the analyses, the piles at the abutments (Bents 1 and 17) are assumed to be driven from the plan pile cap bottom elevations to the plan tip elevations. The piles at the interior bents and piers are assumed to be jetted to 5 ft above the plan tip elevations. The final 5 ft of penetration is assumed to be achieved by impact hammer. Graphical results of the drivability analyses are provided in Appendix G. The results of the drivability analyses are summarized in Table 1b below.

Table 1b: Results of Driveability Analyses

Bent No.	Pile Diameter, in.	Pile Penetration, ft	Hammer Energy, ft-kips	Max Blow Count, Blows/ft	Max Comp Stress, ksi	Comments
Bent 1	18	63	66.2	209	31.1	Drivable
Bent 3	18	41	51.2	231	31.6	Drivable
Bent 5	18	55	73.8	203	37.1	Drivable
Bent 7	18	50	59.7	183	35.6	Drivable
Bent 8	18	55	66.2	240	39.0	Drivable
Bent 10	24	58	96.5	177	30.9	Drivable
Pier 2	24	64	125	236	37.1	Drivable
Bent 11	18	61	75.4	222	38.6	Drivable
Bent 13	18	48	51.2	214	28.5	Drivable
Bent 14	18	50	59.7	155	32.5	Drivable
Bent 17	18	53	66.2	223	31.7	Drivable

With the recommended hammer energy, the required number of hammer blows indicated by the WEAP analyses is typically limited to 20 blows per in. (240 blows per ft) for the steel shell piles. The calculated compressive and tensile stresses in the piles determined from the WEAP analyses are also in the acceptable range, less than 90 percent of the yield strength of 45 ksi (40.5 ksi), as per AHTD Standard Specifications Section 805.07. A specific review and analysis of the pile-hammer system proposed by the Contractor should be performed by the Engineer or Department (AHTD) prior to hammer acceptance and beginning of driving.

Piling Construction. We recommend a minimum of four (4) test piles be installed at the bridge location prior to installation of production piles. One (1) test pile should be installed at a bent location at each side of the navigation channel and at least one (1) test pile should be performed at each pier location. Test piles should have a length at least 10 ft longer than that anticipated for production piles.

Piles should be installed in compliance with AHTD Standard Specifications Section 805. Jetting may be required for pile installation. If jetting is to be utilized, containment of jetted materials must be provided unless approved by environmental agencies or other authorities. Safe bearing capacity of test piles and production piles should be determined by AHTD Standard Specifications Section 805.09, Method B. Driving records should be available for review by the Engineer or Department during pile installation.

Pile points are recommended at the pile tips to facilitate pile penetration. Jetting is expected to be utilized to assist pile penetration at the interior bents /piers. However, the final 5 ft of pile penetration must be achieved by driving. The jetting pressure and flow rate through jet pipes will directly affect jetting effect. Too much flow and pressure may result in poor controllability and poor alignment of the pile being worked and/or misalignment and compromising of the adjacent piles. Too little water flow or pressure could make the jetting technique ineffective. The Contractor should have demonstrable experience in installing steel shell piles of similar sizes in subsurface conditions similar to those at this site. The Contractor should have appropriate equipment with sufficient jetting pressure / flow rate and adequate hammer energy to install piles to the plan tip elevation.

Lateral Load Analyses

To evaluate foundation soil stiffness and responses to lateral loads and to determine lateral resistance, lateral load analyses were performed on representative bents and pier using the results of the borings and reactions provided by the Engineer. These representative bents and pier are selected by the Engineer and summarized in Table 2a below. As noted, the steel shell piles will conform to ASTM A252, Grade 3 with a minimum yield strength (f_y) of 45 kips per sq inch. Wall thickness (t) of ½ in. is planned for the 18-in.-diameter steel shell piles while a wall thickness of ¾ in. is planned for the 24-in.-diameter steel shell piles. The steel shells will be filled with Class S concrete (AHTD Standard Specifications Section 805.03) with a minimum compressive strength at 28 days of 3500 psi (AHTD Standard Specifications Section 802.05).

Lateral load analyses of single piles were performed using the computer program LPILE[7]. Preliminary analyses were first performed to evaluate stiffness of the foundation soils using assumed pile head reactions. We understand that the calculated foundation soil stiffness values were utilized by the Engineer as input parameters to determine pile head reactions. The calculated pile head reactions, as provided by the Engineer, are also summarized in Table 2a. These reactions include calculated lateral loads (shear loads in transverse and longitudinal directions) and axial compression load at the plan cap bottom elevations of single piles.

A fixed-head boundary condition was used in the analyses. The effects of group interaction of the piles at each bent were modelled by applying a P-multiplier to p (P_m) to define

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[7] LPILE Plus, Version 5.0; Lymon C. Reese and Shin Tower Wang; Ensoft, 2004.

the foundation p-y relationship. The P_m values are determined based on the pile space-diameter ratio and in accordance with the AASHTO LRFD Bridge Design Specifications. The top 5 ft of pile penetration is neglected from lateral resistance evaluation. At Pier 1, the results of scour analysis performed by the Engineer indicate the potential localized scour at the design flood (100-year flood) is at El 76, about 35 ft below the plan pile footing bottom elevation (El 111). To model the lateral resistance of the foundation soils above the localized, temporary scour depth, a reduced shear strength parameter, i.e., an internal friction angle (ϕ') of 30° which is typical of loose sand, is utilized in the analysis.

Table 2a: Representative Bents Lateral Load Analyses

Bent No.	Pile Diameter, in.	Plan Pile Length, ft	Lateral Load Direction	Pile Spacing, ft	Lateral Load, kips	Axial Load, kips
1	18	63	Long	4.5	20.5	159
2	18	41	Trans	5	2.2	164
			Long	5	2.5	164
3	18	41	Trans	5	2.4	160
			Long	5	2.7	160
8	18	55	Trans	6	37.8	351
			Long	4.5	19.0	351
9	24	33	Trans	6	35.9	348
			Long	6	18.2	348
Pier 1	24	55	Trans	6	22.6	380
			Long	6	11.3	380
10	24	58	Trans	8	36.2	343
			Long	6	18.4	343
17	18	53	Long	4.5	20.5	159

Graphical plots of pile head deflection versus depth, unfactored bending moment versus depth, and shear force versus depth are provided in Appendix H for these representative bridge bents and pier. The results of the lateral load analyses are summarized below in Table 2b.

Table 2b: Lateral Load Analysis Results

Bent No.	Lateral Load Direction	Lateral Load, kips	Calculated Maximum Pile-Head Deflection, in.	Calculated Maximum Unfactored Bending Moment, in.-kips
1	Long	20.5	0.86	2650
2	Trans	2.2	0.04	230
	Long	2.5	0.05	250
	Resultant	3.4	0.06	340
3	Trans	2.4	0.04	230
	Long	2.7	0.04	260
	Resultant	3.6	0.06	350
8	Trans	37.8	0.48	4600
	Long	19.0	1.13	2200
	Resultant	42.3	1.31	5200
9	Trans	35.9	0.37	4400
	Long	18.2	0.15	2100
	Resultant	40.3	0.44	5100
Pier 1	Trans	22.6	0.38	3200
	Long	11.3	0.18	1600
	Resultant	25.2	0.43	3600
10	Trans	36.2	0.41	4700
	Long	18.4	0.16	2200
	Resultant	40.6	0.47	5300
18	Long	20.5	1.07	2850

Wingwall and Abutment Wall Lateral Earth Pressures

It is expected that wingwalls and abutment walls, if planned, will be backfilled with unclassified borrow or select material. Recommendations regarding lateral earth pressures for wingwalls and abutments are summarized below.

- Total unit weight (γ) for unclassified backfill: 130 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: 96 lbs per sq ft per ft depth.
- Angle of internal friction (ϕ) for SM-1 backfill: 32°
- Total unit weight (γ) for SM-1: 125 lbs per cu ft
- Equivalent fluid pressure for SM-1 backfill:
    - 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.
- Nominal/ultimate sliding resistance:
    - Interaction friction angle (δ) for concrete on stable bearing stratum: 19°.
    - Interaction friction factor ($\tan \delta$) for concrete on stable bearing stratum: 0.34.
    - A resistance factor (φ) for sliding resistance: 0.8

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

Approach Embankments

General. Preliminary stability analyses were initially performed to evaluate various alternative embankment configurations (i.e., various embankment height, slope, and geosynthetics reinforcement layout, etc.) at the west and east bridge ends. We understand that the existing embankments will be utilized to the extent possible and the south portion of the plan embankments will incorporate the existing embankment fill to some degree. Consequently, two (2) embankment sections had been evaluated for each bridge end in the preliminary analyses. These included: (1) an embankment section comprised completely of new embankment fill (new embankment section) and, (2) an embankment section incorporating the existing embankment fill (composite embankment section). Based on the results of these preliminary stability analyses, it had been concluded that the composite embankment section is generally more critical with lower factors of safety against slope instability for all the loading conditions analyzed and for both bridge end slopes. Consequently, the final analyses evaluate/verify only composite embankment sections.

As noted, the bridge layout dated December 16, 2013, as provided by the Engineer, has been utilized to develop Plans of Borings that are included in this final report as Plates 2b through 2f. In addition, some representative cross sections of the approach embankments were also provided by the Engineer. These cross sections are included in Appendix I. The bridge layout and embankment cross sections indicate that geogrid-reinforced embankments with maximum embankment height of up to about 35 ft and a 3-horizontal to 1-vertical (3H:1V) slope configuration have been adopted by the Engineer. It is understood the bridge layout and embankment cross sections were developed based on the results of preliminary stability analyses.

The purposes of the final stability analyses are to verify stability of the design embankment configuration with respect to shear strength of embankment fill and foundation soils. To model the lower strength boundary of unclassified embankment fill placed for new embankments, a cohesion value of 750 lbs per sq ft and an internal friction angle (φ) of 0° were assumed. Cohesive soils were assumed for the embankment fill. Cohesive soils are recommended for embankment fill due to improved response to seismic loading as compared to cohesionless soils. The *in-situ* soil properties have been modeled for use in stability analyses based on the results of the laboratory testing program and our experience with similar soils. For the purposes of stability analyses, a uniform surcharge of 250 lbs per sq ft has been included to accommodate vehicle traffic loads.

Stability analyses have been performed using the computer program SLOPE/W 2007[8] and a Morgenstern-Price analysis. The loading conditions evaluated for the maximum 35-ft embankments include the following.

- End of construction with total stresses.
- Long term with effective stresses and groundwater at the embankment toe.
- Long term with effective stresses and the embankment saturated to a groundwater level approximately equal to the design 500-yr flood at El 192.9.
- Seismic condition with effective stresses and groundwater at the embankment toe.

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

  - As per FHWA guidelines[9] utilized by AHTD and assuming an embankment reinforced with geogrid, the analyses for the seismic condition have utilized a horizontal acceleration coefficient (k_h) value of one-half of the peak ground acceleration value, i.e., 0.10.
  - For analyses of unreinforced embankments, a horizontal acceleration coefficient (k_h) value of 0.19, which is the peak ground acceleration value, was utilized.
- Rapid drawdown with effective stresses and a saturated embankment at the design 500-yr flood of El 192.9 and groundwater at the embankment toe elevation. Only the upstream slopes (i.e., end slopes and north side of the side slopes) were evaluated in stability analysis of rapid drawdown condition.

Geosynthetic (biaxial geogrid) internal reinforcement is incorporated into embankment model and stability analyses. For the purposes of the analyses, the internal reinforcement is assumed to be structural geogrids with a minimum allowable tensile strength of 4000 lbs/ft. The geogrid reinforcement has been assumed to be spaced at 3-ft vertical intervals starting at the ground level and continuing to within three (3) ft of the final grade. Laterally, the reinforcement has been assumed to extend from one side slope face to the other side slope face, consistent with the construction requirement. The longitudinal extent of the reinforcement was determined through an iterative process by performing stability analyses on unreinforced slopes of decreasing height. The results of detailed stability analyses are discussed in the following report sections.

Stability analyses have been performed to verify the suitability of the plan approach embankments. Results of the stability analyses performed on six (6) embankment sections are provided in this final report. These sections, as well as location and plan height of the slopes, are summarized in Table 3 below. Section view drawings, with material parameters shown on, have been developed for these sections to facilitate stability analysis modeling. These sections are included in respective appendices containing the results of stability analyses (see Table 3).

Table 3: Summary of Embankment Sections Utilized for Stability Analyses

Location	Slope	Reinforcement Condition	Height, ft	Appendix
West Abutment (Sta 1883+41.92)	3H:1V End Slope	Geogrid-Reinforced	35	J
	3H:1V Side Slopes	Geogrid-Reinforced	15 to 35	K
East Abutment (Sta 1911+84.08)	3H:1V End Slope	Geogrid-Reinforced	32	L

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[9] 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.

Location	**Slope**	**Reinforcement Condition**	**Height, ft**	**Appendix**
	3H:1V Side Slopes	Geogrid-Reinforced	10 to 32	M
Sta 1875+00	3H:1V Side Slopes	Unreinforced	17 to 25	N
Sta 1913+00	3H:1V Side Slopes	Unreinforced	10 to 29	O

Results of Stability Analyses. The results of stability analyses are summarized in Tables 4 through 9. Detailed graphical results for each case analyzed are provided in appropriate appendices summarized in Table 3 above.

Table 4: Stability Analysis Results – End Slope @ West Abutment (H = 35 ft)

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	Groundwater @ embankment toe	2.1
Long Term	Groundwater @ embankment toe	1.8
Long Term	Historical high water @ El 192.9	1.9
Seismic (kh = 0.5A_S = 0.10)	Groundwater @ embankment toe	1.3
Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

Table 5: Stability Analysis Results – Side Slopes @ West Abutment (H = 15 to 35 ft)

Design Loading Condition	**Embankment Side**	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	North	Groundwater @ embankment toe	2.2
	South		4.1
Long Term	North	Groundwater @ embankment toe	1.9
	South		2.0
	North	Historical high water @ El 192.9	1.9
	South		2.0
Seismic (k_h = 0.5A_S = 0.10)	North	Groundwater @ embankment toe	1.4
	South		1.5
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.5

Table 6: Stability Analysis Results – End Slope @ East Abutment (H = 32 ft)

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	Groundwater @ embankment toe	2.8
Long Term	Groundwater @ embankment toe	2.5
Long Term	Historical high water @ El 192.9	2.4
Seismic ($k_h = 0.5A_S = 0.10$)	Groundwater @ embankment toe	1.8
Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.3

Table 7: Stability Analysis Results – Side Slopes @ East Abutment (H = 10 to 32 ft)

Design Loading Condition	**Embankment Side**	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	North	Groundwater @ embankment toe	2.5
	South		4.5
Long Term	North	Groundwater @ embankment toe	2.3
	South		3.4
	North	Historical high water @ El 192.9	2.3
	South		3.4
Seismic ($k_h = 0.5A_S = 0.10$)	North	Groundwater @ embankment toe	1.7
	South		1.9
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	2.1

Table 8: Stability Analysis Results – Side Slopes @ Sta 1875+00 (H = 17 to 25 ft)

Design Loading Condition	**Embankment Side**	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	North	Groundwater @ embankment toe	2.9
	South		3.4
Long Term	North	Groundwater @ embankment toe	2.2
	South		2.4
	North	Historical high water @ El 192.9	3.0
	South		3.2
Seismic ($k_h = 1.0A_S = 0.19$)	North	Groundwater @ embankment toe	1.1
	South		1.2
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.6

Table 9: Stability Analysis Results – Side Slopes @ Sta 1913+00 (H = 10 to 29 ft)

Design Loading Condition	Embankment Side	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	North	Groundwater @ embankment toe	2.9
	South		3.4
Long Term	North	Groundwater @ embankment toe	1.8
	South		4.3
	North	Historical high water @ El 192.9	1.8
	South		3.9
Seismic ($k_h = 1.0A_S = 0.19$)	North	Groundwater @ embankment toe	1.1
	South		1.6
Rapid Drawdown	North	Drawdown from 192.9 to embankment toe	1.6

Conclusions and Recommendations. The results of slope stability analyses summarized in Tables 4 through 7 indicate acceptable factors of safety against sliding for the geogrid-reinforced end and side slopes with maximum height of 32 to 35 ft and a simple 3H:1V slope configuration for all the loading conditions analyzed and for both end and side slopes evaluated at the bridge ends. Consequently, the design slope configuration is considered to be adequate and suitable in conjunction with the assumed geogrid reinforcement properties and arrangement.

The embankment alignments which may be unreinforced was determined through an iterative process by performing stability analyses on unreinforced slopes of decreasing height. A trial-and-error procedure was utilized in the analyses to determine the maximum unreinforced embankment height until the calculated minimum factor of safety is just greater than 1.0. Based on the results of these analyses summarized in Tables 8 and 9, we recommend the embankment alignment between Sta 1875+00 and Sta 1883+42 (Begin Bridge) and the alignment between Sta 1911+84 (End Bridge) and Sta 1913+00 be reinforced with geogrid. Beyond these limits, geogrid reinforcement is not required. Recommended limits of geogrid-reinforced embankment alignments are shown on the site layout included in Appendix P.

Embankment Fill Considerations. We recommend that embankment fill consist of cohesive soils. Suitable borrow includes sandy clay, clayey sand, silty clay, and clay. We recommend the following properties for imported borrow for use as embankment fill.

- Maximum liquid limit of 50
- Minimum plasticity index (PI) of 10
- A minimum of 60 percent passing the No. 4 sieve (4.76mm)

- A minimum of 35 percent passing the No. 200 sieve (0.074mm)
- Non-dispersive

We recommend that the top 24 in. of embankment fill in slopes have a maximum liquid limit of 40 and a PI between 5 and 18. All fill and backfill must be free of organic materials. Maximum particle size in embankment fill should be limited to about 6 inches.

Where fill is placed against existing embankment slopes, short vertical cuts should be benched into the existing slope faces to facilitate application of geogrid reinforcement and bonding of horizontal fill lifts. Maximum bench height should be limited to 3 feet. A typical bench width of 9 ft is recommended. Detailed benching pattern during construction must be based on specific site and construction conditions. A nominal horizontal bench width of 8 to 12 ft and a maximum bench height of 4 ft are recommended.

Stability of East River Bank near Pier 2

The east river bank where Pier No. 2 is planned is relatively high (±51 feet) and steep. The upper half portion of the slope is steeper and has a slope configuration of approximately 1.1H:1V. Consequently, engineering analyses were performed to evaluate stability of the river bank at the east side of the White River. Like those analyses performed for the approach embankments, stability analyses were also performed on the existing east river bank using the computer program SLOPE/W 2007 and a Morgenstern-Price method. Likewise, four (4) loading conditions were evaluated. These loading conditions are: 1) long term condition with groundwater at El 164, 2) long term condition with historical high water at El 192.9, 3) seismic condition with groundwater at El 164, and 4) rapid drawdown from El 192.9 to El 159 (design low water).

The results of stability analyses performed on the east river bank are summarized in Table 10 below. Detailed graphical results are provided in Appendix Q.

Table 10: Stability Analysis Results – Existing East River Bank (H = 51 ft)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
Long Term	Groundwater @ El 164	1.3
Long Term	Historical high water @ El 192.9	1.5
Seismic (kh = 1.0A_S = 0.19)	Groundwater @ El 164	0.7
Rapid Drawdown	Drawdown from 192.9 to El 159	0.8

The results of the slope stability analyses summarized above indicate that the east river bank is generally stable for long term conditions, but with marginally acceptable factors of safety against sliding. For seismic and rapid drawdown conditions, the calculated factors of safety are less than 1.0, indicating a substantial risk of sliding with these loading conditions. Consequently, design will be required to maintain stability of the river bank and to protect the planned Pier 2 from lateral loads due to sliding.

Detailed design for the remediation of the east river bank is not presently available. However, we understand that the preliminary concept is to design Pier 2 as a retaining wall-like structure or to include a pressure relief wall. The pier or retaining wall will extend from the top of pile footing (approximately El 116) up to some elevation (e.g., El 186) to retain ground and resist the lateral earth pressure. We also understand that individual columns will be utilized above the top of the retaining wall elevation to support the bridge superstructure.

The thick retaining wall structure is expected to be fixed at the top by the superstructure with the columns and will be designed to allow minimal deflection. Consequently, at-rest earth pressure will be mobilized on the retained side. On the retaining side, passive earth pressure will be developed to resist wall deflection. Recommendations regarding lateral earth pressures for the Pier 2 retaining wall are summarized below.

- Retained side (behind retaining wall) – at-rest earth pressure
    - Design (equivalent) surface elevation of retained soils: El 175
    - Effective unit weight (γ'): 56 lbs per cu ft
    - Angle of internal friction (ϕ): 24°
    - Coefficient of static at-rest earth pressure (K_0): 0.60
    - Coefficient of total (static and seismic) at-rest earth pressure (K_{0e}): 1.02
- Retaining side (in front of retaining wall) – passive earth pressure
    - Design (equivalent) surface elevation of retaining soils: El 145
    - Effective unit weight (γ'): 66 lbs per cu ft
    - Angle of internal friction (ϕ): 36°
    - Coefficient of static passive earth pressure (K_p): 3.85
    - Coefficient of total (static and seismic) at-rest earth pressure (K_{pe}): 3.32
- Nominal/ultimate sliding resistance:
    - Interaction friction angle (δ) for concrete (pile footing) on stable bearing stratum: 19°
    - Interaction friction factor ($\tan \delta$) for concrete (pile footing) on stable bearing stratum: 0.34
    - A resistance factor (φ) for sliding resistance: 0.8

Stability analyses were performed to verify global stability of the east river bank with the retaining structure. Two (2) loading conditions, i.e., seismic and rapid drawdown, were evaluated. The soil on the retaining side was conservatively assumed to slide or be eroded by scour to the dense silty fine sand stratum near El 145. In addition, the retaining structure was assumed to be rigid enough to drive the failure surface below El 110. The retaining structure design must include verification of internal stability of the retaining wall.

The results of stability analyses performed on the east river bank with a retaining structure are summarized in Table 11. Detailed graphical results are also provided in Appendix Q. The results shown in Table 11 indicate acceptable factors of safety against sliding with respect to seismic and rapid drawdown conditions. The factors of safety for long term conditions are expected to be greater and will be acceptable.

Table 11: Stability Analysis Results –East River Bank Remedied by Retaining Structure

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
Seismic ($k_h = 1.0A_S = 0.19$)	Groundwater @ El 164	1.9
Rapid Drawdown	Drawdown from 192.9 to El 159	5.9

Pavement Design Subgrade Support Parameters

It is understood that the new approach roads will be constructed mostly on embankments. In light of the roadway on the embankment, surface drainage in the new roadway alignment is expected to be good.

Based on the results of the borings performed in the existing embankment (i.e., Borings 29 and 30), the subgrade soils are expected to be predominantly silty clay and clay (AASHTO A-6 and A-7-6). Locally available borrow for use as unclassified embankment fill is expected to be comprised of similar soils.

We recommend that A-7-5 and A-7-6 silty clay and clay be excluded within 18 in. of the plan subgrade elevation. The top 18 in. of subgrade soils should have a maximum plasticity index (PI) of 18. The as-built pavement subgrade should be approved by the Engineer based on site observations. Areas of unsuitable subgrade should be improved by undercut or treatment with hydrated lime or Portland cement.

Based on correlation with the AASHTO classification of the anticipated subgrade soils, subgrade support characteristics are expected to be fair. For the purposes of developing an effective resilient modulus (M_R) for use in pavement design, correlation has been based on the fair subgrade support expected for a subgrade comprised of a minimum 18 in. of low-plasticity soils. We recommend an effective resilient modulus (M_R) value of 3200 lb per sq in. for use in approach road pavement design. An effective subgrade modulus (k) value of 165 lbs per cu in. is also recommended.

Scour Analysis Parameters

Scour protection will be warranted to limit the potential for loss of ground and reduction in foundation capacities. We understand that specific scour analyses will be performed by the Engineer. For use in scour analyses, the particle size through which 50 percent of soil by weight passing, D_{50}, are indicated on the generalized subsurface profile and respective grain size distribution curves included in Appendix R.

Site Grading and Earthwork

We understand that the replacement bridges will incorporate the existing embankments to the extent possible. Some site grading/reshaping of the existing embankments will be required. After bridge demolition, site grading and subgrade preparation should include necessary clearing and grubbing of trees and underbrush and stripping the organic-containing surface soils in work areas. Where fill depths in excess of 3 ft are planned, stumps may be left after close cutting trees to grade, as per AHTD criteria. Otherwise, the tree stumps must be completely excavated and properly backfilled. The depth of stripping will be variable, with deeper stripping depths in the low-lying, poorly drained, and/or wooded areas, and less stripping required in the higher-terrain areas. In general, the stripping depth is estimated to be about 6 to 12 in. in cleared areas, but may be 18 to 24 in. or more in the localized wooded areas. The zone of organic surface soils should be completely stripped in the embankment footprints.

Where the existing shoulder pavements are within 3 ft of the plan subgrade elevation, the existing pavement surface should be scarified to a minimum depth of 6 inches. The scarified soil should be recompacted to a stable condition. Where pavements are to be demolished, consideration may be given to utilizing the processed asphalt concrete, Portland cement concrete, and/or aggregate base for embankment fill in areas/zones where piling is not planned. In this case, the demolished materials should be thoroughly blended and processed to a reasonably well-graded mixture with a maximum particle size of 2 inches.

Following demolition, stripping and grubbing, and prior to fill placement or otherwise continuing with subgrade preparation, the extent of weak and unsuitable soils should be determined. Proof-rolling is recommended to evaluate subgrade stability. Proof-rolling should be performed with a loaded tandem-wheel dump truck or similar equipment. Unstable soils exhibiting a tendency to rut and/or pump should be undercut and replaced with suitable fill. Care should be taken that undercuts, stump holes, and other excavations or low areas resulting from subgrade preparation are properly backfilled with compacted fill.

Based on the results of the borings, undercuts on the order of 2 to 4 ft, more or less, will be required at the bridge abutments. It should be noted that the estimated undercut depths are below existing grade and are based on the results of the borings and test pits. Required as-built depth of undercut will vary with seasonal site conditions and final grading plans. As-built undercut requirements must be field verified by the Engineer or Department.

Undercuts for embankments may be backfilled with suitable embankment fill. Should excavations or deeper undercuts encounter shallow water or seepage, or if areas of seepage are encountered during the work, backfill should consist of clean sand (AHTD Standard Specifications Section 302, SM-1 with less than 10 percent passing the No. 200 sieve), stone backfill (AHTD Standard Specifications, Section 207), or clean aggregate (AHTD Standard Specifications Subsections 403.01 and 403.02 Class 3 mineral aggregate) extending up to an elevation above the inflow of seepage. 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.

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

Subgrade preparation and mass undercuts should extend at least 10 ft beyond the embankment toes to the extent possible. Subgrade preparation in roadway areas should extend at least 3 ft outside pavement shoulder edges to the extent possible. The existing drainage features should be completely mucked out and all loose and/or organic soils removed prior to fill placement.

Fill and backfill may consist of unclassified borrow free of organics and other deleterious materials as per AHTD Standard Specifications Subsection 210.06. Granular soils must be protected from erosion with a minimum 18-in.-thick armor of clayey soil with a PI in the range of 5 to 18.

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

CONSTRUCTION CONSIDERATIONS

Groundwater and Seepage Control

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

Groundwater was encountered at 4- to 16-ft depth or at approximately El 178 to El 160 (average El 166) over the period of April 2013 to March 2014. In addition, shallow perched groundwater may be encountered in the near-surface soils. Seepage into excavations and cuts can typically be controlled by ditching or sump-and-pump methods. If seepage into excavations becomes a problem, backfill should consist of clean sand (AHTD Standard Specifications

Section 302, SM-1 with less than 10 percent passing the No. 200 sieve), stone backfill (AHTD Standard Specifications Section 207), or clean aggregate (AHTD Standard Specifications Subsections 403.01 and 403.02 Class 3 mineral aggregate) to an elevation above the inflow of seepage. In areas of seepage infiltration, the granular fill should be encapsulated with a filter fabric complying with AHTD Standard Specifications Subsection 625.02, Type 2 and vented to positive discharge.

Where perched water is 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. We also recommend that blanket drains be constructed in any drainage features encountered during the grading work.

Piling

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.

Pile installation should be monitored by qualified personnel to maintain specific and complete driving records and observe pile installation procedures. Driving records should be available for review by the Engineer during pile installation. Compatible driving equipment should be utilized based on the results of drivability analyses performed by the Department. Blow counts for steel piles should be limited to about 20 blows per inch. As-built pile capacities should be evaluated by use of wave equation analysis of piles (WEAP) in accordance of AHTD Standard Specifications, Section 805.09, Method B.

CLOSURE

The Engineer 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 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 final report:

Plate 1	Site Vicinity Map
Plates 2a through 2f	Plans of Borings
Plates 3 through 58	Boring Logs
Plate 59	Key to Terms and Symbols
Appendix A	Summary of Subsurface Exploration Program
Appendix B	Generalized Subsurface Profiles
Appendix C	Results of Laboratory Test Results
Appendix D	Results of Site-Specific Ground Motion Response Analyses
Appendix E	Results of Liquefaction Analyses
Appendix F	Ultimate Single Pile Capacity
Appendix G	Results of WEAP Driveability Analyses
Appendix H	Results of Lateral Load Analyses
Appendix I	Relevant Cross Sections
Appendix J	Results of Stability Analyses – Reinforced End Slope at West Abutment
Appendix K	Results of Stability Analyses – Reinforced Side Slopes at West Abutment
Appendix L	Results of Stability Analyses – Reinforced End Slope at East Abutment
Appendix M	Results of Stability Analyses – Reinforced Side Slopes at East Abutment
Appendix N	Results of Stability Analyses – Conventional Side Slopes at Sta 1875+00
Appendix O	Results of Stability Analyses – Conventional Side Slopes at Sta 1913+00
Appendix P	Recommended Approximate Limits of Geogrid-Reinforced Embankments
Appendix Q	Stability of East River Bank near Pier 2
Appendix R	Scour Analysis Parameters

* * * * *

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

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

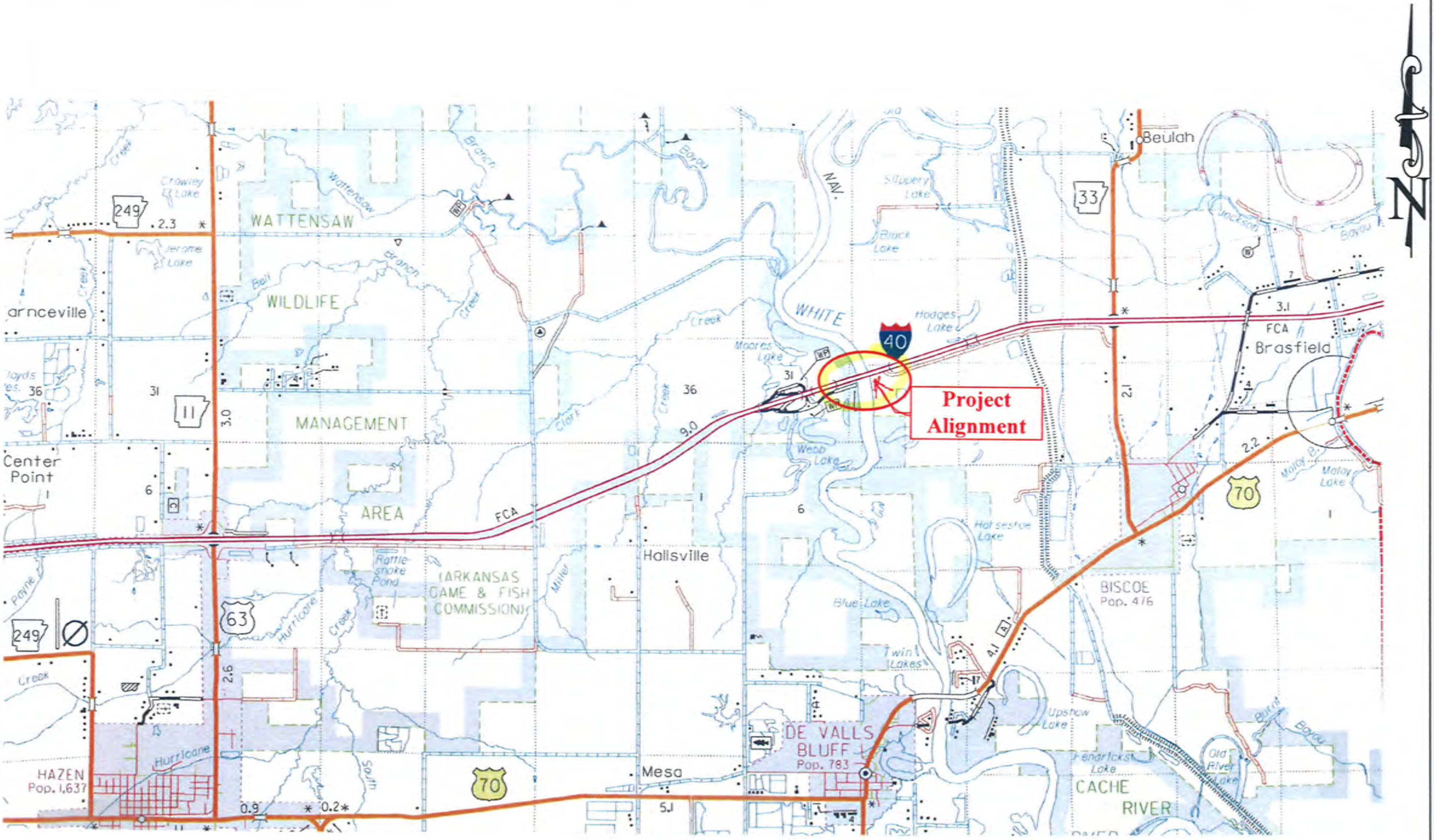
Yongsheng Zhao, Ph.D., P.E.
Project Engineer

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

Mark E. Wyatt, P.E.
President

YZ/MEW:jw

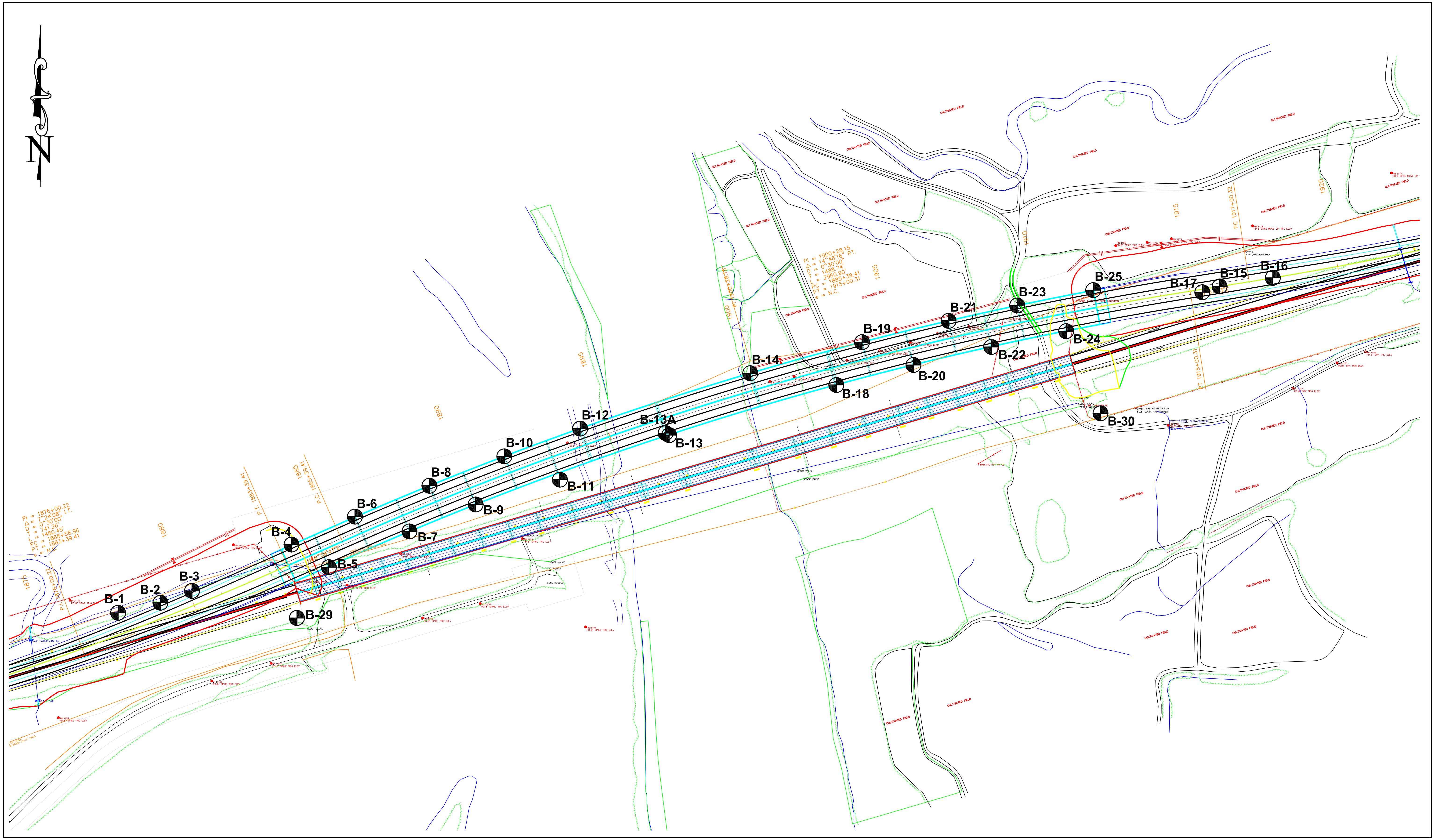
Copies submitted:	Jacobs Engineering Group Inc.		
	Attn:	Mr. Mark A. Asher, P.E.	(2+electronic)
	Attn:	Mr. Chris Criswell, P.E.	(1-electronic)
	Attn:	Mr. Mark Schurk, P.E.	(1-electronic)
	Attn:	Mr. Charles Wise, P.E.	(1-electronic)



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

Plan of Borings
I-40 over White River
AHTD Job No. BB0610
Prairie County, Arkansas

Job No.: 13-017	As Shown
June 2013	Plate 1



Note: Base drawing provided by Jacobs.

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

Plan of Borings
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Job No.: 13-017

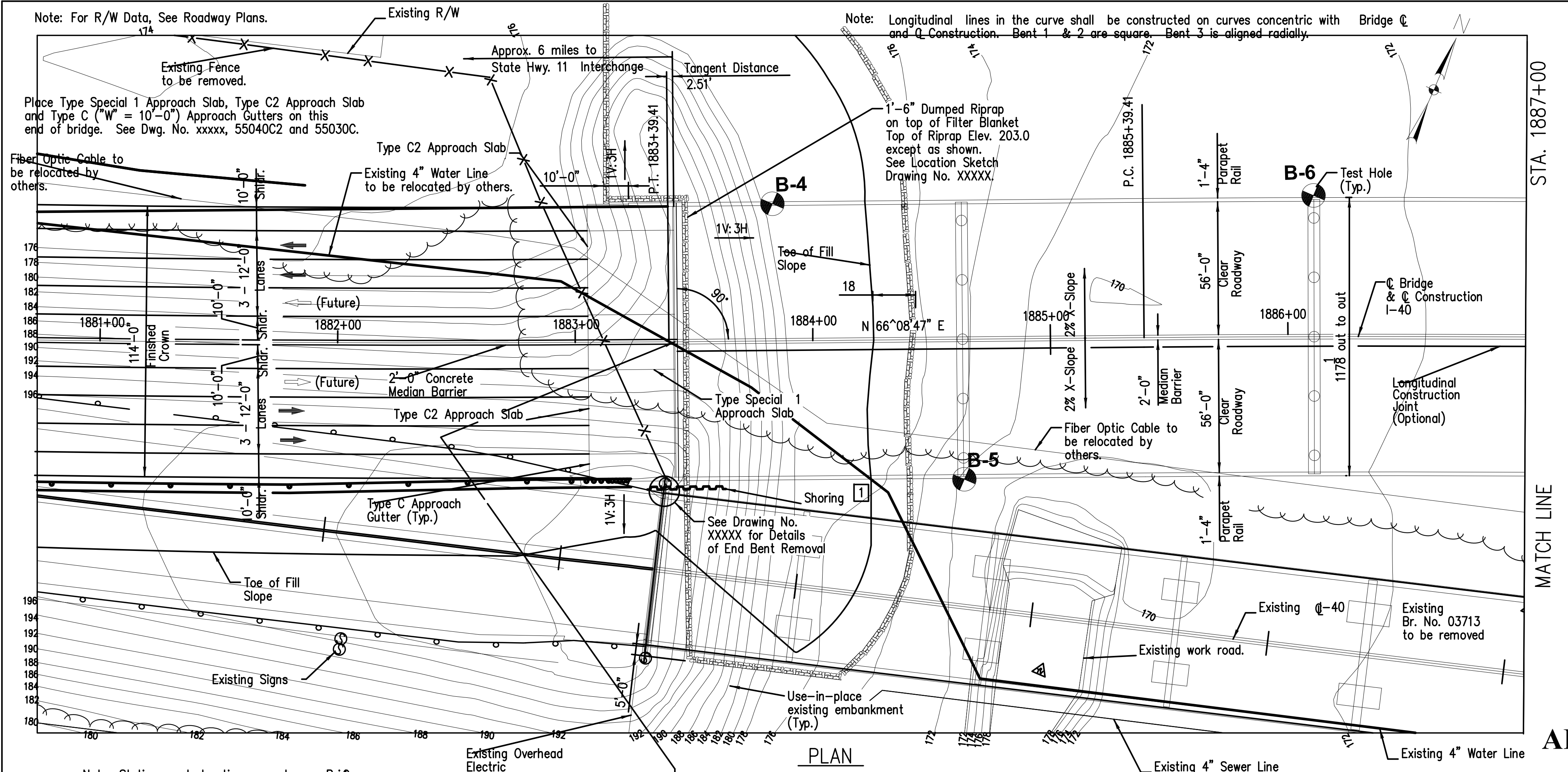
Scale: As Shown

March 26, 2014

Plate 2a

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX



Plan of Borings - Plate 2b

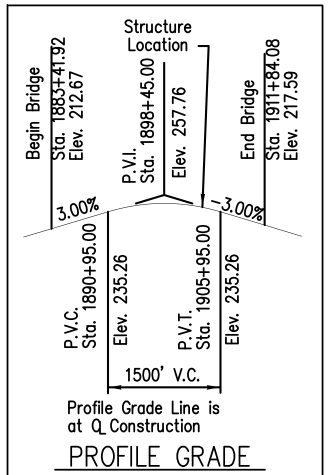
AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas

Curve Data
℄C Construction HORIZONTAL CURVE DATA
PI = 1876+00.22 R = 11,459.16' Δ = 7^24'08" Lt. D = 0^30'00" T = 741.26' L = 1480.45' PC = 1868+58.96 PT = 1883+39.41 e = Normal Crown

Curve Data
℄C Construction HORIZONTAL CURVE DATA
PI = 1900+28.15 R = 11,459.16' Δ = 14^48'16" Rt. D = 0^30'00" T = 1488.74' L = 2960.90' PC = 1885+39.41 PT = 1915+00.31 e = Normal Crown

Profile Grade Detail



Profile Grade Line is at ℄ Construction

PROFILE GRADE

Note: All elevations in detail above are shown at working point.

"N" VALUES

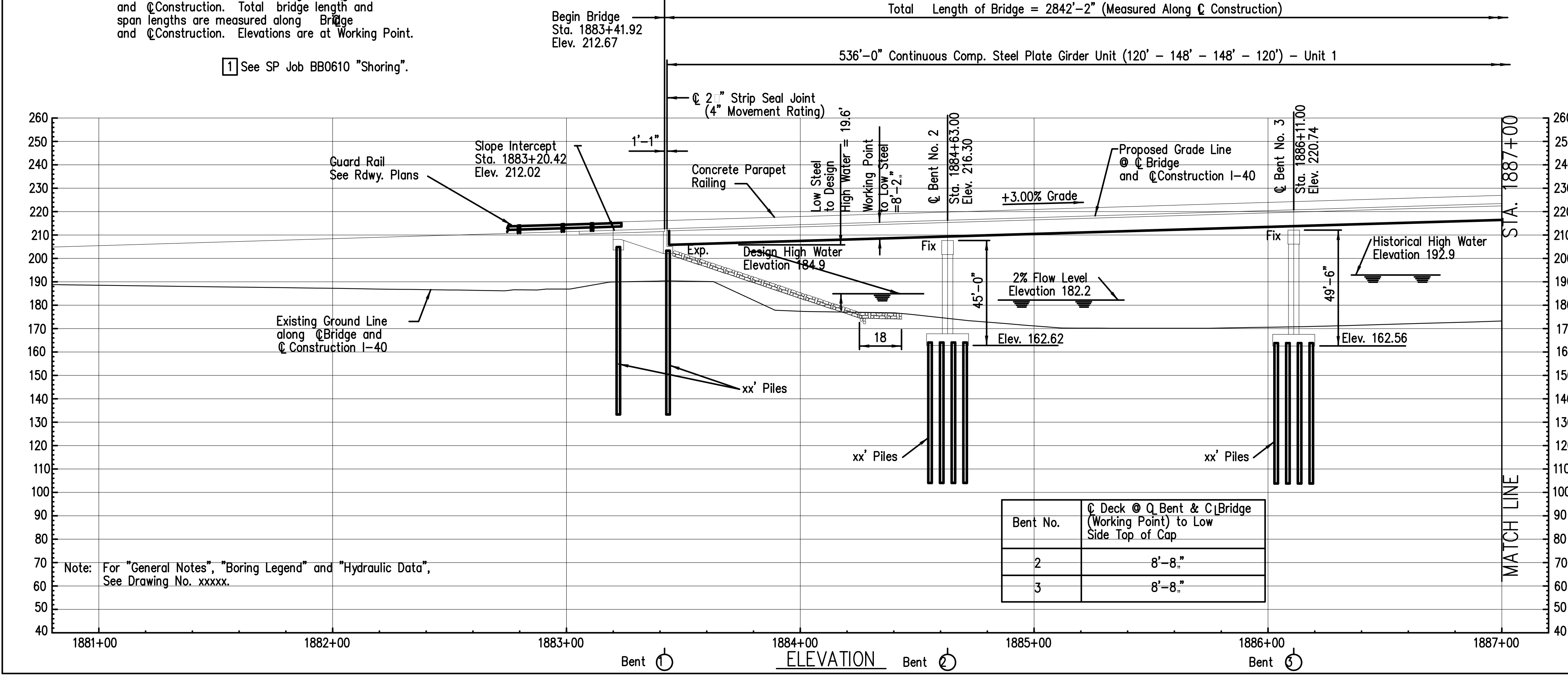
Sta. 1883+42 – 60' Left of ℄C Construction

Sta. 1884+63 – 60' Right of ℄C Construction

Sta. 1886+11 – 60' Left of ℄C Construction

Note: Stations and elevations are along Bridge and ℄Construction. Total bridge length and span lengths are measured along Bridge and ℄Construction. Elevations are at Working Point.

1 See SP Job BB0610 "Shoring".



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
2	8'–8"
3	8'–8"

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. xxxxx.

30 ft 0 30 60 ft

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 1 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'–0"
BRIDGE NO. XXXXX DRAWING NO.XXXXX

Note: Layout provided by Jacobs.

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

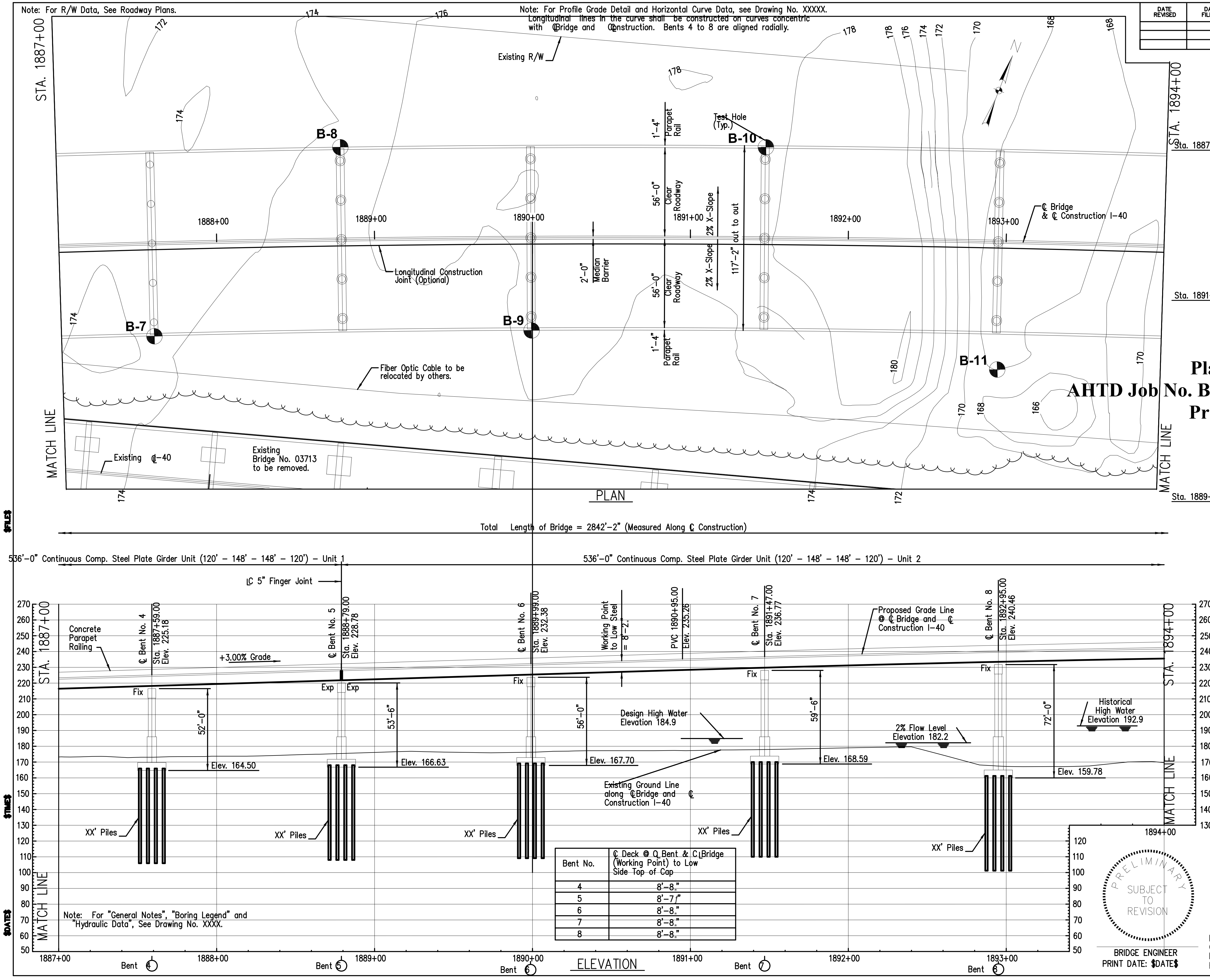
Note: For R/W Data, See Roadway Plans.

Note: For Profile Grade Detail and Horizontal Curve Data, see Drawing No. XXXXX. Longitudinal lines in the curve shall be constructed on curves concentric with ℄Bridge and ℄Construction. Bents 4 to 8 are aligned radially.

Plan of Borings - Plate 2c

AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
4	8'-8."
5	8'-7ƒ"
6	8'-8."
7	8'-8."
8	8'-8."

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. XXXX.

"N" VALUES

Sta. 1887+59 – 60' Right of ℄ Construction

Sta. 1888+79 – 60' Left of ℄ Construction

Sta. 1891+47 – 60' Left of ℄ Construction

Sta. 1889+99 – 60' Right of ℄ Construction

Sta. 1889+99 – 60' Right of ℄ Construction

Sta. 1891+47 – 60' Left of ℄ Construction

Sta. 1892+95 – 60' Right of ℄ Construction

Note:
Stations and elevations are along ℄Bridge and ℄Construction. Total bridge length and span lengths are measured along ℄Bridge and ℄Construction. Elevations are at Working Point.

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 2 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: LHG DATE: 12/30/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

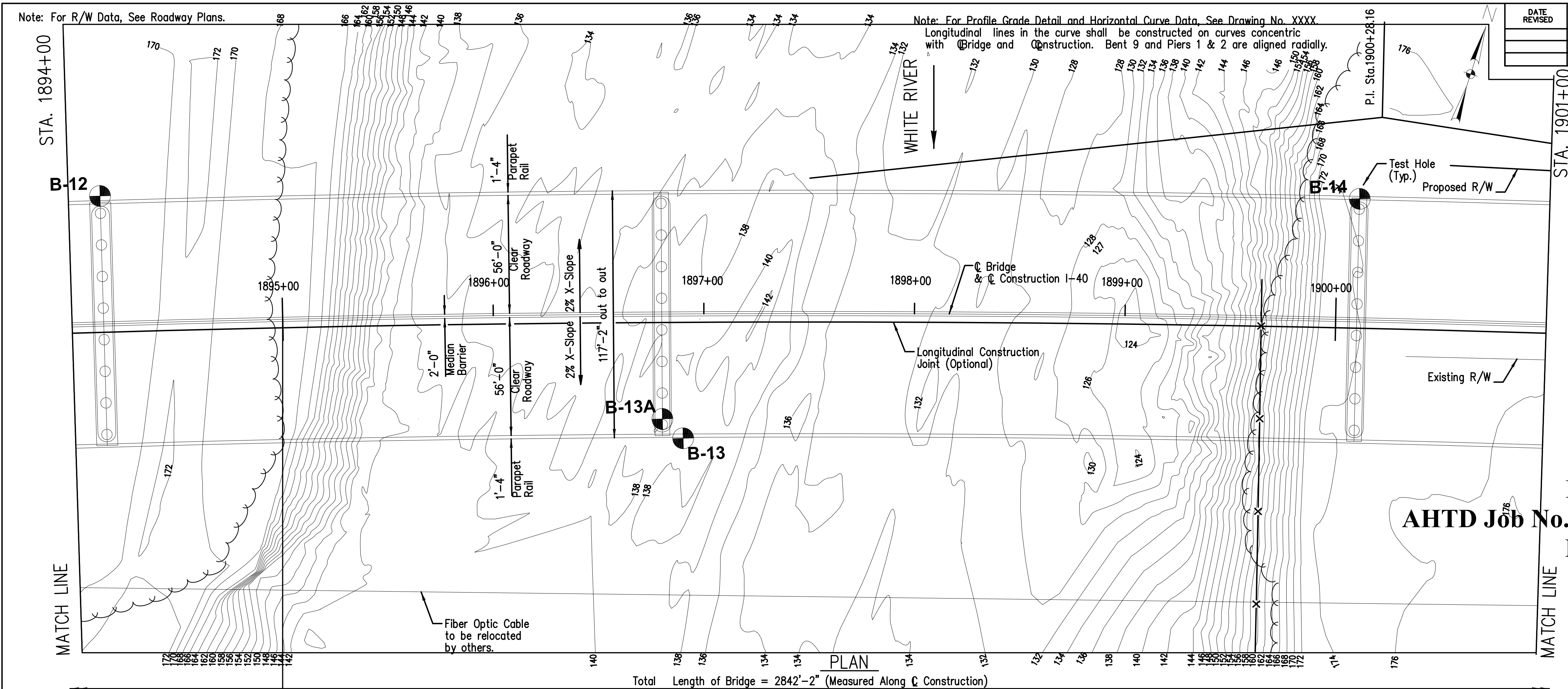
$FILE$
$TIME$
$DATE$

Note: Layout provided by Jacobs.

JACOBS

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

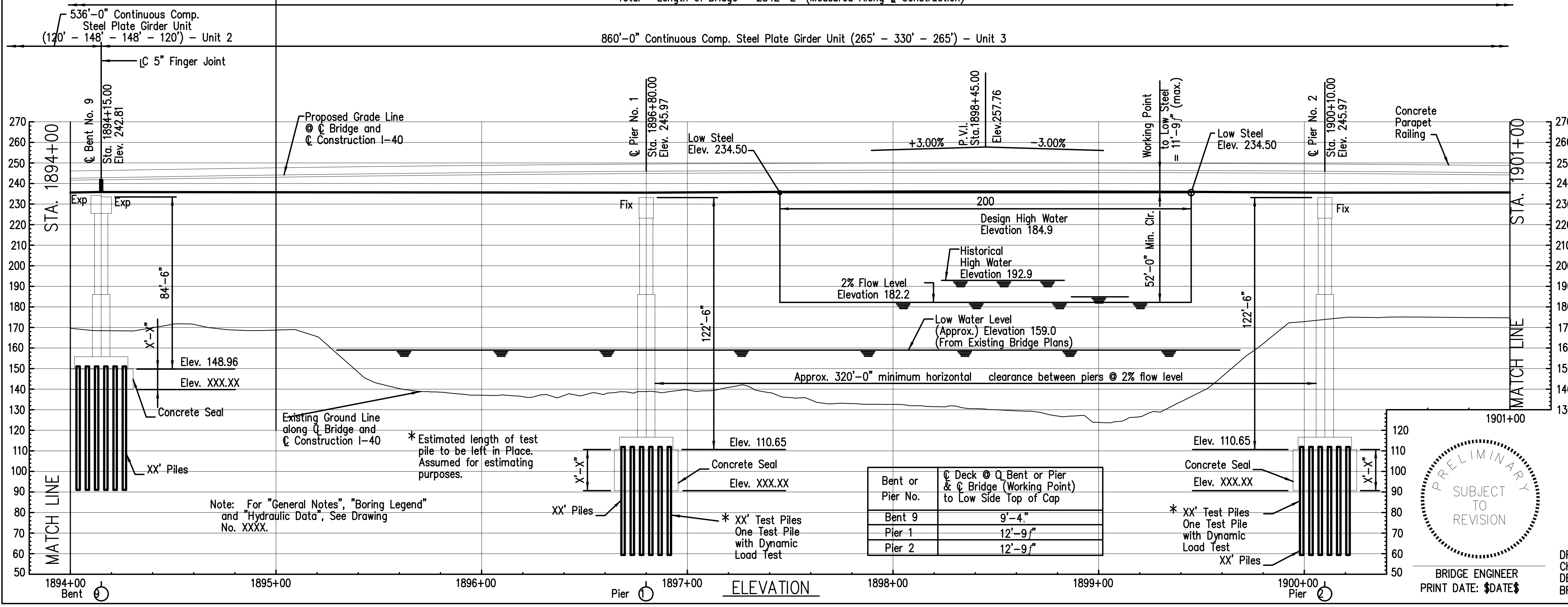


Plan of Borings - Plate 2d
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

"N" VALUES

Sta. 1894+15 – 60' Left of ℄ Construction

Sta. 1896+80 – 60' Right of ℄ Construction



*Estimated length of test pile to be left in Place. Assumed for estimating purposes.

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. XXXX.

Bent or Pier No.	℄ Deck @ ℄ Bent or Pier & ℄ Bridge (Working Point) to Low Side Top of Cap
Bent 9	9'-4,"
Pier 1	12'-9ƒ"
Pier 2	12'-9ƒ"

Boring No. 14
Sta. 1900+10 – 60' Left of ℄ Construction

.05–1.5, N=2
2.5–3.5, N=7
4.5–5.5, N=5
6.5–7.5, N=4
9.0–10.0, N=8
14.0–15.0, N=6
19.0–20.0, N=5
24.0–25.0, N=14
29.0–30.0, N=32
34.0–35.0, N=36
39.0–40.0, N=40
44.0–45.0, N=41
49.0–50.0, N=45
58.0–59.0, N=50(11")
68.0–69.0, N=50(11")
78.0–79.0, N=50(10")
88.0–88.5, N=50(6")
97.0–97.5, N=50(7")
108.0–108.5, N=50(6")
118.0–118.5, N=50(4")

Note: Stations and elevations are along ℄ Bridge and ℄ Construction. Total bridge length and span lengths are measured along ℄ Bridge and ℄ Construction. Elevations are at Working Point.

SHEET 3 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

$FILE$
$TIME$
$DATE$

Note: Layout provided by Jacobs.

JACOBS

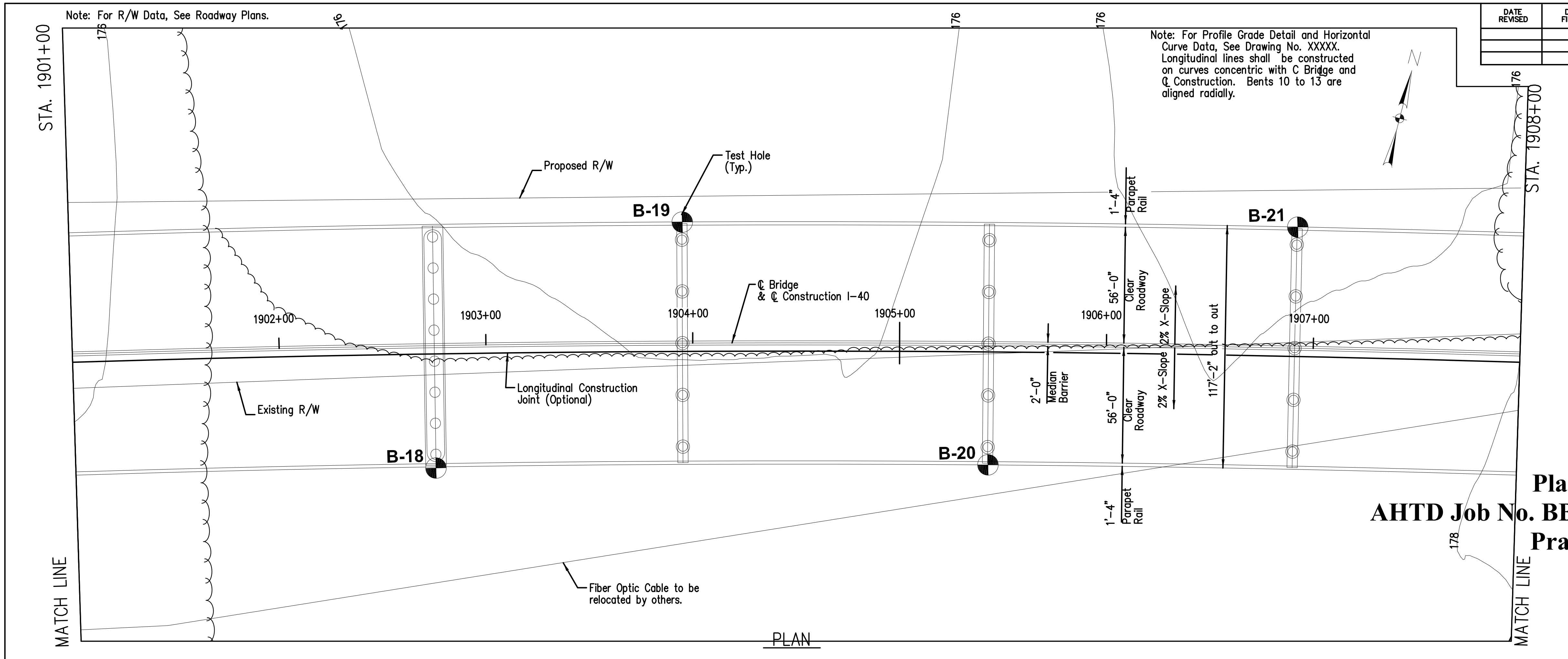
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX

Plan of Borings - Plate 2e
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: For R/W Data, See Roadway Plans.

Note: For Profile Grade Detail and Horizontal Curve Data, See Drawing No. XXXXX. Longitudinal lines shall be constructed on curves concentric with C Bridge and ℄ Construction. Bents 10 to 13 are aligned radially.



"N" VALUES

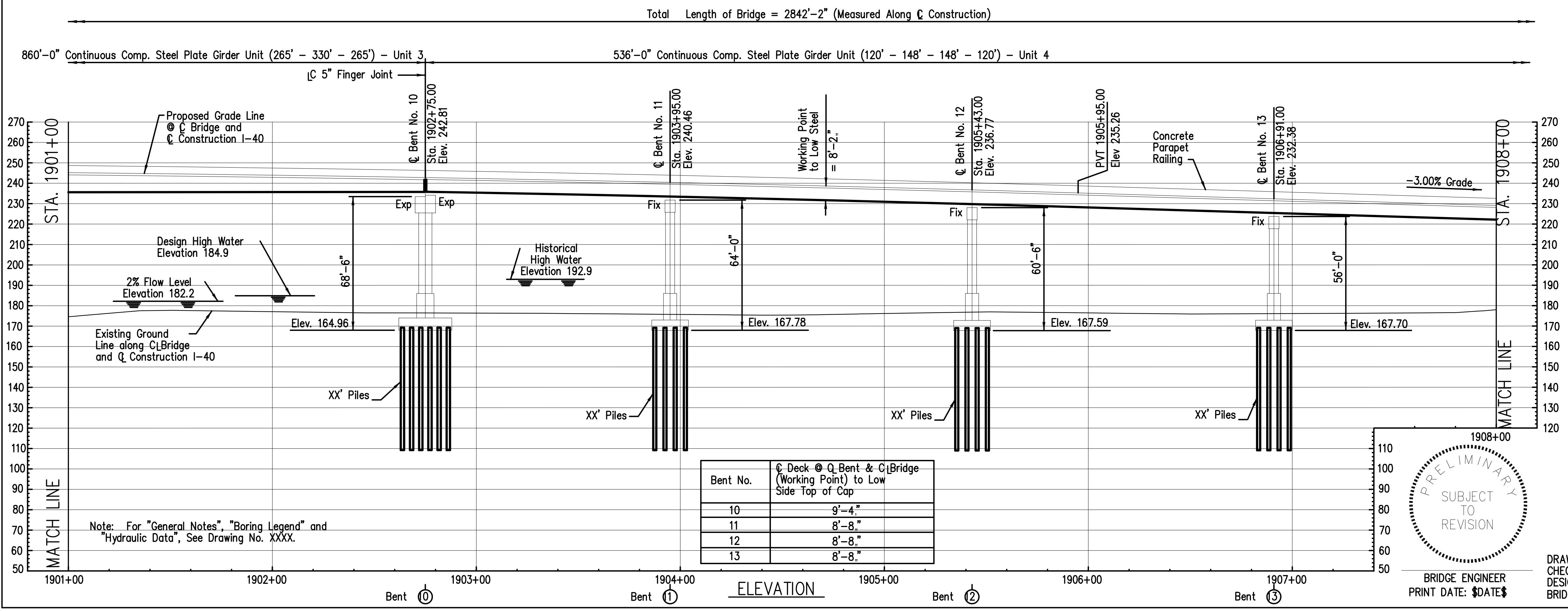
Sta. 1902+75 – 60' Right of ℄ Construction

Sta. 1903+95 – 60' Left of ℄ Construction

Sta. 1905+43 – 60' Right of ℄ Construction

30 ft 0 30 60 ft

Sta. 1906+91 – 60' Left of ℄ Construction



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
10	9'-4."
11	8'-8."
12	8'-8."
13	8'-8."

Note: For "General Notes", "Boring Legend" and "Hydraulic Data", See Drawing No. XXXX.

Note: Stations and elevations are along ℄ Bridge C ℄ Construction. Total bridge length and span lengths are measured along C L Bridge and C ℄ Construction. Elevations are at Working Point.

PRELIMINARY SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

SHEET 4 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

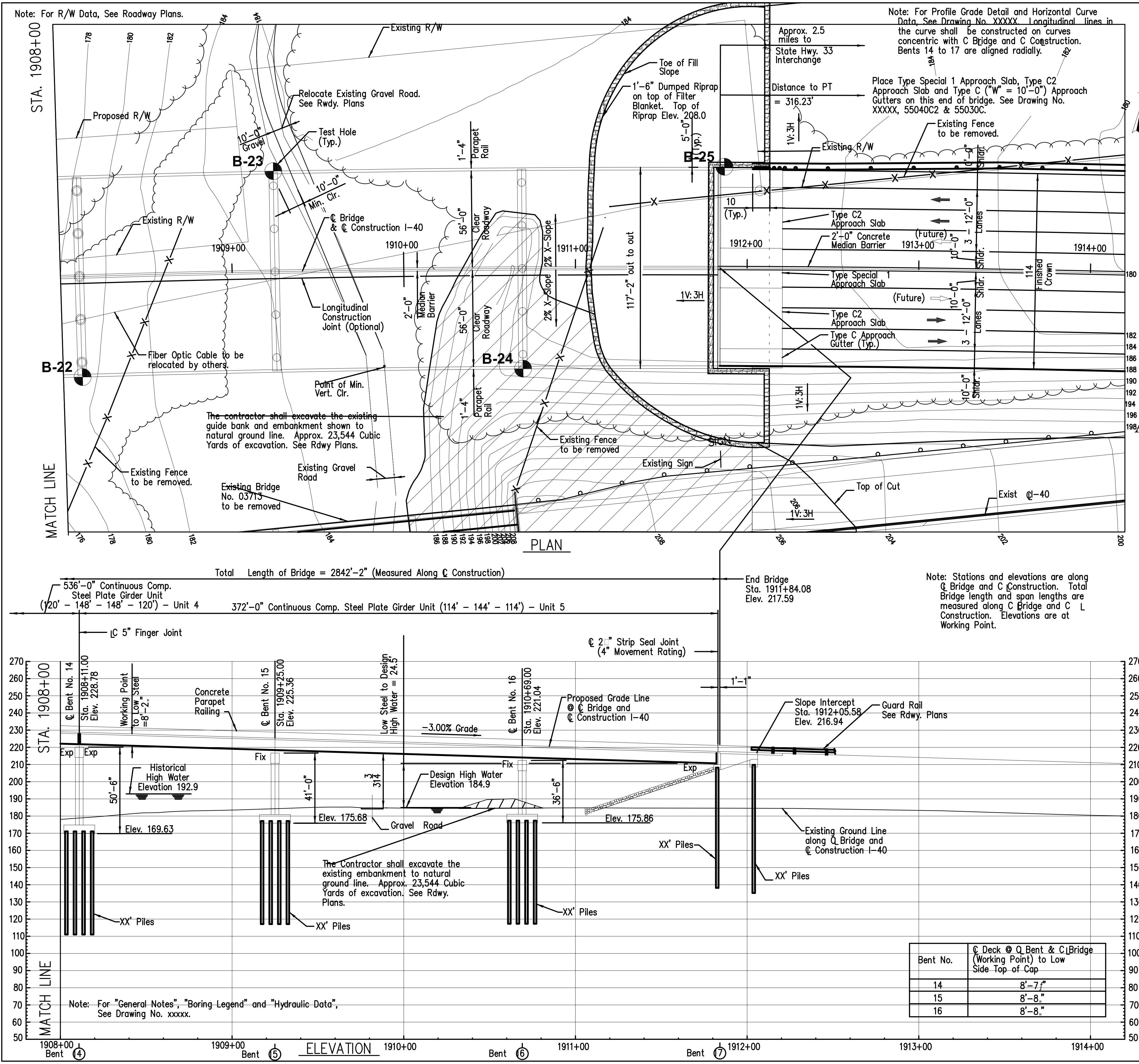
$FILES$
$TIME$
$DATE$

Note: Layout provided by Jacobs.

JACOBS

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.		BB0610	XX	XXX

① XXXXX – LAYOUT – XXXXX



Bent No.	℄ Deck @ ℄ Bent & ℄ Bridge (Working Point) to Low Side Top of Cap
14	8'-7 1/2"
15	8'-8."
16	8'-8."

"N" VALUES

Sta. 1908+11 – 60' Right of ℄ Construction

Sta. 1909+25 – 60' Left of ℄ Construction

Plan of Borings - Plate 2f
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Sta. 1910+69 – 60' Right of ℄ Construction

Sta. 1911+84 – 60' Left of ℄ Construction

30 ft 0 30 60 ft

SHEET 5 OF 6
LAYOUT OF
BRIDGE OVER WHITE RIVER
WHITE RIVER STR. & APPRS. (F)
PRAIRIE COUNTY
ROUTE 40 SECTION 42
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARKANSAS

PRELIMINARY
SUBJECT TO REVISION

BRIDGE ENGINEER
PRINT DATE: $DATE$

DRAWN BY: LHG DATE: 12/16/13 FILENAME: XXXXXXXX.XXX
CHECKED BY: MAA DATE: 2/1/14
DESIGNED BY: CJC DATE: 12/11/13 SCALE: 1" = 30'-0"
BRIDGE NO. XXXXX DRAWING NO. XXXXX

Note: Layout provided by Jacobs.

JACOBS

13-017

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

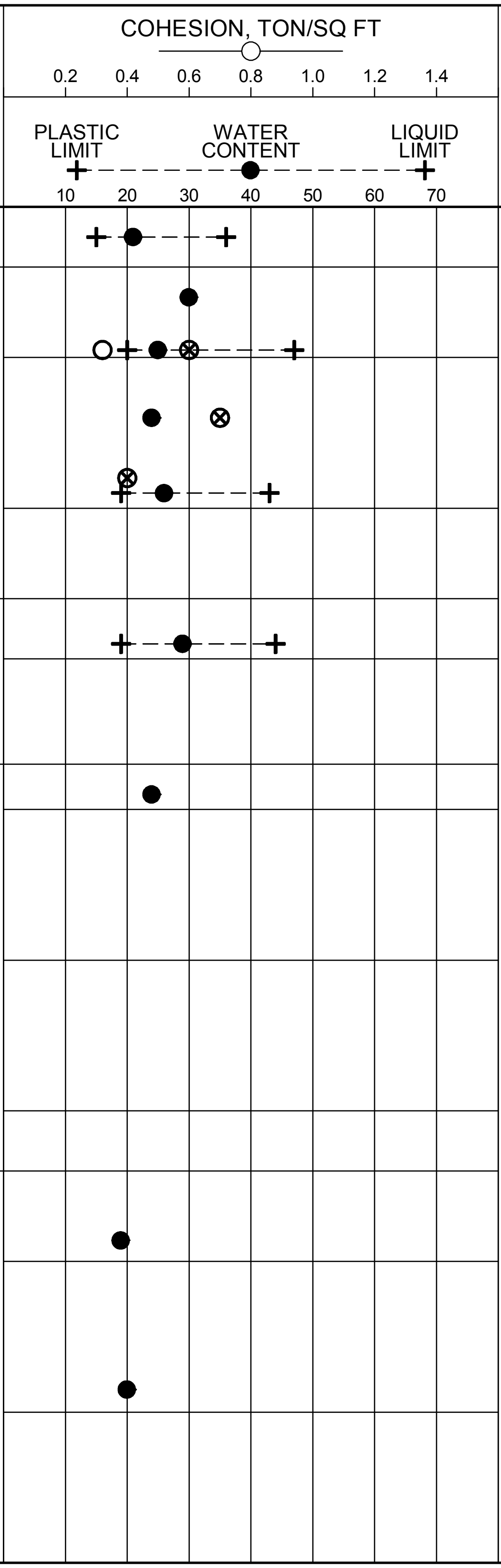
LOG OF BORING NO. 1

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1877+60, 70 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 184.3	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm reddish brown silty clay w/clayey silt pockets (fill)	8		82
	Soft to firm brown and gray silty clay w/ferrous stains - firm to stiff at 2 to 4 ft	5		
5	- with organic stains to 3 ft - water at 4 ft - stiff at 4 to 8 ft		96	96
	- firm below 8 ft			
10			95	97
	Soft brownish gray silty clay, moist			
15		6		97
20	Medium dense brown and gray silty fine sand w/occasional silty clay pockets	11		13
25	- dense, less silty and coarser below 24 ft	35		
30		41		
	Dense brown and gray fine to medium sand, slightly silty			
35		50/7"		6
40	- with clayey fine sand seams at 39 to 45 ft	50/6"		16
		41		



COMPLETION DEPTH: 70.0 ft
DATE: 4-30-13

DEPTH TO WATER
IN BORING: 4 ft

DATE: 4/30/2013

LGBNEW 13-017.GPJ 12-2-13

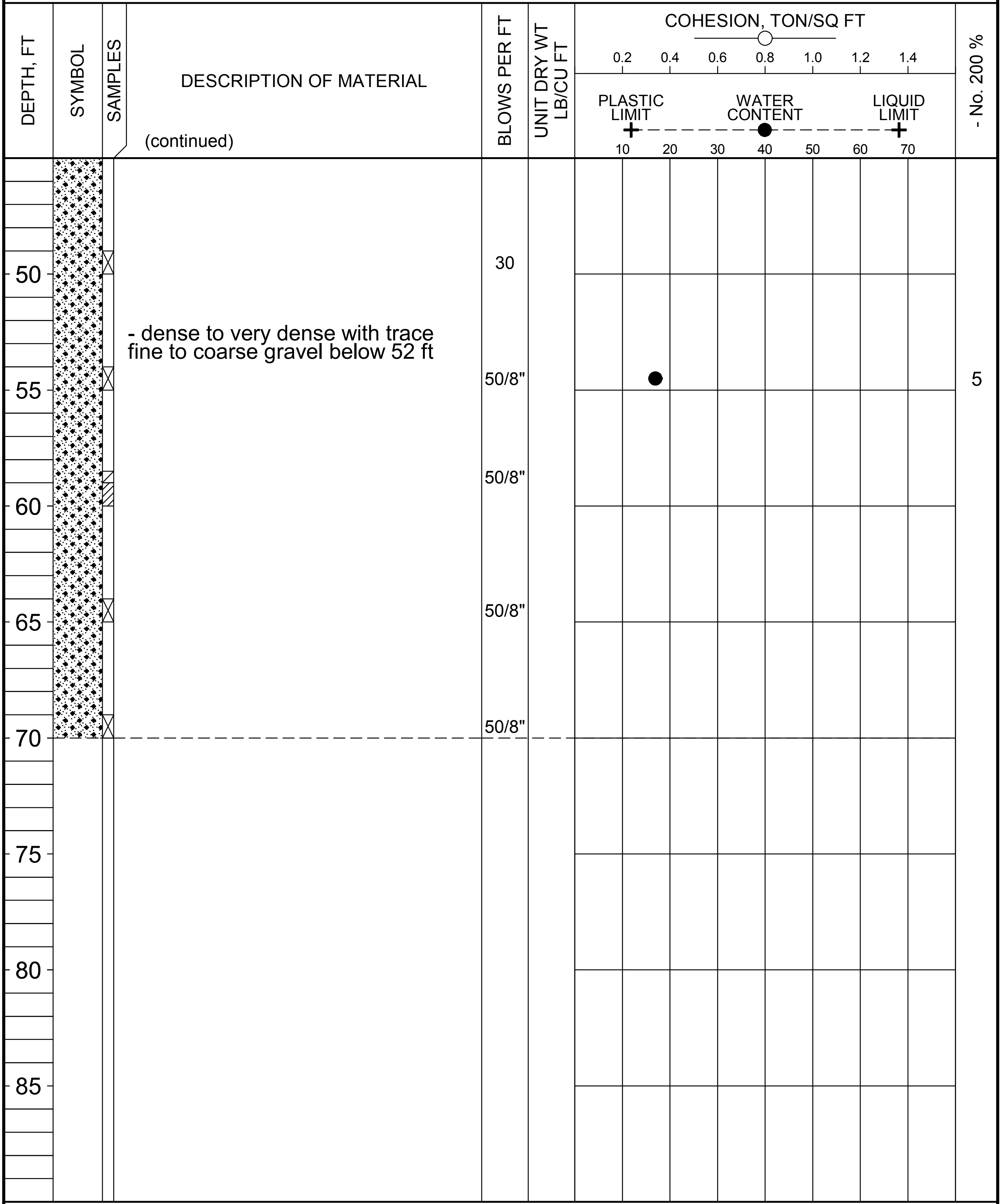
Grubbs, Hoskyn,
Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 1

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1877+60, 70 ft Lt



- dense to very dense with trace fine to coarse gravel below 52 ft

COMPLETION DEPTH: 70.0 ft
DATE: 4-30-13

DEPTH TO WATER
IN BORING: 4 ft

DATE: 4/30/2013

LGBNEW 13-017.GPJ 12-2-13

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

LOG OF BORING NO. 2

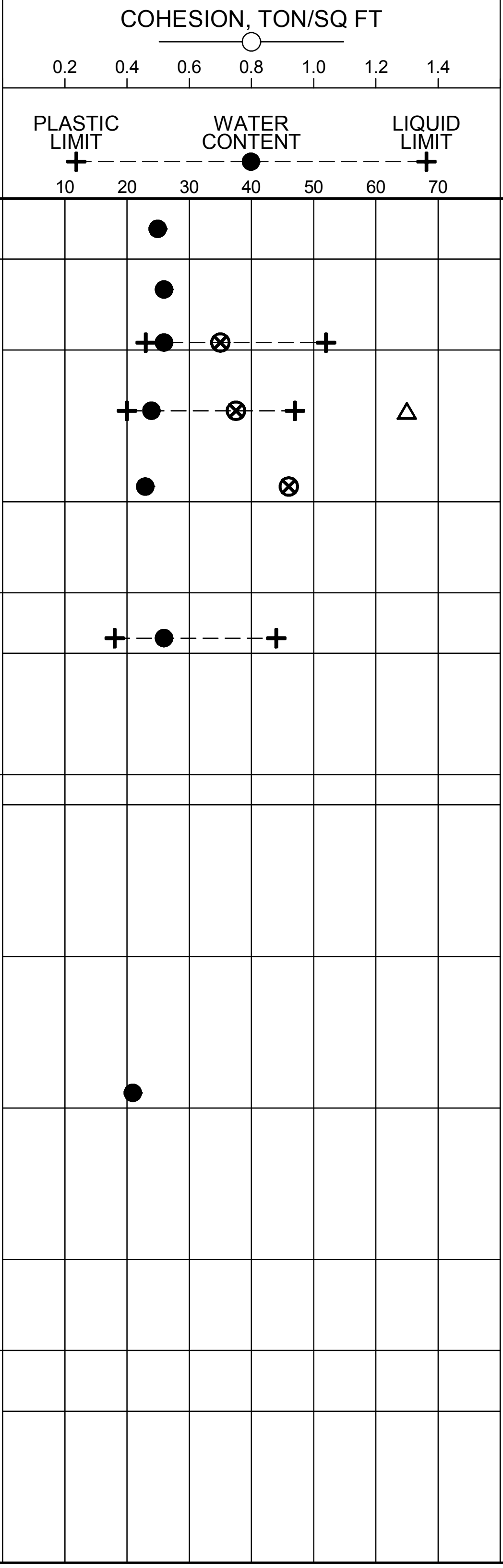
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 13.5 ft /Wash

LOCATION: Sta 1879+00, 50 ft Lt

SURF. EL: 191.3

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Very soft reddish tan silty clay w/clayey silt pockets (fill)	2		
Stiff brown and gray silty clay w/clay pockets and ferrous stains	15		
		95	98
- stiff to very stiff at 6 to 8 ft		99	98
Firm brownish gray clay w/ferrous stains	7		98
Medium dense brown and gray silty fine sand	23		
	24		
- coarser, less silty below 28 ft - dense at 28 - 32 ft	40		14
- medium dense below 32 ft			
	14		
Dense to very dense brown and gray fine to medium sand, slightly silty	50/10"		
	48		



COMPLETION DEPTH: 70.0 ft
DATE: 5-1-13

DEPTH TO WATER
IN BORING: Dry to 13.5 ft

DATE: 5/1/2013

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

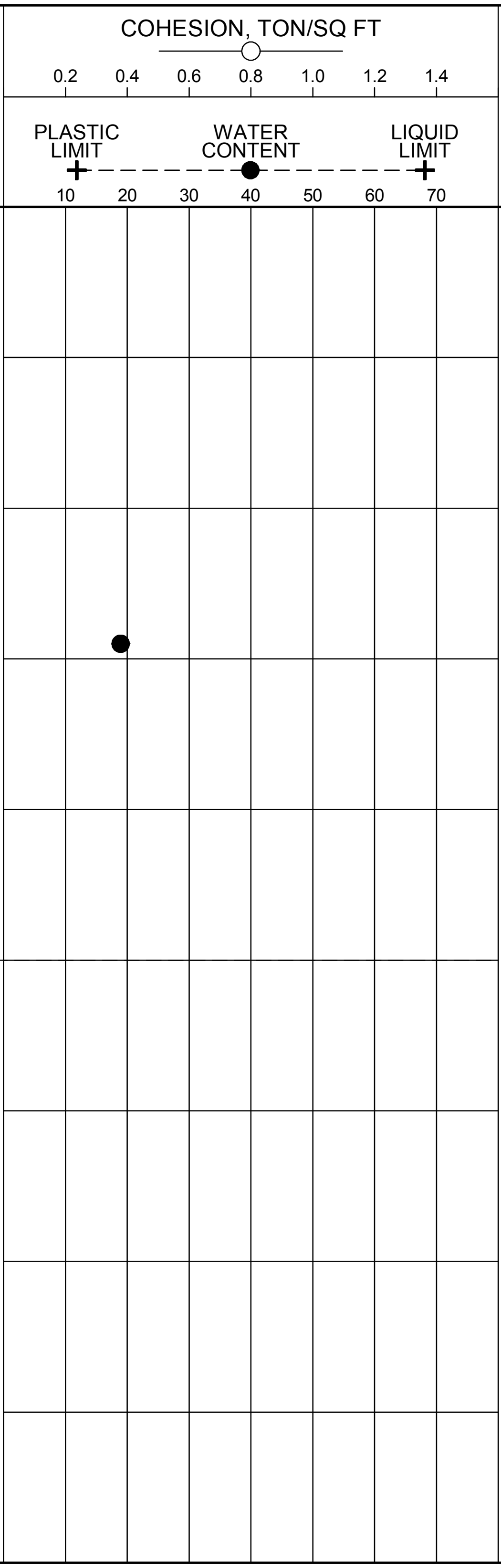
LOG OF BORING NO. 2

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 13.5 ft /Wash

LOCATION: Sta 1879+00, 50 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50		☒	- medium dense with silty clay layers at 48 - 52 ft	20		
55		☒	- dense to very dense with trace fine to coarse gravel below 52 ft	50/8"		
60		☒		50/8"		7
65		☒		50/6"		
70		☒		50/10"		
75						
80						
85						



COMPLETION DEPTH: 70.0 ft
DATE: 5-1-13

DEPTH TO WATER
IN BORING: Dry to 13.5 ft

DATE: 5/1/2013

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

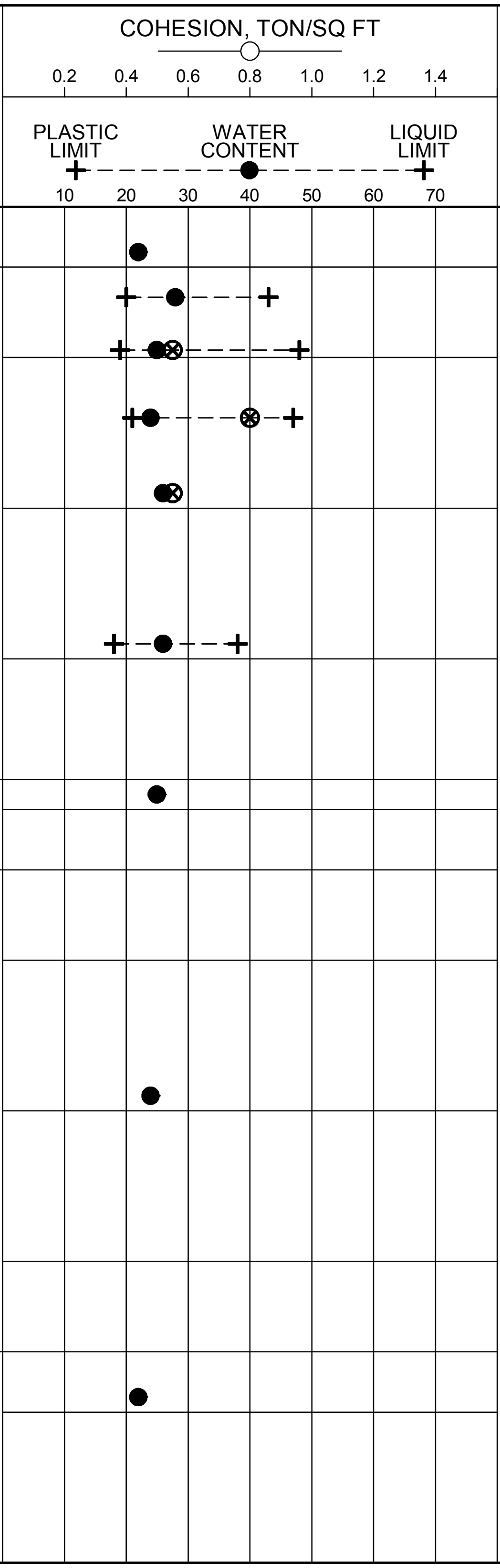
LOG OF BORING NO. 3

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1880+15 , 50 ft Lt

SURF. EL: 190.5



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Soft reddish tan and brown clay w/clayey silt pockets and surface organics	4		
Soft brown and gray silty clay w/ferrous stains	6		97
- stiff at 4 - 12 ft		99	95
		100	95
- firm to stiff below 12 ft	10		93
Medium dense clayey fine sand, wet	15		39
Dense brown and tan silty fine sand w/trace silty clay pockets, wet	39		
- brown and gray below 28 ft	31		15
	30		
Medium dense brown and gray fine to medium sand, slightly silty	22		6
- dense below 42 ft	39		

COMPLETION DEPTH: 70.0 ft
DATE: 5-2-13

DEPTH TO WATER
IN BORING: Dry to 8 ft

DATE: 5/2/2013

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

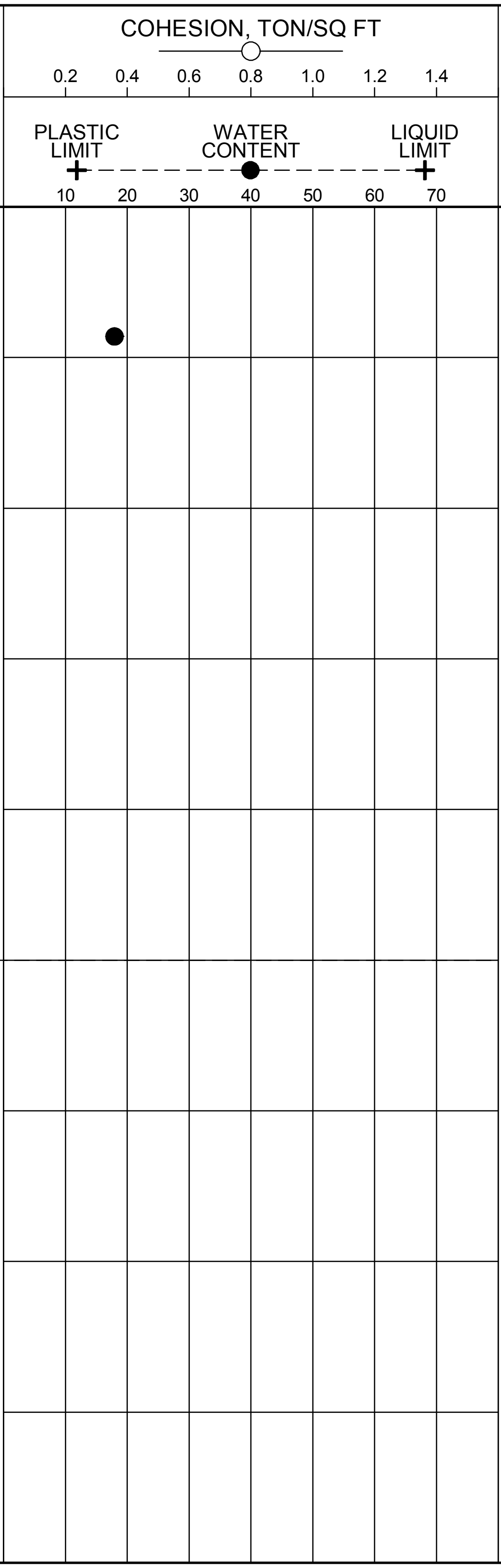
LOG OF BORING NO. 3

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 1880+15 , 50 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
50	- dense to very dense below 48 ft	50/7"		11
55	- with trace fine to coarse gravel below 52 ft	50/8"		
60		50/6"		
65		50/8"		
70		50/6"		
75				
80				
85				



COMPLETION DEPTH: 70.0 ft
DATE: 5-2-13

DEPTH TO WATER
IN BORING: Dry to 8 ft

DATE: 5/2/2013

13-017

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

L O G O F B O R I N G N O. 4

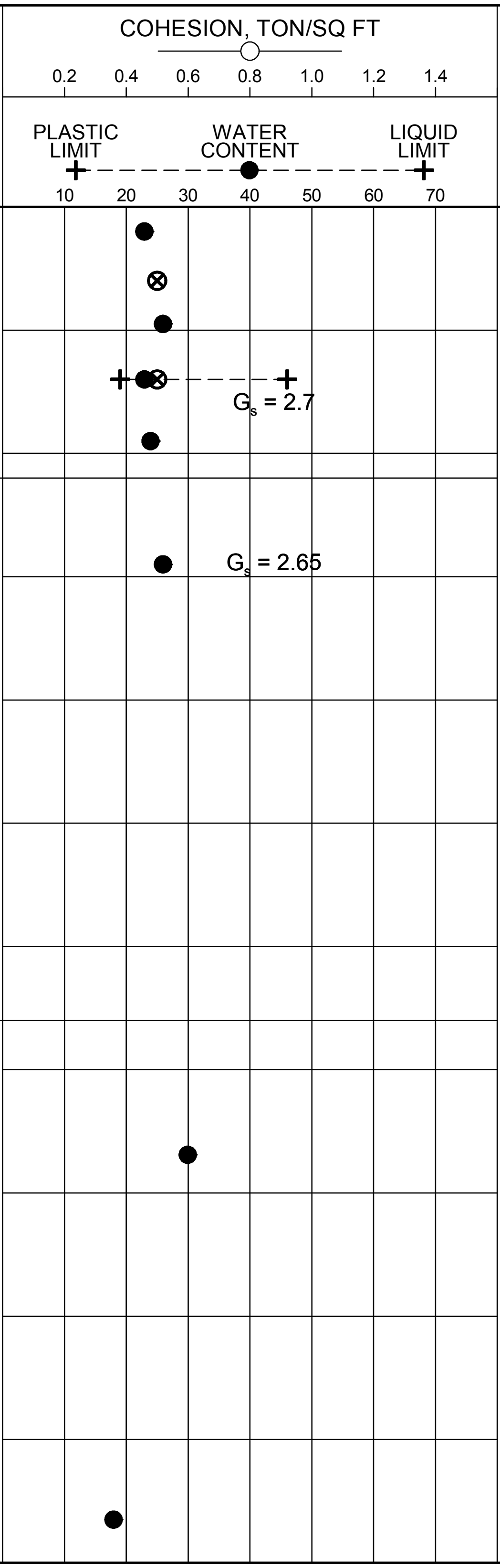
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1883+83, 60 ft Lt

SURF. EL: 171.9

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm gray and brown silty clay w/silt pockets and ferrous nodules and stains	9		
- stiff below 4 ft	13		
		100	37
- water at 8 ft	23		
Medium dense gray and brown fine to medium sand, slightly silty	22		5
	23		
	26		
- dense below 28 ft	45		
Dense gray fine to medium sand	50		
- dense to very dense below 38 ft	50/9"		4
	50/8"		
	50/9"		
	50/6"		10



COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

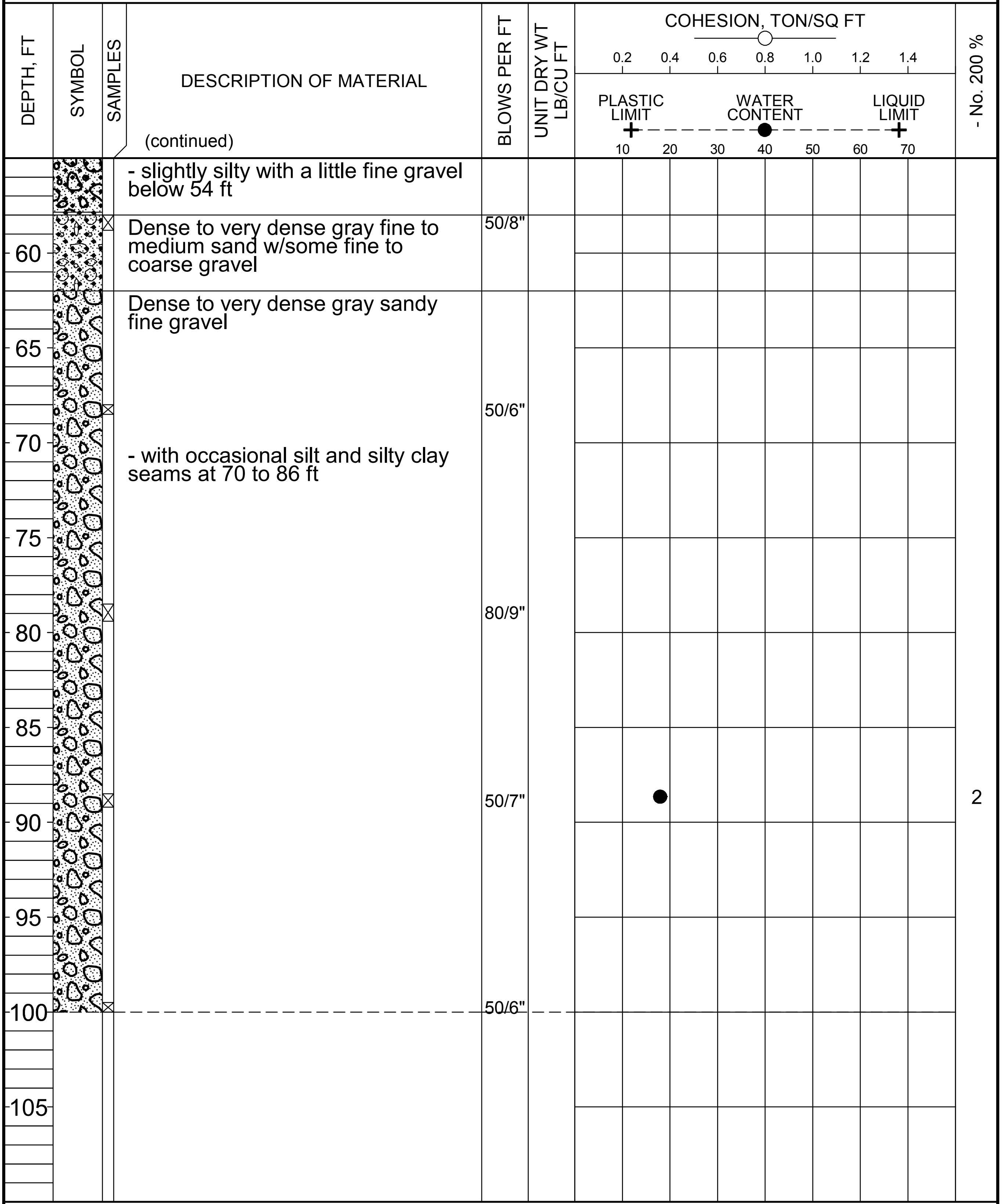
LGBNEW 13-017.GPJ 4-7-14

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

LOG OF BORING NO. 4

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1883+83, 60 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	- slightly silty with a little fine gravel below 54 ft			
	Dense to very dense gray fine to medium sand w/some fine to coarse gravel	50/8"		
	Dense to very dense gray sandy fine gravel			
		50/6"		
	- with occasional silt and silty clay seams at 70 to 86 ft			
		80/9"		
		50/7"		2
100		50/6"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

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

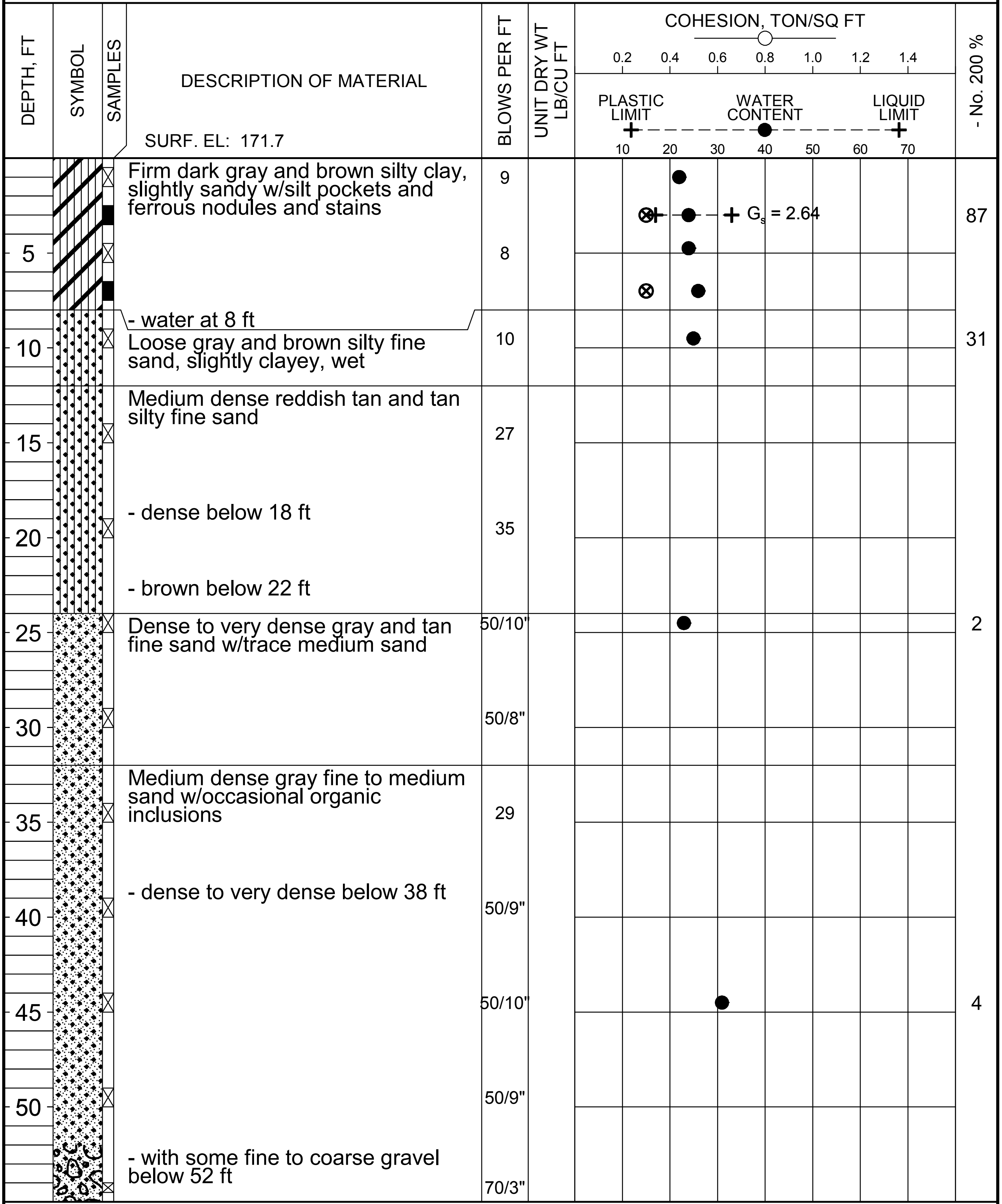
LOG OF BORING NO. 5

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1884+63, 60 ft Rt

SURF. EL: 171.7



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm dark gray and brown silty clay, slightly sandy w/silt pockets and ferrous nodules and stains	9		
			87
	8		
- water at 8 ft			
Loose gray and brown silty fine sand, slightly clayey, wet	10		31
Medium dense reddish tan and tan silty fine sand	27		
- dense below 18 ft	35		
- brown below 22 ft			
Dense to very dense gray and tan fine sand w/trace medium sand	50/10"		2
	50/8"		
Medium dense gray fine to medium sand w/occasional organic inclusions	29		
- dense to very dense below 38 ft	50/9"		
	50/10"		4
	50/9"		
- with some fine to coarse gravel below 52 ft	70/3"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-25-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/25/2014

13-017

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

LOG OF BORING NO. 5

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

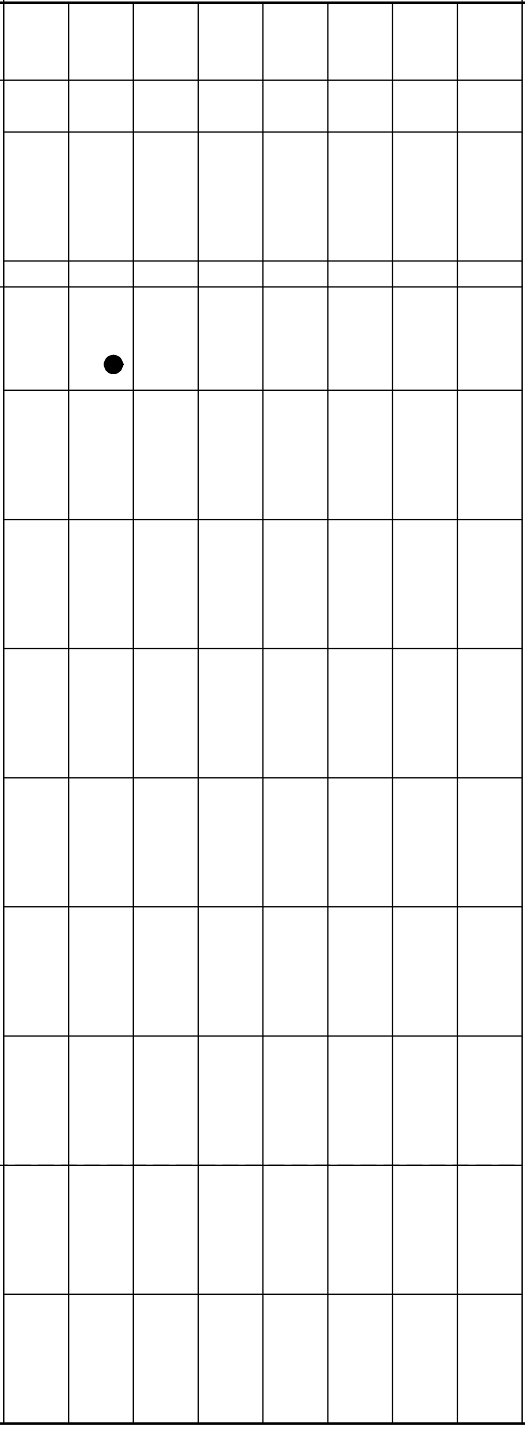
TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1884+63, 60 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Dense to very dense gray sandy fine to coarse gravel	50/6"		
60				
65				
	Dense to very dense gray fine to coarse sand, slightly silty w/some fine to coarse gravel	50/11"		8
70				
75				
		70/9"		
80				
85				
		50/8"		
90	- with more gravel below 90 ft			
95		70/6"		
100				
105				

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

PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70



COMPLETION DEPTH: 100.0 ft
DATE: 2-25-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/25/2014

LGBNEW 13-017.GPJ 4-7-14

13-017

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

LOG OF BORING NO. 6

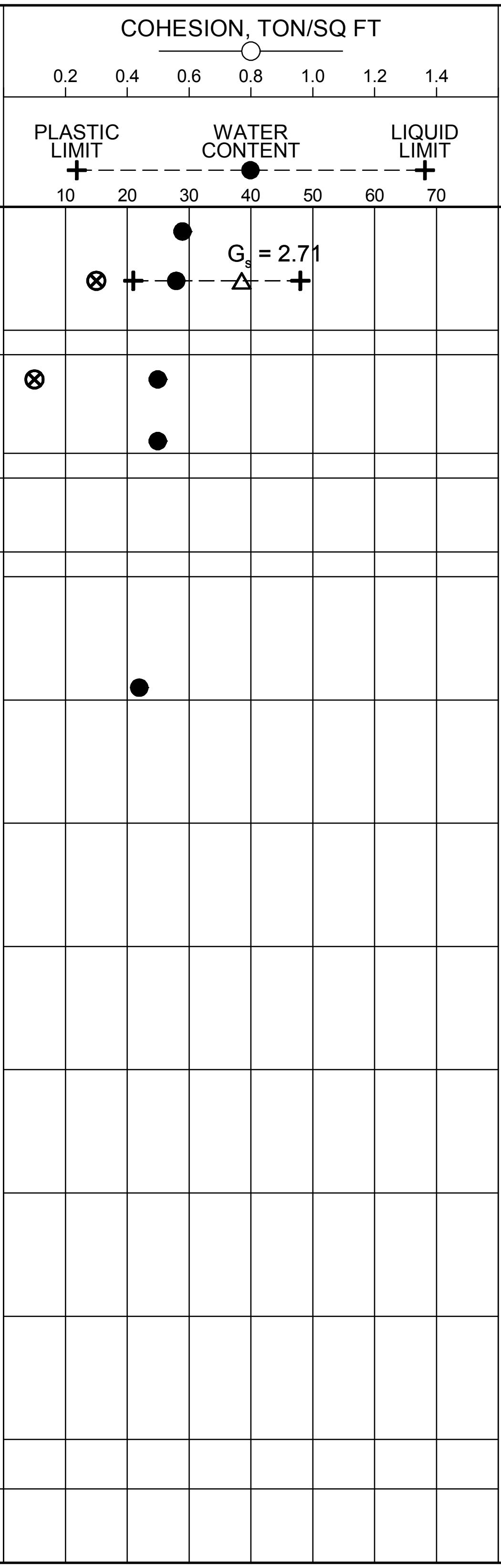
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1886+11, 60 ft Lt

SURF. EL: 171.0

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft gray and brown silty clay w/ferrous stains and nodules	5		
	- firm at 2 - 4 ft		95	99
5	- stiff, moist below 4 ft	12		
10	Soft brown clayey silt w/occasional clay pockets from 4 - 5 ft - firm below 8 ft	9		
	Medium dense brown silty fine sand w/occasional clay pockets and seams			
15	Medium dense gray fine sand, slightly silty	14		
20		25		5
25	- dense 25 to 34 ft	37		
30		44		
35	- dense to very dense below 34 ft	50/11"		
40	- with occasional organic inclusions below 40 ft	50/10"		
45		72		
50		50/10"		
	Dense to very dense gray fine to medium sand, silty w/some fine gravel and occasional clay pockets	50/9"		



COMPLETION DEPTH: 100.0 ft
DATE: 3-11-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/11/2014

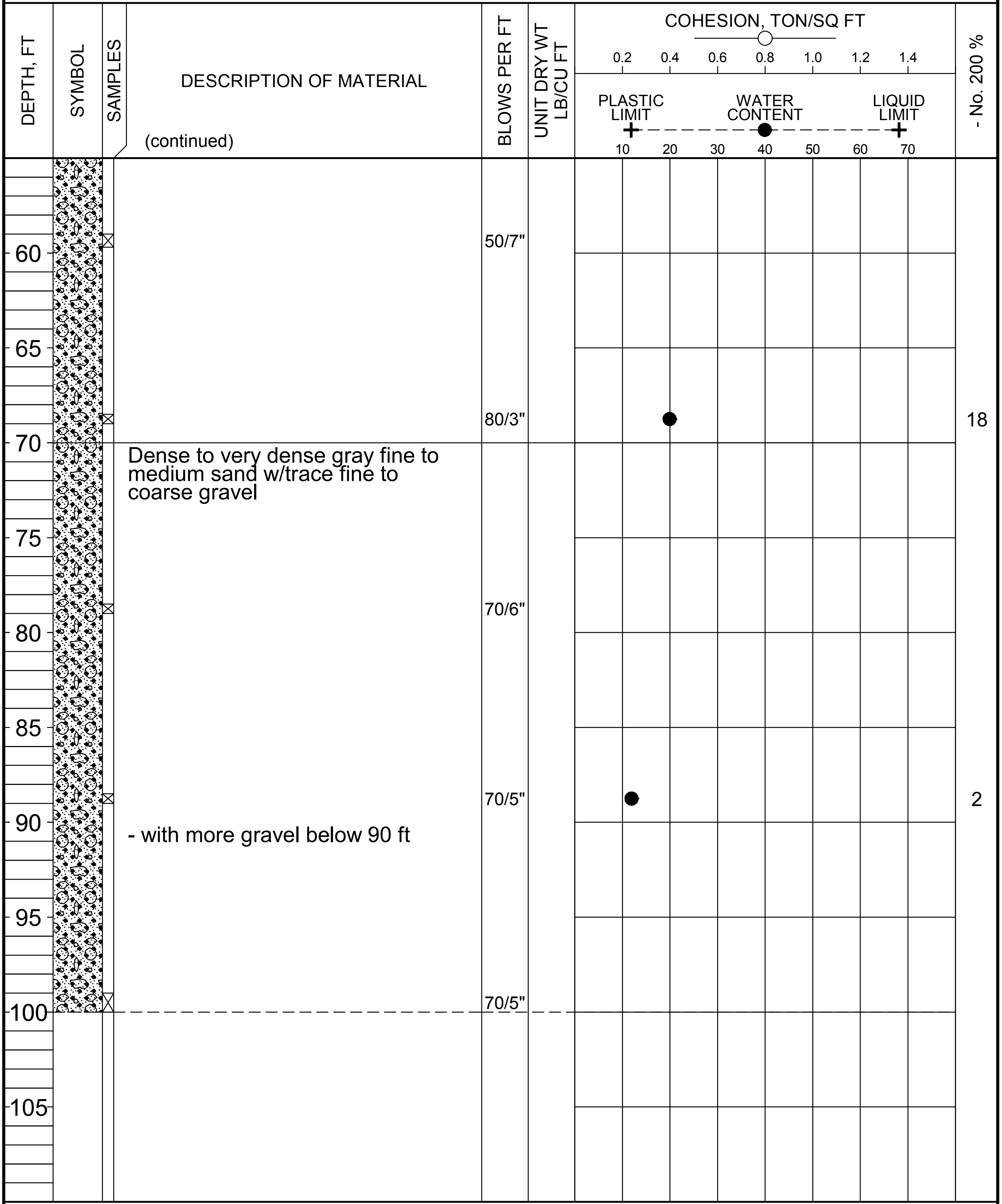
LGBNEW 13-017.GPJ 4-7-14

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

LOG OF BORING NO. 6

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1886+11, 60 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
60		50/7"		
70		80/3"		18
70	Dense to very dense gray fine to medium sand w/trace fine to coarse gravel			
80		70/6"		
90		70/5"		2
90	- with more gravel below 90 ft			
100		70/5"		

COMPLETION DEPTH: 100.0 ft
DATE: 3-11-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/11/2014

13-017

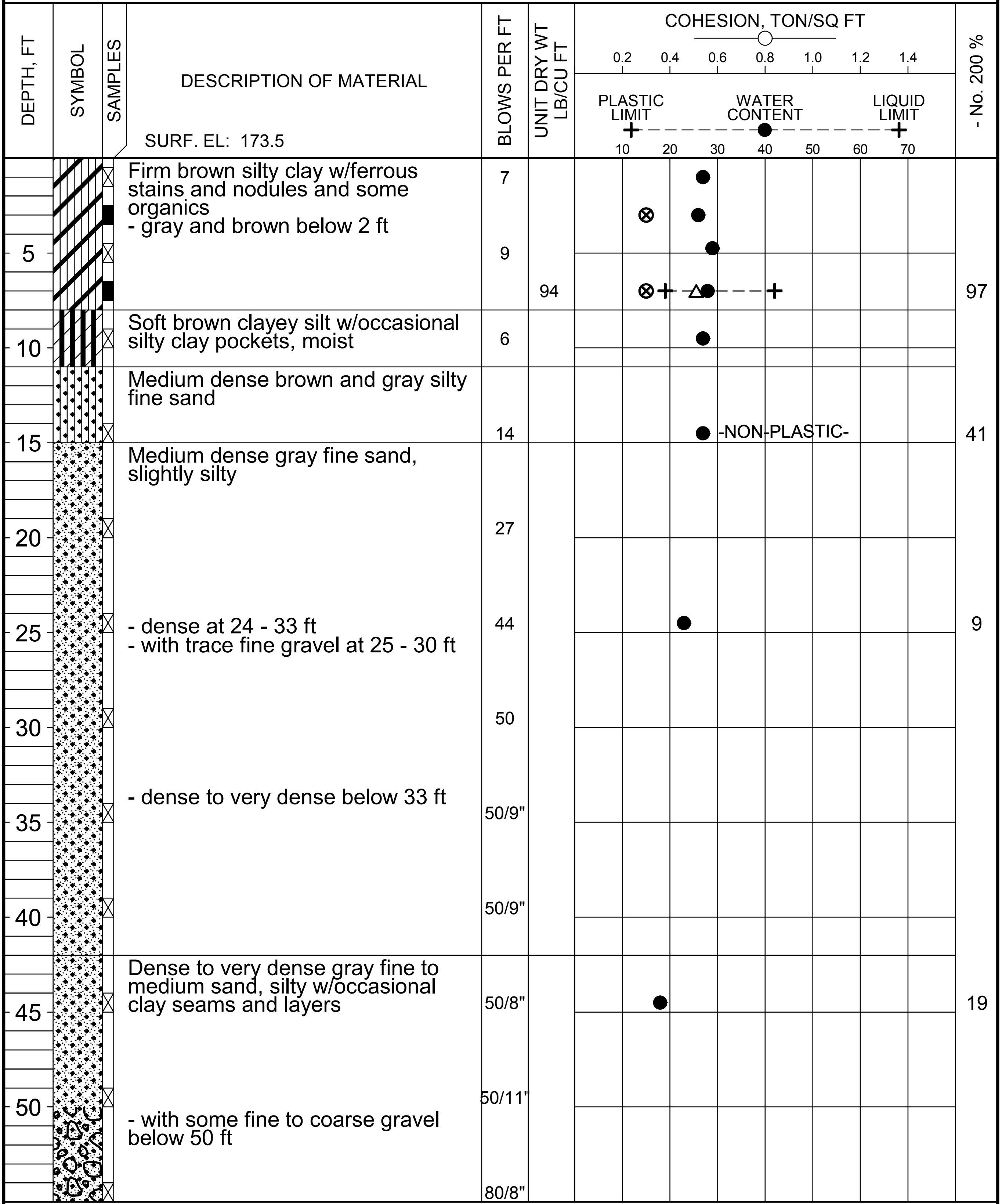
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 7

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1887+59, 60 ft Rt



SURF. EL: 173.5

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT		- No. 200 %
Firm brown silty clay w/ferrous stains and nodules and some organics - gray and brown below 2 ft	7			
	9			
		94		97
Soft brown clayey silt w/occasional silty clay pockets, moist	6			
Medium dense brown and gray silty fine sand	14		-NON-PLASTIC-	41
Medium dense gray fine sand, slightly silty	27			
- dense at 24 - 33 ft - with trace fine gravel at 25 - 30 ft	44			9
	50			
- dense to very dense below 33 ft	50/9"			
	50/9"			
Dense to very dense gray fine to medium sand, silty w/occasional clay seams and layers	50/8"			19
	50/11"			
- with some fine to coarse gravel below 50 ft	80/8"			

COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/12/2014

LGBNEW 13-017.GPJ 4-7-14

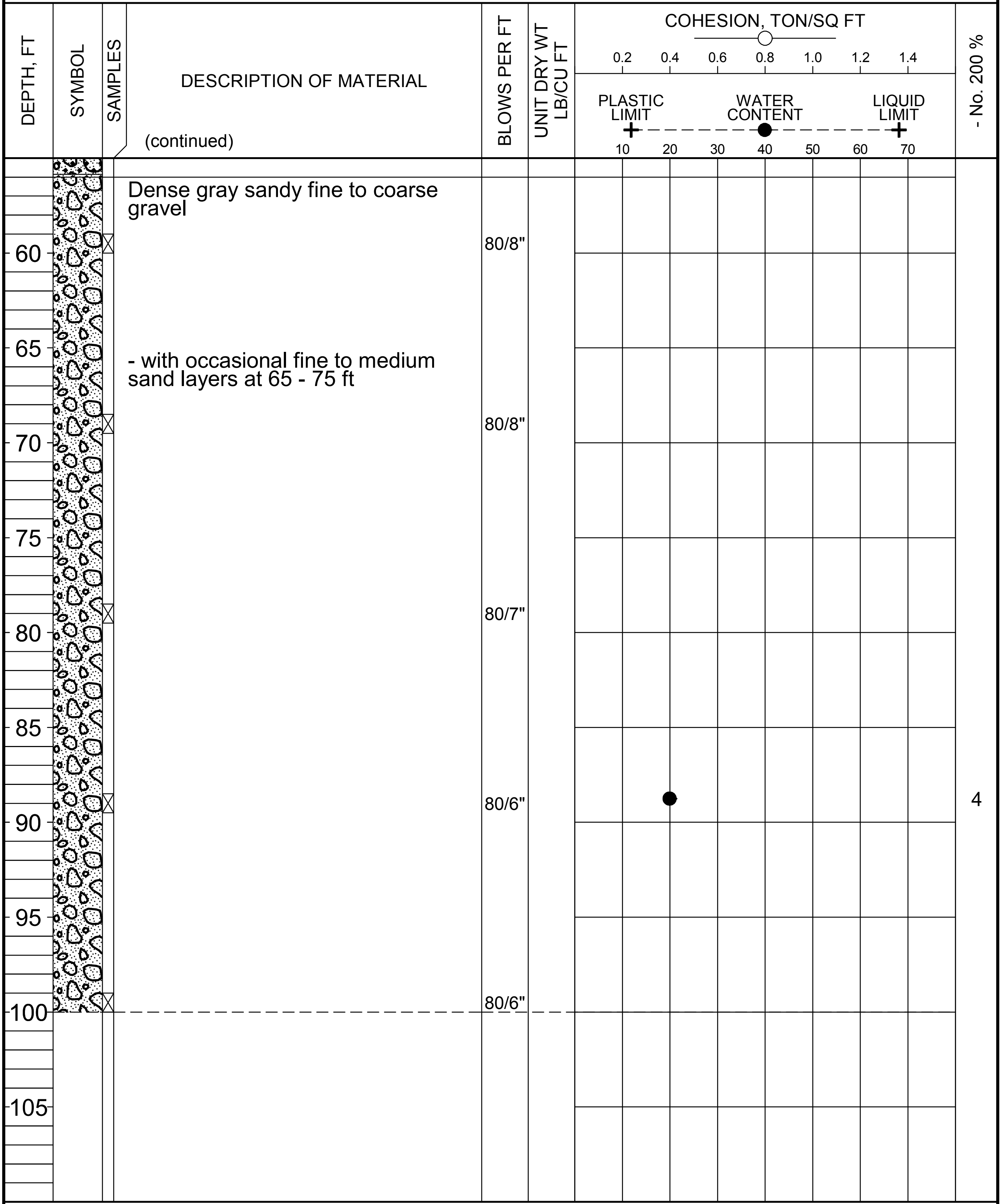
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 7

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1887+59, 60 ft Rt



Dense gray sandy fine to coarse gravel

- with occasional fine to medium sand layers at 65 - 75 ft

BLOWS PER FT	- No. 200 %
80/8"	
80/8"	
80/7"	
80/6"	4
80/6"	

COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/12/2014

13-017

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

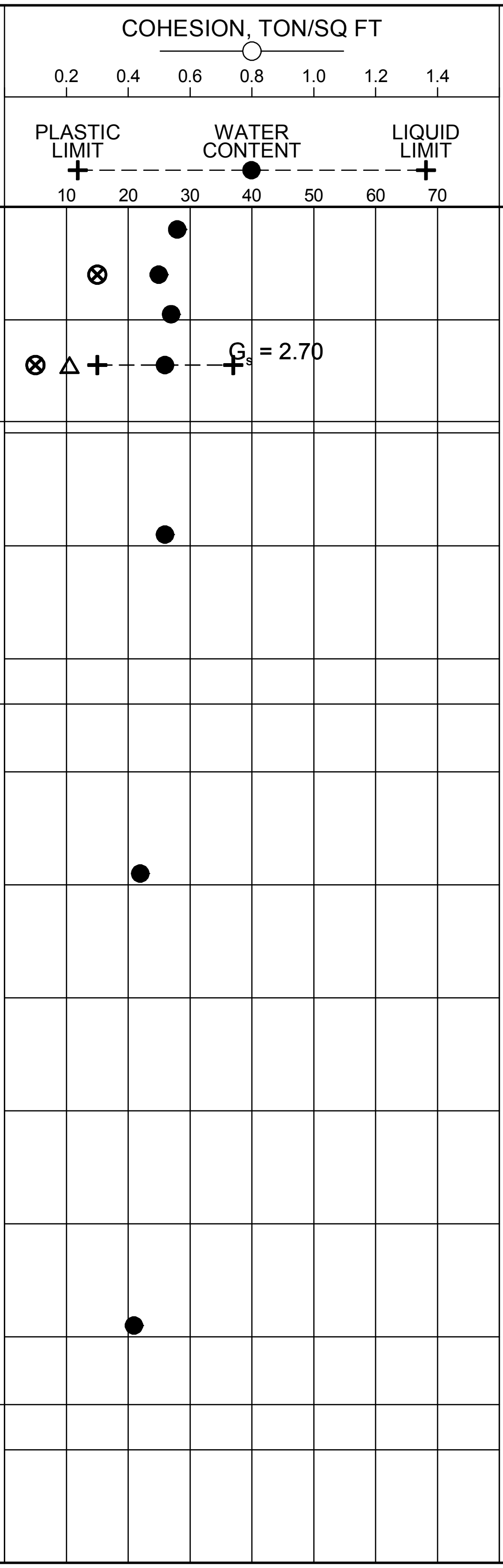
LOG OF BORING NO. 8

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1888+79, 60 ft Lt

SURF. EL: 174.4

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm to stiff gray and brown silty clay, sandy w/occasional silt pockets and ferrous nodules and stains	10		
5		14		
	- soft below 6 ft			81
	- water at 8 ft			
10	Medium dense brown silty fine sand	16		
	- brown and tan below 13 ft			
15	- loose at 13 - 18 ft	10		29
	- medium dense below 18 ft			
20		21		
	Dense gray fine sand, slightly silty			
25		52		
	- dense to very dense at 28 to 43 ft			
30		50/11"		5
35	- with organic inclusions below 34 ft	50/8"		
40		50/11"		
	- dense with trace fine gravel below 43 ft			
45		50		
50		49		12
55	Dense to very dense gray fine to medium sand w/trace fine to coarse gravel	50/10"		
		50/9"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

LGBNEW 13-017.GPJ 4-7-14

13-017

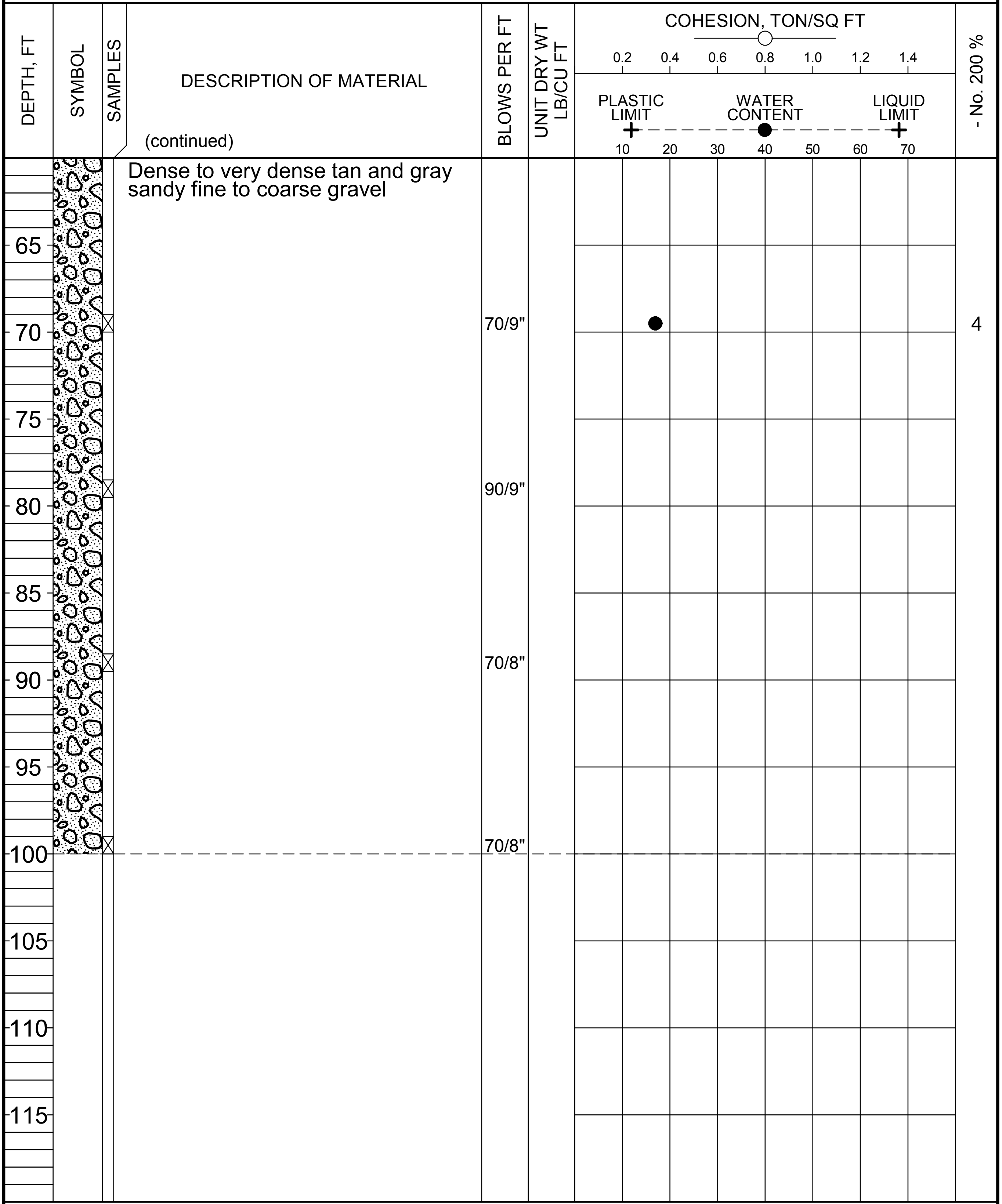
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 8

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1888+79, 60 ft Lt



Dense to very dense tan and gray sandy fine to coarse gravel

DEPTH, FT	BLOWS PER FT	- No. 200 %
70	70/9"	4
79	90/9"	
89	70/8"	
100	70/8"	

COMPLETION DEPTH: 100.0 ft
DATE: 2-26-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/26/2014

LGBNEW 13-017.GPJ 4-7-14

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

LOG OF BORING NO. 9

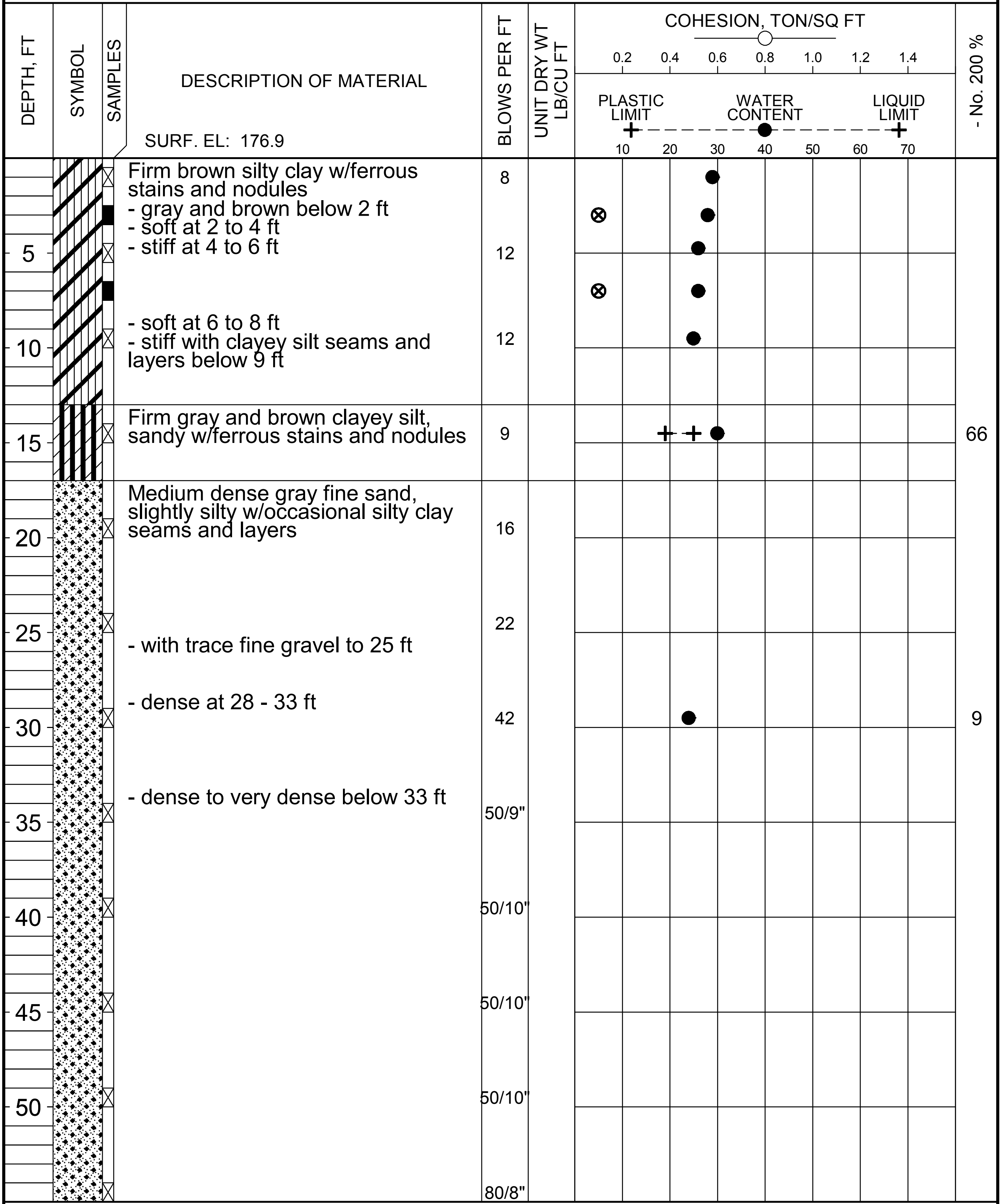
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1889+99, 60 ft Rt

SURF. EL: 176.9

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm brown silty clay w/ferrous stains and nodules	8		
- gray and brown below 2 ft - soft at 2 to 4 ft			
- stiff at 4 to 6 ft	12		
- soft at 6 to 8 ft - stiff with clayey silt seams and layers below 9 ft	12		
Firm gray and brown clayey silt, sandy w/ferrous stains and nodules	9		66
Medium dense gray fine sand, slightly silty w/occasional silty clay seams and layers	16		
- with trace fine gravel to 25 ft	22		
- dense at 28 - 33 ft	42		9
- dense to very dense below 33 ft	50/9"		
	50/10"		
	50/10"		
	50/10"		
	80/8"		



COMPLETION DEPTH: 100.0 ft
DATE: 3-10-14

DEPTH TO WATER
IN BORING: 9 ft
DATE: 3/10/2014

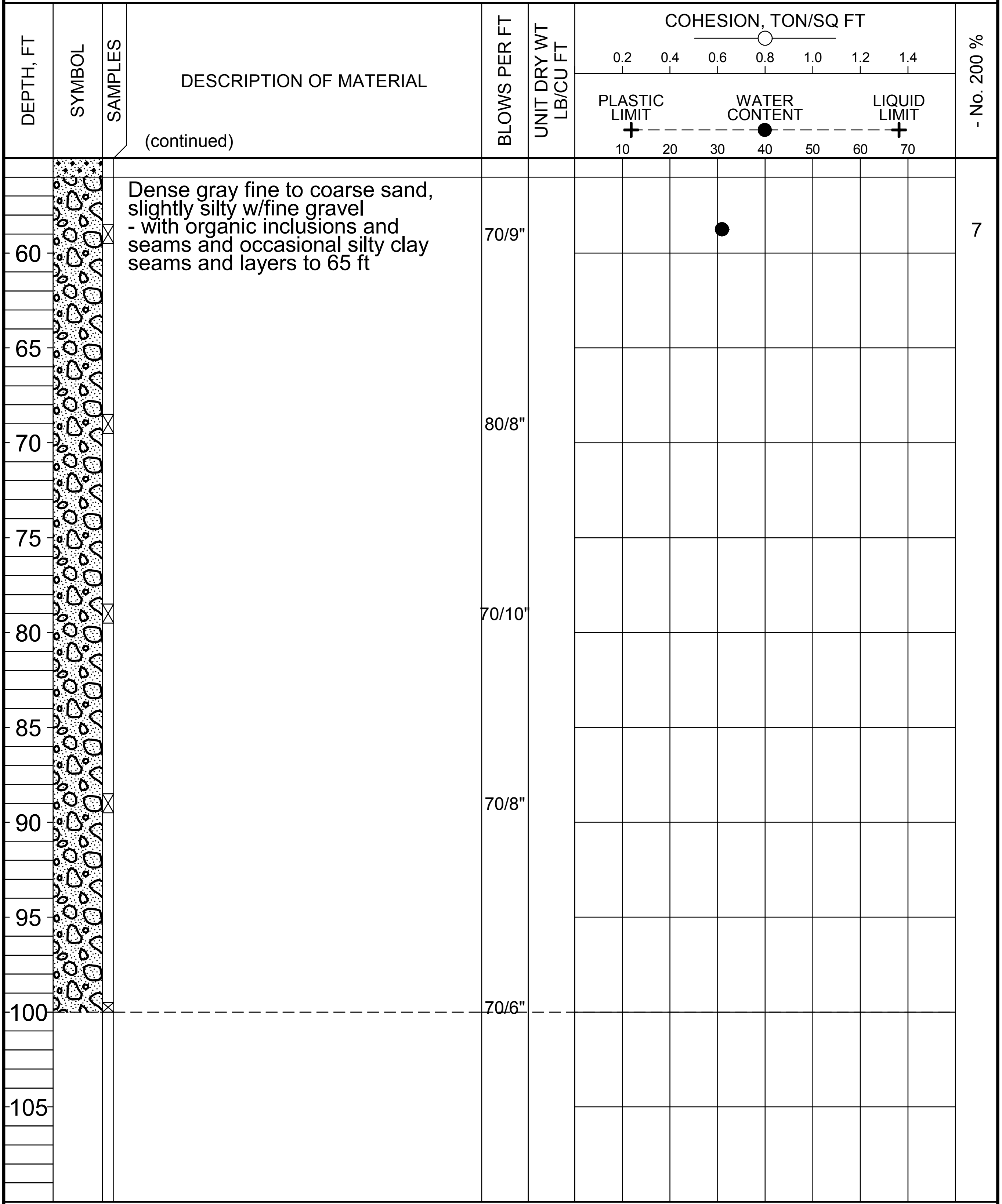
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 9

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1889+99, 60 ft Rt



Dense gray fine to coarse sand, slightly silty w/fine gravel
- with organic inclusions and seams and occasional silty clay seams and layers to 65 ft

70/9"

7

80/8"

70/10"

70/8"

70/6"

COMPLETION DEPTH: 100.0 ft
DATE: 3-10-14

DEPTH TO WATER
IN BORING: 9 ft

DATE: 3/10/2014

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

LOG OF BORING NO. 10

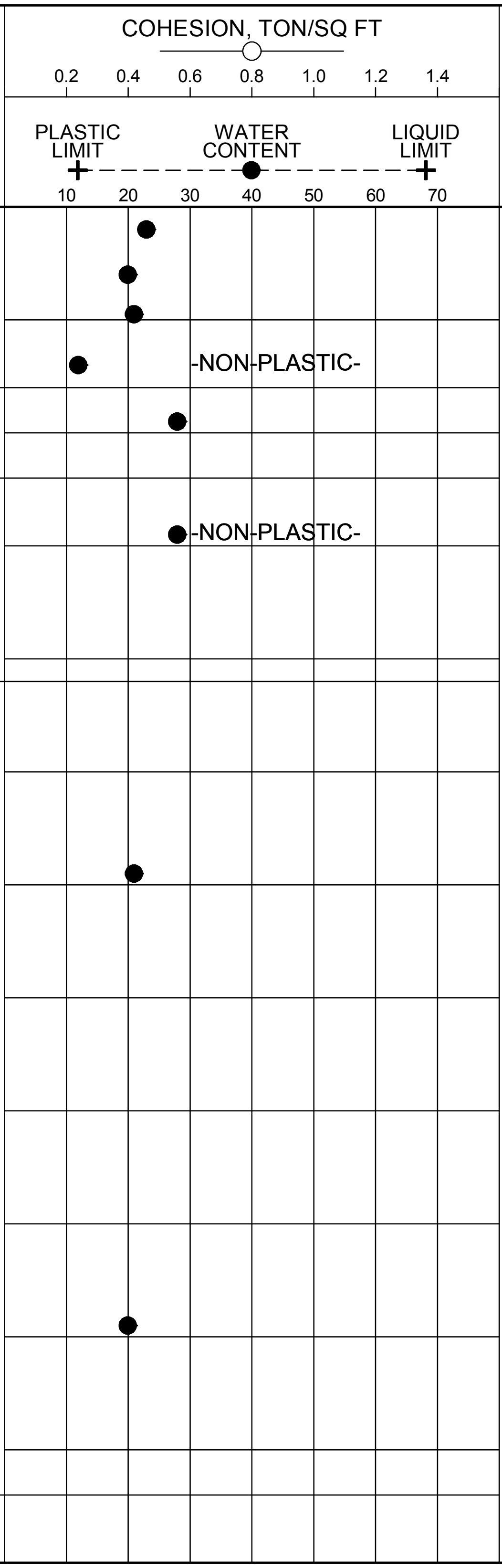
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1891+47, 60 ft Lt

SURF. EL: 177.2

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
0	Loose brown silty fine sand w/silty clay seams and layers	9		
		10		
5		9		
		10		20
8	Stiff gray and brown silty clay w/ferrous stains and nodules	13		
12	Loose brown silty fine sand	6		38
		5		
21	Medium dense gray fine sand w/a little medium sand	20		
	- dense at 28 to 33 ft	45		4
	-dense to very dense at 33 to 42 ft	50/10"		
		50/10"		
	- with occasional silty clay pockets below 40 ft - dense at 42 - 45 ft	40		
	- dense to very dense with trace coarse sand and fine gravel below 45 ft	50/8"		4
		50		
57	Dense gray fine to medium sand w/some fine to coarse gravel	80/8"		



COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 3/12/2014

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

LOG OF BORING NO. 10

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1891+47, 60 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
65							
70		☒		80/8"			
			Dense gray sandy fine to coarse gravel				
75							
80		☒		80/7"			
85							
90		☒		80/7"			
95							
100		☒		80/7"			
105							
110							
115							

COMPLETION DEPTH: 100.0 ft
DATE: 3-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 3/12/2014

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

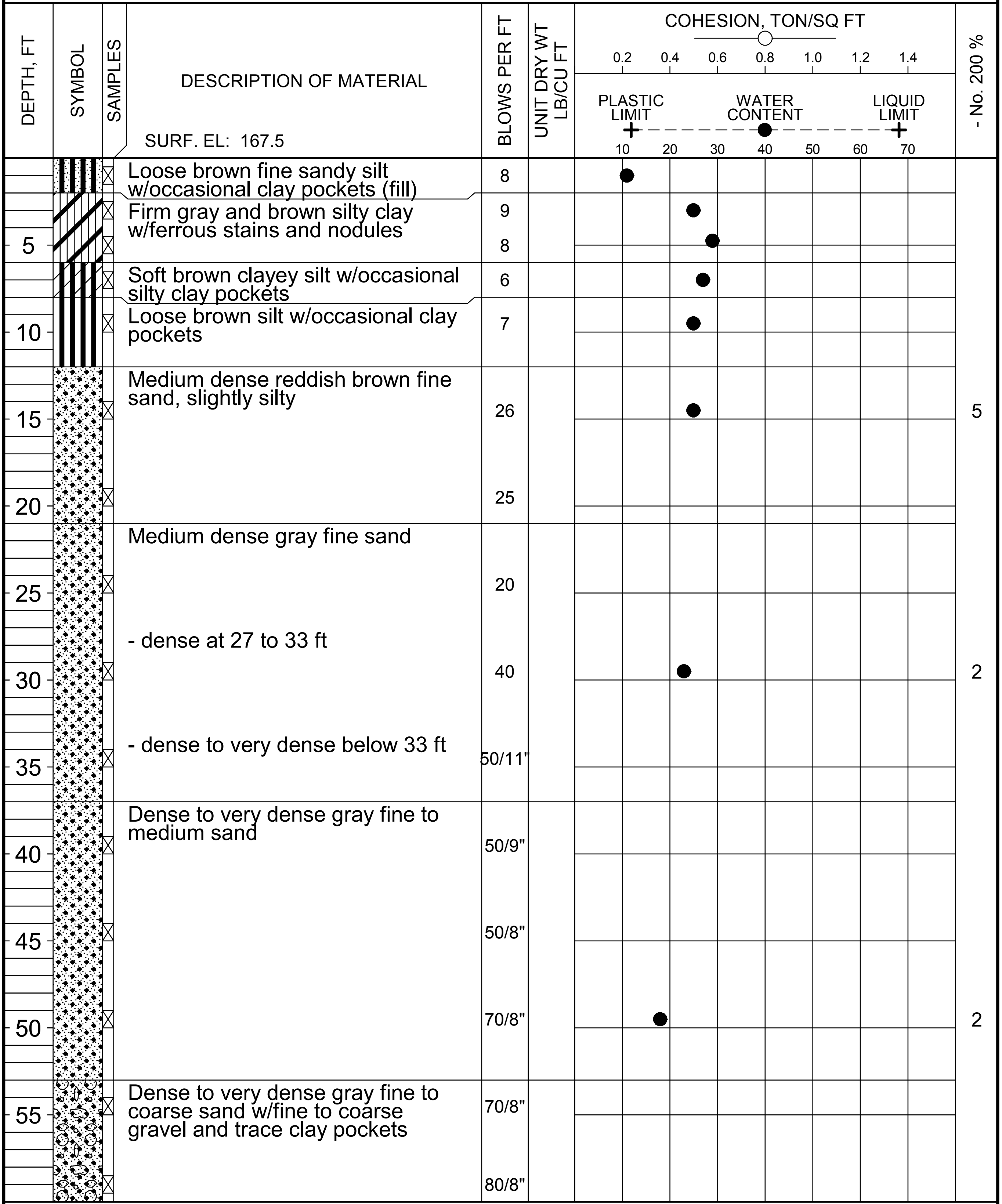
LOG OF BORING NO. 11

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to10 ft /Wash

LOCATION: Sta 1892+95, 80 ft Rt

SURF. EL: 167.5



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Loose brown fine sandy silt w/occasional clay pockets (fill)	8		
Firm gray and brown silty clay w/ferrous stains and nodules	9 8		
Soft brown clayey silt w/occasional silty clay pockets	6		
Loose brown silt w/occasional clay pockets	7		
Medium dense reddish brown fine sand, slightly silty	26 25		5
Medium dense gray fine sand - dense at 27 to 33 ft - dense to very dense below 33 ft	20 40 50/11"		 2
Dense to very dense gray fine to medium sand	50/9" 50/8" 70/8"		 2
Dense to very dense gray fine to coarse sand w/fine to coarse gravel and trace clay pockets	70/8" 80/8"		

COMPLETION DEPTH: 100.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/13/2014

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

LOG OF BORING NO. 11

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to10 ft /Wash

LOCATION: Sta 1892+95, 80 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT ○ 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT + 10 20 30 40 50 60 70	- No. 200 %
65							
70		☒		80/7"			
75							
80		☒		80/8"			
85							
90		☒		80/10"			
95			Dense gray sandy fine to coarse gravel				
100		☒		80/6"			
105							
110							
115							

COMPLETION DEPTH: 100.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 3/13/2014

13-017

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

LOG OF BORING NO. 12

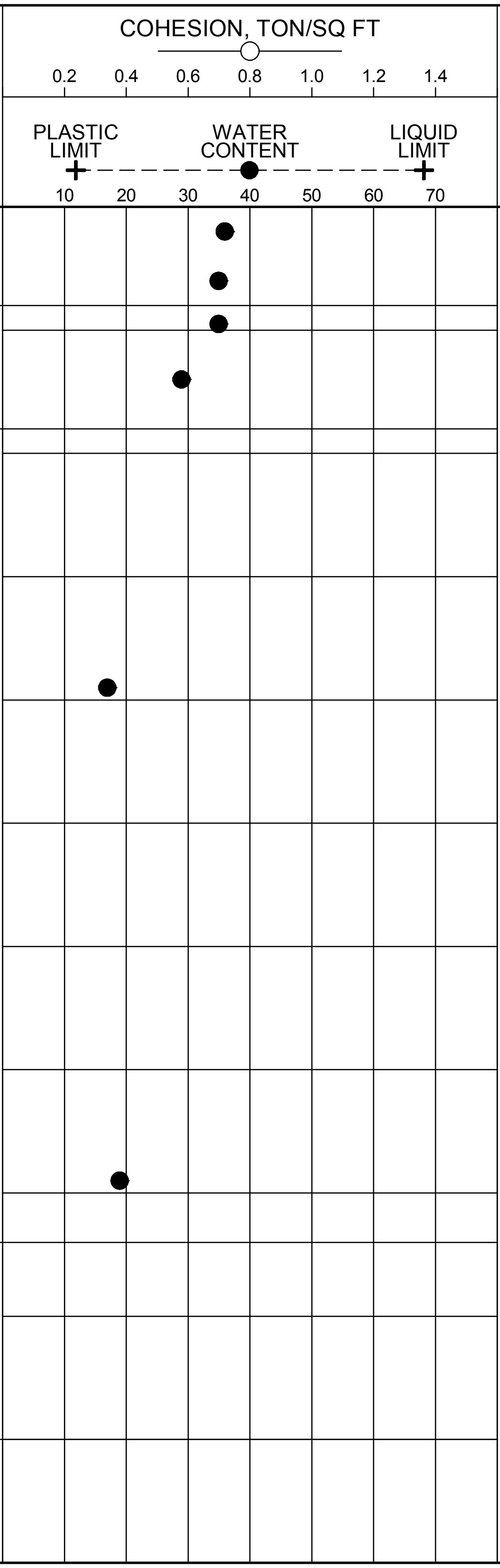
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1894+15, 60 ft Lt

SURF. EL: 168 ±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very soft gray and brown silty clay w/occasional silt pockets and seams	2		
		2		
5	Very soft brown clayey silt w/silty clay seams	3		
		3		
10	Medium dense brown and gray fine sand - gray below 13 ft	18		
15		18		
20		29		2
25		29		
30	- with occasional organic inclusions to 35 ft - dense at 28 to 34 ft	46		
35	- dense to very dense with some medium sand below 34 ft	55		
40		50/11"		3
45	Dense gray silty fine sand w/trace medium sand and occasional silty clay seams	32		
50	- with wood fragments at 50 ft	45		16
	Dense to very dense gray sandy fine gravel w/occasional fine to medium sand seams and layers	50/11"		



COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

DATE: 3/13/2014

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

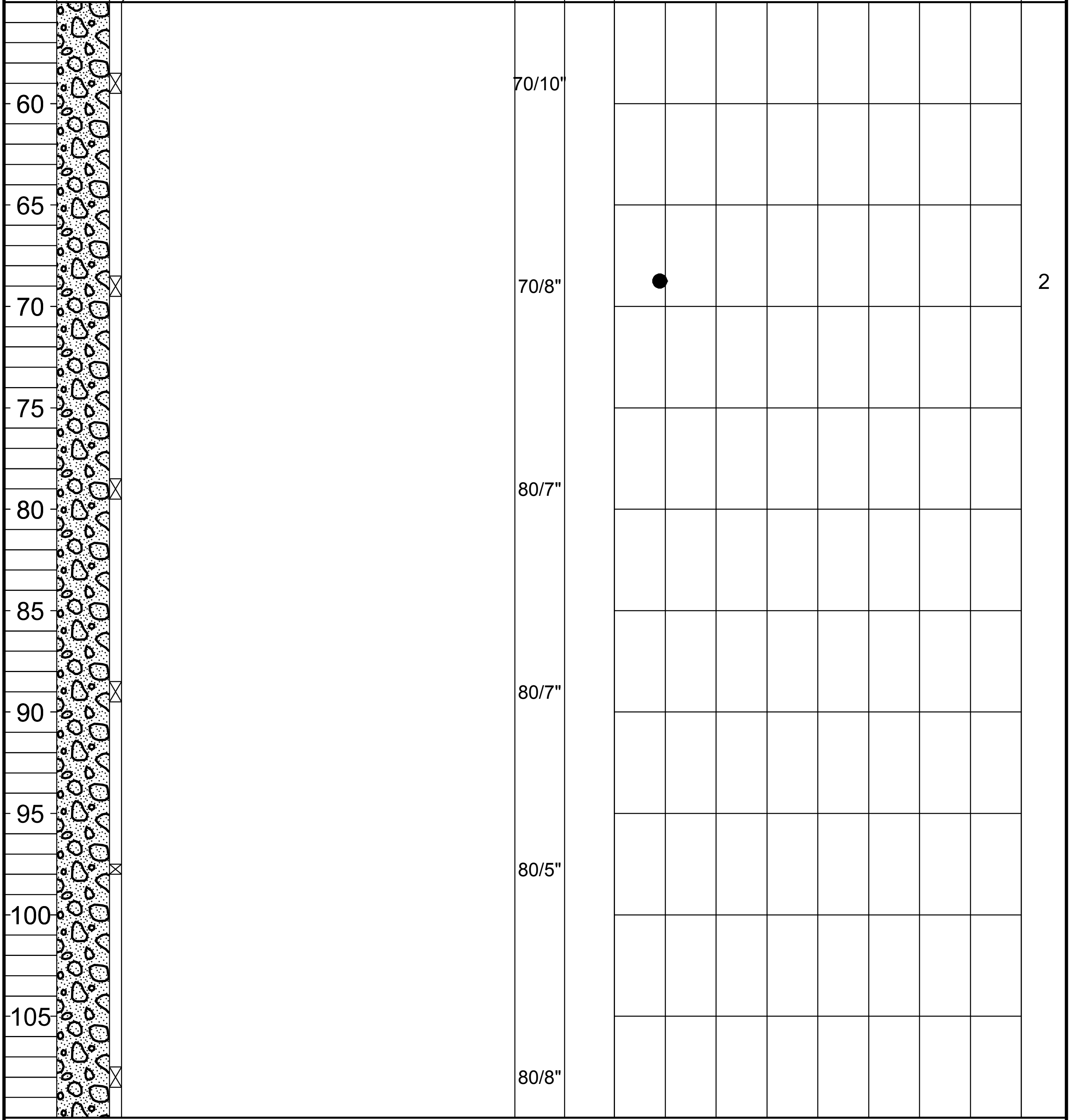
LOG OF BORING NO. 12

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1894+15, 60 ft Lt

DEPTH, FT | SYMBOL | SAMPLES | DESCRIPTION OF MATERIAL (continued) | BLOWS PER FT | UNIT DRY WT LB/CU FT | COHESION, TON/SQ FT / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT | - No. 200 %



BLOWS PER FT	- No. 200 %
70/10"	
70/8"	2
80/7"	
80/7"	
80/5"	
80/8"	

COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

DATE: 3/13/2014

13-017

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

LOG OF BORING NO. 12

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1894+15, 60 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
			- borehole caving at 110 ft				
115							
120							
125							
130							
135							
140							
145							
150							
155							
160							

COMPLETION DEPTH: 130.0 ft
DATE: 3-13-14

DEPTH TO WATER
IN BORING: 4 ft

DATE: 3/13/2014

LGBNEW 13-017.GPJ 6-13-14

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

LOG OF BORING NO. 13

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+90, 60 ft Rt

SURF. EL: 137±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT	- No. 200 %
	Medium dense brown and gray silty fine sand w/a little medium sand	15			
5		17		● -NON-PLASTIC- G_s = 2.66	31
		15			
10		16			
15	- dense at 15 to 20 ft	30			
20	- dense to very dense with some fine to coarse gravel below 20 ft	75			
25					
30		63			
35		100/11"			
40	Dense to very dense brownish gray fine to medium sand w/some fine to coarse gravel	75			

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

WATER CONTENT scale: 10 20 30 40 50 60 70

COMPLETION DEPTH: 78.0 ft
DATE: 2-28-14

DEPTH TO WATER
IN BORING: NA

DATE: 2/28/2014

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

LOG OF BORING NO. 13

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+90, 60 ft Rt

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	- No. 200 %
		☒		73			
50							
		☒		75			
55							
		☒		100/6"			
60			Dense to very dense gray fine to coarse sand w/some fine to coarse gravel				
65							
		☒		100/5"			
70							
75			- brown and gray below 75 ft				
		☒	- 100% water loss at 78 ft	100/5"			
80			NOTE 1: Water depth: 27 ft. NOTE 2: Set 30 ft casing to 3 ft below mudline. NOTE 3: Boring caving. Abandoned at 78 ft				
85							

COMPLETION DEPTH: 78.0 ft
DATE: 2-28-14

DEPTH TO WATER
IN BORING: NA

DATE: 2/28/2014

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

LOG OF BORING NO. 13A

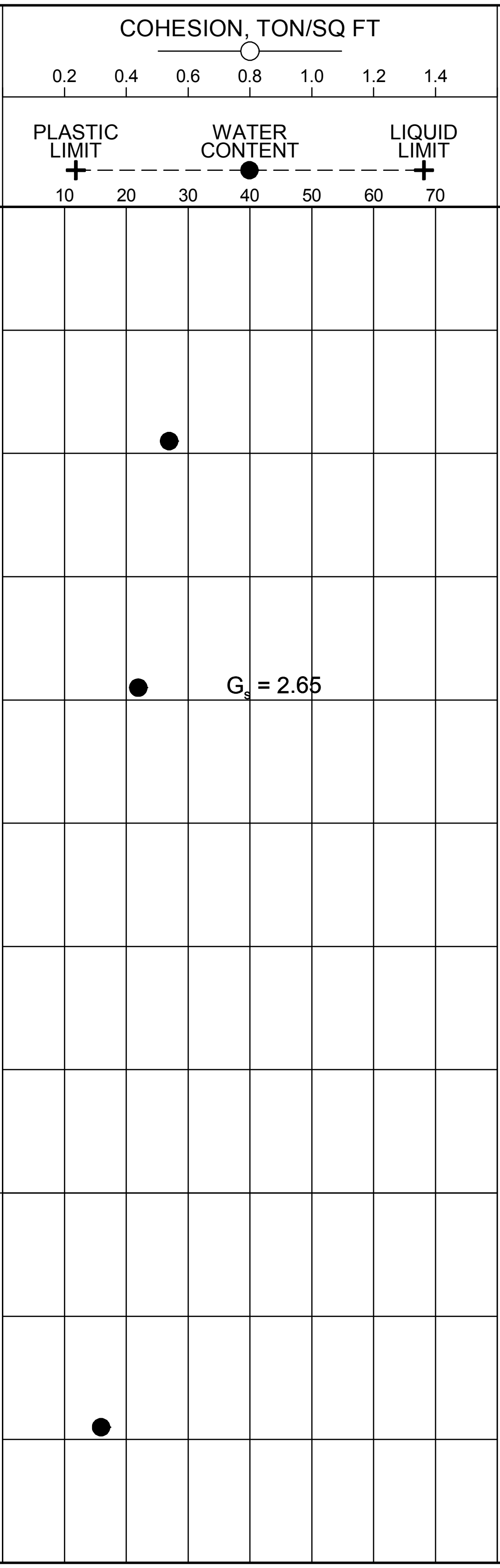
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt

SURF. EL: 137±

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Dense brown and tan fine sand w/a little medium sand	50/8"		
	45		2
	36		4
	50/10"		
- dense to very dense with some gravel below 30 ft			
	100/7"		
Dense to very dense gray fine to coarse sand, slightly silty w/fine gravel			
	100/8"		6
	50/8"		



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

COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

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

LOG OF BORING NO. 13A

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt

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

PLASTIC LIMIT + - - - WATER CONTENT ● - - - LIQUID LIMIT +: 10 20 30 40 50 60 70

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT	- No. 200 %
60	- with trace coarse gravel below 60 ft	100/7"			
65		100/5"		17	5
70					
75	Dense to very dense gray fine to coarse sand w/fine gravel				
80	- more gravel below 80 ft	105/7"			
	- 100% water loss at 82 ft, borehole caving				
85					
90		100/7"			
95					
100				20	1
105					

COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

LGBNEW 13-017.GPJ 4-8-14

13-017

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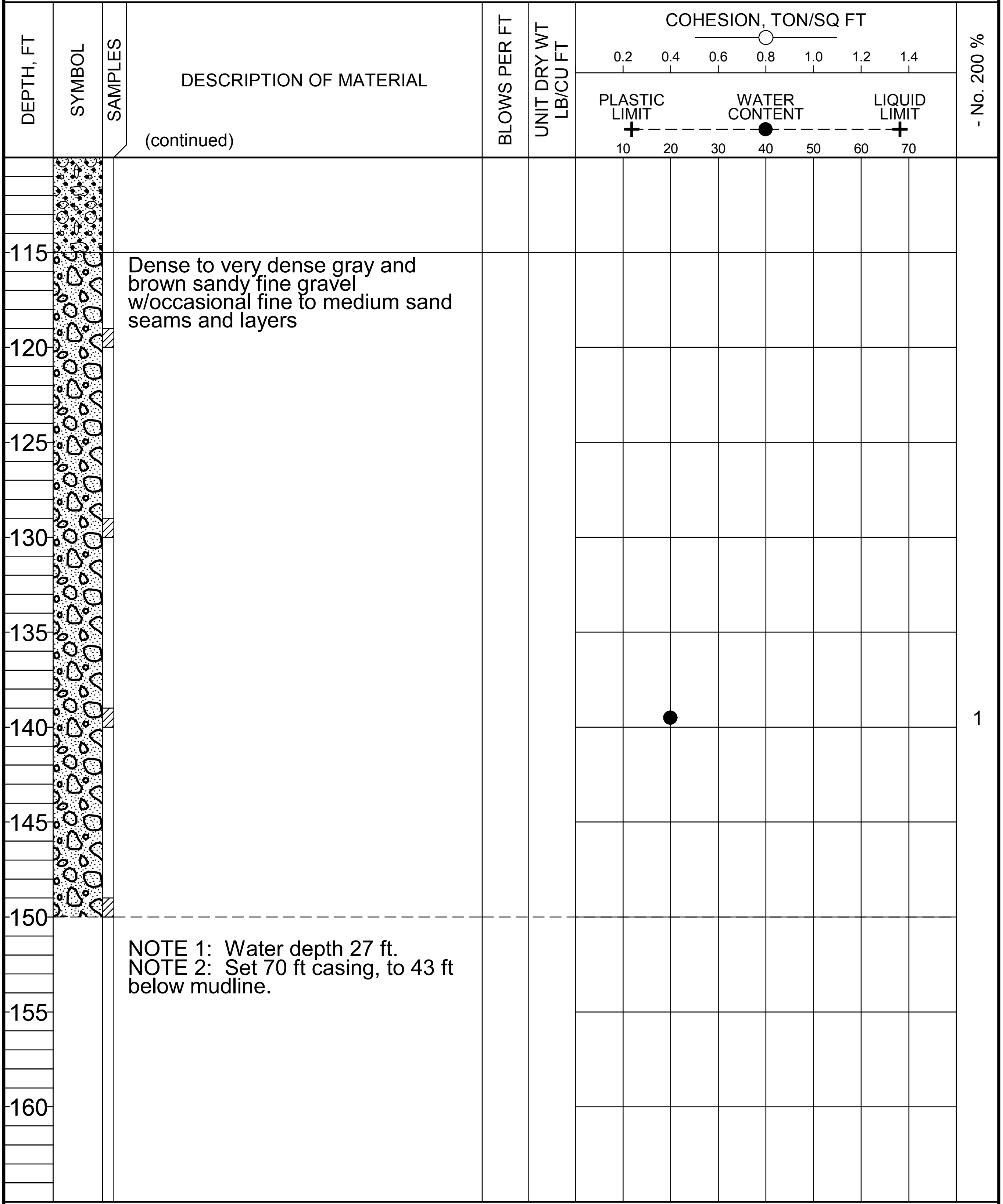
Consulting Engineers

LOG OF BORING NO. 13A

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Wash

LOCATION: Sta 1896+80, 50 ft Rt



DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	WATER CONTENT	- No. 200 %
115			Dense to very dense gray and brown sandy fine gravel w/occasional fine to medium sand seams and layers				
140						20	1
150			NOTE 1: Water depth 27 ft. NOTE 2: Set 70 ft casing, to 43 ft below mudline.				

COMPLETION DEPTH: 150.0 ft
DATE: 3-1-14

DEPTH TO WATER
IN BORING: NA

DATE: 3/7/2014

13-017

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

LOG OF BORING NO. 14

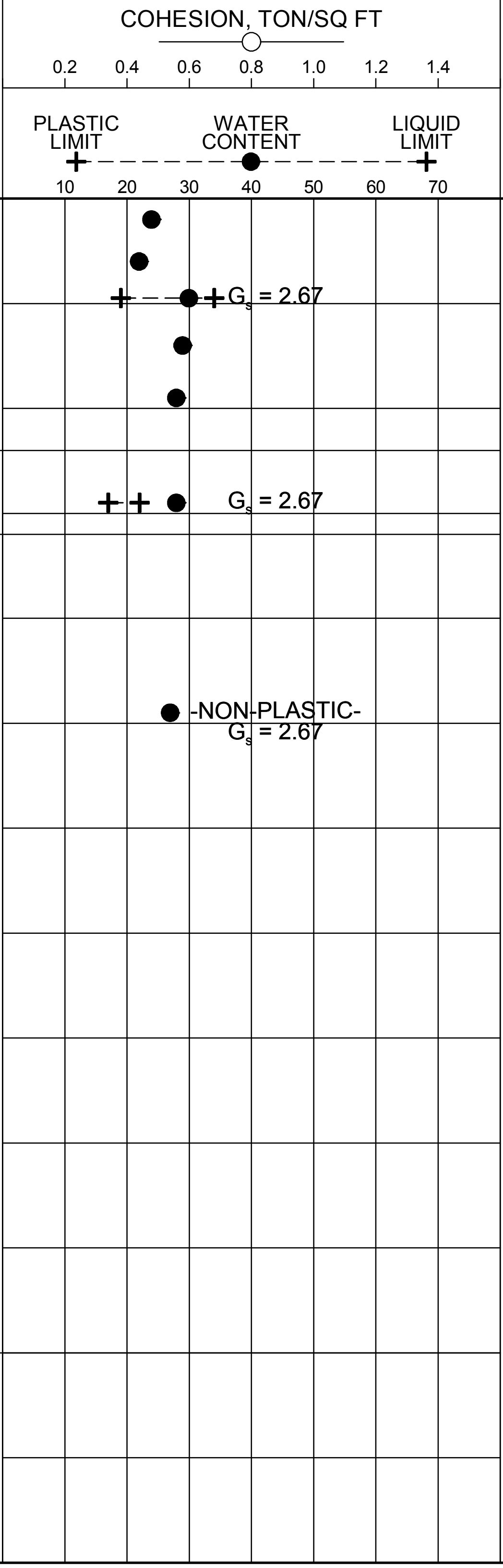
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1900+10, 60 ft Lt

SURF. EL: 176.6

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
		☒	Very soft brown silty clay w/organic inclusions, moist	2		
		☒	- soft to firm below 2 ft	7		
5		☒		5		95
		☒		4		
10		☒		8		
			Soft gray clayey silt			
15		☒		6		62
			Loose gray silty fine sand w/occasional silty clay pockets			
20		☒		5		
			- medium dense at 22 to 28 ft			
25		☒		14		36
			- dense below 28 ft			
30		☒		32		
35		☒		36		
40		☒		40		
45		☒		41		24
50		☒		45		
55			Dense brownish gray fine sand, slightly silty w/trace medium sand			
		☒	- with trace fine gravel below 57 ft	50/11"		11
60						



COMPLETION DEPTH: 125.0 ft
DATE: 2-13-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/13/2014

13-017

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

L O G O F B O R I N G N O. 14

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1900+10, 60 ft Lt

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

PLASTIC LIMIT + WATER CONTENT ● LIQUID LIMIT +
10 20 30 40 50 60 70

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
		50/11"		
70	Dense to very dense brown sandy fine gravel			
75				
		50/10"		
80				
85				
		50/6"		
90				
95				
		50/7"		2
100				
105				
		50/6"		
110				
115				
		50/4"		
120				
125				

COMPLETION DEPTH: 125.0 ft
DATE: 2-13-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/13/2014

LGBNEW 13-017.GPJ 4-8-14

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

LOG OF BORING NO. 15

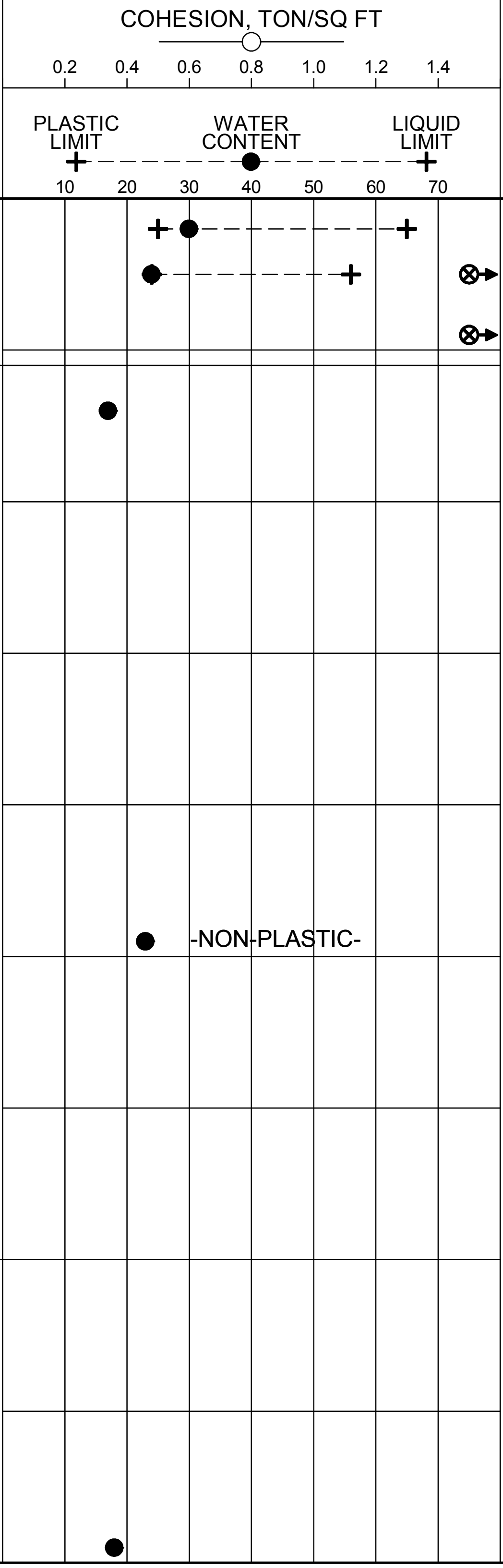
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1916+00, CL

SURF. EL: 180.1

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brown clay	14		99
	- very stiff below 2 ft			94
5	- with fine sandy silt pockets below 4 ft			
	Medium dense brown silty fine sand	23		
10	- loose to medium dense at 8 - 12 ft	10		
	- medium dense at 12 - 18 ft			
15		18		
	- wet at 9 - 18 ft - dense at 18 - 28 ft	36		
20				
25	- with silty clay pockets below 24 ft	42		12
	- dense to very dense at 28 - 32 ft	50/8"		
30				
	- dense, grayish brown, wet below 32 ft			
35		35		
	Dense brown and gray fine to medium sand, slightly silty w/silty clay pockets and trace fine gravel			
40		45		
		38		9



COMPLETION DEPTH: 70.0 ft
DATE: 6-24-13

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 6/24/2013

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

LOG OF BORING NO. 15

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1916+00, CL

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT 0.2 0.4 0.6 0.8 1.0 1.2 1.4 / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT 10 20 30 40 50 60 70	- No. 200 %
50		☒	- dark gray and brown with numerous organic inclusions below 48 ft	34			
55		☒	Dense to very dense gray fine sand, slightly silty - dense, coarser below 55 ft	50/9"			
60		☒		37			6
65		☒	Dense to very dense brown and gray fine to medium sand, slightly silty	50/8"			
70		☒		50/10"			5
75							
80							
85							

COMPLETION DEPTH: 70.0 ft
DATE: 6-24-13

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 6/24/2013

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

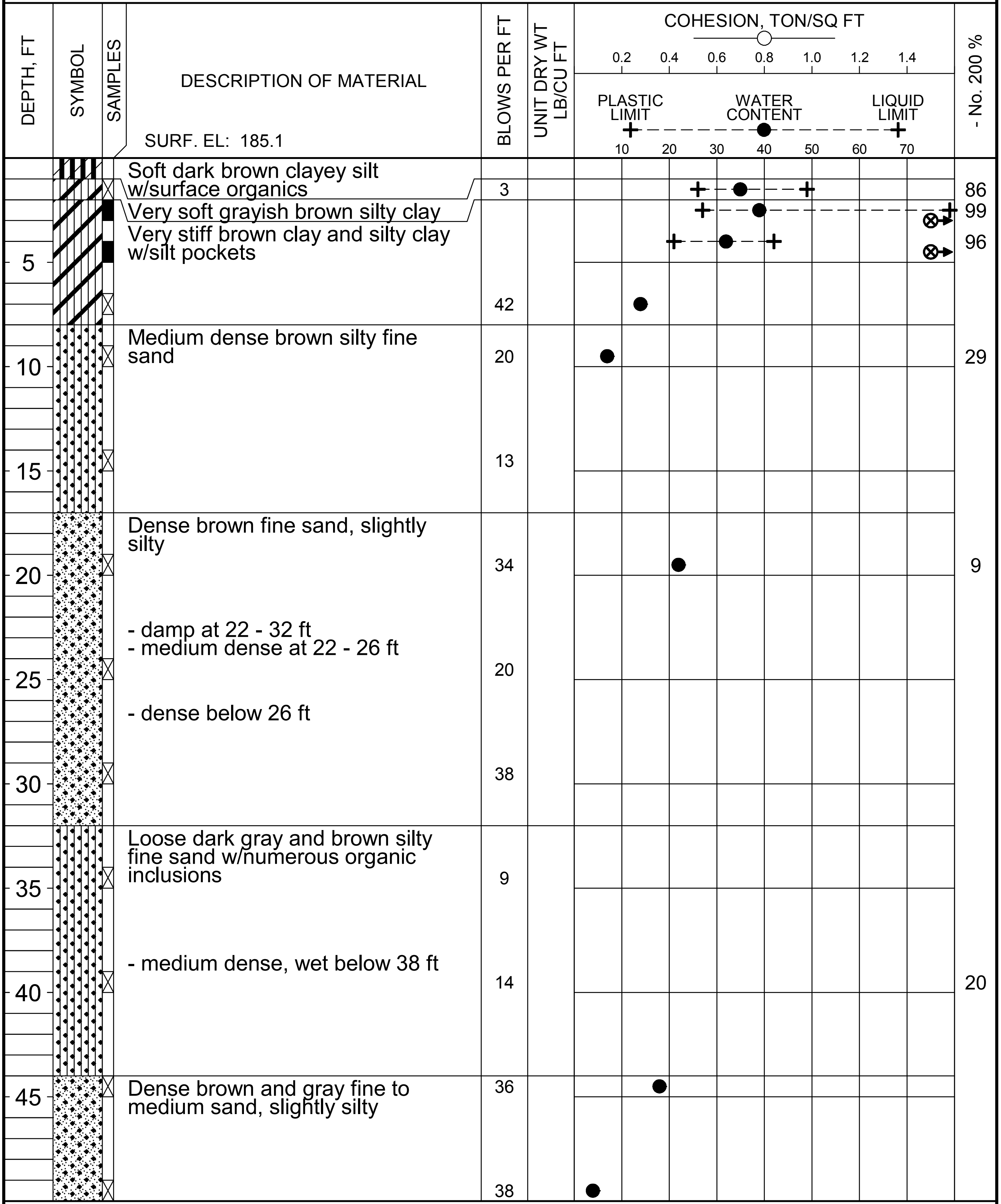
LOG OF BORING NO. 16

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1917+80, CL

SURF. EL: 185.1



DESCRIPTION OF MATERIAL	BLOWS PER FT	- No. 200 %
Soft dark brown clayey silt w/surface organics	3	86
Very soft grayish brown silty clay		99
Very stiff brown clay and silty clay w/silt pockets		96
	42	
Medium dense brown silty fine sand	20	29
	13	
Dense brown fine sand, slightly silty	34	9
- damp at 22 - 32 ft - medium dense at 22 - 26 ft	20	
- dense below 26 ft	38	
Loose dark gray and brown silty fine sand w/numerous organic inclusions	9	
- medium dense, wet below 38 ft	14	20
Dense brown and gray fine to medium sand, slightly silty	36	
	38	

COMPLETION DEPTH: 70.0 ft
DATE: 6-19-13

DEPTH TO WATER
IN BORING: Dry to 15 ft
DATE: 6/19/2013

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

LOG OF BORING NO. 16

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash
LOCATION: Sta 1917+80, CL

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	- No. 200 %
55		30			
	Dense gray and brown silty fine sand, wet				
60		47			13
	- dense to very dense below 63 ft	50/9"			
65	Dense to very dense brown and gray fine to medium sand w/trace fine gravel				
70		50/10"			4

COMPLETION DEPTH: 70.0 ft
DATE: 6-19-13

DEPTH TO WATER
IN BORING: Dry to 15 ft
DATE: 6/19/2013

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

LOG OF BORING NO. 17

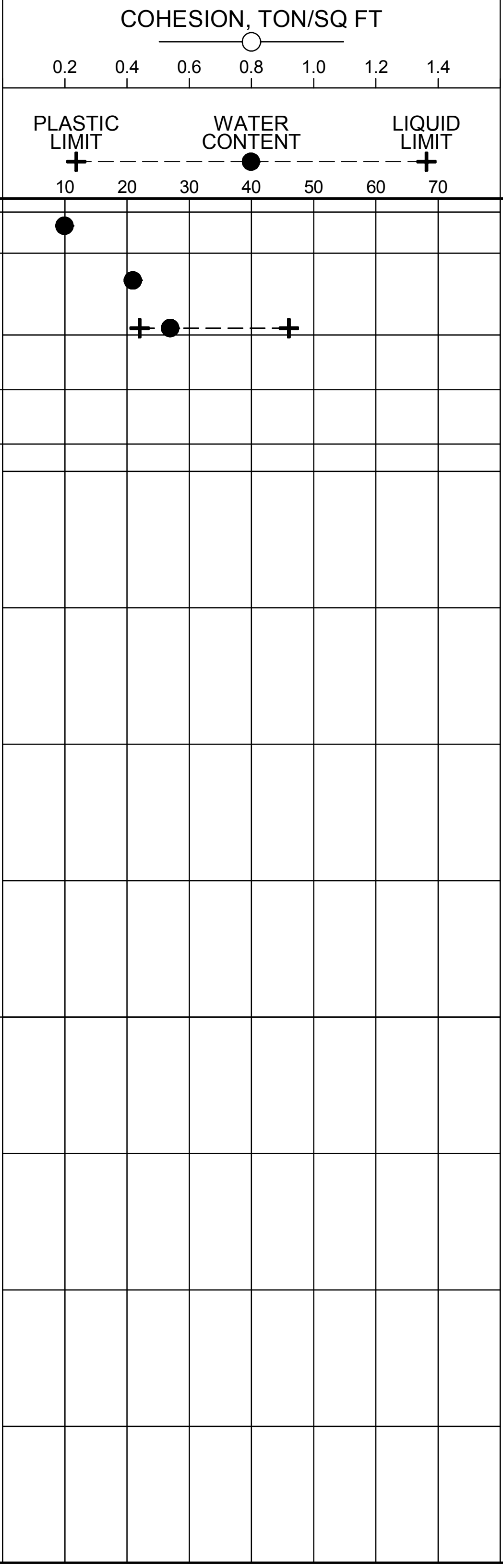
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1915+40, 10 ft Rt

SURF. EL: 178.4

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Very stiff brown silty clay	16		
	Medium dense brown silty fine sand	14		81
5	Stiff dark brown silty clay w/organic stains - light gray and tan below 4.5 ft	15		88
		11		
	Medium dense brown silty fine sand			
10	Loose brown and tan fine sand, slightly silty	7		
	- medium dense at 12 - 18 ft			
15		13		
	- dense at 18 to 23 ft			
20		35		
	- dense to very dense at 23 to 28 ft			
25		50/11"		
	- medium dense below 28 ft			
30		16		9
	Dense gray and brown fine to medium sand, slightly silty			
35		39		
	- medium dense below 38 ft			
40		28		
	- gray below 42 ft			
45		19		6
	- gray and dark gray with organic inclusions below 47 ft			
		16		



COMPLETION DEPTH: 88.0 ft
DATE: 6-26-13

DEPTH TO WATER
IN BORING: 8 ft

DATE: 6/26/2013

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

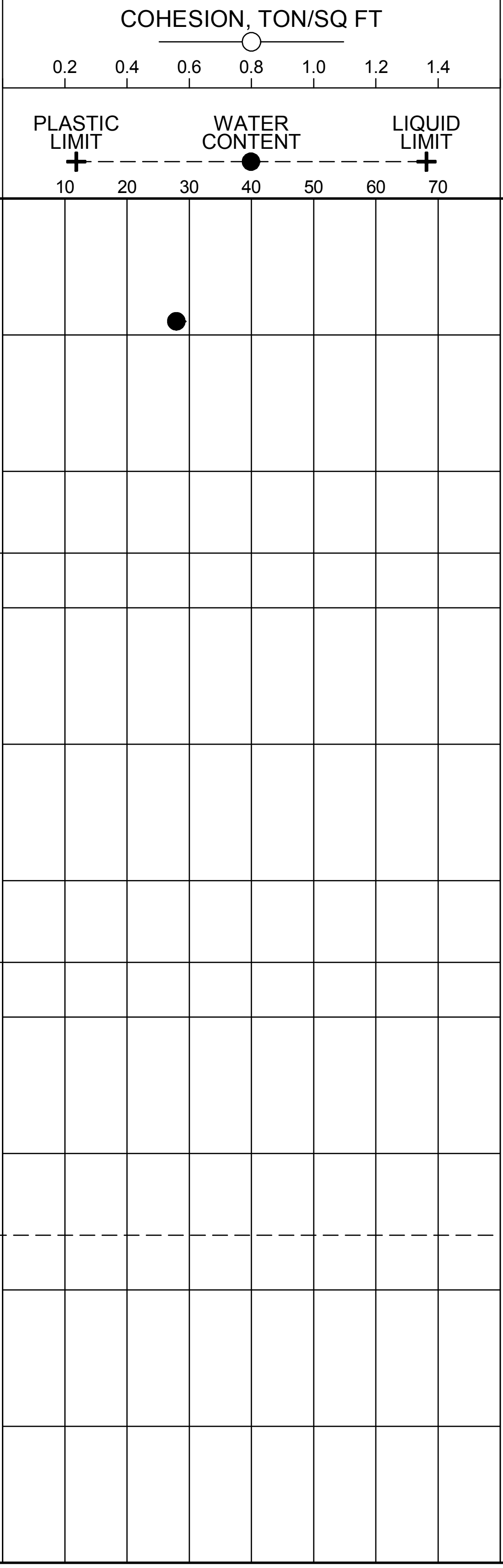
LOG OF BORING NO. 17

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1915+40, 10 ft Rt

DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	- dense below 52 ft			
55	- with silty clay pockets and seams at 54 - 56 ft	38		17
60		50		
65	Dense gray and brown fine to medium sand	38		
	- dense to very dense below 67 ft			
70		50/8"		4
75		50/8"		
80	Dense to very dense brown and gray sandy fine to coarse gravel	50/8"		
85		50/8"		3
90				
95				



COMPLETION DEPTH: 88.0 ft
DATE: 6-26-13

DEPTH TO WATER
IN BORING: 8 ft

DATE: 6/26/2013

13-017

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

LOG OF BORING NO. 18

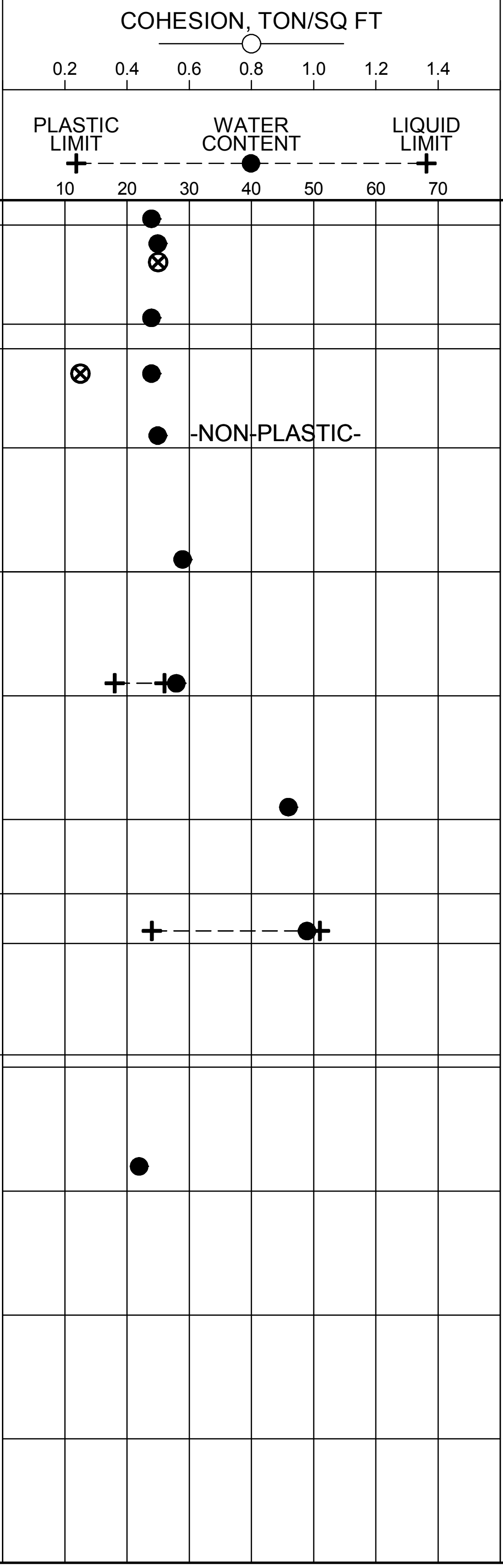
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1902+75, 60 ft Rt

SURF. EL: 177.1

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft dark brown silty clay Firm to stiff brown clay	8		
5	- stiff below 4 ft	16		
	Soft brown and light brown silty clay			
10	- with interbedded fine sandy silt layers below 8.5 ft - firm at 9 - 10 ft	9		51
	- soft below 12 ft			
15		4		
	Very soft brownish gray fine sandy clay, silty			
20		3		77
25	- firm below 23 ft	8		
	Stiff brown clay			
30		21		99
	- very stiff below 33 ft	44		
35	Dense gray silty fine sand			
40	- dense to very dense below 38 ft	50/11"		20
45		50/11"		
50		50/10"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-14-14

DEPTH TO WATER
IN BORING: 7 ft

DATE: 2/14/2014

13-017

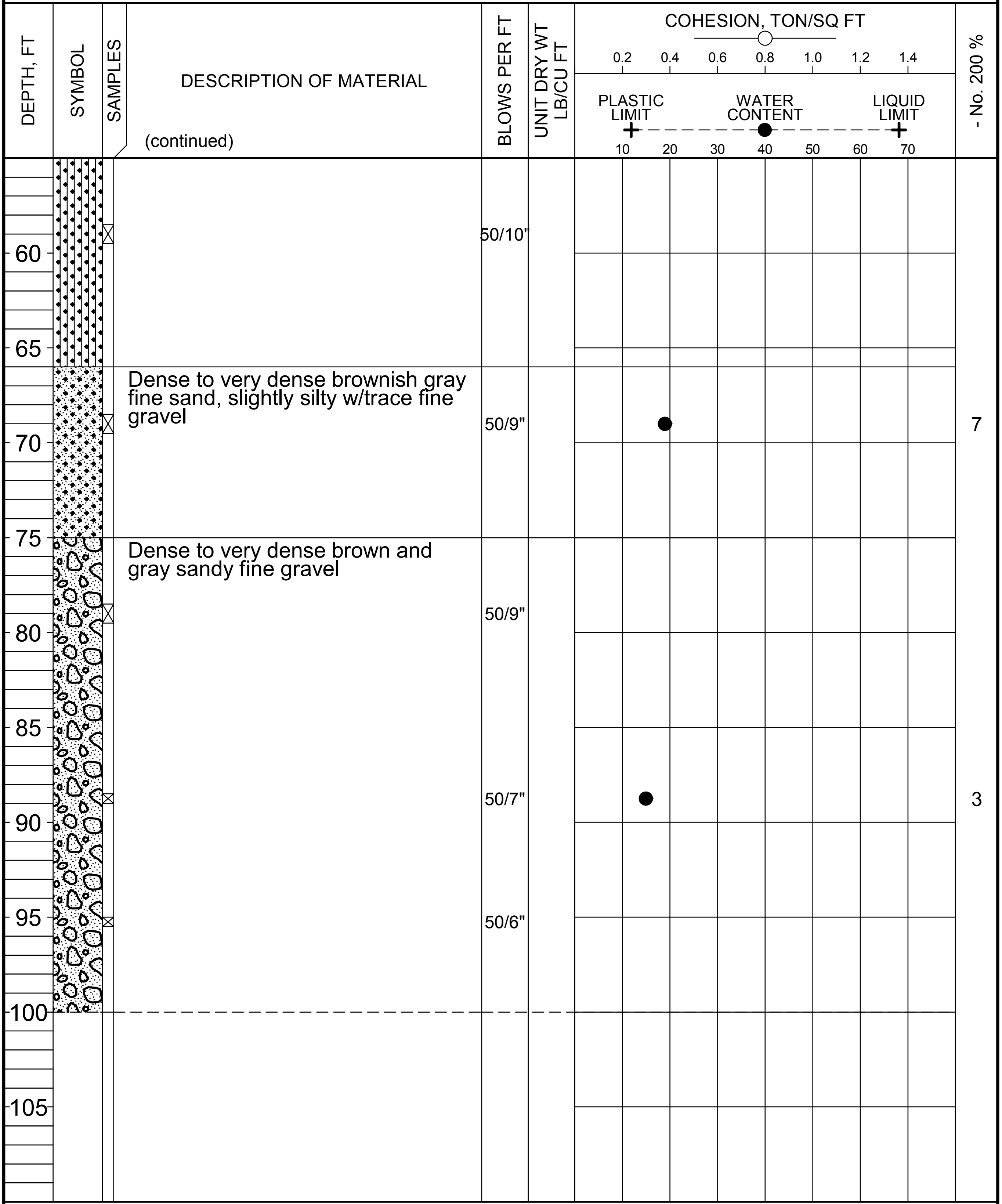
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

L O G O F B O R I N G N O. 18

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1902+75, 60 ft Rt



Dense to very dense brownish gray fine sand, slightly silty w/trace fine gravel

Dense to very dense brown and gray sandy fine gravel

COMPLETION DEPTH: 100.0 ft
DATE: 2-14-14

DEPTH TO WATER
IN BORING: 7 ft

DATE: 2/14/2014

LGBNEW 13-017.GPJ 4-8-14

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

LOG OF BORING NO. 19

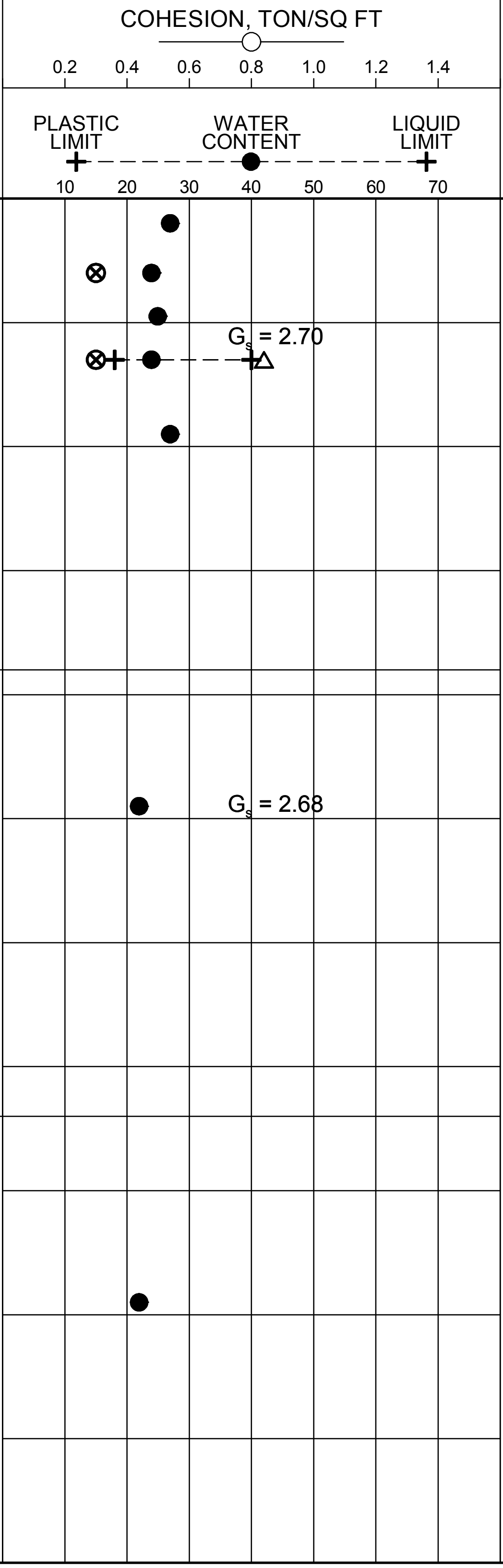
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1903+95, 60 ft Lt

SURF. EL: 174.5

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm dark brown and brown silty clay w/ferrous stains and nodules	8		
	- firm to stiff at 2 - 6 ft			
	- gray and brown below 4 ft			
5		13		
	- firm below 6 ft		102	96
10		9		
15		7		
	- gray below 15 ft			
20	Medium dense gray fine sand, slightly silty	23		
	- dense below 23 ft			
25		43		8
30		48		
35		55		
	Dense to very dense brownish gray fine sand, slightly silty			
40		50/9"		
45		50/10"		8
50		50/10"		
	- with some organic inclusions, occasional silty clay pockets and a little fine to coarse gravel below 50 ft			



COMPLETION DEPTH: 100.0 ft
DATE: 2-17-14

DEPTH TO WATER
IN BORING: 13 ft

DATE: 2/17/2014

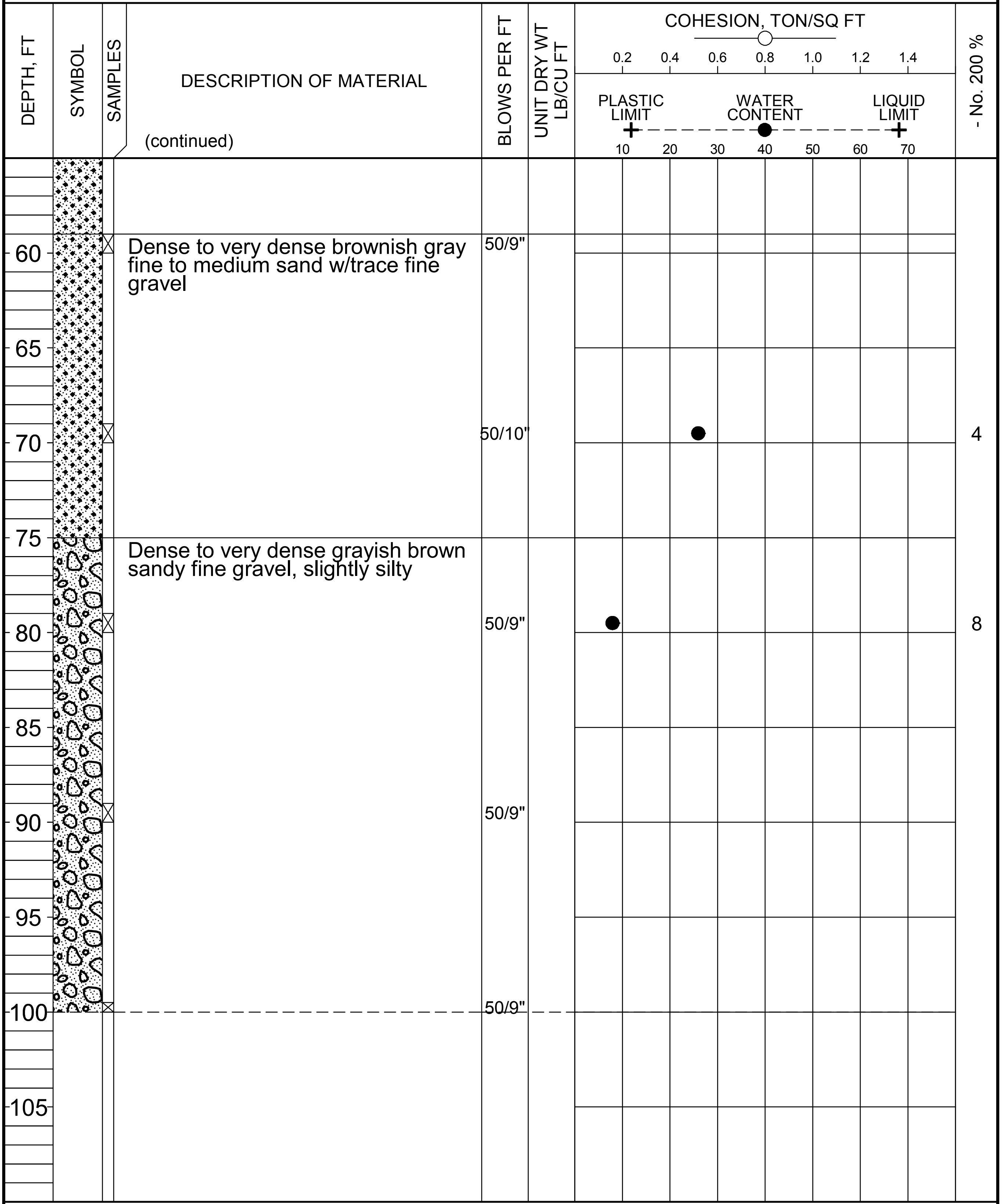
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 19

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1903+95, 60 ft Lt



DEPTH, FT	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
60	Dense to very dense brownish gray fine to medium sand w/trace fine gravel	50/9"		
70		50/10"		4
75	Dense to very dense grayish brown sandy fine gravel, slightly silty			
80		50/9"		8
90		50/9"		
100		50/9"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-17-14

DEPTH TO WATER
IN BORING: 13 ft

DATE: 2/17/2014

13-017

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

LOG OF BORING NO. 20

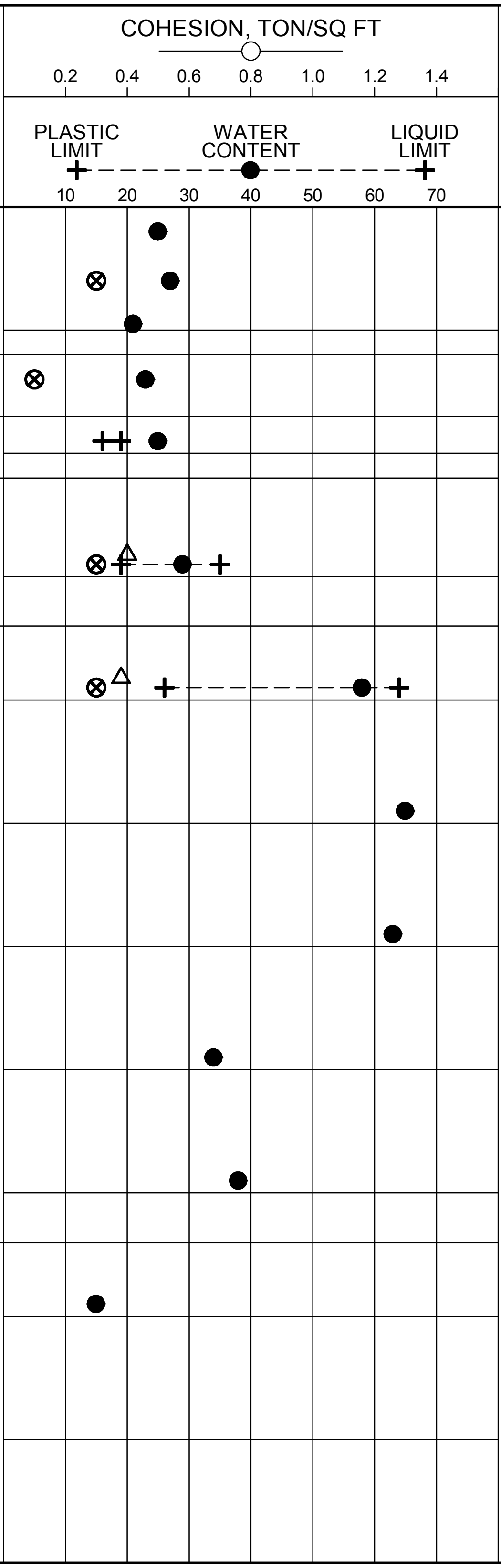
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1905+43, 60 ft Rt

SURF. EL: 177.0

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm to stiff brown silty clay w/ferrous stains and organics	10		
5	- stiff, gray and brown below 3.5 ft	14		
	Soft brown silty clay w/clayey silt pockets			
10	Medium dense brown fine sandy silt w/silty clay seams and layers	14		52
	Soft to firm brown and gray silty clay			
15			96	99
	Firm gray clay			
20			64	99
25	- with some organic inclusions below 23 ft - stiff at 23 to 32 ft	15		
30		13		
	- firm below 32 ft			
35		9		
40		8		
45	Dense gray fine to coarse sand, slightly silty w/a little fine gravel and occasional silty clay pockets	50		12
50		50		



COMPLETION DEPTH: 100.0 ft
DATE: 2-18-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/18/2014

LGBNEW 13-017.GPJ 6-13-14

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

LOG OF BORING NO. 20

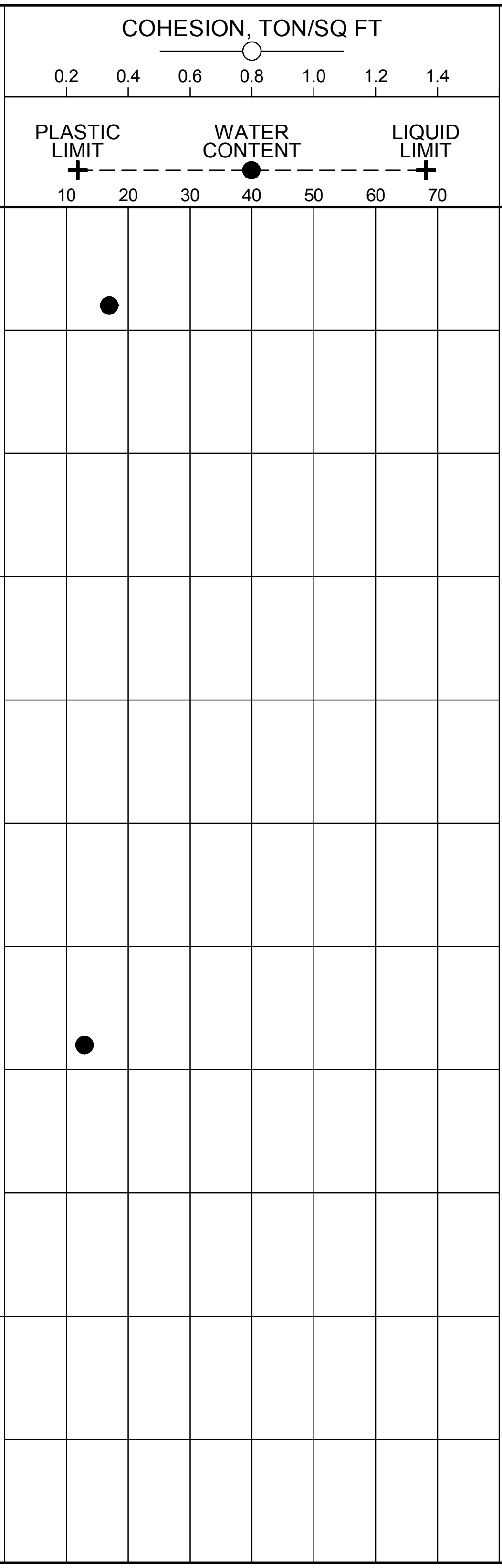
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1905+43, 60 ft Rt

(continued)

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
60	- dense to very dense below 58 ft	50/11"		6
65				
70	Dense to very dense brown sandy fine gravel w/occasional coarse sand seams and layers	50/10"		
75				
80		50/9"		
85				
90		50/8"		3
95				
100		50/6"		
105				



COMPLETION DEPTH: 100.0 ft
DATE: 2-18-14

DEPTH TO WATER
IN BORING: 8 ft

DATE: 2/18/2014

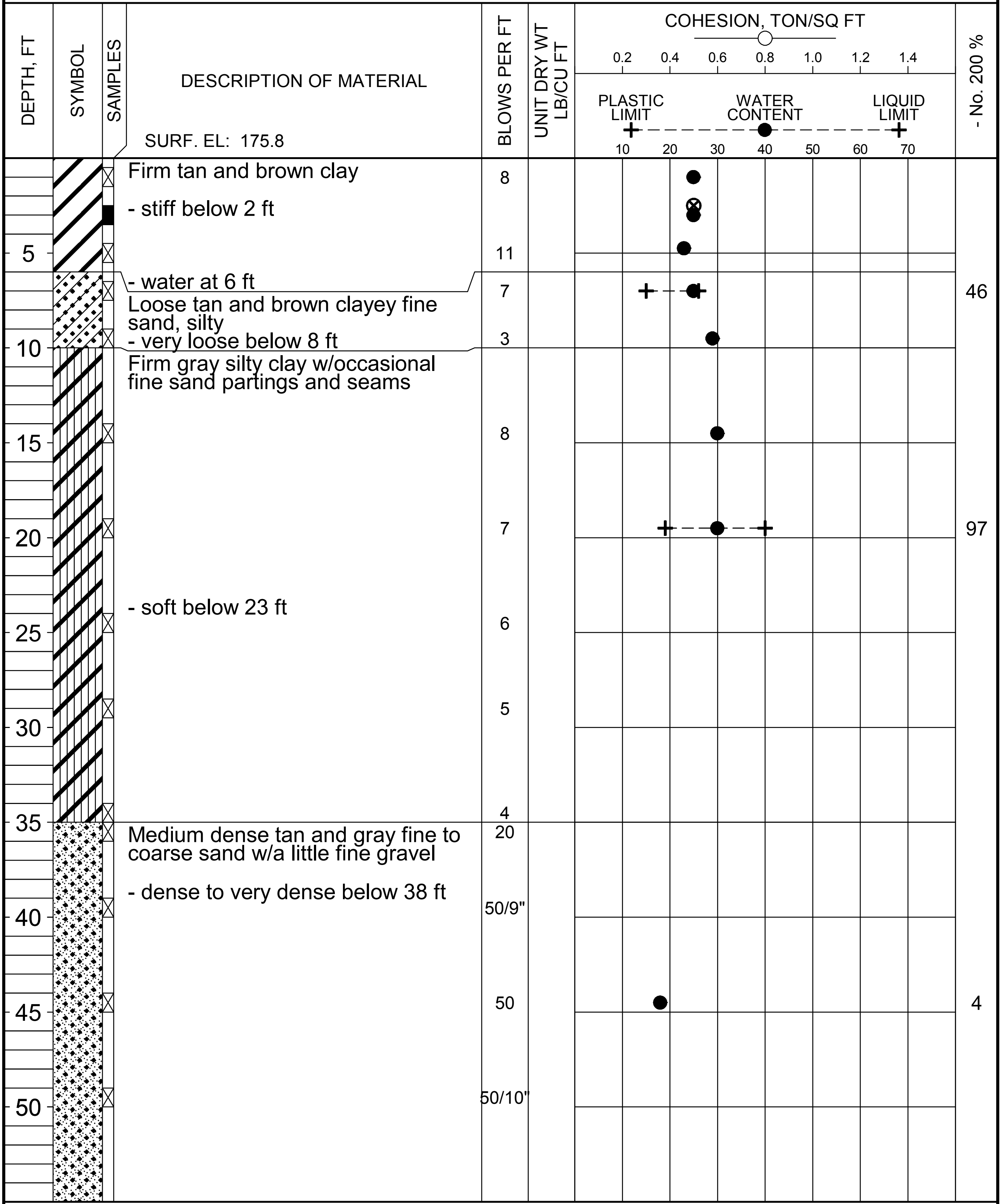
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 21

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash
LOCATION: Sta 1906+91, 60 ft Lt

SURF. EL: 175.8



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Firm tan and brown clay	8		
- stiff below 2 ft			
	11		
- water at 6 ft			
Loose tan and brown clayey fine sand, silty	7		46
- very loose below 8 ft	3		
Firm gray silty clay w/occasional fine sand partings and seams			
	8		
	7		97
- soft below 23 ft	6		
	5		
	4		
Medium dense tan and gray fine to coarse sand w/a little fine gravel	20		
- dense to very dense below 38 ft	50/9"		
	50		4
	50/10"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-21-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/21/2014

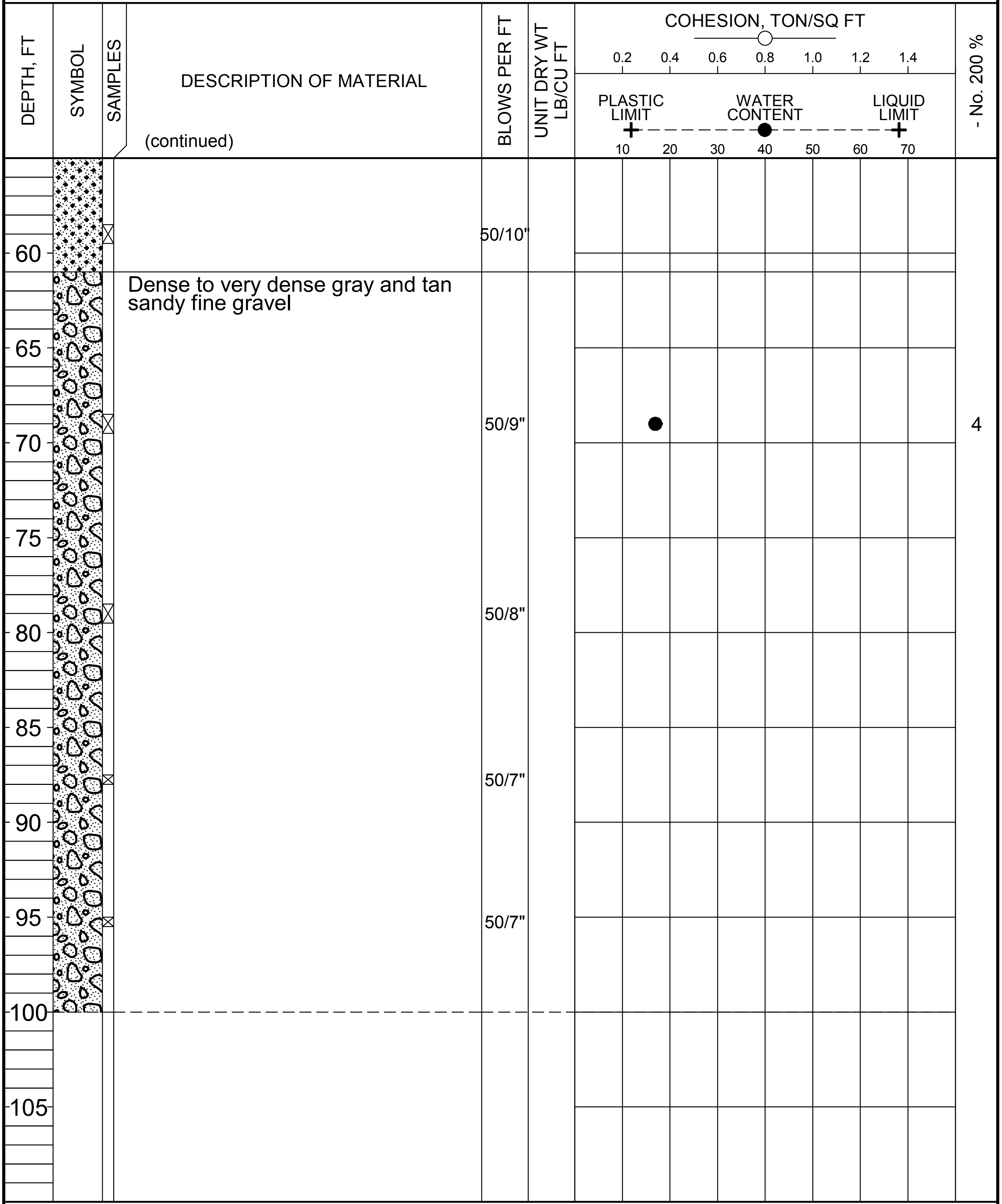
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 21

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1906+91, 60 ft Lt



COMPLETION DEPTH: 100.0 ft
DATE: 2-21-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/21/2014

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

LOG OF BORING NO. 22

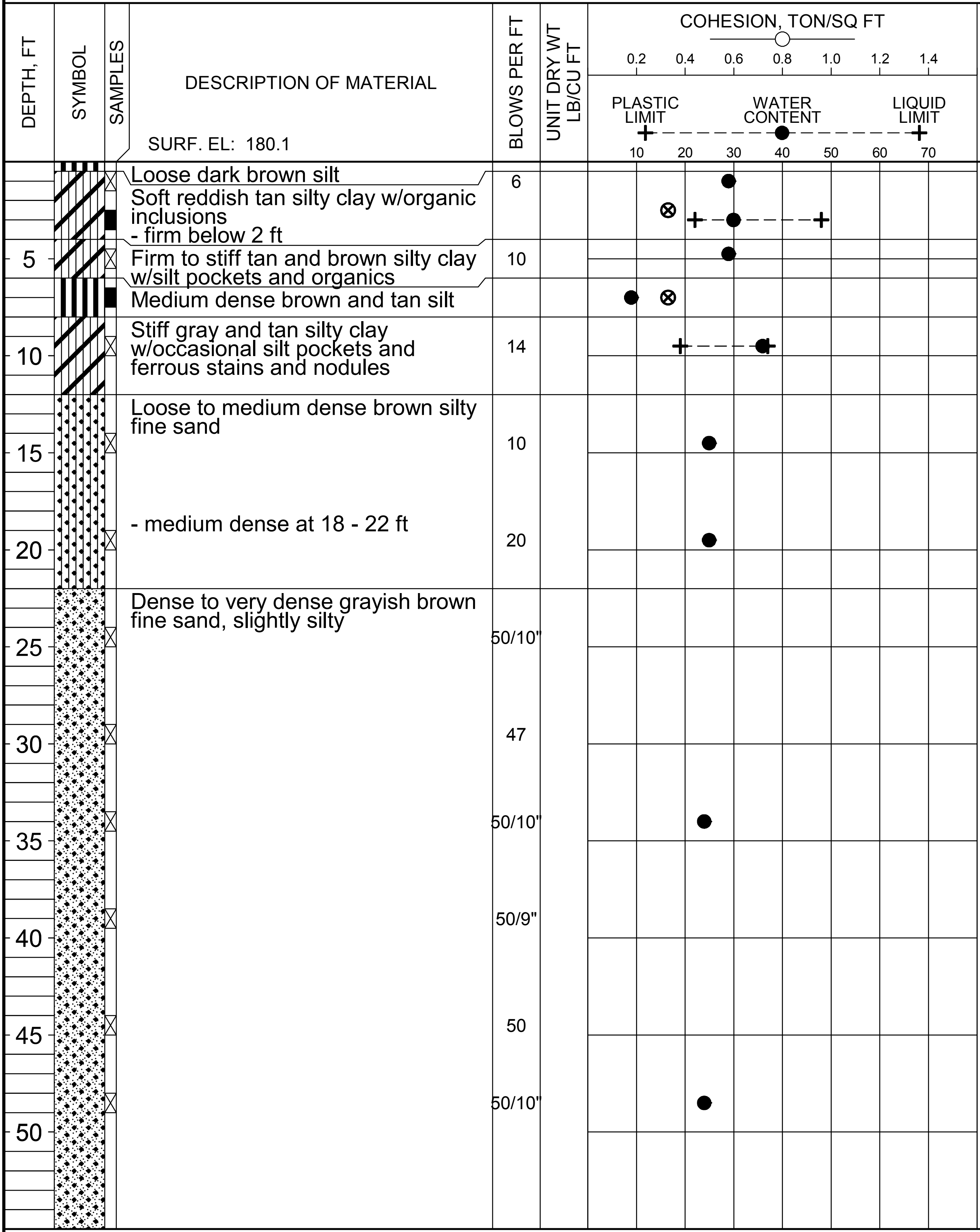
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1908+11, 60 ft Rt

SURF. EL: 180.1

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
Loose dark brown silt	6		
Soft reddish tan silty clay w/organic inclusions - firm below 2 ft			99
Firm to stiff tan and brown silty clay w/silt pockets and organics	10		
Medium dense brown and tan silt			
Stiff gray and tan silty clay w/occasional silt pockets and ferrous stains and nodules	14		95
Loose to medium dense brown silty fine sand	10		13
- medium dense at 18 - 22 ft	20		
Dense to very dense grayish brown fine sand, slightly silty	50/10"		
	47		
	50/10"		9
	50/9"		
	50		
	50/10"		6



COMPLETION DEPTH: 100.0 ft
DATE: 2-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 2/12/2014

LGBNEW 13-017.GPJ 4-8-14

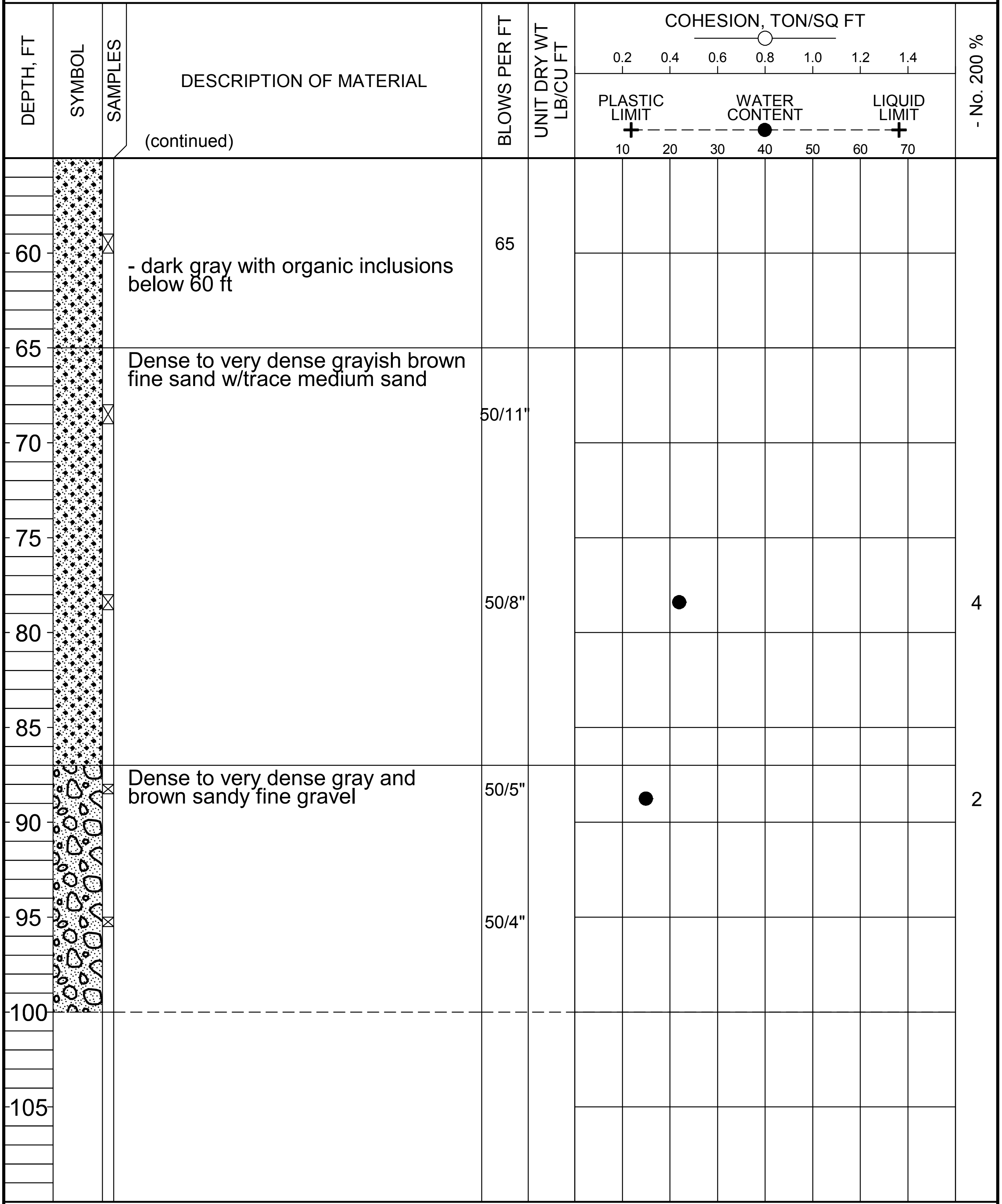
13-017

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

LOG OF BORING NO. 22

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash
LOCATION: Sta 1908+11, 60 ft Rt



Depth, ft	Description of Material	Blows per ft	- No. 200 %
60	- dark gray with organic inclusions below 60 ft	65	
65	Dense to very dense grayish brown fine sand w/trace medium sand		
68		50/11"	
78		50/8"	4
87	Dense to very dense gray and brown sandy fine gravel		
88		50/5"	2
95		50/4"	

COMPLETION DEPTH: 100.0 ft
DATE: 2-12-14

DEPTH TO WATER
IN BORING: 12 ft

DATE: 2/12/2014

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

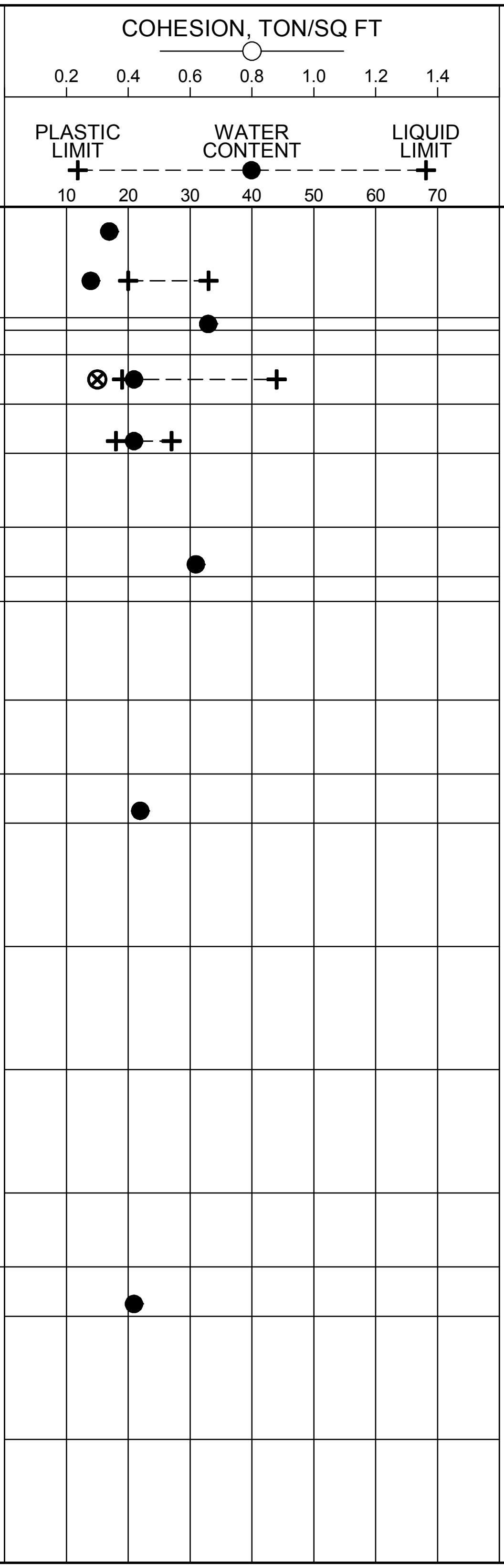
LOG OF BORING NO. 23

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1909+25, 60 ft Lt

DEPTH, FT	DESCRIPTION OF MATERIAL SURF. EL: 185.4	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Stiff brown and light brown silty clay w/organics	15		
		20		87
5	Stiff light tan clay	11		
	Firm brown and tan silty clay w/ferrous stains and nodules		100	97
10	Stiff brown clayey silt	22		71
15	Loose to medium dense gray fine sandy silt w/occasional silty clay pockets	10		76
20	Loose brown silty fine sand	6		
25	Dense to very dense grayish brown fine sand	50/10"		4
30		43		
35		50/7"		
40		50		
45	Dense brownish gray fine to medium sand, slightly silty	40		6
50	- dense to very dense below 48 ft - with organic inclusions below 49 ft	50/11"		



COMPLETION DEPTH: 100.0 ft
DATE: 2-11-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/11/2014

13-017

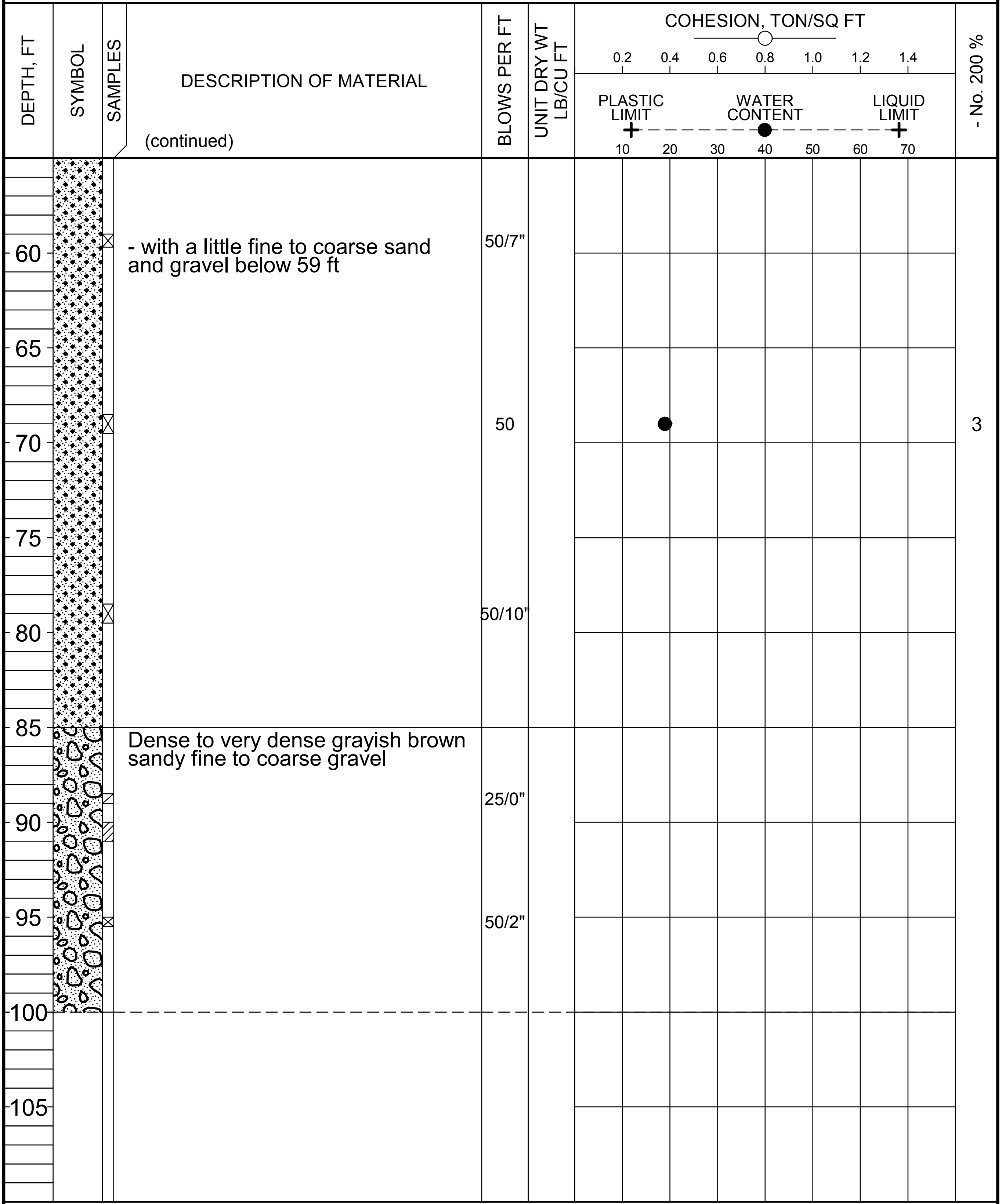
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 23

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1909+25, 60 ft Lt



DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
- with a little fine to coarse sand and gravel below 59 ft	50/7"		
	50		3
	50/10"		
Dense to very dense grayish brown sandy fine to coarse gravel			
	25/0"		
	50/2"		

COMPLETION DEPTH: 100.0 ft
DATE: 2-11-14

DEPTH TO WATER
IN BORING: 16 ft

DATE: 2/11/2014

Grubbs, Hoskyn, Barton & Wyatt, Inc.
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L O G O F B O R I N G N O. 24

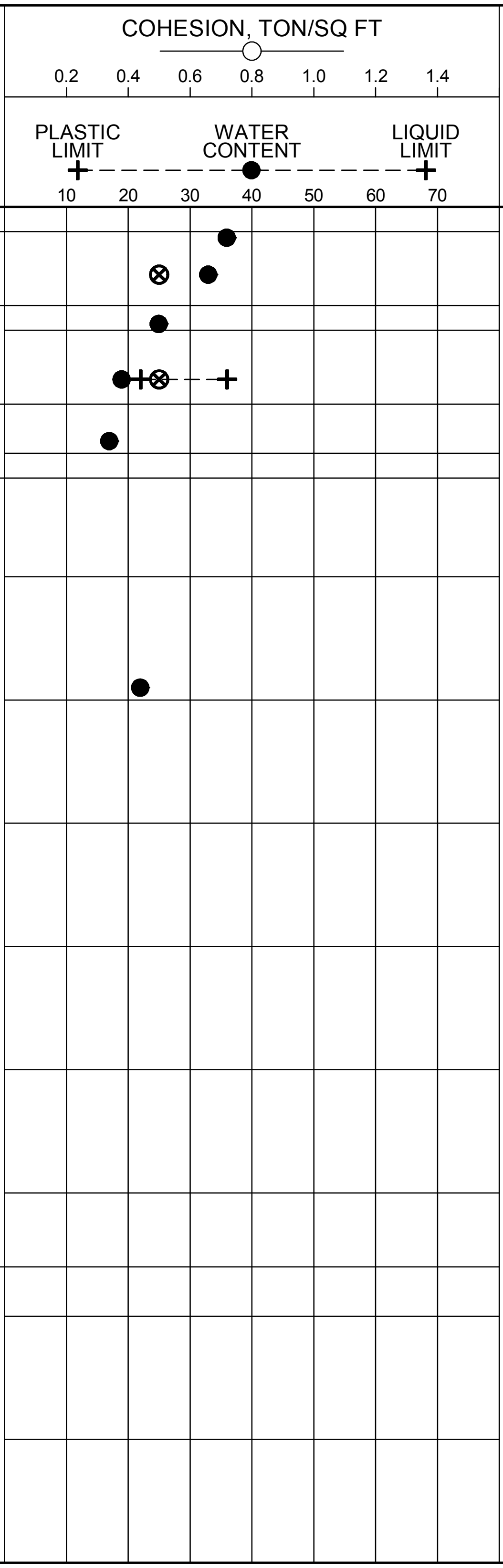
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1910+69, 60 ft Rt

SURF. EL: 187.3

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Soft brown silty clay	8		
	Firm brown clay			
	- firm to stiff below 2.5 ft			
5	Very stiff tan and brown silty clay	27		
	- stiff below 6 ft			88
10	Medium dense tan and brown fine sandy silt	28		67
	Medium dense brown and tan fine sand, slightly silty			
15		28		
	- dense below 18 ft			
20		38		7
25		50		
30		50		
35		32		
40		46		
45	Dense to very dense tan and gray fine sand, slightly silty	50/8"		
50		50/10"		
		50/8"		



COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 2/20/2014

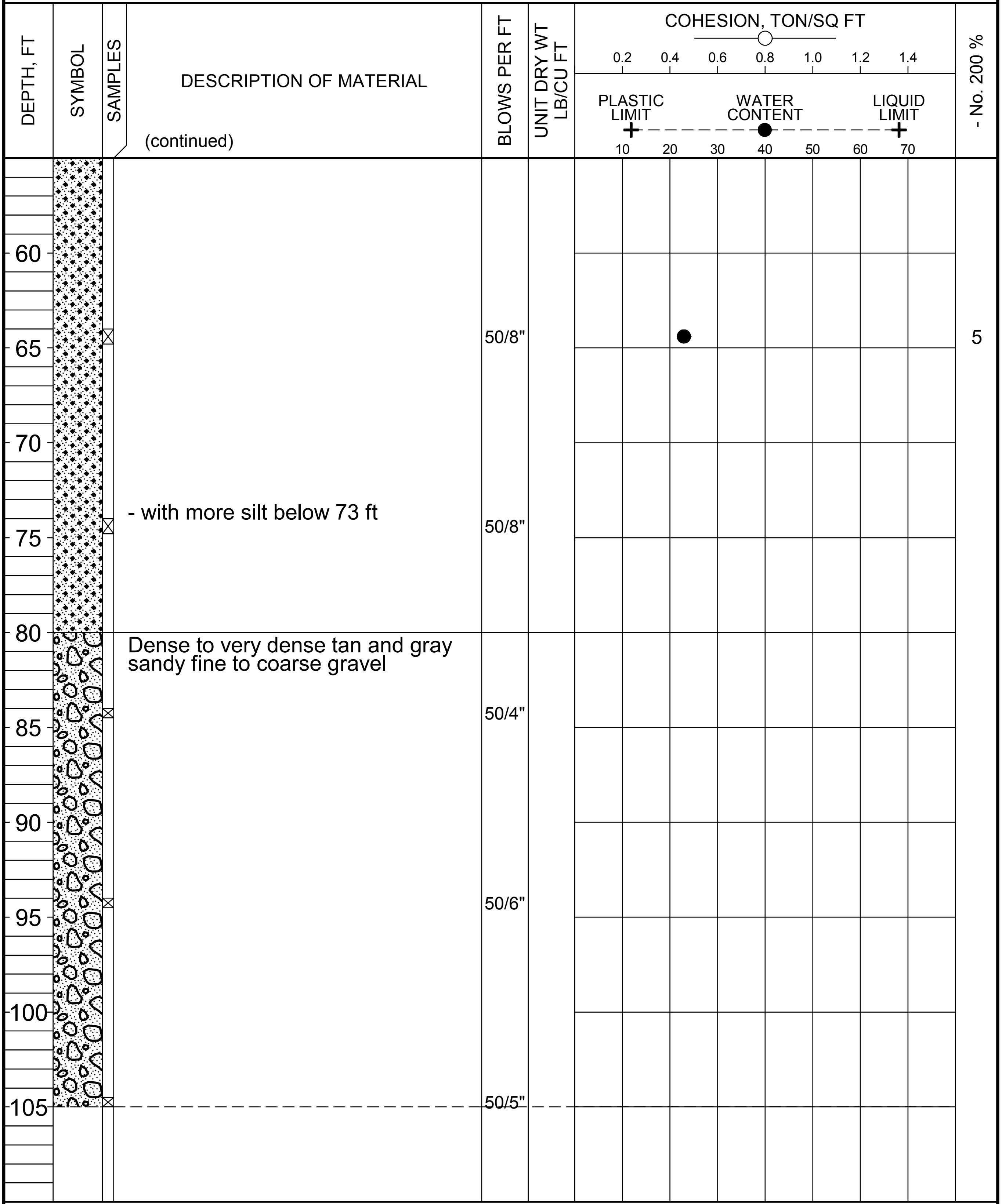
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 24

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1910+69, 60 ft Rt



- with more silt below 73 ft

Dense to very dense tan and gray sandy fine to coarse gravel

BLOWS PER FT	- No. 200 %
50/8"	5
50/8"	
50/4"	
50/6"	
50/5"	

COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 2/20/2014

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

LOG OF BORING NO. 25

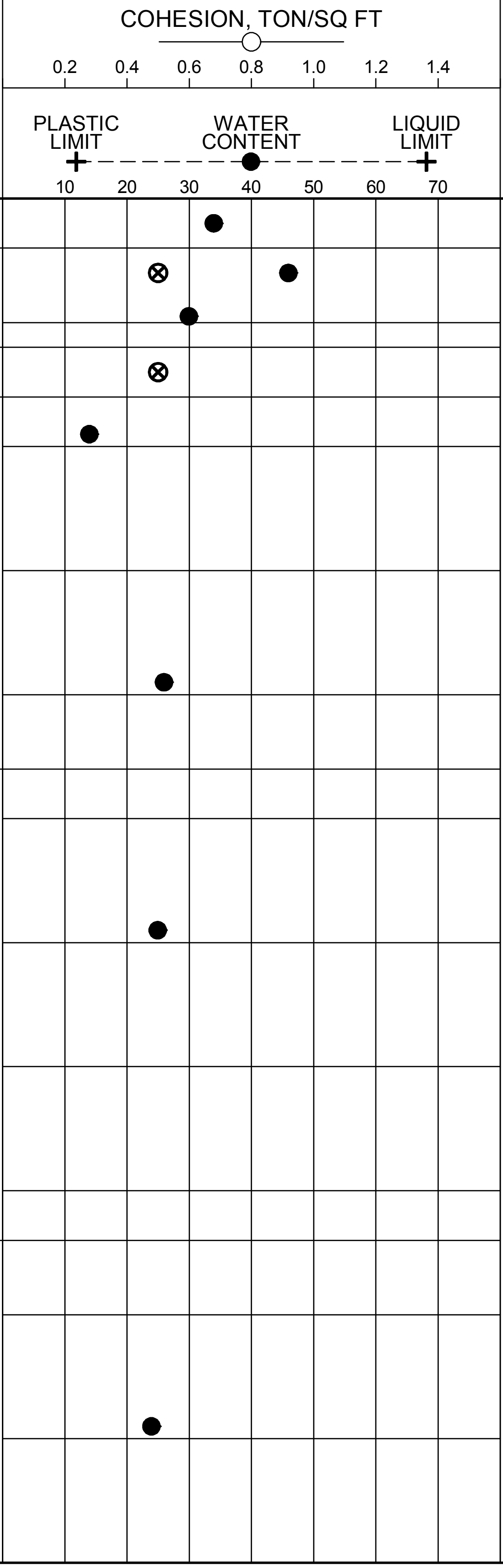
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1911+84, 60 ft Lt

SURF. EL: 184.3

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	Firm tan and brown silty clay	8		
	Firm to stiff tan clay			
5	- stiff below 4 ft	16		
	Firm to stiff tan and brown clayey silt w/silty clay pockets			
10	Medium dense tan and brown silty fine sand w/occasional silty clay pockets	16		35
	- less silty below 13 ft			
15		21		
20		27		15
25	Dense tan and gray fine sand, slightly silty	35		
30		38		10
	- dense to very dense beloe 33 ft			
35		59		
40		50/11"		
45	Dense to very dense tan and gray fine sand w/trace medium sand and fine gravel	50/11"		
50		50/10"		4
		50/8"		



COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/20/2014

13-017

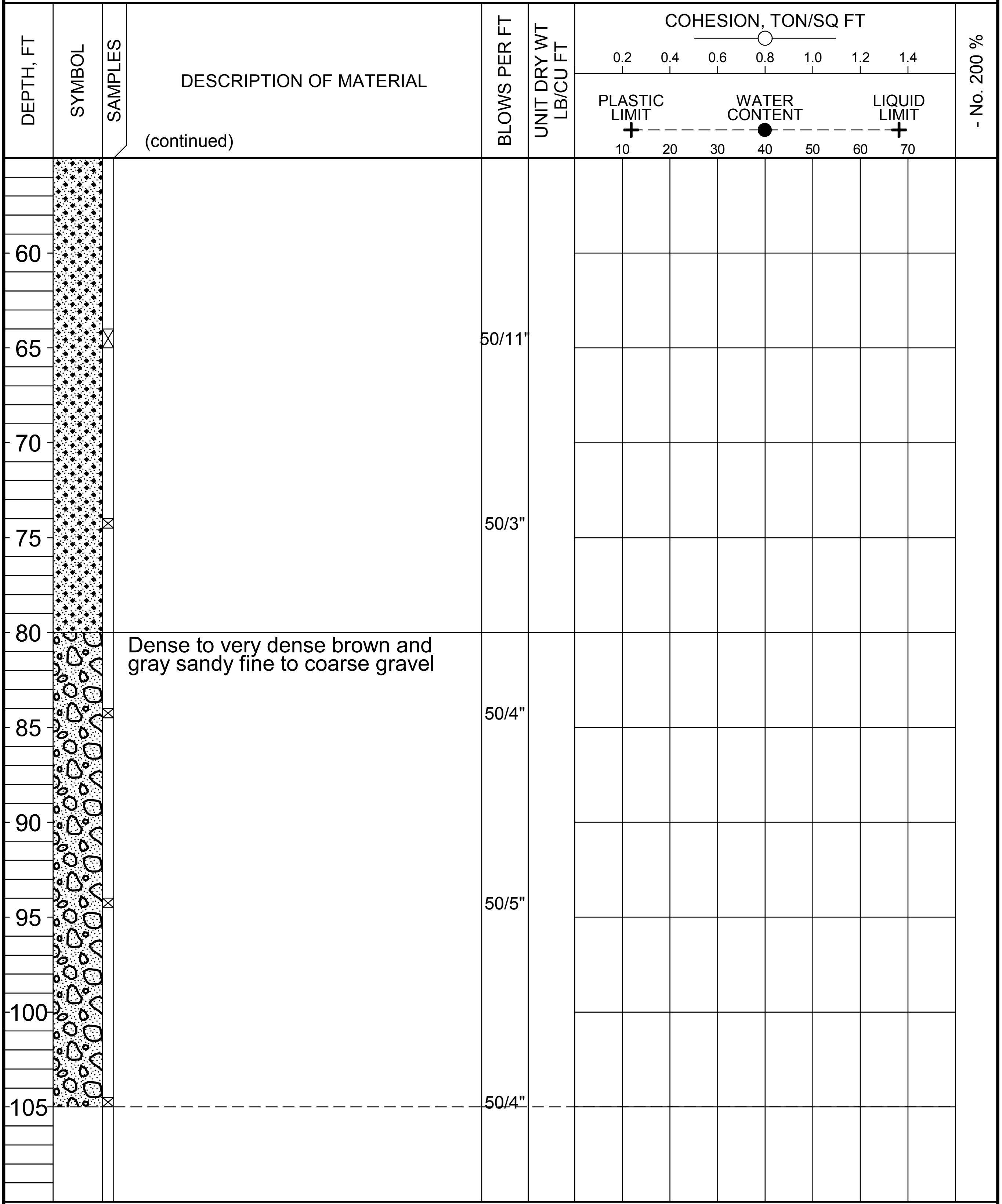
Grubbs, Hoskyn,
Barton & Wyatt, Inc.
Consulting Engineers

LOG OF BORING NO. 25

I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1911+84, 60 ft Lt



COMPLETION DEPTH: 105.0 ft
DATE: 2-20-14

DEPTH TO WATER
IN BORING: 6 ft

DATE: 2/20/2014

LGBNEW 13-017.GPJ 4-8-14

13-017

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

LOG OF BORING NO. 29

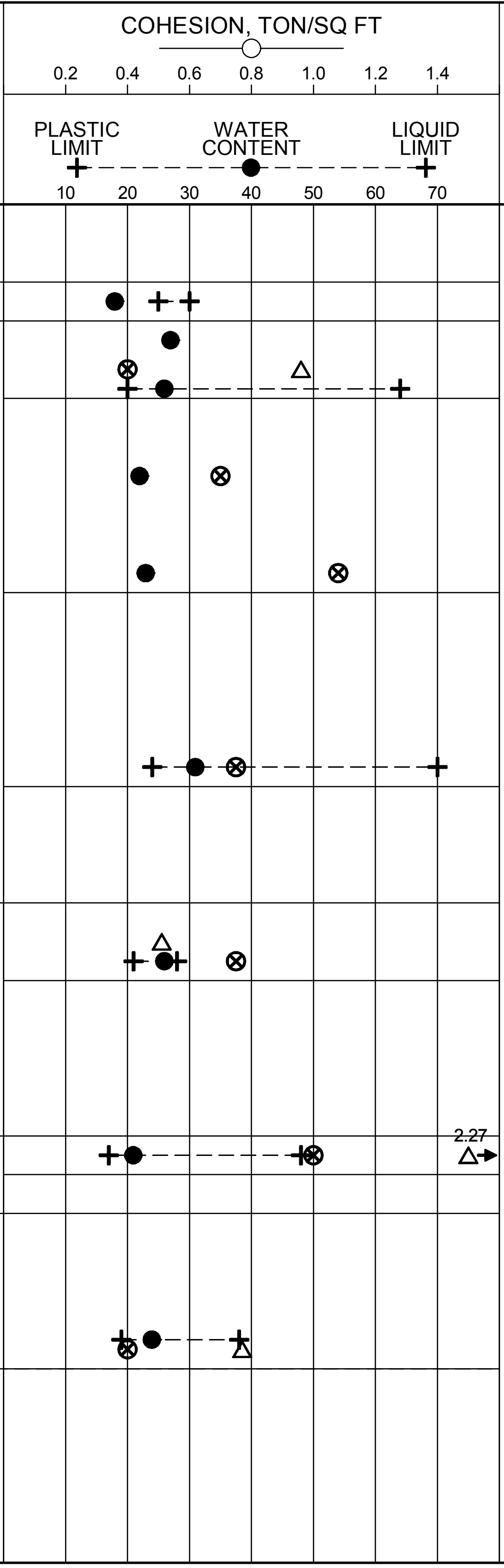
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger

LOCATION: Sta 1883+15, 80 ft Rt - Existing West Bridge End

SURF. EL: 205±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	10 inches: Asphalt Concrete 14 inches: Crushed Stone Base			
	Stiff reddish brown clayey silt, sandy (fill)	15		82
	Firm to stiff tan and gray clay w/fine sandy silt pockets (fill)	7		
5			101	96
	- perched water at 5.5 ft - stiff at 6 to 8 ft			
	- very stiff at 8 to 12 ft			
10				
	- stiff, reddish brown at 12 - 24 ft			
15			90	99
	Firm to stiff reddish brown clayey silt (fill)		95	97
20				
25	Stiff to very stiff tan and gray silty clay		106	97
	Firm to stiff brownish gray silty clay w/organic inclusions		100	97
30				
	NOTE: Borehole backfilled with cement-sand mix, pavement patched.			



COMPLETION DEPTH: 30.0 ft
DATE: 4-15-13

DEPTH TO WATER
IN BORING: Dry

DATE: 4/15/2013

13-017

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

L O G O F B O R I N G N O. 30

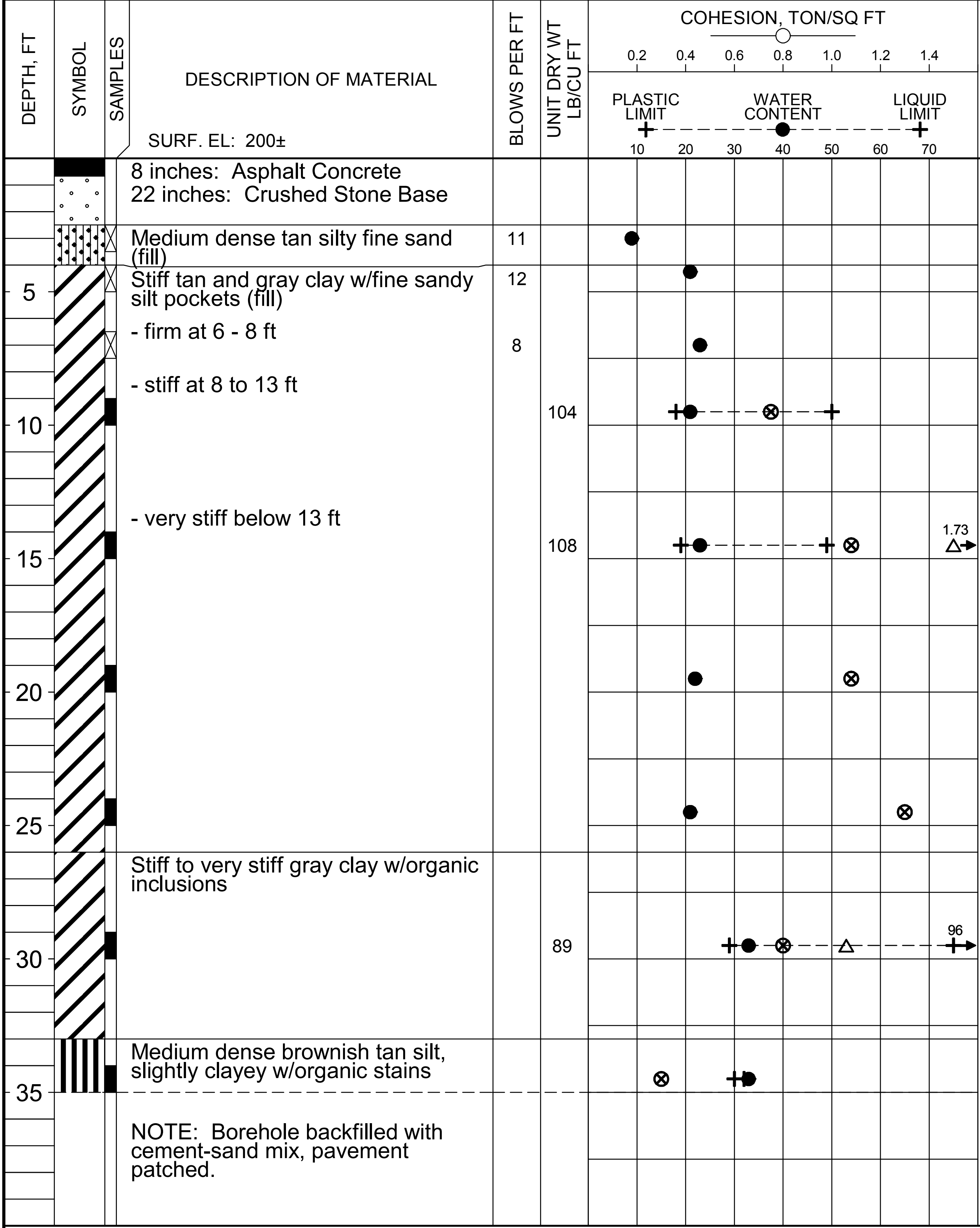
I-40 over White River - AHTD Job #BB0610
Prairie County, Arkansas

TYPE: Auger

LOCATION: Sta 1910+85, 185 ft Rt - Existing East Bridge End

SURF. EL: 200±

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
8 inches: Asphalt Concrete 22 inches: Crushed Stone Base			
Medium dense tan silty fine sand (fill)	11		
Stiff tan and gray clay w/fine sandy silt pockets (fill)	12		
- firm at 6 - 8 ft	8		
- stiff at 8 to 13 ft		104	87
- very stiff below 13 ft		108	95
Stiff to very stiff gray clay w/organic inclusions		89	99
Medium dense brownish tan silt, slightly clayey w/organic stains			99
NOTE: Borehole backfilled with cement-sand mix, pavement patched.			



COMPLETION DEPTH: 35.0 ft
DATE: 4-15-13

DEPTH TO WATER
IN BORING: Dry

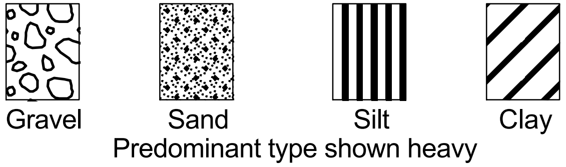
DATE: 4/15/2013

LGBNEW 13-017.GPJ 4-8-14

SYMBOLS AND TERMS USED ON BORING LOGS

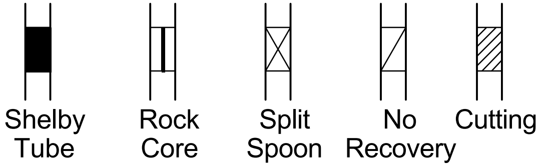
SOIL TYPES

(SHOWN IN SYMBOLS COLUMN)



SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



TERMS DESCRIBING CONSISTENCY OR CONDITION

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

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

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

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

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

TERMS CHARACTERIZING SOIL STRUCTURE

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

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

KEY 3-14-12

APPENDIX A

Summary of Subsurface Exploration Program

Project: AHTD Job No. BB0610: White River Str. & Apprs. (F)
Location: Prairie County, Arkansas
GHBW Job No.: 13-017

Boring No.	Approximate Station	Approximate Offset	Ground Surface El, ft	Completion Depth, ft	Project Feature
1	1877+60	70' Lt	184.3	70	West Bridge End
2	1879+00	50' Lt	191.3	70	West Bridge End
3	1880+15	50' Lt	190.5	70	West Bridge End
4	1883+83	60' Lt	171.9	100	Bent 1
5	1884+63	60' Rt	171.7	100	Bent 2
6	1886+11	60' Lt	171.0	100	Bent 3
7	1887+59	60' Rt	173.5	100	Bent 4
8	1888+79	60' Lt	174.4	100	Bent 5
9	1889+99	60' Rt	176.9	100	Bent 6
10	1891+47	60' Lt	177.2	100	Bent 7
11	1892+95	80' Rt	167.5	100	Bent 8
12	1894+15	60' Lt	168±	130	Bent 9
13	1896+90	60' Rt	137±	78	Pier 1
13A	1896+80	50' Rt	137±	150	Pier 1

Summary of Subsurface Exploration Program

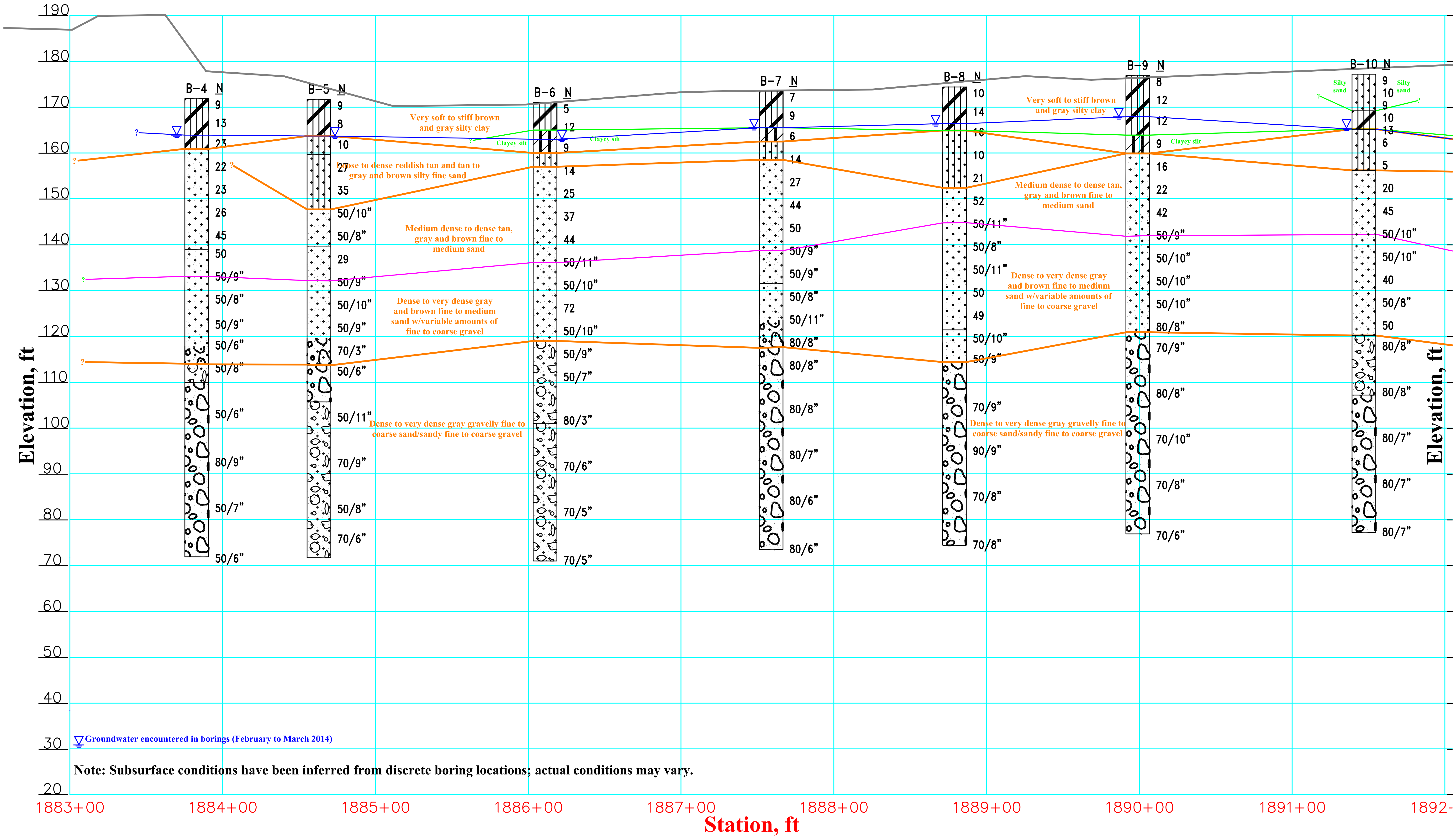
Project: AHTD Job No. BB0610: White River Str. & Apprs. (F)

Location: Prairie County, Arkansas

GHBW Job No.: 13-017

14	1900+10	60' Lt	176.6	125	Pier 2
15	1916+00	CL	180.1	70	East Bridge End
16	1917+80	CL	185.1	70	East Bridge End
17	1915+40	10' Rt	178.4	88	East Bridge End
18	1902+75	60' Rt	177.1	100	Bent 10
19	1903+95	60' Lt	174.5	100	Bent 11
20	1905+43	60' Rt	177.0	100	Bent 12
21	1906+91	60' Lt	175.8	100	Bent 13
22	1908+11	60' Rt	180.1	100	Bent 14
23	1909+25	60' Lt	185.4	100	Bent 15
24	1910+69	60' Rt	187.3	105	Bent 16
25	1911+81	60' Lt	184.3	105	Bent 17
29	1883+15	80' Rt	205±	30	Existing West Bridge End
30	1910+85	185' Rt	200±	35	Existing East Bridge End

APPENDIX B



▽ Groundwater encountered in borings (February to March 2014)

Note: Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.

Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

Grubbs, Hoskyn,
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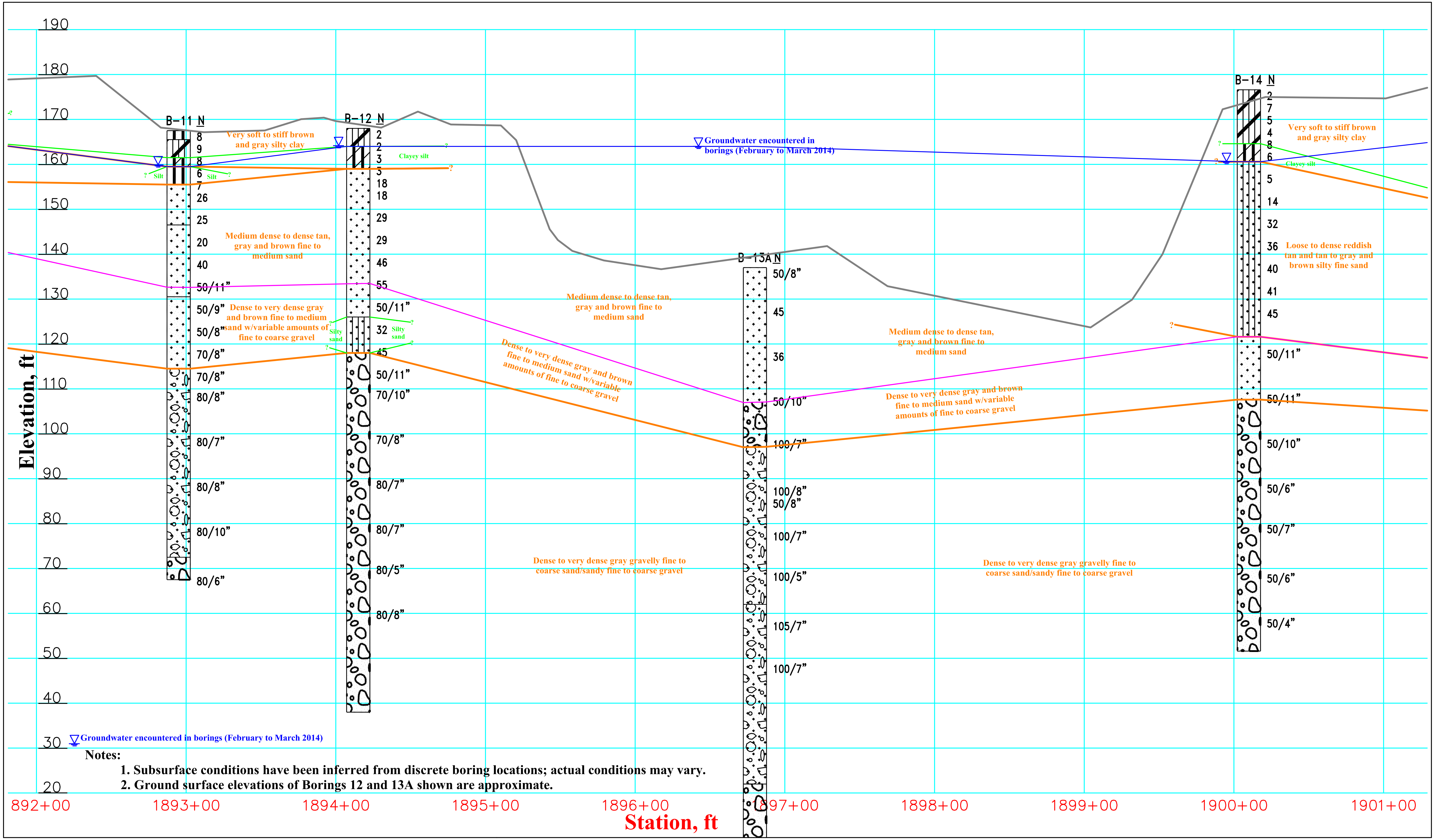
Generalized Subsurface Profile - Page 1 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	Scale: As Shown
April 8, 2014	Plate



▽ Groundwater encountered in borings (February to March 2014)

Notes:

1. Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.
2. Ground surface elevations of Borings 12 and 13A shown are approximate.

Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

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Consulting Engineers

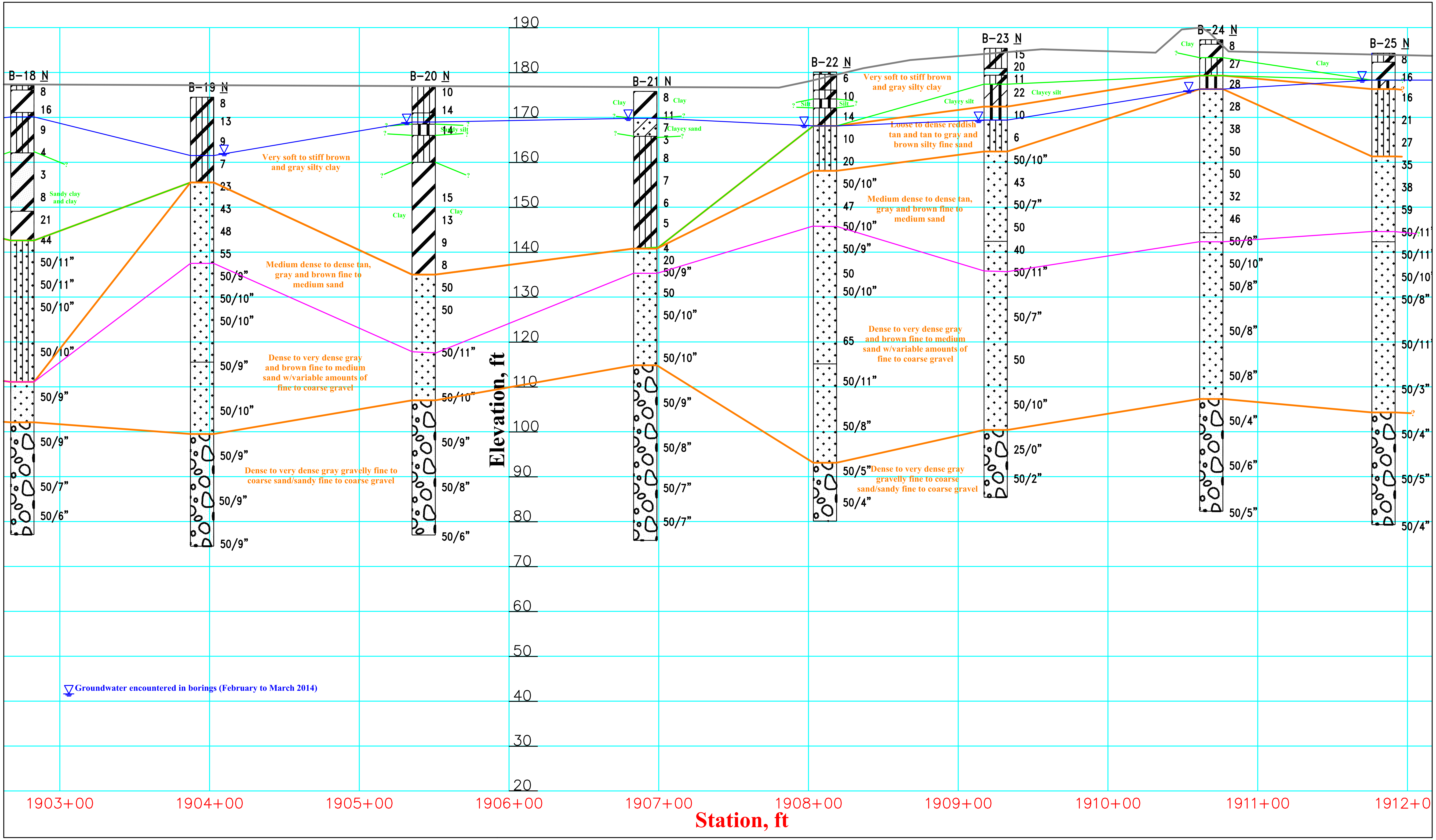
Generalized Subsurface Profile - Page 2 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

April 8, 2014

Plate



Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

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Consulting Engineers

Generalized Subsurface Profile - Page 3 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

April 8, 2014

Plate

APPENDIX C

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
1	0.5-1.5	21	36	15	21	----	----	----	----	----	----	82	CL	A-6
1	5-5.5	25	47	20	27	----	----	----	----	----	----	96	CL	A-7-6
1	9.5-10	26	43	19	24	100	100	100	100	100	98	97	CL	A-7-6
1	14-15	29	44	19	25	---	---	---	---	---	---	97	CL	A-7-6
1	19-20	24	----	----	----	100	100	100	100	100	96	13	SM	A-2-4
1	34-35	19	----	----	----	100	100	100	100	100	73	6	SP-SM	A-3
1	39-40	20	----	----	----	100	100	100	100	100	57	16	SM	A-2-4
1	54-55	17	----	----	----	100	100	96	95	94	62	5	SP-SM	A-3
2	4.5-5.5	25	52	23	29	100	100	100	100	100	99	98	CH	A-7-6
2	7-7.5	24	47	20	27	----	----	----	----	----	----	98	CL	A-7-6
2	14-15	26	44	18	26	----	----	----	----	----	----	98	CL	A-7-6
2	29-30	21	----	----	----	100	100	100	100	100	82	14	SM	A-2-4
2	59-60	19	----	----	----	100	100	92	89	88	52	7	SP-SM	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
3	2.5-3.5	28	43	20	23	----	----	----	----	----	----	97	CL	A-7-6
3	4.5-5	25	48	19	29	100	100	100	100	99	97	95	CL	A-7-6
3	7-7.5	24	47	21	26	100	100	100	100	99	98	95	CL	A-7-6
3	14-15	26	38	18	20	----	----	----	----	----	----	93	CL	A-6
3	19-20	25	19	11	8	----	----	----	----	----	----	39	SC	A-4
3	29-30	24	----	----	----	100	100	100	100	100	76	15	SM	A-2-4
3	39-40	22	----	----	----	100	100	100	100	100	34	6	SP-SM	A-1-b
3	49-50	18	----	----	----	100	100	98	98	95	57	11	SP-SM	A-2-4
4	7-7.5	23	46	19	27	100	100	100	100	100	99	97	CL	A-7-6
4	14-15	26	----	----	----	100	100	100	100	100	60	5	SP-SM	A-3
4	38.5-39.5	30	----	----	----	100	100	100	100	100	77	4	SP	A-3
4	53.5-54	18	----	----	----	100	100	89	87	86	58	10	SP-SM	A-3
4	88.5-89.5	18	----	----	----	100	100	92	51	26	7	2	GW	A-1-a

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
5	3-3.5	24	33	17	16	100	100	100	100	100	100	87	CL	A-6
5	9-10	25	----	----	----	----	----	----	----	----	----	31	SM	A-2-4
5	24-25	23	----	----	----	100	100	100	100	100	95	2	SP	A-3
5	43.5-44.5	31	----	----	----	100	100	100	100	100	77	4	SP	A-3
5	68.5-69.5	17	----	----	----	100	96	88	80	73	54	8	SP-SM	A-3
6	3-3.5	28	48	21	27	100	100	100	100	100	100	99	CL	A-7-6
6	19-20	22	----	----	----	100	100	100	100	100	99	5	SP-SM	A-3
6	58.5-59.5	20	----	----	----	100	100	89	81	75	51	18	SM	A-2-4
6	88.5-89	12	----	----	----	100	100	100	100	100	77	4	SP	A-3
7	7-7.5	28	42	19	23	----	----	----	----	----	----	97	CL	A-7-6
7	14-15	27	----	----	----	----	----	----	----	----	----	41	SM	A-2-4
7	24-25	23	----	----	----	100	100	100	100	100	90	9	SP-SM	A-3
7	43.5-44.5	18	----	----	----	100	100	97	94	91	42	19	SM	A-1-b

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
7	88.5-89	20	----	----	----	100	100	99	77	54	24	4	SW	A-1-b
8	7-7.5	26	37	15	22	100	100	100	100	100	100	81	CL	A-6
8	14-15	26	----	----	----	100	100	100	100	100	100	29	SM	A-2-4
8	29-30	22	----	----	----	100	100	100	100	100	95	5	SP-SM	A-3
8	49-50	21	----	----	----	100	100	100	100	100	95	12	SP-SM	A-2-4
8	68.5-69.5	17	----	----	----	100	100	95	85	74	44	4	SP	A-1-b
9	14-15	30	25	19	6	----	----	----	----	----	----	66	CL-ML	A-4
9	29-30	24	----	----	----	100	100	100	100	100	95	9	SP-SM	A-3
9	58.5-59.5	31	----	----	----	100	100	99	81	49	24	7	SP-SM	A-1-a
10	6.5-7.5	12	Non-plastic			----	----	----	----	----	----	20	SM	A-2-4
10	14-15	28	Non-plastic			----	----	----	----	----	----	38	SM	A-4
10	29-30	21	----	----	----	100	100	100	100	99	83	4	SP	A-3
10	49-50	20	----	----	----	100	100	100	100	98	77	4	SP	A-3

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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**		
11	14-15	25	----	----	----	----	----	----	----	----	----	5	SP-SM	A-3
11	29-30	23	----	----	----	100	100	100	100	100	98	2	SP	A-3
11	48.5-49.5	18	----	----	----	100	100	100	98	97	31	2	SP	A-1-b
12	19-20	17	----	----	----	----	----	----	----	----	----	2	SP	A-3
12	38.5-39.5	19	----	----	----	100	100	100	99	98	66	3	SP	A-3
12	49-50	----	----	----	----	100	100	100	100	100	93	16	SM	A-2-4
12	68.5-69.5	9	----	----	----	100	100	90	45	18	6	2	GW	A-1-a
13	6.5-7.5	24	Non-plastic			100	100	100	100	100	88	31	SM	A-2-4
13A	9-10	27	----	----	----	100	100	100	98	98	85	1	SP	A-3
13A	19-20	22	----	----	----	100	97	86	84	84	79	4	SP	A-3
13A	48.5-49.5	16	----	----	----	100	100	94	81	76	26	6	SW-SM	A-1-b
13A	68.5-69	17	----	----	----	100	92	84	74	66	32	5	SW-SM	A-1-b

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
13A	99-100	20	----	----	----	100	100	75	70	65	36	1	SP	A-1-b
13A	139-140	20	----	----	----	100	100	86	68	55	29	1	SP	A-1-b
14	4.5-5.5	30	34	19	15	100	100	100	100	100	100	95	CL	A-6
14	14-15	28	22	17	5	100	100	100	100	100	100	62	CL-ML	A-4
14	24-25	27	Non-plastic			100	100	100	100	100	100	36	SM	A-4
14	44-45	----	----	----	----	100	100	100	100	100	99	24	SM	A-2-4
14	58.5-59.5	----	----	----	----	100	100	100	100	100	90	11	SP-SM	A-2-4
14	100-118	----	----	----	----	100	100	88	50	29	14	2	GP	A-1-a
15	0.5-1.5	30	65	25	40	----	----	----	----	----	----	89	CH	A-7-6
15	2-2.5	24	56	24	32	----	----	----	----	----	----	94	CH	A-7-6
15	6.5-7.5	17	----	----	----	100	100	100	100	100	100	13	SM	A-2-4
15	24-25	23	Non-plastic			100	100	100	100	100	99	12	SP-SM	A-2-4
15	44-45	18	----	----	----	100	95	95	92	90	46	9	SP-SM	A-1-b
15	59-60	----	----	----	----	100	100	100	100	100	98	6	SP-SM	A-3

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
15	69-70	----	----	----	----	100	100	95	94	93	60	5	SP-SM	A-3
16	1-2	35	49	26	23	----	----	----	----	----	----	86	CL	A-7-6
16	2-2.5	39	79	27	52	----	----	----	----	----	----	99	CH	A-7-6
16	4.5-5	32	42	21	21	----	----	----	----	----	----	96	CL	A-7-6
16	9-10	7	----	----	----	100	100	100	100	100	99	29	SM	A-2-4
16	19-20	----	----	----	----	100	100	100	100	100	100	9	SP-SM	A-3
16	39-40	----	----	----	----	100	100	100	100	100	90	19	SM	A-2-4
16	49-50	----	----	----	----	100	100	100	99	98	48	4	SP	A-1-b
16	59-60	----	----	----	----	100	100	100	100	100	98	13	SM	A-2-4
16	69-70	----	----	----	----	100	100	93	90	82	39	4	SP	A-1-b
17	2.5-3.5	21	----	----	----	----	----	----	----	----	----	81	CL	A-7-6
17	4.5-5.5	27	46	22	24	----	----	----	----	----	----	88	CL	A-7-6
17	6.5-7.5	22	Non-plastic			----	----	----	----	----	----	42	SM	A-4
17	9-10	----	----	----	----	----	----	----	----	----	----	3	SP	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
17	14-15	----	----	----	----	----	----	----	----	----	----	5	SP-SM	A-3
17	29-30	----	----	----	----	100	100	100	100	100	86	9	SP-SM	A-3
17	44-45	----	----	----	----	100	100	100	100	100	87	6	SP-SM	A-3
17	54-55	----	----	----	----	100	100	100	100	100	98	17	SM	A-2-4
17	68.5-69	----	----	----	----	100	100	100	100	99	74	3	SP	A-3
17	83.5-84	----	----	----	----	100	94	78	59	41	18	3	SW	A-1-a
18	9-10	25	Non-plastic			----	----	----	----	----	----	51	ML	A-4
18	19-20	28	26	18	8	----	----	----	----	----	----	77	CL	A-4
18	29-30	49	51	24	27	----	----	----	----	----	----	99	CH	A-7-6
18	38.5-39.5	22	----	----	----	100	100	100	100	100	97	20	SM	A-2-4
18	68.5-69.5	19	----	----	----	100	100	94	90	89	84	7	SP-SM	A-3
18	88.5-89.5	15	----	----	----	100	100	78	35	19	11	3	GP	A-1-a
19	6.5-7	24	40	18	22	100	100	100	100	100	99	96	CL	A-6
19	24-25	22	----	----	----	100	100	100	100	100	90	8	SP-SM	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
19	43.5-44.5	22	----	----	----	100	100	100	100	100	82	8	SP-SM	A-3
19	68.5-69.5	26	----	----	----	100	100	100	100	99	69	4	SP	A-3
19	78.5-79.5	11	----	----	----	100	100	96	79	58	35	8	SP-SM	A-1-b
20	9-10	25	19	16	3	----	----	----	----	----	----	52	ML	A-4
20	14-14.5	29	35	19	16	----	----	----	----	----	----	99	CL	A-6
20	19-19.5	53	64	26	38	----	----	----	----	----	----	99	CH	A-7-6
20	44-45	15	----	----	----	100	100	90	86	78	47	12	SP-SM	A-1-b
20	58.5-59.5	17	----	----	----	100	100	98	94	90	63	6	SP-SM	A-3
20	88.5-89.5	13	----	----	----	100	100	93	56	39	20	3	SW	A-1-a
21	6.5-7.5	25	26	15	11	----	----	----	----	----	----	46	SC	A-6
21	19-20	30	40	19	21	----	----	----	----	----	----	97	CL	A-6
21	44-45	18	----	----	----	100	100	100	97	87	55	4	SP	A-3
21	68.5-69.5	17	----	----	----	100	100	100	73	38	8	4	SP	A-1-b

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
22	2.5-3.5	30	48	22	26	----	----	----	----	----	----	99	CL	A-7-6
22	9-10	36	37	19	18	----	----	----	----	----	----	95	CL	A-6
22	14-15	25	----	----	----	100	100	100	100	100	99	13	SM	A-2-4
22	33.5-34.5	24	----	----	----	100	100	100	100	100	100	9	SP-SM	A-3
22	48.5-49.5	24	----	----	----	100	100	100	100	100	100	6	SP-SM	A-3
22	78.5-79.5	22	----	----	----	100	100	100	100	100	92	4	SP	A-3
22	88.5-89	15	----	----	----	100	100	87	71	65	46	2	SP	A-1-b
23	2.5-3.5	14	33	20	13	----	----	----	----	----	----	87	CL	A-6
23	7-7.5	21	44	19	25	----	----	----	----	----	----	97	CL	A-7-6
23	9-10	21	27	18	9	----	----	----	----	----	----	71	CL	A-4
23	14-15	31	----	----	----	100	100	100	100	99	95	76	ML	A-4
23	24-25	22	----	----	----	100	100	100	100	100	95	4	SP	A-3
23	44-45	21	----	----	----	100	100	100	100	99	75	6	SP-SM	A-3
23	68.5-69.5	19	----	----	----	100	94	87	86	85	75	3	SP	A-3

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
24	6.5-7.5	19	36	22	14	----	----	----	----	----	----	88	CL	A-6
24	9-10	17	----	----	----	----	----	----	----	----	----	67	ML	A-4
24	19-20	22	----	----	----	100	100	96	96	96	96	7	SP-SM	A-3
24	64-65	23	----	----	----	100	100	100	100	100	92	5	SP-SM	A-3
25	9-10	14	----	----	----	----	----	----	----	----	----	35	SM	A-2-4
25	19-20	26	----	----	----	100	100	100	100	100	98	15	SM	A-2-4
25	29-30	24	----	----	----	100	100	100	100	100	100	10	SP-SM	A-3
25	48.5-49.5	24	----	----	----	100	100	100	100	99	96	4	SP	A-3
29	1.5-2.5	18	30	25	5	----	----	----	----	----	----	82	CL-ML	A-4
29	5-5.5	26	64	20	44	----	----	----	----	----	----	96	CH	A-7-6
29	14.5-15	30	70	24	46	----	----	----	----	----	----	99	CH	A-7-6
29	19.5-20	26	28	21	7	----	----	----	----	----	----	97	CL-ML	A-4
29	24.5-25	21	48	17	31	----	----	----	----	----	----	97	CL	A-7-6
29	29.5-30	24	38	19	19	----	----	----	----	----	----	97	CL	A-6

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-40 over White River - AHTD Job #BB0610
LOCATION: Prairie County, AR
JOB NUMBER: 13-017

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		
30	9.5-10	21	50	18	32	----	----	----	----	----	----	87	CH	A-7-6
30	14.5-15	23	49	19	30	----	----	----	----	----	----	95	CL	A-7-6
30	29.5-30	33	85	26	59	----	----	----	----	----	----	99	CH	A-7-6
30	34-34.5	33	32	30	2	----	----	----	----	----	----	99	ML	A-4

GRUBBS, HOSKYN, BARTON & WYATT, INC.
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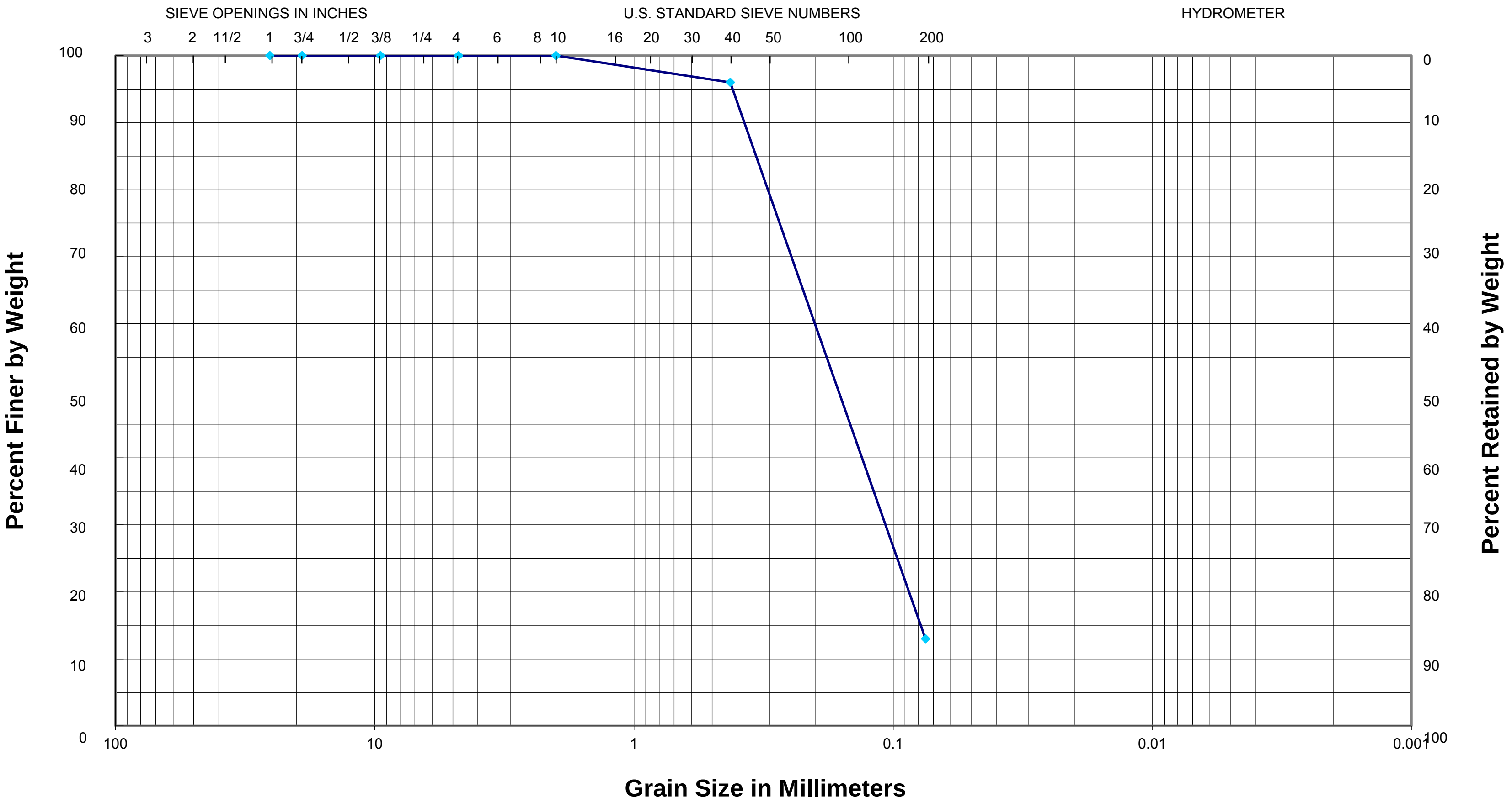
13-017

GRAIN SIZE CURVE



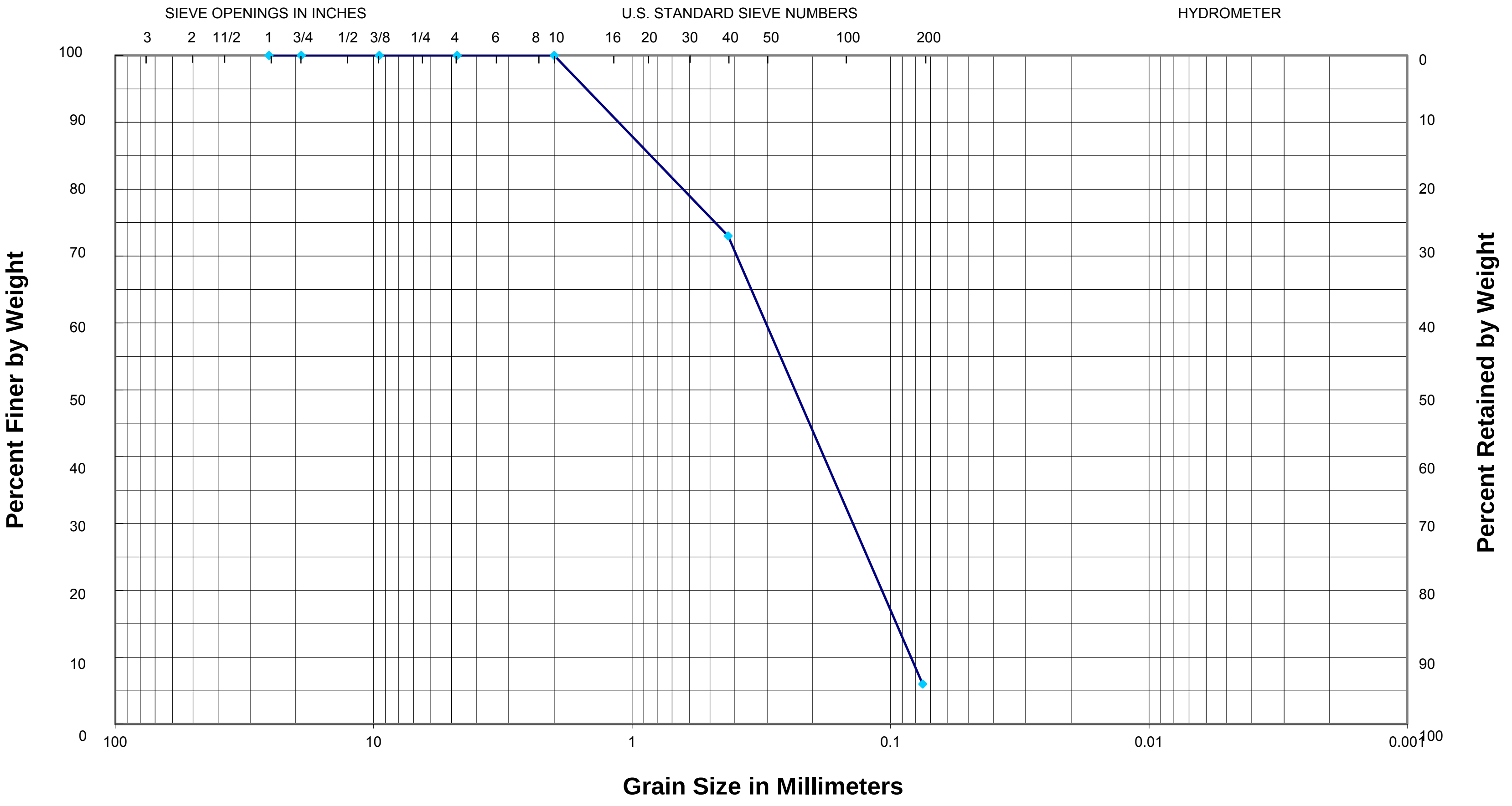
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



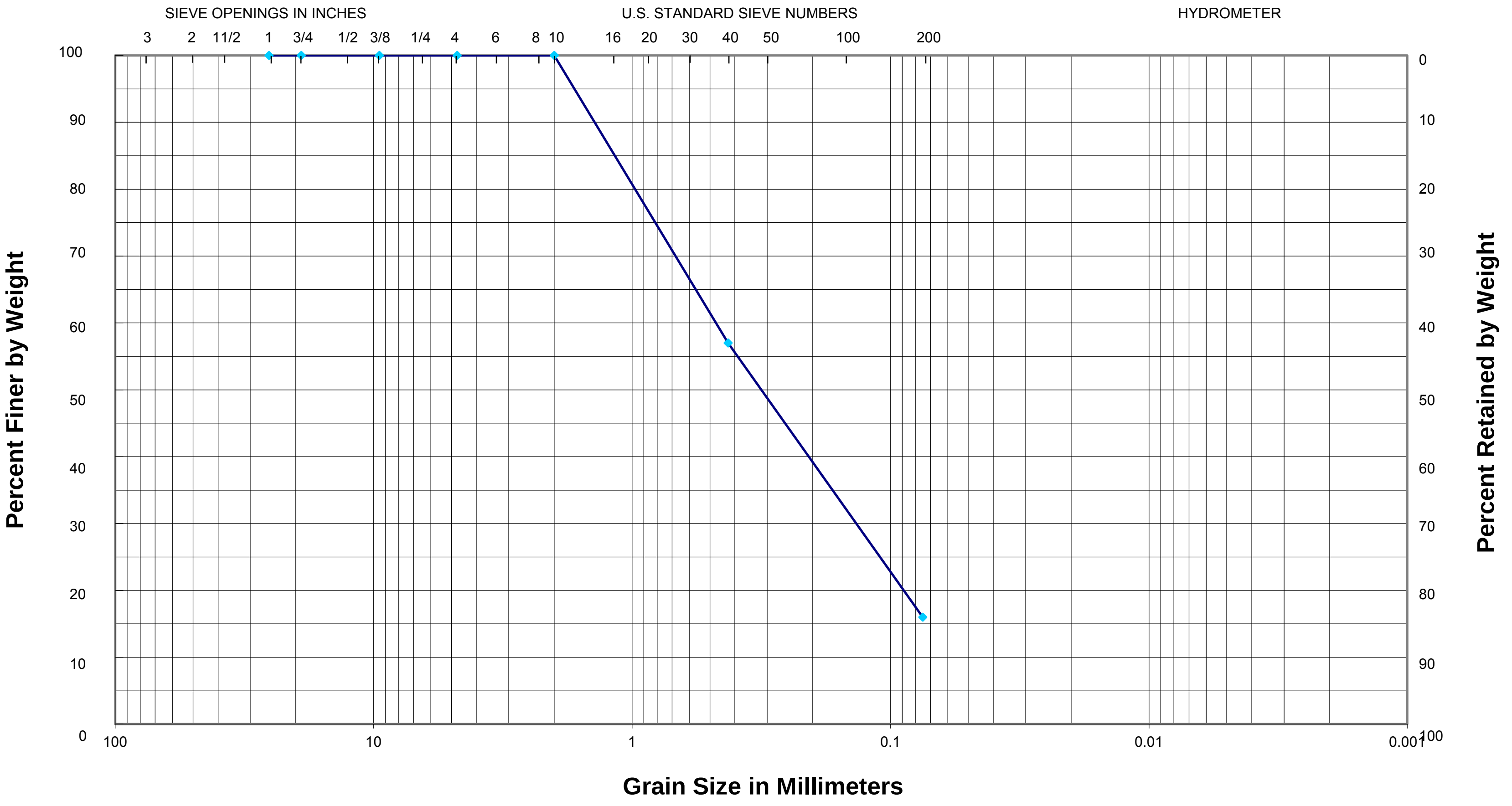
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 34-35 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



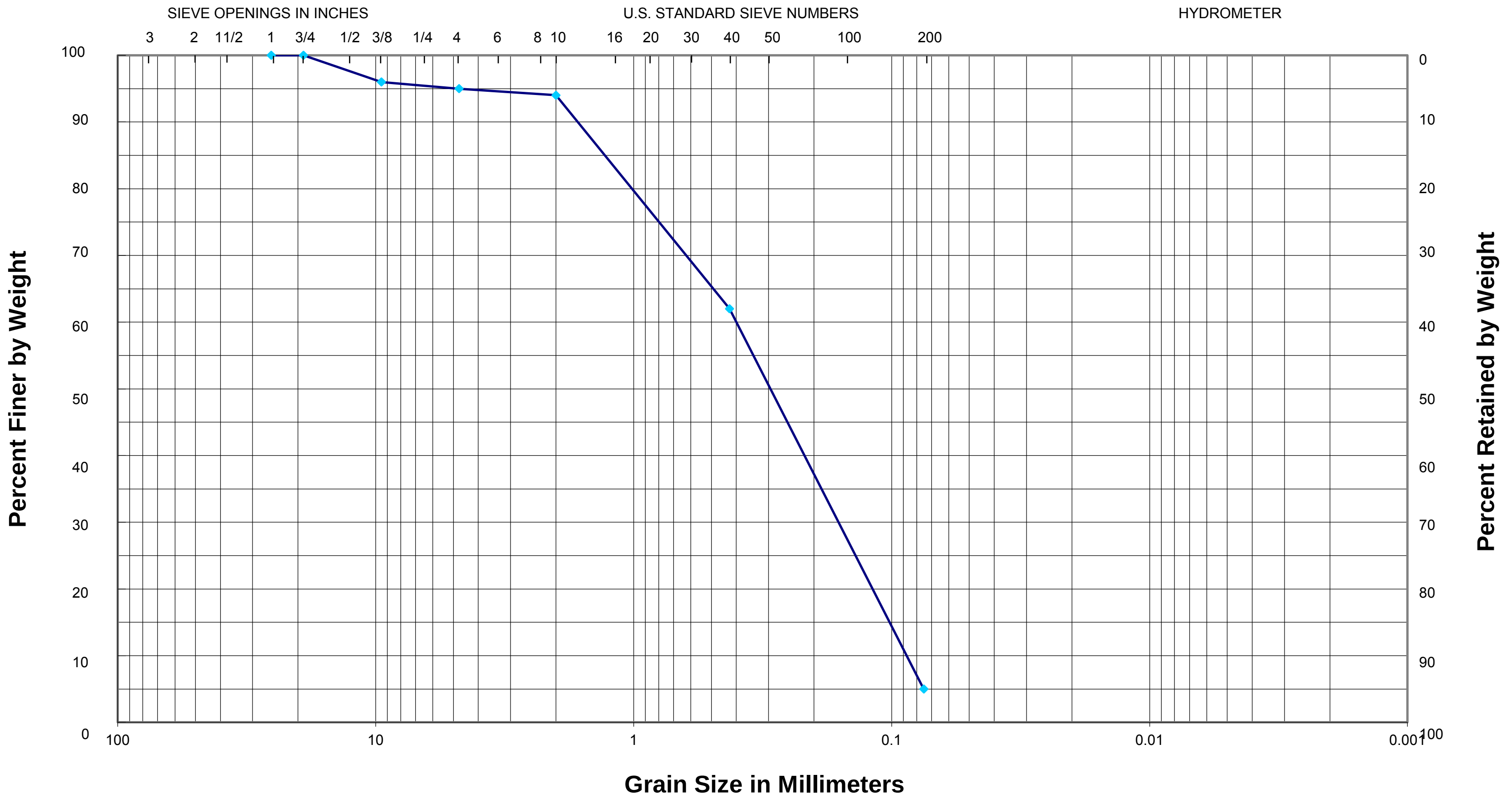
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 39-40 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand w/clayey fine sand seams
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



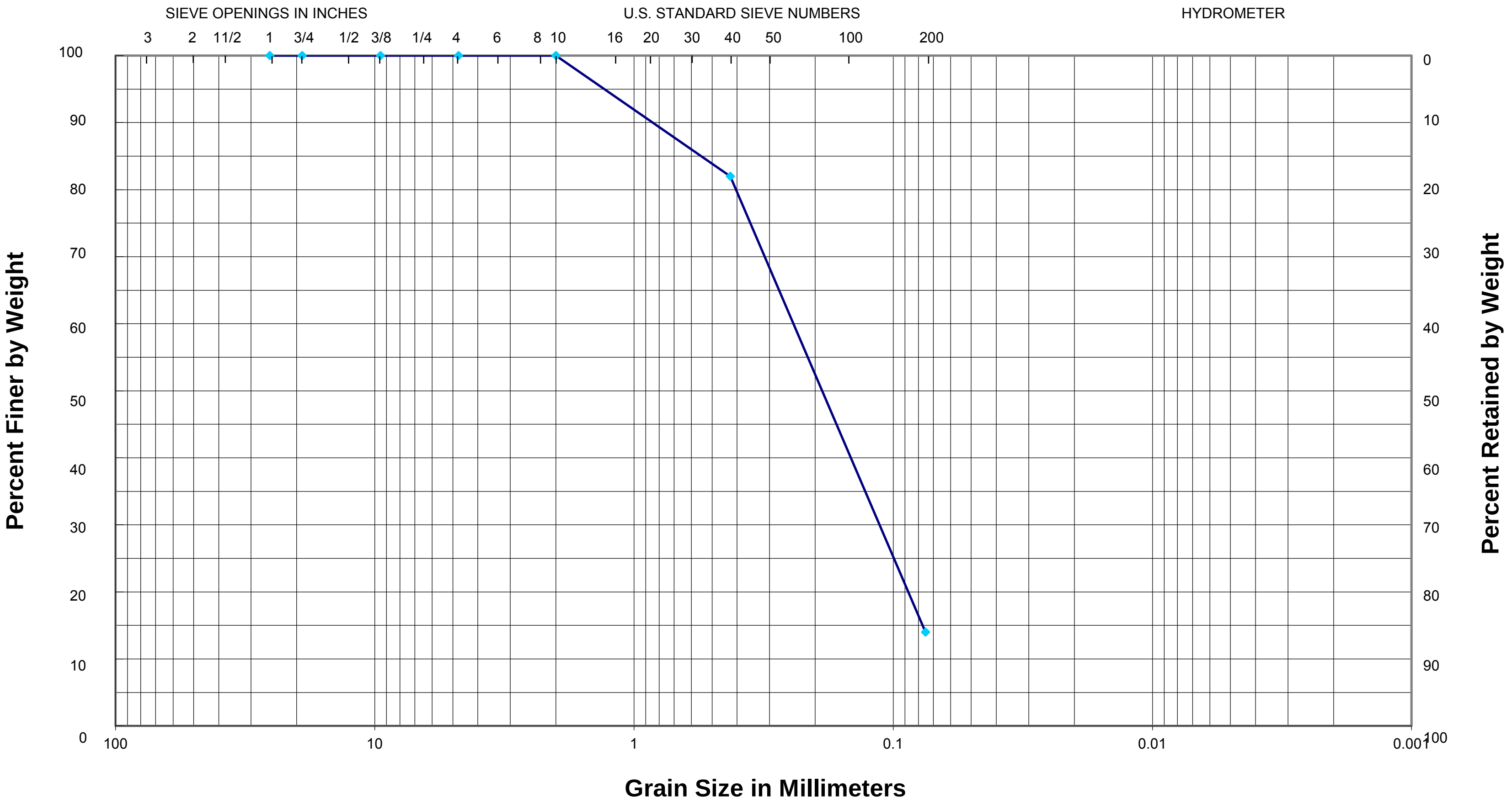
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 54-55 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



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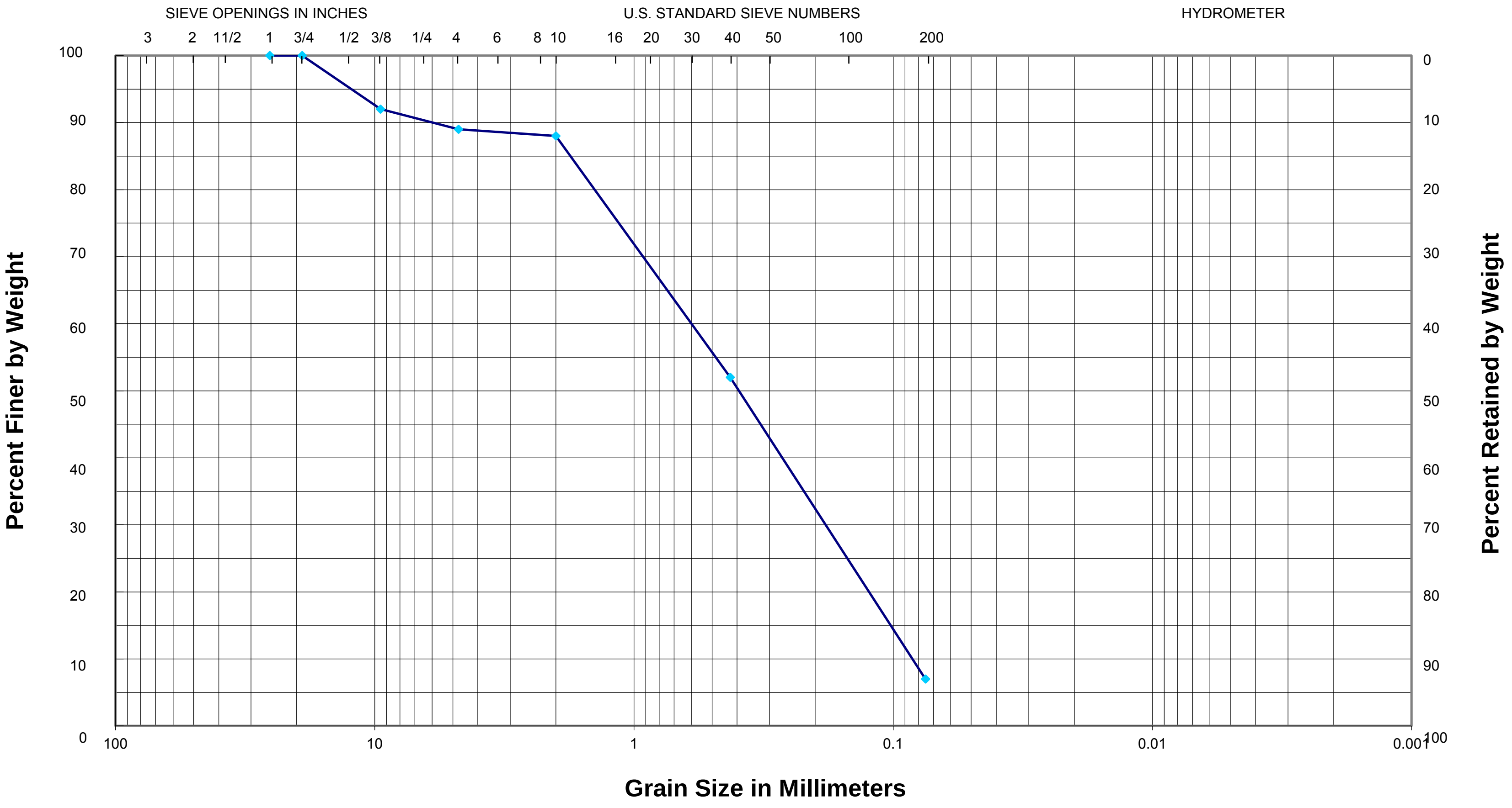
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Sample: Boring 2, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



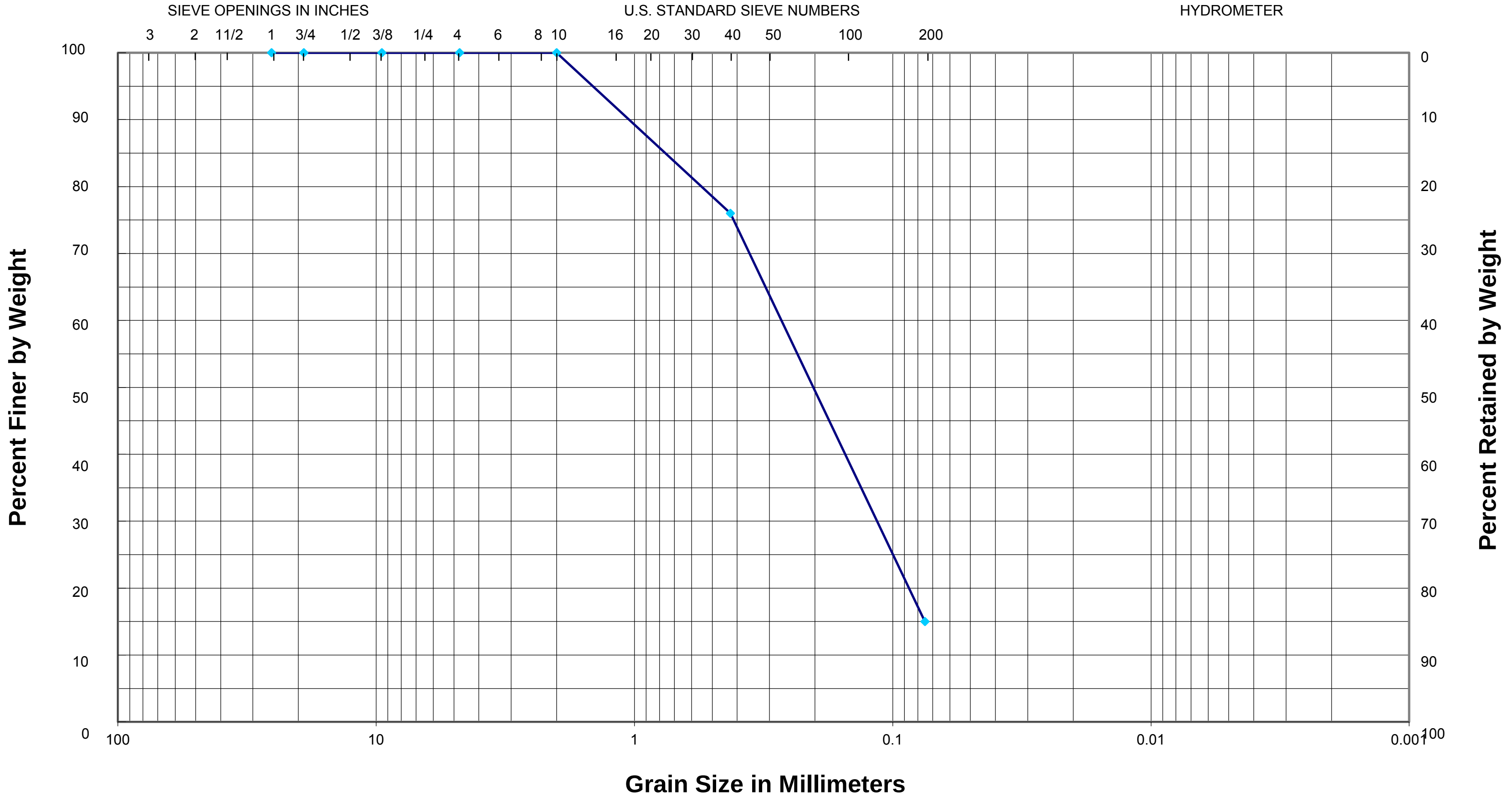
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 2, 59-60 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



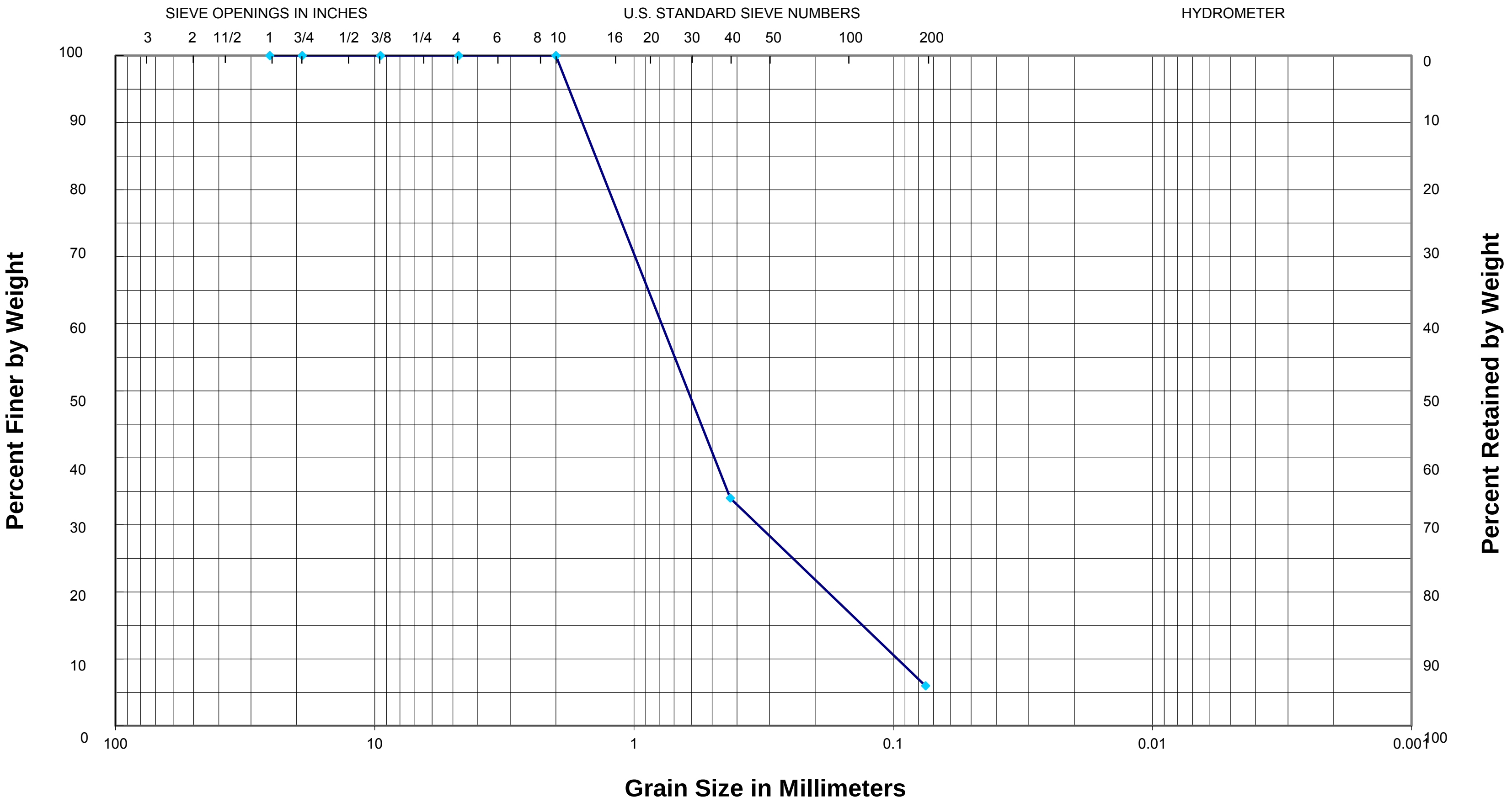
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 3, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



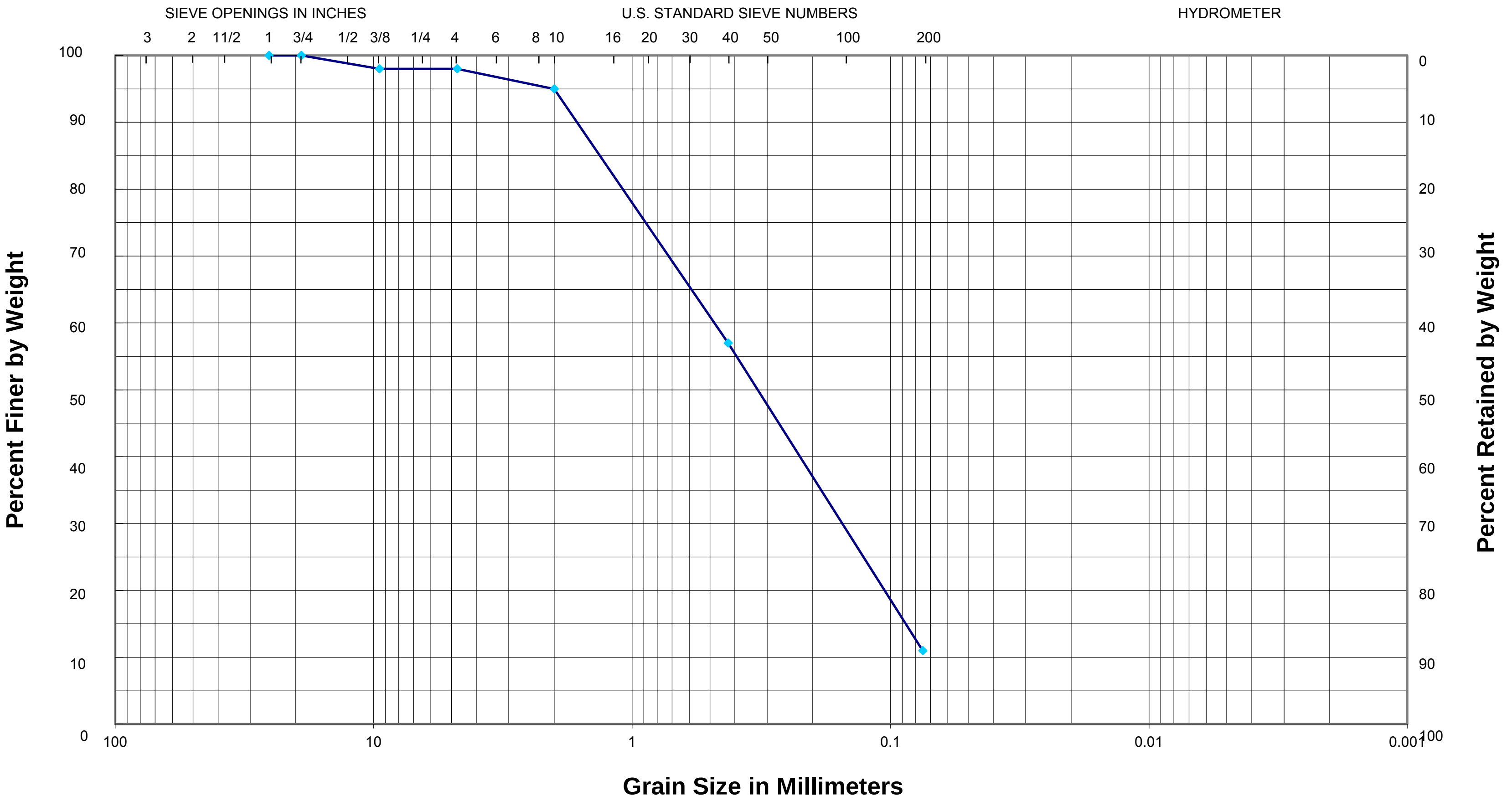
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 3, 39-40 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

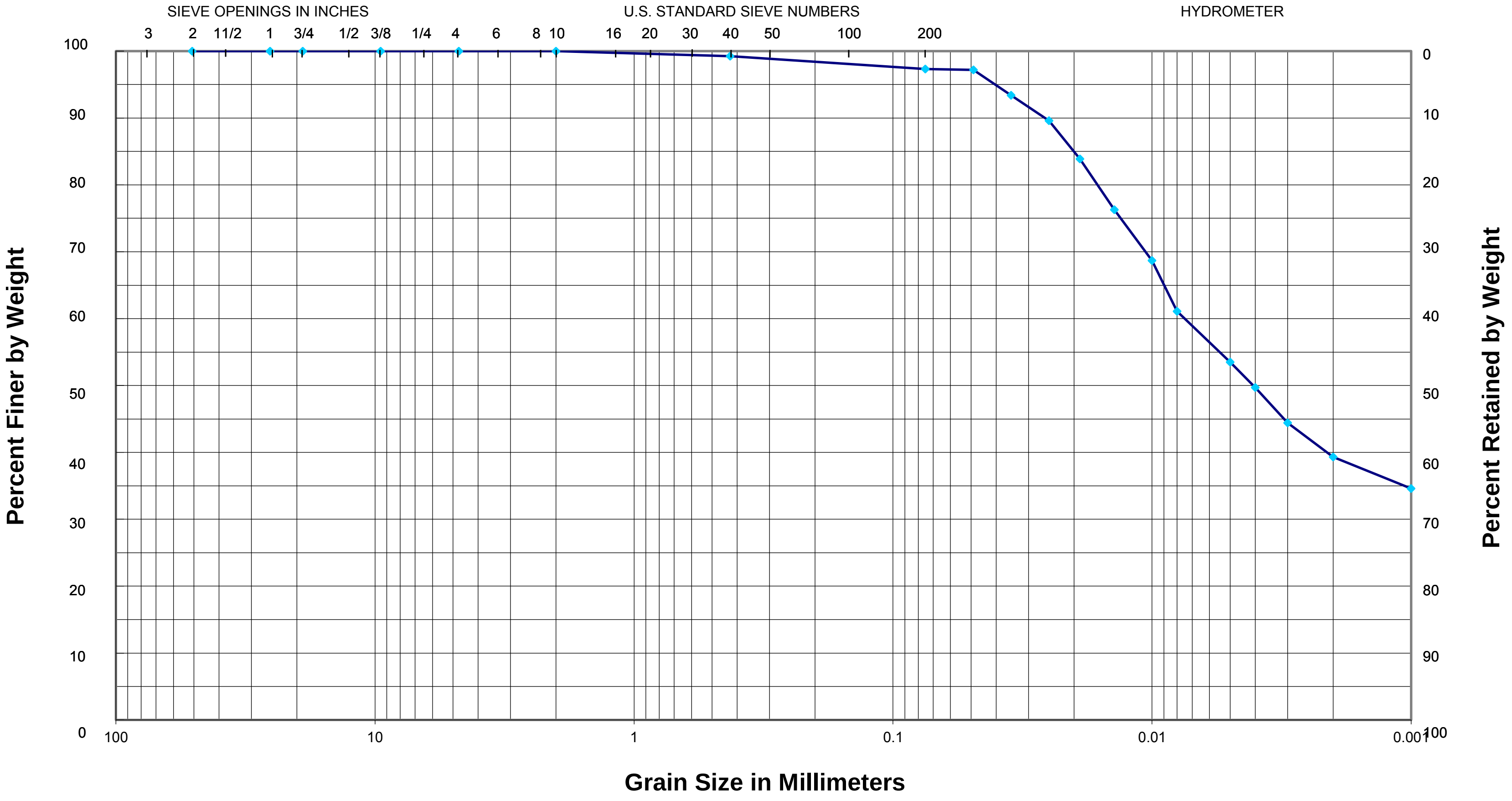
Sample: Boring 3, 49-50 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

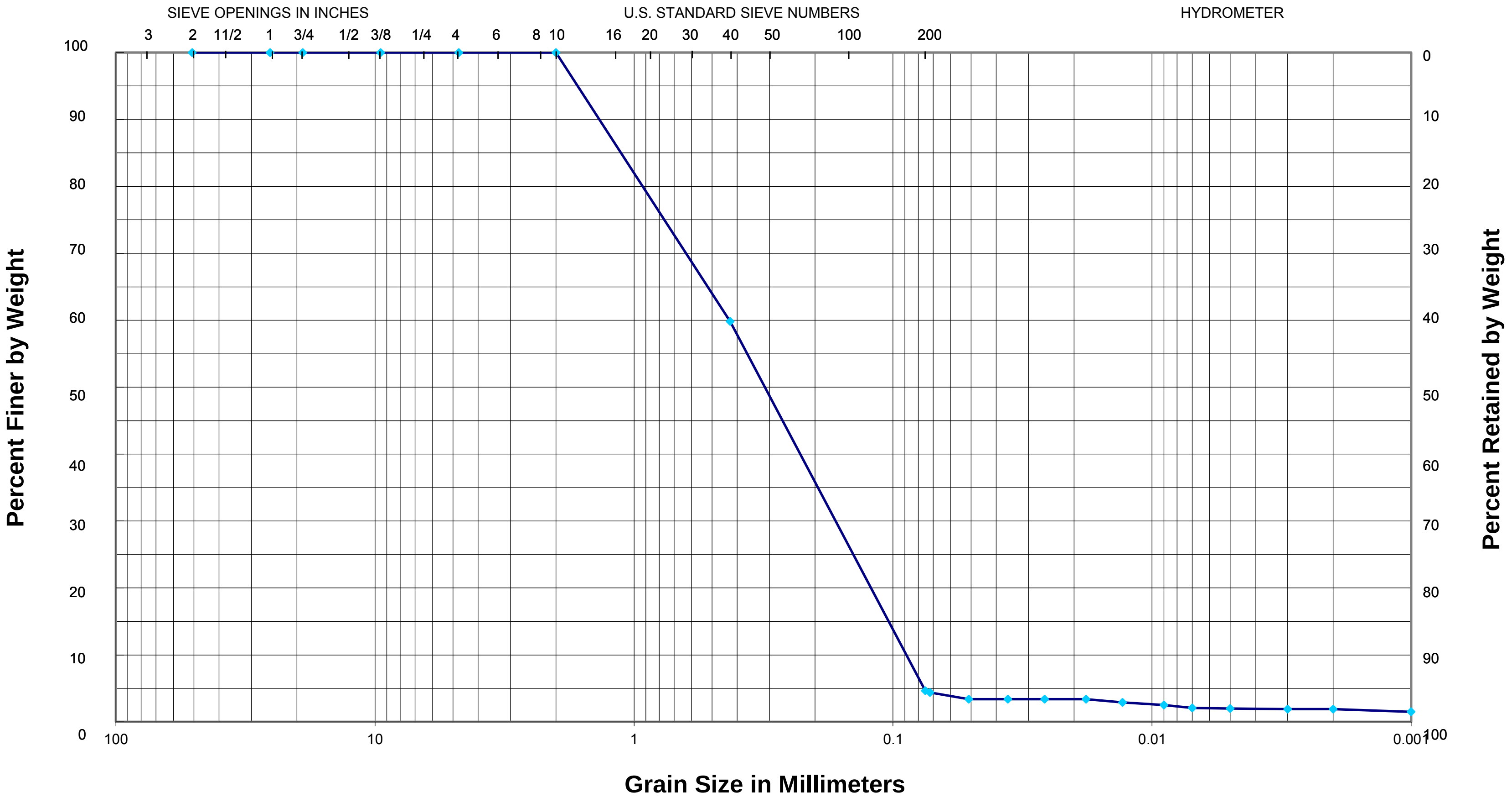
Sample: Boring 4, 7-7.5 ft
Properties: G_s = 2.700; LL = 46, PL = 19, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 4, 14-15 ft
Properties: G_s = 2.653; Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

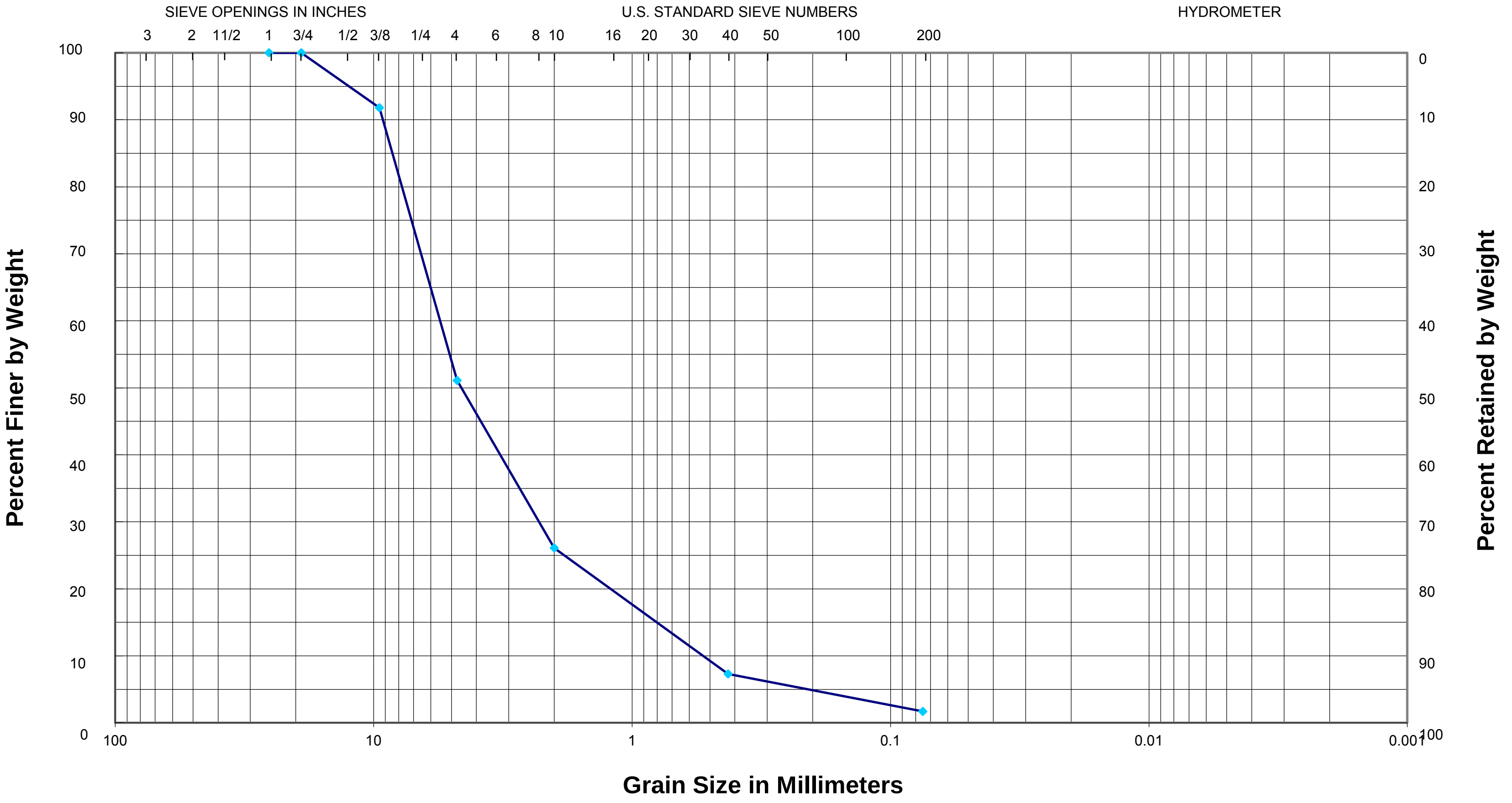
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 4, 53.5-54 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand w/a little fine gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

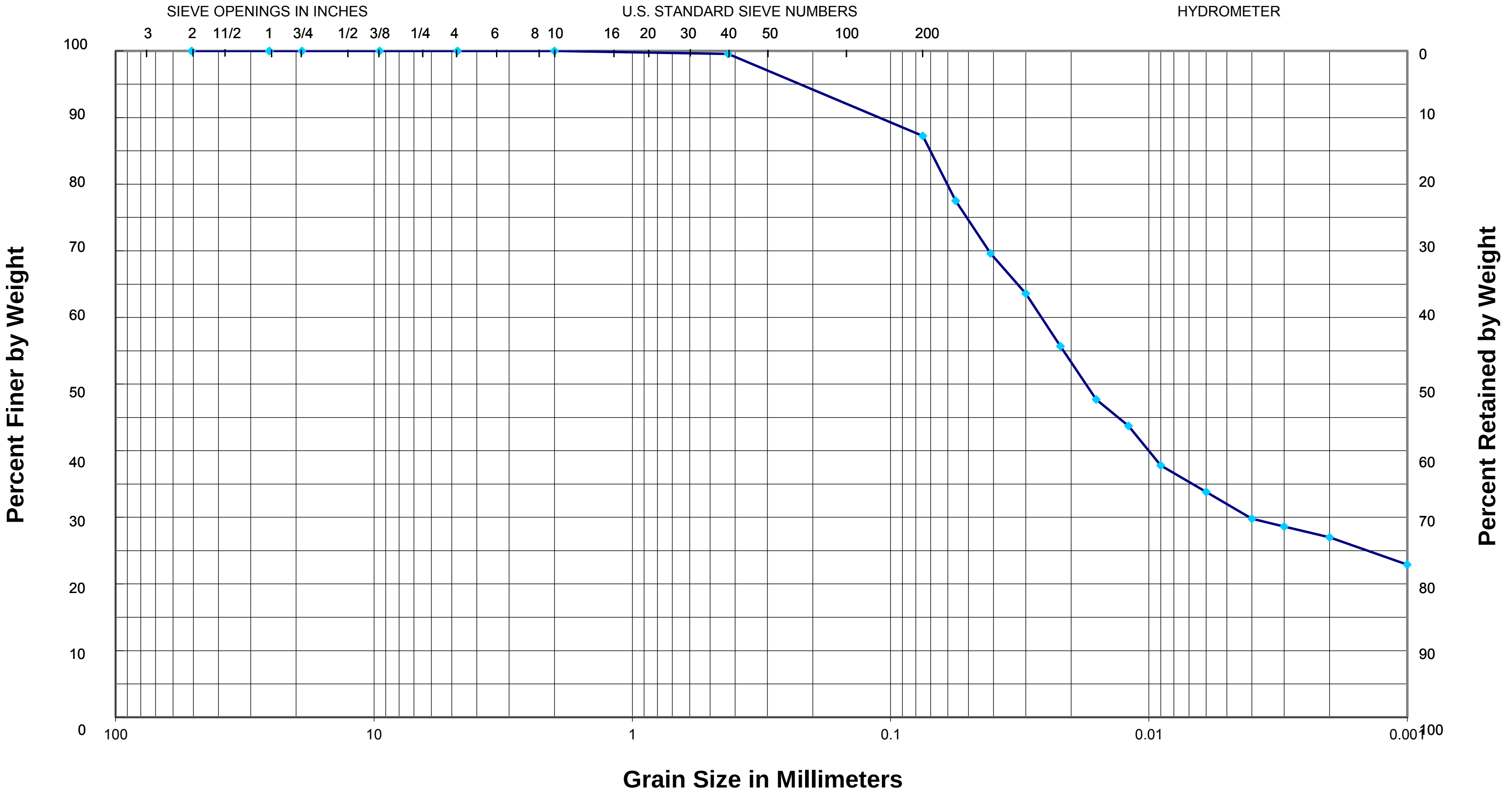
Sample: Boring 4, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Gray sandy fine gravel
Classification: USCS = GW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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



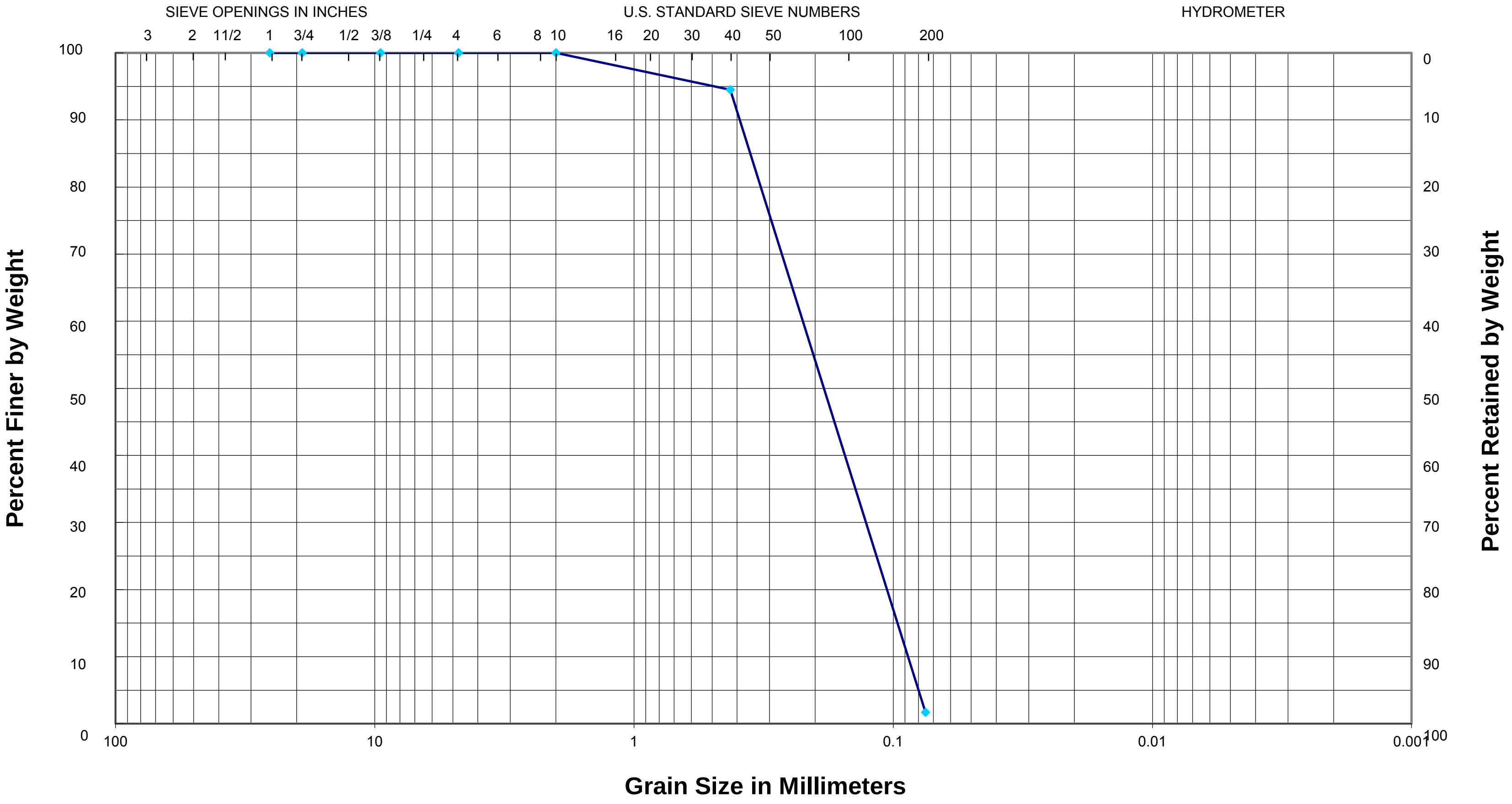
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 5, 3-3.5 ft
Properties: G_s = 2.637; LL = 33, PL = 17, PI = 16

Description: Dark gray and brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



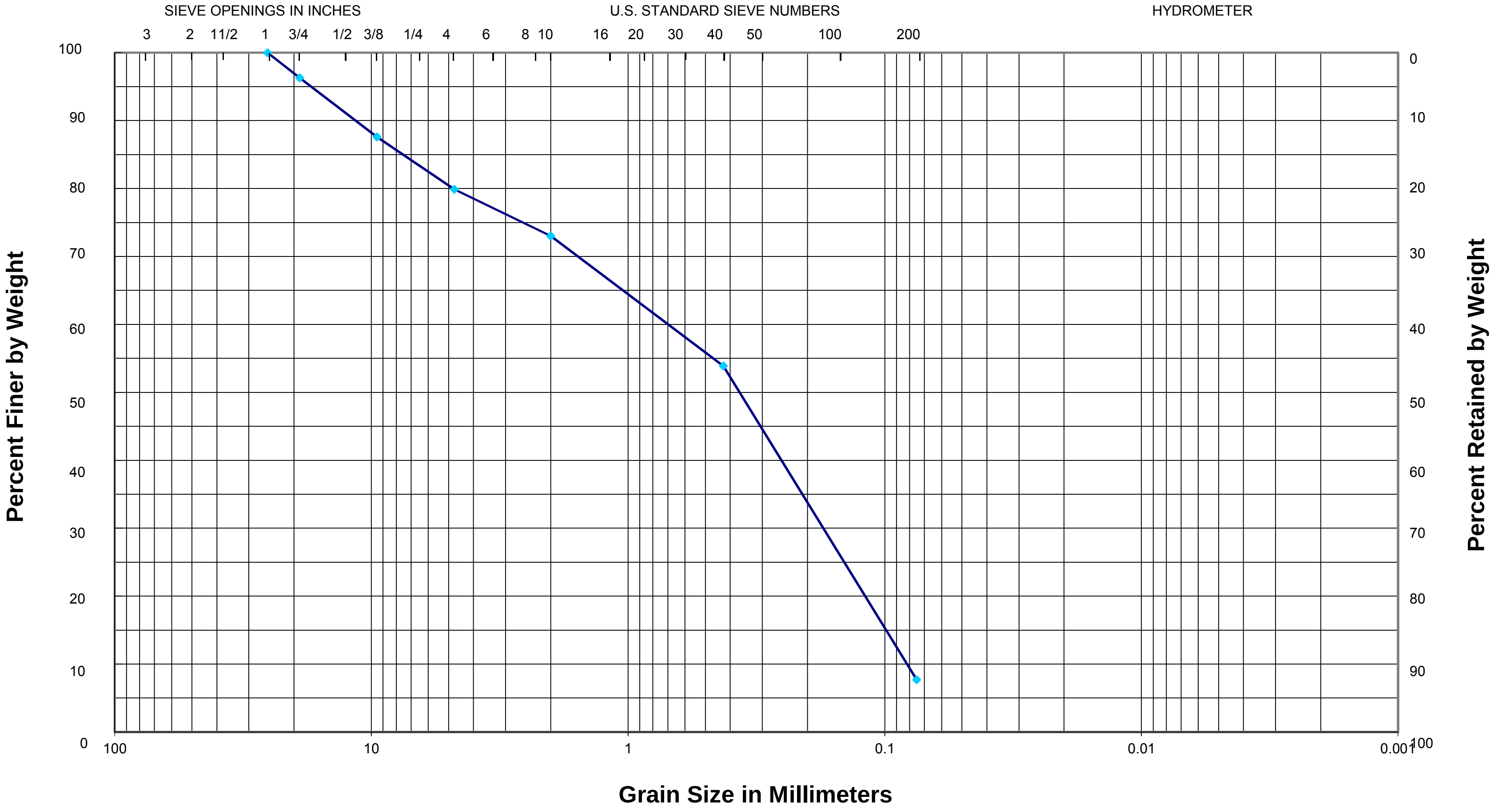
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 5, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray and tan fine sand w/trace medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

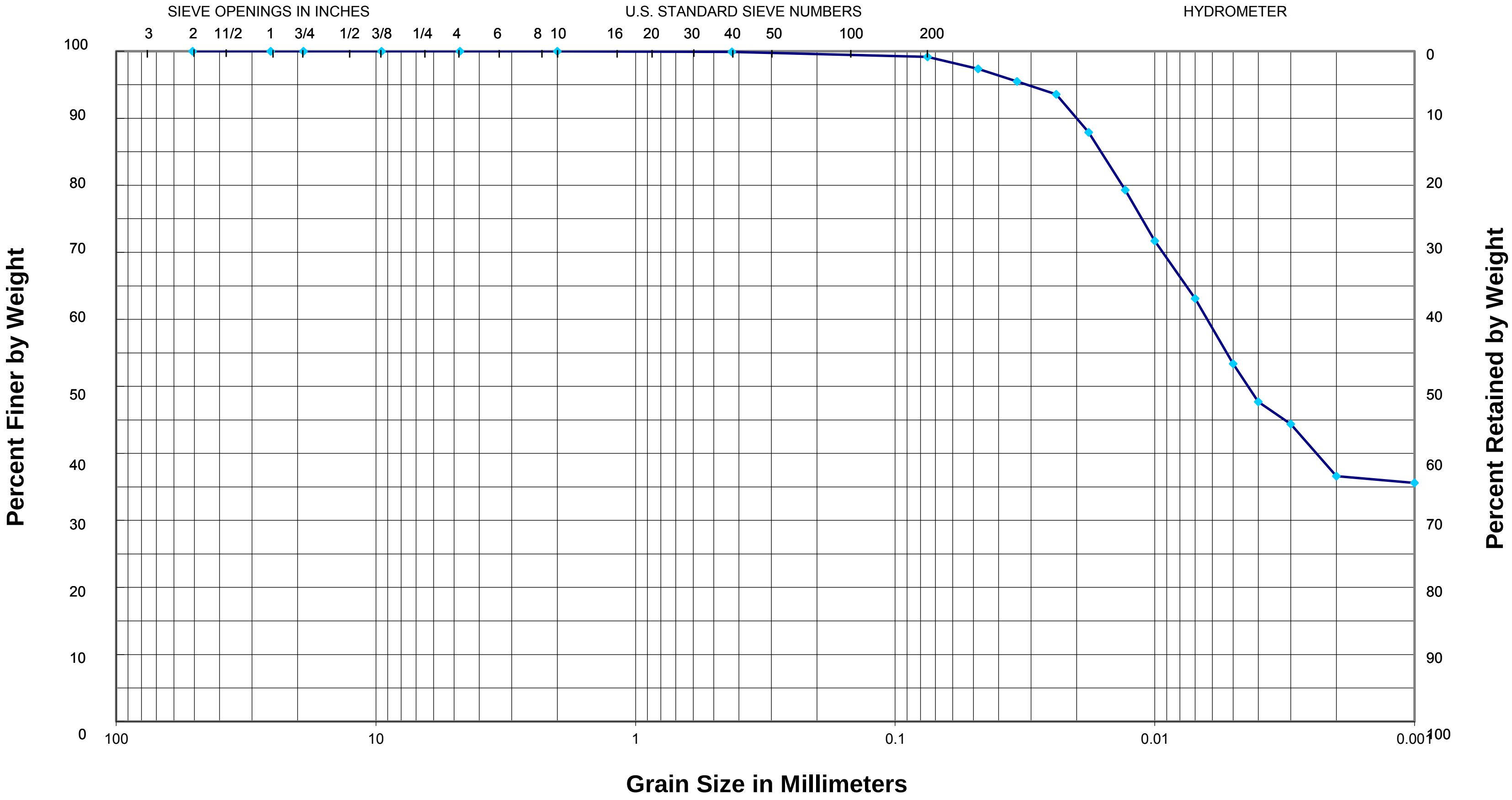
Sample: Boring 5, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/fine to coarse gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



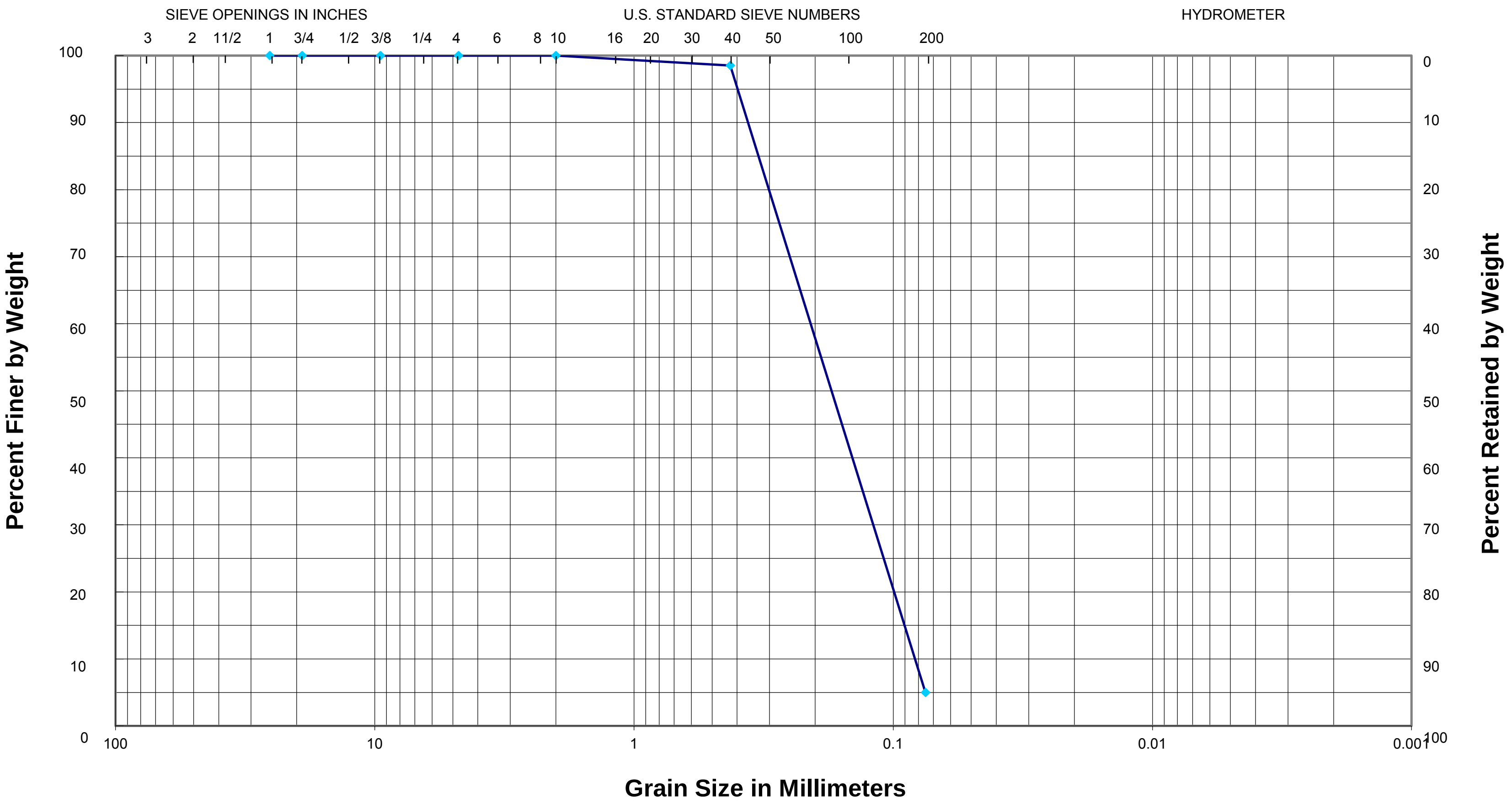
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 6, 3-3.5 ft
Properties: G_s = 2.707; LL = 48, PL = 21, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE



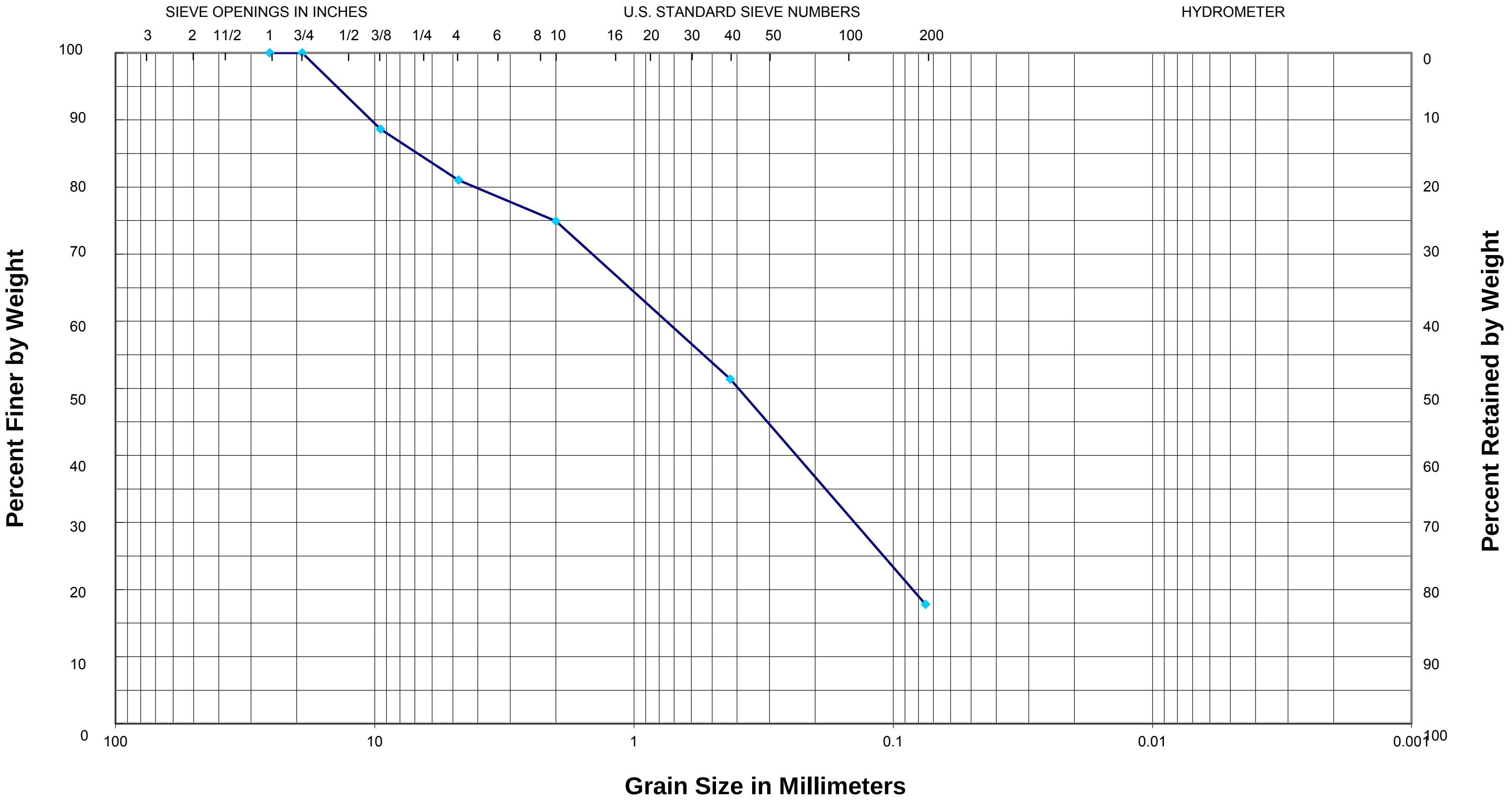
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 6, 19-20 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



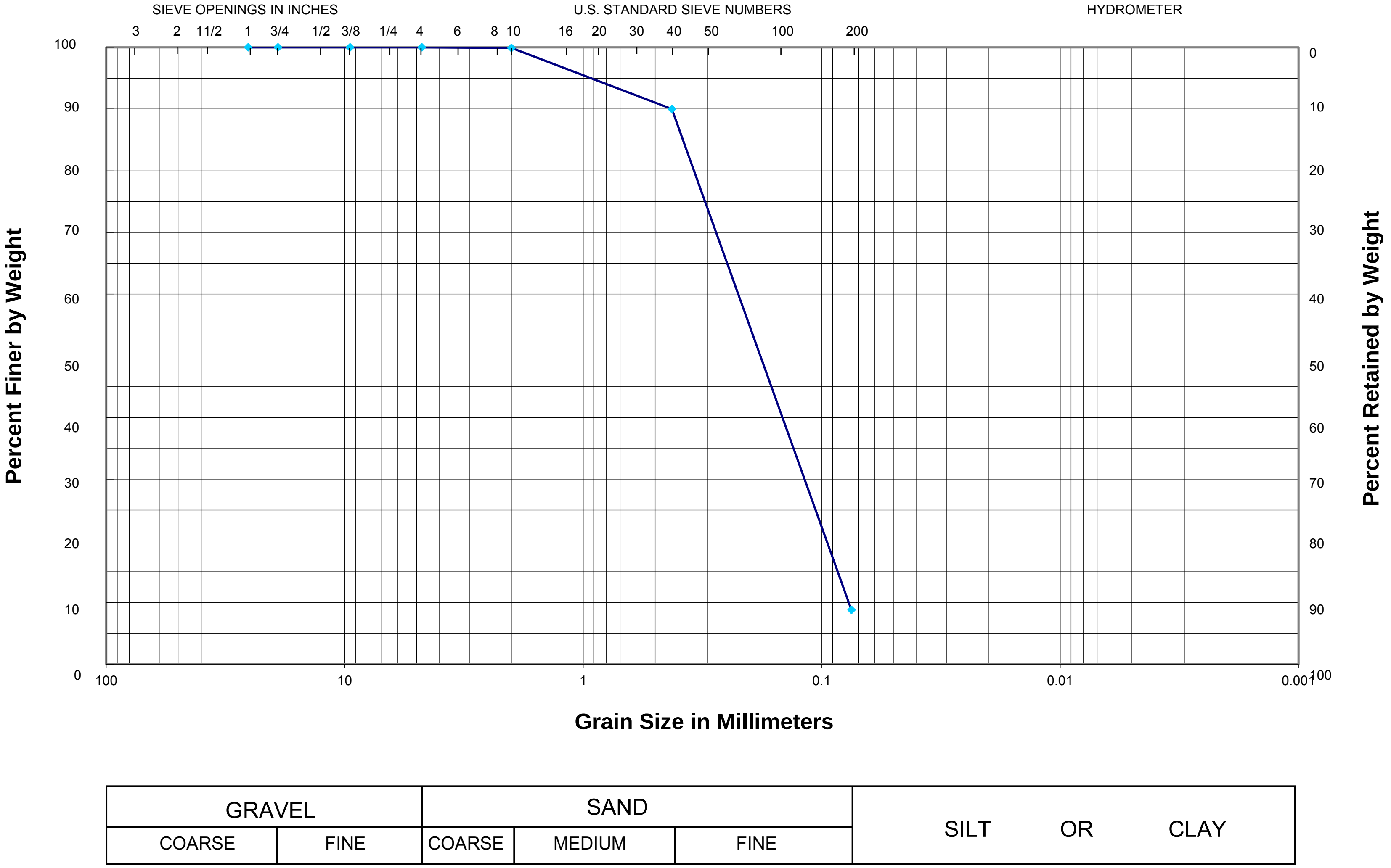
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 6, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand, silty w/some fine gravel
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

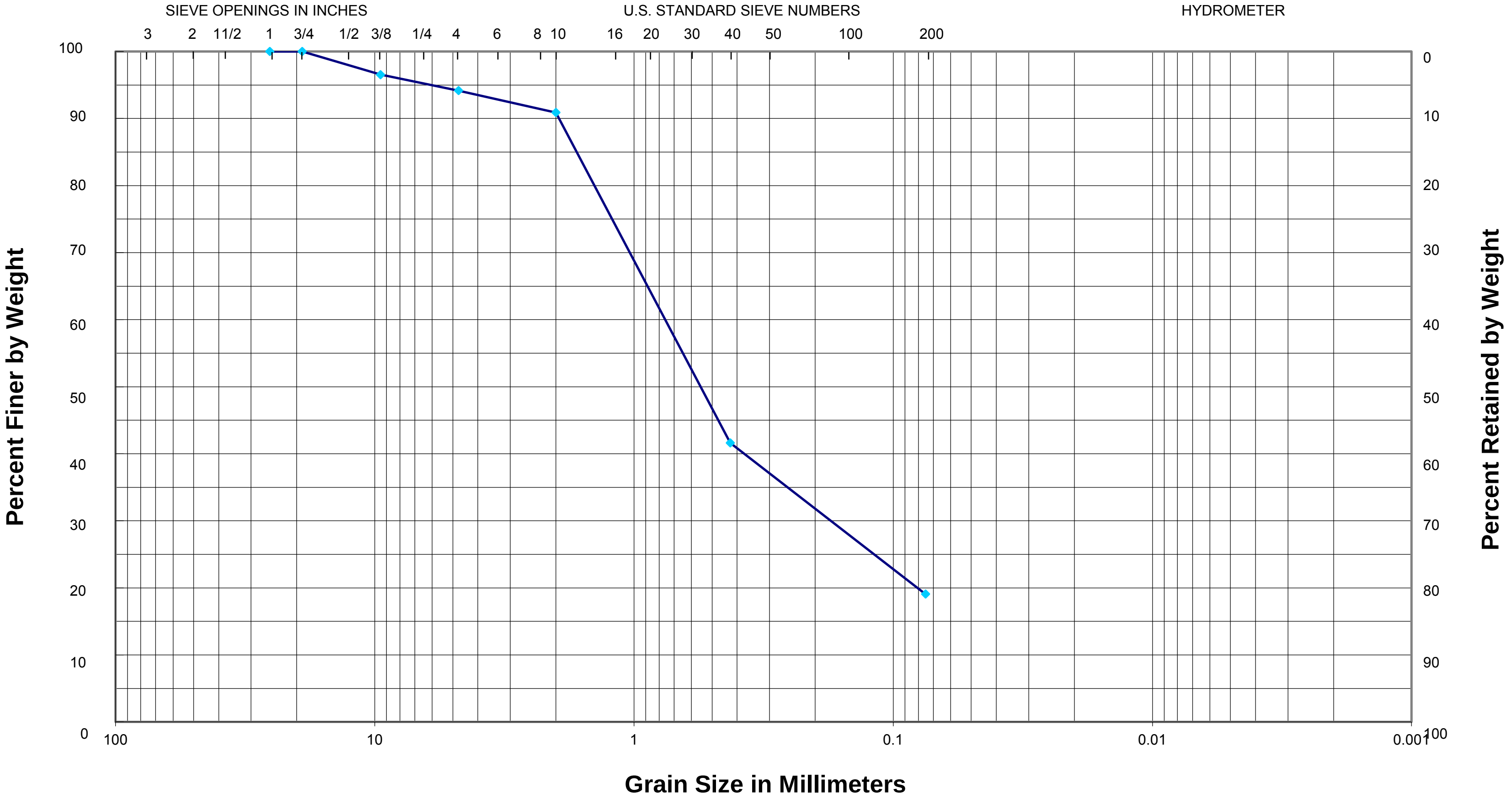


Sample: Boring 7, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



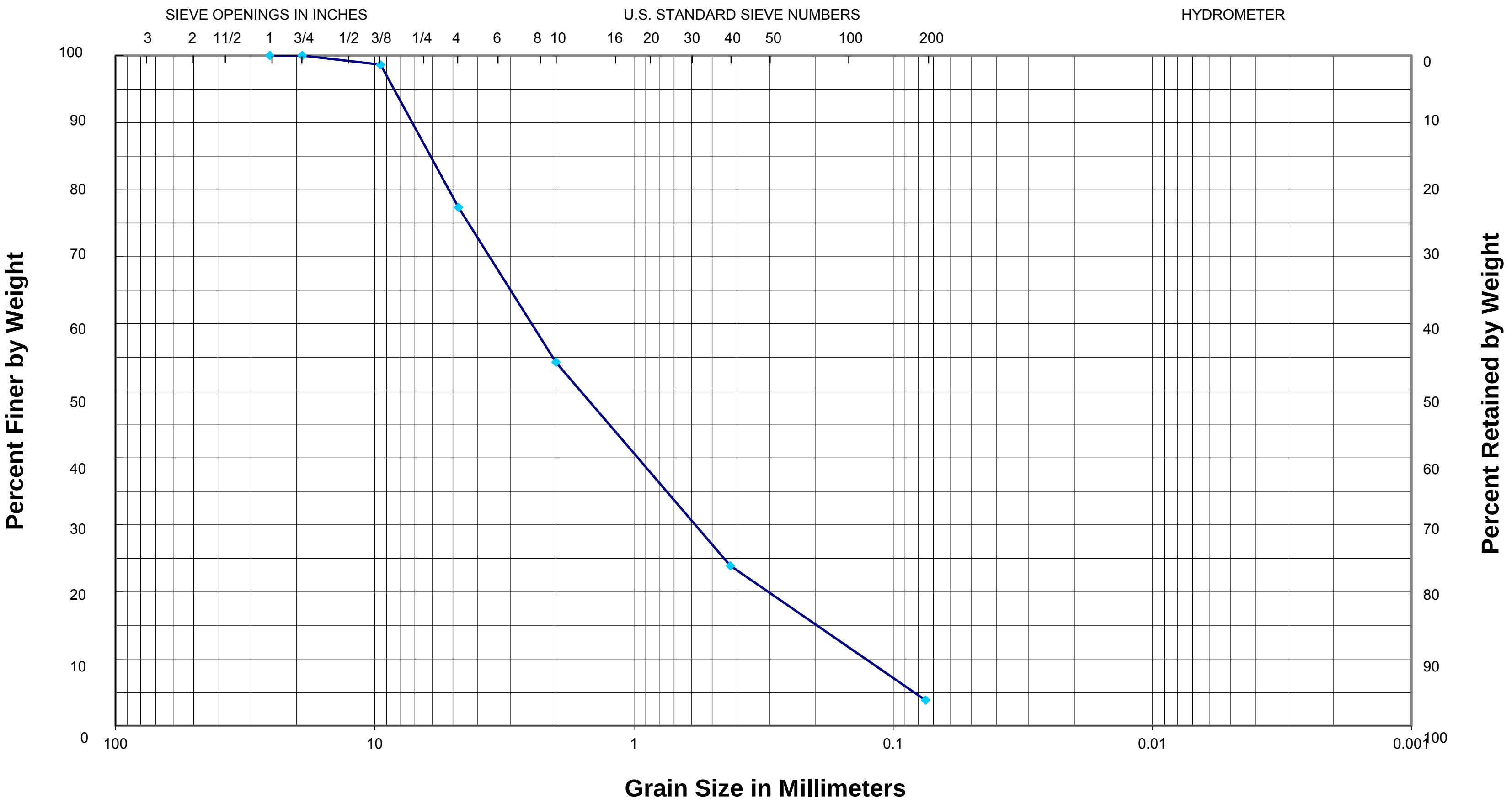
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 7, 43.5-44.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand, silty
Classification: USCS = SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 7, 88.5-89 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SW; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

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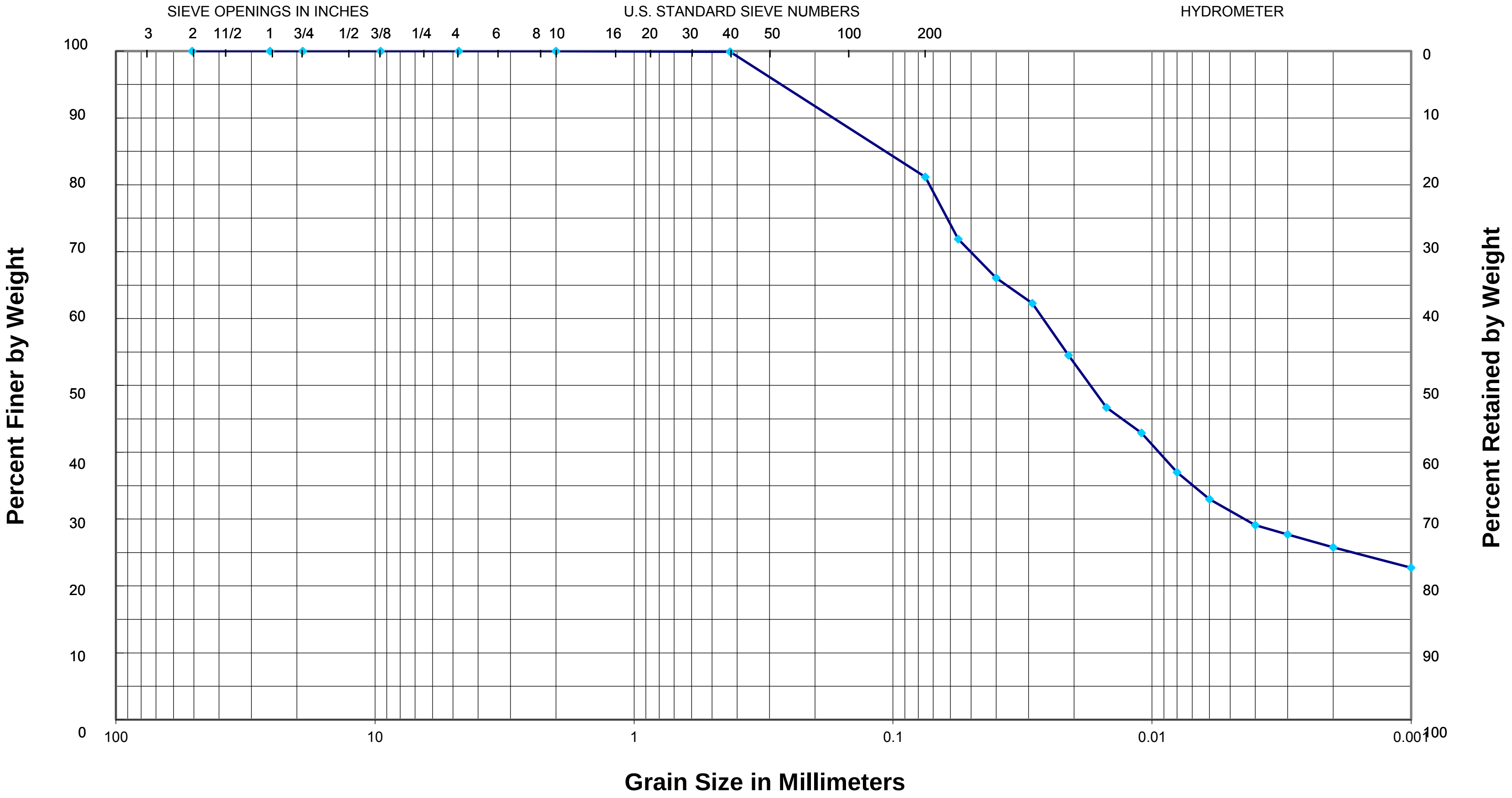


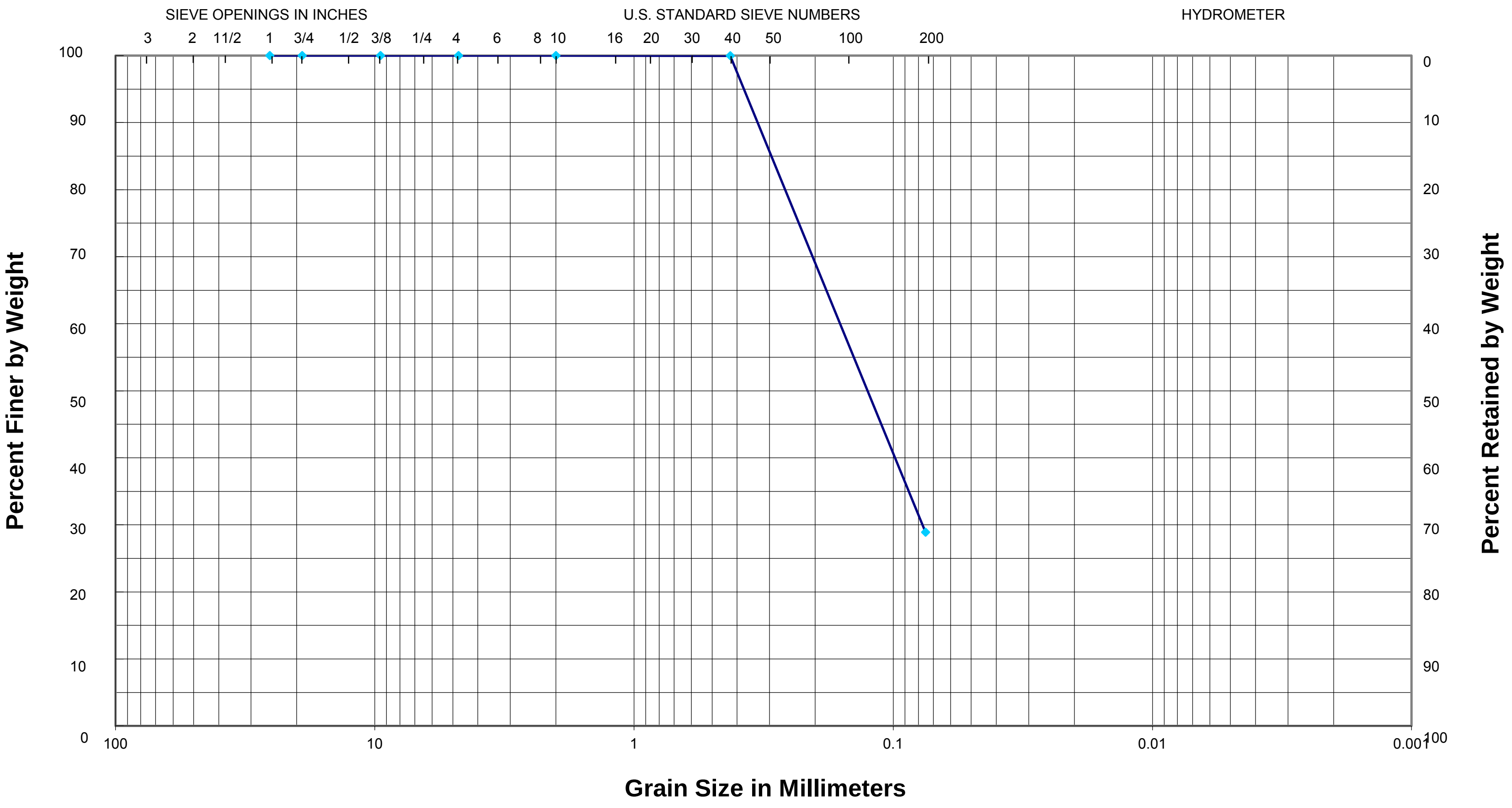
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 8, 7-7.5 ft
Properties: G_s = 2.697; LL = 37, PL = 15, PI = 12

Description: Gray and brown silty clay, sandy
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

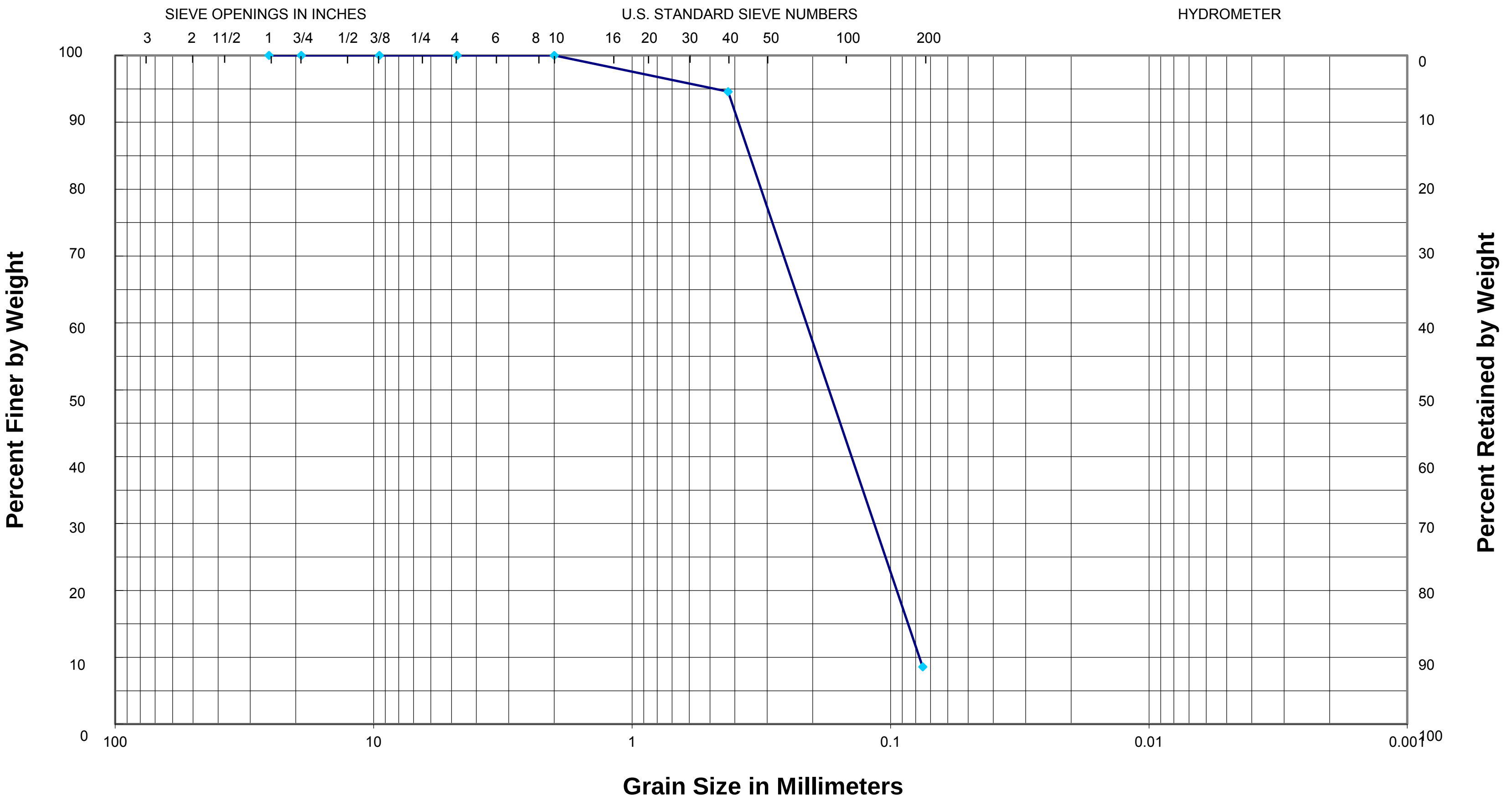
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine to coarse sand w/fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 9, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

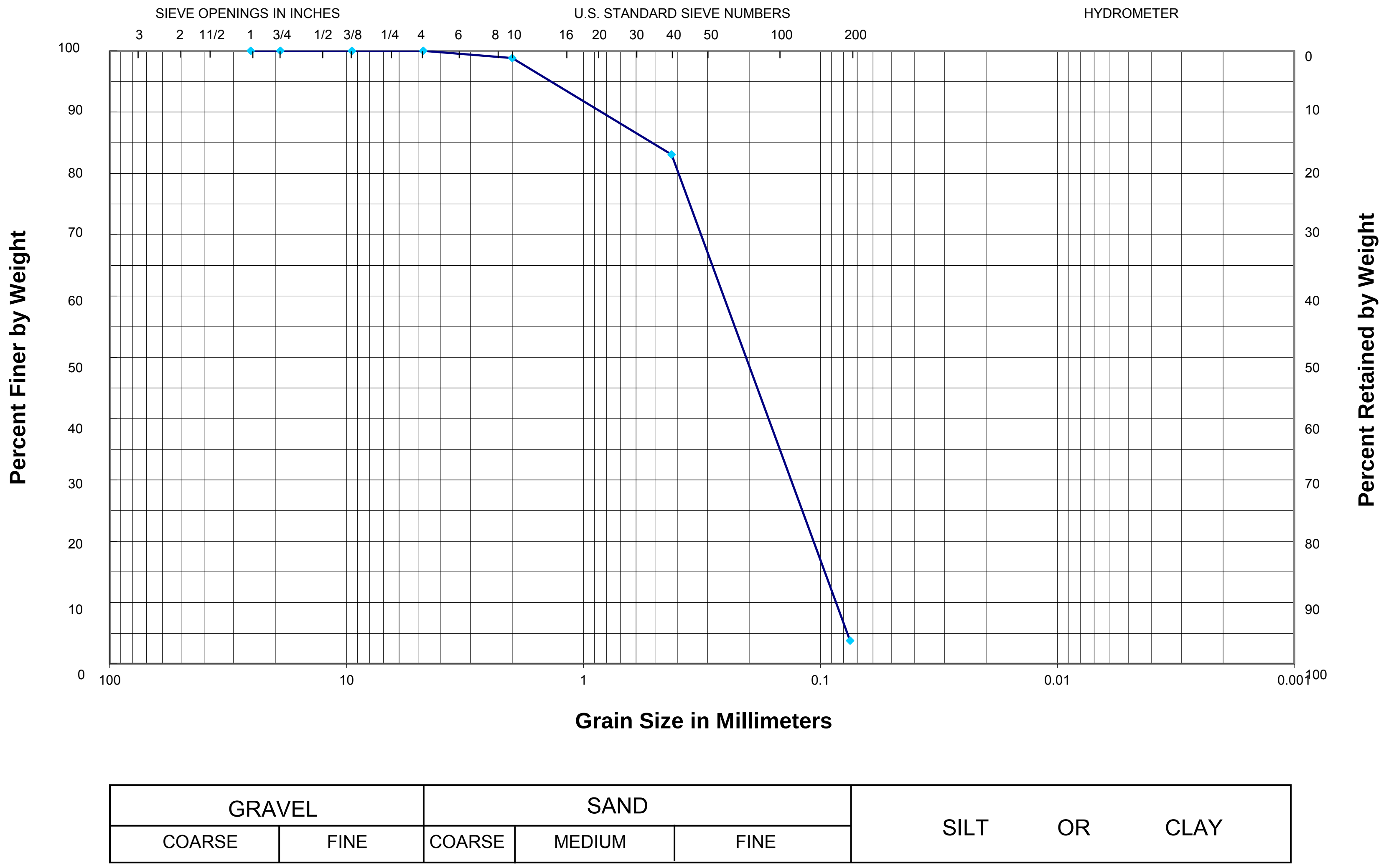
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 9, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/fine gravel
Classification: USCS = SP-SM; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

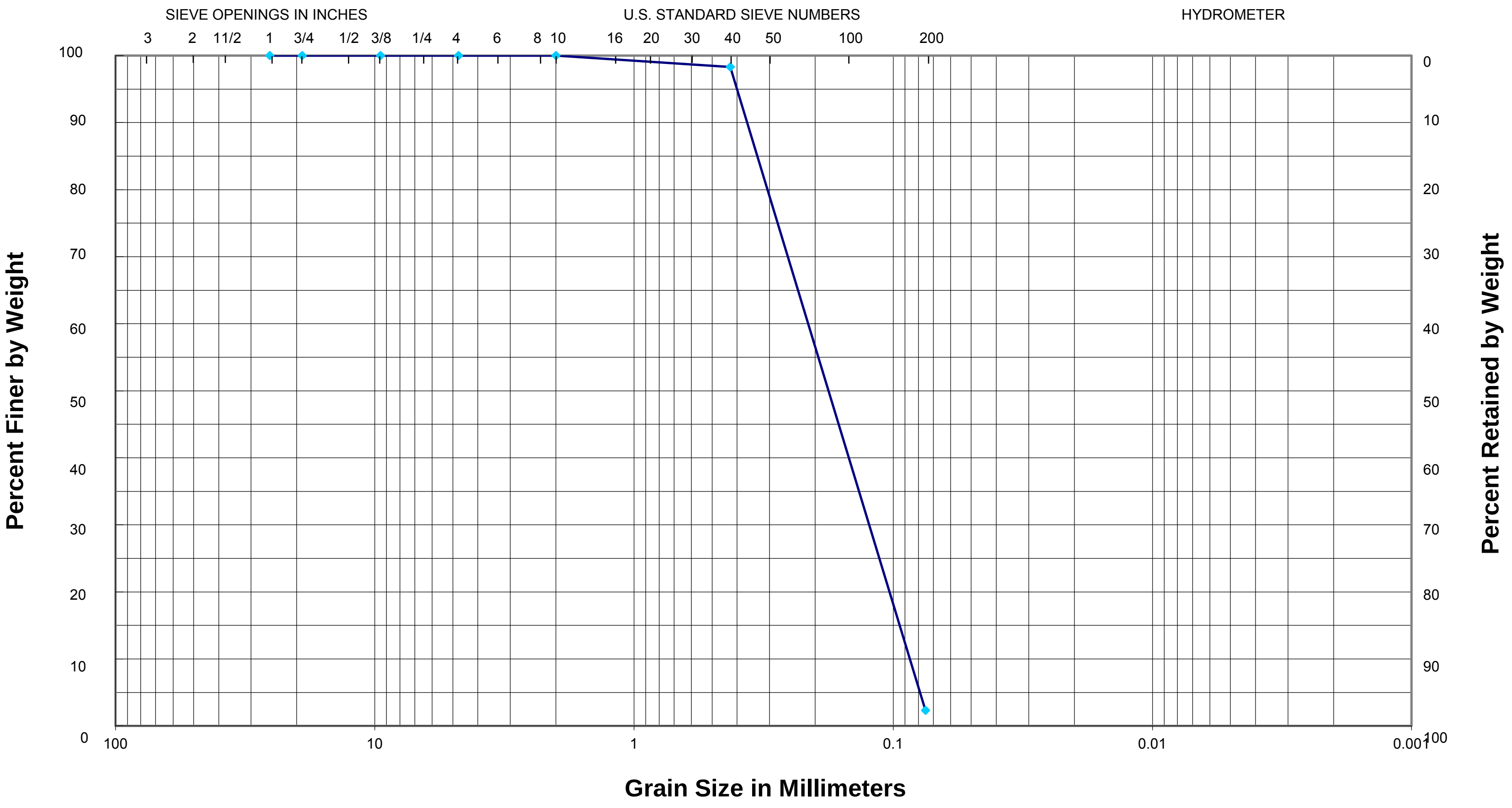


Sample: Boring 10, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



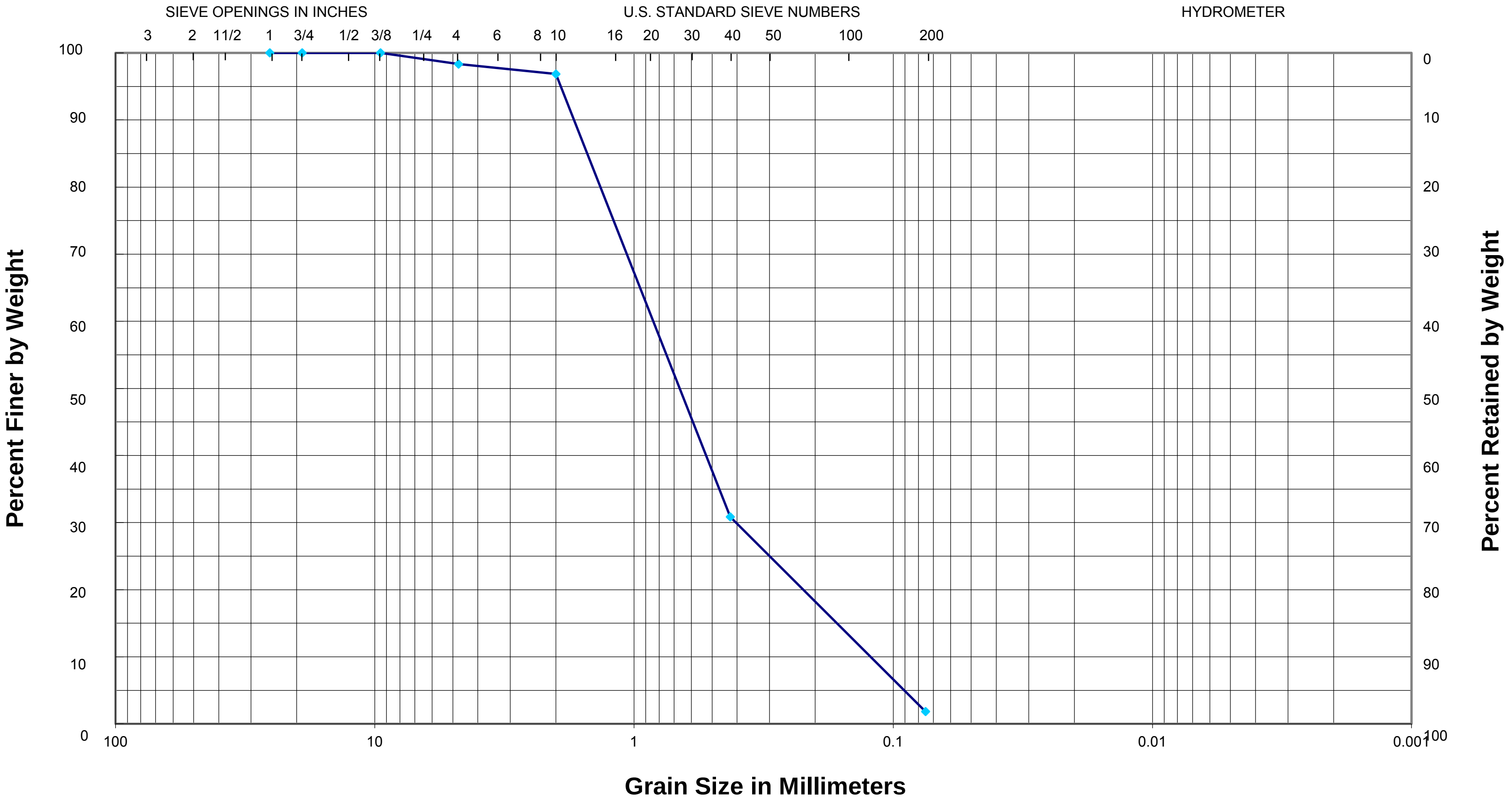
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 11, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



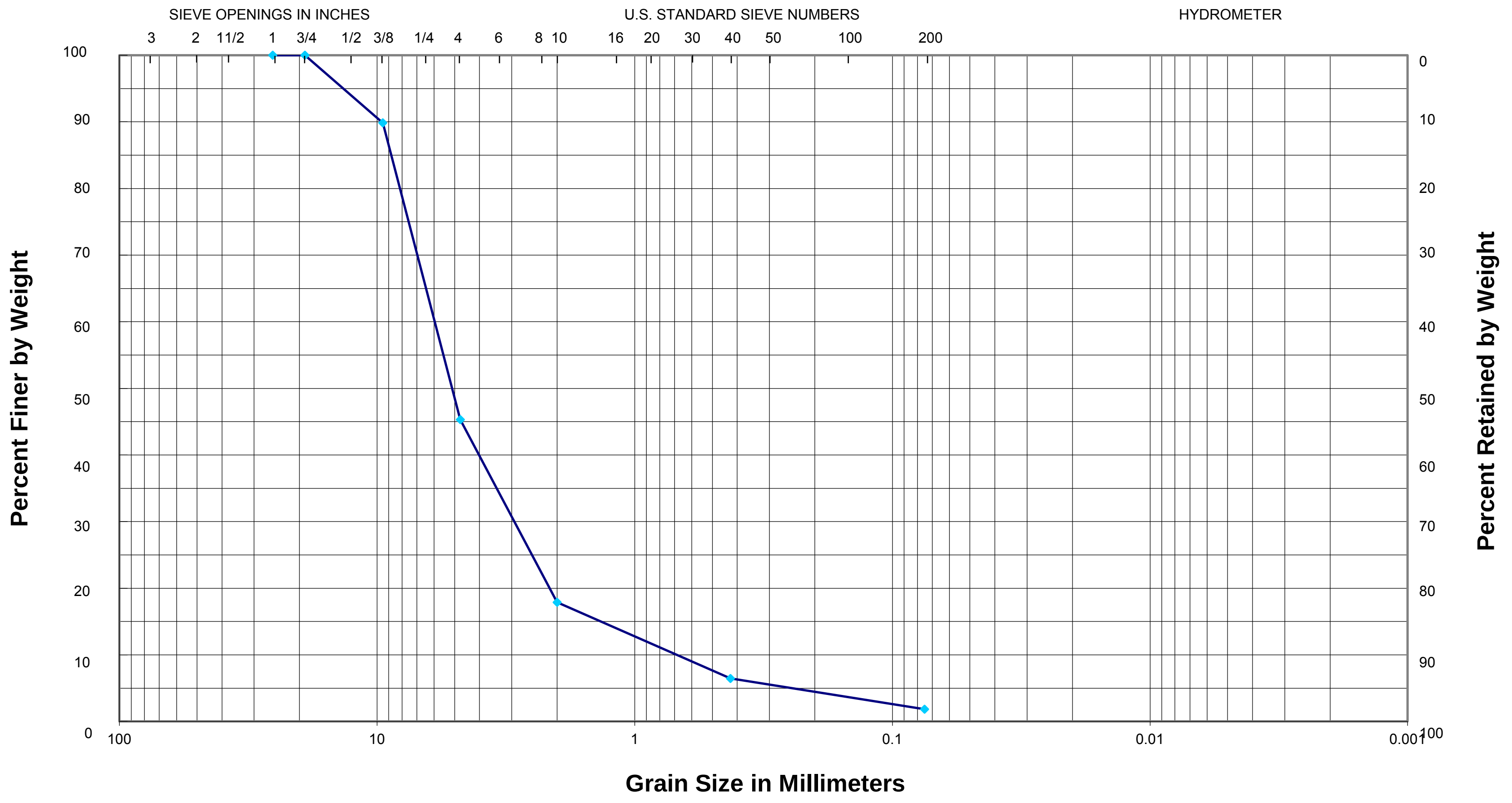
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 11, 48.5-49.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to medium sand
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

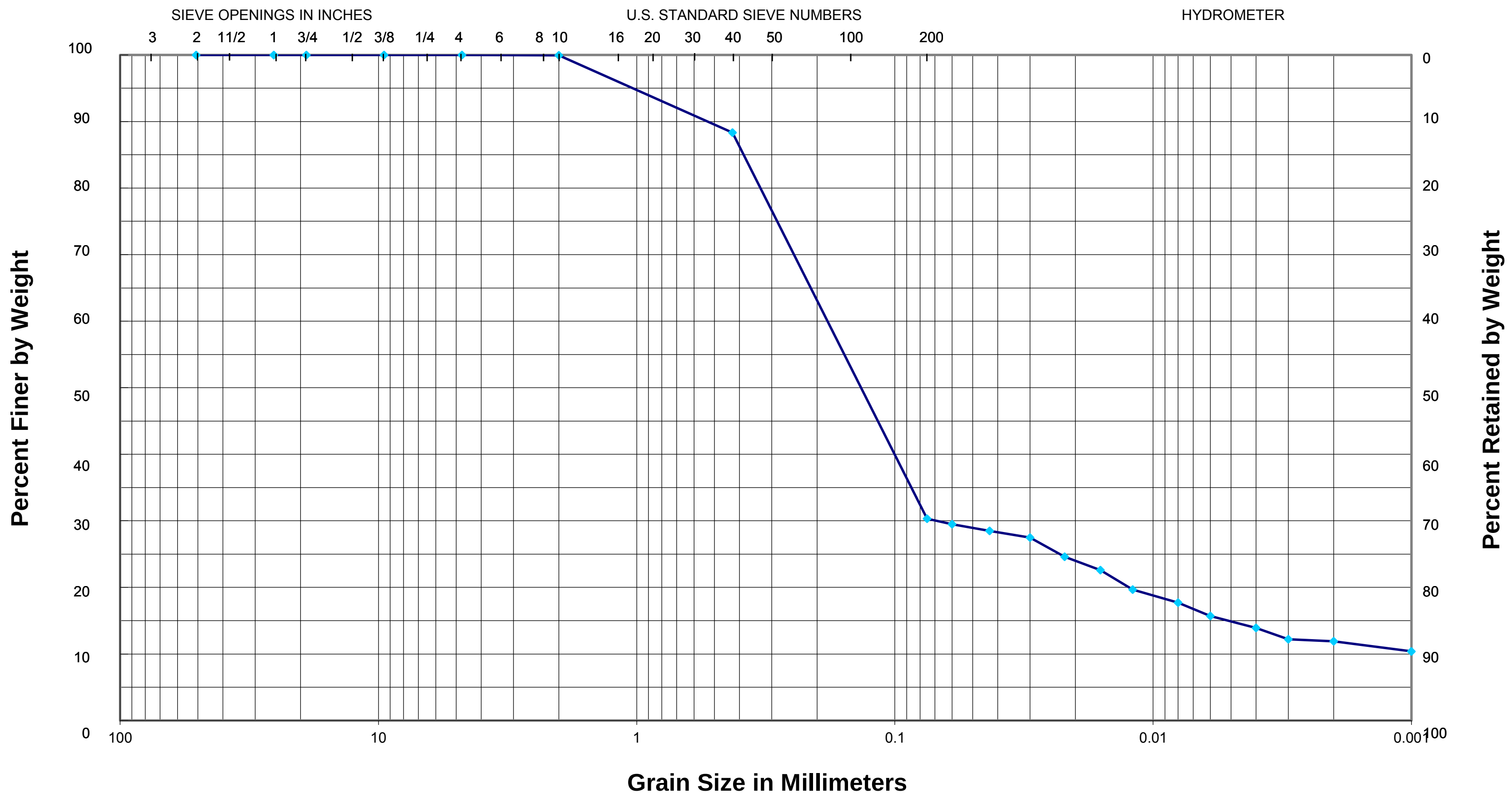
Sample: Boring 12, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray sandy fine gravel
Classification: USCS = GW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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



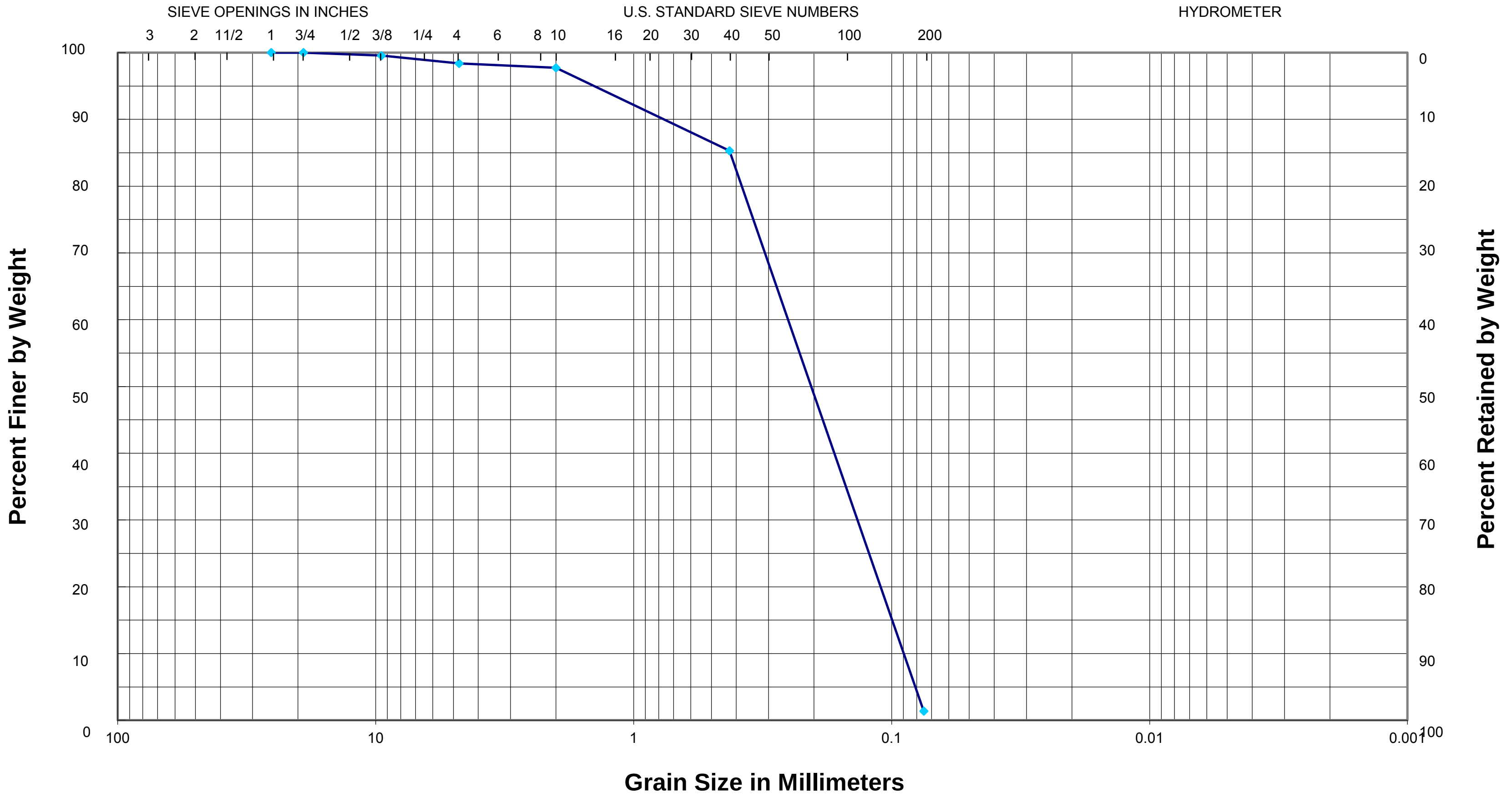
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13, 6.5-7.5 ft
Properties: G_s = 2.661; Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

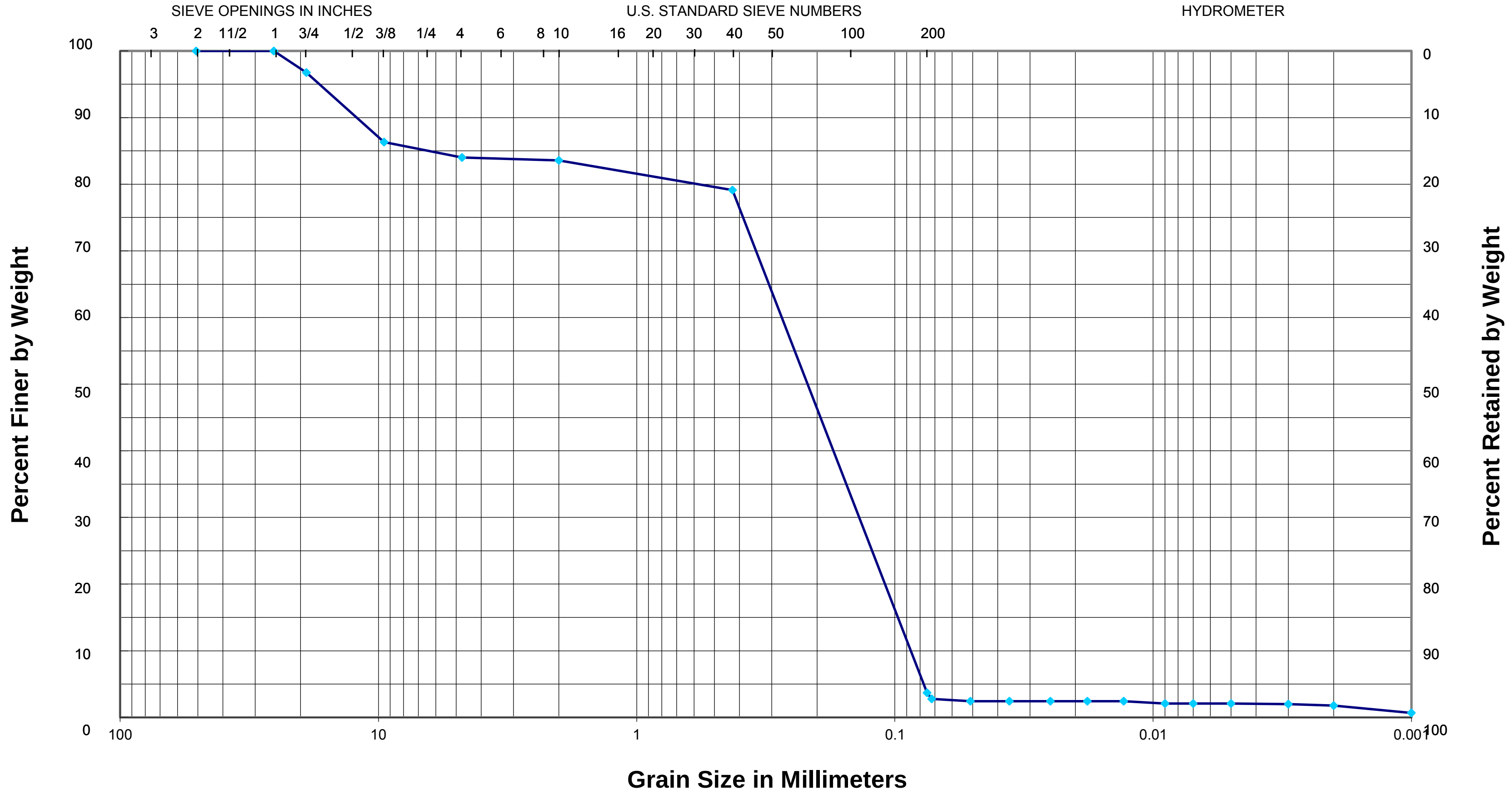
Sample: Boring 13A, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13A, 19-20 ft
Properties: G_s = 2.654; Non-plastic

Description: Brown and tan fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 48.5-49.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/some fine gravel
Classification: USCS = SW-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand, slightly silty w/some fine to coarse gravel
Classification: USCS = SW-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

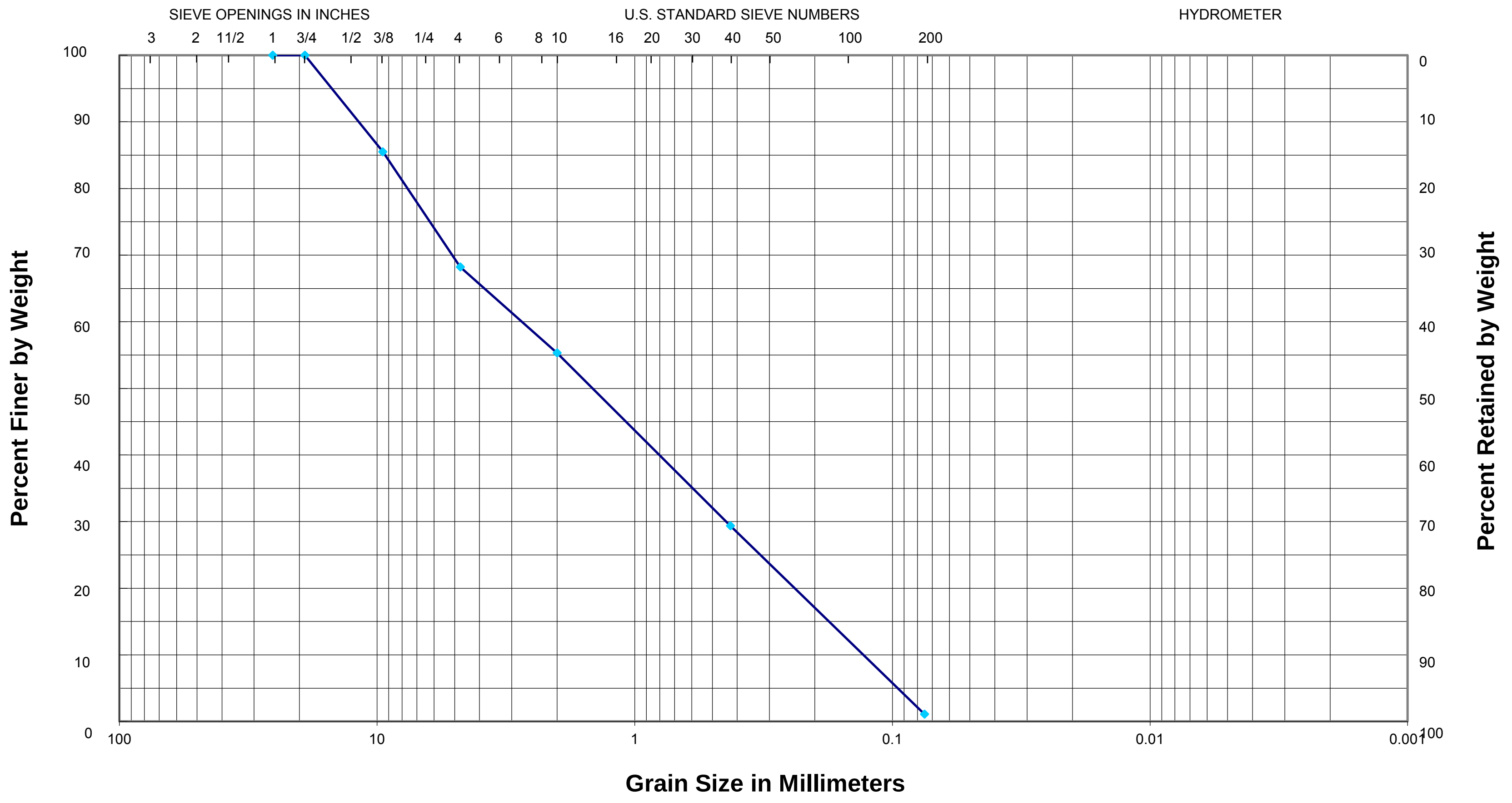
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 13A, 99-100 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

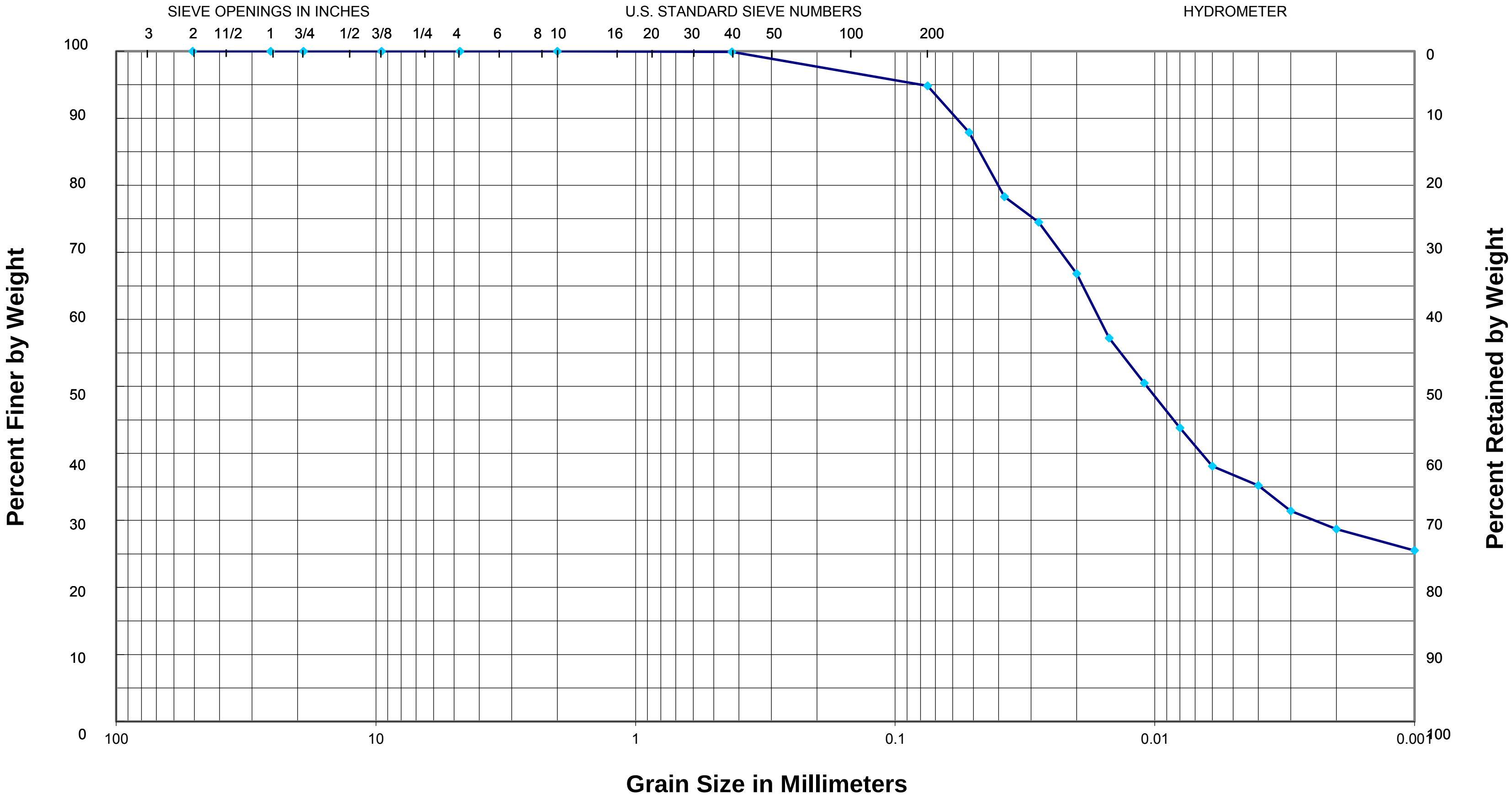
Sample: Boring 13A, 139-140 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand w/some fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

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



<table>
<tr><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT</td><td rowspan="2">CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

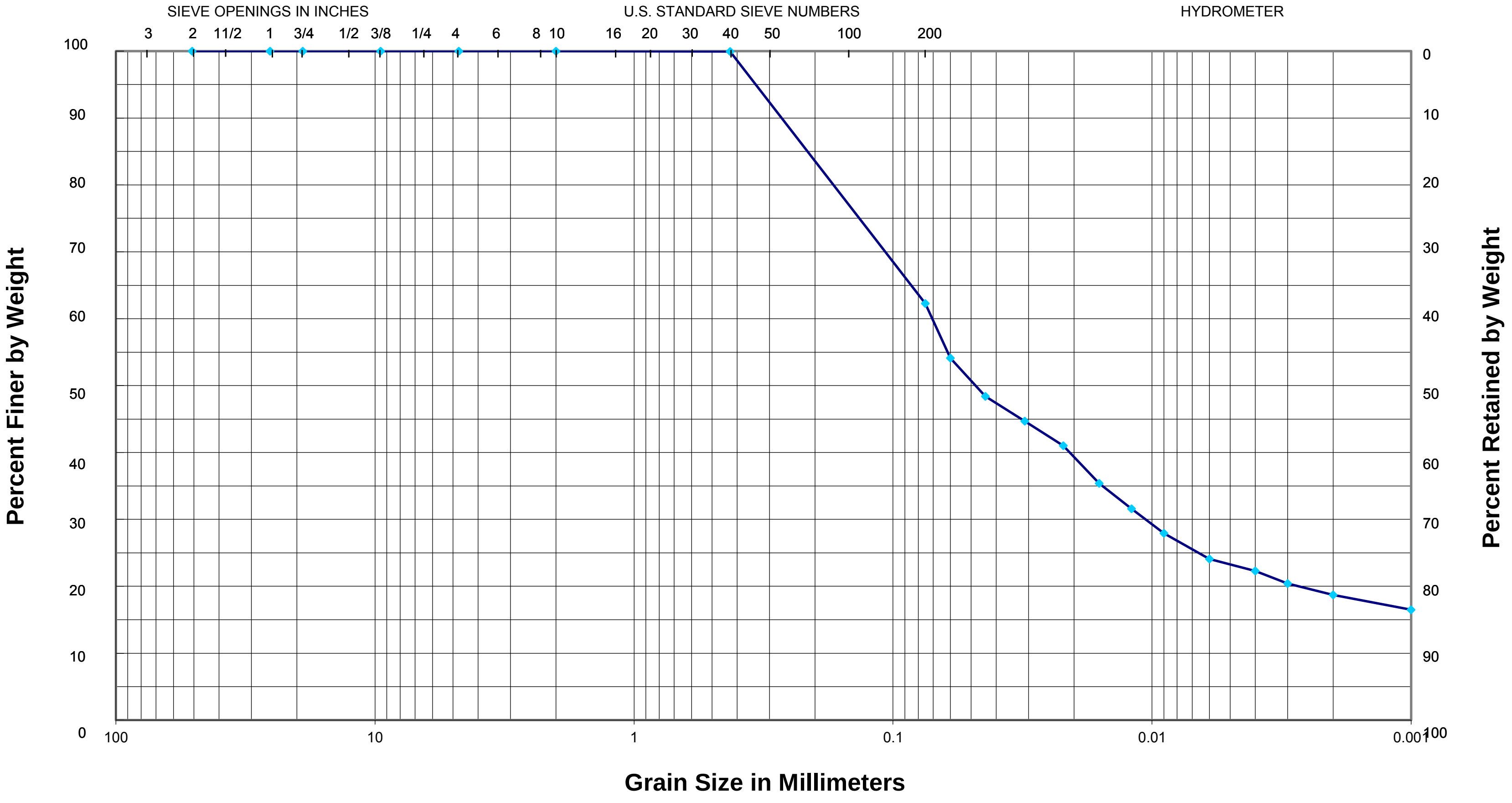
Sample: Boring 14, 4.5-5.5 ft
Properties: G_s = 2.669; LL = 34, PL = 19, PI = 15

Description: Brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

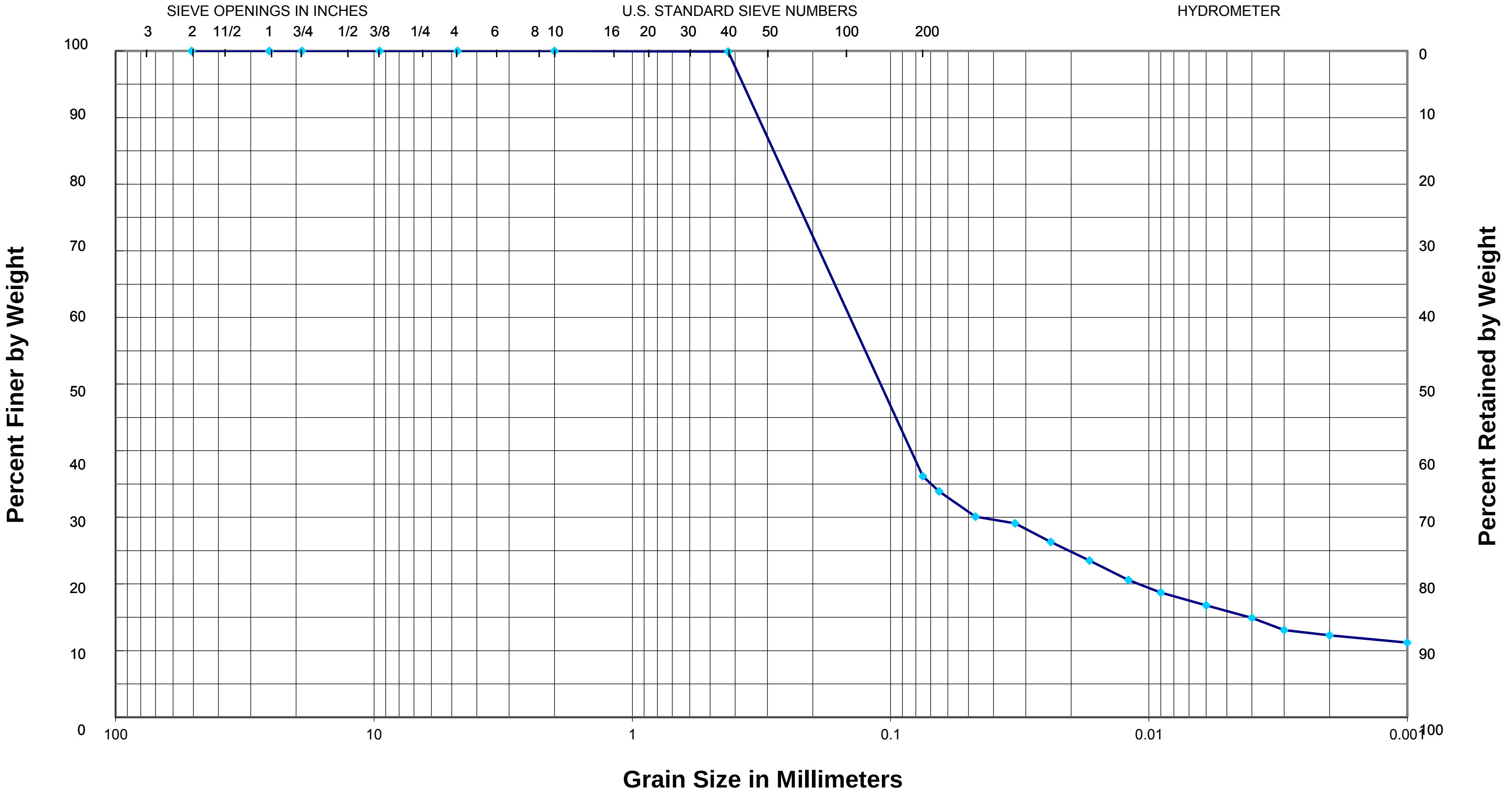
Sample: Boring 14, 14-15 ft
Properties: G_s = 2.668; LL = 22, PL = 17, PI = 5

Description: Gray clayey silt
Classification: USCS = CL-ML; AASHTO = A-4

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 14, 24-24.5 ft
Properties: G_s = 2.668; Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-4

13-017

GRAIN SIZE CURVE

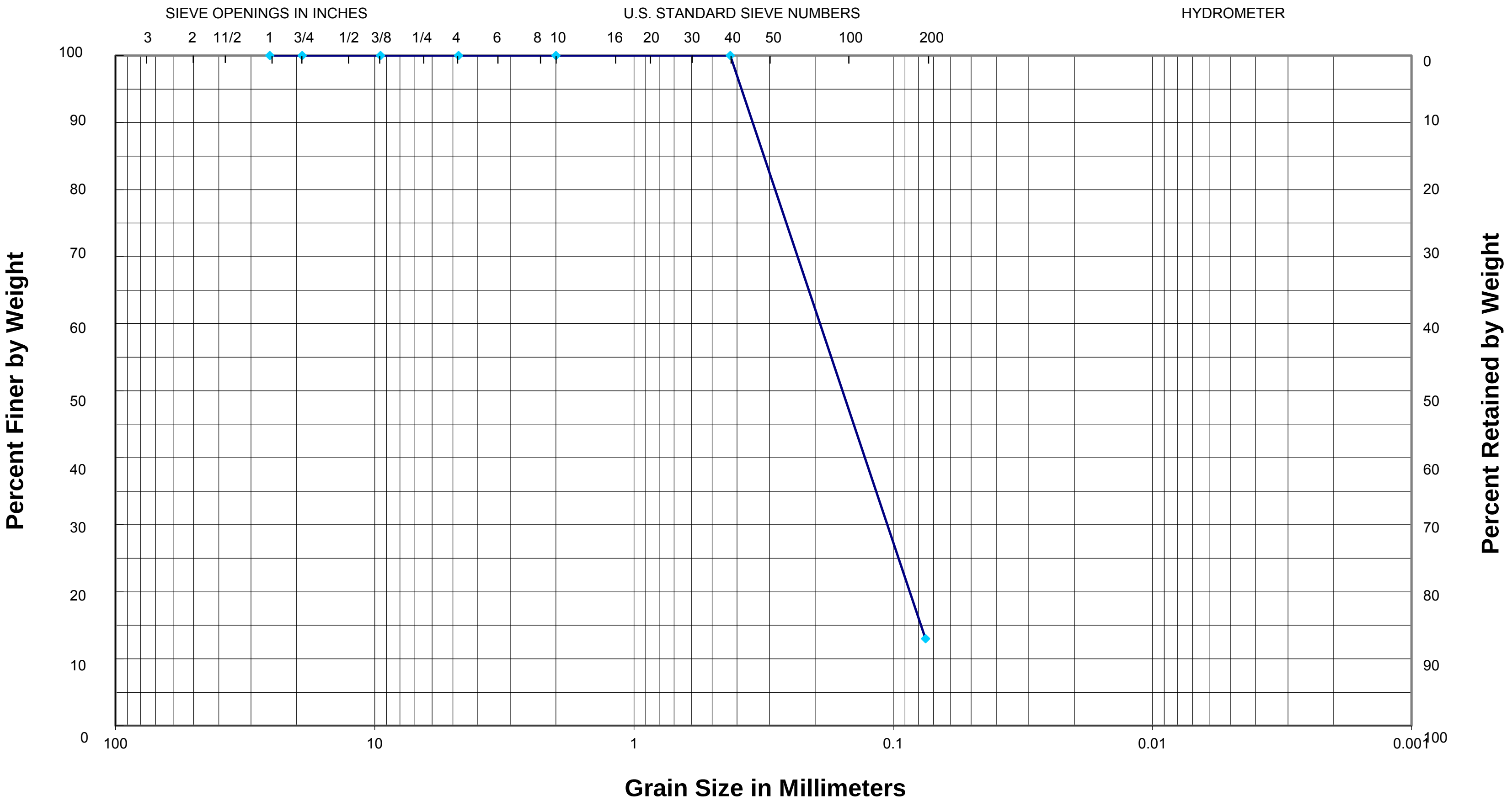
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 14, 100-118 ft (composite)
Atterberg Limits: Non-plastic

Description: Brown sandy fine gravel
Classification: USCS = GP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



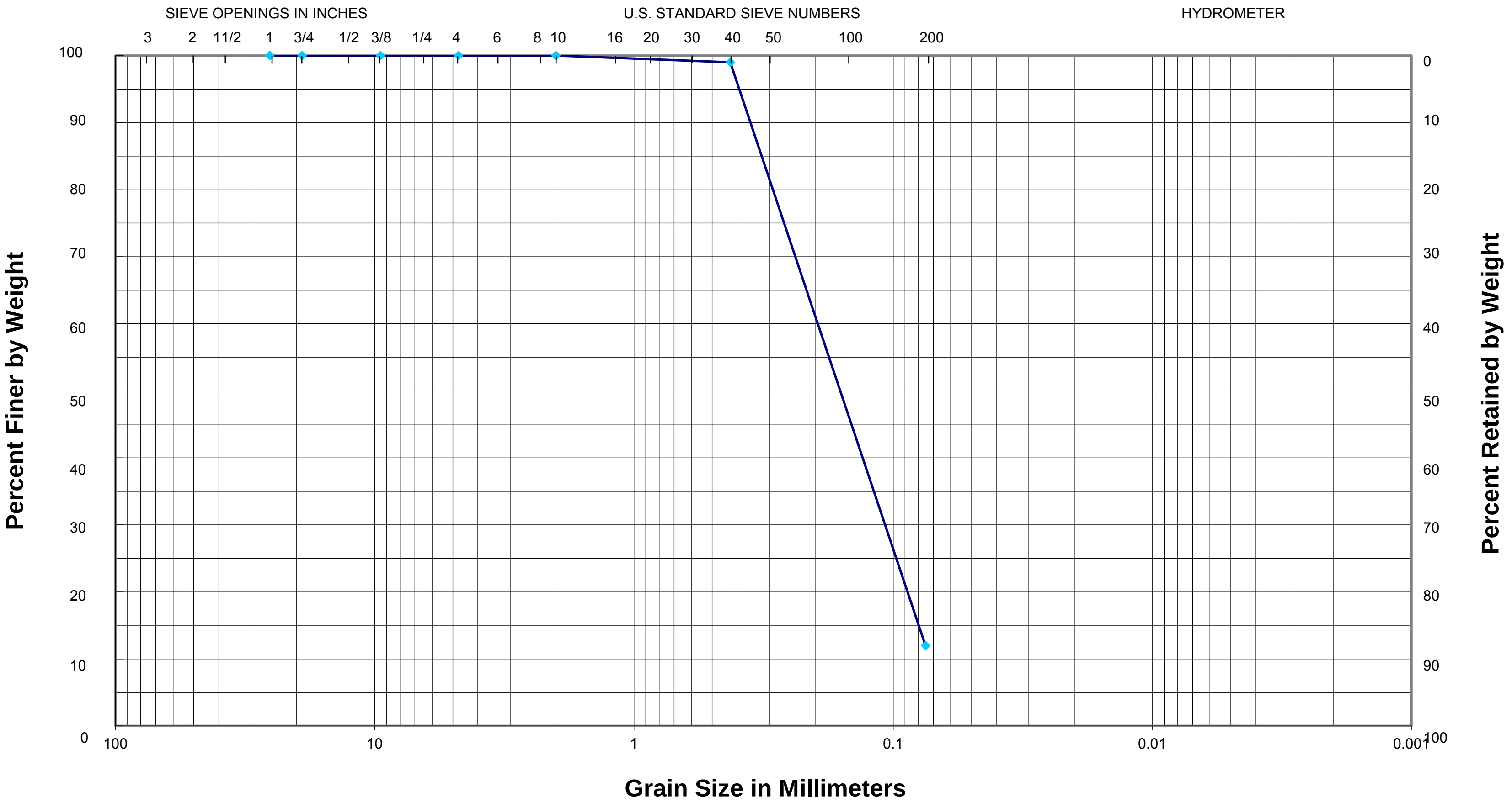
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 6.5-7.5 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



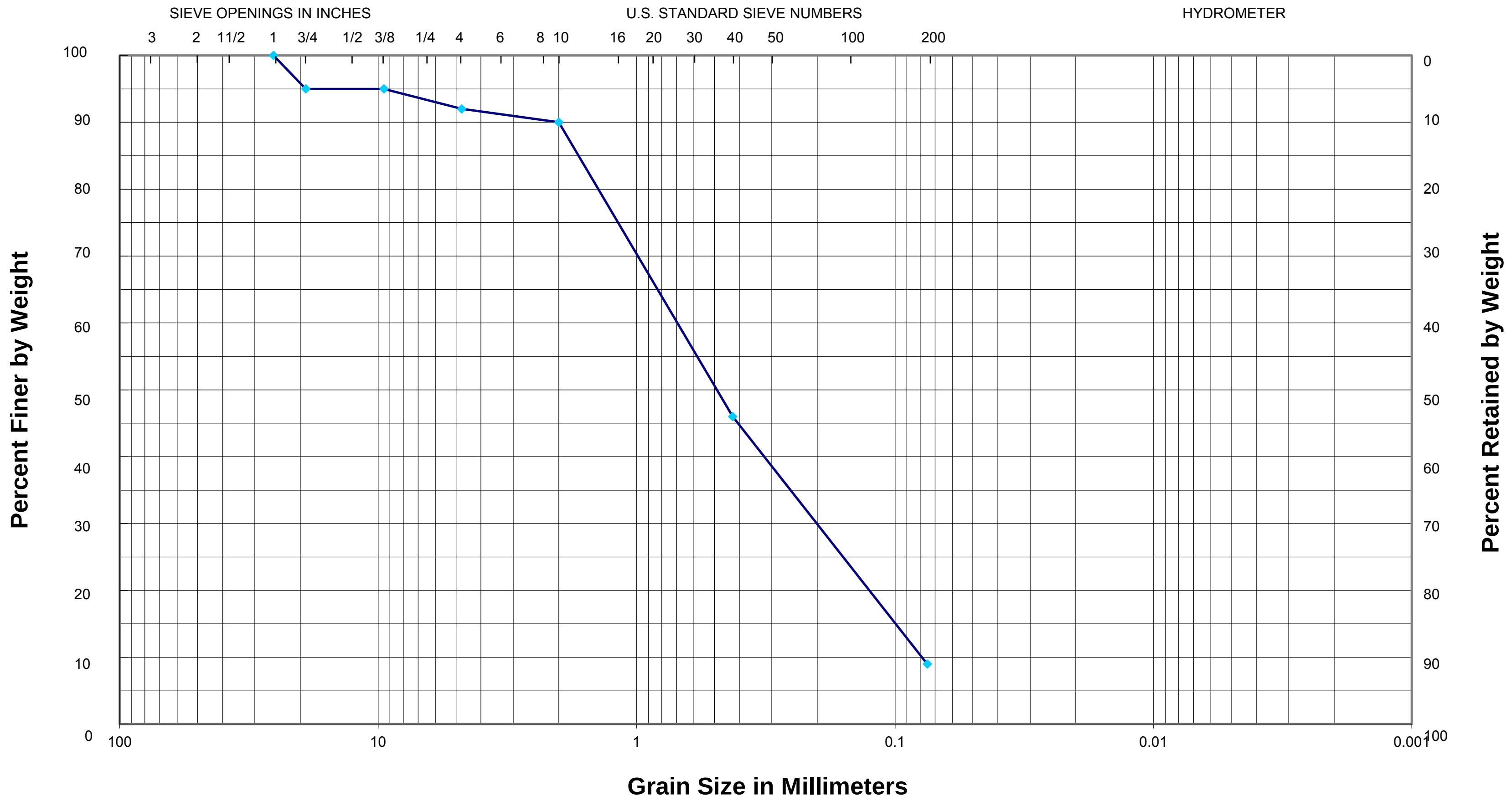
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 24-25 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



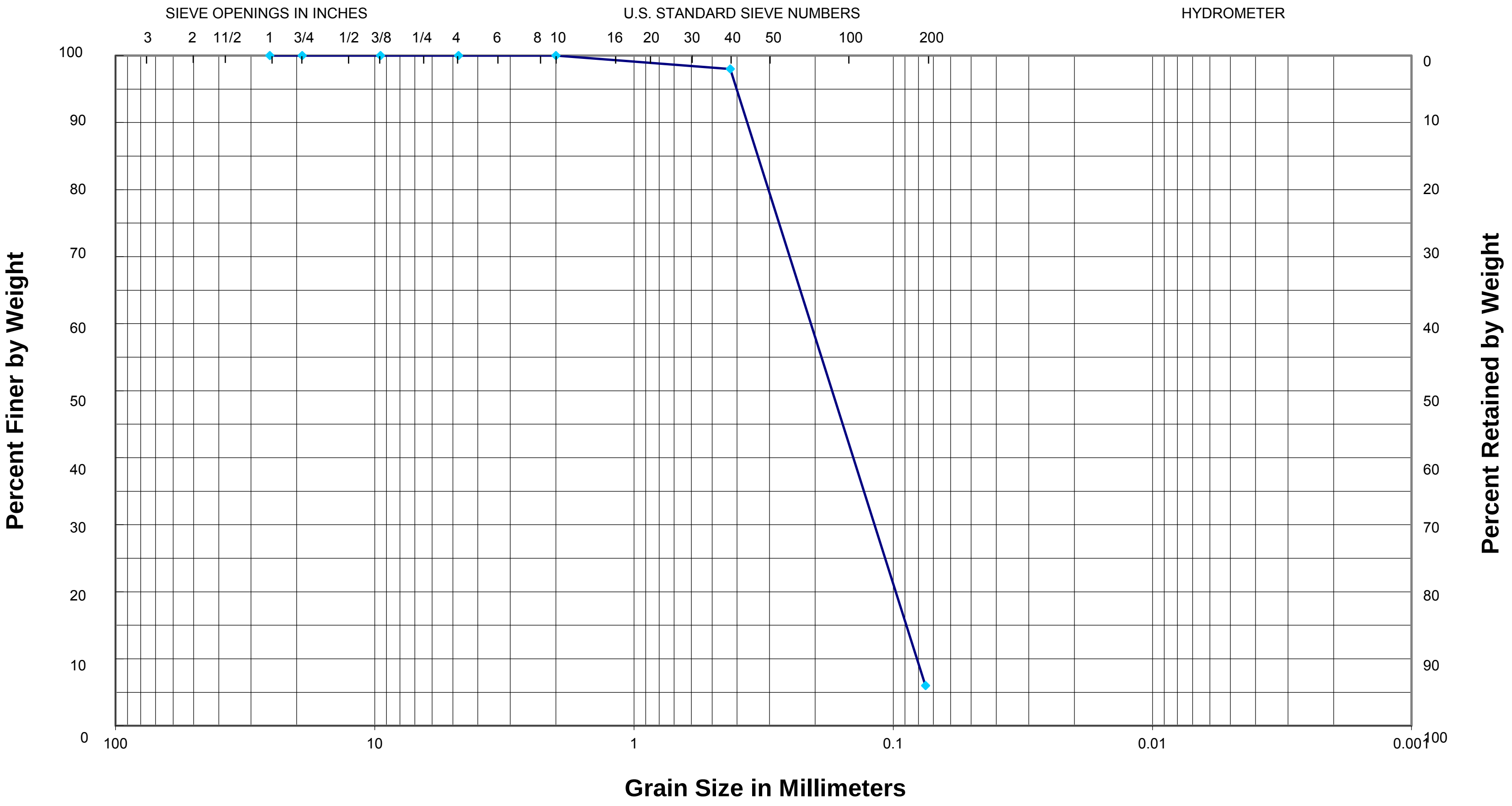
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 44-45 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



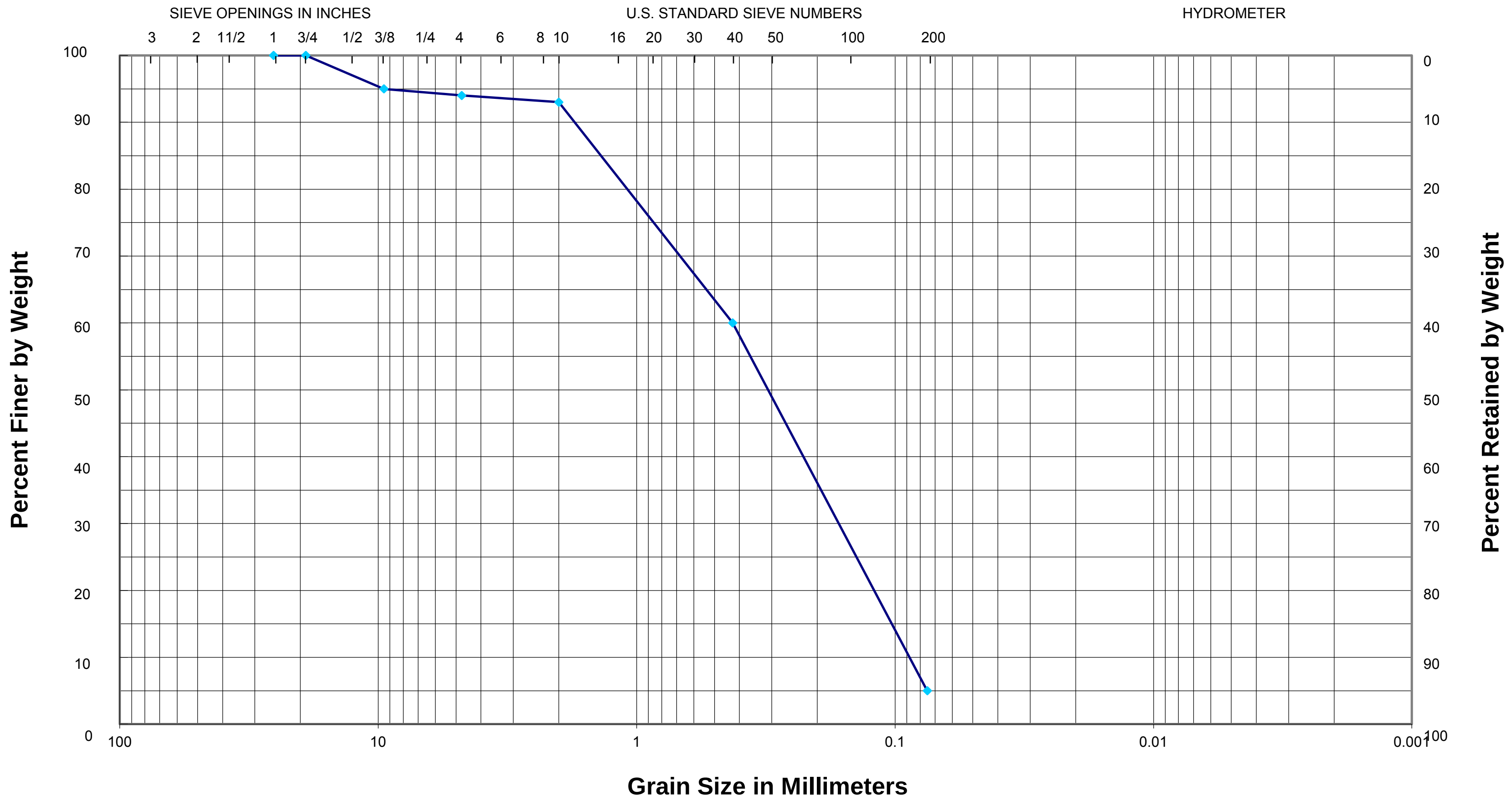
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 59-60 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



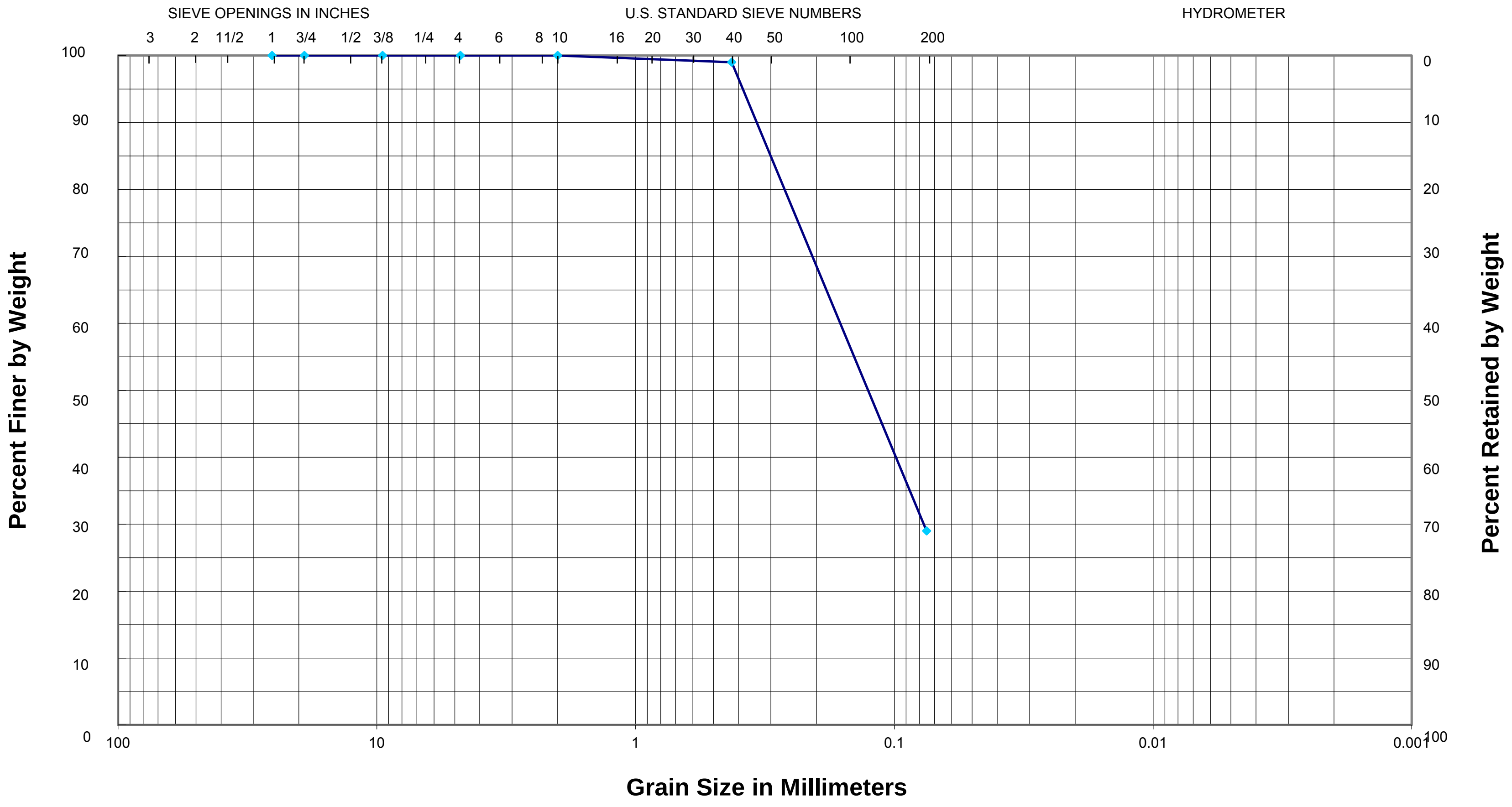
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 69-70 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



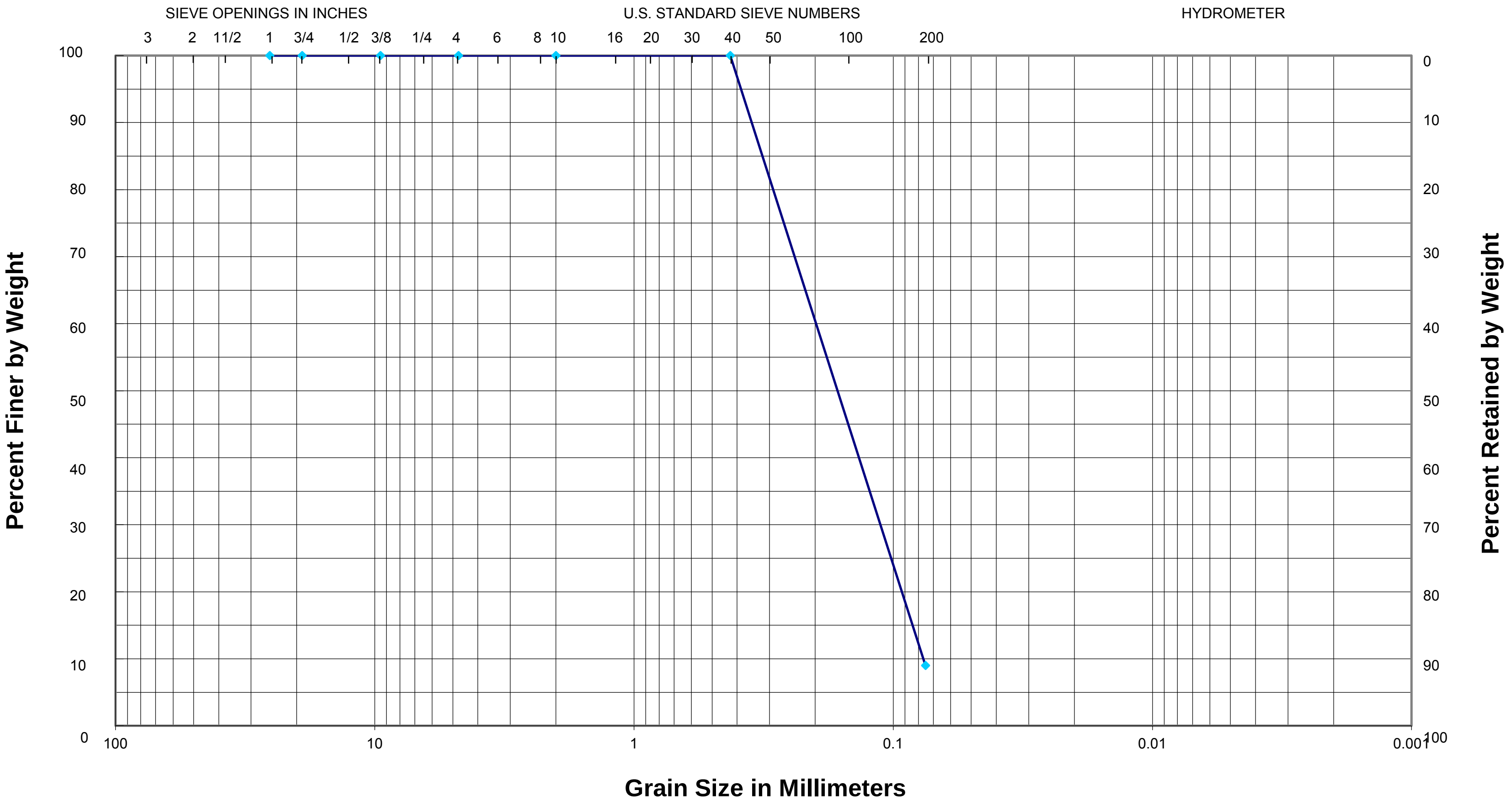
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

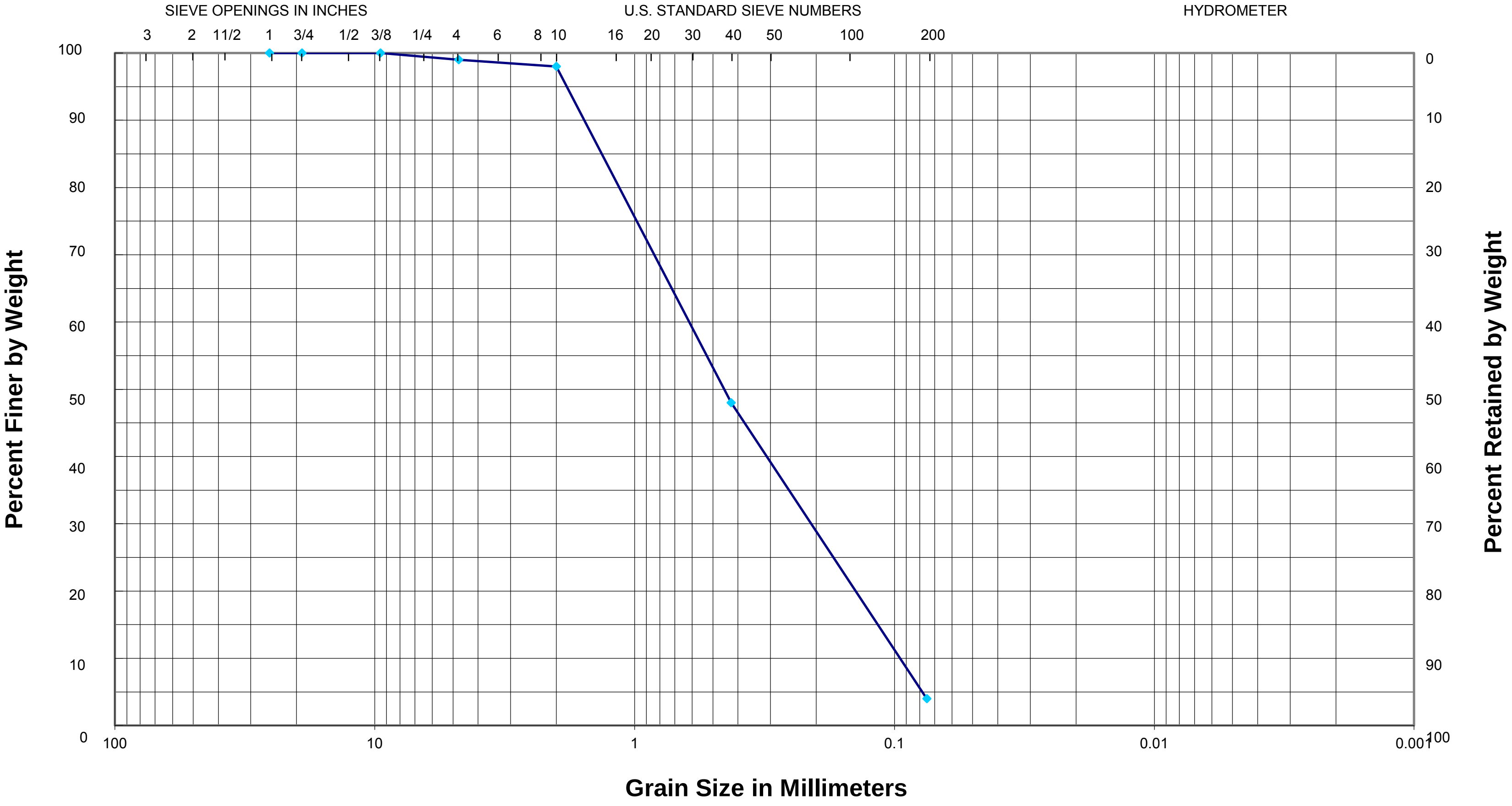
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 39-40 ft
Atterberg Limits: Non-plastic

Description: Dark gray and brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



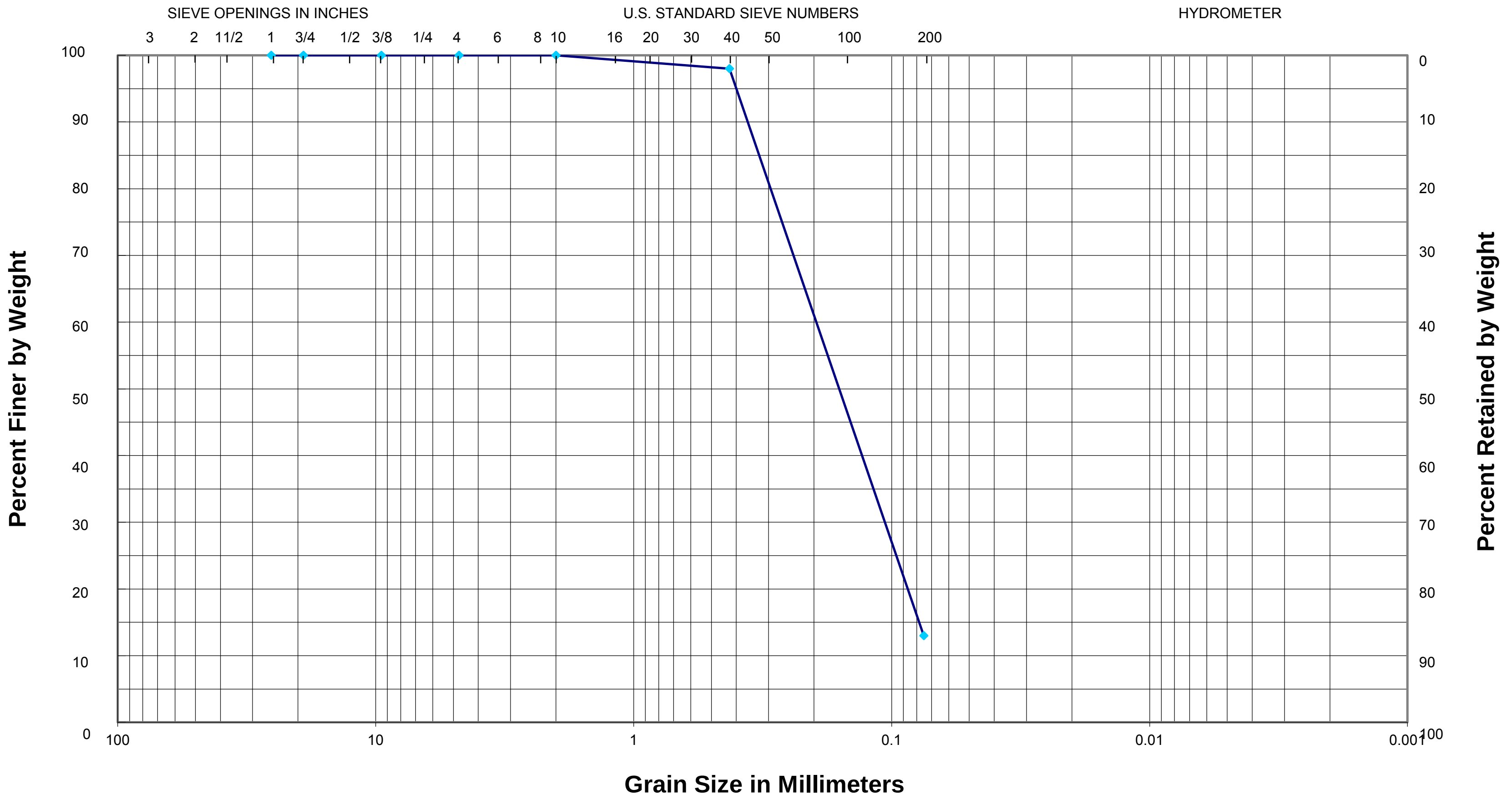
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 49-50 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand, slightly silty
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



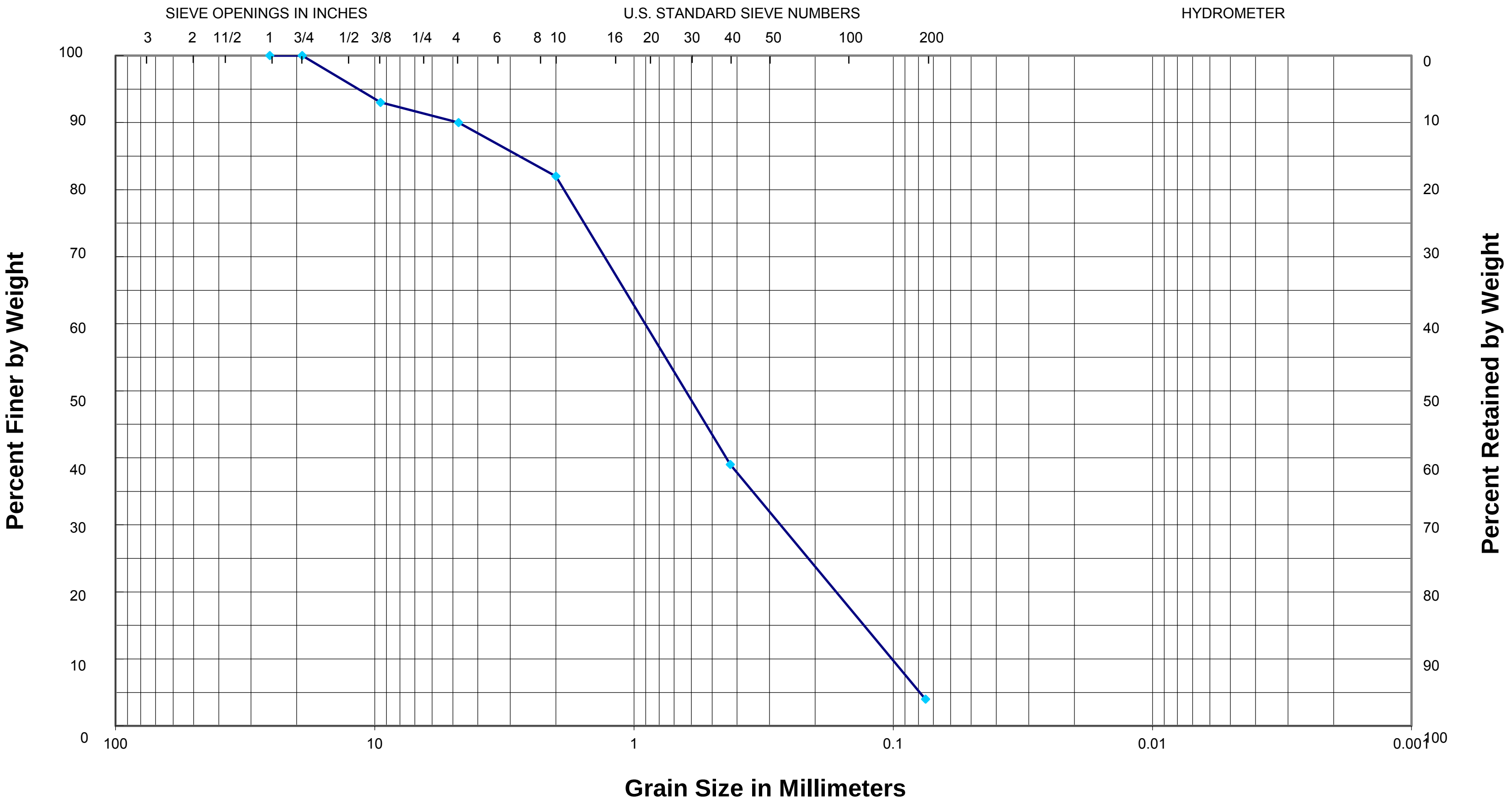
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 59-60 ft
Atterberg Limits: Non-plastic

Description: Gray and brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



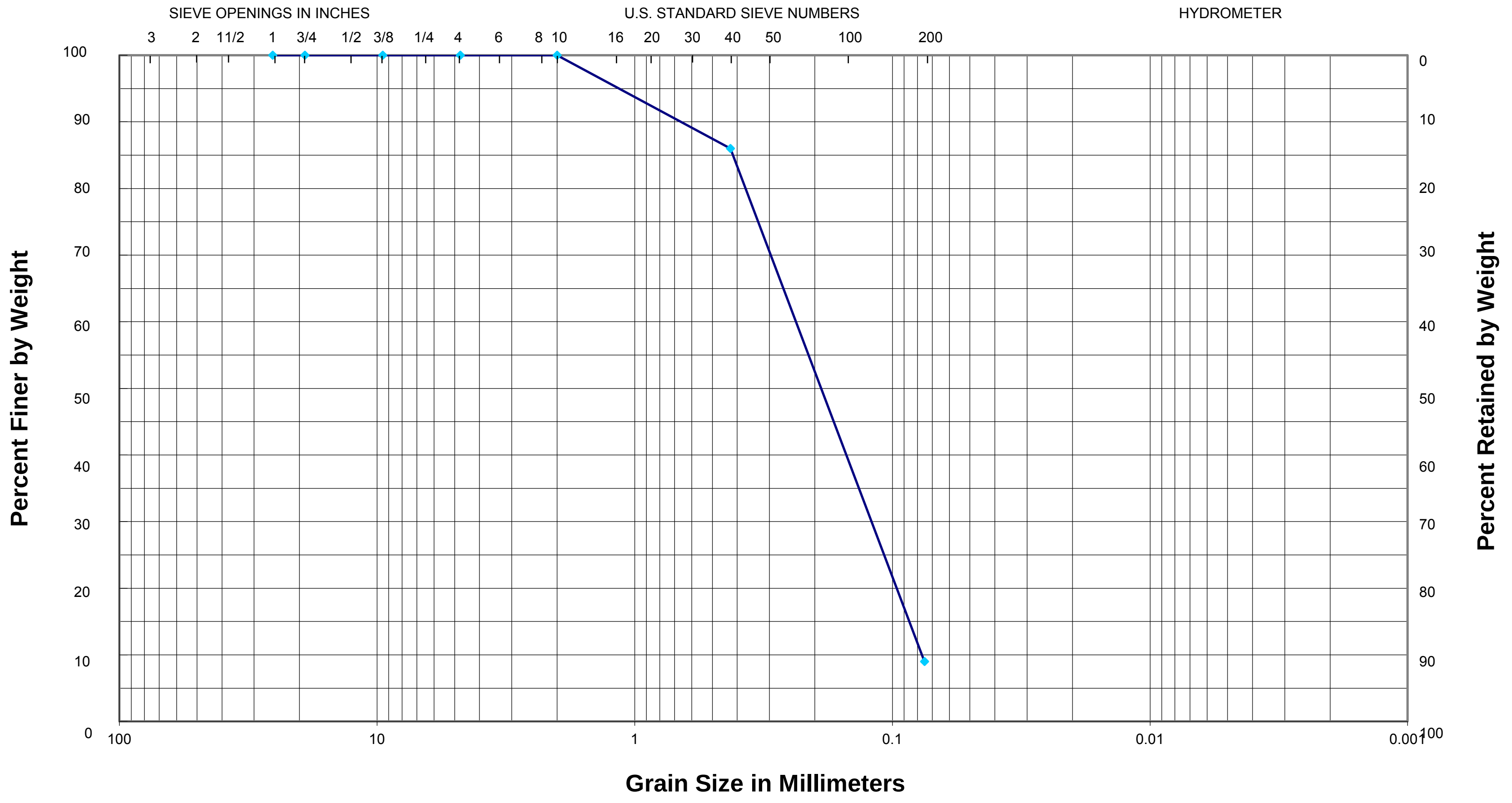
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 69-70 ft
Atterberg Limits: Non-plastic

Description: Brown and gray fine to medium sand w/trace fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



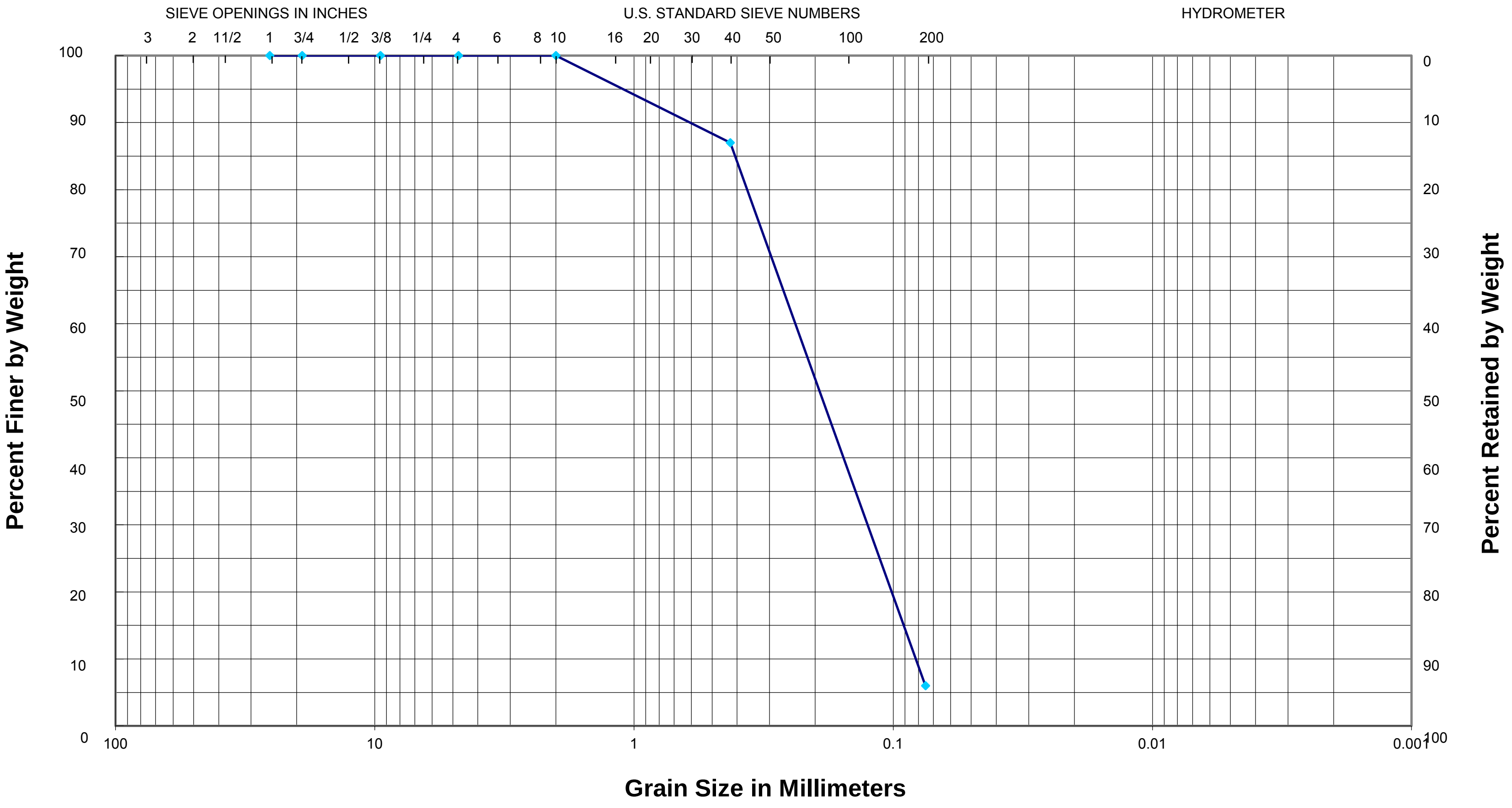
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 44-45 ft
Atterberg Limits: Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

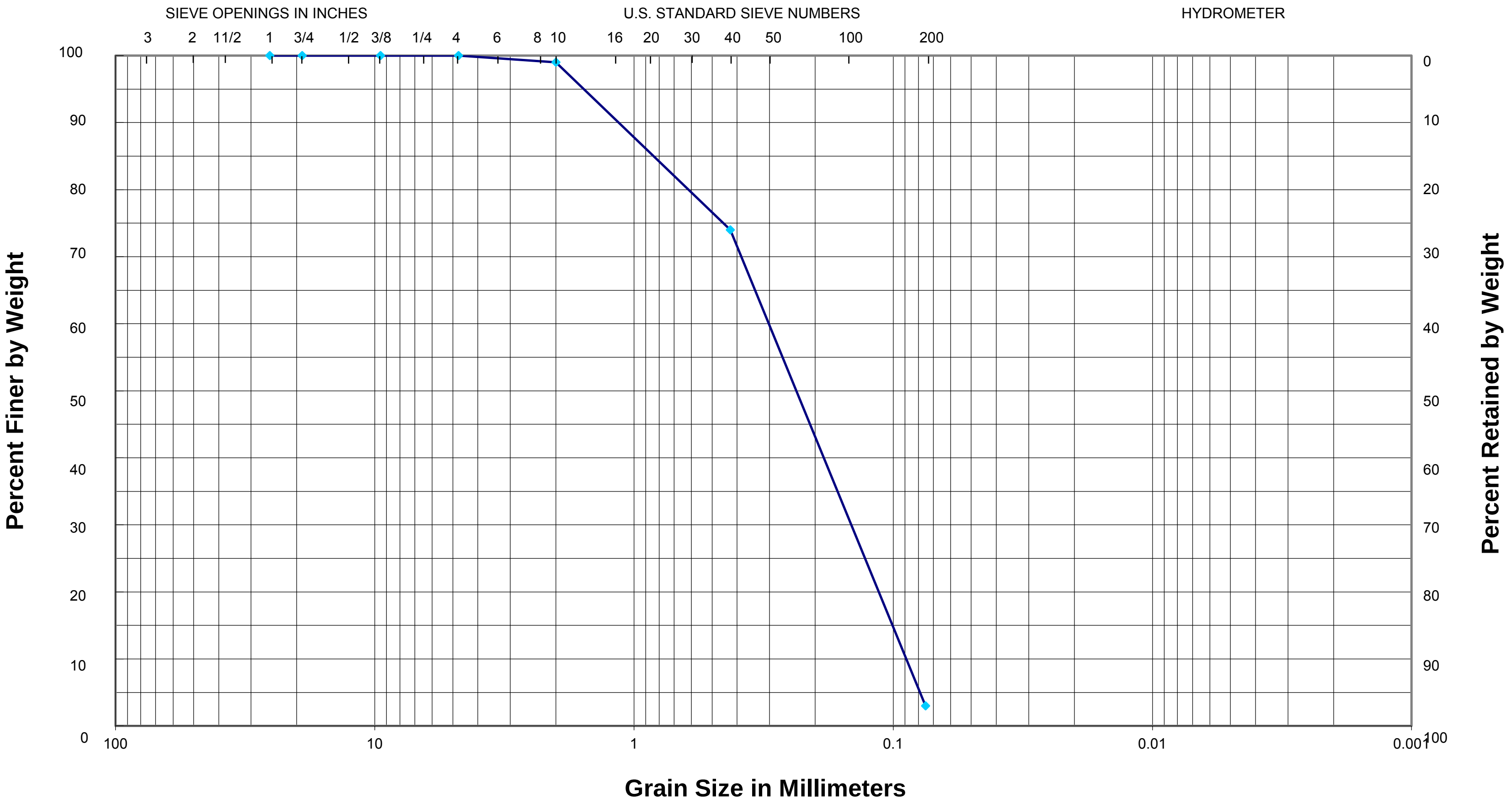
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 54-55 ft
Atterberg Limits: Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



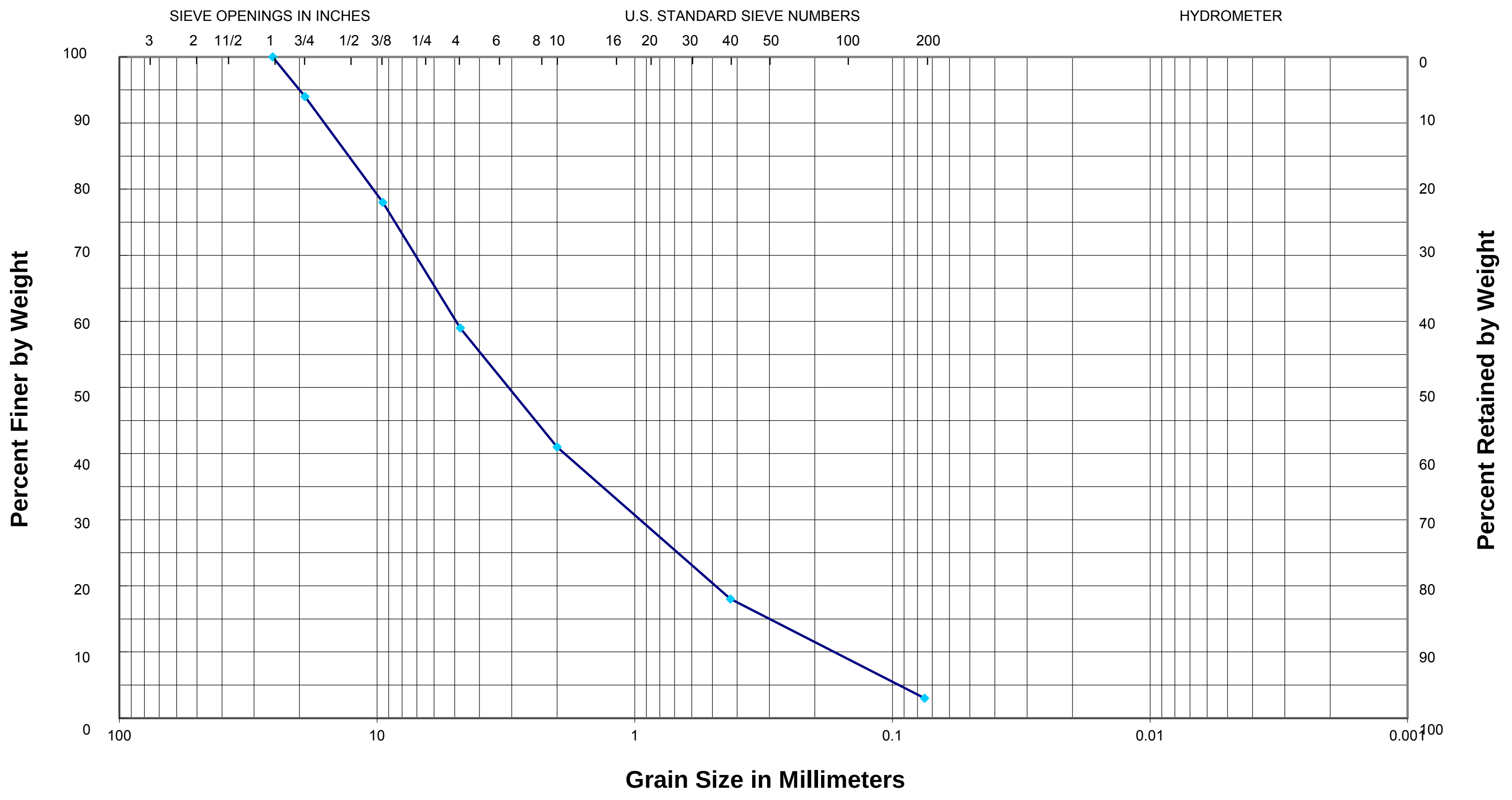
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 68.5-69 ft
Atterberg Limits: Non-plastic

Description: Brown fine to medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 17, 83.5-84 ft
Atterberg Limits: Non-plastic

Description: Brown and gray sandy fine to coarse gravel
Classification: USCS = SW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 18, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Brown and gray sandy fine gravel
Classification: USCS = GP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE

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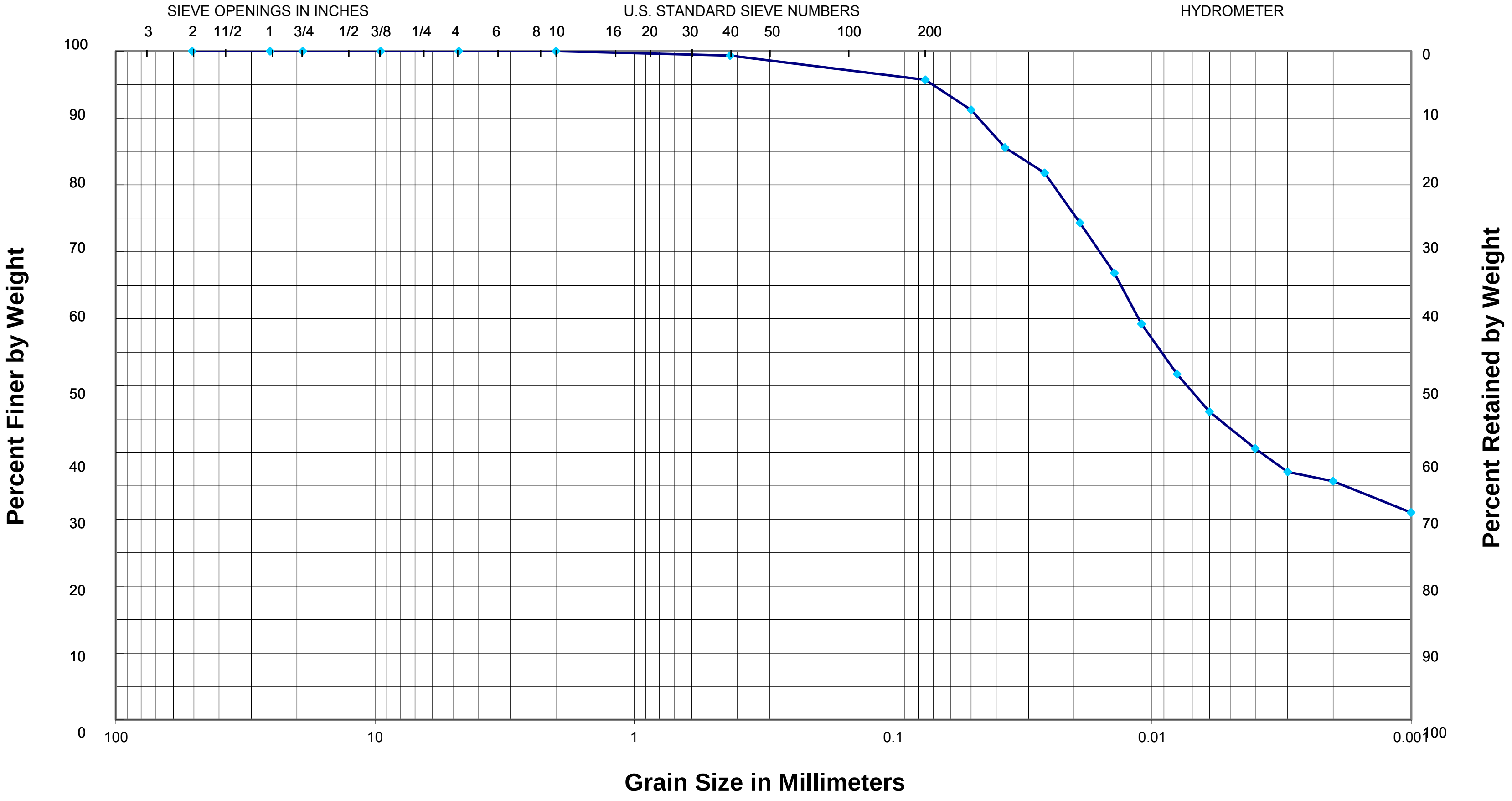

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<tr><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT</td><td rowspan="2">CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

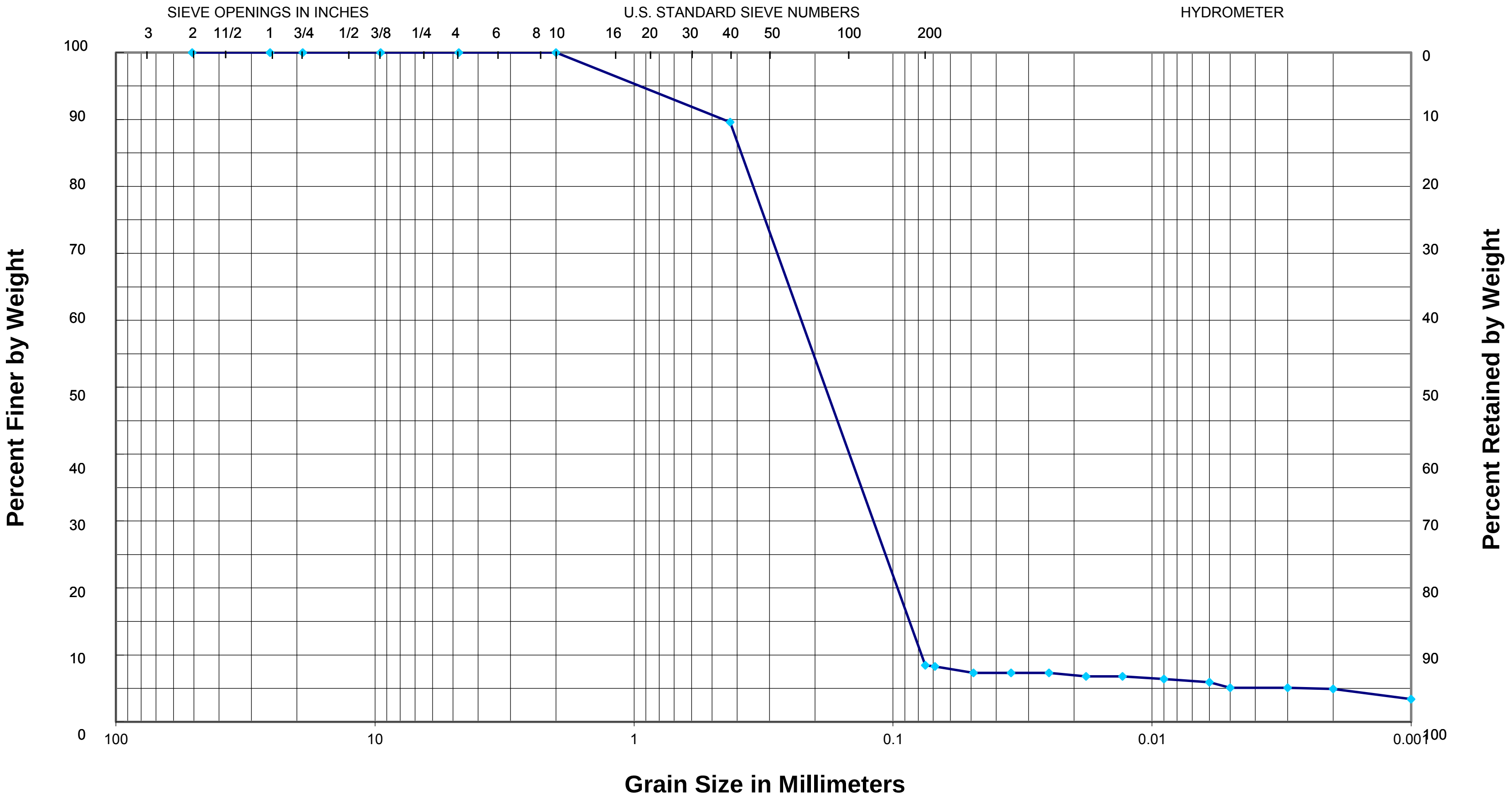
Sample: Boring 19, 6.5-7 ft
Properties: G_s = 2.696; LL = 40, PL = 18, PI = 22

Description: Gray and brown silty clay w/ferrous stains and nodules
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 19, 24-25 ft
Properties: G_s = 2.678; Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = sp-sm; AASHTO = A-3

13-017

GRAIN SIZE CURVE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 19, 78.5-79.5 ft
Atterberg Limits: Non-plastic

Description: Grayish brown sandy fine gravel, slighly silty
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE

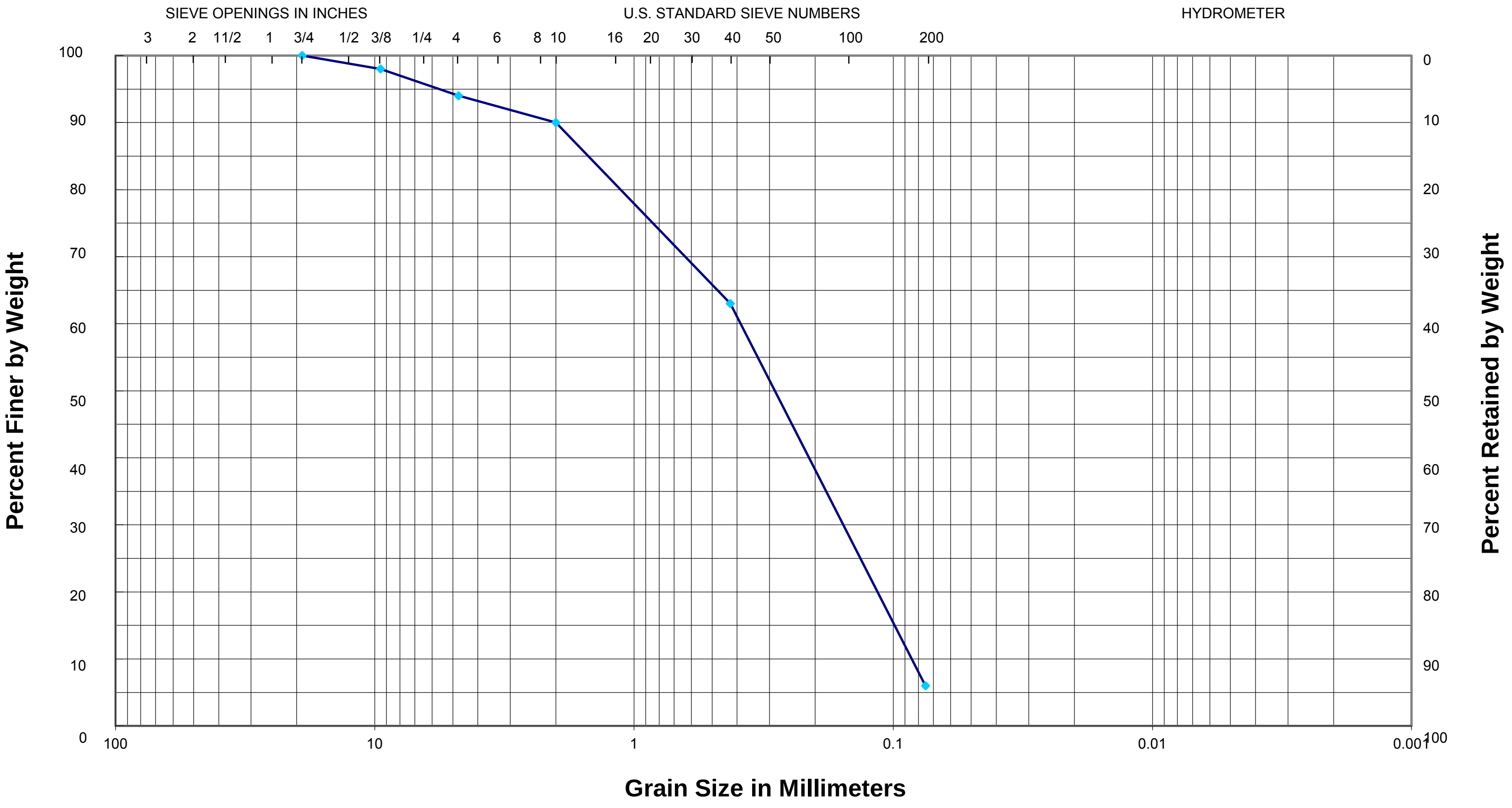
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 20, 44-45 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand with a little fine gravel
Classification: USCS = SP-SM; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



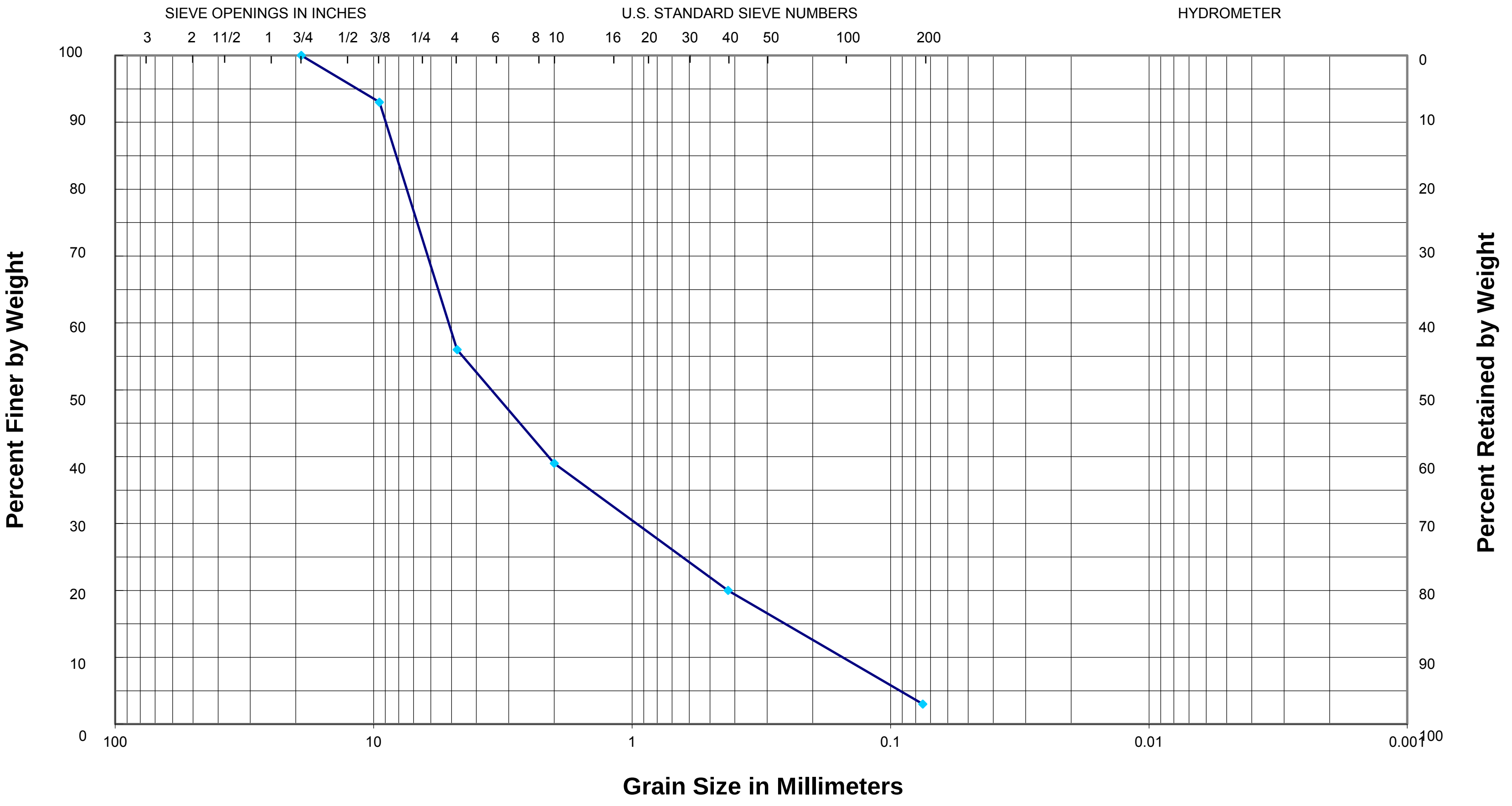
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 20, 58.5-59.5 ft
Atterberg Limits: Non-plastic

Description: Gray fine to coarse sand with trace fine gravel
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



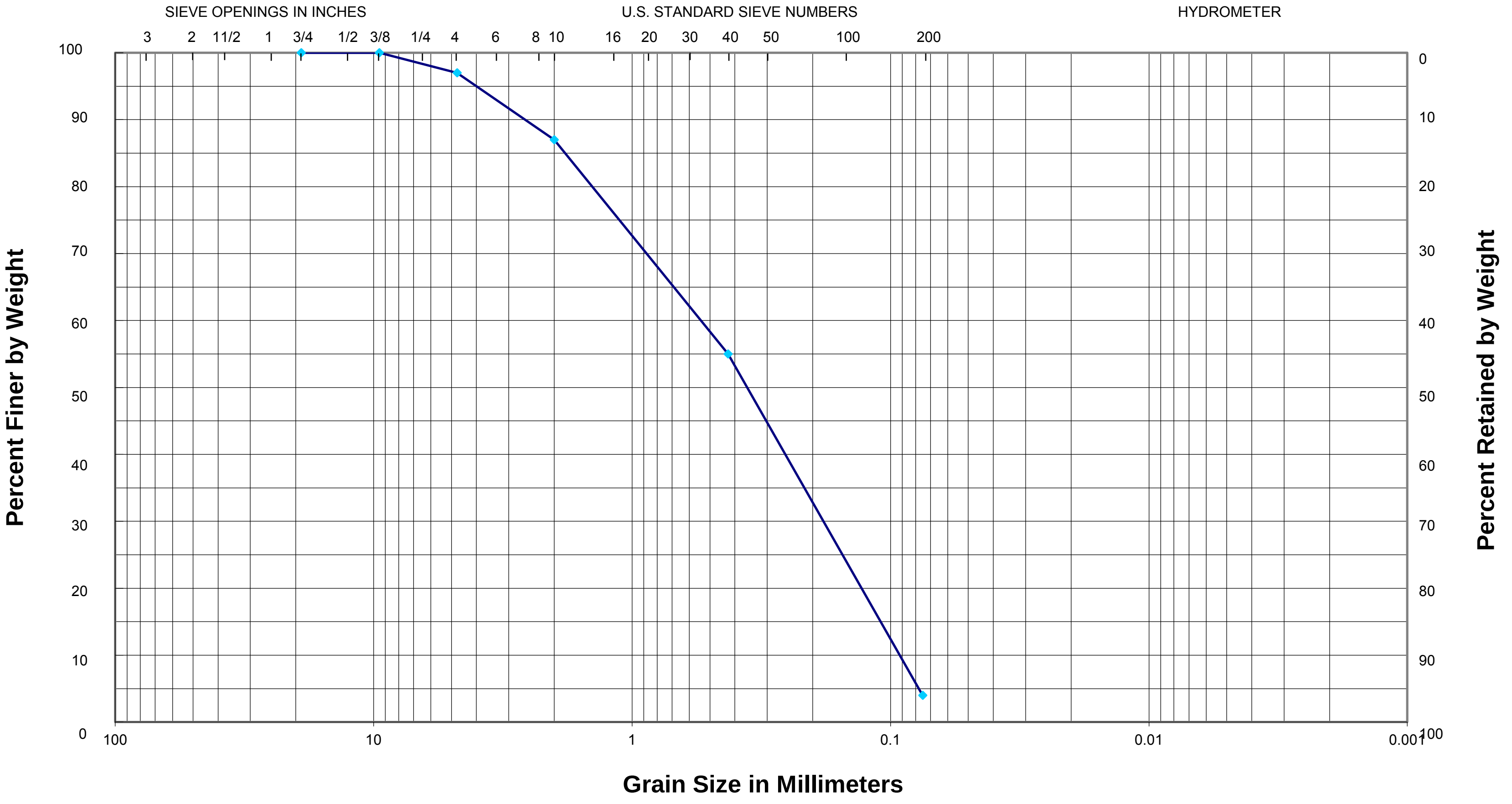
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 20, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Brown sandy fine gravel
Classification: USCS = SW; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



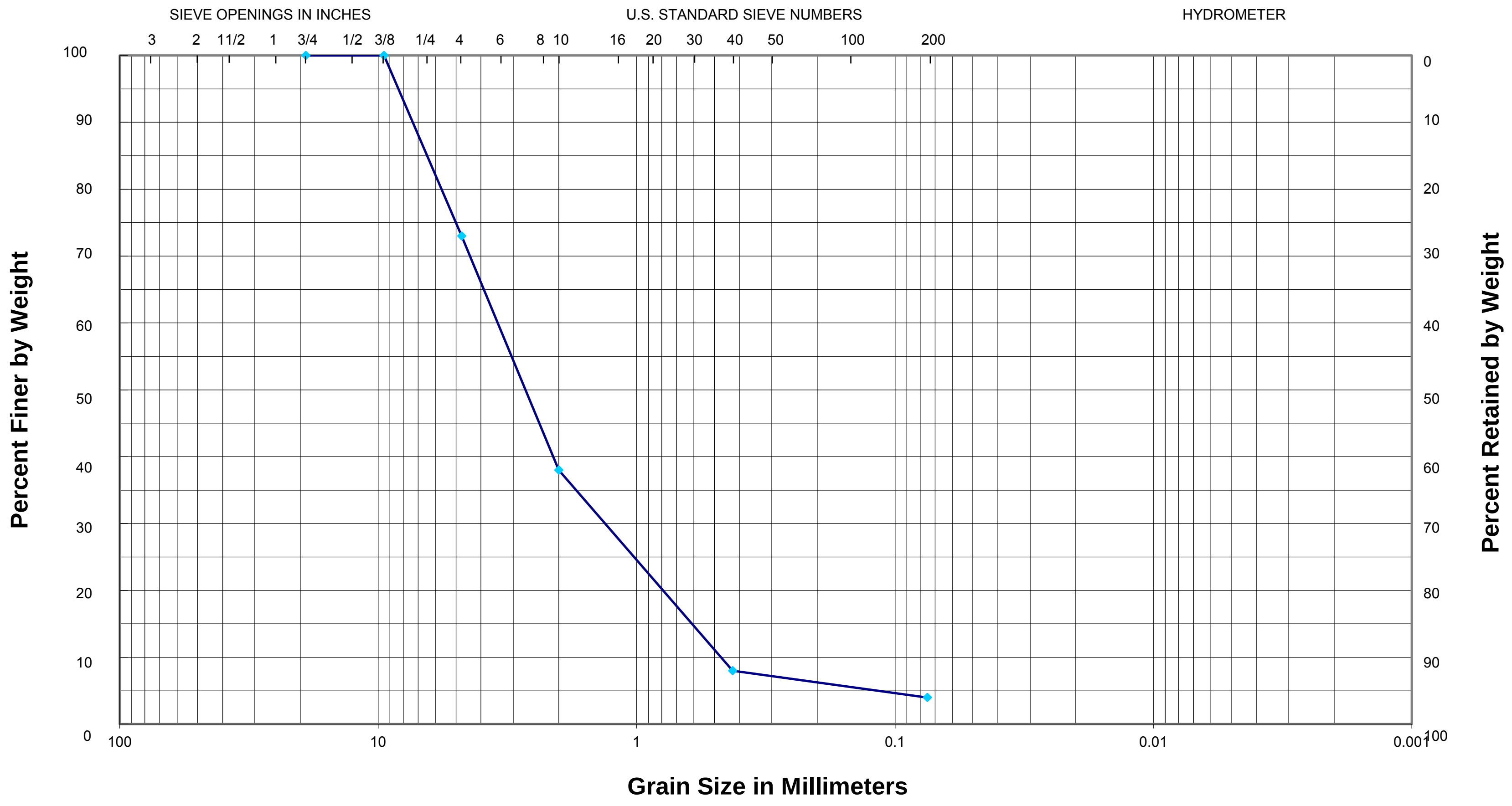
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 21, 44-45 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine to coarse sand w/a little fine gravel
Classification: USCS = SP; AASHTO = A-1-3

13-017

GRAIN SIZE CURVE



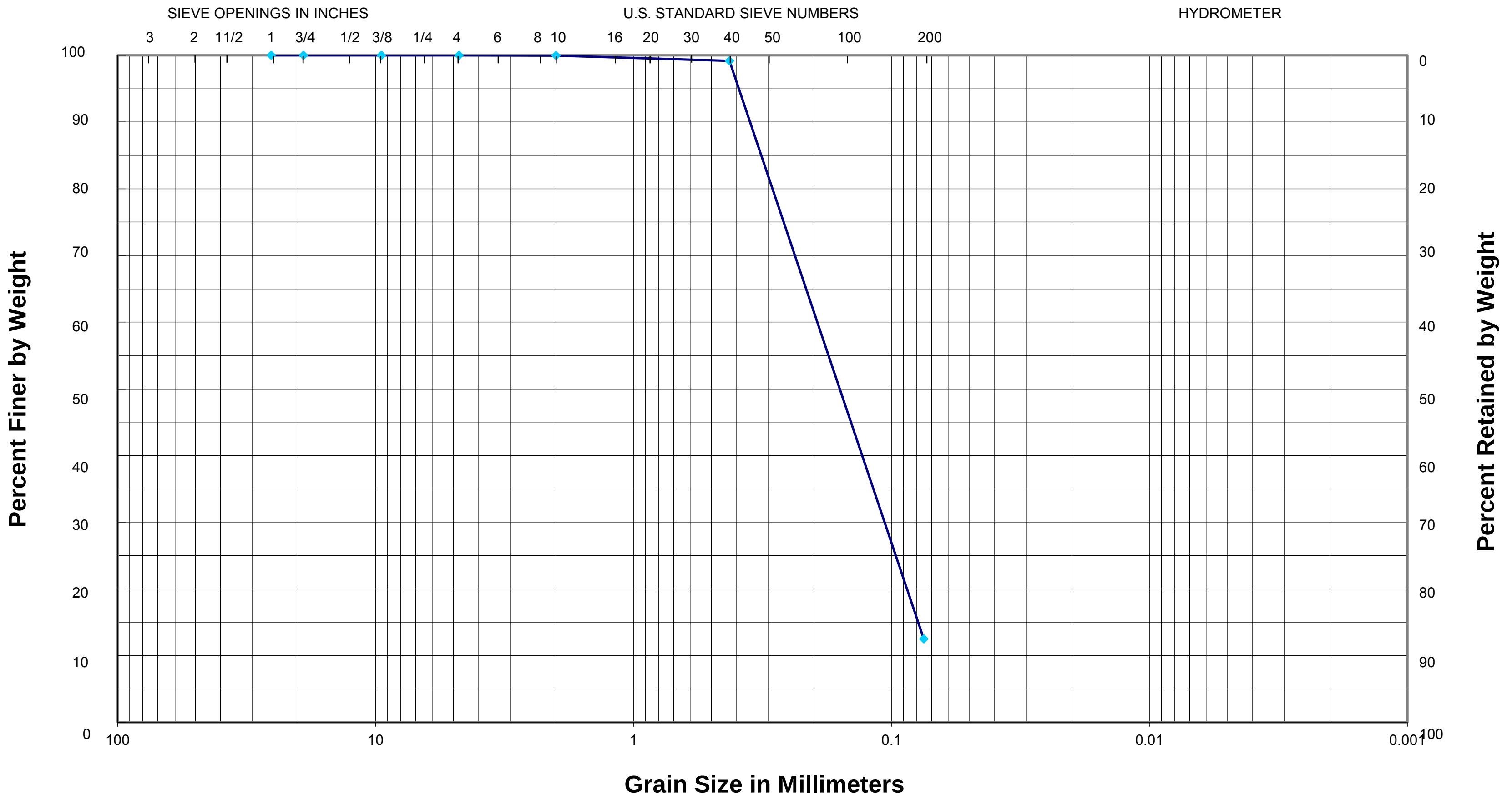
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 21, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Gray and tan sandy fine gravel
Classification: USCS = SP; AASHTO = A-1-a

13-017

GRAIN SIZE CURVE



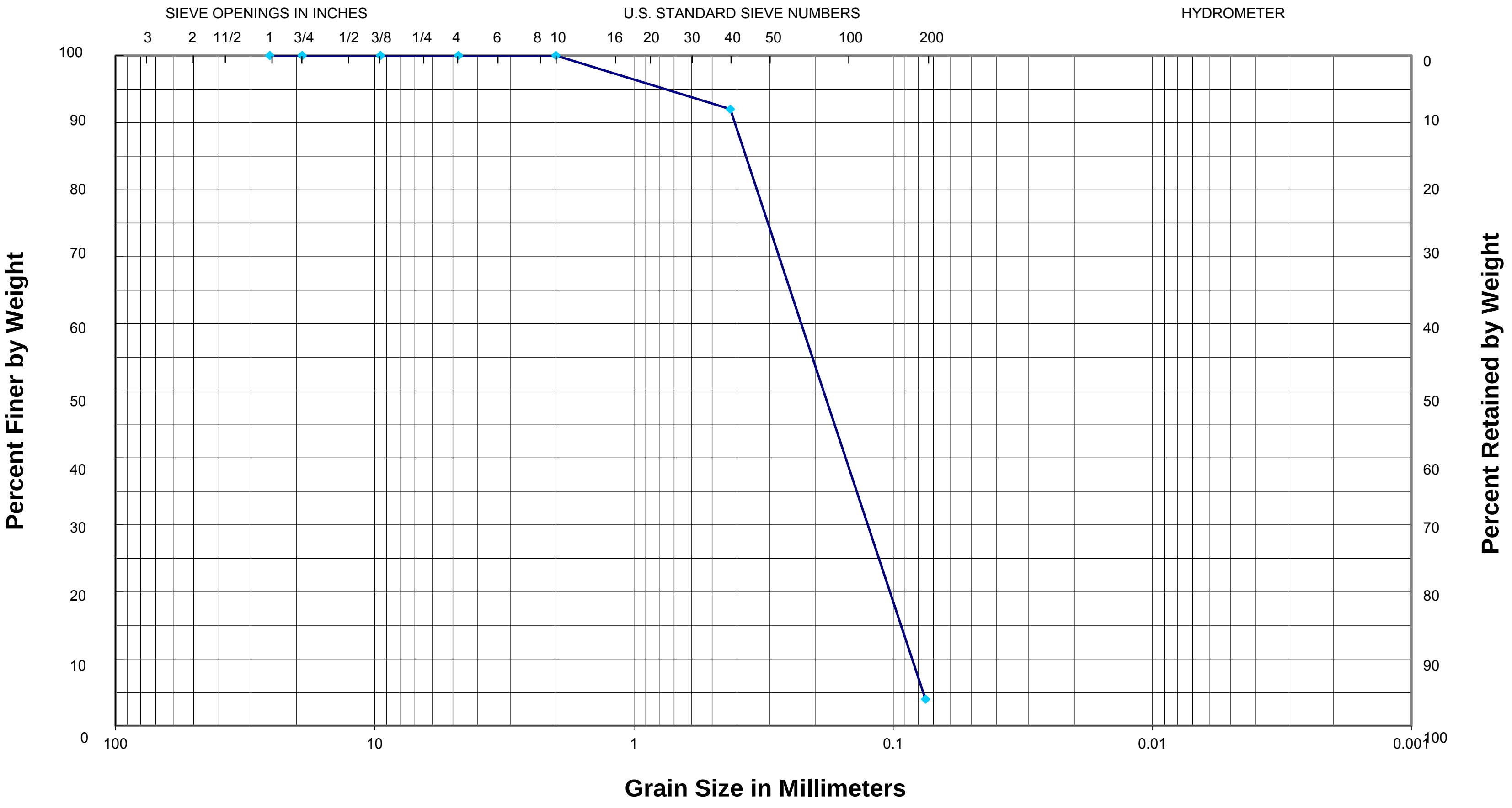
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



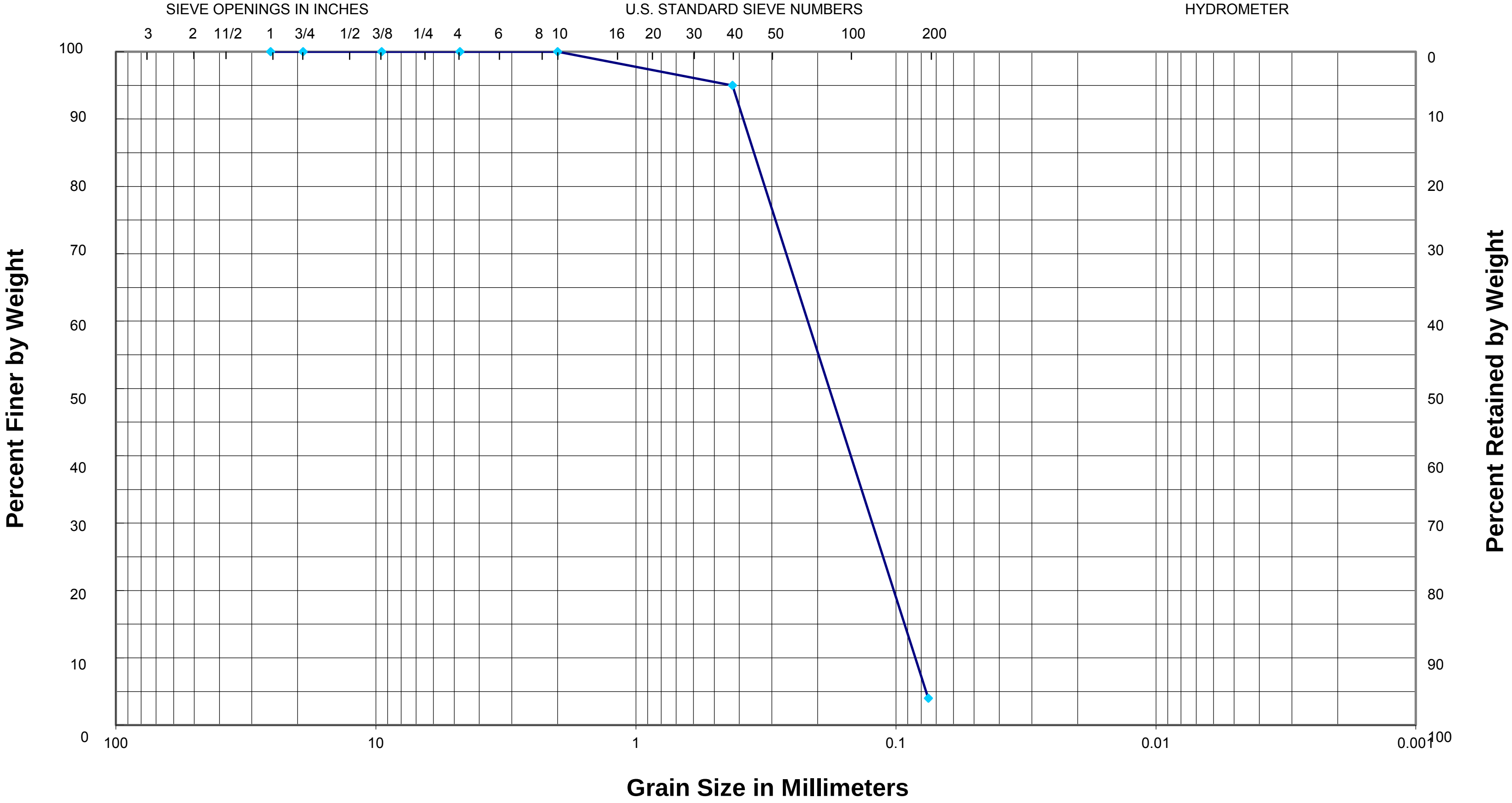
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 88.5-89.5 ft
Atterberg Limits: Non-plastic

Description: Gray and brown sandy fine gravel
Classification: USCS = SP; AASHTO = A-1-b

13-017

GRAIN SIZE CURVE



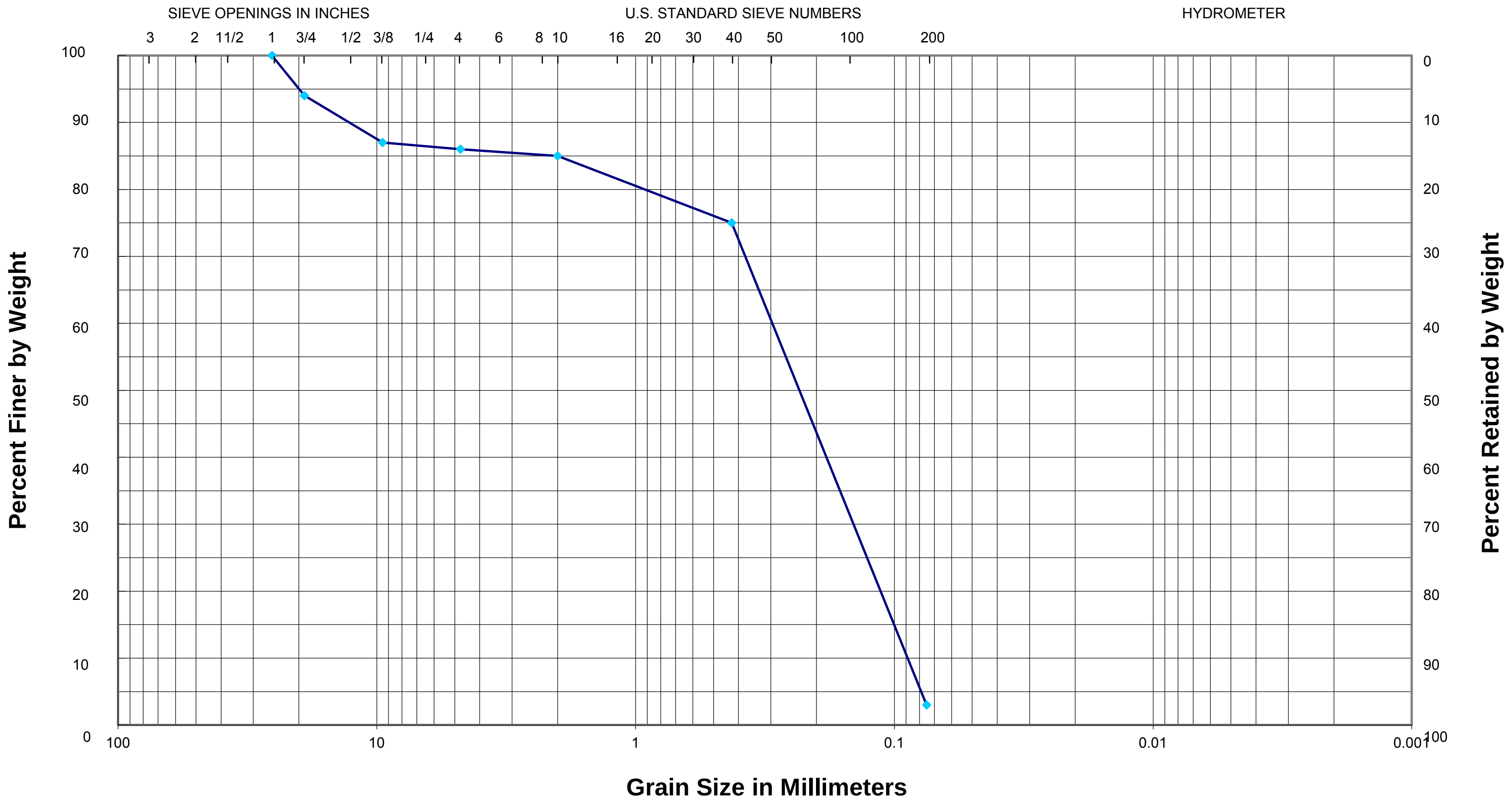
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 24-25 ft
Atterberg Limits: Non-plastic

Description: Grayish brown fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



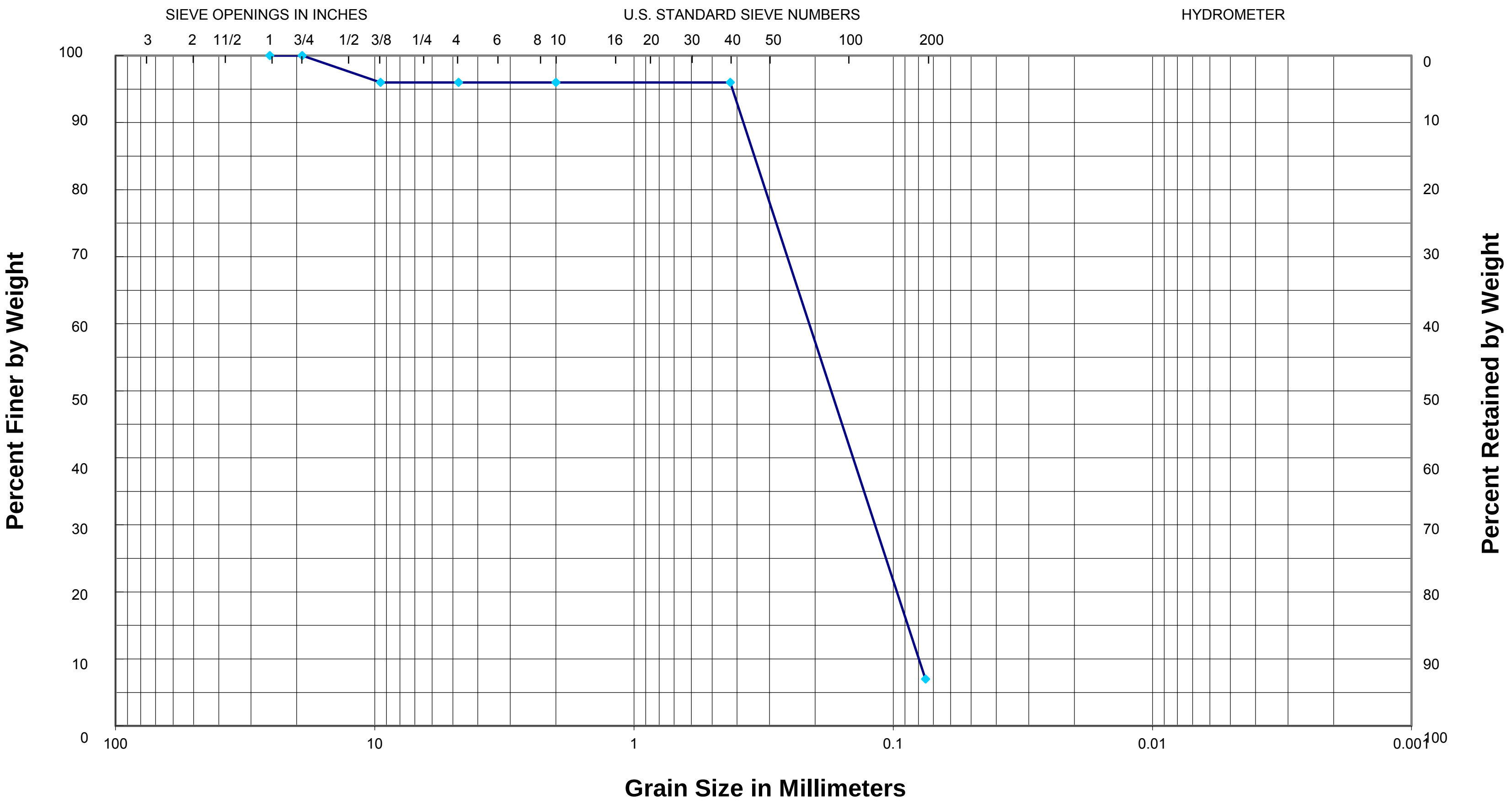
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 68.5-69.5 ft
Atterberg Limits: Non-plastic

Description: Brownish gray fine to medium sand w/a little coarse sand and fine gravel
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



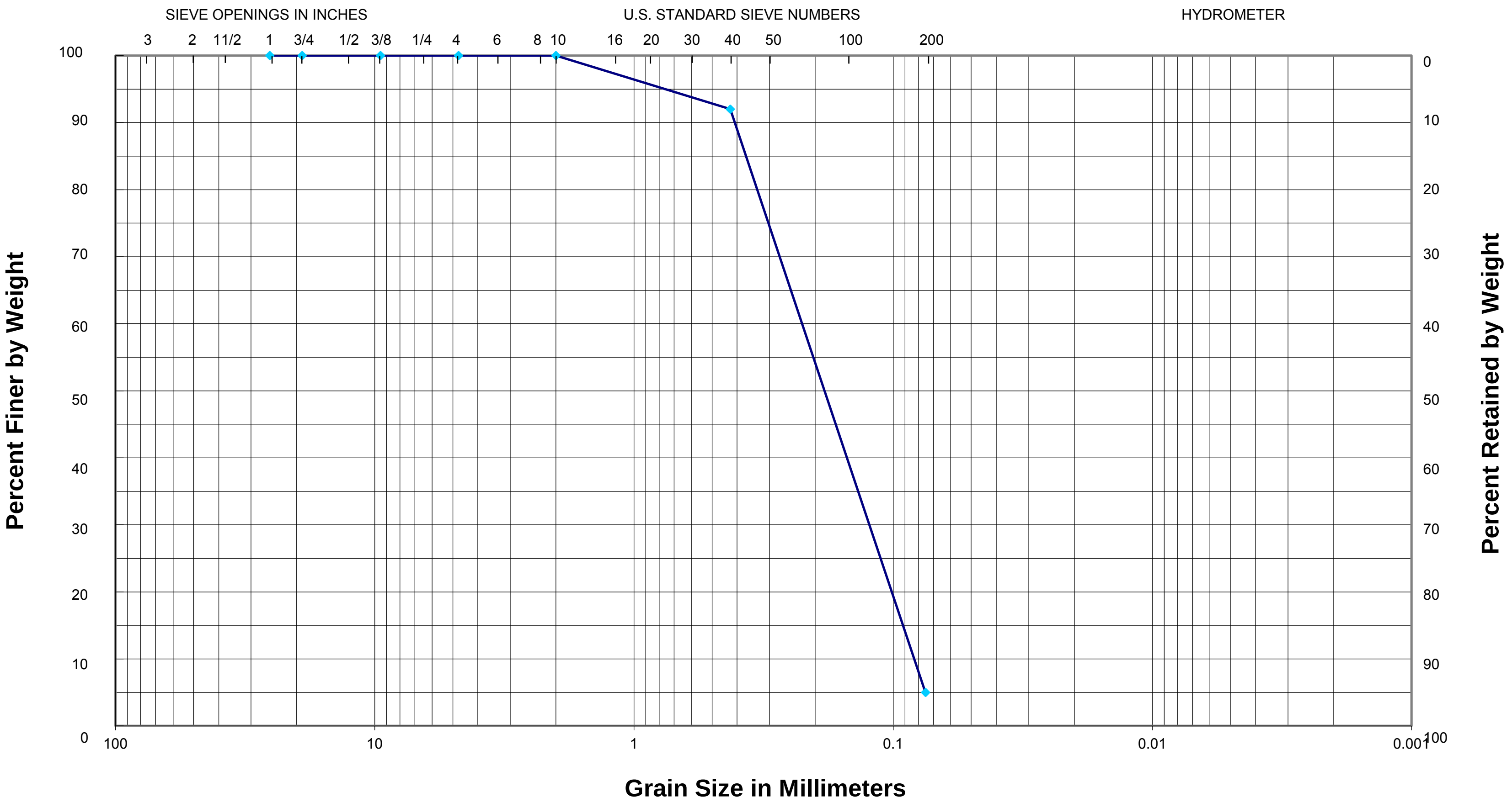
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



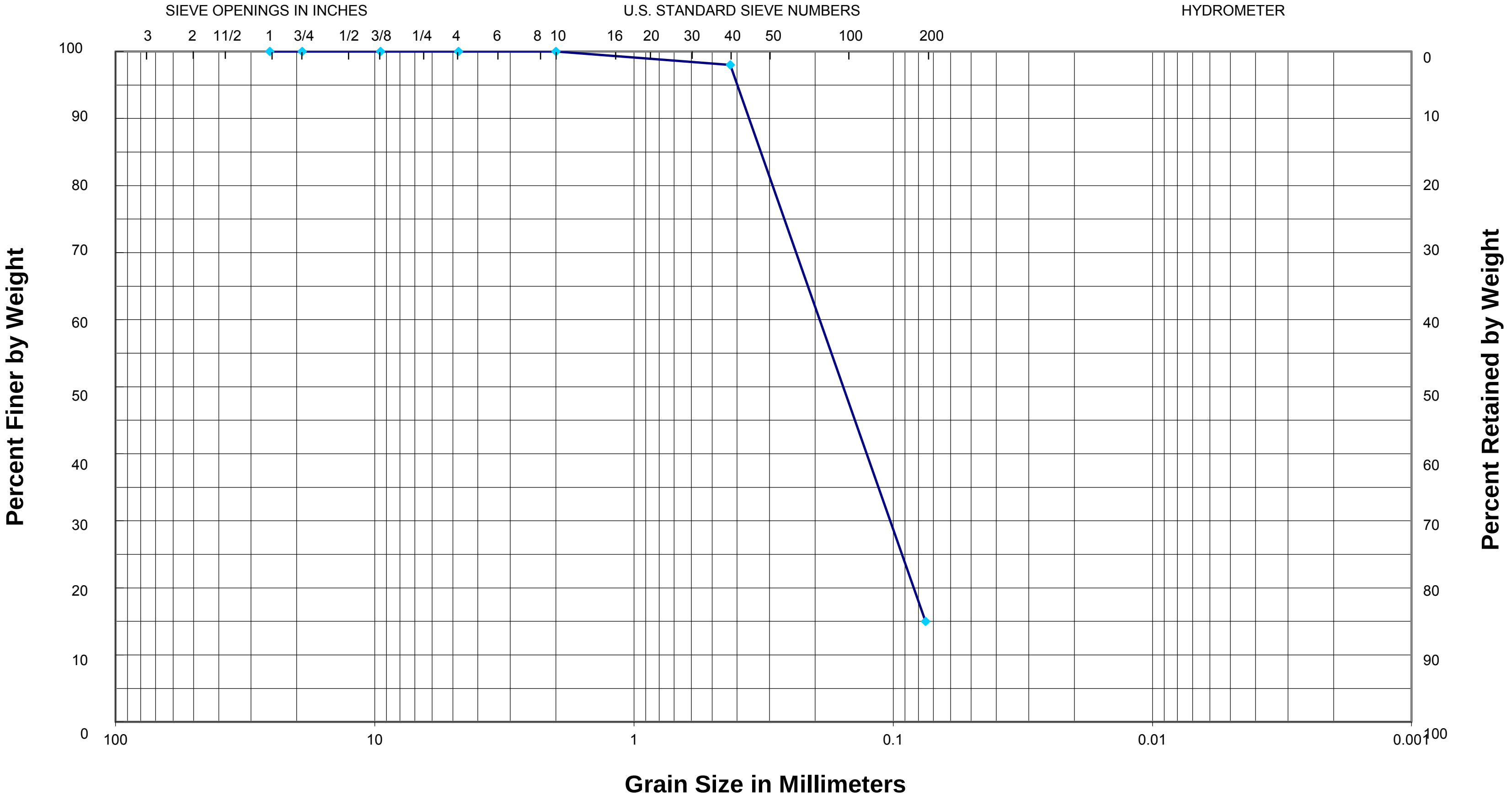
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 64-65 ft
Atterberg Limits: Non-plastic

Description: Tan and gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 25, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

DIRECT SHEAR (AASHTO T-236) TEST RESULTS

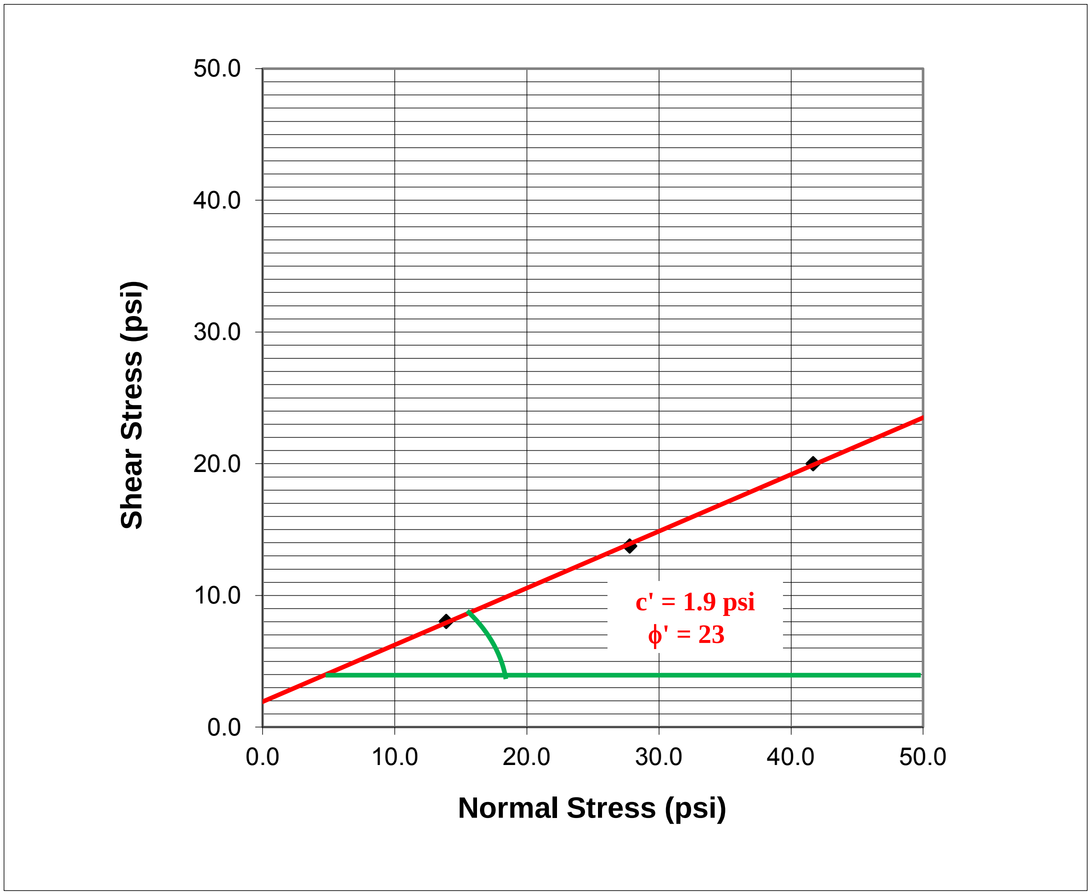
(Consolidated Drained)

Job No.: 13-017
Project: I-40 over White River
Prairie County, Arkansas
Boring No.: 2
Depth, ft: 4.5-5
Sample Description: Brown and gray clay with clay pockets and ferrous stains

Tested By: MM
Reported by: YZ
Test Date: 5/21/2013

Material Properties: LL = 52, PL = 23, PI = 29; Minus No. 200 Sieve = 98%; USCS = CH; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	13.9	8.0	93.6	27.7
2	27.8	13.8	96.5	25.0
3	41.7	20.0	95.4	26.6



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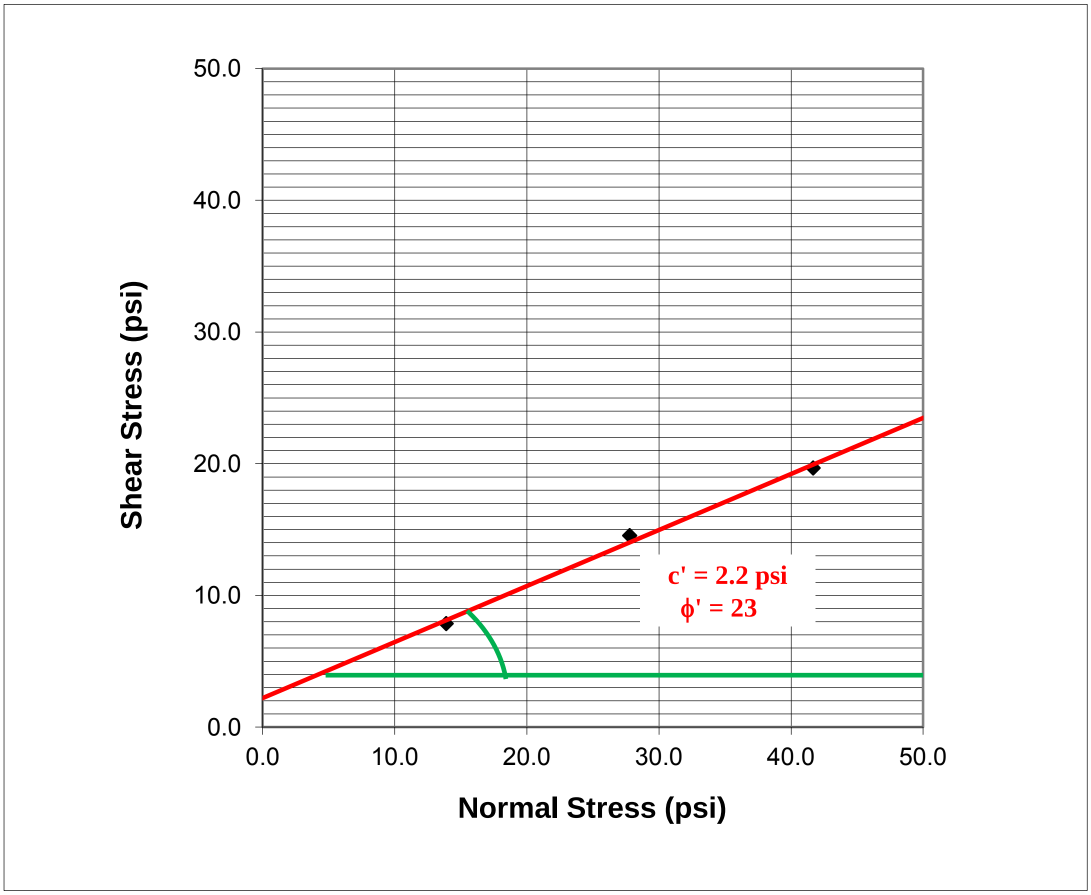
DIRECT SHEAR (AASHTO T-236) TEST RESULTS
(Consolidated Drained)

Job No.: 13-017
Project: I-40 over White River
Prairie County, Arkansas
Boring No.: 3
Depth, ft: 7-7.5
Sample Description: Brown and gray silty clay with ferrous stains

Tested By: MM
Reported by: YZ
Test Date: 5./30/2013

Material Properties: LL = 47, PL = 21, PI = 26; Minus No. 200 Sieve = 95%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	13.9	7.9	99.0	23.7
2	27.8	14.5	99.3	24.0
3	41.7	19.7	100.6	22.9



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DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.: 13-017

Project: I-40 over White River
Prairie County, Arkansas

Boring No.: 4

Depth, ft: 7-7.5

Sample Description: Gray and brown silty clay

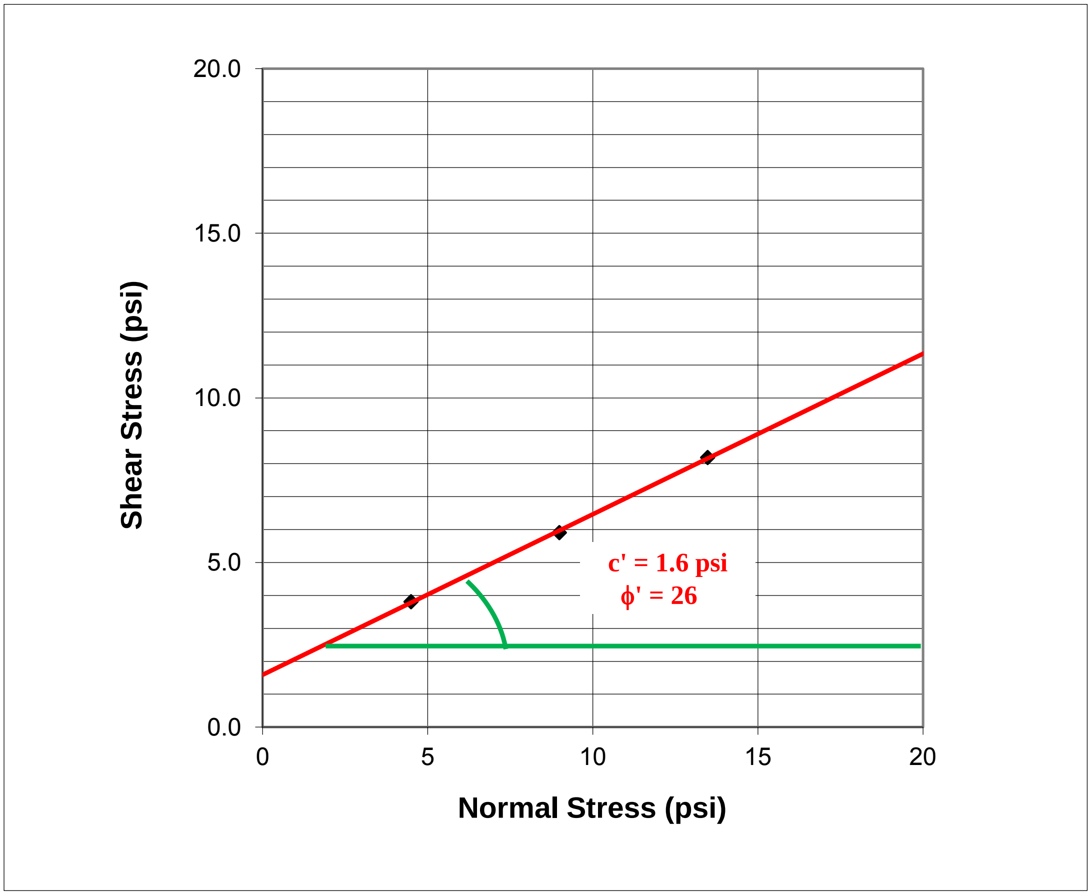
Tested By: MM

Reported by: YZ

Test Date: 3/12/2014

Material Properties: LL = 46, PL = 19, PI = 27; Minus No. 200 Sieve = 97%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	3.8	100.2	23.5
2	9.0	5.9	99.2	23.5
3	13.5	8.2	99.0	23.5



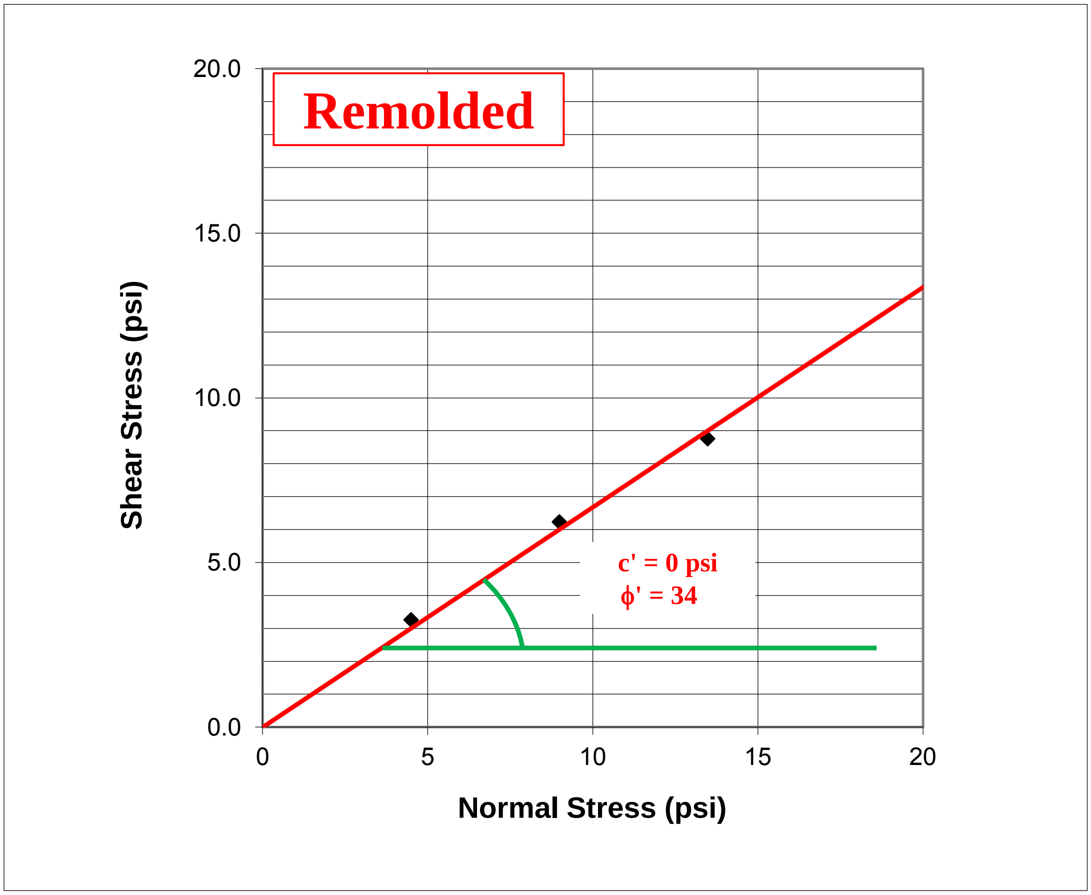
Grubbs, Hoskyn, Barton & Wyatt, Inc.
CONSULTING ENGINEERS

DIRECT SHEAR (AASHTO T-236) TEST RESULTS
(Consolidated Drained)

Job No.:	13-017	Tested By:	MM
Project:	I-40 over White River	Reported by:	YZ
	Prairie County, Arkansas	Test Date:	2/25/2014
Boring No.:	22		
Depth, ft:	14-25		
Sample Description:	Brown silty fine sand		

Material Properties: Non-plastic; Minus No. 200 Sieve = 13%; USCS = SM; AASHTO = A-4

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	3.3	96.8	24.5
2	9.0	6.2	97.4	24.5
3	13.5	8.8	97.8	24.5



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

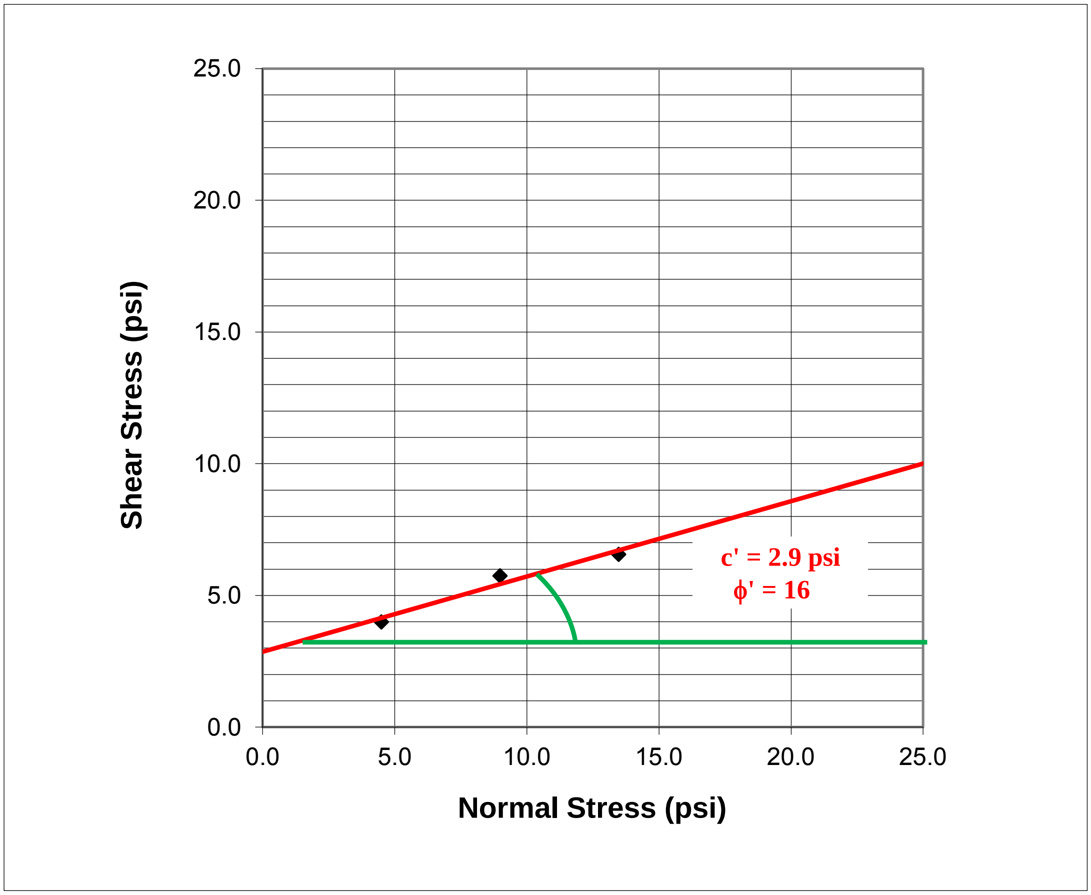
DIRECT SHEAR (AASHTO T-236) TEST RESULTS

(Consolidated Drained)

Job No.:	13-017	Tested By:	MM
Project:	I-40 over White River	Reported by:	YZ
	Prairie County, Arkansas	Test Date:	5/13/2013
Boring No.:	29		
Depth, ft:	14-14.5		
Sample Description:	Reddish brown clay		

Material Properties: LL = 70, PL = 24, PI = 46; Minus No. 200 Sieve = 99%; USCS = CH; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	4.0	90.2	30.5
2	9.0	5.7	89.5	31.5
3	13.5	6.6	91.6	30.3



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DIRECT SHEAR (AASHTO T-236) TEST RESULTS

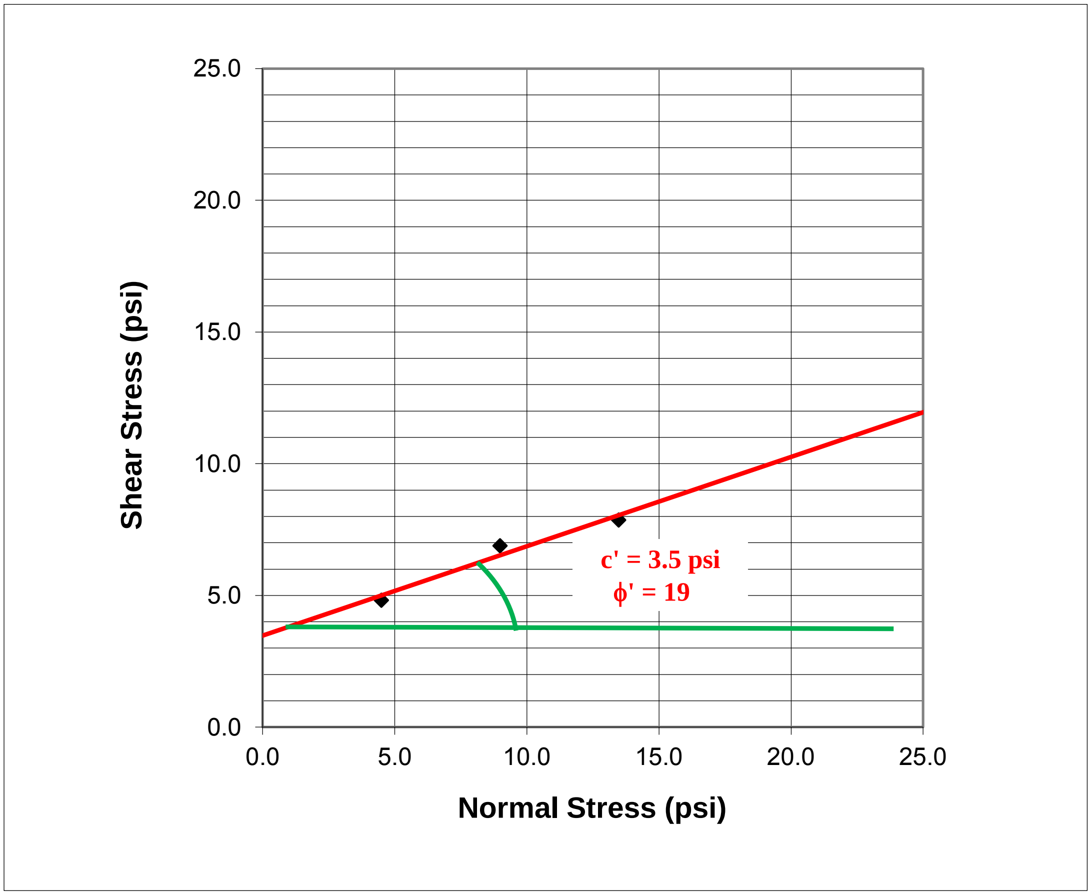
(Consolidated Drained)

Job No.:	13-017	Tested By:	MM
Project:	I-40 over White River	Reported by:	YZ
	Prairie County, Arkansas	Test Date:	5/13/2013
Boring No.:	30		
Depth, ft:	9.5-10		

Sample Description: Tan and gray clay with fine sandy silt pockets

Material Properties: LL = 50, PL = 18, PI = 32; Minus No. 200 Sieve = 87%; USCS = CL; AASHTO = A-7-6

Specimen	Normal Stress, σ'_v (psi)	Shear at failure, τ_f (psi)	Unit Dry Wt, γ_d (lb/ft^3)	Moisture Content, w (%)
1	4.5	4.8	101.0	24.0
2	9.0	6.9	105.6	19.5
3	13.5	7.9	104.4	20.5



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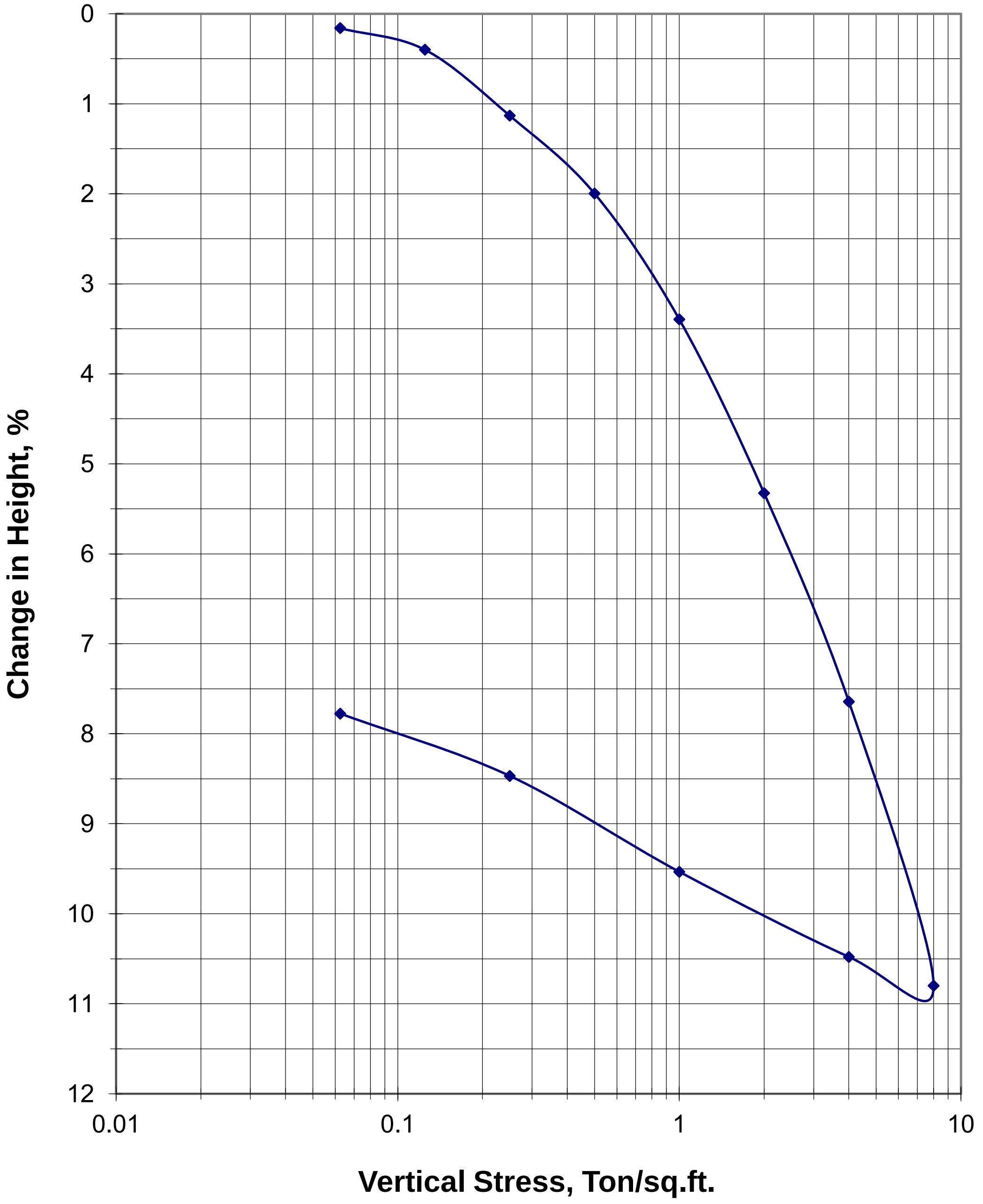
CONSULTING ENGINEERS

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 1
Depth: 9.5-10 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 95.0 pcf
Initial Water Content: 26.4%
Final Water Content: 24.5%
Liquid Limit: 43
Plastic Limit: 19
Minus #200: 97%



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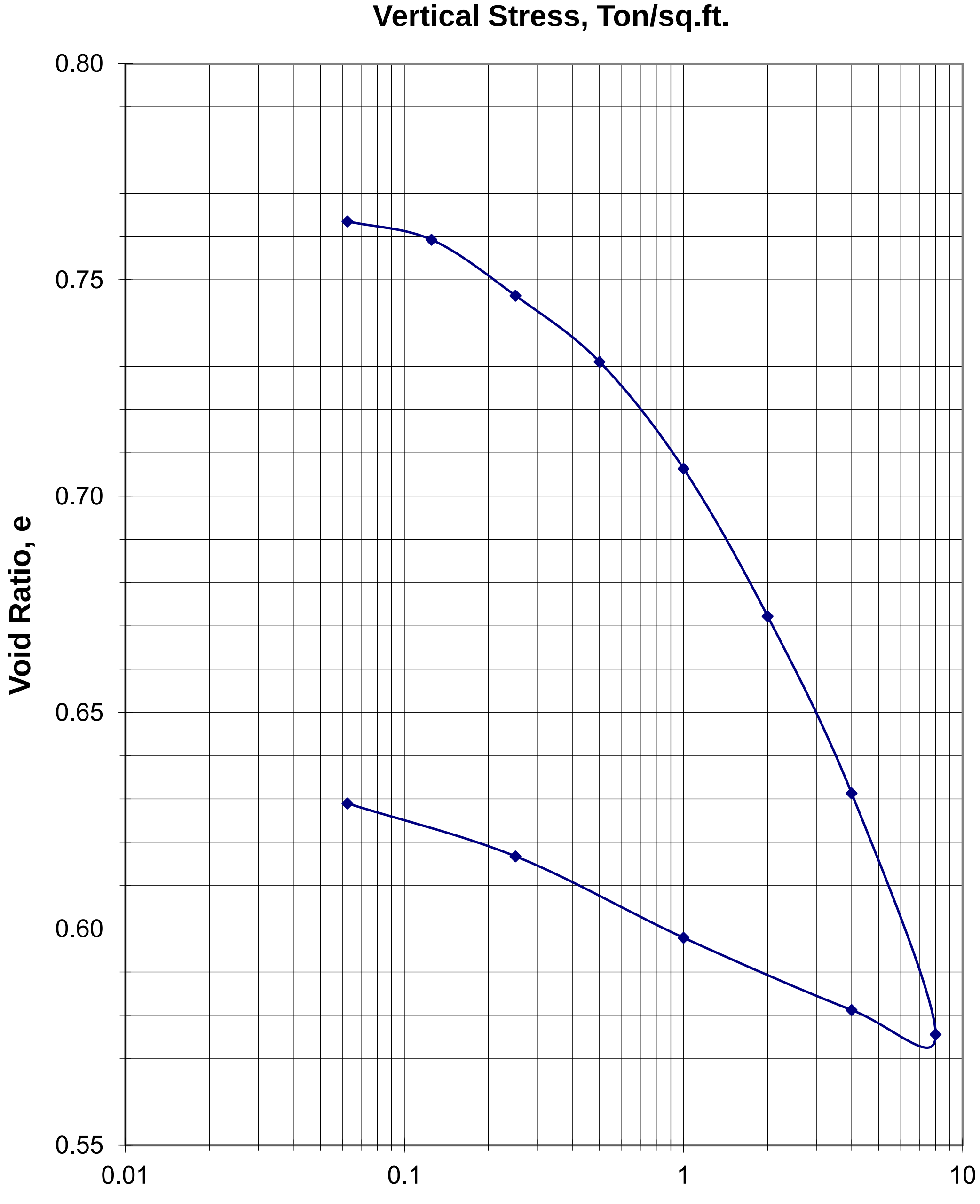
CONSULTING ENGINEERS

PLATE

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 1
Depth: 9.5-10 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 95.0 pcf
Initial Water Content: 26.4%
Final Water Content: 24.5%
Liquid Limit: 43
Plastic Limit: 19
Minus #200: 97%



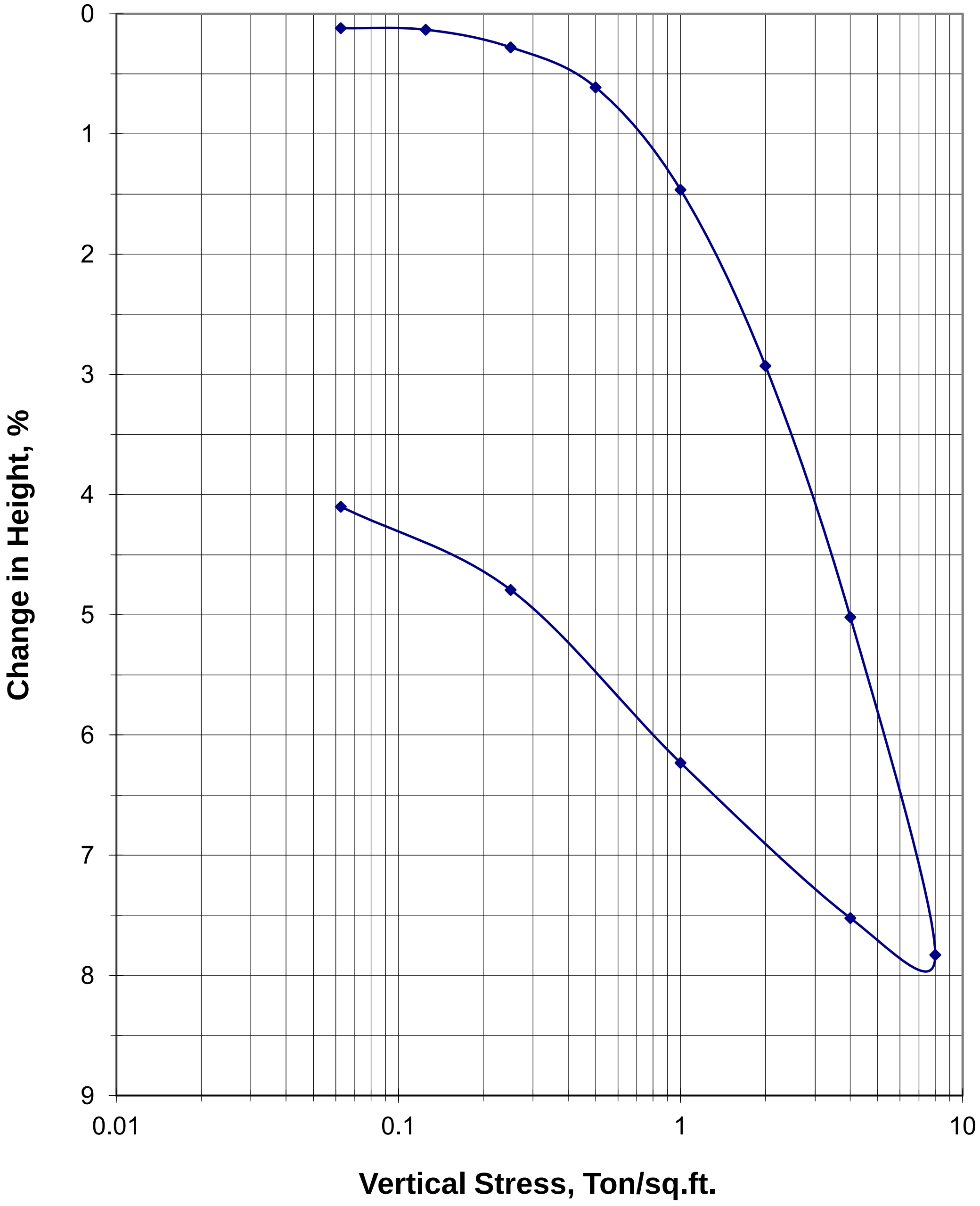
Grubbs, Hoskyn, Barton & Wyatt, Inc.
CONSULTING ENGINEERS

PLATE

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 3
Depth: 4.5-5 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

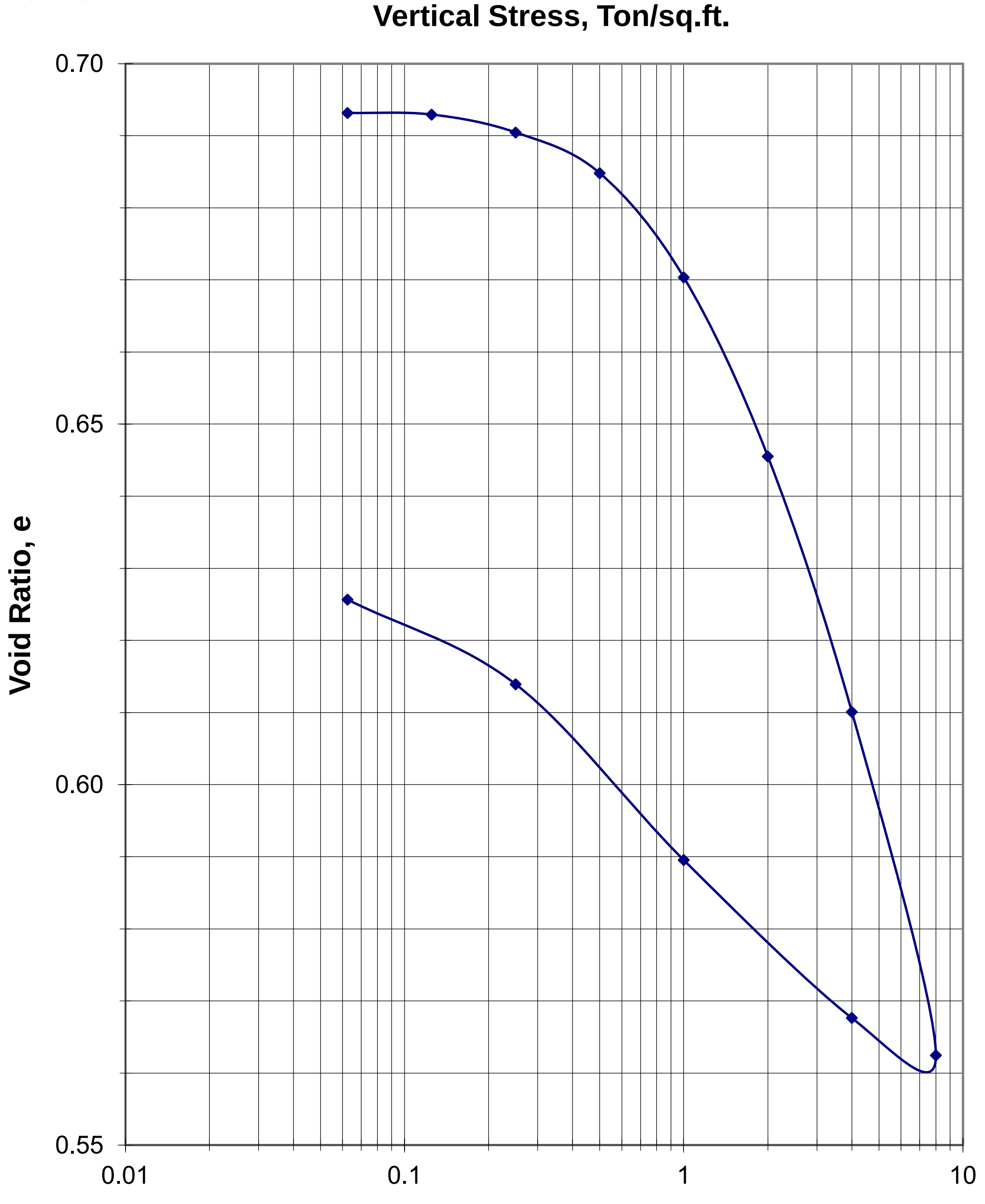
Unit Dry Weight: 99.0 pcf
Initial Water Content: 24.5%
Final Water Content: 25.1%
Liquid Limit: 48
Plastic Limit: 19
Minus #200: 95%



CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 3
Depth: 4.5-5 ft
Description: Brown and gray silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 99.0 pcf
Initial Water Content: 24.5%
Final Water Content: 25.1%
Liquid Limit: 48
Plastic Limit: 19
Minus #200: 95%



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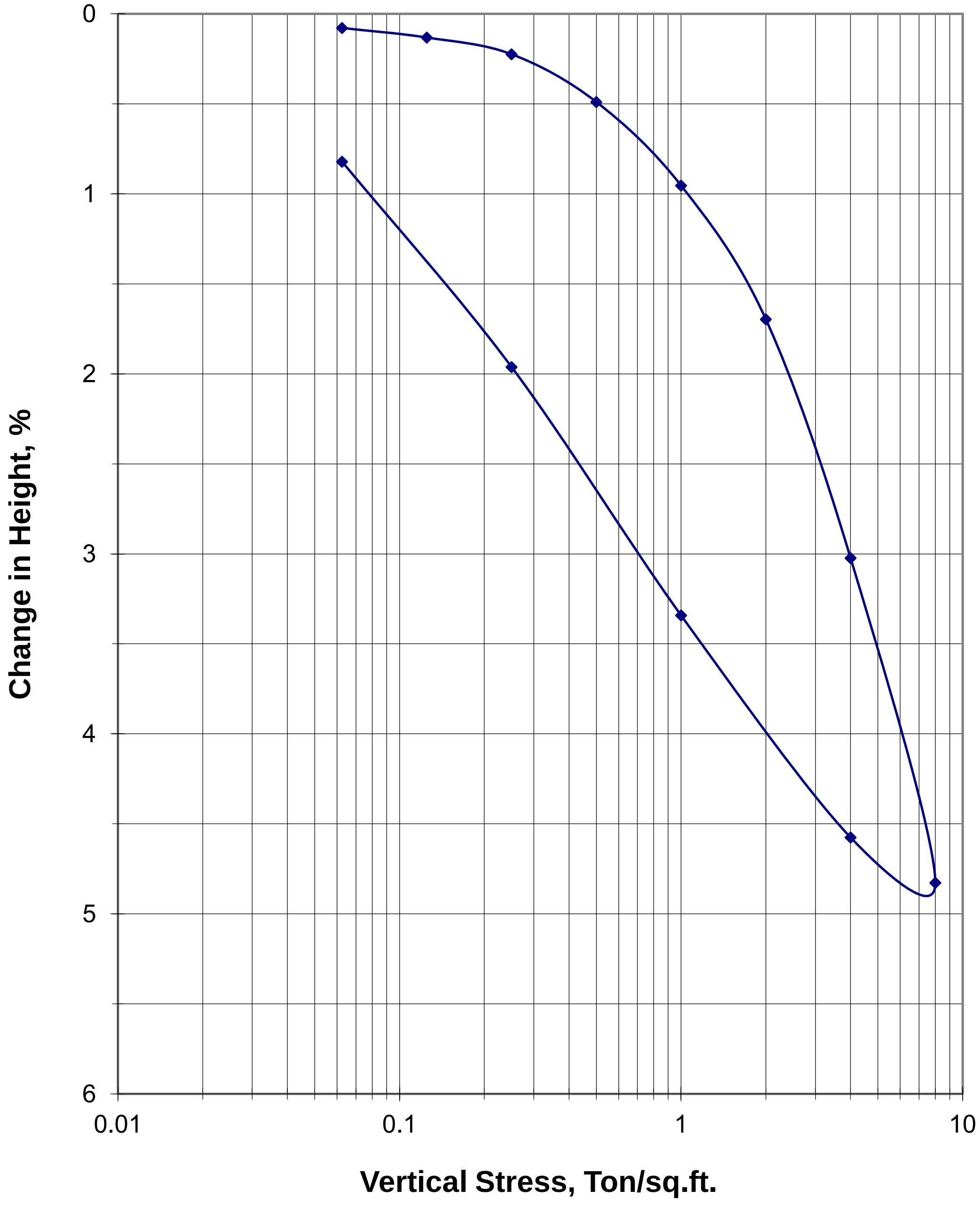
CONSULTING ENGINEERS

PLATE

CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 23
Depth: 7-7.5 ft
Description: Brown and tan silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 99.9 pcf
Initial Water Content: 21.3%
Final Water Content: 27.1%
Liquid Limit: 44
Plastic Limit: 19
Minus #200: 97%



CONSOLIDATION TEST RESULTS
(AASHTO T-216)

GHBW Job No.: 13-017
Project: I-40 over White River
Boring: 23
Depth: 7-7.5 ft
Description: Brown and tan silty clay
USCS = CL
AASHTO = A-7-6

Unit Dry Weight: 99.9 pcf
Initial Water Content: 21.3%
Final Water Content: 27.1%
Liquid Limit: 44
Plastic Limit: 19
Minus #200: 97%

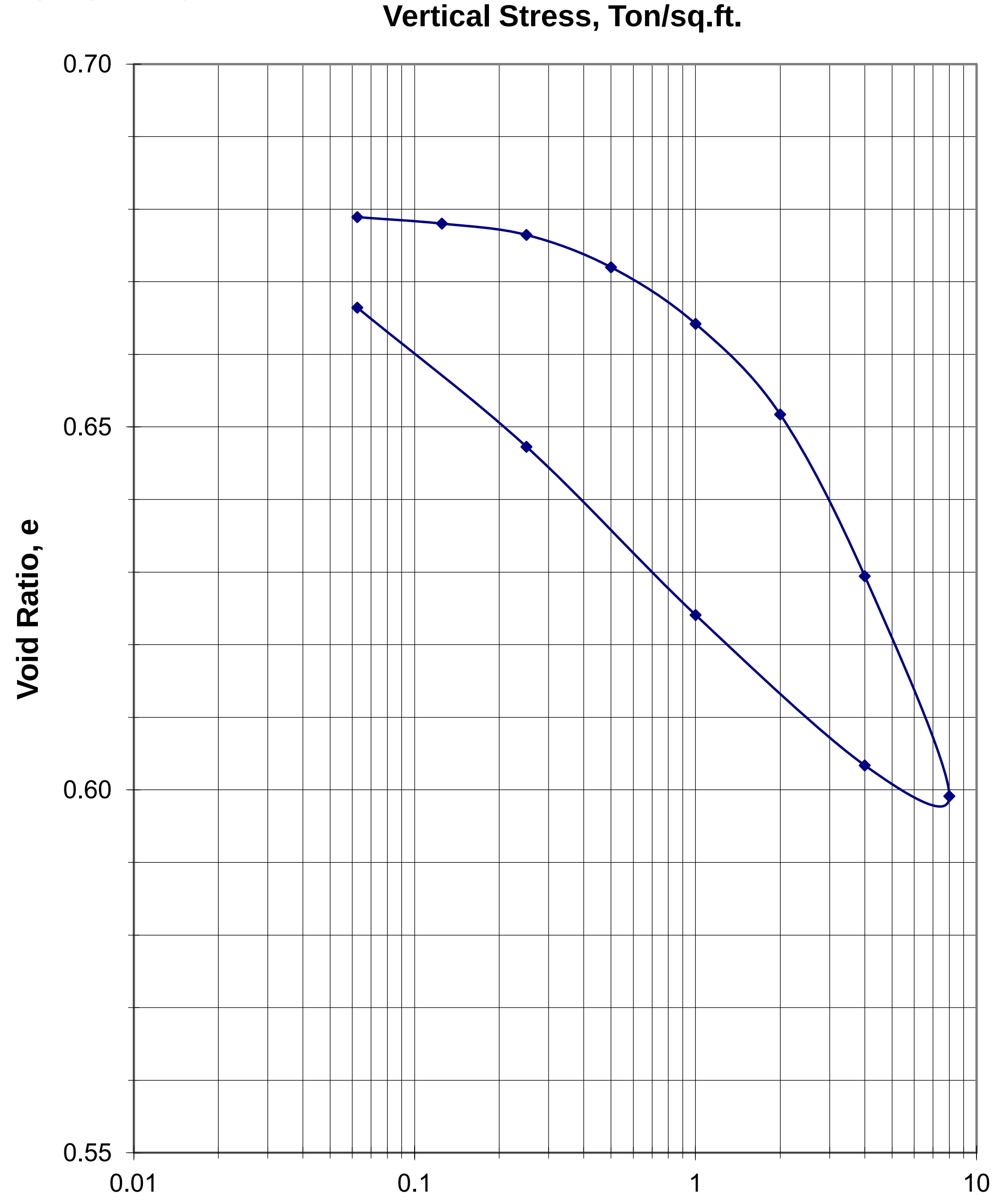


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PLATE

APPENDIX D

Final Report:

Site-Specific Ground Motion Response Analysis Results for the I-40 Replacement Bridge over the White River
(AHTD Job BB0610)

for:

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President
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The University of Texas

May 27, 2014

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EXECUTIVE SUMMARY

Site-specific seismic ground motion response analyses using a weighted combination of equivalent linear (EQL) and nonlinear (NL) methods were used to develop median ground surface spectral acceleration (Sa) estimates for both sides the I-40 replacement bridge over the White River. The final results (i.e., delineated design acceleration response spectra) are summarized below in Figures 3-5 and Table 1. All site-specific ground motion response analyses were conducted in accordance with Section 3.4.3.2 in the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition.

Shear wave velocity (Vs) profiles were determined on both sides of the White River (hereafter referred to as the West and East sides) using a combination of active-source and ambient-wavefield surface wave methods. The use of active-source methods (i.e., the MASW method with both sledgehammer and vibroseis truck sources) allowed for resolution of near-surface layering details, while the use of 2D ambient-wavefield methods (i.e., microtremor array measurements, or MAM) allowed for deep subsurface profiling. The median Vs profile developed for the **West side of the river resulted in an average Vs30m/Vs100ft value of 171 m/s (560 ft/sec); a Site Class E** designation according to Table 3.4.2.1-1 in AASHTO (2011). The median Vs profile developed for the **East side of the river resulted in an average Vs30m/Vs100ft value of 207 m/s (680 ft/sec); a Site Class D** designation. The Vs profiles developed for the West and East side of the river are provided in Figures E1 and E2, respectively. They are discussed in greater detail in the main body of this report.

The Vs profiles on both sides of the river extend to depths exceeding 400 m (1300 ft), where material with an average shear wave velocity greater than 760 m/s (2500 ft/sec) (i.e., Site Class B rock) was encountered. The uncertainty in Vs was accounted for in the site response analyses by adjusting the derived Vs profiles by +/- 15%, and depth to "engineering bedrock", where ground motions were input, by +/- 20%. Uncertainty in the dynamic soil properties assigned to each soil and rock layer was accounted for by using the median and +/- one standard deviation normalized modulus reduction (G/Gmax) and damping (D) curves proposed by Darendeli (2001). Sets of these curves were strength corrected at high shear strains for compatibility with the measured soil shear strengths in the near-surface soil layers.

NL and EQL site response analyses were performed using the 1D site response program DEEPSOIL (http://deepsoil.cee.illinois.edu/). Four ground motions from the NUREG database were spectrally matched to the Site Class B Uniform Hazard Spectrum and used as the input ground motions for this study. The log-normal median surface response resulting from propagating these four input ground motions through each distinct set of Vs and dynamic soil properties was weighted and combined via a logic tree approach in order to determine an overall/"fully weighted" surface response for the West and East sides of the river, respectively. These fully weighted results were used to determine spectral ratios between the median input ground motions and median surface ground motions, which were then multiplied by the Site Class B generic/general acceleration response spectrum in order to determine the median site-specific response spectrum on each side of the river.

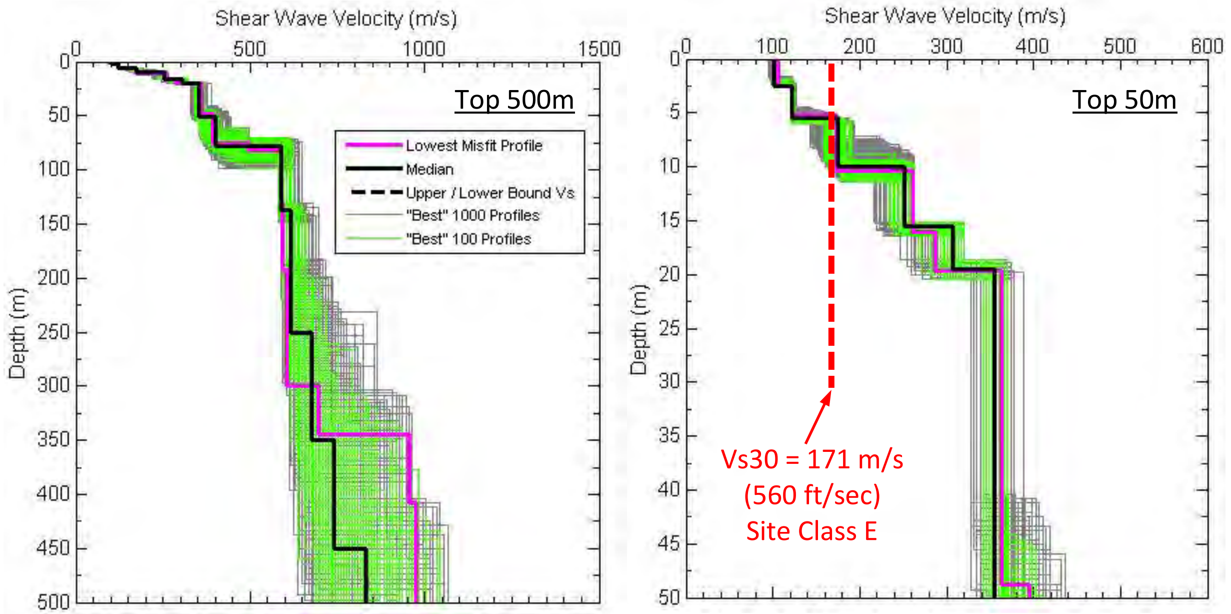

Figure E1. Shear wave velocity (Vs) profiles for the West side of the I-40 White River Bridge. The "best" 1000, "best" 100, and "best"/lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 171 m/s (560 ft/sec), resulting in a Site Class E seismic site classification.

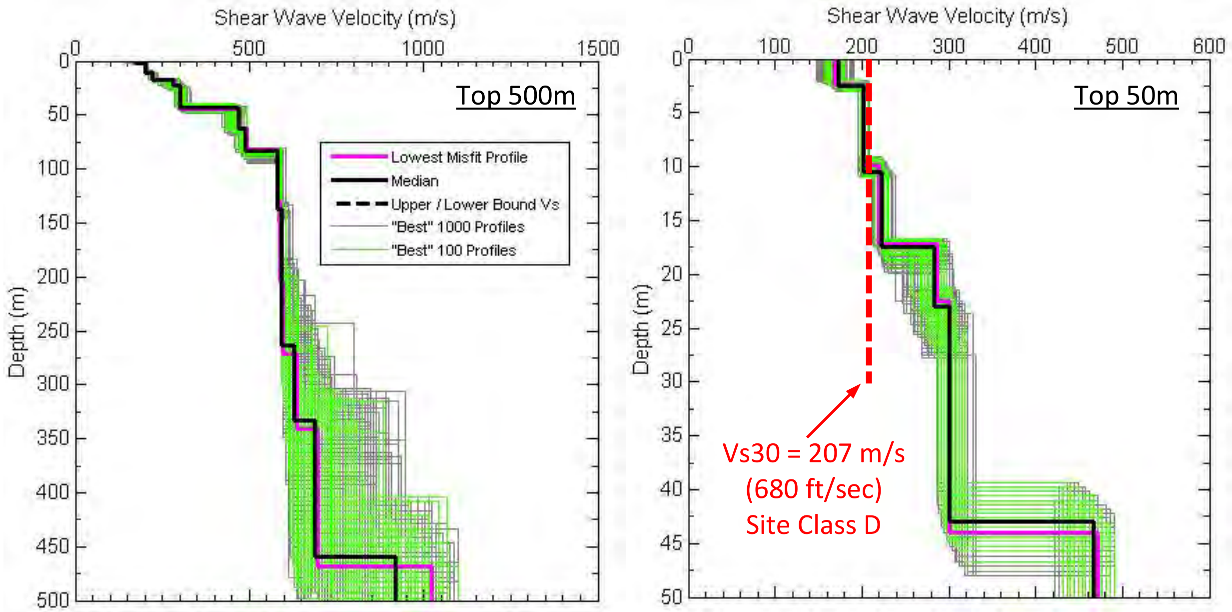

Figure E2. Shear wave velocity (Vs) profiles for the East side of the I-40 White River Bridge. The "best" 1000, "best" 100, and "best"/lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 207 m/s (680 ft/sec), resulting in a Site Class D seismic site classification.

The final results from the site-specific ground motion response analyses are detailed in Figures E3 and E4 for the West and East side of the river, respectively. As noted above, the West side of the river was found to classify as Site Class E in the vicinity where Vs testing was conducted (refer to Figure 1, below, for location). Therefore, the Site Class E spectrum based on the general AASHTO (2011) procedure (Section 3.4.1) is shown for reference in Figure E3. Also shown for reference is a spectrum that is equivalent to 2/3 of the Site Class E general spectrum, because according to Section 3.4.3 of AASHTO (2011), unless otherwise approved by the Owner, the response spectrum used in design shall not be lower than 2/3 of the general site-adjusted spectrum even if site-specific results indicate it is possible. The median site-specific response spectrum on the West side of the river is provided in Figure E3. This response spectrum is less than the allowable minimum of 2/3 Site Class E up to a period (T) of just over 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit, while still remaining below the general Site Class E spectrum, between periods of approximately 0.2 – 1.0 seconds, after which it falls slightly below the limit. The delineated site-specific design response spectrum, highlighted in Figure E3, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the West side of the river.

The East side of the river was found to classify as Site Class D in the vicinity where Vs testing was conducted (refer to Figure 1, below, for location). Therefore, the Site Class D spectrum based on the general AASHTO (2011) procedure is shown for reference in Figure E4, along with a spectrum equivalent to 2/3 of the Site Class D spectrum. The median site-specific response spectrum on the East side of the river is also provided in Figure E4. This response spectrum is generally less than the allowable minimum of 2/3 Site Class D up to a period (T) of approximately 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit between periods of approximately 0.2 – 3.0 seconds, and is essentially equal to the Site Class D general spectrum between periods of 0.7 – 1.0 seconds. The delineated site-specific design response spectrum, highlighted in Figure E4, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the East side of the river.

The site-specific ground motion response analysis results for both the West and East side of the White River are compared, along with their respective delineated design response spectra, in Figure E5. **Tabulated values of Sa, including PGA/As values for use in liquefaction analyses, for each delineated design response spectrum are also provided in Table E1. A corresponding moment magnitude (Mw) of 7.7 should be used in liquefaction analyses**. This is the modal magnitude at the bridge site for every available period in the 2002 USGS deaggregation tool. While the modal magnitude always stays fixed at Mw = 7.7, the mean magnitude consistently shifts upward from approximately Mw = 7.35 for PGA to Mw = 7.57 for the 2-second period data (maximum period available). Basically, this means the modal magnitude and the mean magnitude are expected to grow closer to one another for soft and/or deep soil sites with large natural periods. Since this site has an approximate natural period of 3 seconds, based on the deep Vs profiles shown above, the mean and modal magnitudes are expected to be very close to one another and approximately equal to Mw = 7.7.

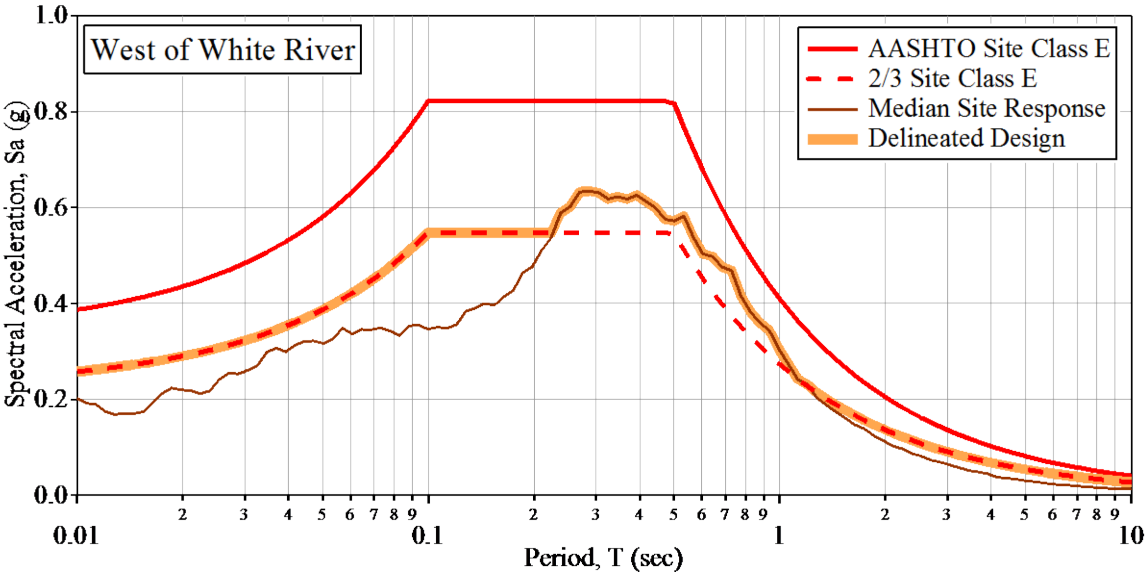

Figure E3. Site-specific ground motion response analysis results for the <u>West side</u> of the White River at I-40.

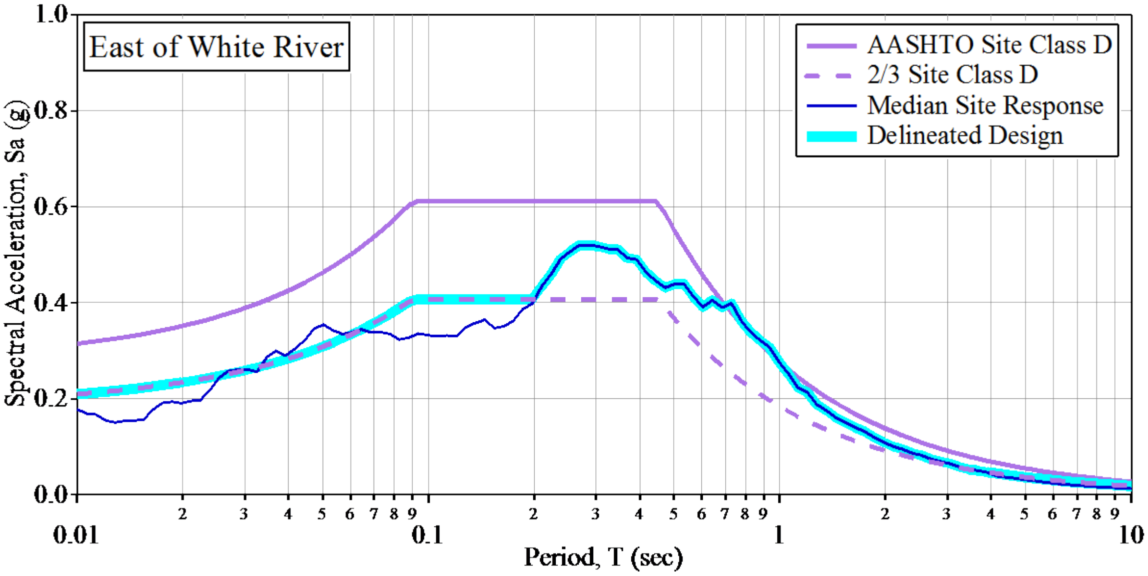

Figure E4. Site-specific ground motion response analysis results for the <u>East side</u> of the White River at I-40.

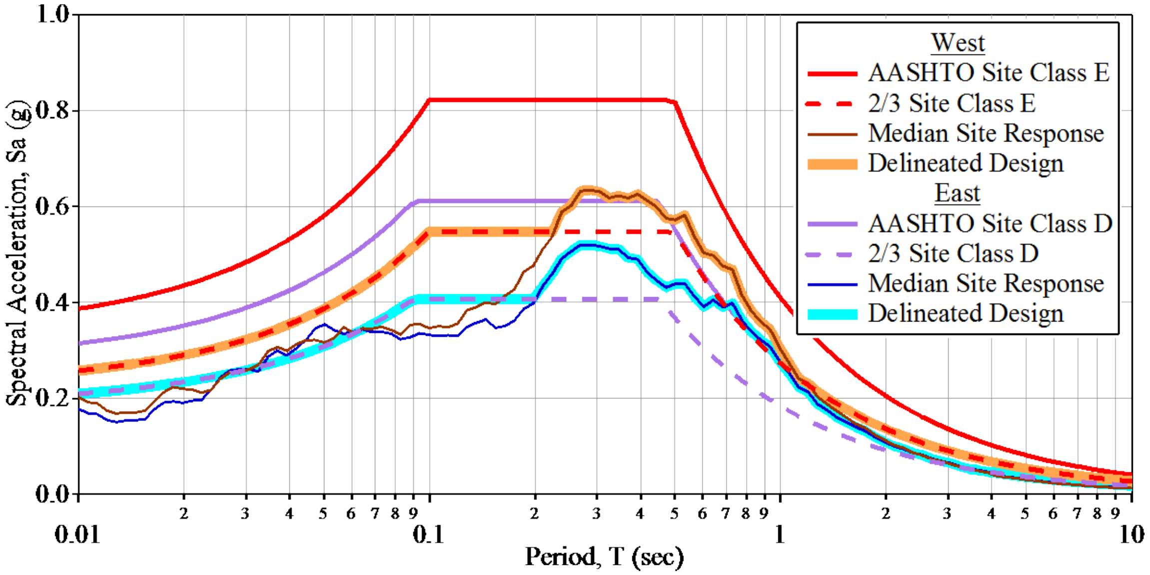

Figure E5. Comparison of site-specific ground motion response analysis results for the West and East side of the White River at I-40.

Table E1. Spectral acceleration (Sa) values for the delineated design response spectra obtained from site-specific ground motion response analyses for the West and East side of the White River at I-40.

West of White River	
Period (sec)	Delineated Design (g)
PGA	0.23
0.010	0.258
0.014	0.271
0.018	0.284
0.022	0.297
0.026	0.310
0.030	0.323
0.040	0.355
0.050	0.387
0.060	0.419
0.070	0.452
0.080	0.484
0.090	0.516
0.10	0.547
0.20	0.547
0.22	0.547
0.24	0.590
0.26	0.615
0.28	0.634
0.30	0.632
0.34	0.621
0.36	0.619
0.42	0.611
0.48	0.574
0.54	0.574
0.60	0.508
0.70	0.473
0.80	0.399
0.90	0.353
1.0	0.302
1.2	0.231
1.4	0.195
1.6	0.171
1.8	0.152
2.0	0.137
2.5	0.109
3.0	0.091
3.5	0.078
4.0	0.068
5.0	0.055
6.0	0.045
7.0	0.039
8.0	0.034
9.0	0.030
10.0	0.027

East of White River	
Period (sec)	Delineated Design (g)
PGA	0.17
0.010	0.210
0.014	0.220
0.018	0.230
0.022	0.239
0.026	0.249
0.030	0.259
0.040	0.284
0.050	0.309
0.060	0.333
0.070	0.358
0.080	0.383
0.090	0.404
0.10	0.408
0.20	0.413
0.22	0.453
0.24	0.494
0.26	0.512
0.28	0.520
0.30	0.518
0.34	0.511
0.36	0.500
0.42	0.461
0.48	0.433
0.54	0.435
0.60	0.394
0.70	0.393
0.80	0.351
0.90	0.316
1.0	0.275
1.2	0.213
1.4	0.169
1.6	0.144
1.8	0.126
2.0	0.108
2.5	0.083
3.0	0.067
3.5	0.053
4.0	0.046
5.0	0.037
6.0	0.031
7.0	0.026
8.0	0.023
9.0	0.020
10.0	0.018

INTRODUCTION

Site-specific seismic ground motion response analyses have been conducted to develop ground surface spectral acceleration (Sa) estimates for both sides the I-40 replacement bridge over the White River in Prairie County, Arkansas (AHTD Job BB0610). All site-specific ground motion response analyses were performed in accordance with Section 3.4.3.2 in the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition. Furthermore, this work has closely followed procedures that were developed for Mack Blackwell Rural Transportation Center (MBTC) project number MBTC 3032, "Site-Specific Seismic Ground Motion Analyses for Transportation Infrastructure in the New Madrid Seismic Zone". The report from that project, written by Cox et al. (2011), is available online at:

http://ww2.mackblackwell.org/web/research/ALL_RESEARCH_PROJECTS/3000s/3032/MBTC-3032FinalReport.pdf

The MBTC 3032 report includes thorough descriptions of the entire site-specific ground motion response analysis process. Herein, we primarily document critical details specific to understanding the analyses conducted for the White River bridge site, and refer the reader to MBTC 3032 for additional, more detailed information.

The approximate latitude and longitude coordinates for the proposed bridge site are provided in Table 1. An aerial photograph of the bridge site is provided in Figure 1.

SHEAR WAVE VELOCITY PROFILES

A combination of active-source and ambient-wavefield surface wave data were acquired on both the West and East sides of the river in order to develop deep shear wave velocity (Vs) profiles for use in the site response analyses. Active-source testing was conducted using the multi-channel analysis of surface waves method (MASW) with both a sledgehammer impact source and a vibroseis truck. Ambient-wavefield surface waves were recorded using 2-dimensional (2D) arrays of sensors; a technique referred to as microtremor array measurements (MAM). The locations of the MASW and MAM arrays are shown in Figure 1.

The following data was collected at the bridge site during field testing: (a) active-source (sledgehammer) stress wave records from 48, 4.5-Hz vertical geophones spaced at 2 m and collected using source offsets of 5, 10, 20 and 30 m, (b) active-source (vibroseis) records from 24, 4.5-Hz vertical geophones spaced at 2 m and collected using source offsets of 5, 10, 20 and 30 m, and (c) ambient-wavefield records from 10 broadband, 3-component seismometers placed in circular arrays with diameters of approximately 60, 200 and 400 m. Note that only one MAM array, roughly 200 m in diameter, was used on the West side of the river due to the difficulty of finding locations for receivers in the flooded timber.

Active-source sledgehammer and vibroseis data were analyzed using the frequency domain beamformer (FDBF) method (Zywicki 1999) coupled with the multiple source-offset technique for identifying near-field contamination and quantifying dispersion uncertainty. The ambient-wavefield MAM data were analyzed using both the 2D high resolution frequency-

Table 1. Site Coordinates for the I-40 replacement bridge over the White River (AHTD Job BB0610).

Latitude	34°50’22.62”	34.83967°
Longitude	-91°27’25.92”	-91.4572°

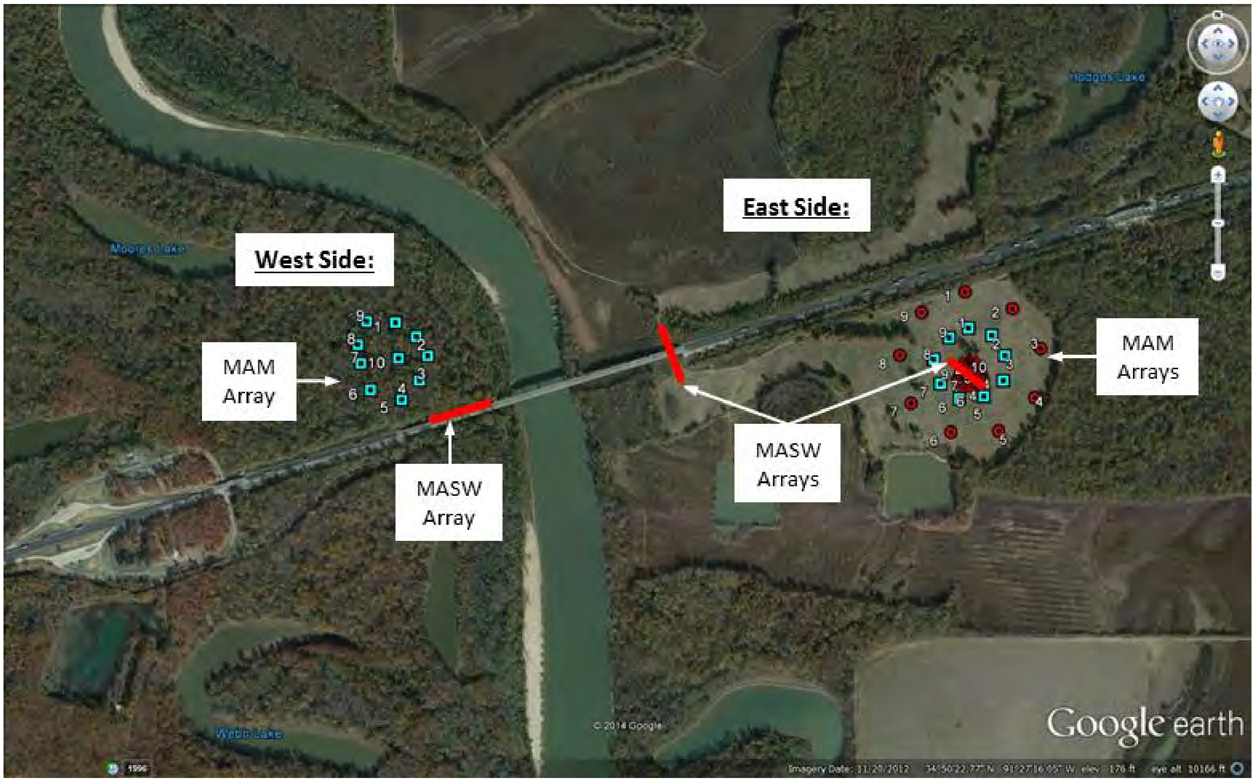

Figure 1. Current I-40 bridge over the White River (location 34.83967° N 91.4572° W). Also shown are the locations of the sensor arrays used for active-source multi-channel analysis of surface waves (MASW) testing and ambient-wavefield microtremor array measurements (MAM) on the West and East sides of the river.

wavenumber (HRFK) method and the Modified Spatial Autocorrelation (MSPAC) method programed in the Geopsy software package (http://www.geopsy.org). More detailed information about the procedures used to develop dispersion estimates from the active and ambient surface wave data may be found in Cox and Wood (2011) and Wood et al. (2014).

The MASW and MAM Rayleigh wave phase velocity dispersion data derived for the West and East sides of the river are compared in Figures 2 and 3, respectively. The agreement between the MASW and MAM data is excellent, particularly considering that the MASW and MAM arrays were not always co-located (refer to Figure 1). Furthermore, dispersion estimates were developed from approximately 1 – 20 Hz on both sides of the river, despite only having one MAM array on the West side. The dispersion data shown in Figures 2 and 3 were averaged in areas of overlap and resampled at a coarser frequency spacing in order to prepare the data for inversion. The resampled data is shown in Figures 4 and 5 for the West and East sides of the river, respectively.

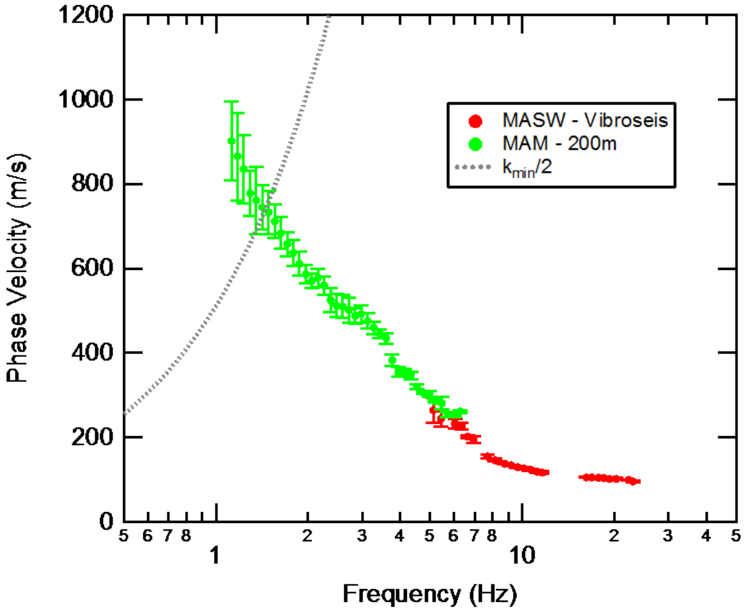

Figure 2. Comparison of MASW and MAM experimental Rayleigh wave phase velocity dispersion data on the West side of the I-40 White River Bridge.

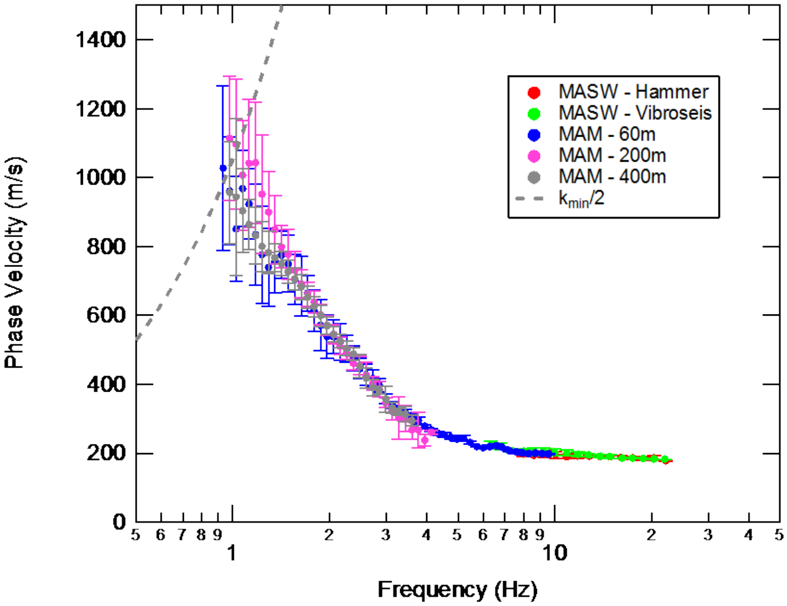

Figure 3. Comparison of MASW and MAM experimental Rayleigh wave phase velocity dispersion data on the East side of the I-40 White River Bridge.

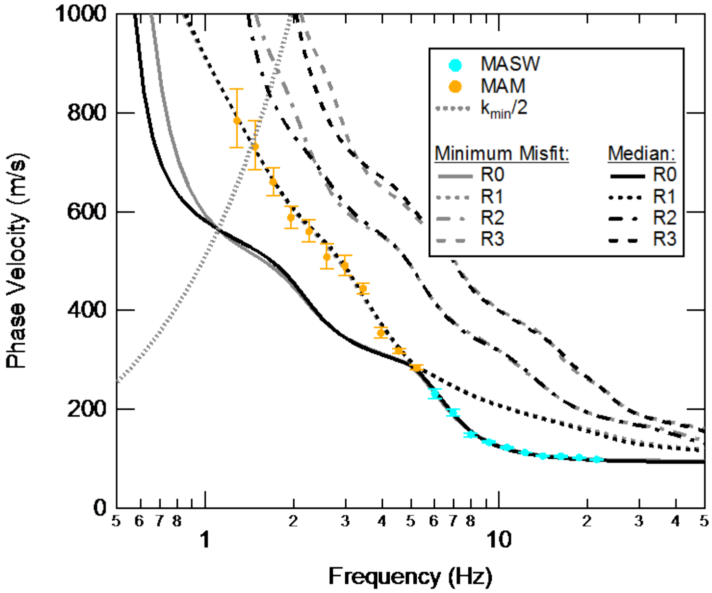

Figure 4. Combined and resampled experimental Rayleigh wave phase velocity dispersion data on the West side of the I-40 White River Bridge. Also shown are the theoretical dispersion curves for the minimum misfit and median shear wave velocity (Vs) profile resulting from the multi-modal surface wave inversion. Note that R0, R1, R2 and R3 correspond to the fundamental, 1st-higher, 2nd-higher and 3rd-higher theoretical modes of Rayleigh wave propagation for the inverted Vs profiles.

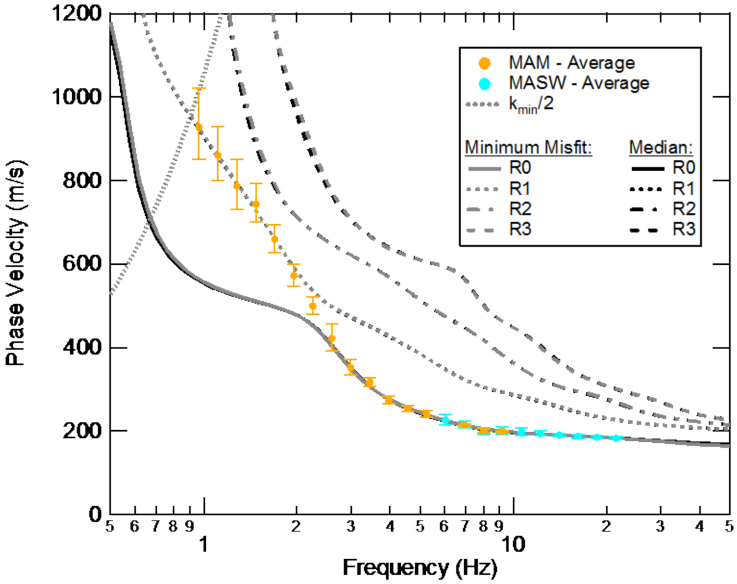

Figure 5. Combined and resampled experimental Rayleigh wave phase velocity dispersion data on the East side of the I-40 White River Bridge. Also shown are the theoretical dispersion curves for the minimum misfit and median shear wave velocity (Vs) profile resulting from the multi-modal surface wave inversion. Note that R0, R1, R2 and R3 correspond to the fundamental, 1st-higher, 2nd-higher and 3rd-higher theoretical modes of Rayleigh wave propagation for the inverted Vs profiles.

The dispersion data shown in Figures 4 and 5 were used to perform a multi-modal inversion in order to obtain a median shear wave velocity profile on each side of the river. Once again, the Geopsy software package was used to perform these inversions. During inversion, over 2 million layered soil profiles were considered using a neighborhood search algorithm. The shear wave velocities for the soil layers were initially estimated as a function of material type/geology and confining pressure, yet given the flexibility to vary within reasonable limits. The saturation depth was set at approximately 2-5 m (6-26 ft) deep (due to the relatively shallow water table near the river), thus constraining the P-wave velocity to 1500 m/s [5000 ft/s] (i.e., effectively fixing Poisson's ratio near 0.5) below this depth; however, at depths where the shear wave velocity exceeded approximately 750 m/s (2500 ft/s), the P-wave velocity constraint was relaxed and Poisson's ratio was allowed to vary between 0.25-0.35. The density of each layer was held constant at 2000 kg/m^3 (125 pcf) regardless of depth or suspected material type. The fundamental mode of Rayleigh wave propagation (R0) was considered, along with the first (R1), second (R2) and third (R3) higher modes. A misfit function between the experimental dispersion data and the theoretical dispersion data resulting from each layered soil profile was used to obtain the shear wave velocity profiles with the minimum misfit (i.e., best fit). The theoretical dispersion curves resulting from the layered profile with the minimum misfit are superimposed on the experimental data in Figures 4 and 5 for the West and East sides of the river, respectively. On the West side of the river, the experimental data was found to transition from R0 to R1 at approximately 5 Hz. On the East side of the river, the experimental data was found to transition from R0 to R1 at approximately 2 Hz. A very good fit to the experimental data was obtained in both cases.

The shear wave velocity profiles resulting from inversion of the dispersion data collected on the West and East sides of the river are shown in Figures 6 and 7, respectively. In both figures, the plot on the left displays the Vs profiles over the top 500 m (1600 ft), while the plot on the right displays the Vs profiles over the top 50 m (160 ft). The 1000 "best", 100 "best" and "best"/minimum misfit Vs profiles are all indicated, along with the median Vs profile of the 1000 "best" profiles. The median Vs profiles on both sides of the river were used as the basis for determining seismic site classification and as the starting profiles for incorporating Vs uncertainty into the site response analyses. The median Vs profile on the West side of the river (refer to Figure 6) resulted in a Vs30/Vs100 value of 171 m/s (560 ft/sec), yielding a Site Class E seismic site classification. The median Vs profile on the East side of the river (refer to Figure 7) resulted in a Vs30/Vs100 value of 207 m/s (680 ft/sec), yielding a Site Class D seismic site classification.

The median Vs profiles were used as the starting profiles for incorporating Vs uncertainty into the site response analyses. Ultimately, three different Vs profiles were used for the site response calculations on each side of the bridge. The three Vs profiles used for the West side of the river are shown in Figure 8, while the three Vs profiles used for the East side of the river are shown in Figure 9. These profiles were developed as follows: (1) The median Vs profiles were decreased and increased by 15% in order to obtain the lower- and upper-bound profiles, respectively; (2) The depth to "engineering bedrock" (i.e., the depth where Vs > 760 m/s, corresponding to Site Class B rock) was estimated between 350 – 450 m (1150-1500 ft) based on the "best" 1000 Vs profiles in Figures 6 and 7. This range corresponds well with estimates of the

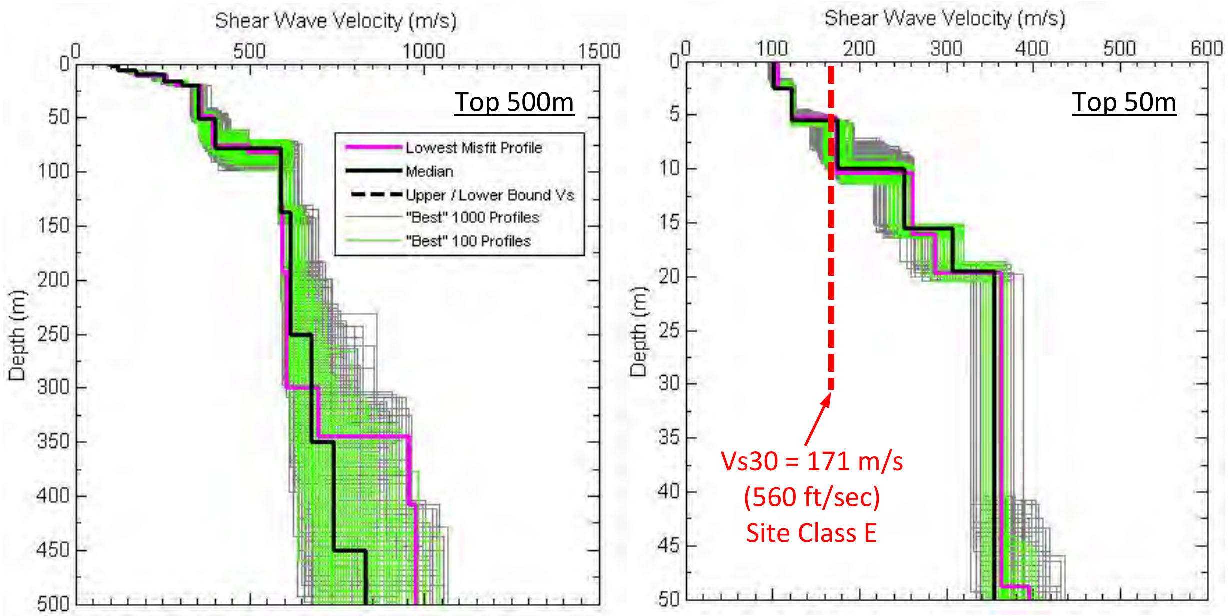

Figure 6. Shear wave velocity (Vs) profiles for the West side of the I-40 White River Bridge. The "best" 1000, "best" 100, and lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 171 m/s (560 ft/sec), resulting in a Site Class E seismic site classification.

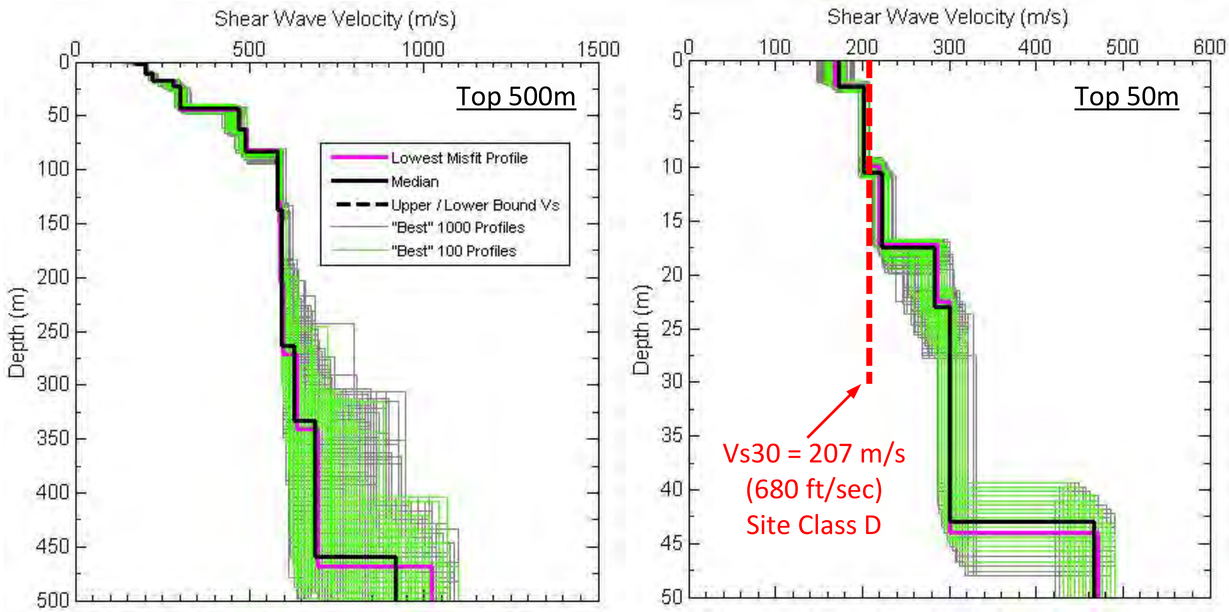

Figure 7. Shear wave velocity (Vs) profiles for the East side of the I-40 White River Bridge. The "best" 1000, "best" 100, and lowest misfit Vs profiles resulting from the surface wave inversion are all shown. Also shown is the median Vs profile derived from the "best" 1000 profiles, which has a Vs30 value of 207 m/s (680 ft/sec), resulting in a Site Class D seismic site classification.

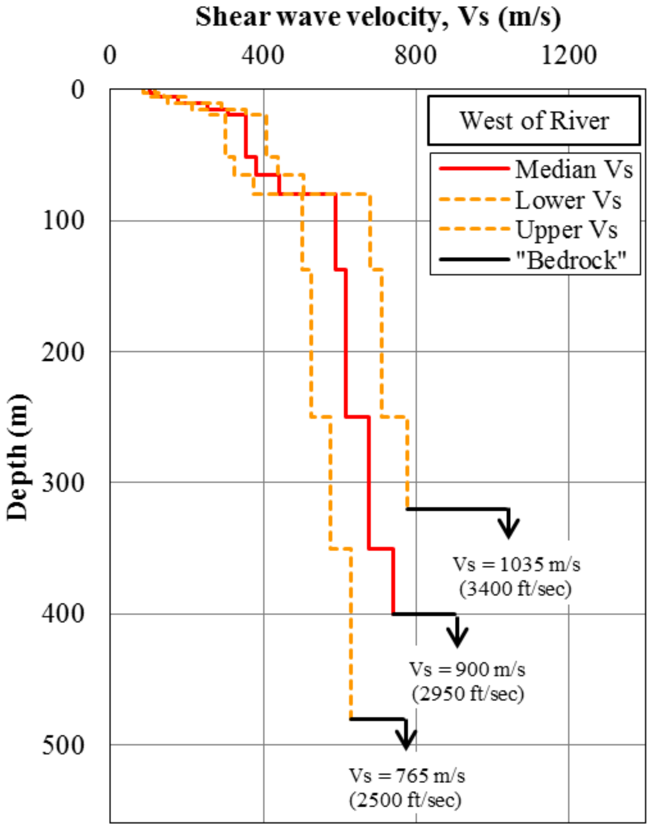

Figure 8. Median, upper- and lower-bound shear wave velocity (Vs) profiles used in the site response analyses conducted for the West side of the I-40 White River Bridge. The arrow locations indicate Vs values and depths assigned to "engineering bedrock", where ground motions were input for site response calculations.

depth to the top of the Mesozoic unit at this location (personal communication from Scott Ausbrooks of AGC and Carson Sloan of AHTD). Therefore, the depth to "engineering bedrock" was set at 400 m (1300 ft) for the median Vs profiles on each side of the river; (3) The depth to "engineering bedrock" was then increased and decreased by 20% in order to account for uncertainty in this depth, with the greater depth to bedrock being associated with the lower-bound Vs profile and the lesser depth to bedrock being associated with the upper-bound Vs profile; (4) An average Vs value of 900 m/s (2950 ft/s) was assigned to the "engineering bedrock", also based on the range of the "best" 1000 profiles on each side of the river. This Vs value was then decreased and increased by 15% in order to account for uncertainty, with the lesser bedrock Vs value associated with the lower-bound Vs profile and the greater bedrock Vs value associated with the upper-bound Vs profile. The depths to bedrock and their corresponding Vs values are indicated in Figures 8 and 9. These depths represent the locations where ground motions were input into the soil profiles for site response analyses.

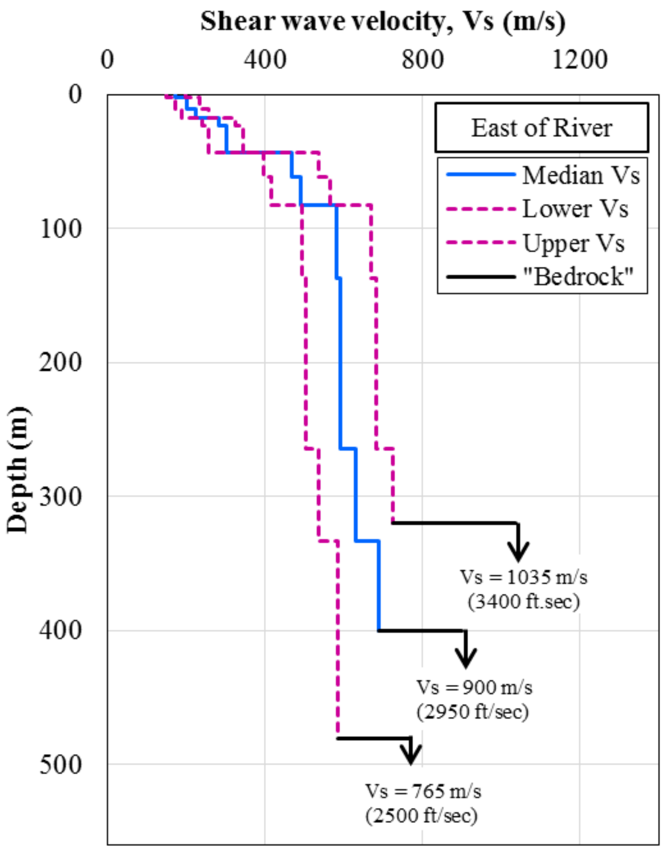

Figure 9. Median, upper- and lower-bound shear wave velocity (Vs) profiles used in the site response analyses conducted for the East side of the I-40 White River Bridge. The arrow locations indicate Vs values and depths assigned to "engineering bedrock", where ground motions were input for site response calculations.

DYNAMIC SOIL PROPERTIES

Dynamic soil properties (i.e., normalized shear modulus reduction [G/G_{max}] and material damping [D]) were developed for each soil layer using the empirical relationships published by Darendeli (2001). Three different sets of dynamic soil properties were utilized in order to account for uncertainty. The standard (i.e., "mean") G/G_{max} and D relationships of Darendeli (2001) were assigned to the median Vs profile, while the plus and minus one standard deviation relationships were assigned to the upper- and lower-bound Vs profiles, respectively. This same convention was followed in MBTC 3032 and the reader may consult that report for additional information.

The dynamic soil property relationships published by Darendeli (2001) are primarily influenced by mean effective confining pressure, soil plasticity index (PI), and overconsolidation ratio (OCR). A simplified soil profile was developed for each side of the bridge to aid in assigning these values to each Vs layer down to bedrock. The near-surface simplified soil profile was primarily derived from boring logs provided by Grubbs, Hoskyn, Barton & Wyatt, Inc., while deeper estimates were made based on geology. If limited information was available, the soil PI was assumed equal to zero and the OCR was assumed equal to one. Reasonable values of soil unit weight were used to estimate effective confining pressures. Figure 10 shows some of the standard G/G_{max} and D curves used for the East side of the river in conjunction with the median Vs profile. These curves were extracted from every 10th layer in the soil profile. In general, the G/G_{max} curves become more linear as the effective confining pressure increases. Thus, the upper-bound G/G_{max} curves in the figure represent soils at greater depths. Conversely, the lower-bound D curves represent soils at greater depths. These curves are primarily shown to provide a visualization that illustrates the range of dynamic soil properties used in the site response analyses with the median Vs profiles. A set of "softer" curves (i.e., minus one standard deviation G/G_{max} curves paired with plus one standard deviation D curves) were used with the lower-bound Vs profiles and a set of "stiffer" curves (i.e., plus one standard deviation G/G_{max} curves paired with minus one standard deviation D curves) were used with the upper-bound Vs profiles.

Implied shear strength corrections, as described in MBTC 3032, were conducted to adjust the G/G_{max} curves on both the West and East sides of the river to depths near 10 m (30 ft). The approximate layer boundaries and soil types for each of these layers are provided in Table 2, along with the target undrained shear strength (Su; for clay layers) or Mohr-Coulomb effective friction angle (ϕ'; for sand layers). These estimates are based on boring logs, discussions with Grubbs, Hoskyn, Barton & Wyatt, Inc., and engineering judgment. Target shear strengths were matched at approximately 3% shear strain.

SITE-SPECIFIC TARGET SPECTRUM

For site-specific ground motion response analyses, a target spectrum (a target for the selection and scaling of input ground motions) was developed to characterize the seismic hazard at the bottom of the soil profile. Herein, the 5% in 50-year uniform hazard spectrum (UHS) serves as that target. The return period for a 5% in 50-year hazard is 1,000 years, which is almost exactly equivalent to the return period for the 7% in 75-year hazard level specified by AASHTO (2011) guidelines. The Java Ground Motion Parameter Calculator (JGMPC) was used to develop a UHS from the 2002 national seismic hazard maps data at periods of 0 (PGA), 0.1, 0.2, 0.3, 0.5, 1.0, and 2.0 seconds. The target spectrum is illustrated in Figure 11. By default, the JGMPC computes hazards for Site Class B. Although the Vs structure varies between the East and West sides of the river, the "engineering bedrock" on both sides is indeed Site Class B. Thus, a single target spectrum was sufficient to describe the seismic hazard on both sides of the river. [Note that even AASHTO 2011 uses the 2002 data rather than the 2008 data; see notes in Figure 3.4.1-2a relative to discussion and references].

INPUT GROUND MOTION SELECTION

Deaggregation results for the site indicate that the seismic hazard is governed by approximately the same earthquake characteristics at each period: a modal magnitude of 7.7 at a distance of approximately 100 km (62 mi). These characteristics were used to select four input

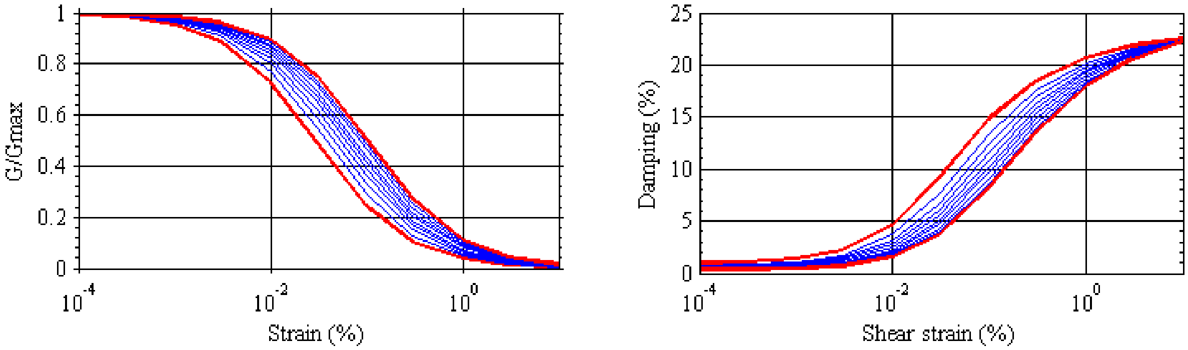

Figure 10. Dynamic soil properties assigned to every 10th layer in the median Vs profile on the East side of the I-40 White River Bridge.

Table 2. Target shear strengths used to perform shear strength corrections for the top three soil layers on both the West and East side of the I-40 White River Bridge.

	Depth	**Assigned Soil Type**	**Target Shear Strength**		
			Lower Vs	**Median Vs**	**Upper Vs**
West	Top 2.5 m (8.2 ft)	Clay	Su = 48 kPa	Su = 57 kPa	Su = 65 kPa
	2.5 - 5.5 m (8.2 - 18 ft)	Sand	φ' = 29°	φ' = 32°	φ' = 35°
	5.5 - 10 m (18 - 32.8 ft)	Dense Sand	φ' = 35°	φ' = 38°	φ' = 40°
East	Top 2.5 m (8.2 ft)	Clay	Su = 57 kPa	Su = 67 kPa	Su = 77 kPa
	2.5 - 10.5 m (8.2 - 34.4 ft)	Sand	φ' = 31°	φ' = 34°	φ' = 37°

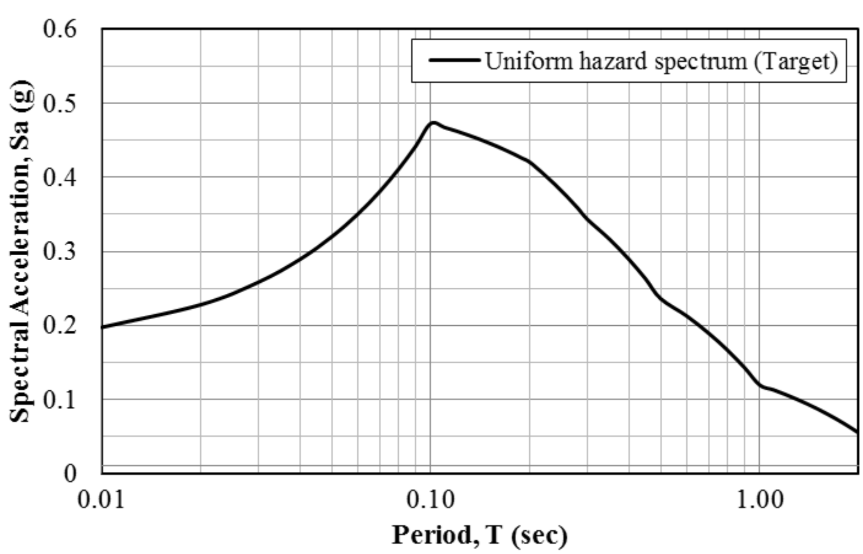

Figure 11. Uniform hazard spectrum (UHS)/target spectrum for the I-40 White River Bridge site developed using the data from the 2002 USGS maps for a 5% in 50-year probability of exceedance (equivalent to 7% in 75-year probability of exceedance).

ground motions from the NUREG database (see MBTC 3032 report for details). Each ground motion was scaled and then spectrally matched to the target spectrum for the White River Bridge site. Figure 12 illustrates the four spectrally matched input ground motions along with the target spectrum. Note that that ground motions match the target spectrum very well at periods greater than 0.1 seconds. The ground motions exceed the target spectrum at shorter periods because the NUREG ground motions were purposely modified to included greater short-period energy as a means to better characterize ground motions expected in the Central and Eastern U.S.

SITE RESPONSE METHODOLOGY

Six distinct sets of site response analyses were conducted for each side of the bridge in an effort to account for uncertainties associated with the input parameters, spatial variability, and methods of analysis. Three types of analyses were conducted to account for uncertainties associated with the soil profile. These include: (1) a softer-bound, defined by the softer set of dynamic soil properties and the lower-bound Vs profile; (2) an average, based on the mean dynamic soil properties and median Vs profile; and (3) a stiffer-bound, defined by the stiffer set of dynamic soil properties and the upper-bound Vs profile. Finally, all of the previous combinations were computed using both equivalent linear (EQL) and non-linear (NL) site response analysis methods. Each of these six distinct analyses were conducted using four scaled and spectrally matched input motions. The result from each type of analysis is described by the lognormal median of the four output (surface) response spectra. This is very similar to the “logic tree” approach that is described in detail in the MTBC 3032 report, except that the West and East sides of the bridge were analyzed separately. For clarity, the weighting factors used for this study are summarized in Figure 13.

RESULTS

All site response analyses were performed using the 1D site response program DEEPSOIL. The log-normal median surface response spectra resulting from six distinct site response analyses were weighted and combined using a logic tree approach in order to determine overall/“fully weighted” surface responses for the West and East sides of the river, respectively.

The output from DEEPSOIL is summarized in Figures 14 and 15 for the West and East sides of the bridge, respectively. The lognormal median response from the four input ground motions is plotted for each of the six distinct site response analyses described above (i.e., three different soil profiles with two types of analyses). The fully weighted response represents a single surface response spectrum obtained by applying the weighting factors from Figure 13 to each of the six lognormal median responses. For both sides of the bridge, ground motion amplification occurs at periods greater than approximately 0.2 sec. These fully weighted results were used to determine spectral ratios between surface ground motions and input ground motions, which were then multiplied by the generic Site Class B acceleration response spectrum (corresponding to the “engineering bedrock”) in order to compute a site-specific response spectrum for each side of the river. The results are plotted along with the generic response

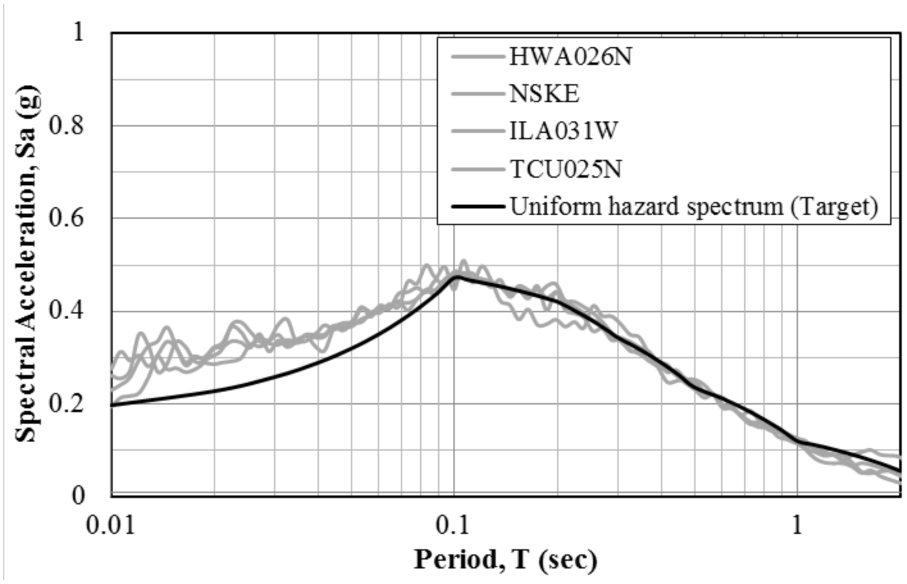

Figure 12. Uniform hazard spectrum (UHS)/target spectrum for the I-40 White River Bridge site with the four scaled and spectrally matched input ground motions selected for use in site response analyses.

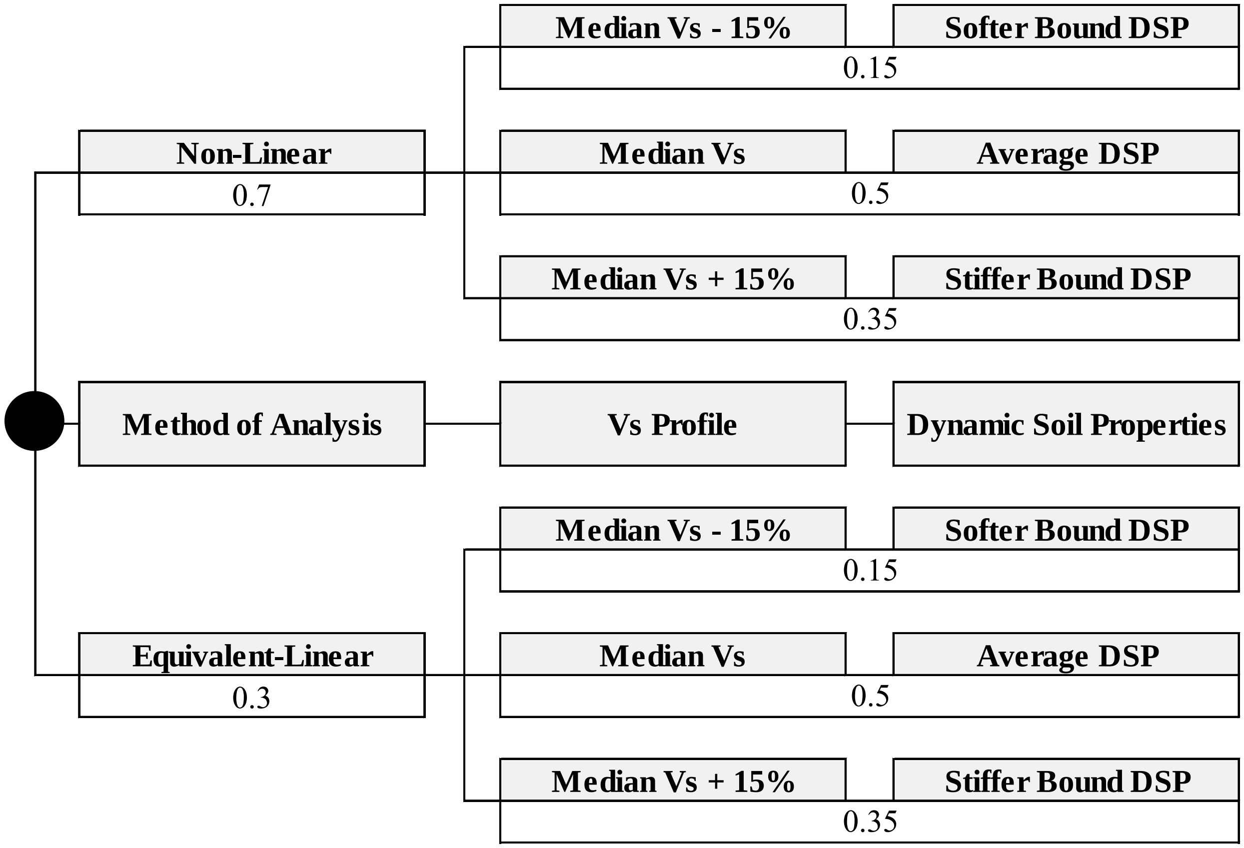

Figure 13. Weighting Factors Used to Combine Site Response Analysis Results that Utilized Different Methods of Analyses, Vs Profiles, and Dynamic Soil Properties (DSP) into a Single, "Fully Weighted" Surface Response Spectrum for Each Side of the Bridge.

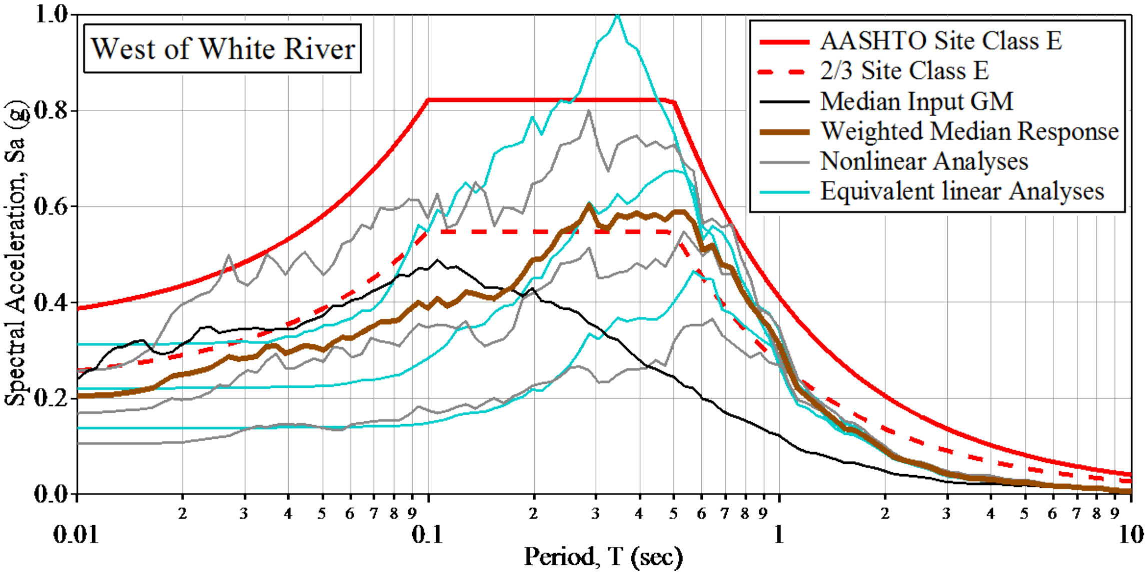

Figure 14. Log-normal median surface response from each distinct set of site response analyses, log-normal median of the input ground motions, generic Site Class E response spectrum, and the fully weighted median response spectra for the West side of the White River bridge over I-40.

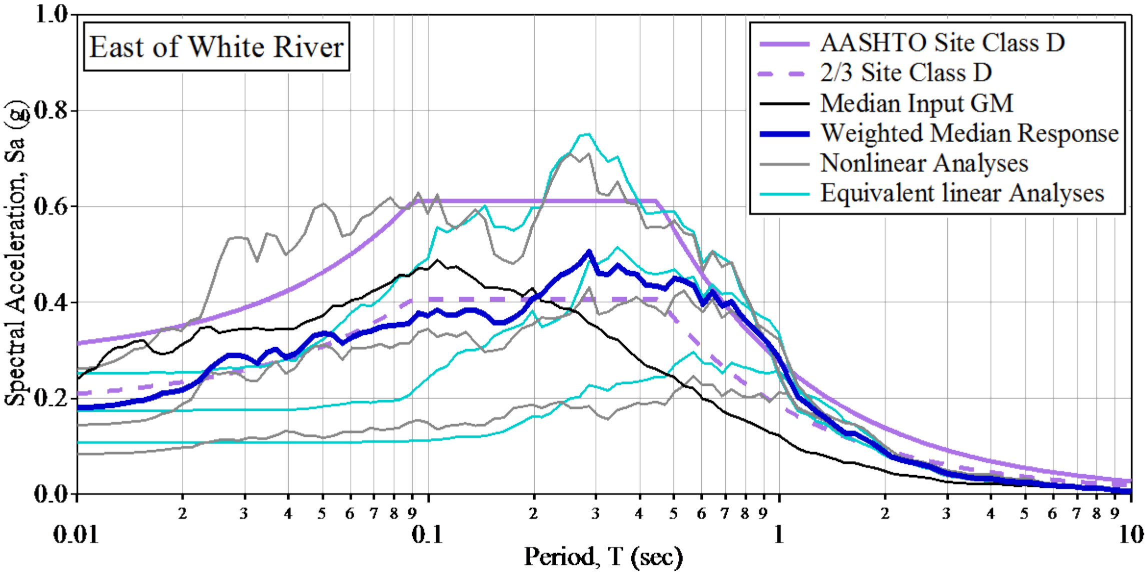

Figure 15. Log-normal median surface response from each distinct set of site response analyses, log-normal median of the input ground motions, generic Site Class D response spectrum, and the fully weighted median response spectra for the East side of the White River bridge over I-40.

spectrum that would be utilized in the absence of site-specific ground motion response analyses. This procedure is described more thoroughly in MBTC-3032.

The final results from the site-specific ground motion response analyses are detailed in Figures 16 and 17 for the West and East side of the river, respectively. As noted above, the West side of the river was found to classify as Site Class E in the vicinity where Vs testing was conducted. Therefore, the Site Class E spectrum based on the general AASHTO (2011) procedure (Section 3.4.1) is shown for reference in Figure 16. Also shown for reference is a spectrum that is equivalent to 2/3 of the Site Class E general spectrum, because according to Section 3.4.3 of AASHTO (2011), unless otherwise approved by the Owner, the response spectrum used in design shall not be lower than 2/3 of the general spectrum even if site-specific results indicate it is possible. The median site-specific response spectrum on the West side of the river is provided in Figure 16. This response spectrum is less than the allowable minimum of 2/3 Site Class E up to a period (T) of just over 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit, while still remaining below the general Site Class E spectrum, between periods of approximately 0.2 – 1.0 seconds, after which it falls slightly below the limit. The delineated site-specific design response spectrum, highlighted in Figure 16, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the West side of the river.

The East side of the river was found to classify as Site Class D in the vicinity where Vs testing was conducted. Therefore, the Site Class D spectrum based on the general AASHTO (2011) procedure is shown for reference in Figure 17 along with a spectrum equivalent to 2/3 of the Site Class D spectrum. The median site-specific response spectrum on the East side of the river is also provided in Figure 17. This response spectrum is generally less than the allowable minimum of 2/3 Site Class D up to a period (T) of approximately 0.2 seconds. The site-specific spectrum exceeds the 2/3 limit between periods of approximately 0.2 – 3.0 seconds, and is essentially equal to the Site Class D general spectrum between periods of 0.7 – 1.0 seconds. The delineated site-specific design response spectrum, highlighted in Figure 17, is defined as the greater of either the median site-specific response spectrum or 2/3 of the general response spectrum. This delineated spectrum should be used for design on the East side of the river.

The site-specific ground motion response analysis results for both the West and East side of the White River are compared, along with their respective delineated design response spectra, in Figure 18. **Tabulated values of Sa, including PGA/As values for use in liquefaction analyses, for each delineated design response spectrum are also provided in Table 3. A corresponding moment magnitude (Mw) of 7.7 should be used in liquefaction analyses**. This is the modal magnitude at the bridge site for every available period in the 2002 USGS deaggregation tool. While the modal magnitude always stays fixed at Mw = 7.7, the mean magnitude consistently shifts upward from approximately Mw = 7.35 for PGA to Mw = 7.57 for the 2-second period data (maximum period available). Basically, this means the modal magnitude and the mean magnitude are expected to grow closer to one another for soft and/or deep soil sites with large natural periods. Since this site has an approximate natural period of 3 seconds, based on the deep Vs profiles shown above, the mean and modal magnitudes are expected to be very close to one another and approximately equal to Mw = 7.7.

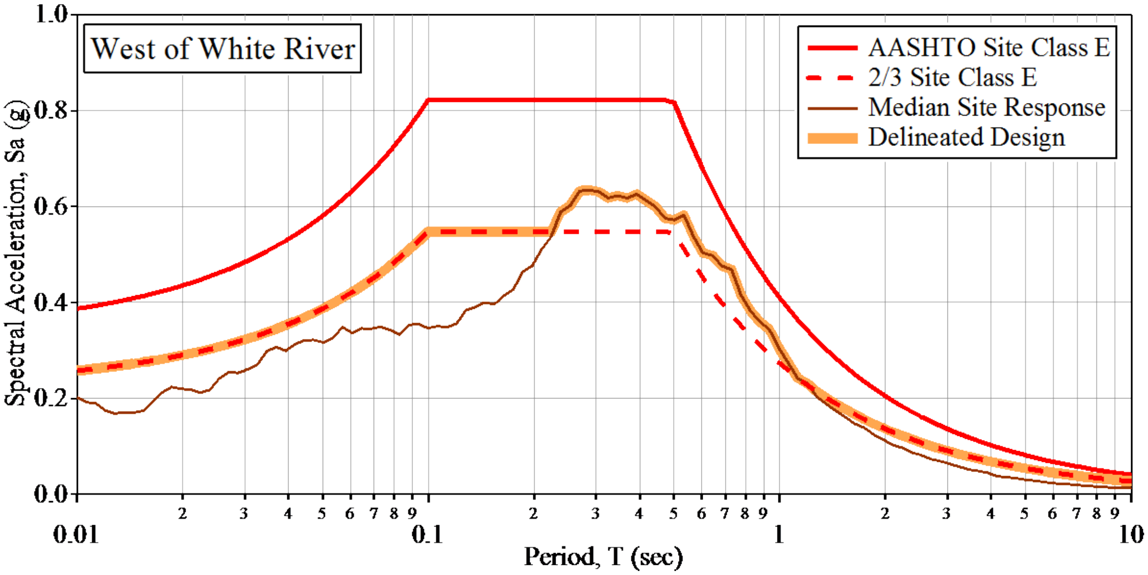

Figure 16. Site-specific ground motion response analysis results for the West side of the White River at I-40.

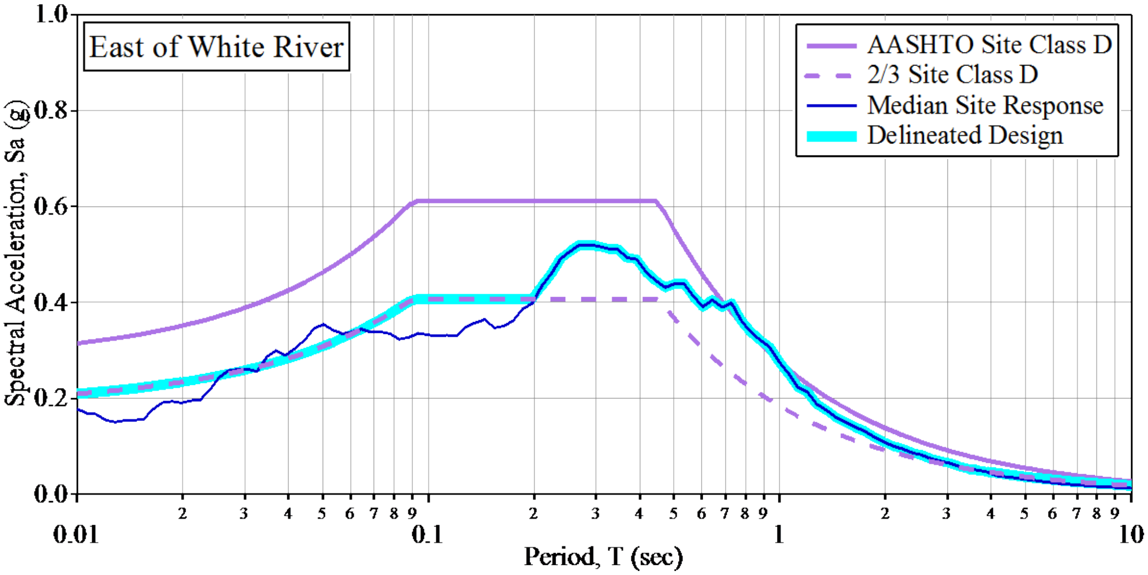

Figure 17. Site-specific ground motion response analysis results for the East side of the White River at I-40.

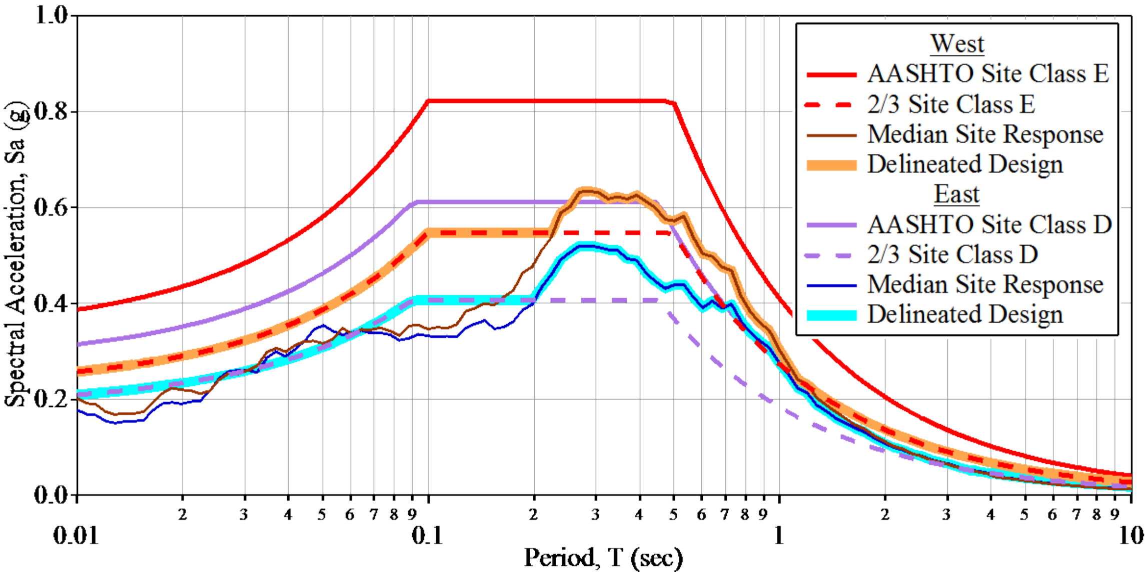

Figure 18. Comparison of site-specific ground motion response analysis results for the <u>West and East side</u> of the White River at I-40.

Table 3. Spectral acceleration (Sa) values for the delineated design response spectra obtained from site-specific ground motion response analyses for the West and East side of the White River at I-40.

West of White River	
Period (sec)	**Delineated Design (g)**
PGA	0.23
0.010	0.258
0.014	0.271
0.018	0.284
0.022	0.297
0.026	0.310
0.030	0.323
0.040	0.355
0.050	0.387
0.060	0.419
0.070	0.452
0.080	0.484
0.090	0.516
0.10	0.547
0.20	0.547
0.22	0.547
0.24	0.590
0.26	0.615
0.28	0.634
0.30	0.632
0.34	0.621
0.36	0.619
0.42	0.611
0.48	0.574
0.54	0.574
0.60	0.508
0.70	0.473
0.80	0.399
0.90	0.353
1.0	0.302
1.2	0.231
1.4	0.195
1.6	0.171
1.8	0.152
2.0	0.137
2.5	0.109
3.0	0.091
3.5	0.078
4.0	0.068
5.0	0.055
6.0	0.045
7.0	0.039
8.0	0.034
9.0	0.030
10.0	0.027

East of White River	
Period (sec)	**Delineated Design (g)**
PGA	0.17
0.010	0.210
0.014	0.220
0.018	0.230
0.022	0.239
0.026	0.249
0.030	0.259
0.040	0.284
0.050	0.309
0.060	0.333
0.070	0.358
0.080	0.383
0.090	0.404
0.10	0.408
0.20	0.413
0.22	0.453
0.24	0.494
0.26	0.512
0.28	0.520
0.30	0.518
0.34	0.511
0.36	0.500
0.42	0.461
0.48	0.433
0.54	0.435
0.60	0.394
0.70	0.393
0.80	0.351
0.90	0.316
1.0	0.275
1.2	0.213
1.4	0.169
1.6	0.144
1.8	0.126
2.0	0.108
2.5	0.083
3.0	0.067
3.5	0.053
4.0	0.046
5.0	0.037
6.0	0.031
7.0	0.026
8.0	0.023
9.0	0.020
10.0	0.018

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Cox, B.R. and Wood, C.M., (2011). “Surface Wave Benchmarking Exercise: Methodologies, Results and Uncertainties,” GeoRisk 2011: Geotechnical Risk Assessment and Management (C.H. Juang et al., eds.), ASCE GSP 224, 845-852.

Cox, B.R., Ellis, T.B., and Griffiths, S.C. (2012) “Site-Specific Ground Motion Analyses for Transportation Infrastructure in the New Madrid Seismic Zone,” Mack-Blackwell Rural Transportation Center (MBTC) project number 3032. Available online at: http://ww2.mackblackwell.org/web/research/ALL_RESEARCH_PROJECTS/3000s/3032/MBTC-3032FinalReport.pdf

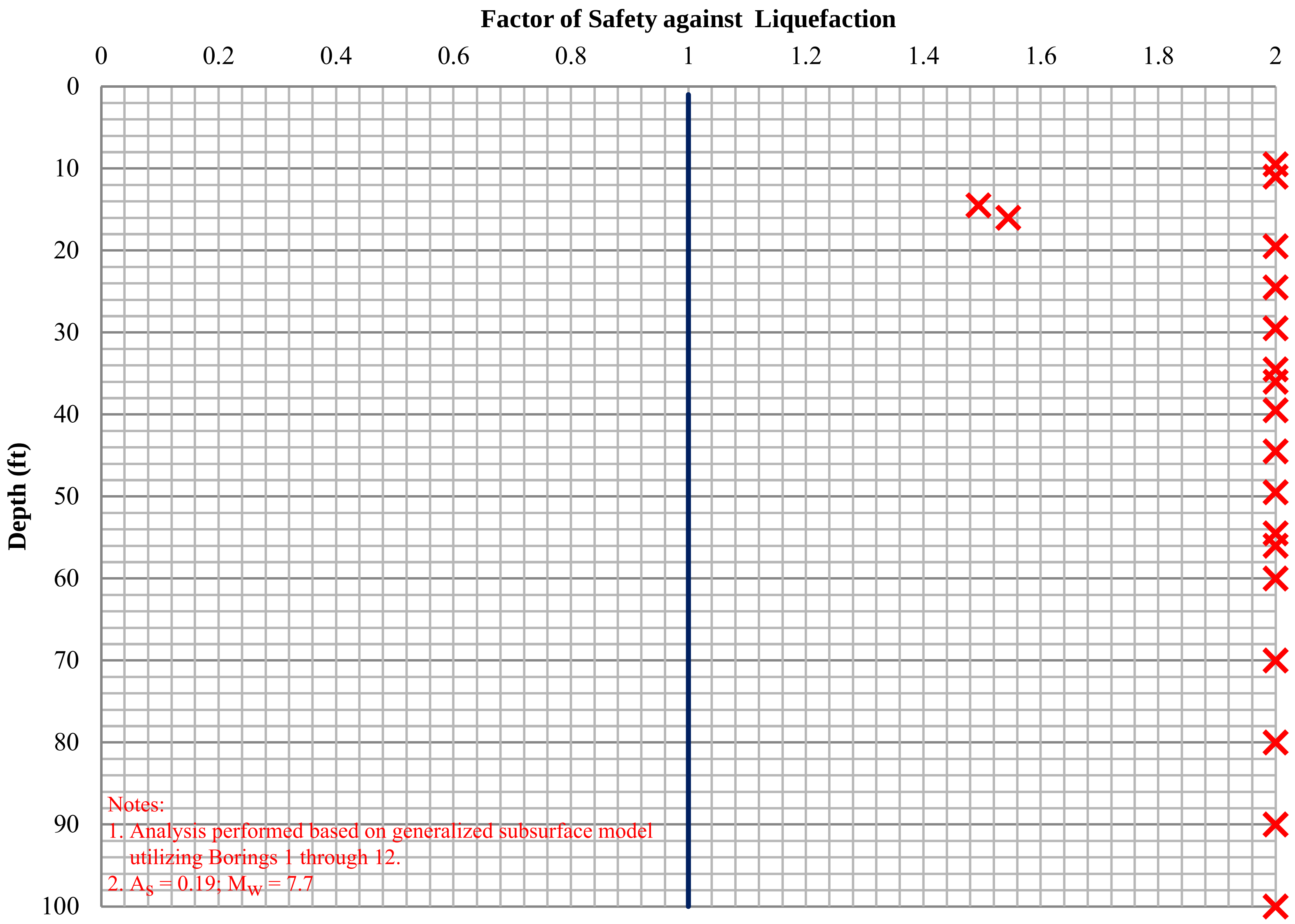
Wood, C.M., Ellis, T.B., Teague, D.P., Cox, B.R. (2014). “Analyst I: Comprehensive Analysis of the UTexas1 Surface Wave Dataset,” ASCE Geo-Congress 2014: Geo-Characterization and Modeling for Sustainability, Atlanta, GA, 23-26 February 2014.

APPENDIX E

Results of Liquefaction Analyses - West Side of River

AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas



Notes:

1. Analysis performed based on generalized subsurface model utilizing Borings 1 through 12.
2. $A_S = 0.19$; $M_W = 7.7$

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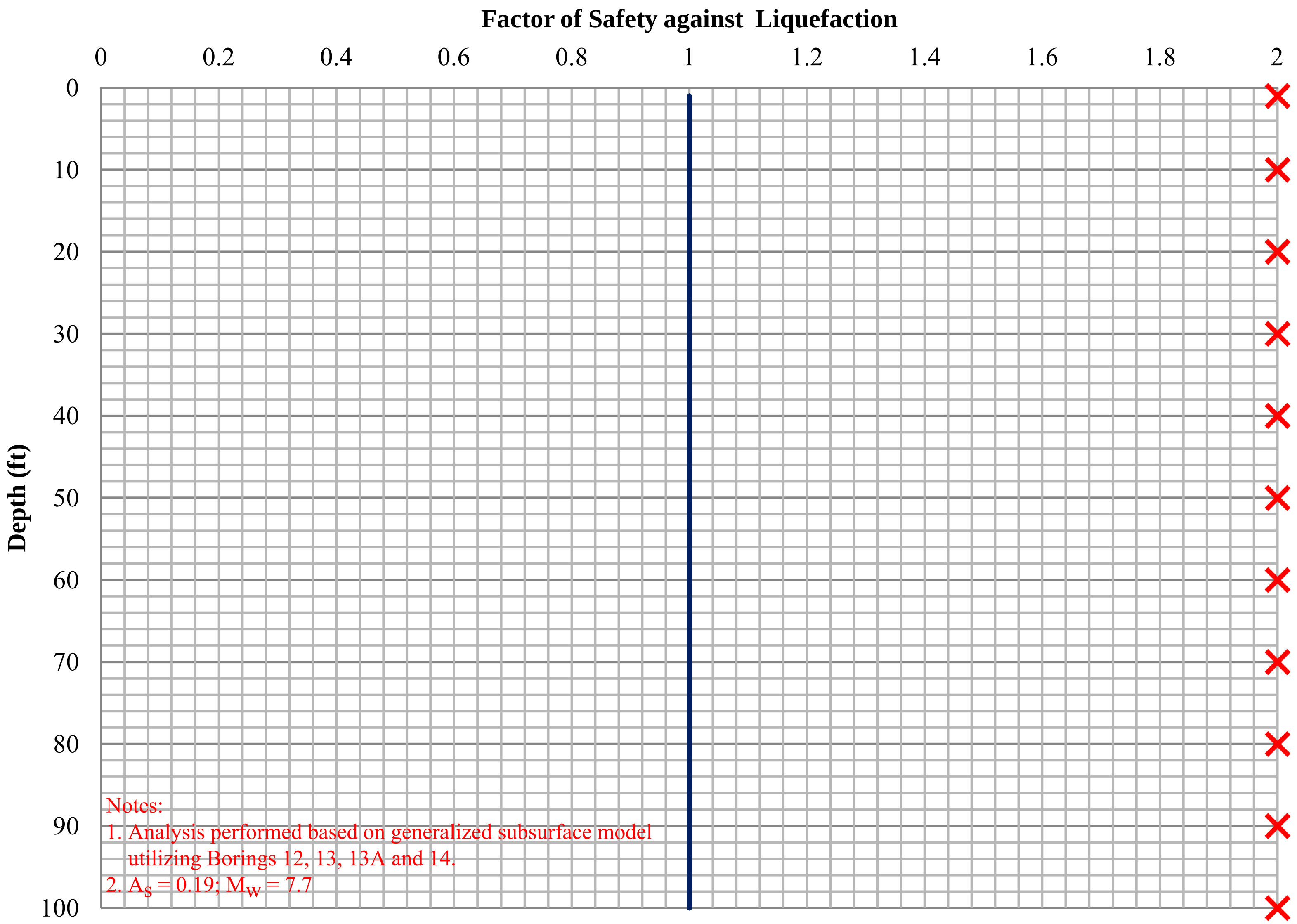
Results of Liquefaction Analyses - River Channel

AHTD Job No. BB0610: White River Str. & Apprs. (F)

Prairie County, Arkansas



Notes:
1. Analysis performed based on generalized subsurface model utilizing Borings 12, 13, 13A and 14.
2. $A_s = 0.19$; $M_W = 7.7$

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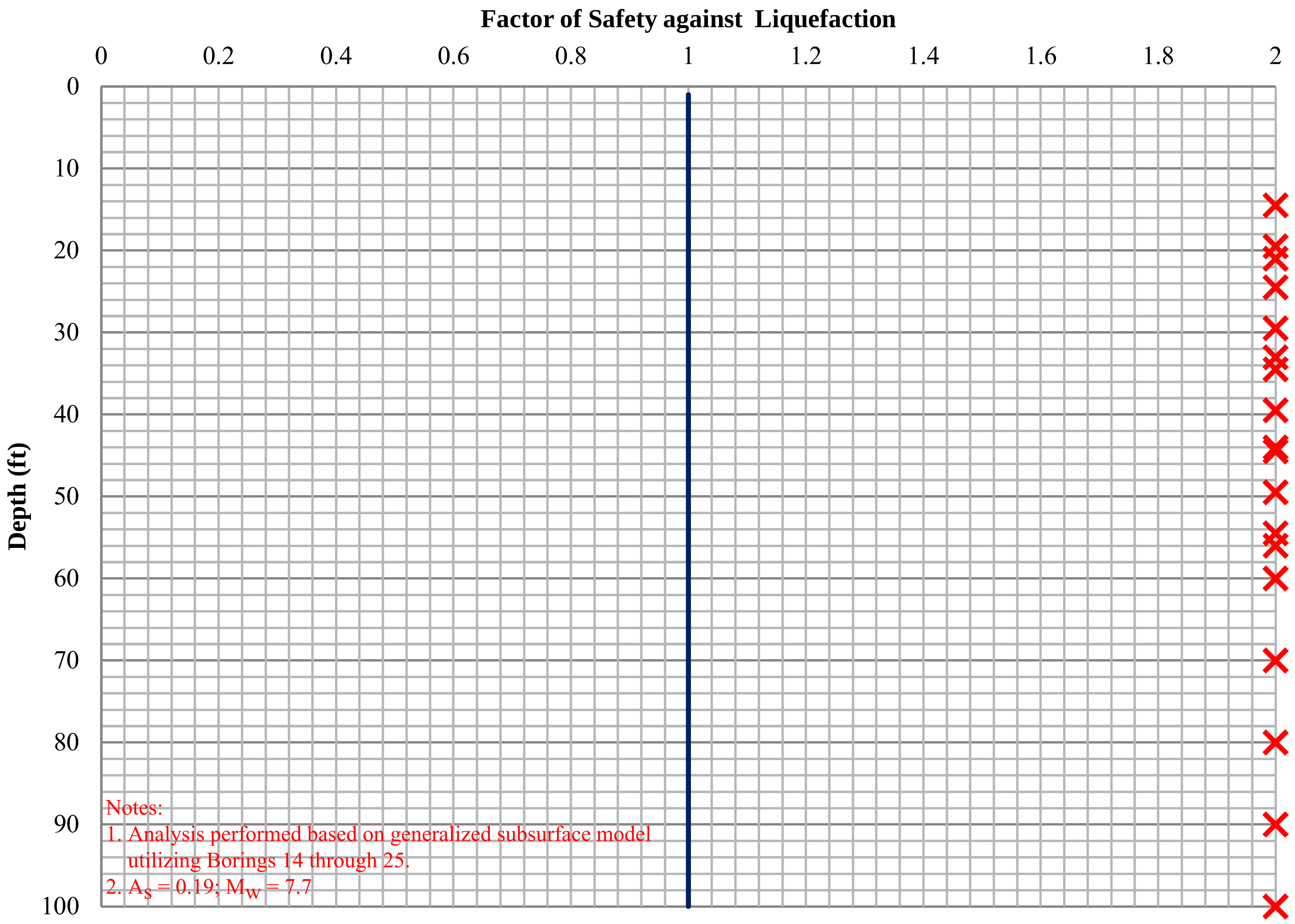
Results of Liquefaction Analyses - East Side of River

AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas



Notes:
1. Analysis performed based on generalized subsurface model utilizing Borings 14 through 25.
2. $A_s = 0.19$; $M_W = 7.7$

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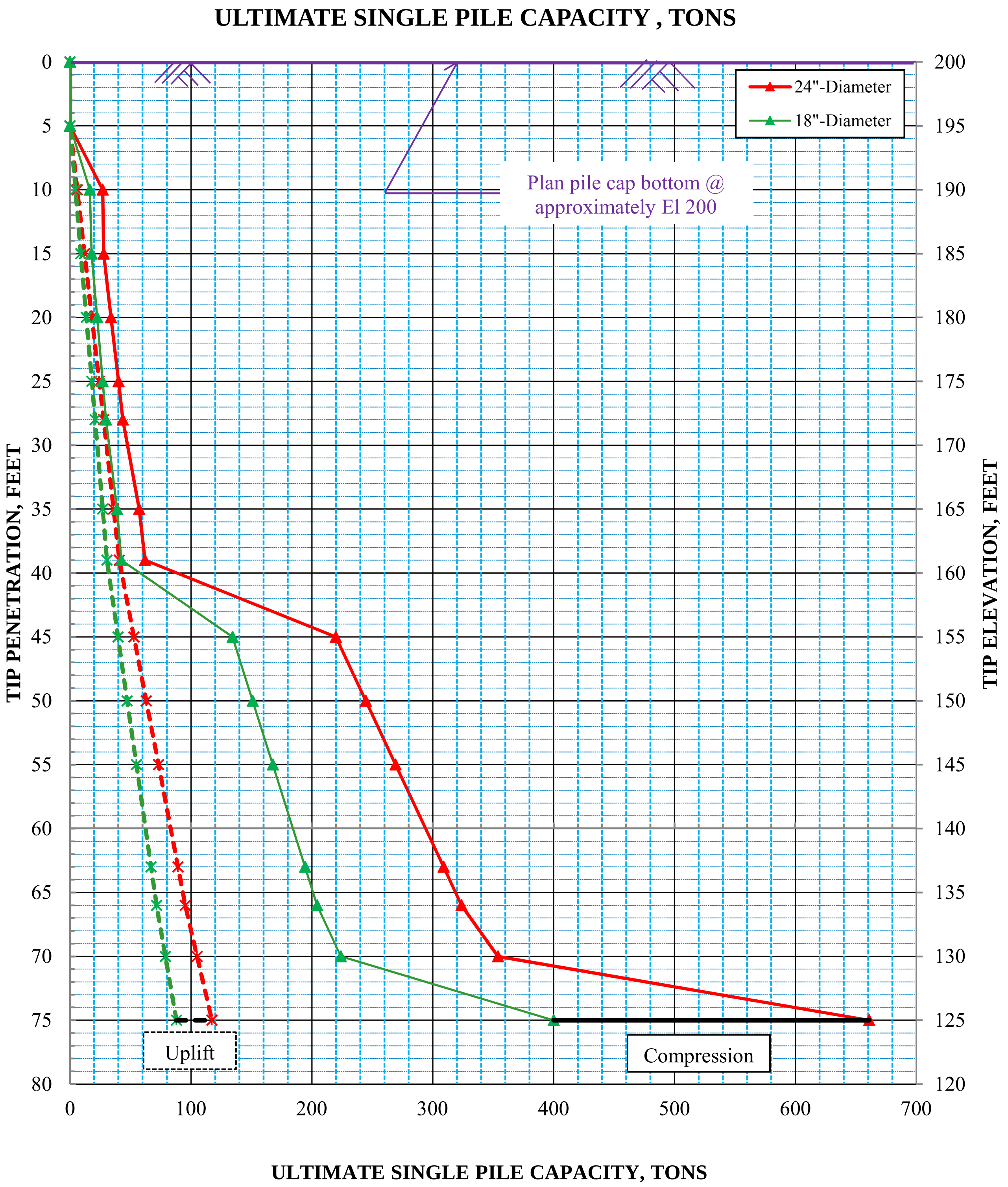
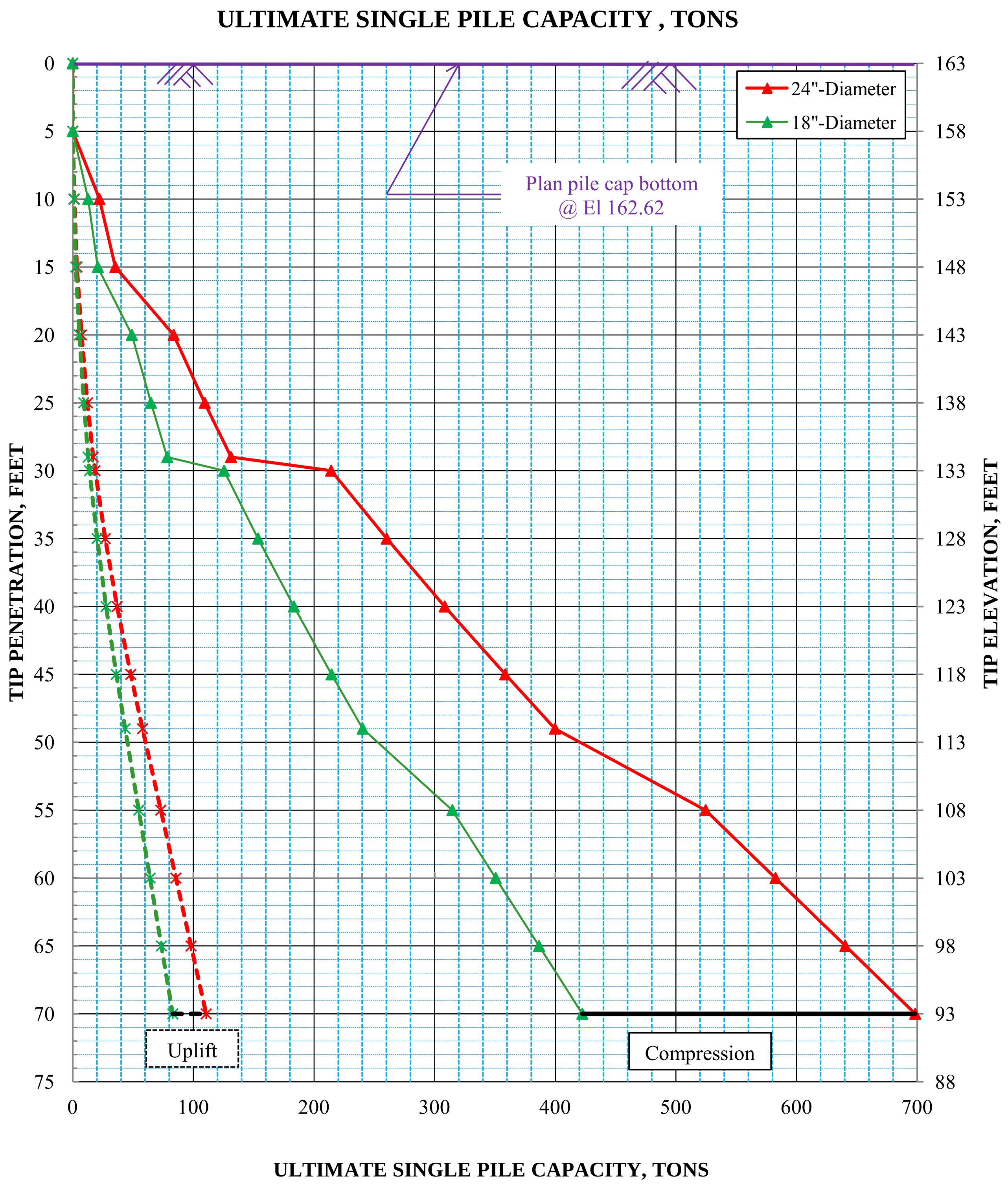
APPENDIX F



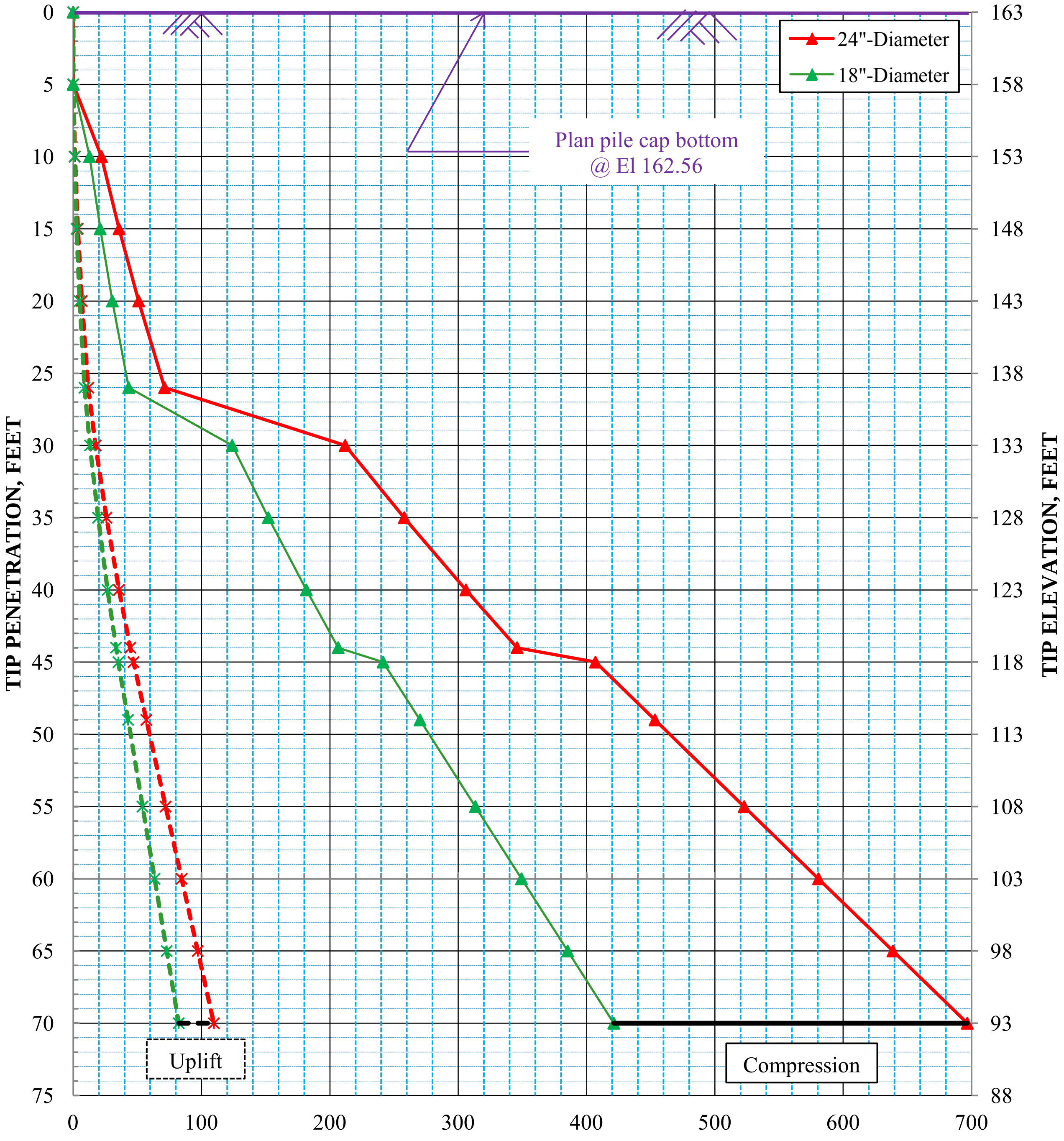
Driven Steel Shells
Bent 1 (West Abutment)
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be driven to plan tip elevation.



Driven Steel Shells
Bent 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

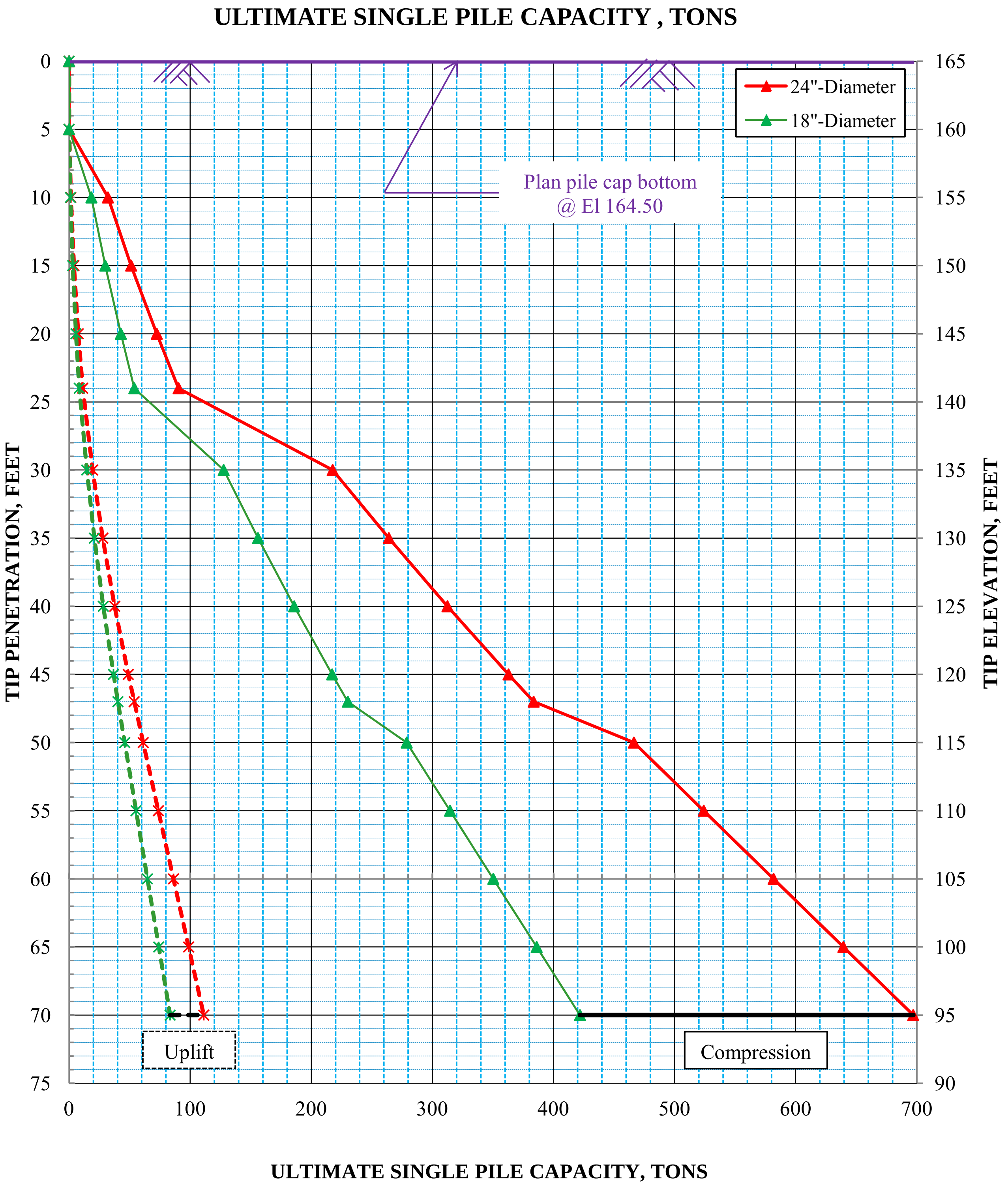


ULTIMATE SINGLE PILE CAPACITY , TONS

ULTIMATE SINGLE PILE CAPACITY, TONS

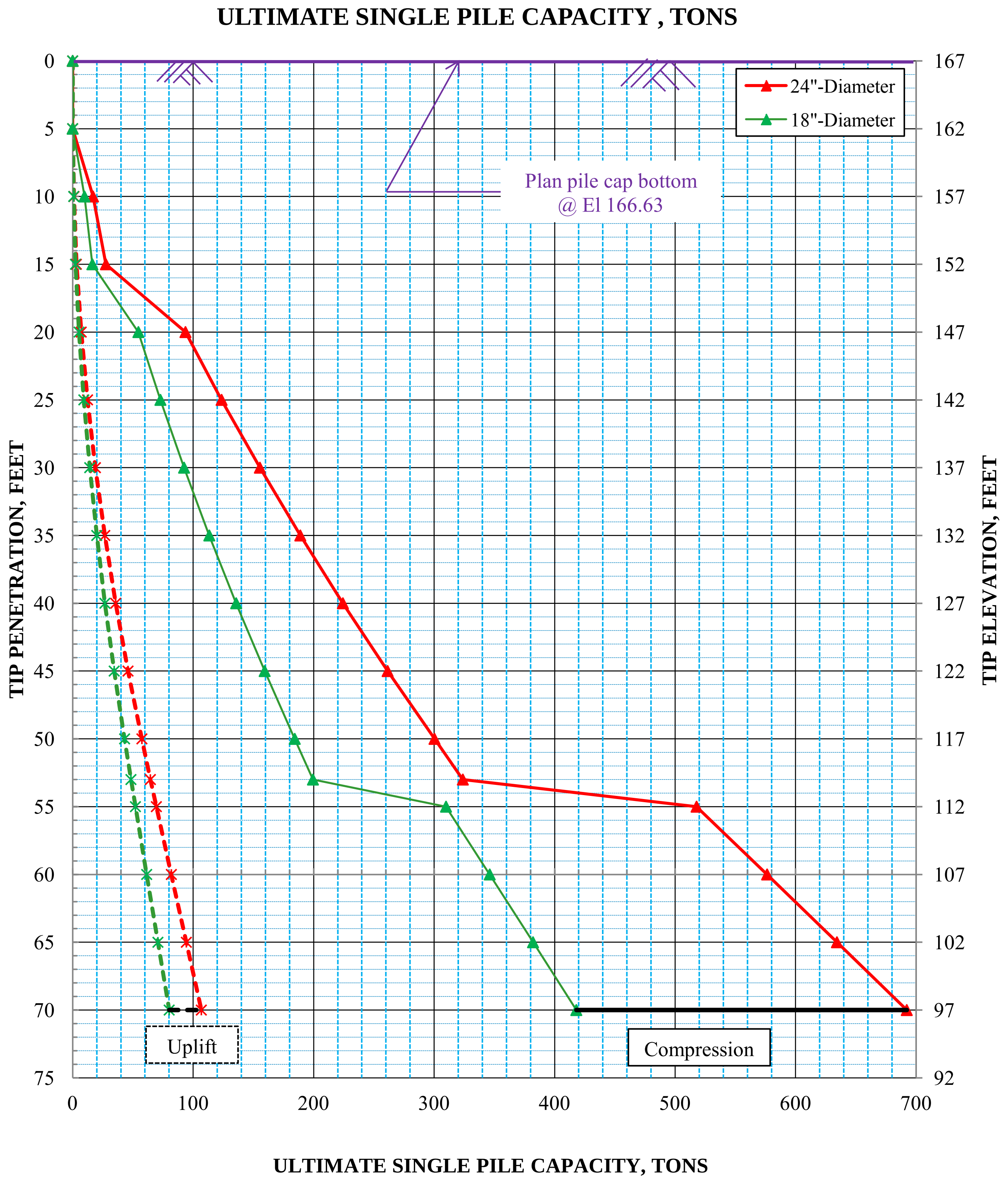
Driven Steel Shells
Bent 3
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



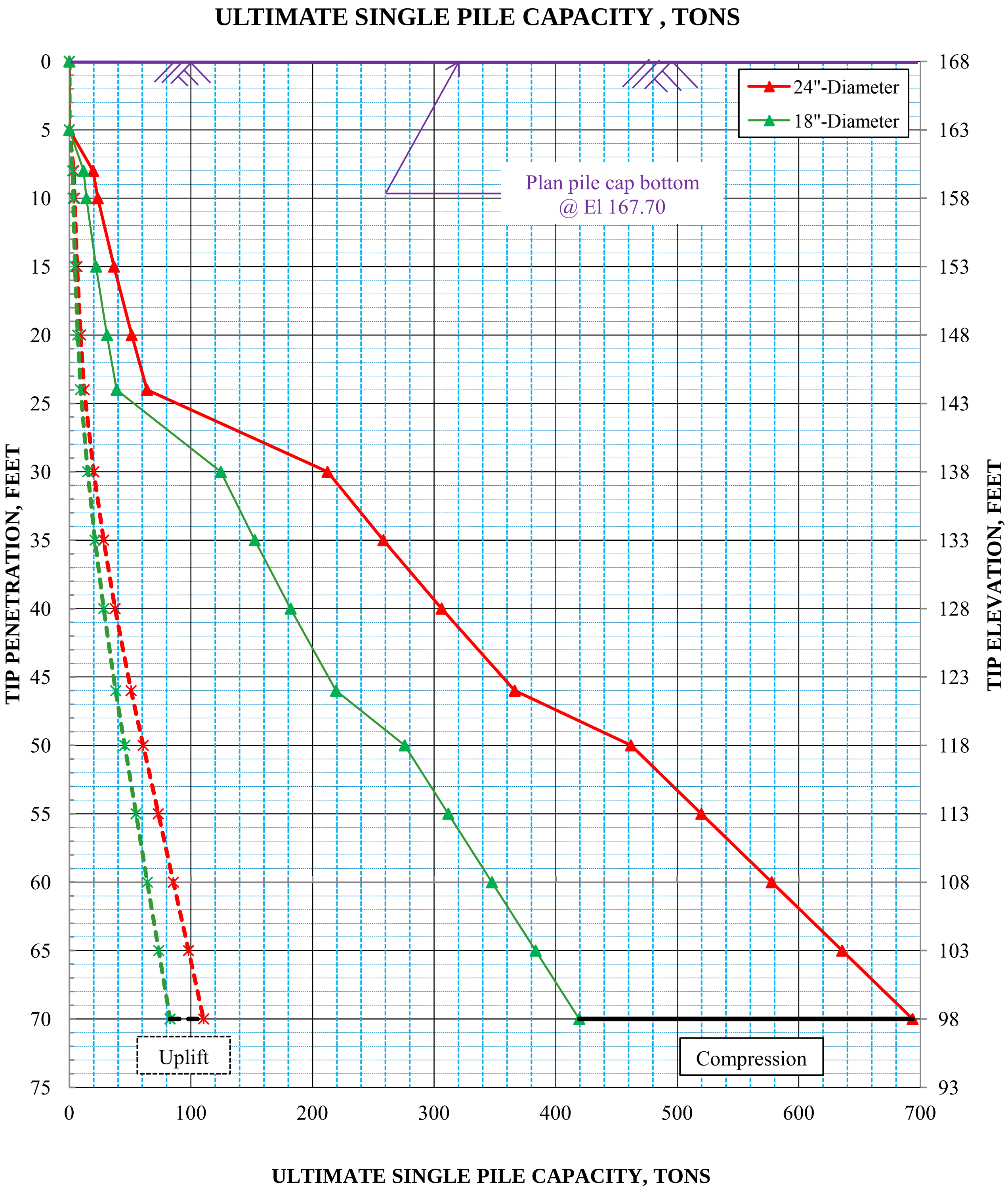
Driven Steel Shells
Bent 4
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 5
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

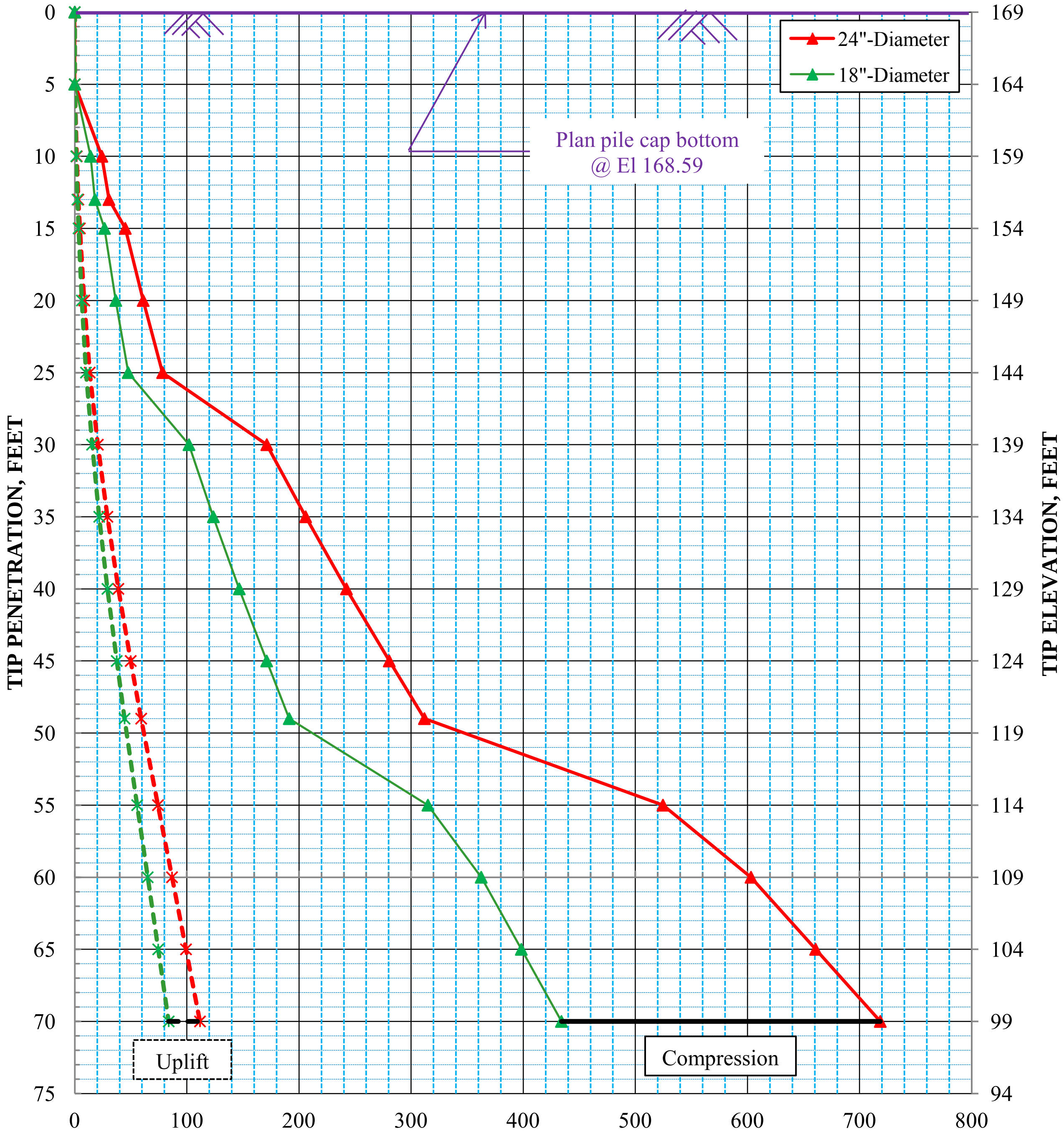
Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 6
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

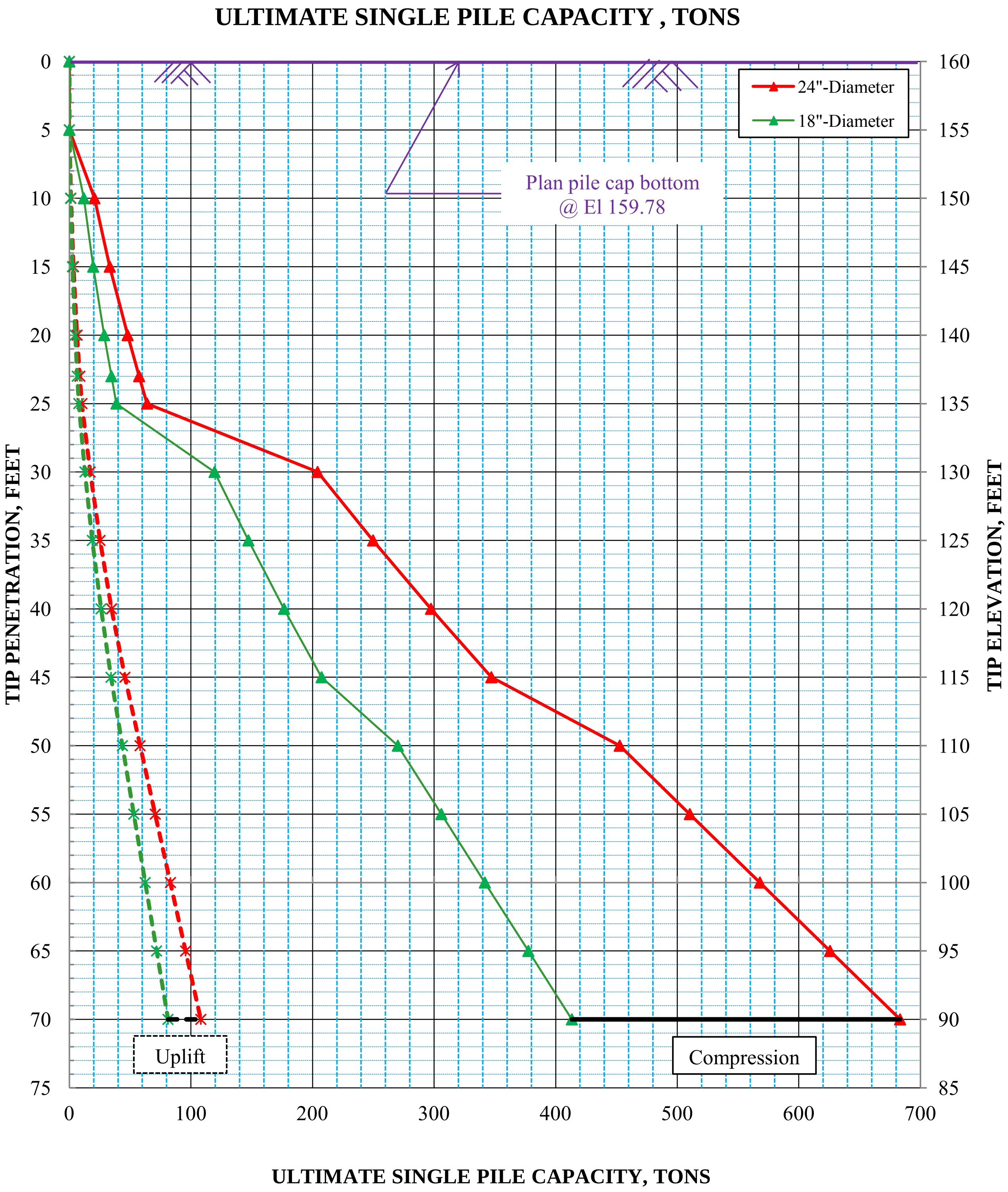
ULTIMATE SINGLE PILE CAPACITY , TONS



ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Bent 7
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

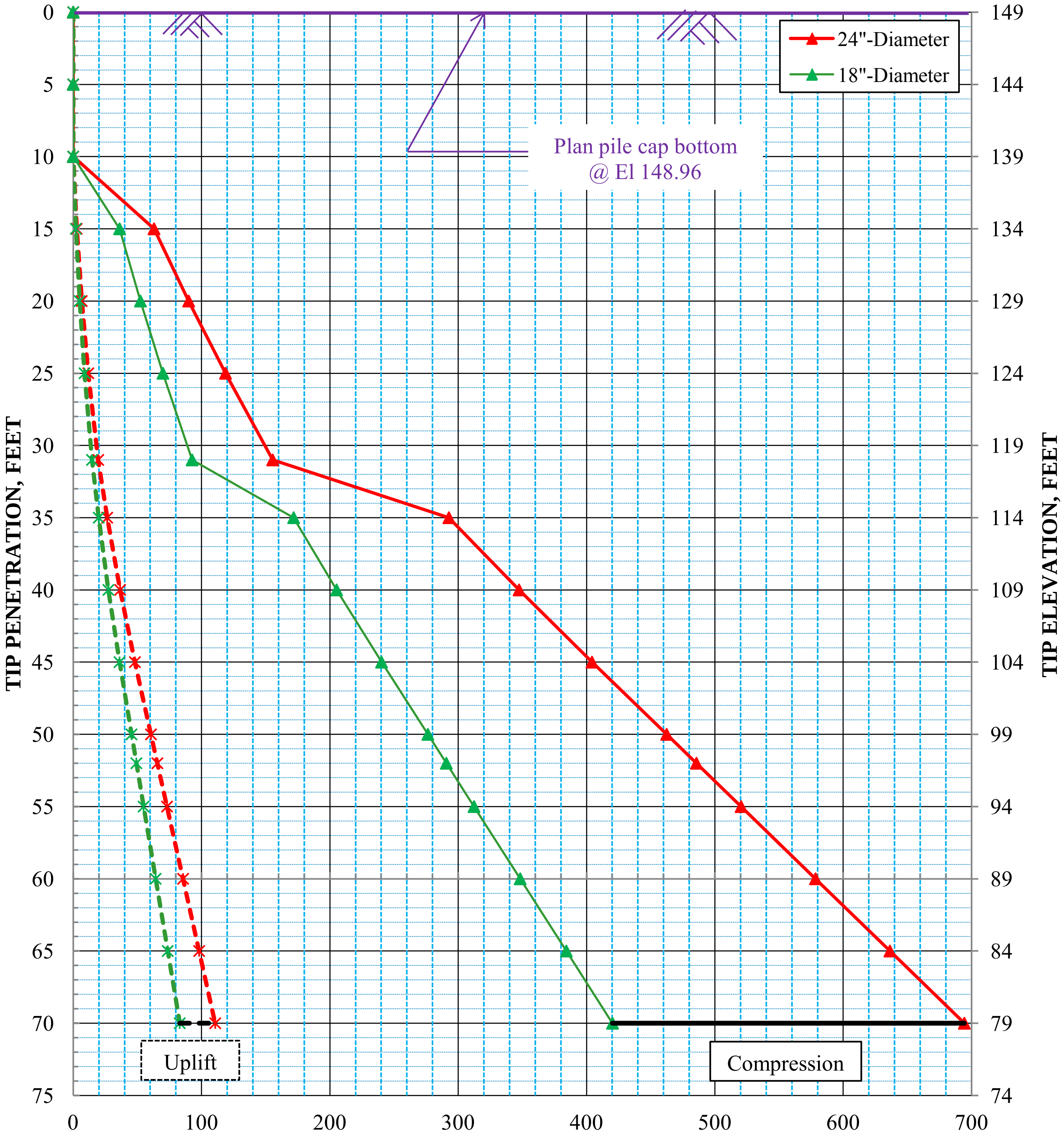
Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 8
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

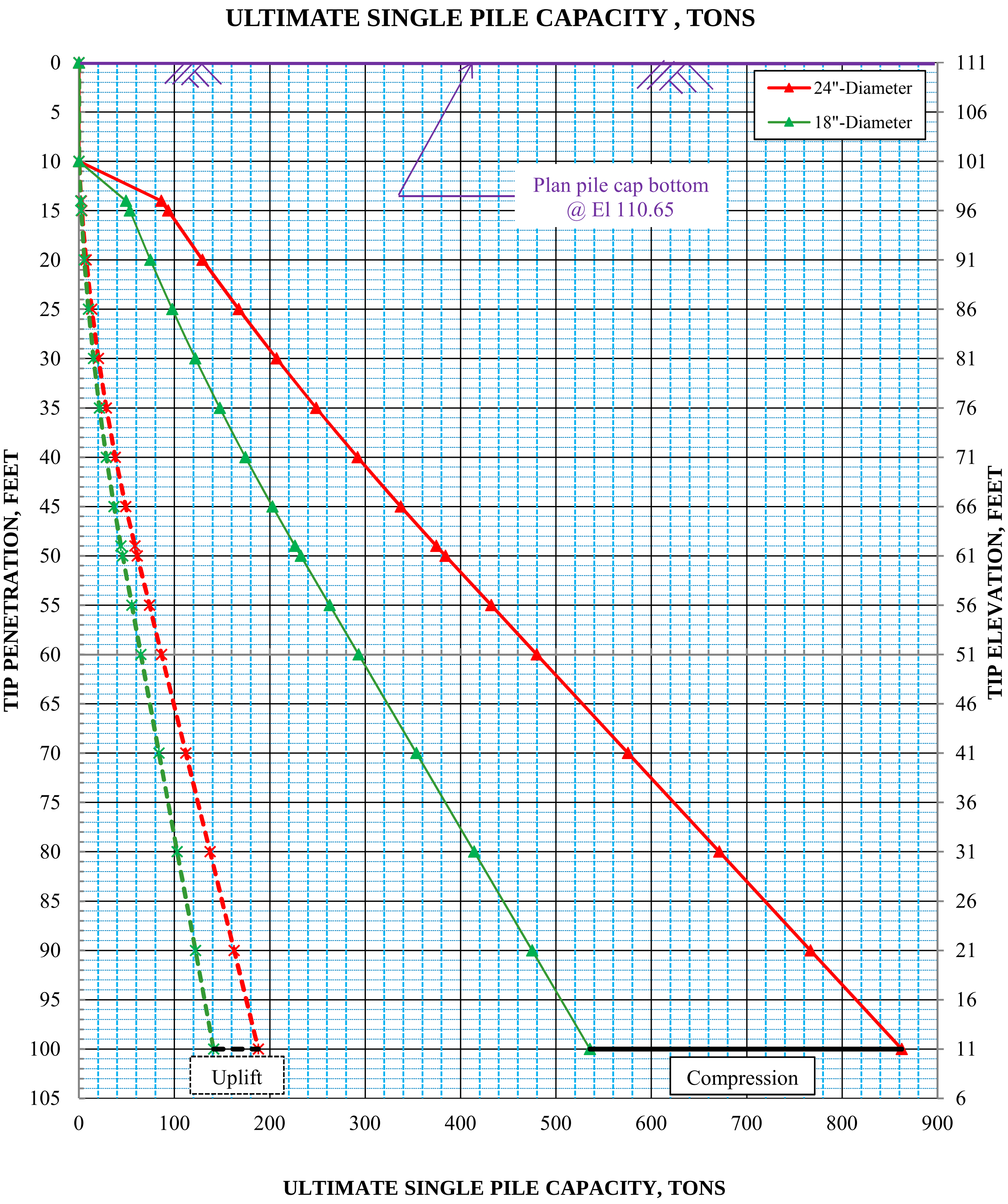
ULTIMATE SINGLE PILE CAPACITY , TONS



ULTIMATE SINGLE PILE CAPACITY, TONS

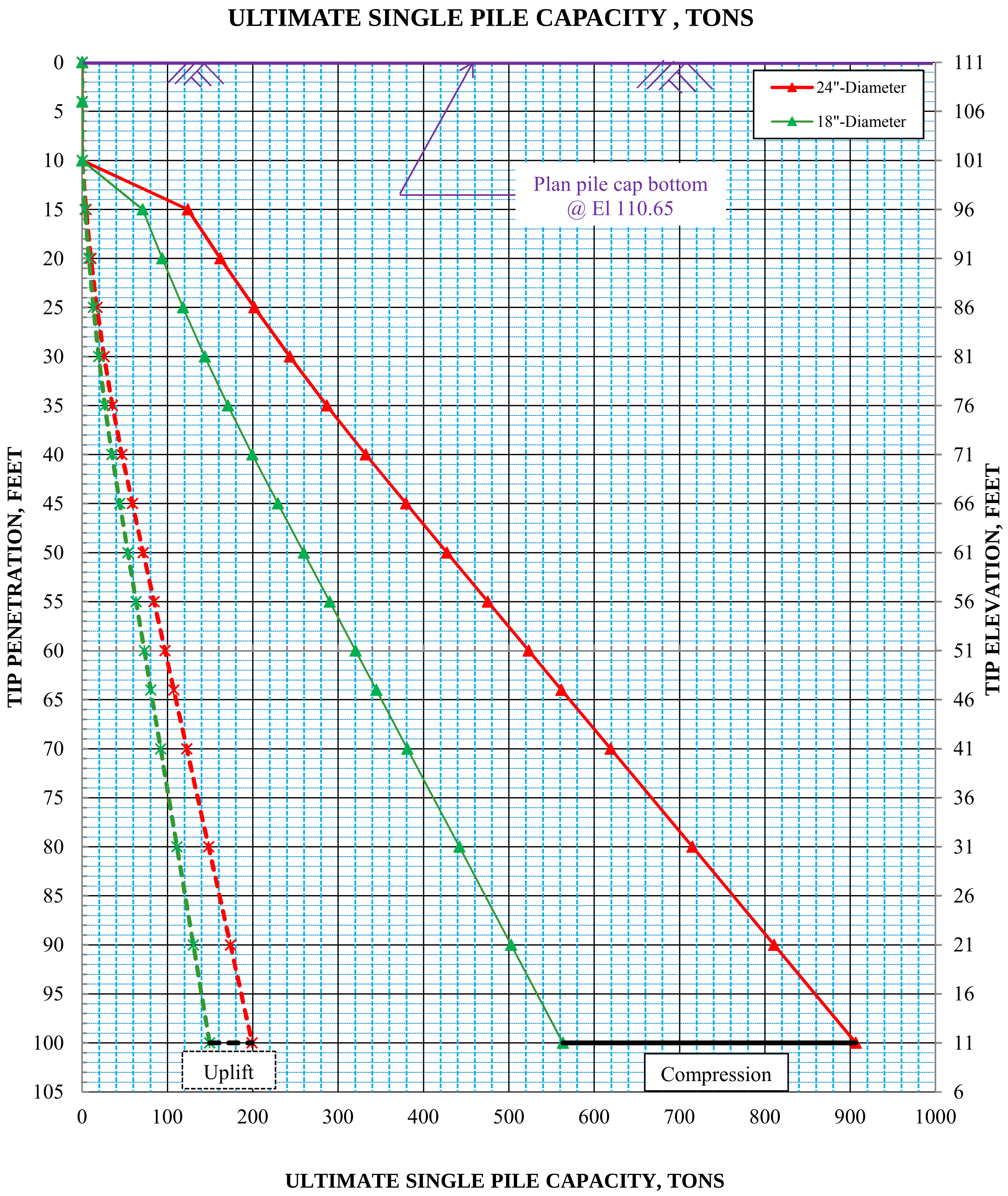
Driven Steel Shells
Bent 9
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



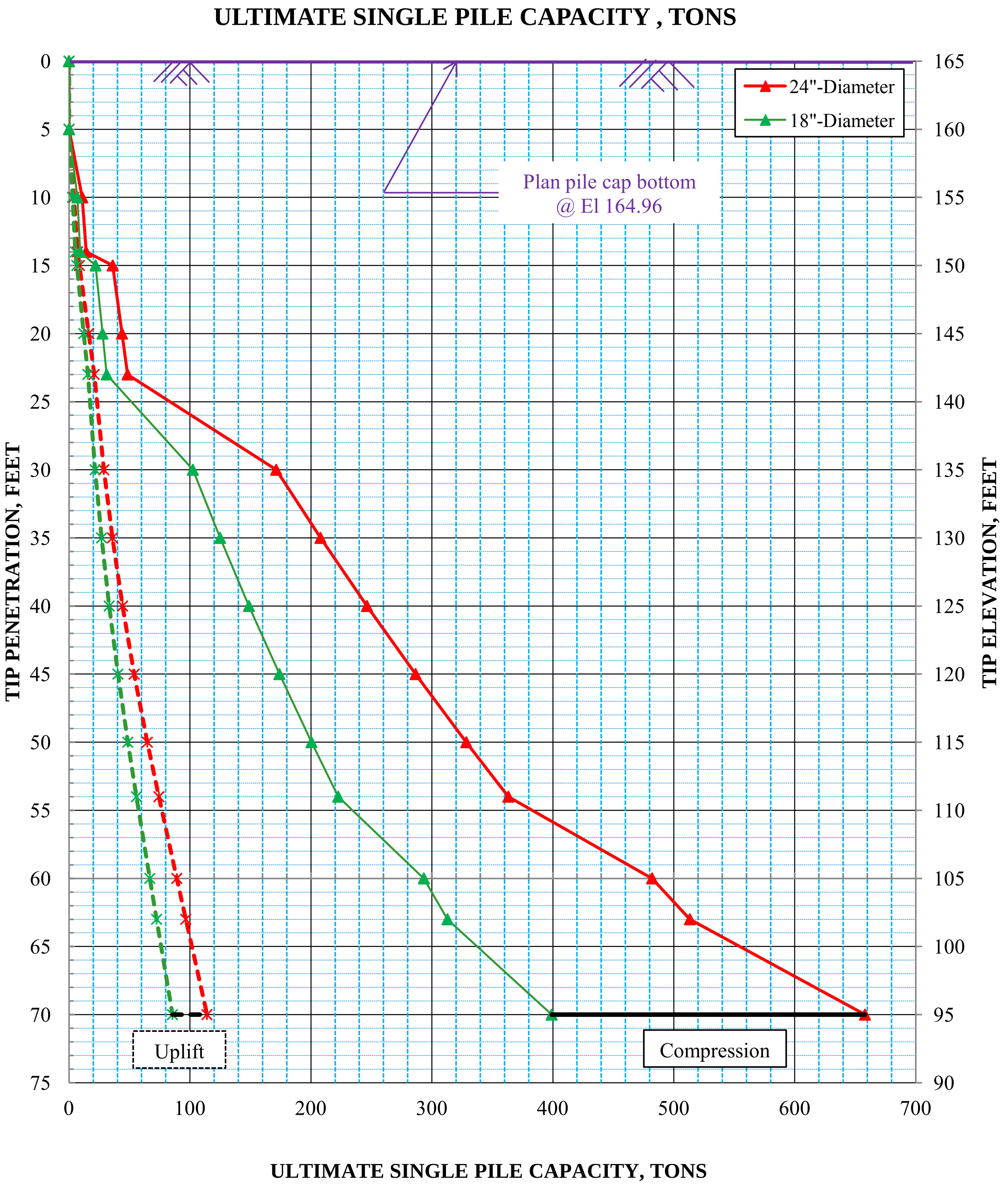
Driven Steel Shells
Pier No. 1
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



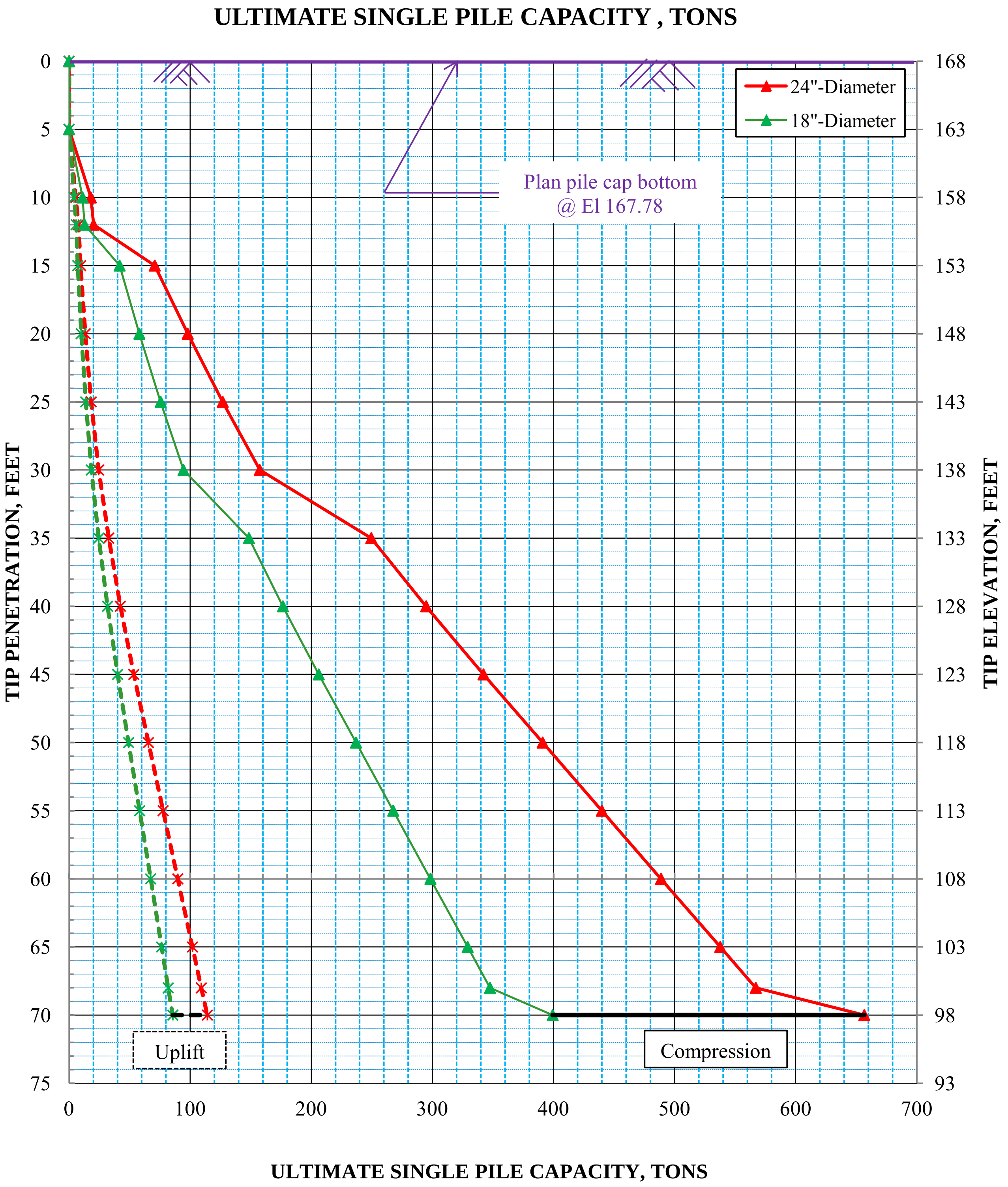
Driven Steel Shells
Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



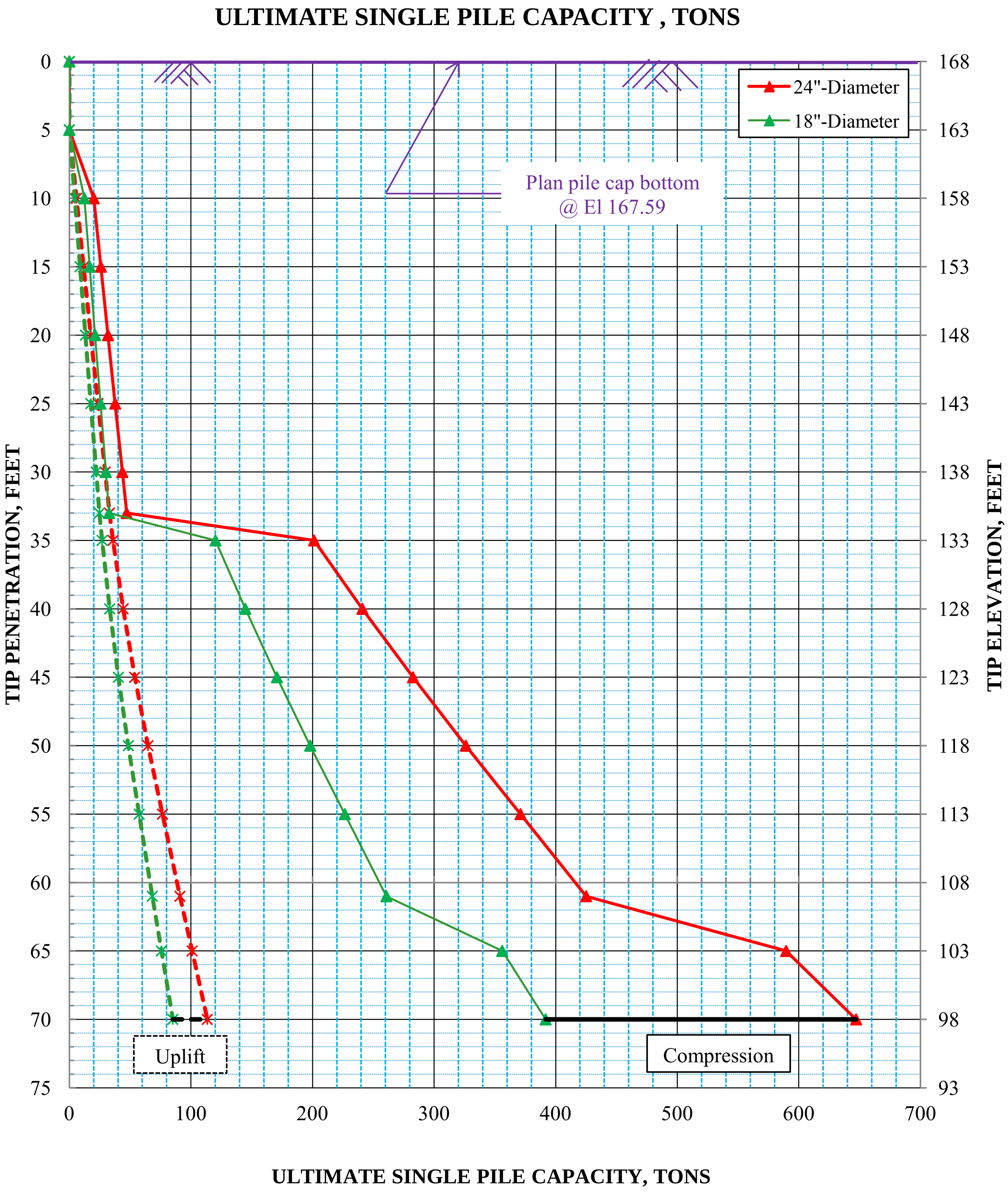
Driven Steel Shells
Bent 10
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



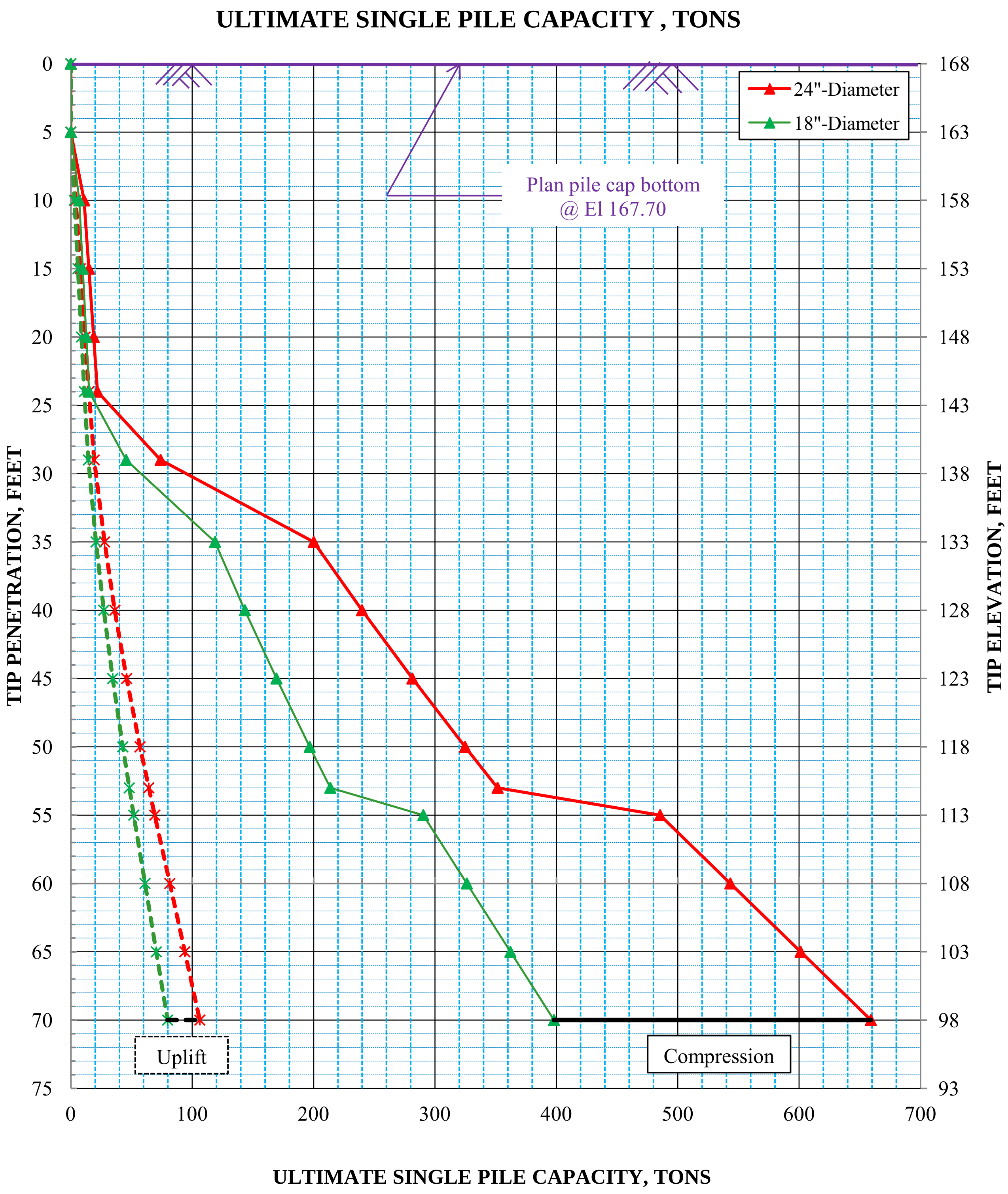
Driven Steel Shells
Bent 11
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 12
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 13
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

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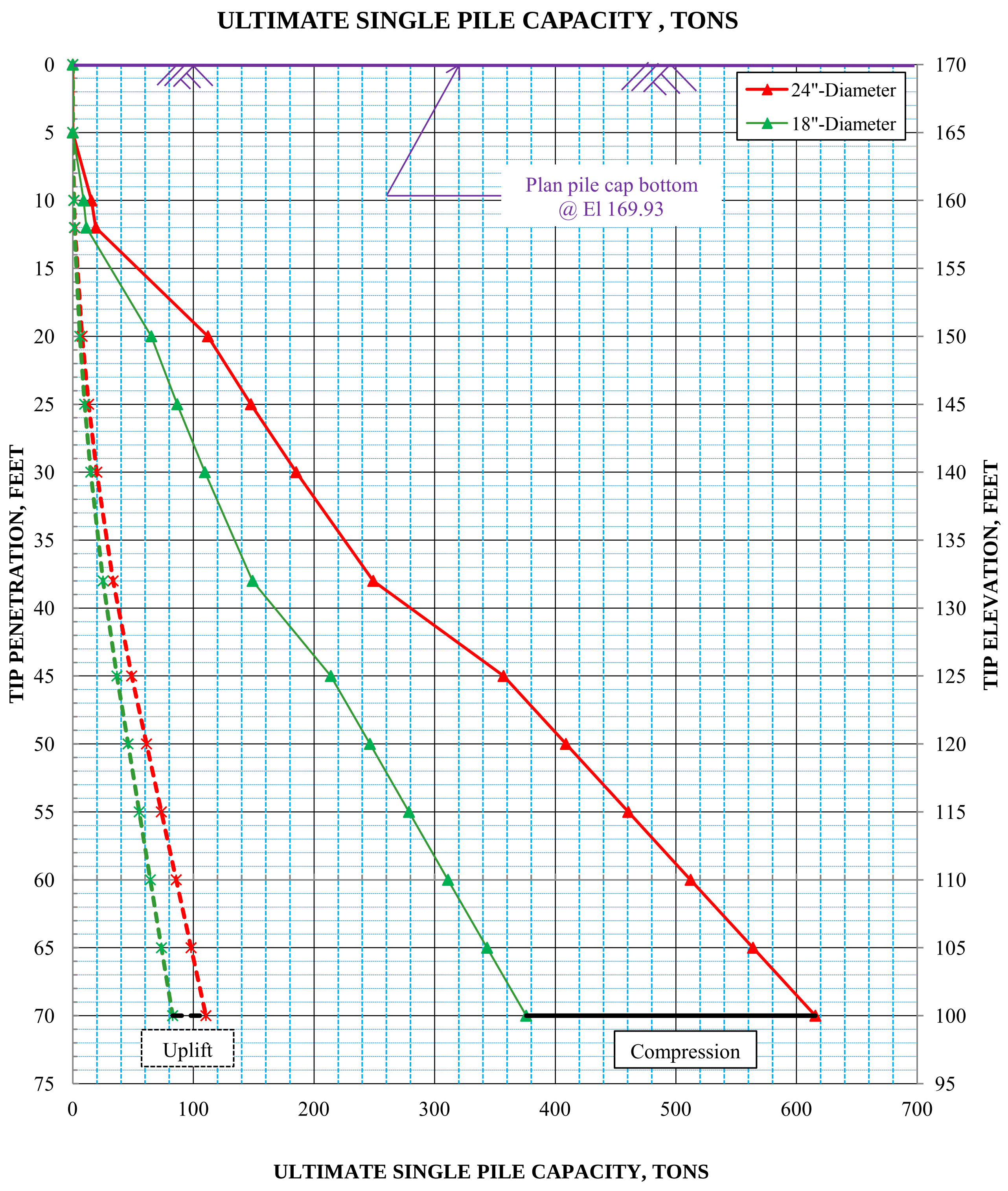
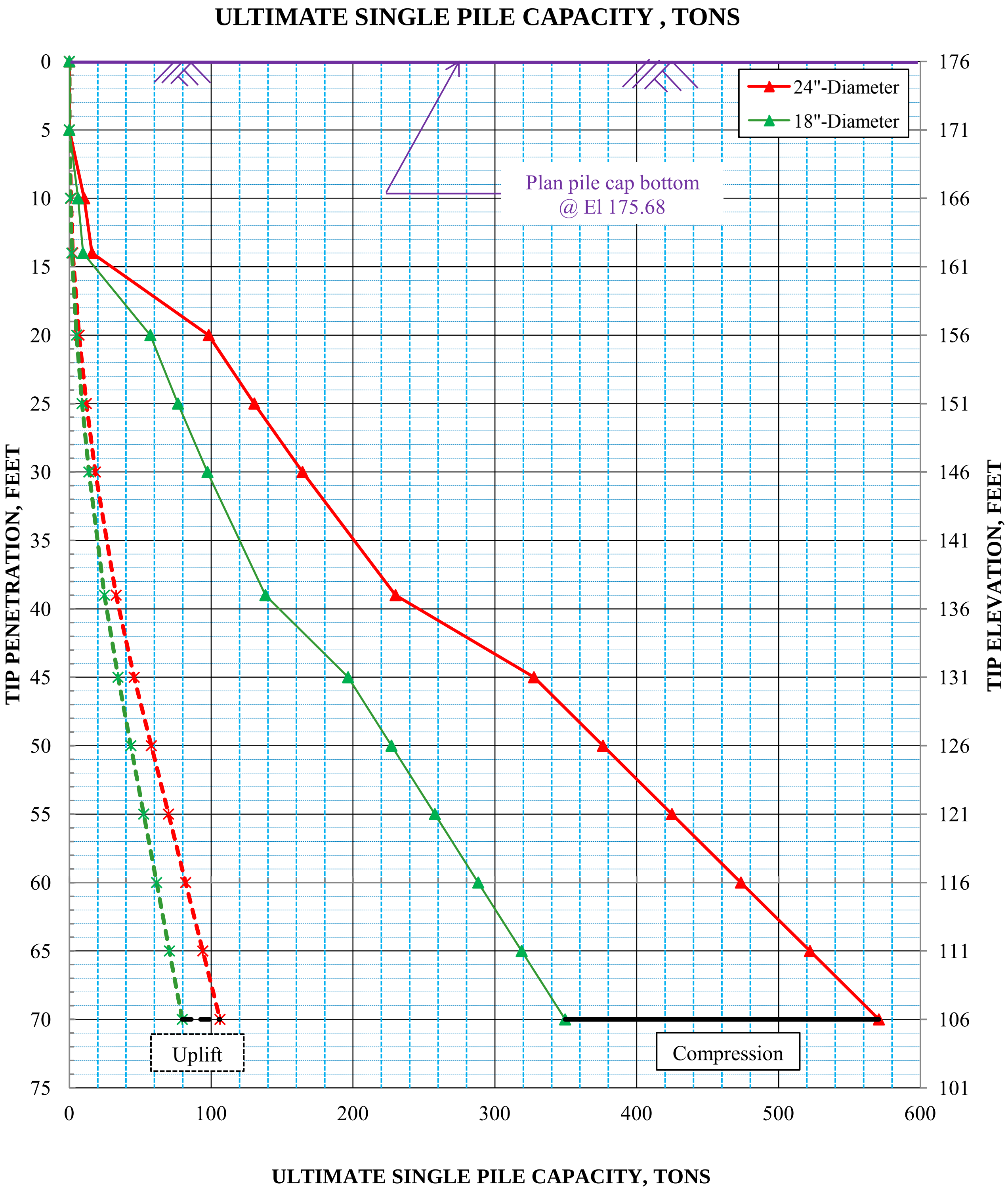


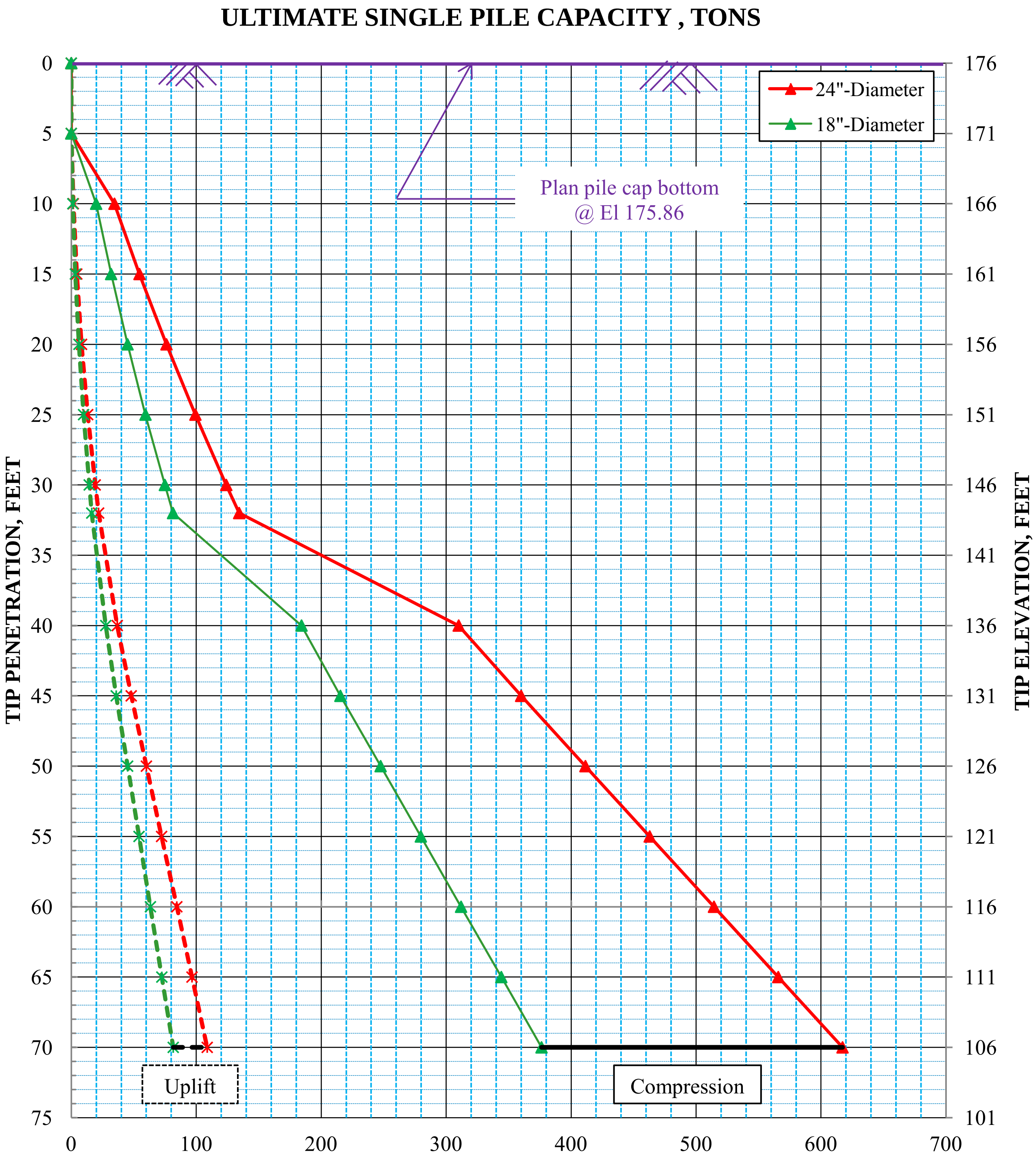
Driven Steel Shells
Bent 14
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.



Driven Steel Shells
Bent 15
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

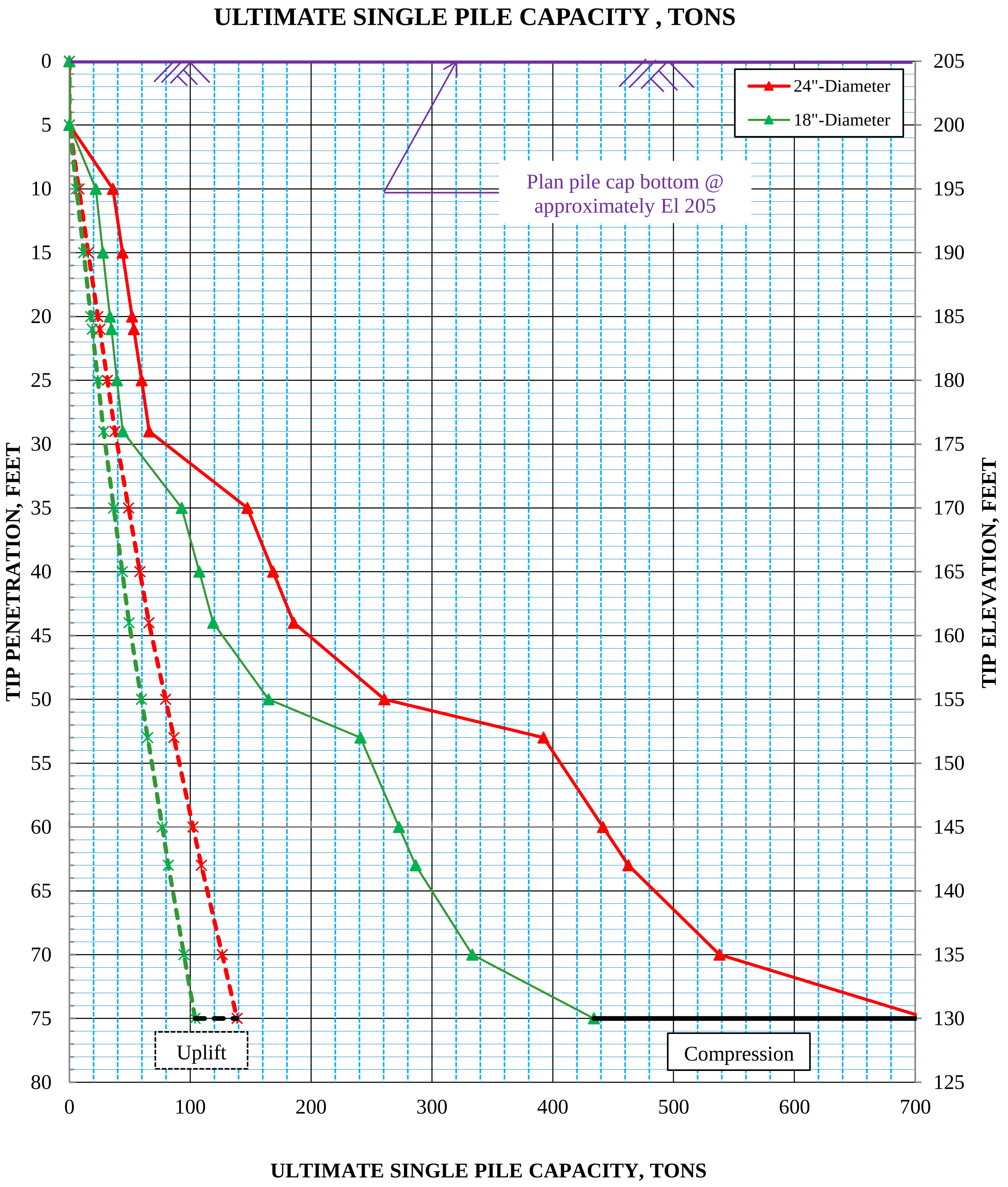
Note: Piles assumed to be jetted to 5 ft above tip elevation.



ULTIMATE SINGLE PILE CAPACITY, TONS

Driven Steel Shells
Bent 16
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be jetted to 5 ft above tip elevation.

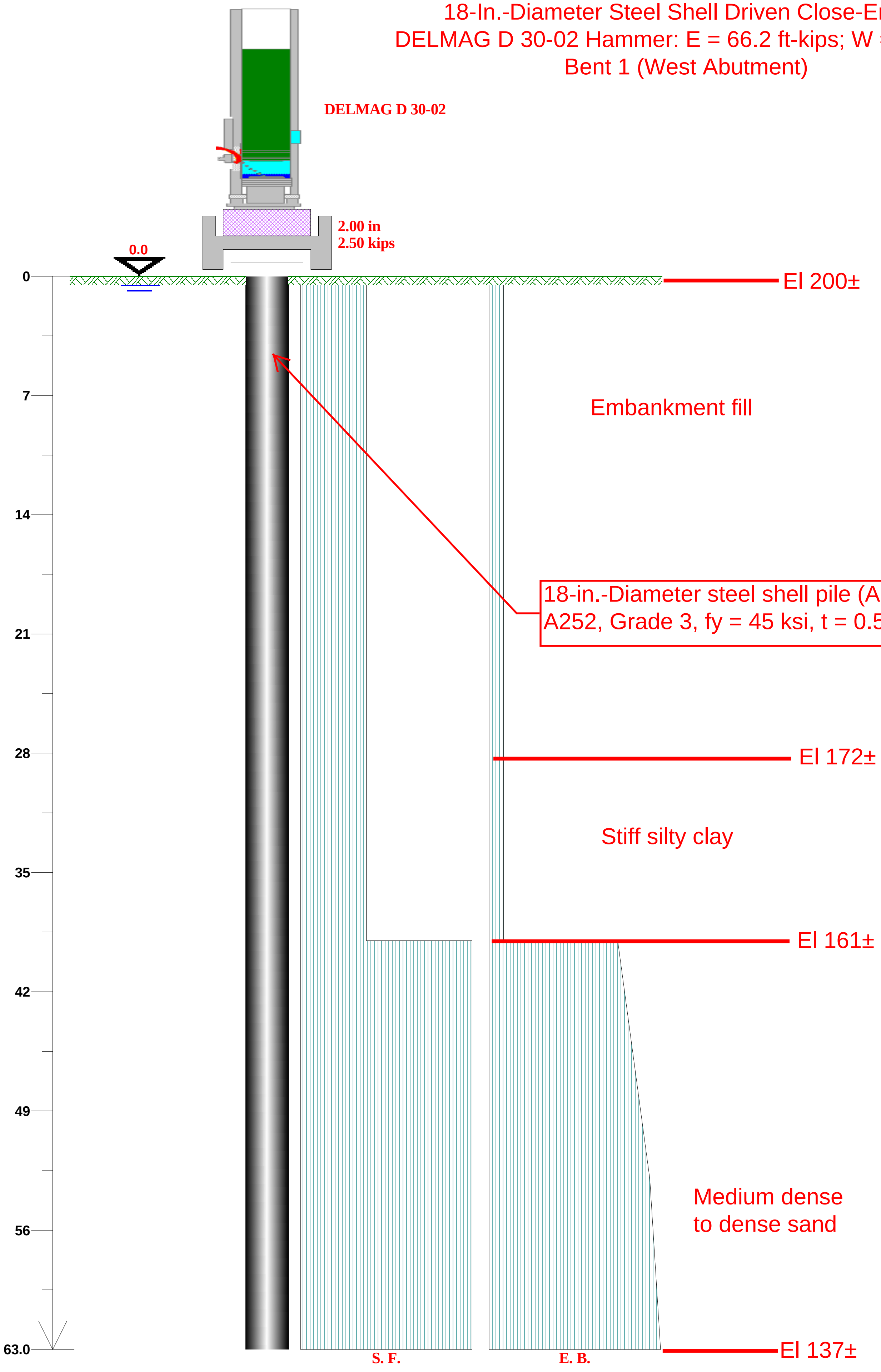


Driven Steel Shells
Bent 17 (East Abutment)
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

Note: Piles assumed to be driven to plan tip elevation.

APPENDIX G

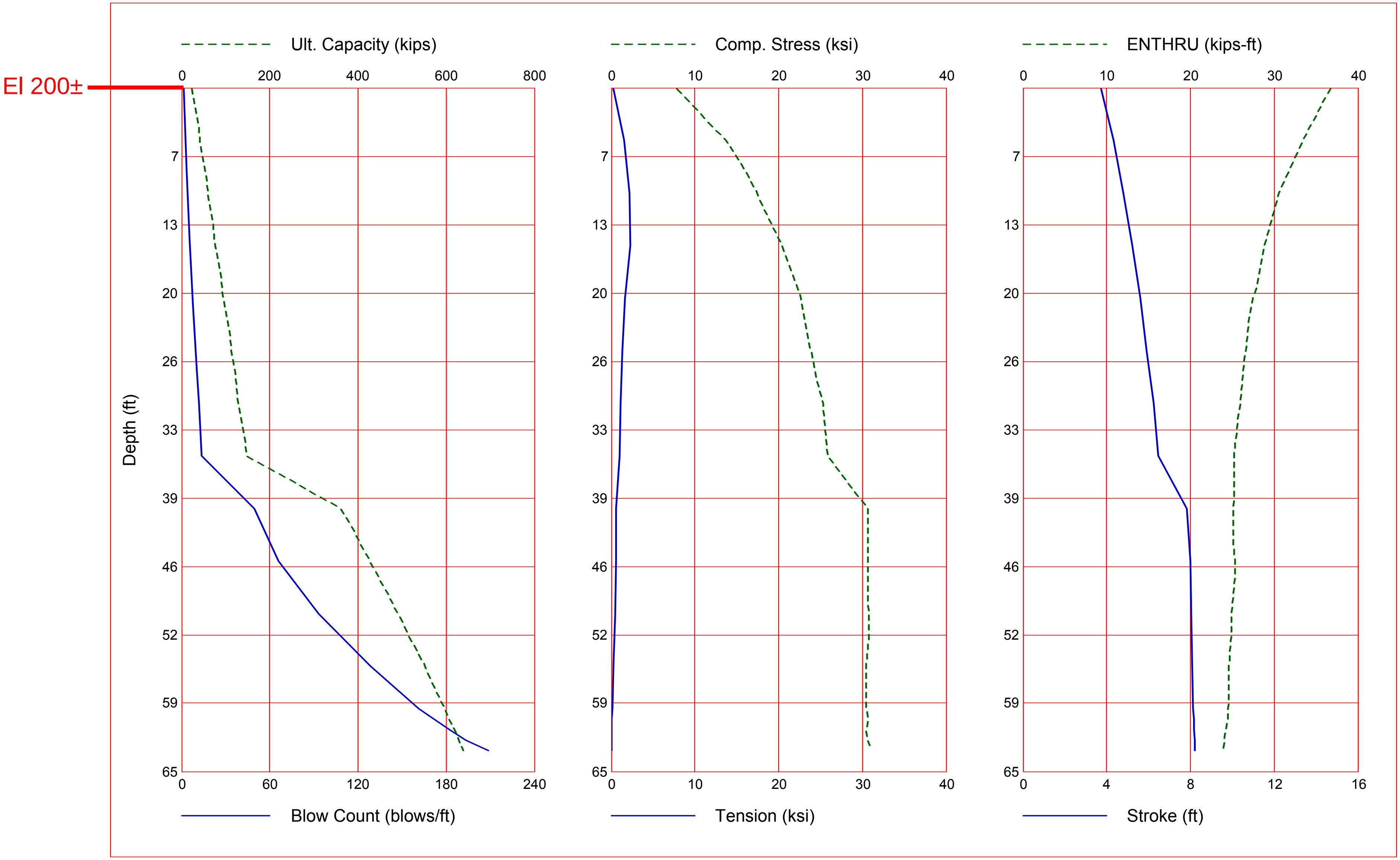
Model for Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1 (West Abutment)



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 1_18 in.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 1_18 in.

Jan 29 2015
GRLWEAP Version 2010

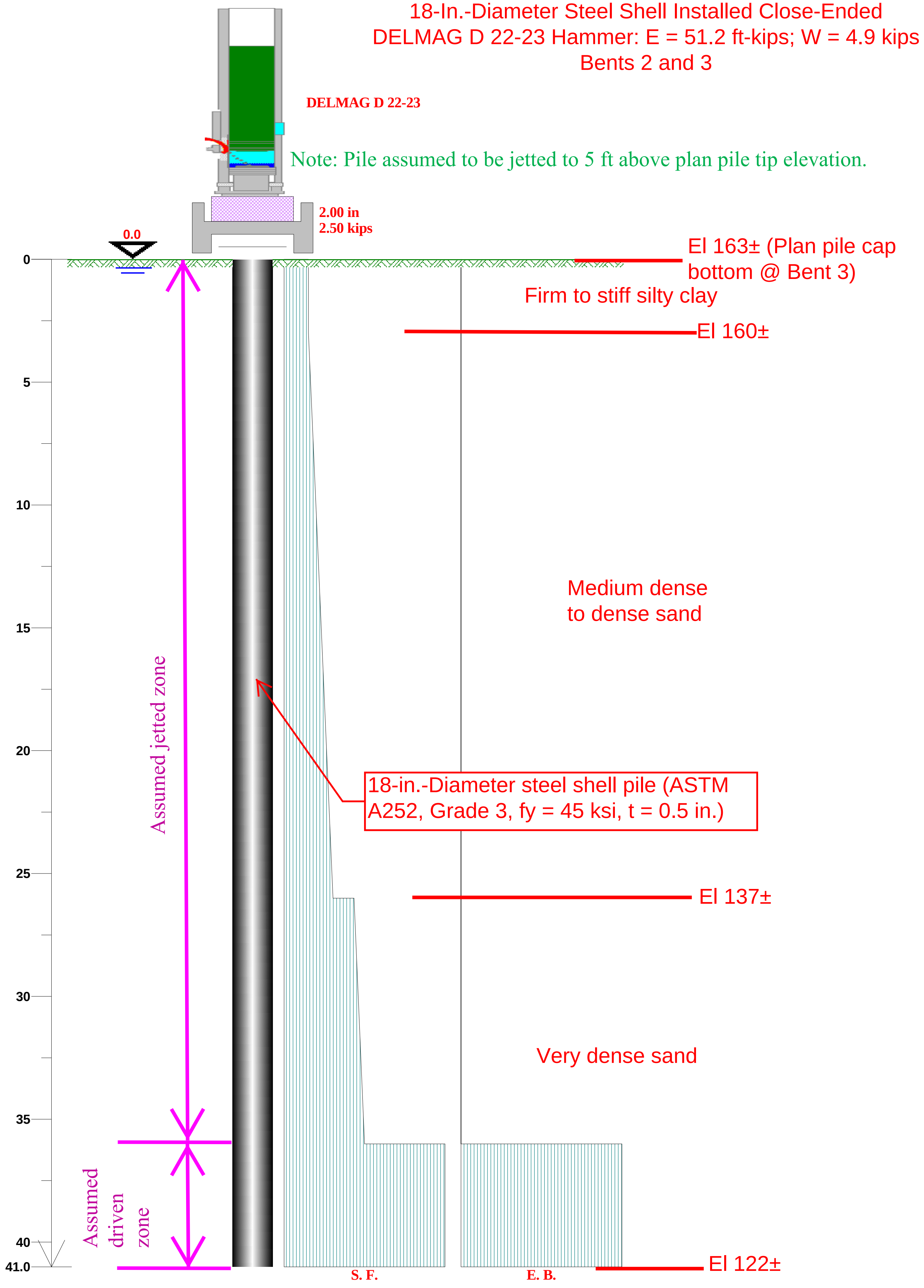
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	24.2	0.4	23.8	1.3	7.834	-0.253	3.73	36.7
5.0	41.4	17.6	23.8	2.5	13.725	-1.502	4.31	33.4
10.0	59.1	35.2	23.8	4.0	17.456	-2.172	4.80	30.6
15.0	76.7	52.9	23.8	5.7	20.475	-2.247	5.23	28.8
20.0	94.3	70.5	23.8	7.5	22.696	-1.617	5.59	27.5
25.0	111.9	88.1	23.8	9.5	23.884	-1.295	5.91	26.6
30.0	129.6	105.7	23.8	11.8	25.323	-1.070	6.24	26.0
35.0	147.2	123.4	23.8	13.9	25.888	-1.039	6.46	25.2
40.0	362.1	146.6	215.4	49.7	30.623	-0.561	7.85	25.1
45.0	426.8	192.5	234.3	66.1	30.643	-0.569	8.02	25.3
50.0	491.5	238.3	253.2	93.0	30.826	-0.533	8.04	24.9
55.0	552.2	284.1	268.1	128.7	30.426	-0.289	8.10	24.6
59.0	595.9	320.8	275.2	161.5	30.495	-0.145	8.14	24.5
60.0	606.9	329.9	276.9	171.4	30.662	-0.085	8.18	24.5
61.0	617.8	339.1	278.7	182.1	30.430	0.000	8.19	24.3
62.0	628.7	348.3	280.5	193.4	30.656	0.000	8.22	24.1
63.0	639.7	357.4	282.2	208.7	31.065	0.000	8.22	23.8

Total Continuous Driving Time 70.00 minutes; Total Number of Blows 2954

Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 1

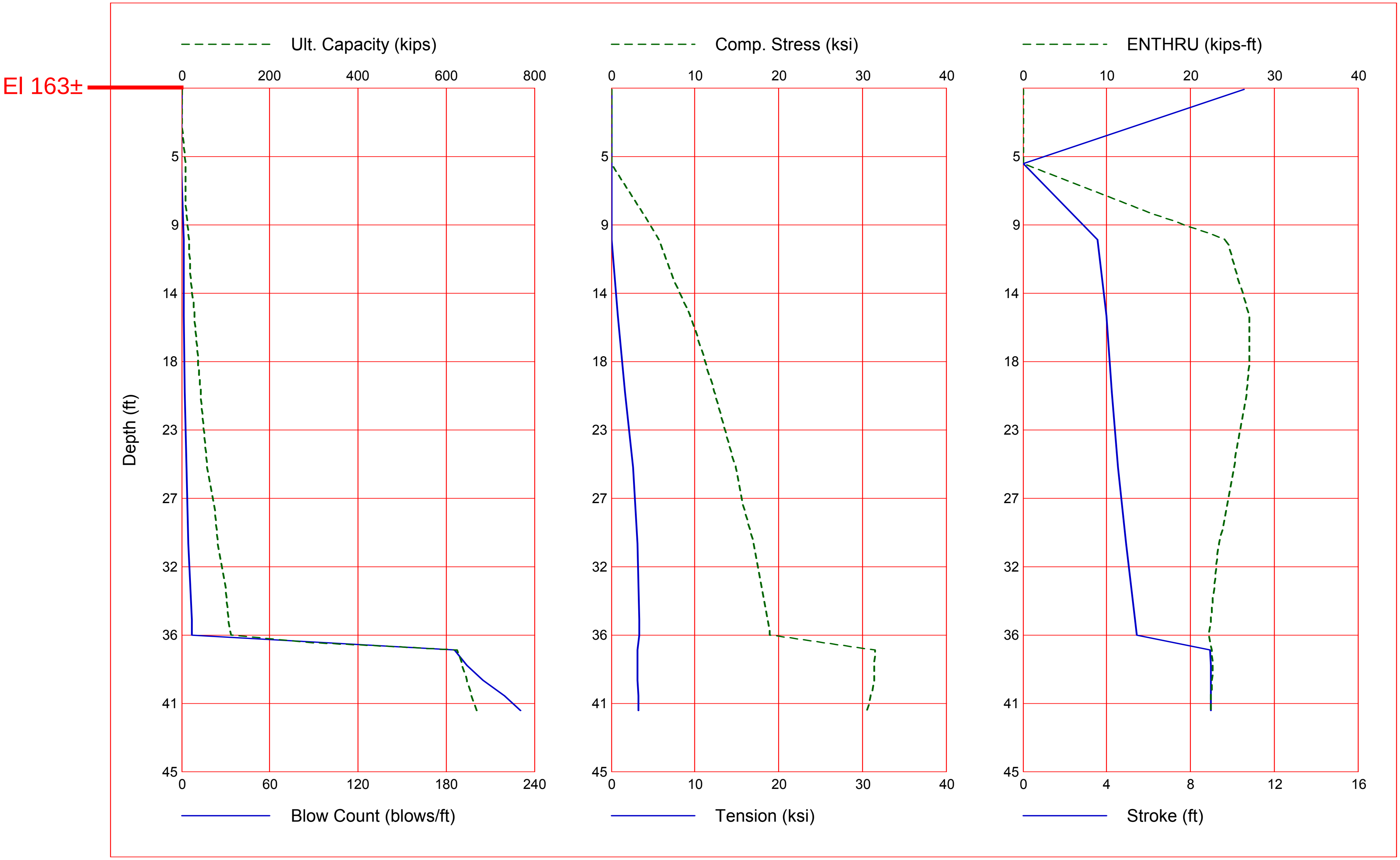
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 3_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 23 2015
13-017_Bent 3_18 in. GRLWEAP Version 2010

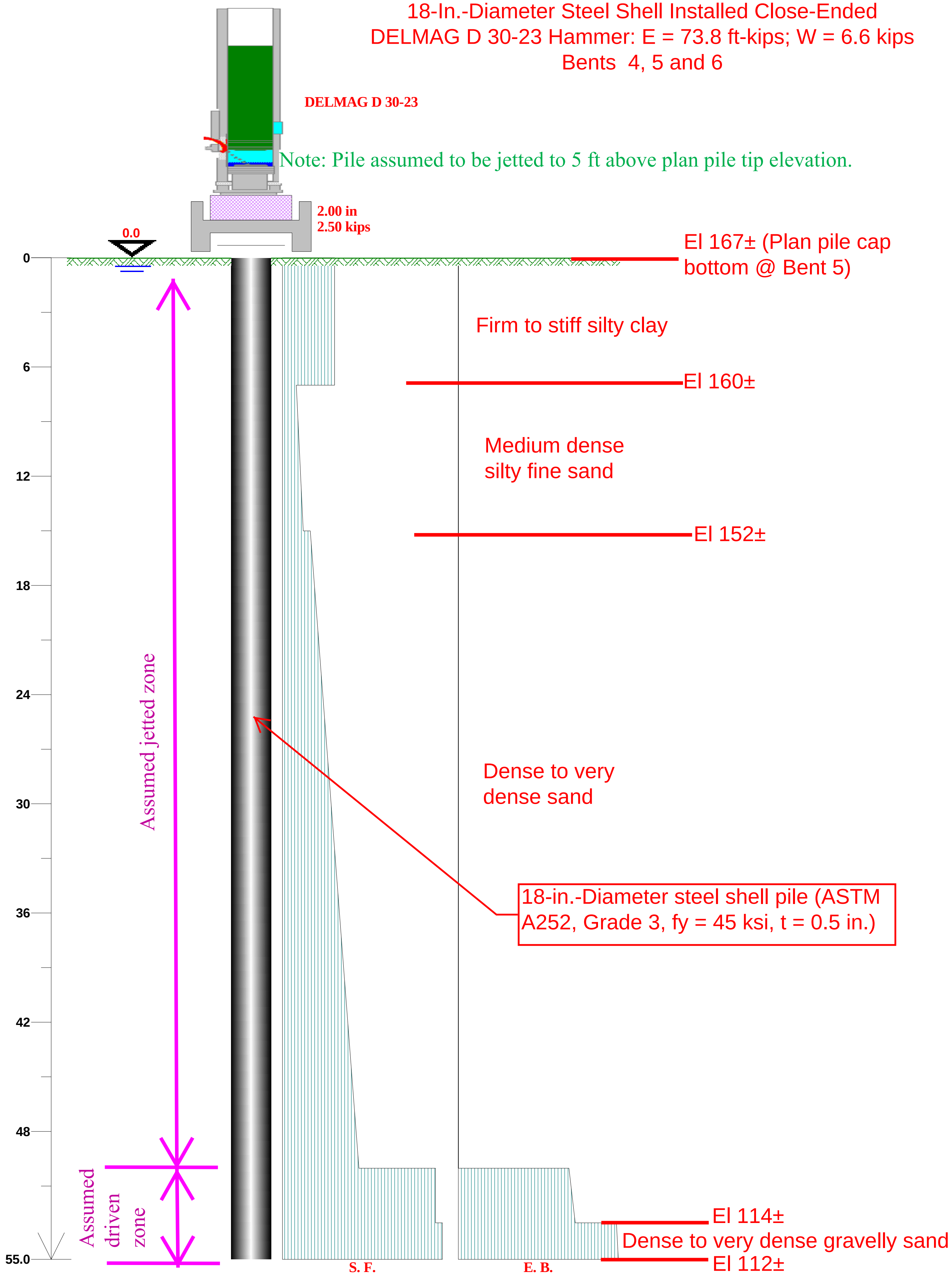
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	10.56	0.0
5.0	8.4	8.4	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	18.2	18.2	0.0	1.5	5.694	0.000	3.58	24.4
15.0	29.8	29.8	0.0	1.8	9.442	-0.757	3.97	27.0
20.0	43.2	43.2	0.0	2.3	12.366	-1.602	4.25	26.7
25.0	58.4	58.4	0.0	3.2	14.879	-2.656	4.56	25.2
30.0	81.1	81.1	0.0	4.8	17.009	-3.102	4.92	23.4
35.0	106.8	106.8	0.0	6.7	18.712	-3.349	5.35	22.4
36.0	112.2	112.2	0.0	7.1	18.951	-3.332	5.42	22.2
37.0	625.7	123.0	502.7	185.4	31.557	-3.135	8.95	22.6
38.0	636.5	133.8	502.7	193.9	31.456	-3.163	8.96	22.7
39.0	647.4	144.6	502.7	204.9	31.362	-3.186	8.97	22.6
40.0	658.2	155.5	502.7	219.6	31.019	-3.208	8.96	22.4
41.0	669.0	166.3	502.7	230.5	30.594	-3.220	8.96	22.4

Total Continuous Driving Time 25.00 minutes; Total Number of Blows 1014

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 2 and 3

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6



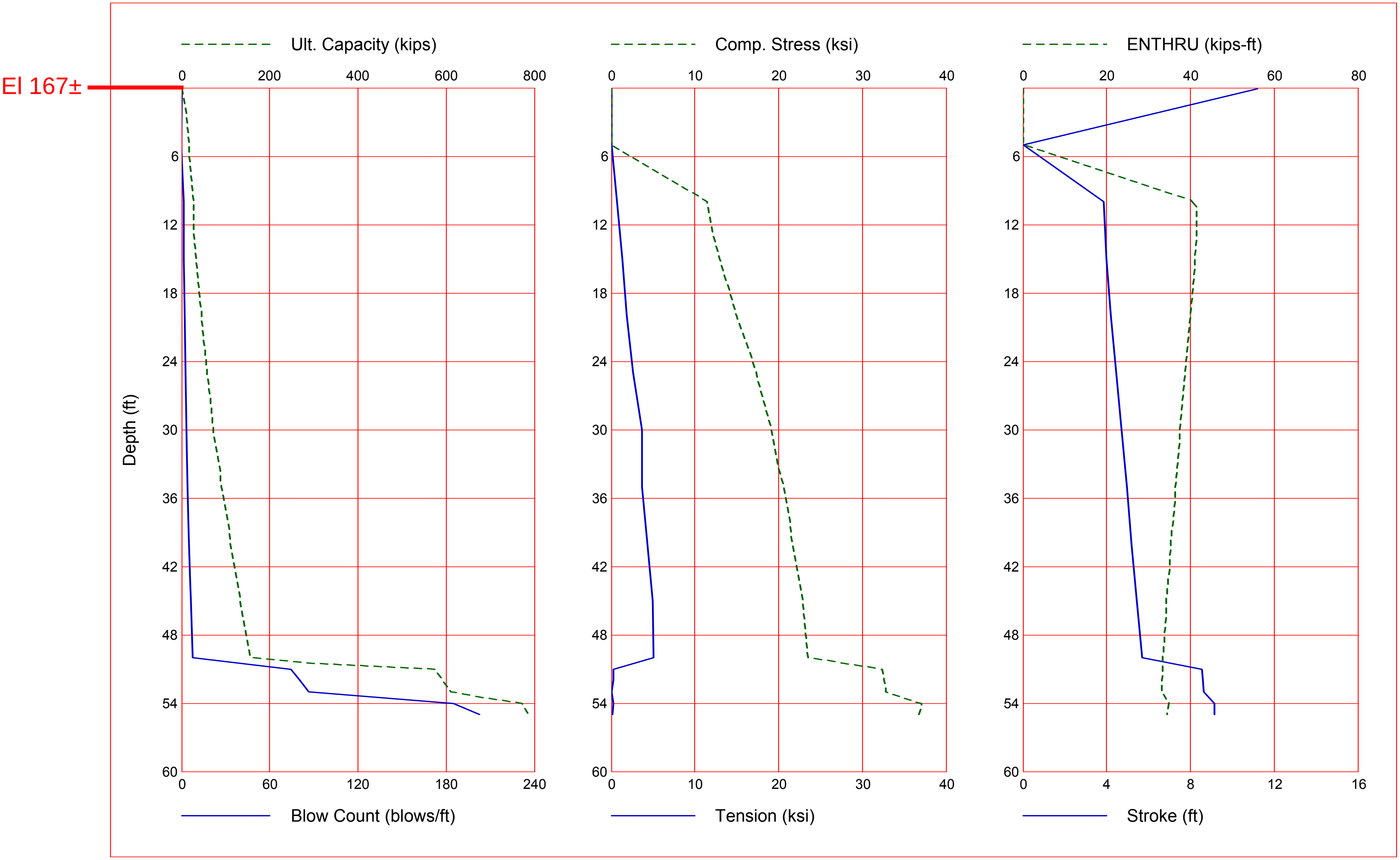
Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

18-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.5 in.)

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 5_18 in.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 29 2015
13-017_Bent 5_18 in. GRLWEAP Version 2010

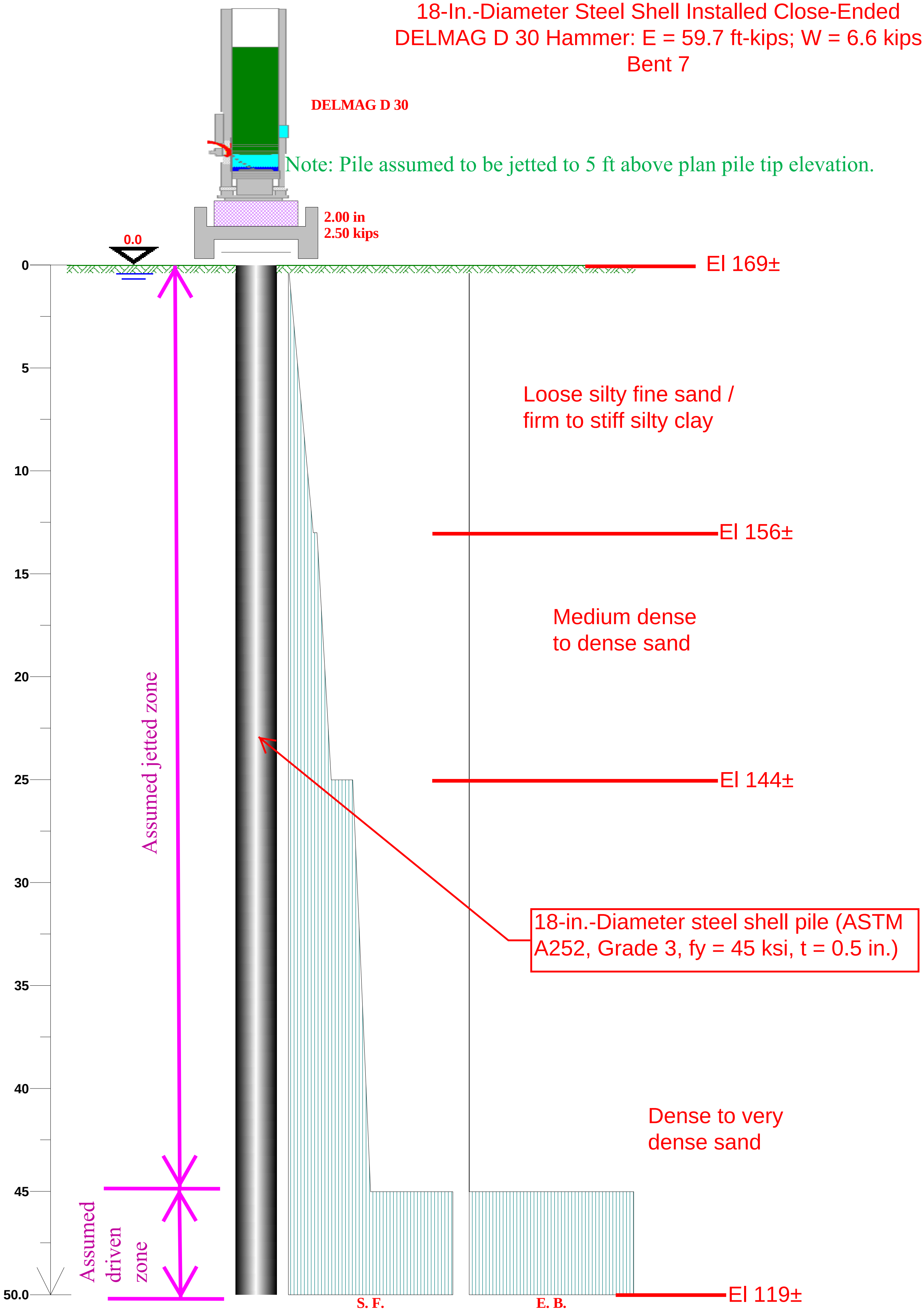
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.4	0.4	0.0	0.0	0.000	0.000	11.18	0.0
5.0	17.6	17.6	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	27.8	27.8	0.0	1.6	11.504	-0.665	3.88	41.5
15.0	34.1	34.1	0.0	1.8	12.936	-1.376	3.99	41.1
20.0	44.6	44.6	0.0	2.2	14.998	-1.879	4.20	40.0
25.0	57.6	57.6	0.0	2.7	17.336	-2.593	4.46	38.8
30.0	72.8	72.8	0.0	3.4	19.134	-3.645	4.70	37.5
35.0	90.5	90.5	0.0	4.1	20.688	-3.670	4.95	36.4
40.0	110.4	110.4	0.0	5.1	21.647	-4.373	5.20	35.3
45.0	132.8	132.8	0.0	6.1	22.886	-4.907	5.45	34.3
50.0	157.4	157.4	0.0	7.5	23.546	-5.080	5.71	33.4
51.0	574.1	167.8	406.3	74.2	32.323	-0.286	8.54	33.3
52.0	592.1	178.1	413.9	80.3	32.561	-0.243	8.59	33.2
53.0	610.0	188.5	421.6	86.3	32.849	-0.069	8.63	33.1
54.0	772.5	199.3	573.3	184.9	37.076	-0.313	9.14	34.9
55.0	786.9	210.1	576.8	202.6	36.761	-0.172	9.14	34.4

Total Continuous Driving Time 16.00 minutes; Total Number of Blows 685

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-23 Hammer: E = 73.8 ft-kips; W = 6.6 kips
Bents 4, 5 and 6

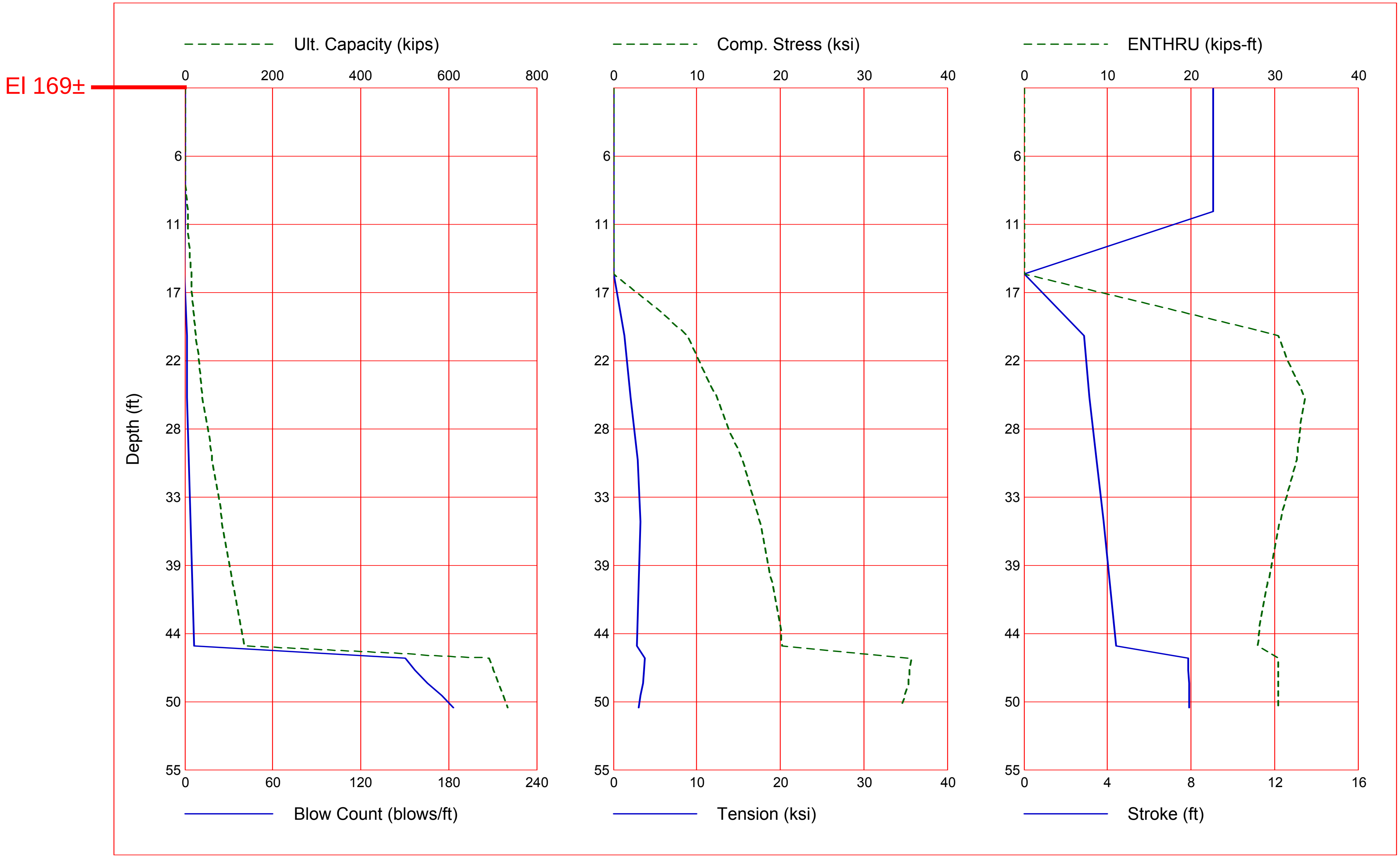
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 7_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 23 2015
13-017_Bent 7_18 in. GRLWEAP Version 2010

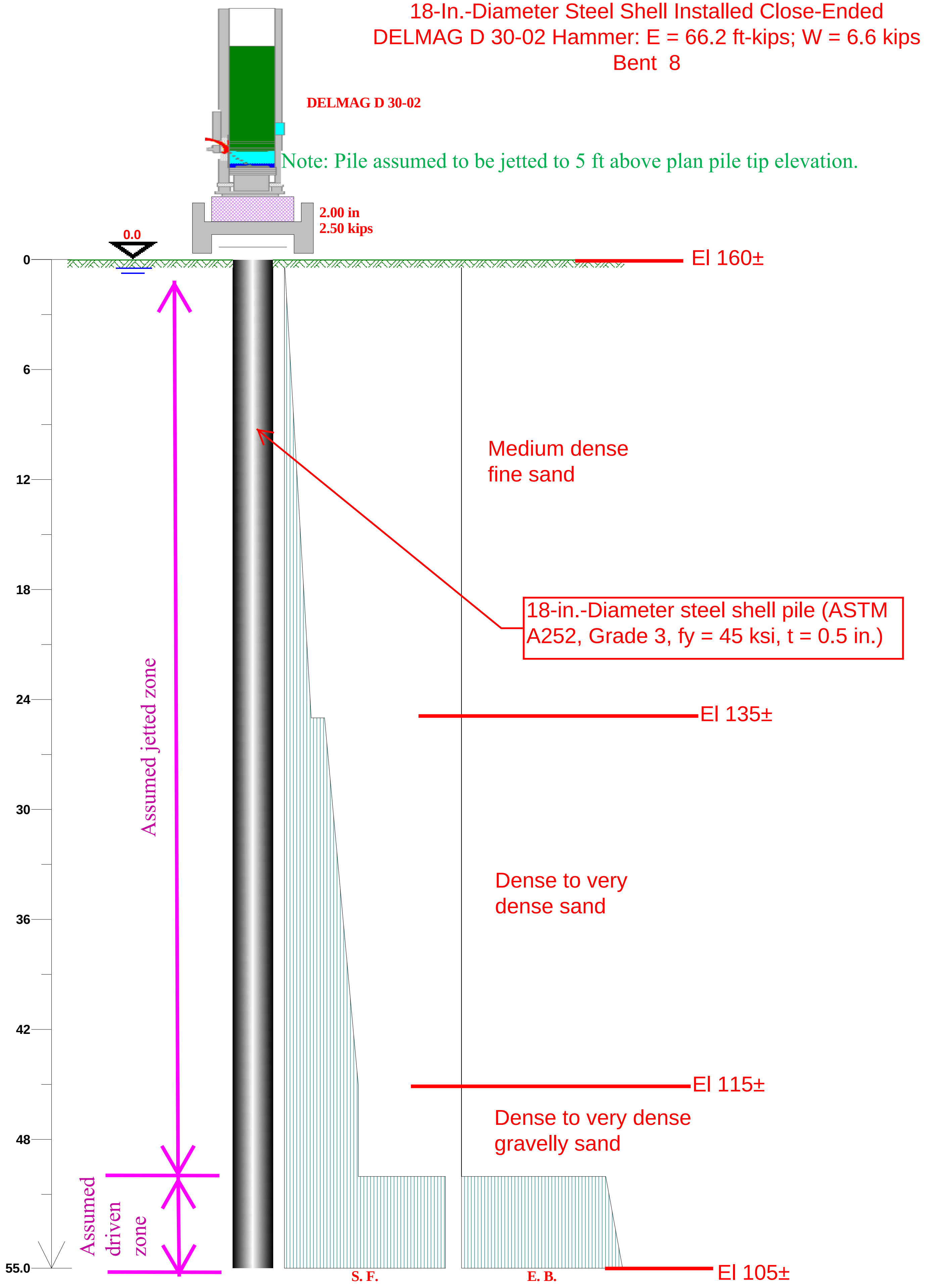
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	9.05	0.0
5.0	1.6	1.6	0.0	0.0	0.000	0.000	9.05	0.0
10.0	6.3	6.3	0.0	0.0	0.000	0.000	9.05	0.0
15.0	14.6	14.6	0.0	-1.0	0.000	0.000	0.00	0.0
20.0	25.8	25.8	0.0	1.4	8.874	-1.325	2.88	30.4
25.0	38.9	38.9	0.0	1.6	12.456	-2.090	3.15	33.6
30.0	60.8	60.8	0.0	2.5	15.531	-2.936	3.48	32.7
35.0	84.1	84.1	0.0	3.7	17.544	-3.215	3.81	30.7
40.0	109.0	109.0	0.0	5.1	19.014	-3.055	4.11	29.2
45.0	135.2	135.2	0.0	6.6	20.251	-2.856	4.41	28.0
46.0	691.1	146.1	545.0	150.4	35.628	-3.823	7.88	30.4
47.0	701.9	156.9	545.0	157.2	35.478	-3.723	7.88	30.5
48.0	712.7	167.7	545.0	165.7	35.333	-3.526	7.90	30.5
49.0	723.5	178.5	545.0	175.1	34.931	-3.305	7.90	30.4
50.0	734.3	189.3	545.0	183.4	34.453	-3.082	7.90	30.5

Total Continuous Driving Time 19.00 minutes; Total Number of Blows 830

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 7

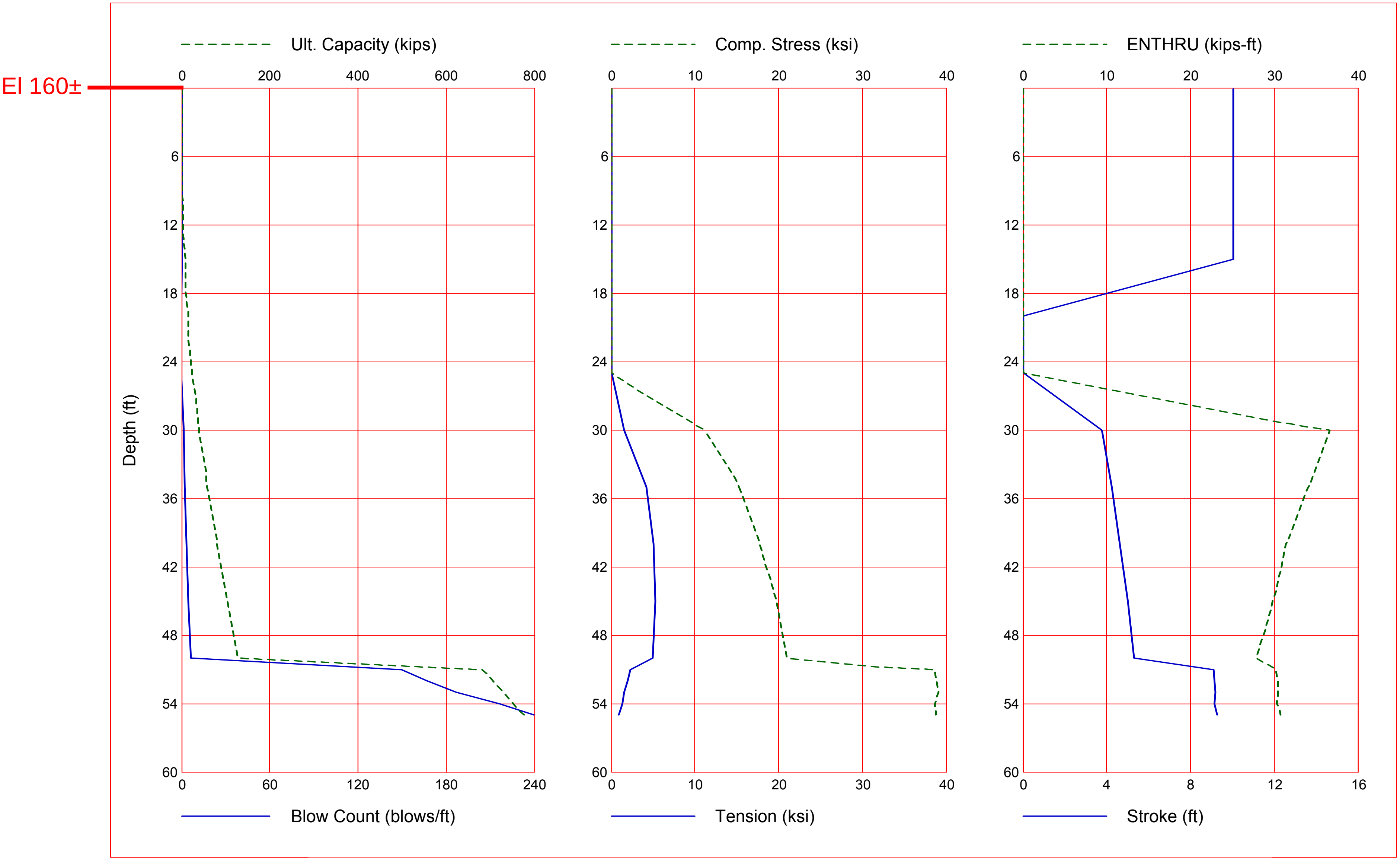
Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 8_18 In.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8

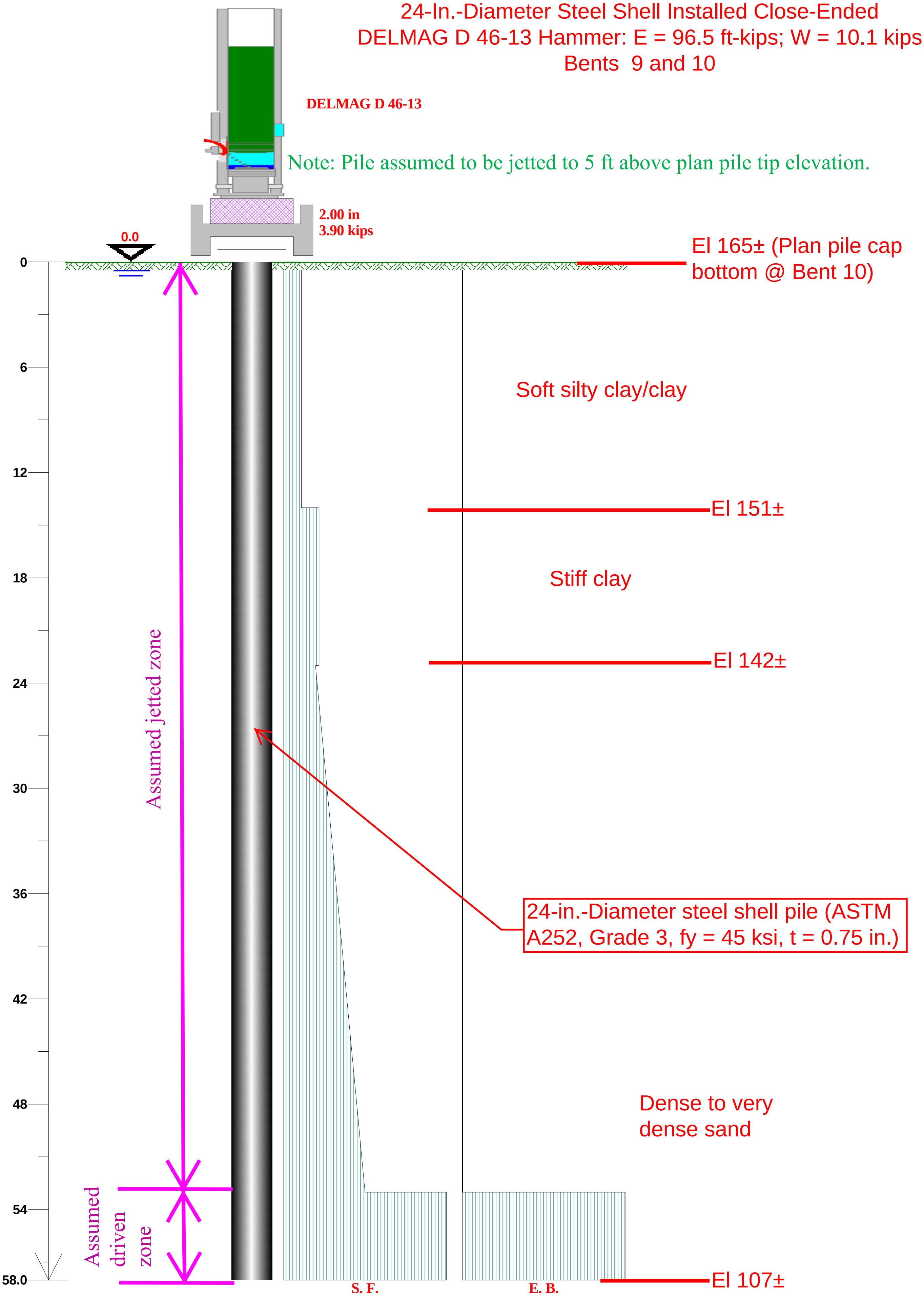
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	10.03	0.0
5.0	0.9	0.9	0.0	0.0	0.000	0.000	10.03	0.0
10.0	3.8	3.8	0.0	0.0	0.000	0.000	10.03	0.0
15.0	8.5	8.5	0.0	0.0	0.000	0.000	10.03	0.0
20.0	15.0	15.0	0.0	-1.0	0.000	0.000	0.00	0.0
25.0	23.5	23.5	0.0	-1.0	0.000	0.000	0.00	0.0
30.0	39.1	39.1	0.0	1.4	11.107	-1.556	3.78	36.6
35.0	57.6	57.6	0.0	2.2	15.285	-4.178	4.24	34.1
40.0	79.0	79.0	0.0	3.3	17.808	-5.035	4.61	31.4
45.0	103.4	103.4	0.0	4.7	19.736	-5.293	5.02	29.8
50.0	129.2	129.2	0.0	6.3	20.979	-4.979	5.32	27.9
51.0	682.0	140.5	541.5	149.9	38.661	-2.340	9.11	30.2
52.0	705.7	151.8	553.9	166.9	38.860	-1.976	9.16	30.4
53.0	729.3	163.1	566.2	186.7	39.048	-1.561	9.20	30.5
54.0	752.9	174.4	578.6	216.5	38.636	-1.370	9.16	30.3
55.0	776.6	185.7	590.9	239.6	38.723	-0.944	9.28	30.8

Total Continuous Driving Time 23.00 minutes; Total Number of Blows 917

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 8

Model for Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10

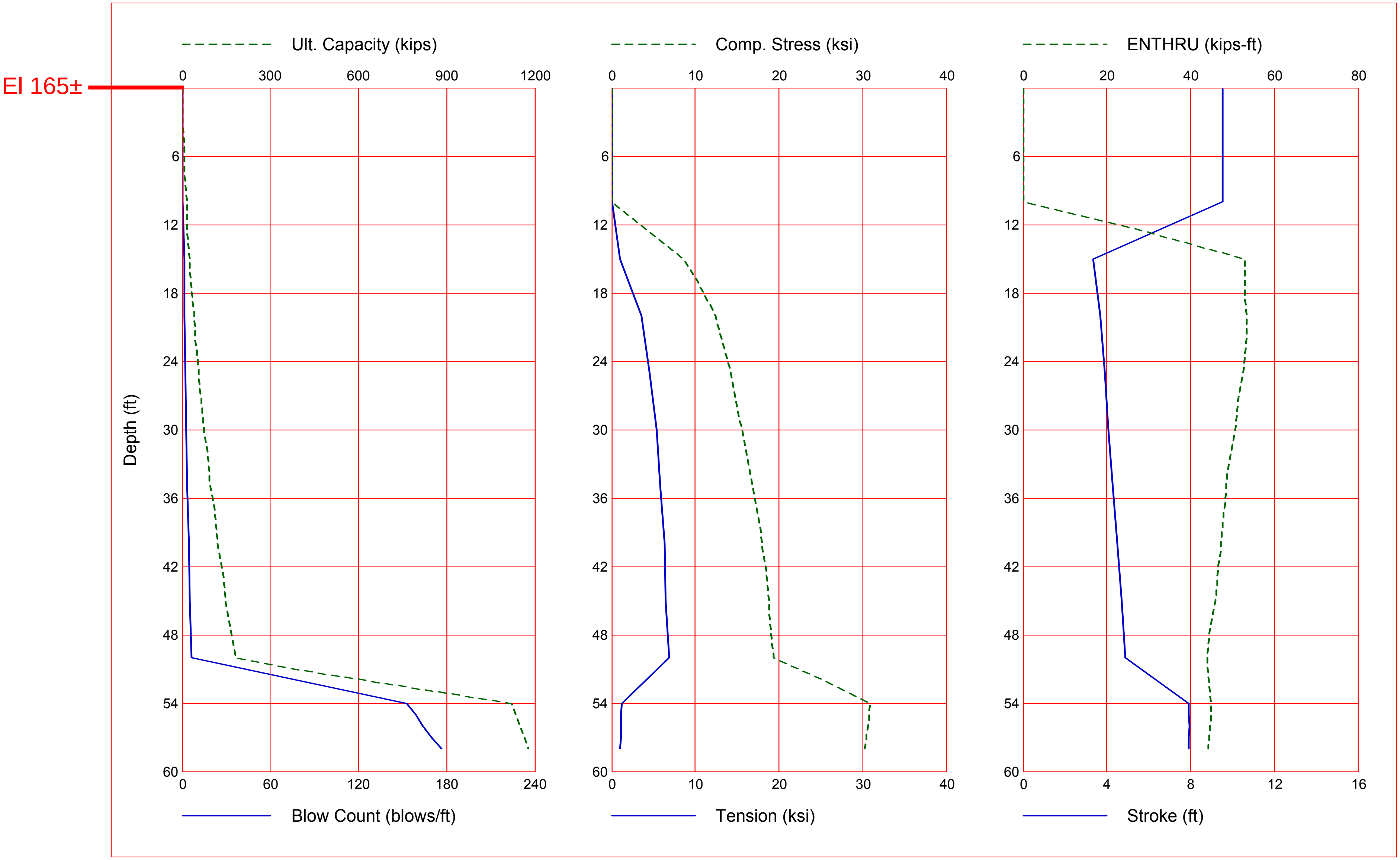


Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 10_24 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10

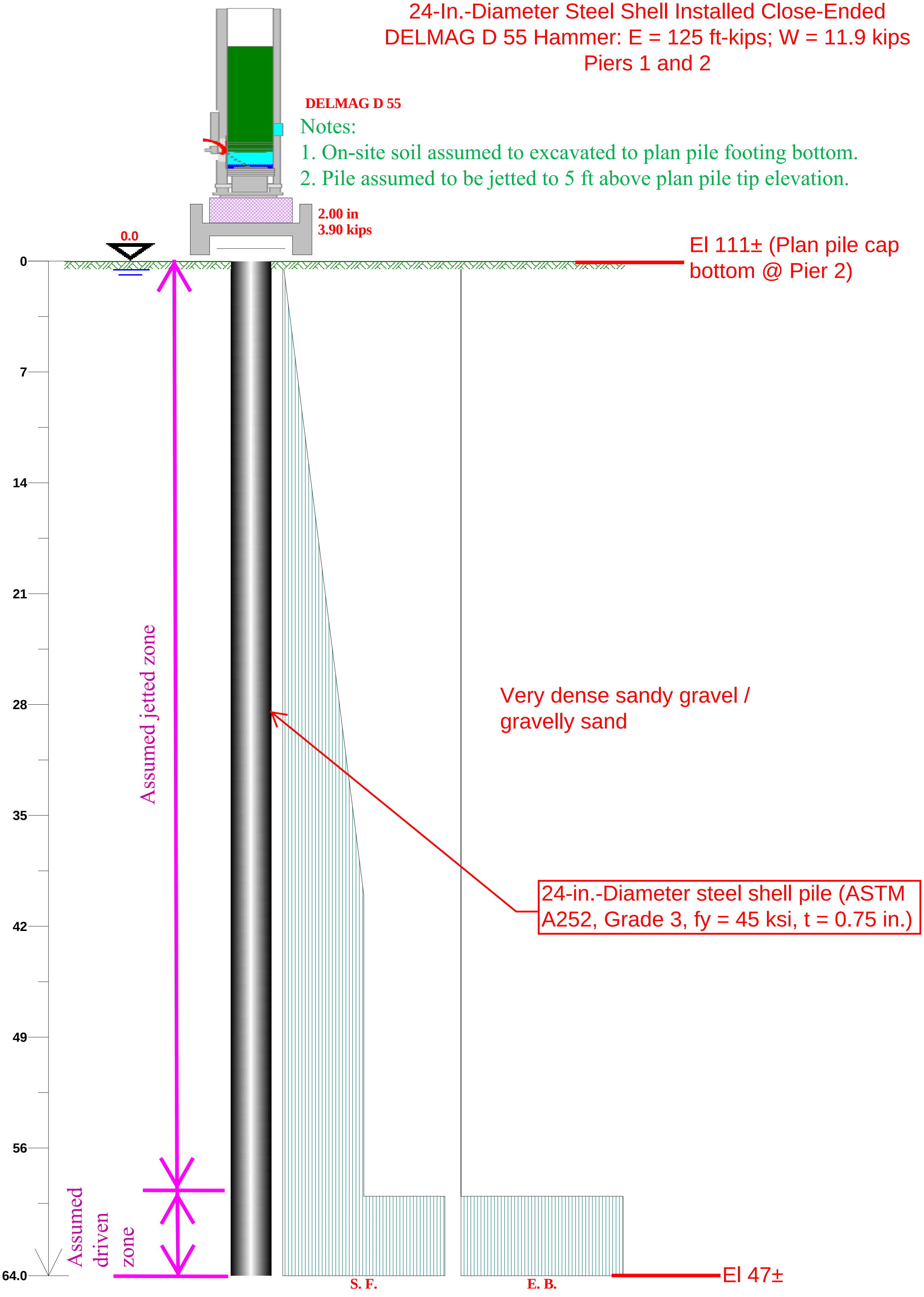
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	9.52	0.0
5.0	7.9	7.9	0.0	0.0	0.000	0.000	9.52	0.0
10.0	15.8	15.8	0.0	0.0	0.000	0.000	9.52	0.0
15.0	25.2	25.2	0.0	1.4	8.624	-1.045	3.34	53.0
20.0	41.0	41.0	0.0	1.8	12.445	-3.533	3.68	53.4
25.0	56.4	56.4	0.0	2.2	14.281	-4.554	3.91	52.5
30.0	73.8	73.8	0.0	2.7	15.649	-5.347	4.09	50.6
35.0	95.0	95.0	0.0	3.5	16.870	-5.858	4.27	48.5
40.0	119.8	119.8	0.0	4.3	17.973	-6.312	4.50	47.2
45.0	148.3	148.3	0.0	5.3	18.841	-6.498	4.71	45.9
50.0	180.5	180.5	0.0	6.5	19.388	-6.865	4.87	44.1
54.0	1120.1	216.1	904.0	153.0	30.883	-1.190	7.92	45.0
55.0	1134.6	230.6	904.0	159.0	30.767	-1.166	7.93	44.8
56.0	1149.1	245.1	904.0	163.6	30.703	-1.122	7.94	44.7
57.0	1163.6	259.6	904.0	169.8	30.470	-1.122	7.93	44.5
58.0	1178.1	274.1	904.0	176.7	30.207	-1.053	7.93	44.3

Total Continuous Driving Time 25.00 minutes; Total Number of Blows 1098

Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 46-13 Hammer: E = 96.5 ft-kips; W = 10.1 kips
Bents 9 and 10

Model for Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips
Piers 1 and 2

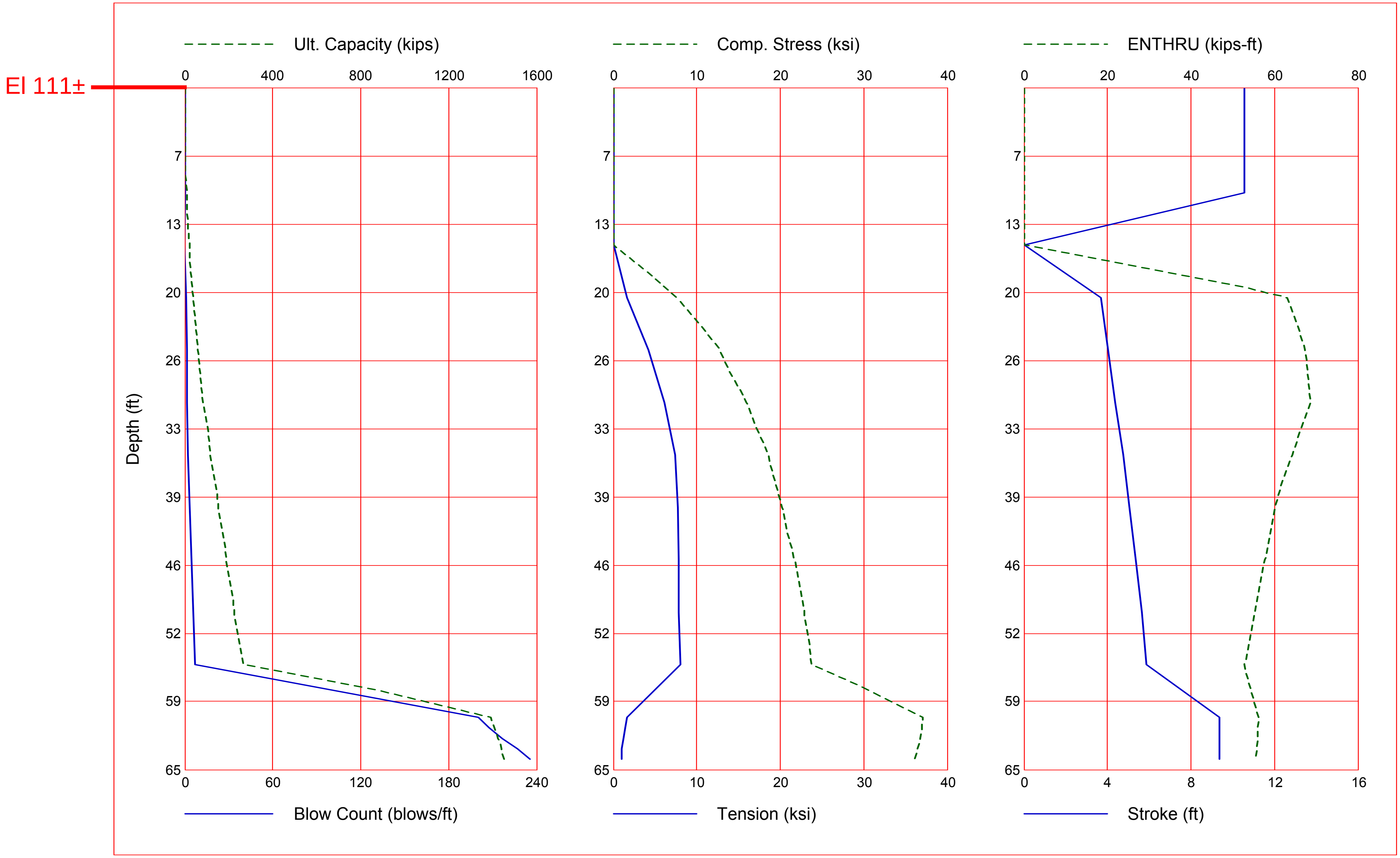


Notes:
1. On-site soil assumed to excavated to plan pile footing bottom.
2. Pile assumed to be jetted to 5 ft above plan pile tip elevation.

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Pier 2_24 in.

Jan 27 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips
Bents 9 and 10 and Piers 1 and 2

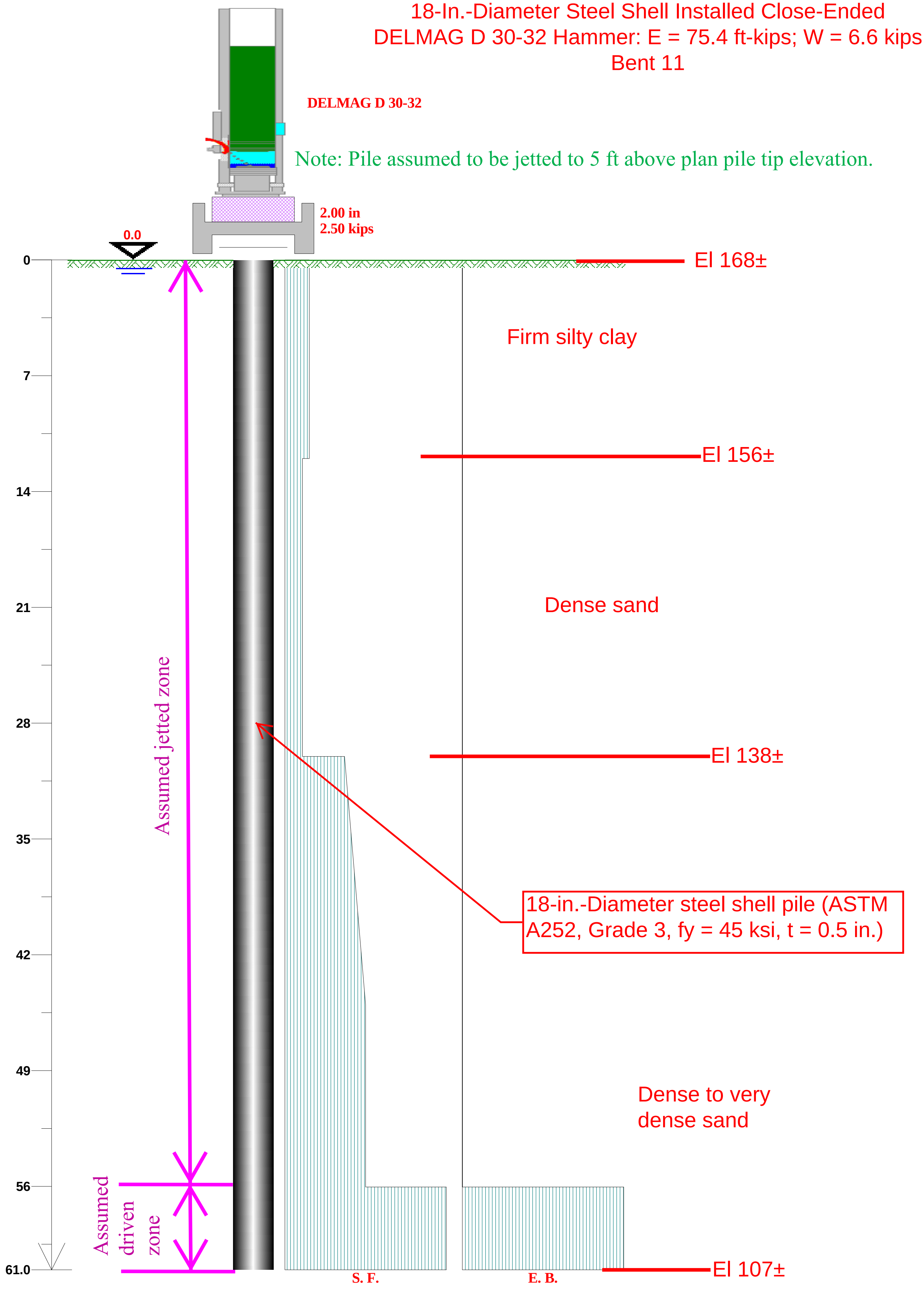
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.0	0.0	0.000	0.000	10.54	0.0
5.0	2.4	2.4	0.0	0.0	0.000	0.000	10.54	0.0
10.0	9.4	9.4	0.0	0.0	0.000	0.000	10.54	0.0
15.0	21.3	21.3	0.0	-1.0	0.000	0.000	0.00	0.0
20.0	37.8	37.8	0.0	1.1	7.624	-1.690	3.71	63.0
25.0	59.1	59.1	0.0	1.3	12.736	-4.235	4.02	67.2
30.0	85.0	85.0	0.0	1.6	15.973	-6.082	4.36	68.5
35.0	115.8	115.8	0.0	2.3	18.580	-7.433	4.74	64.3
40.0	151.2	151.2	0.0	3.3	20.172	-7.698	5.04	60.0
45.0	189.0	189.0	0.0	4.4	21.679	-7.833	5.36	57.6
50.0	226.8	226.8	0.0	5.6	22.928	-7.833	5.66	55.4
55.0	264.6	264.6	0.0	6.8	23.764	-8.002	5.87	52.9
60.0	1392.9	310.0	1082.9	200.4	37.103	-1.614	9.38	56.2
61.0	1408.0	325.1	1082.9	207.6	36.982	-1.391	9.38	56.1
62.0	1423.1	340.2	1082.9	216.4	36.746	-1.228	9.37	55.9
63.0	1438.2	355.3	1082.9	226.8	36.473	-1.040	9.36	55.7
64.0	1453.4	370.4	1082.9	235.6	36.117	-0.989	9.36	55.6

Total Continuous Driving Time 38.00 minutes; Total Number of Blows 1502

Results of Driveability Analysis
24-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 55 Hammer: E = 125 ft-kips; W = 11.9 kips
Bents 9 and 10 and Piers 1 and 2

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11



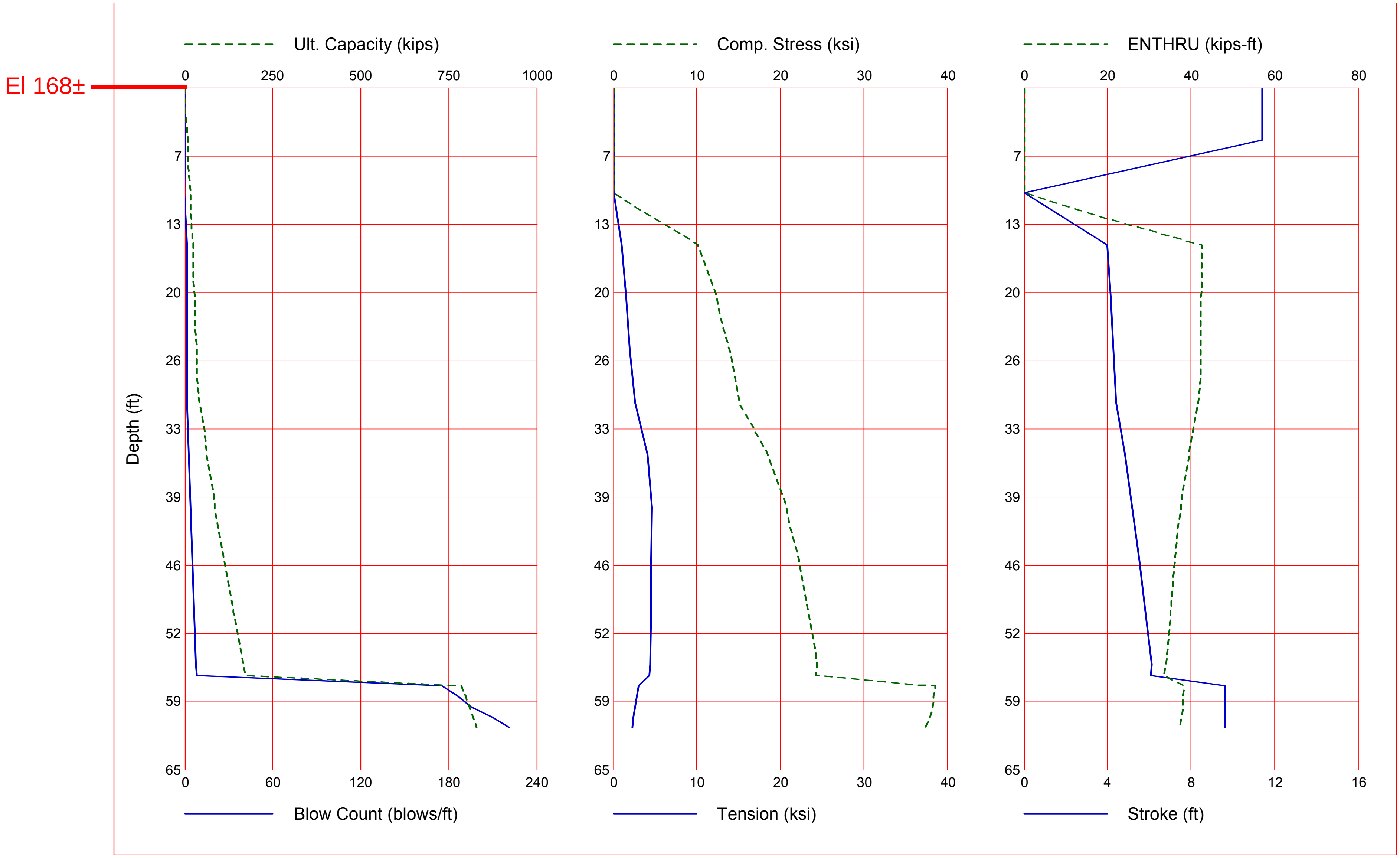
Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

18-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.5 in.)

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 11_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 23 2015
13-017_Bent 11_18 in. GRLWEAP Version 2010

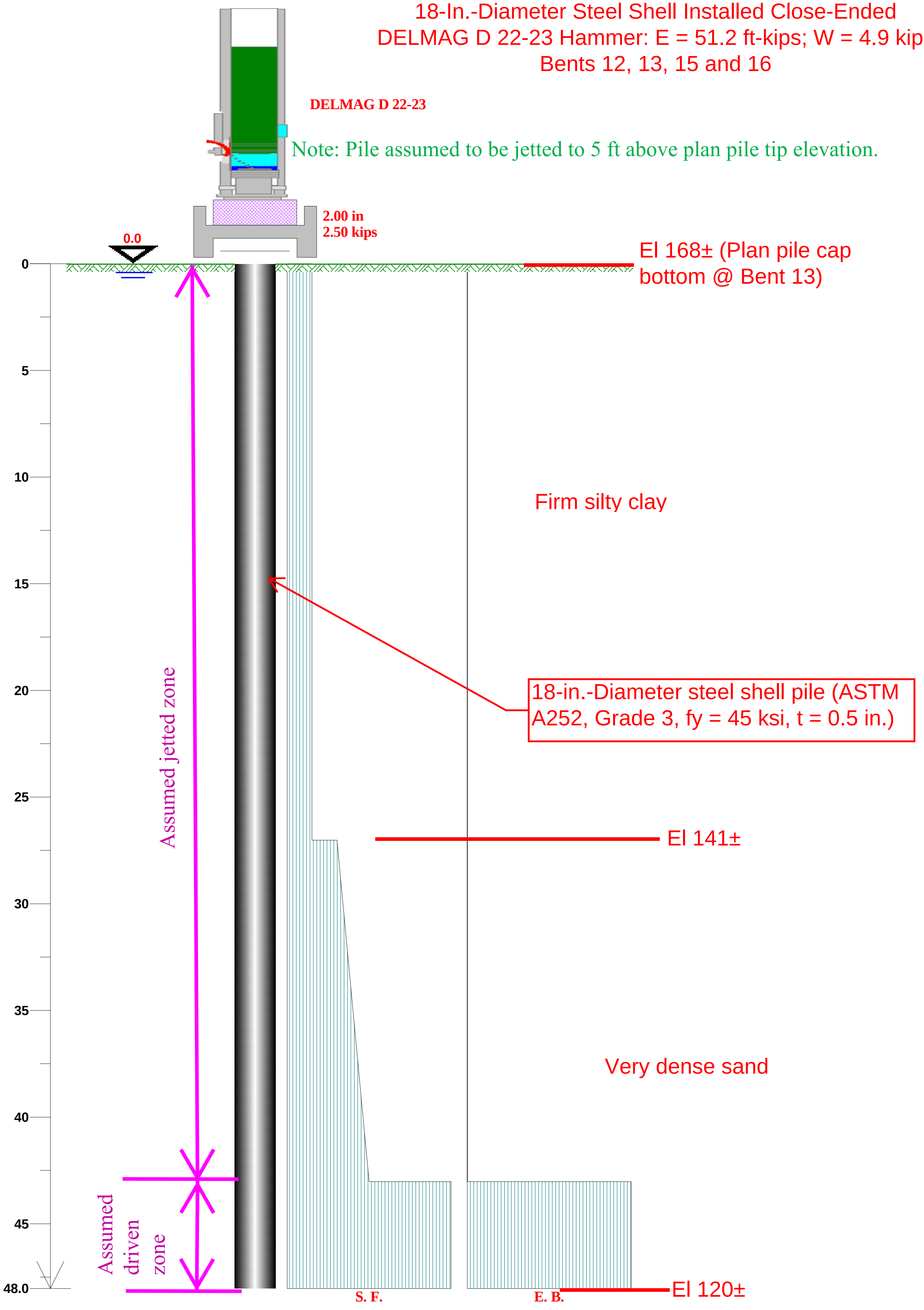
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	11.43	0.0
5.0	8.2	8.2	0.0	0.0	0.000	0.000	11.43	0.0
10.0	16.5	16.5	0.0	-1.0	0.000	0.000	0.00	0.0
15.0	23.3	23.3	0.0	1.4	10.210	-1.019	4.00	42.5
20.0	29.1	29.1	0.0	1.5	12.455	-1.549	4.18	42.3
25.0	35.0	35.0	0.0	1.6	13.895	-1.945	4.30	42.3
30.0	40.9	40.9	0.0	1.8	15.067	-2.619	4.42	41.8
35.0	62.0	62.0	0.0	2.7	18.532	-4.119	4.83	39.6
40.0	85.5	85.5	0.0	3.7	20.787	-4.626	5.19	37.7
45.0	111.4	111.4	0.0	4.9	22.231	-4.506	5.52	36.2
50.0	138.4	138.4	0.0	6.1	23.400	-4.562	5.82	35.1
55.0	165.4	165.4	0.0	7.6	24.346	-4.397	6.11	34.1
56.0	170.8	170.8	0.0	8.0	24.235	-4.269	6.10	33.6
57.0	786.7	181.7	605.0	175.2	38.574	-3.019	9.64	38.3
58.0	797.5	192.5	605.0	186.5	38.344	-2.791	9.63	38.1
59.0	808.3	203.3	605.0	195.6	38.238	-2.630	9.63	38.0
60.0	819.1	214.1	605.0	210.0	37.882	-2.401	9.62	37.6
61.0	829.9	224.9	605.0	221.7	37.439	-2.260	9.61	37.5

Total Continuous Driving Time 26.00 minutes; Total Number of Blows 1028

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30-32 Hammer: E = 75.4 ft-kips; W = 6.6 kips
Bent 11

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16



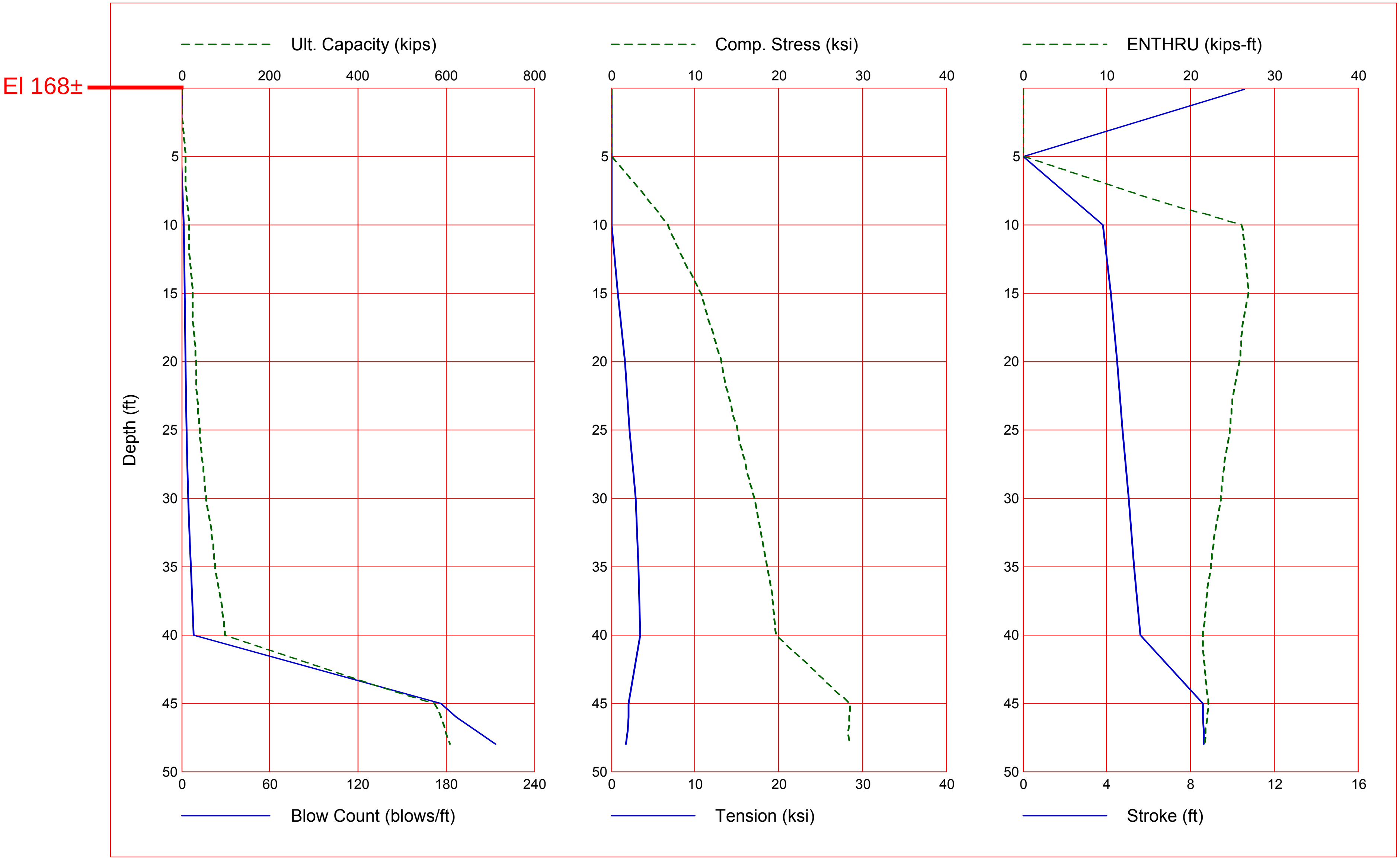
Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

18-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.5 in.)

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 13_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16

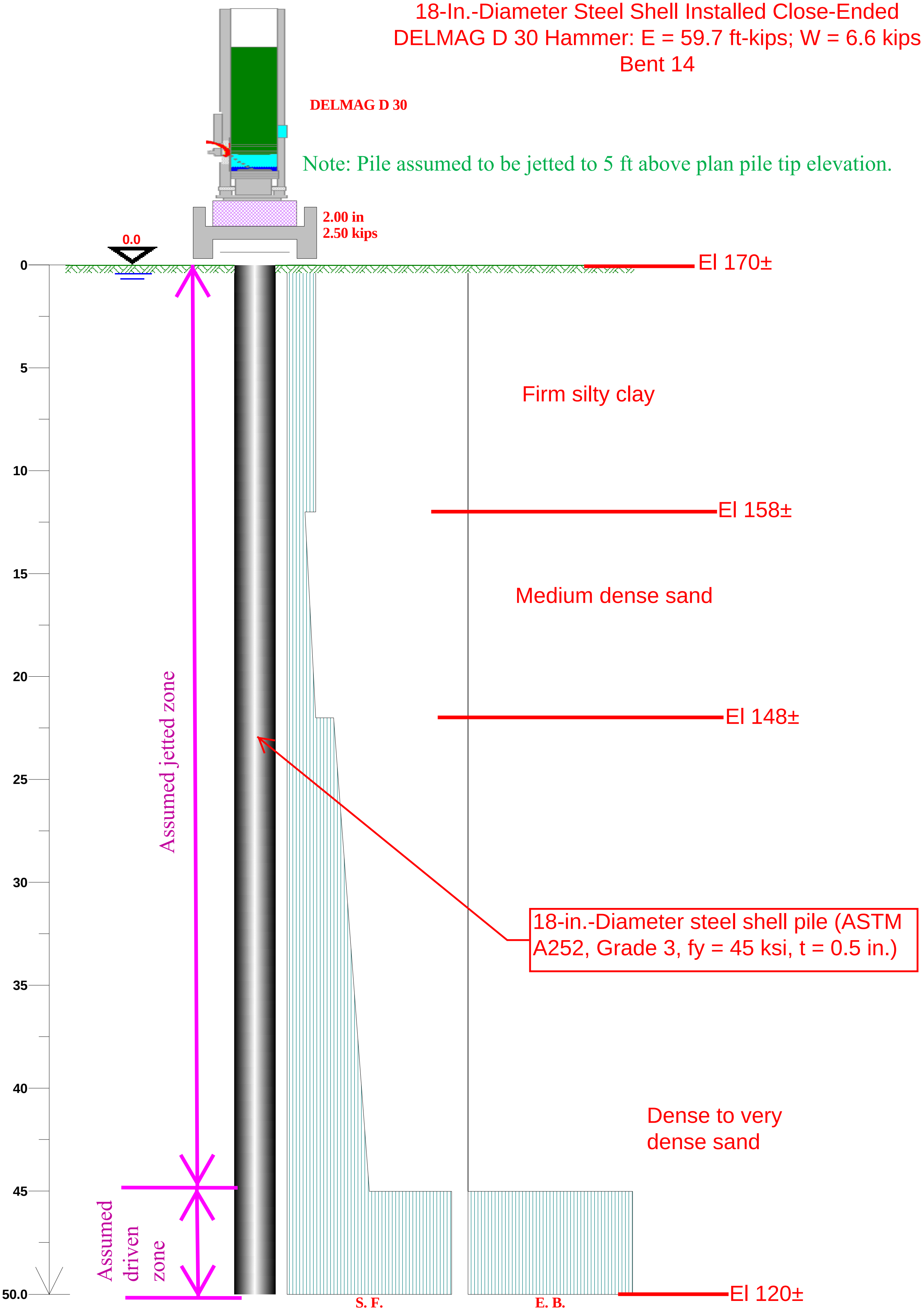
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	10.56	0.0
5.0	8.2	8.2	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	16.4	16.4	0.0	1.7	6.797	0.000	3.82	26.3
15.0	24.7	24.7	0.0	2.1	10.734	-0.771	4.21	26.9
20.0	32.9	32.9	0.0	2.8	13.146	-1.629	4.51	25.9
25.0	41.1	41.1	0.0	3.6	15.053	-2.239	4.77	24.7
30.0	54.9	54.9	0.0	4.7	17.106	-2.913	5.04	23.6
35.0	75.0	75.0	0.0	6.2	18.565	-3.214	5.33	22.5
40.0	98.4	98.4	0.0	8.0	19.702	-3.446	5.62	21.5
45.0	576.6	135.6	441.0	176.4	28.521	-2.075	8.59	22.1
46.0	587.4	146.4	441.0	186.8	28.431	-2.075	8.60	22.0
47.0	598.2	157.2	441.0	200.3	28.356	-1.957	8.62	21.8
48.0	609.0	168.0	441.0	213.6	28.385	-1.812	8.62	21.7

Total Continuous Driving Time 28.00 minutes; Total Number of Blows 1168

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 22-23 Hammer: E = 51.2 ft-kips; W = 4.9 kips
Bents 12, 13, 15 and 16

Model for Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 14



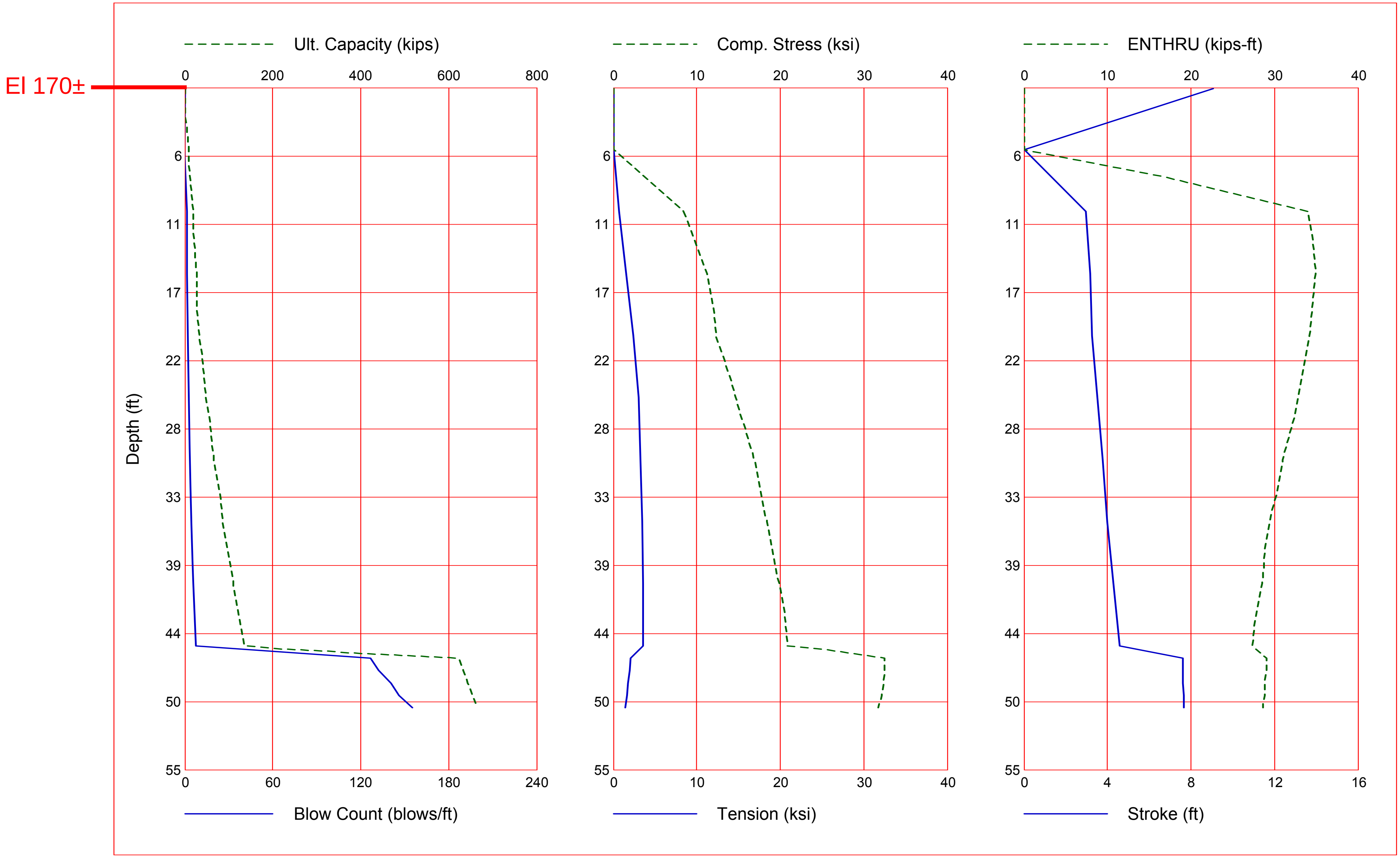
Note: Pile assumed to be jetted to 5 ft above plan pile tip elevation.

18-in.-Diameter steel shell pile (ASTM A252, Grade 3, fy = 45 ksi, t = 0.5 in.)

Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 14_18 in.

Jan 23 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bent 14

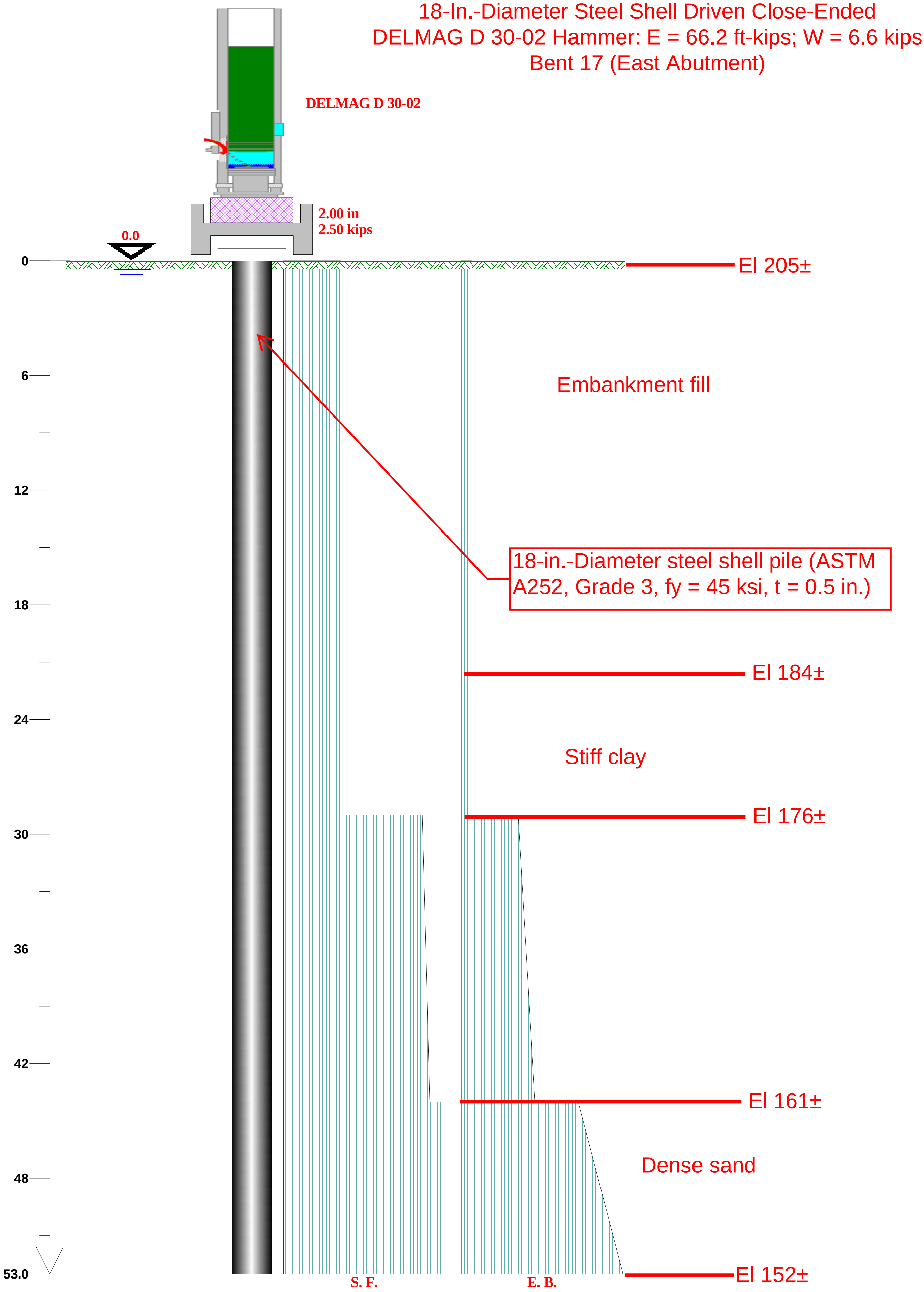
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	0.0	0.0	0.000	0.000	9.05	0.0
5.0	9.4	9.4	0.0	-1.0	0.000	0.000	0.00	0.0
10.0	18.8	18.8	0.0	1.5	8.502	-0.644	2.96	34.1
15.0	26.4	26.4	0.0	1.7	11.242	-1.552	3.16	34.9
20.0	34.2	34.2	0.0	2.0	12.353	-2.388	3.26	34.2
25.0	47.5	47.5	0.0	2.6	14.700	-3.021	3.52	32.8
30.0	65.5	65.5	0.0	3.5	16.959	-3.247	3.76	31.0
35.0	86.2	86.2	0.0	4.7	18.403	-3.439	4.00	29.4
40.0	109.4	109.4	0.0	6.0	19.897	-3.583	4.30	28.5
45.0	135.1	135.1	0.0	7.6	20.810	-3.594	4.57	27.4
46.0	622.2	145.9	476.2	126.9	32.521	-2.071	7.60	29.1
47.0	633.0	156.7	476.2	132.4	32.463	-1.936	7.62	29.1
48.0	643.8	167.6	476.2	140.4	32.390	-1.796	7.63	28.9
49.0	654.6	178.4	476.2	146.1	32.128	-1.692	7.64	28.9
50.0	665.4	189.2	476.2	155.1	31.766	-1.437	7.65	28.6

Total Continuous Driving Time 17.00 minutes; Total Number of Blows 756

Results of Driveability Analysis
18-In.-Diameter Steel Shell Installed Close-Ended
DELMAG D 30 Hammer: E = 59.7 ft-kips; W = 6.6 kips
Bents 14

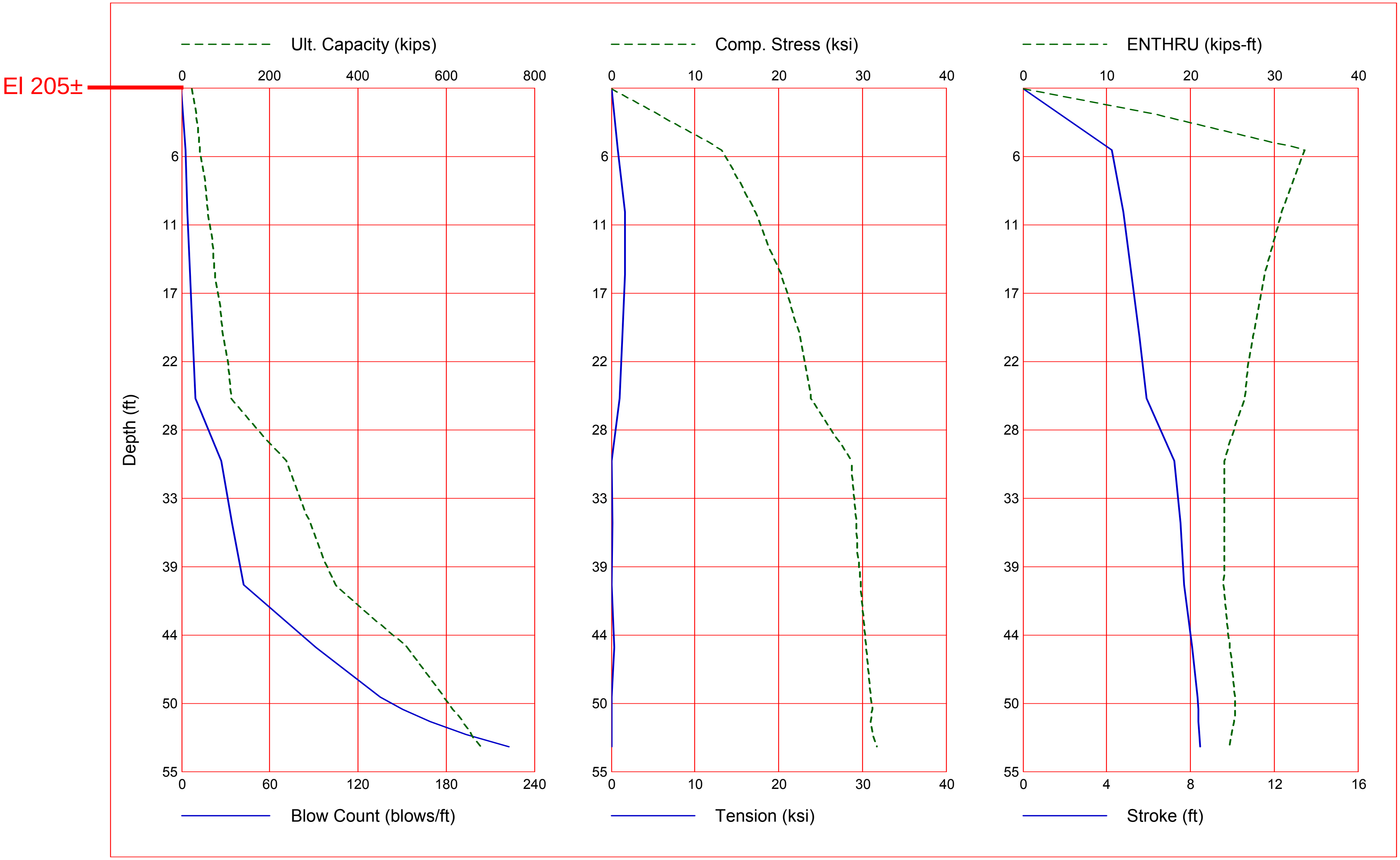
Model for Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17 (East Abutment)



Grubbs, Hoskyn, Barton & Wyatt, Inc.
13-017_Bent 17_18 In.

Jan 29 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17

Grubbs, Hoskyn, Barton & Wyatt, Inc. Jan 29 2015
13-017_Bent 17_18 In. GRLWEAP Version 2010

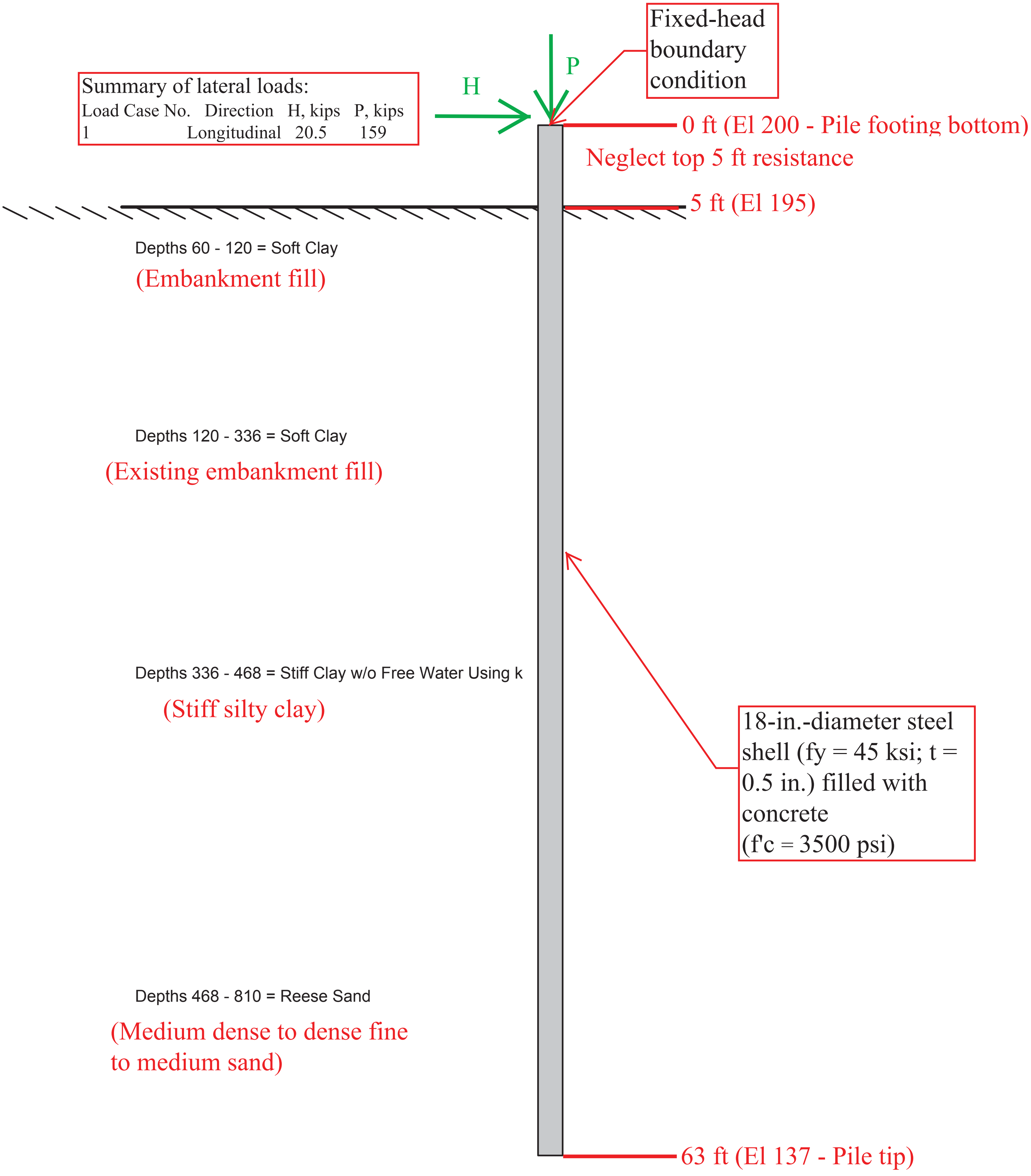
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	24.2	0.4	23.8	-1.0	0.000	0.000	0.00	0.0
5.0	41.4	17.6	23.8	2.5	13.192	-0.818	4.23	33.6
10.0	59.1	35.2	23.8	4.1	17.272	-1.613	4.78	30.9
15.0	76.7	52.9	23.8	5.8	20.335	-1.644	5.20	28.9
20.0	94.3	70.5	23.8	7.6	22.550	-1.284	5.57	27.5
25.0	111.9	88.1	23.8	9.6	23.794	-1.017	5.90	26.5
30.0	238.4	110.7	127.7	27.3	28.710	0.000	7.23	24.1
35.0	293.6	153.5	140.1	34.4	29.254	-0.155	7.53	24.0
40.0	349.6	197.2	152.4	42.2	29.855	0.000	7.70	23.9
45.0	511.0	242.5	268.5	91.5	30.405	-0.360	8.08	24.7
49.0	594.4	282.0	312.4	135.0	30.962	0.000	8.33	25.3
50.0	615.2	291.9	323.4	150.2	31.185	0.000	8.38	25.3
51.0	636.1	301.7	334.4	169.4	31.039	0.000	8.40	25.2
52.0	656.9	311.6	345.3	193.7	31.233	0.000	8.42	24.9
53.0	677.8	321.5	356.3	222.7	31.687	0.000	8.45	24.7

Total Continuous Driving Time 49.00 minutes; Total Number of Blows 2040

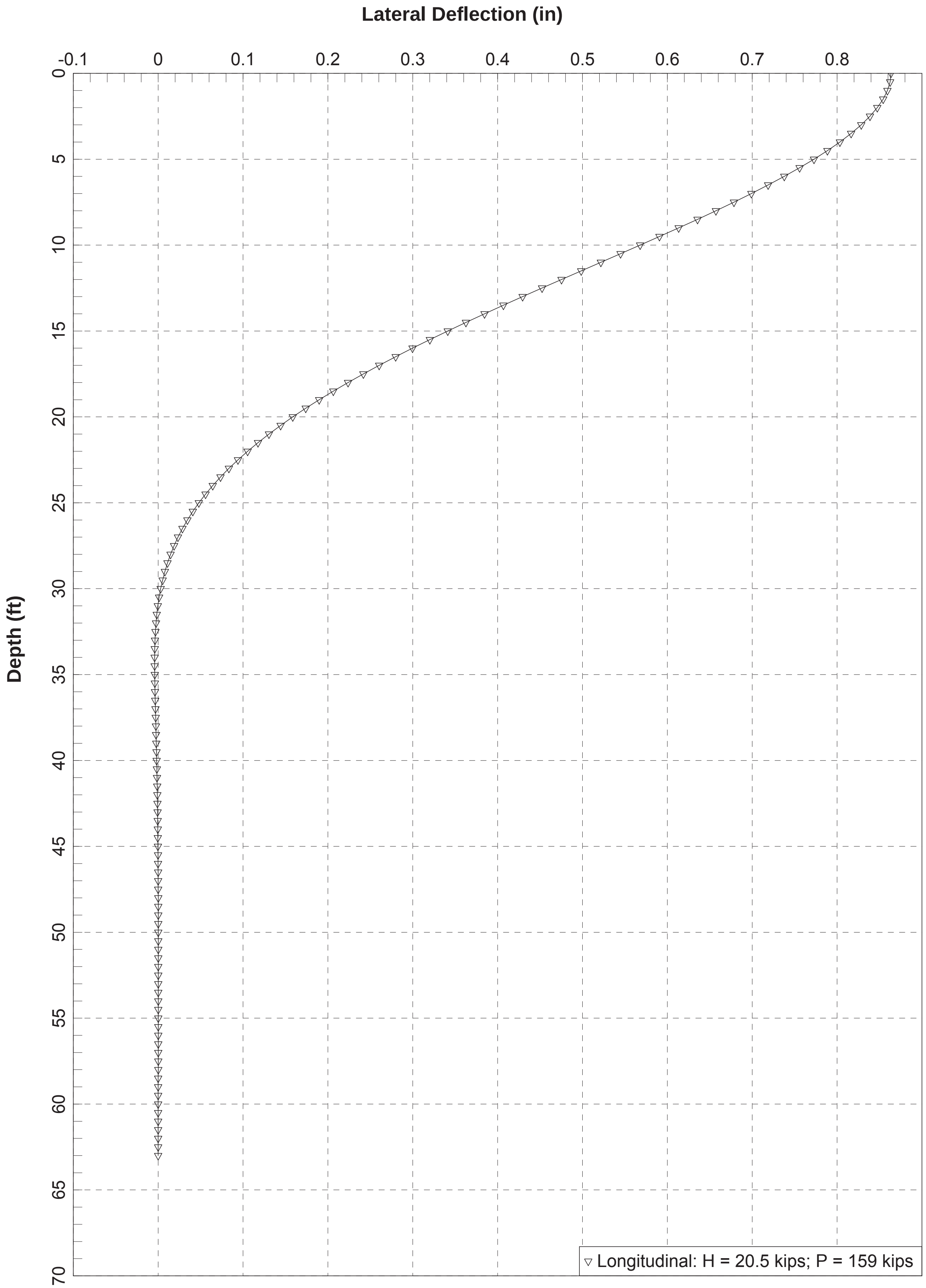
Results of Driveability Analysis
18-In.-Diameter Steel Shell Driven Close-Ended
DELMAG D 30-02 Hammer: E = 66.2 ft-kips; W = 6.6 kips
Bent 17

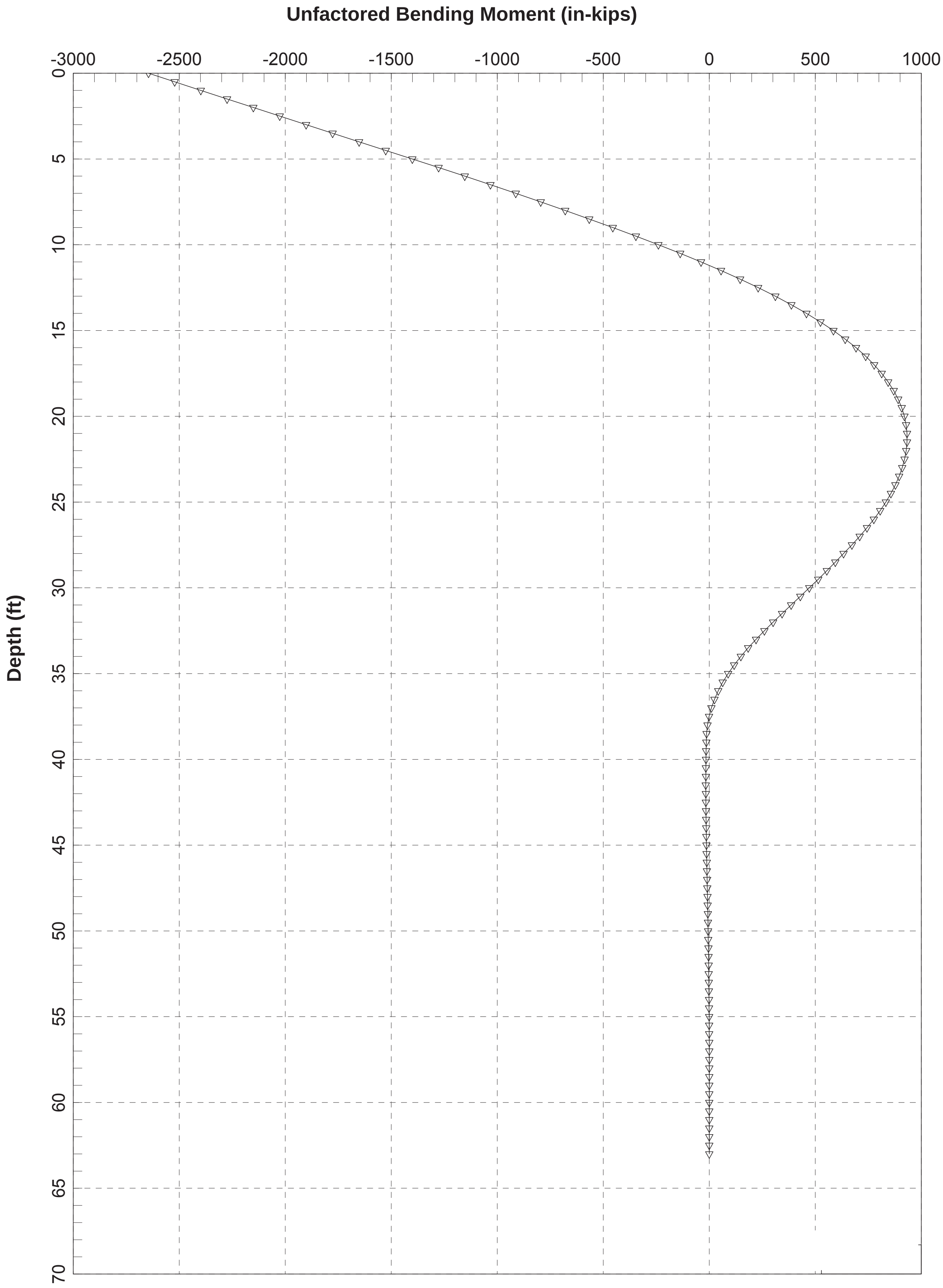
APPENDIX H

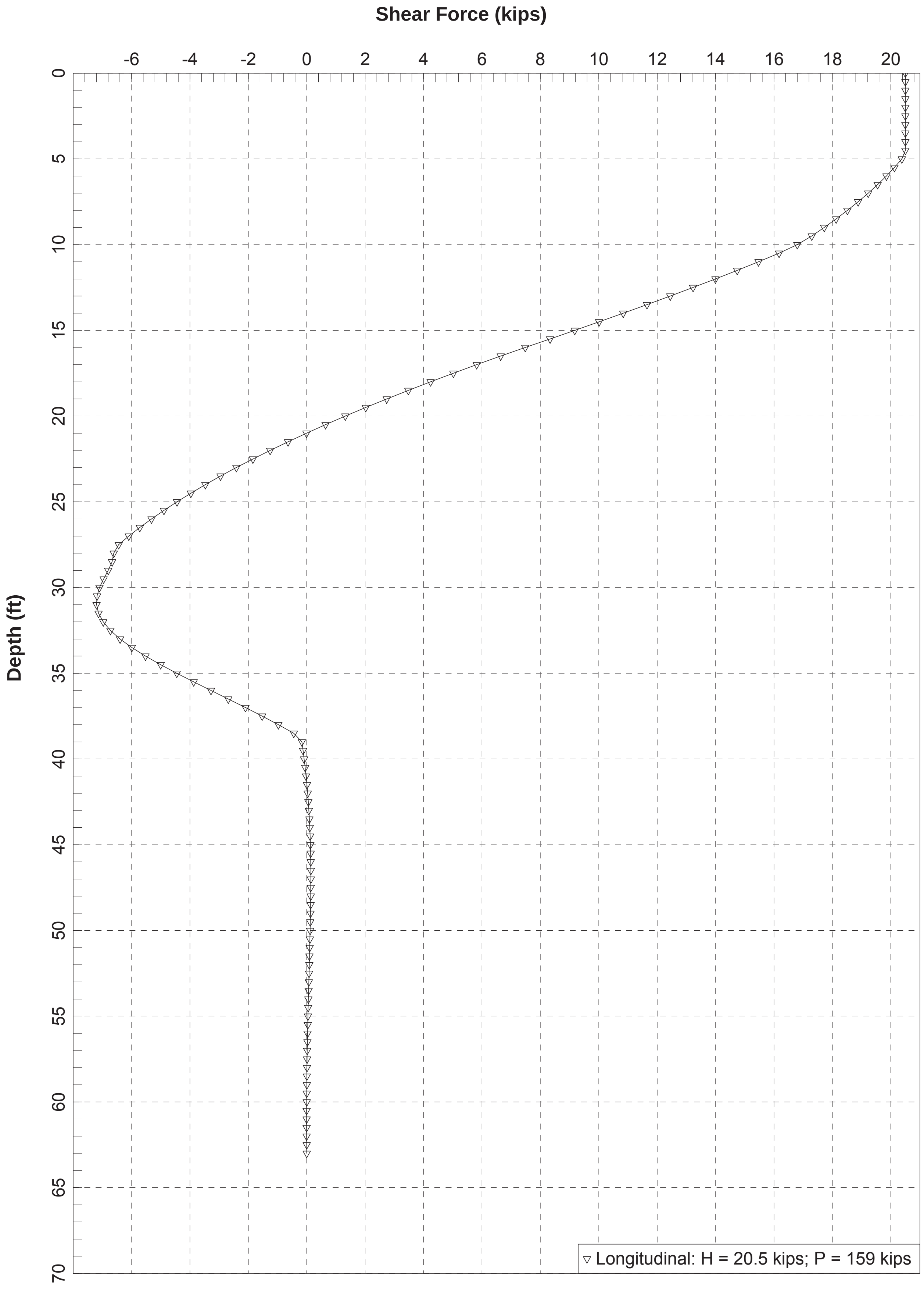


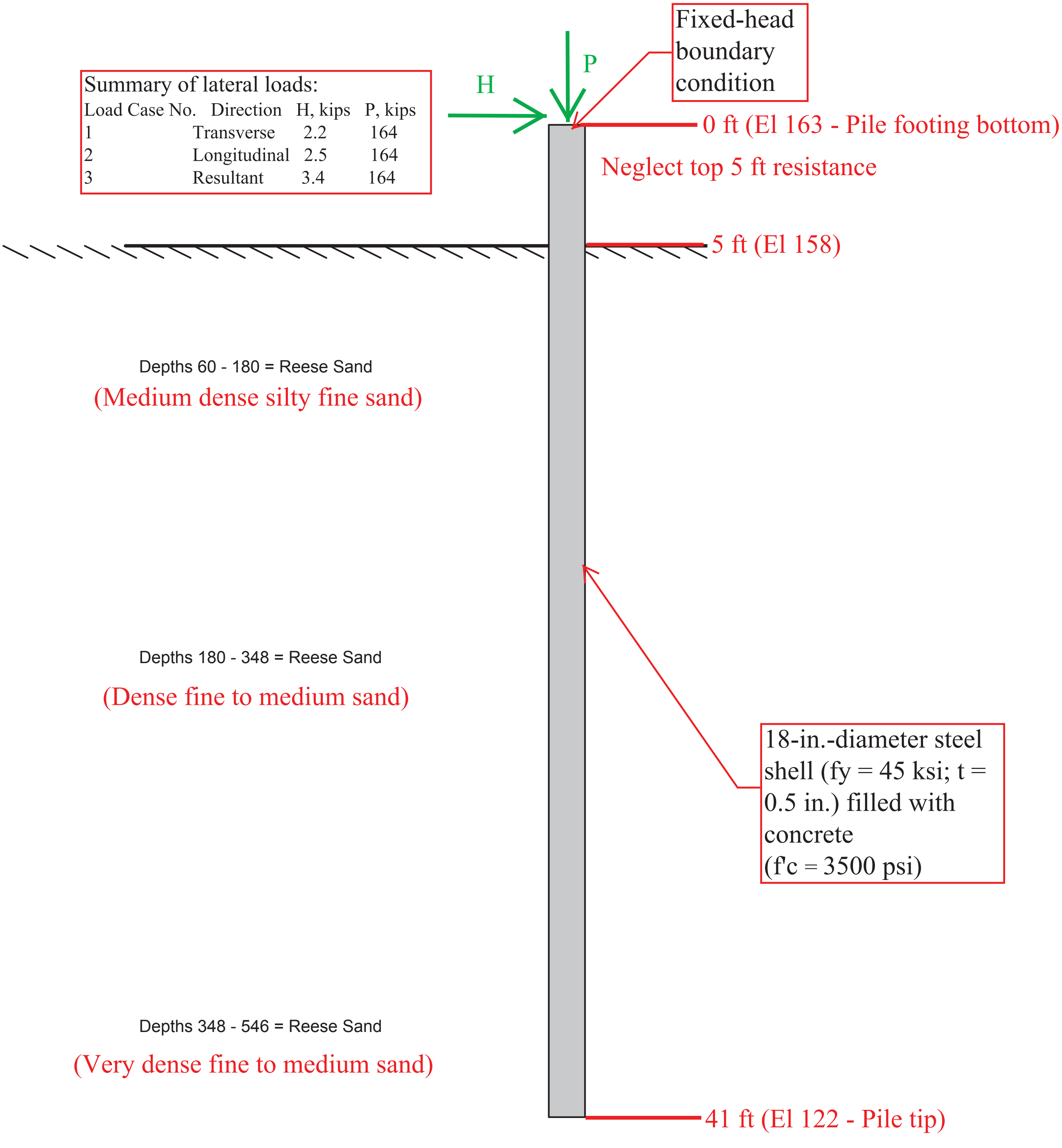
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 1 (West Abutment)
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Longitudinal: H = 20.5 kips; P = 159 kips


Unfactored Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70


Shear Force (kips)
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
20
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Longitudinal: H = 20.5 kips; P = 159 kips

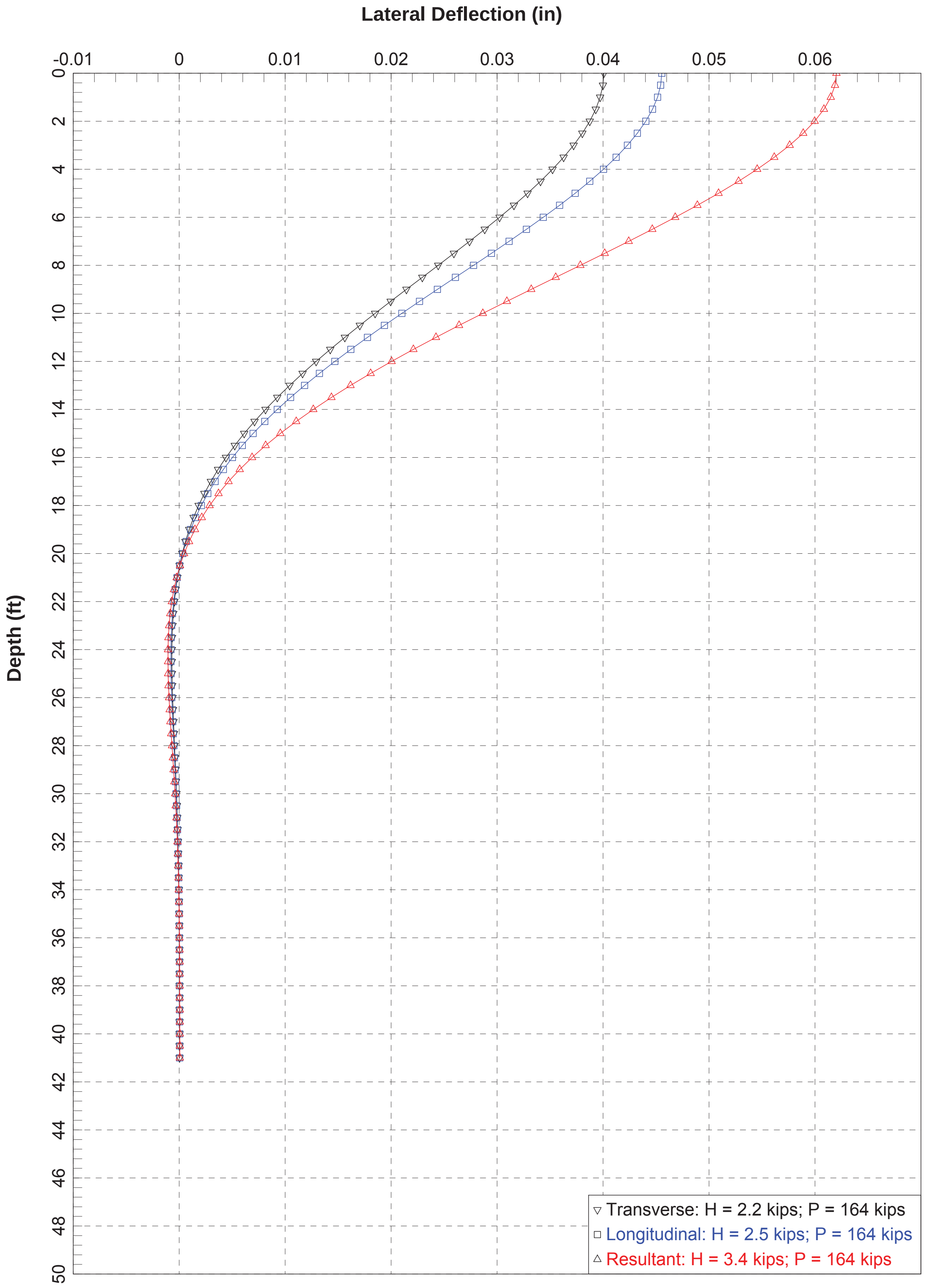


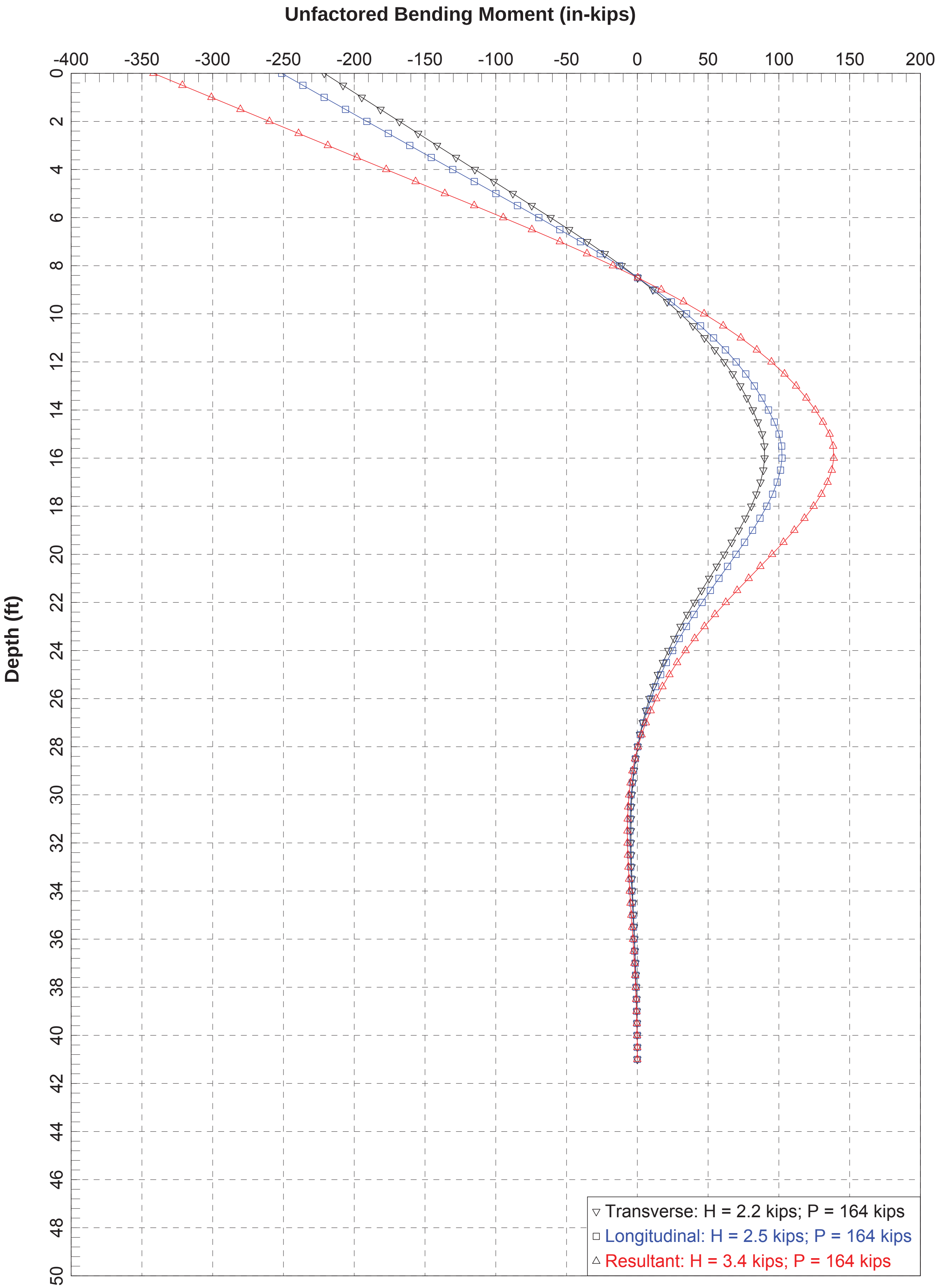
Summary of lateral loads:

Load Case No.	Direction	H, kips	P, kips
1	Transverse	2.2	164
2	Longitudinal	2.5	164
3	Resultant	3.4	164

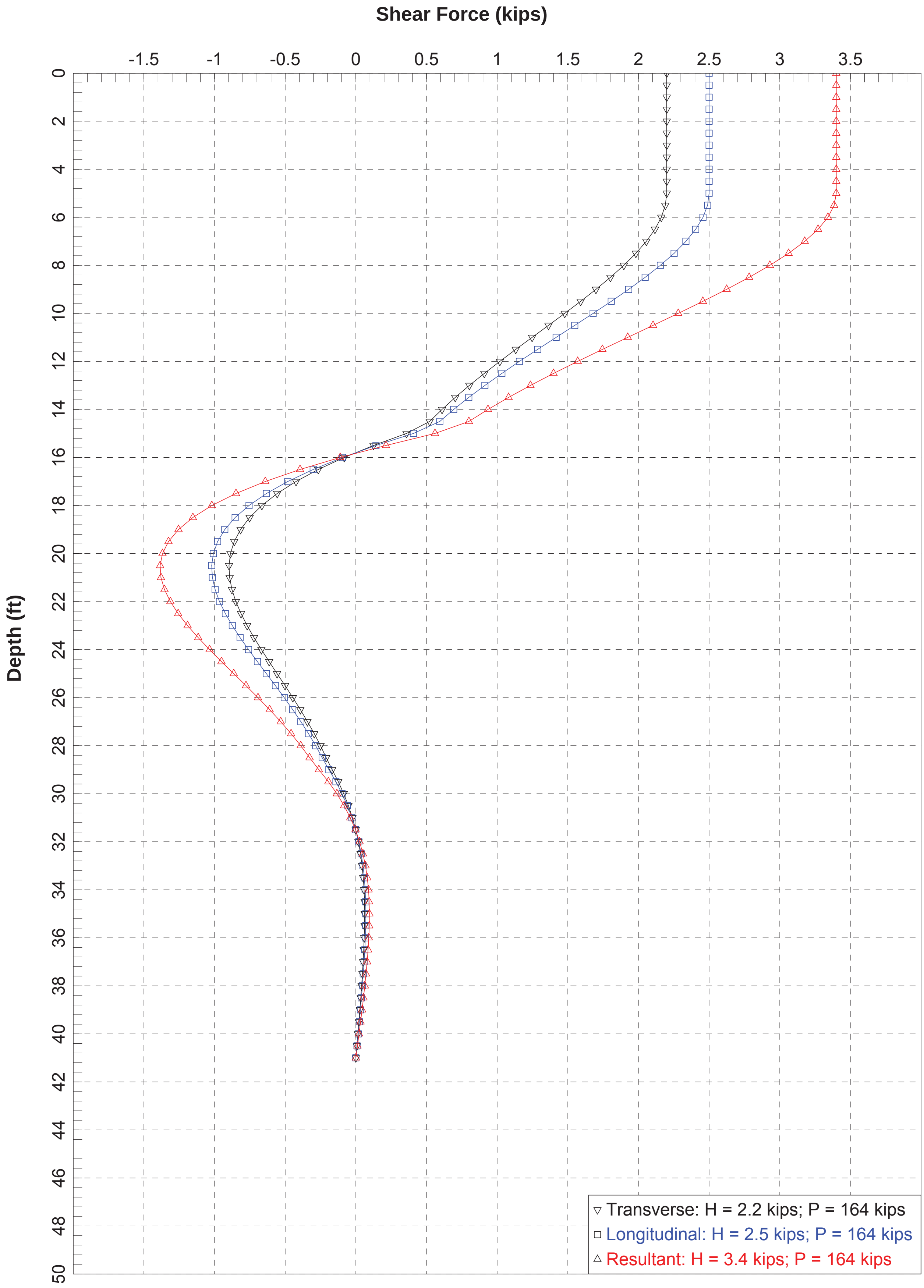
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 2
I-40 over White River

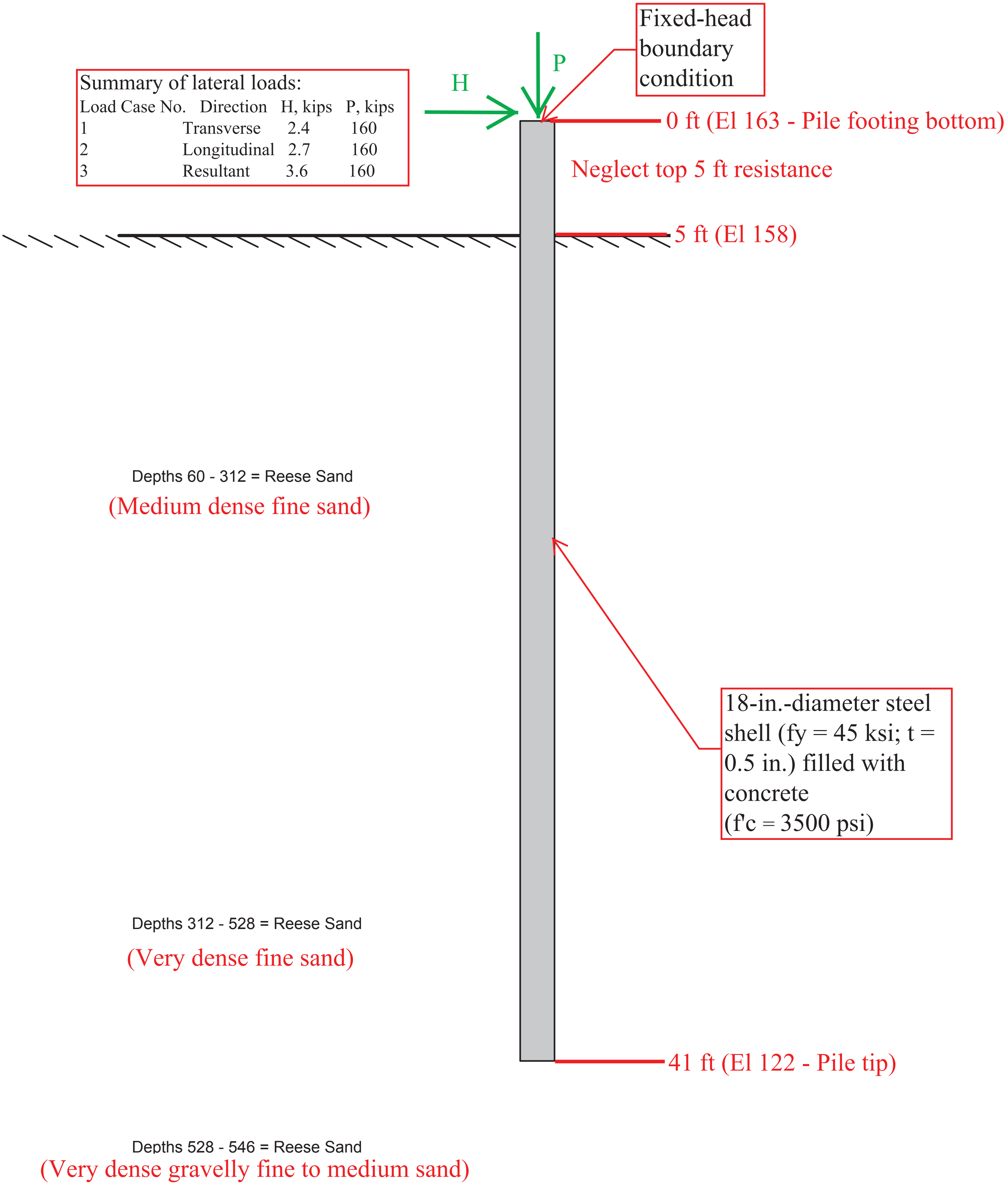
LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.01
0
0.01
0.02
0.03
0.04
0.05
0.06
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips



Unfactored Bending Moment (in-kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips


Shear Force (kips)
-1.5
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.2 kips; P = 164 kips
□ Longitudinal: H = 2.5 kips; P = 164 kips
△ Resultant: H = 3.4 kips; P = 164 kips

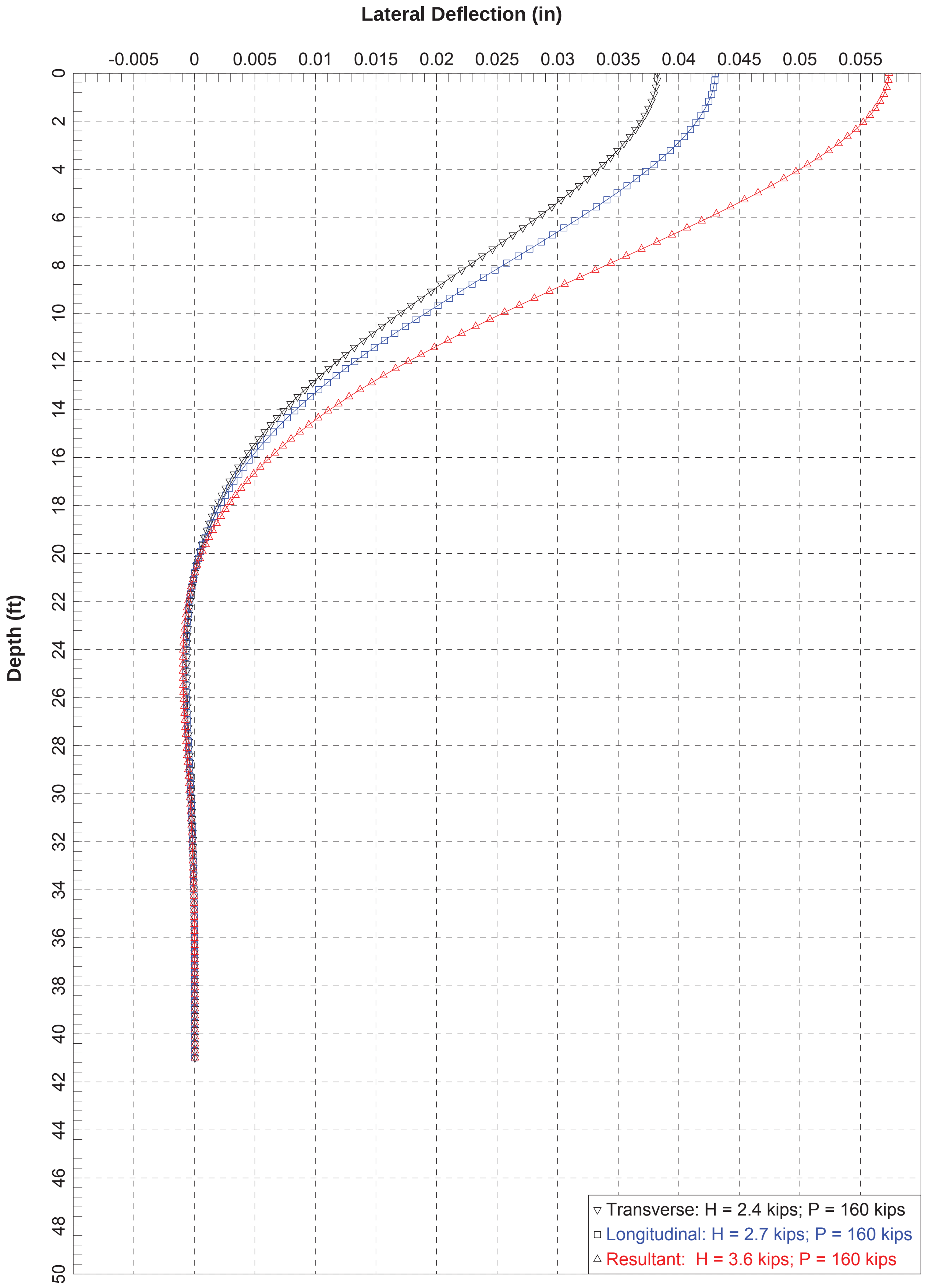


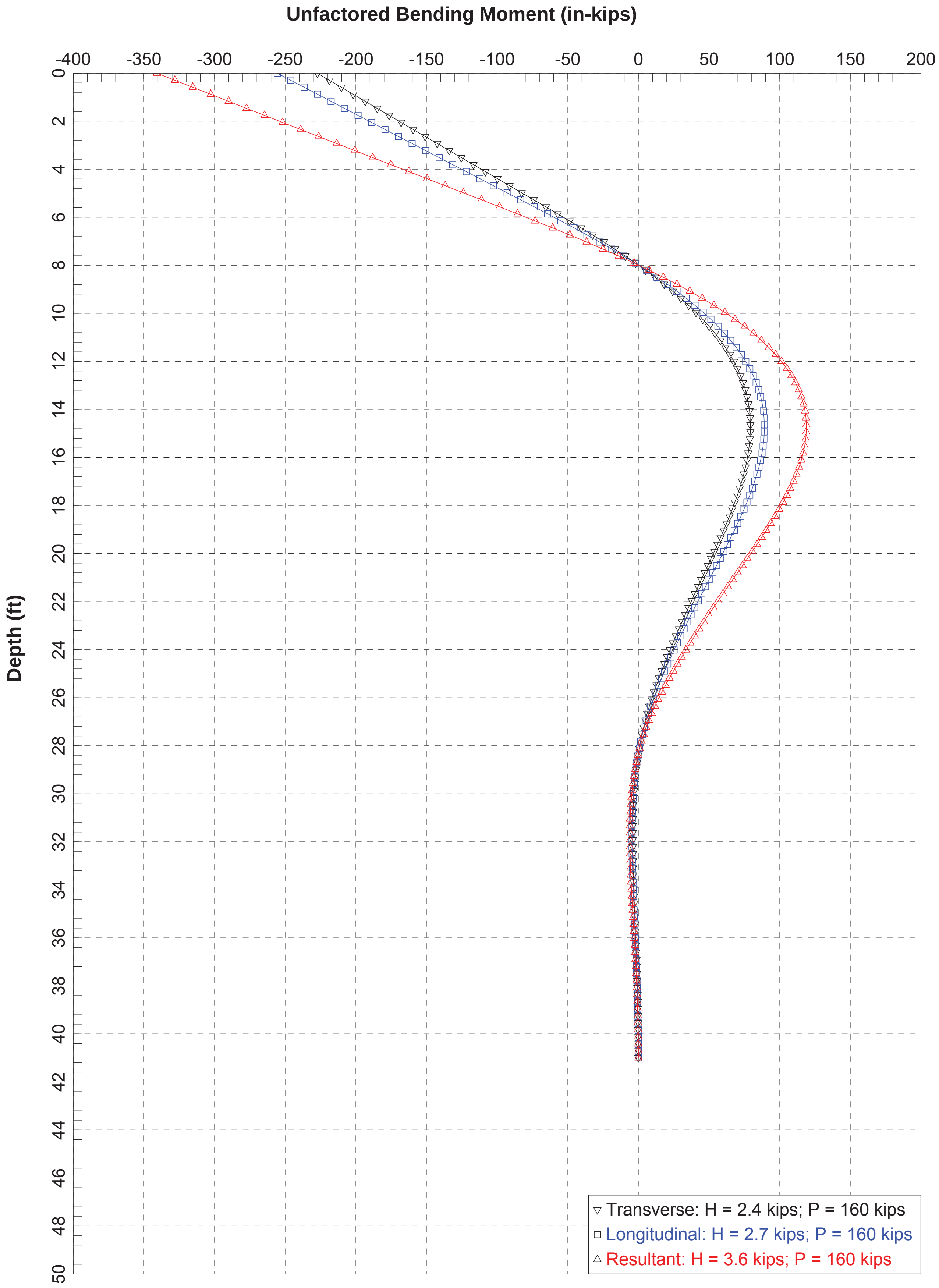
Summary of lateral loads:

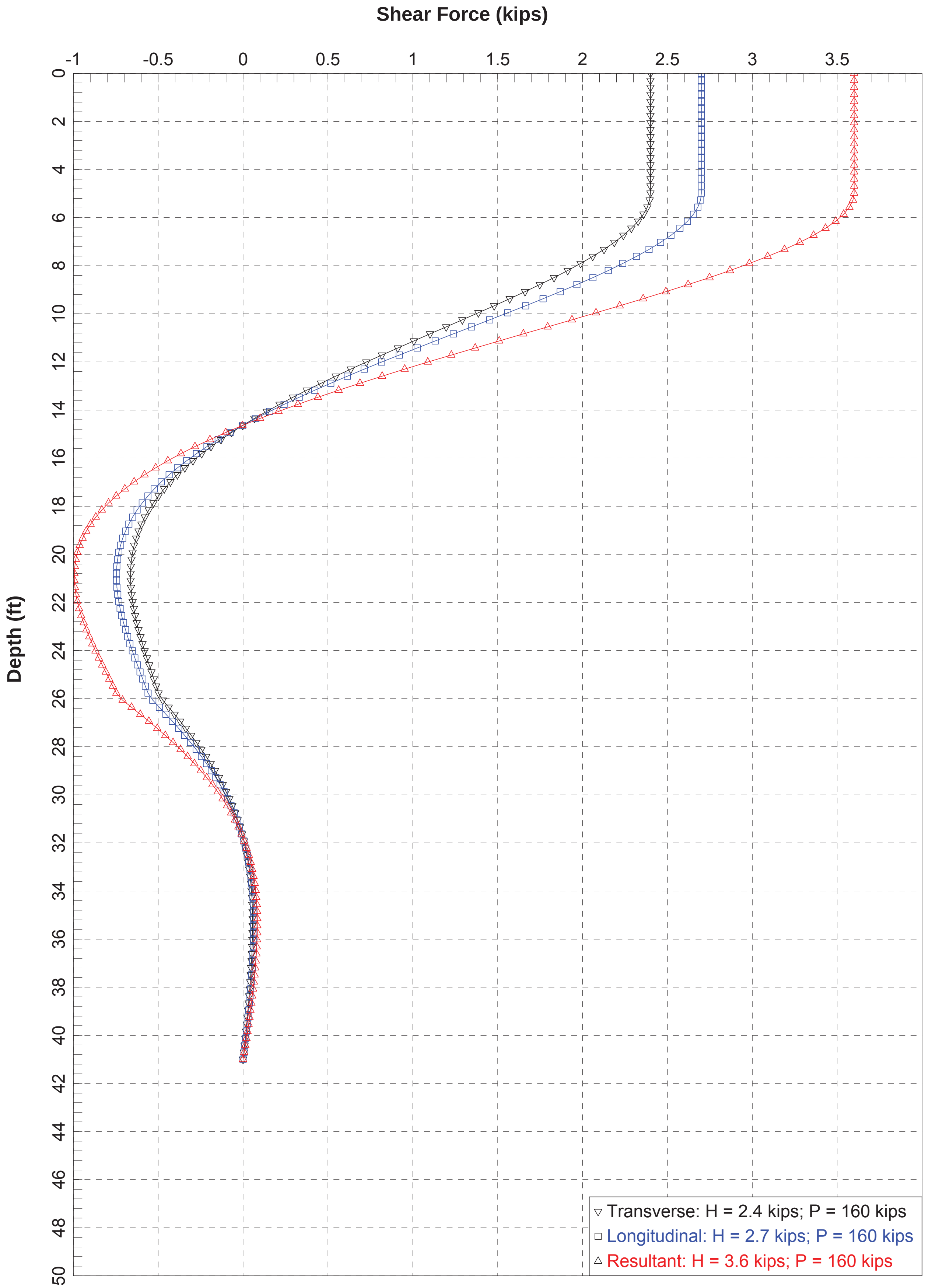
Load Case No.	Direction	H, kips	P, kips
1	Transverse	2.4	160
2	Longitudinal	2.7	160
3	Resultant	3.6	160

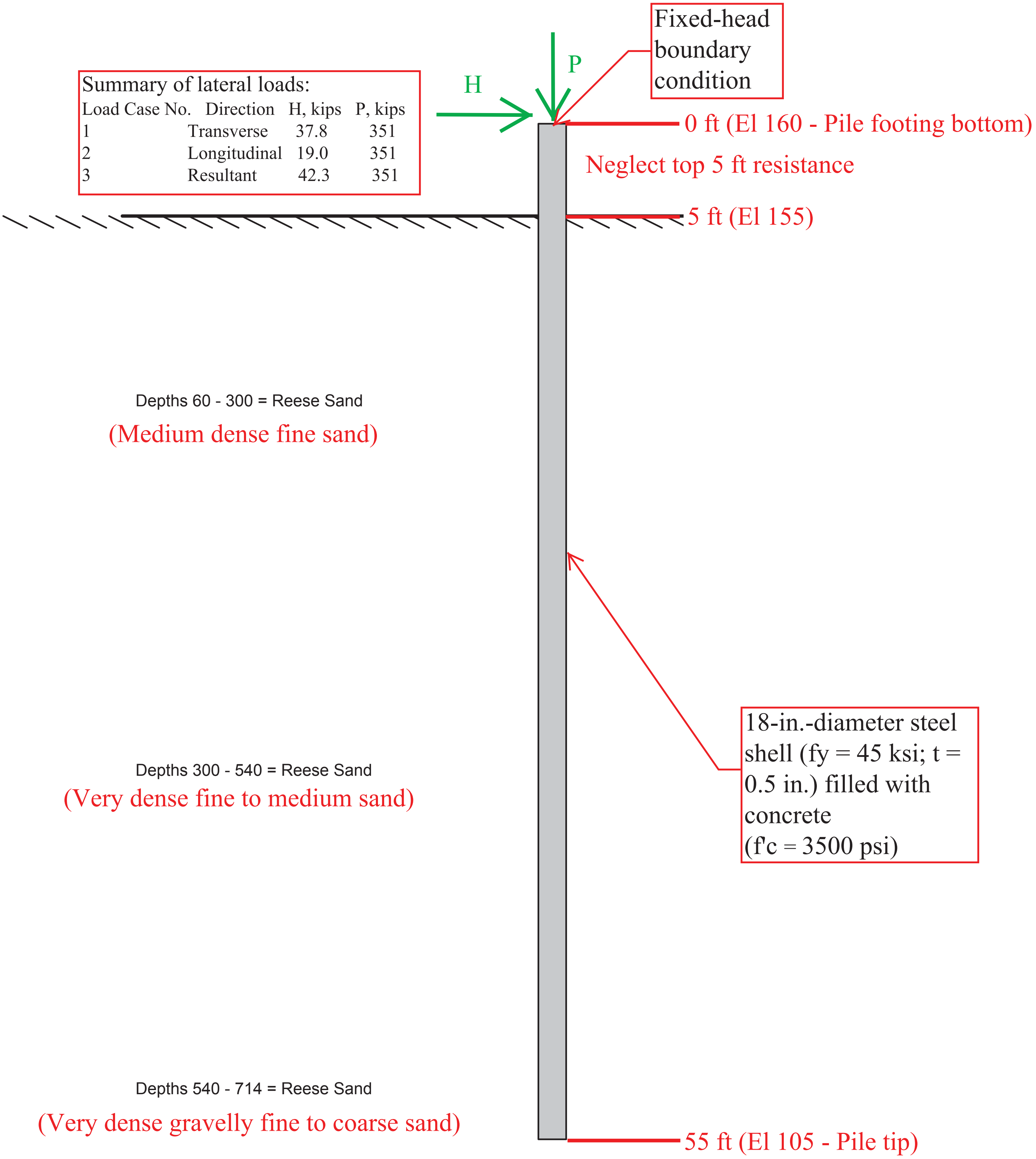
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 3
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
0.04
0.045
0.05
0.055
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips


Unfactored Bending Moment (in-kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
150
200
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips


Shear Force (kips)
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
▽ Transverse: H = 2.4 kips; P = 160 kips
□ Longitudinal: H = 2.7 kips; P = 160 kips
△ Resultant: H = 3.6 kips; P = 160 kips

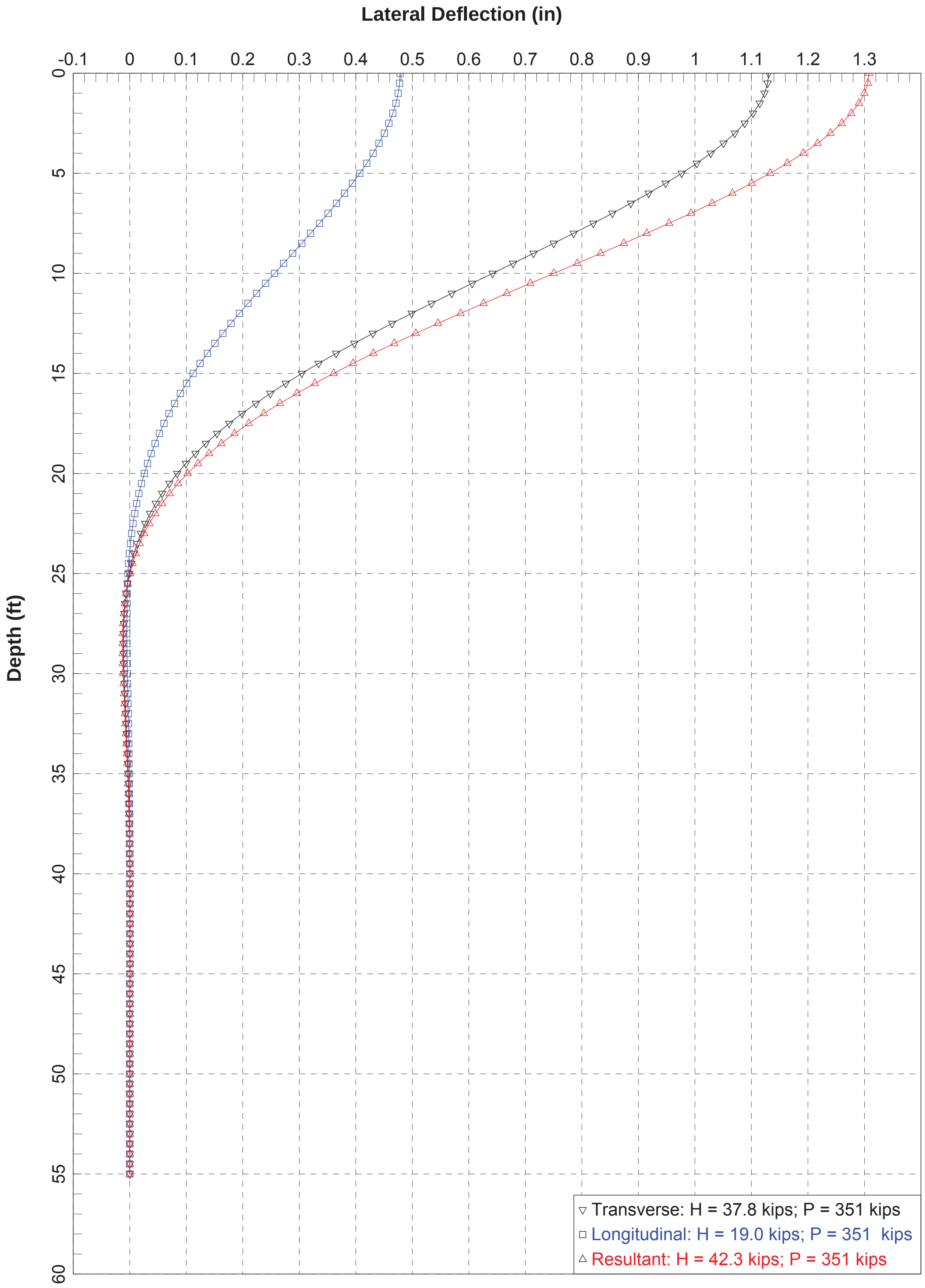


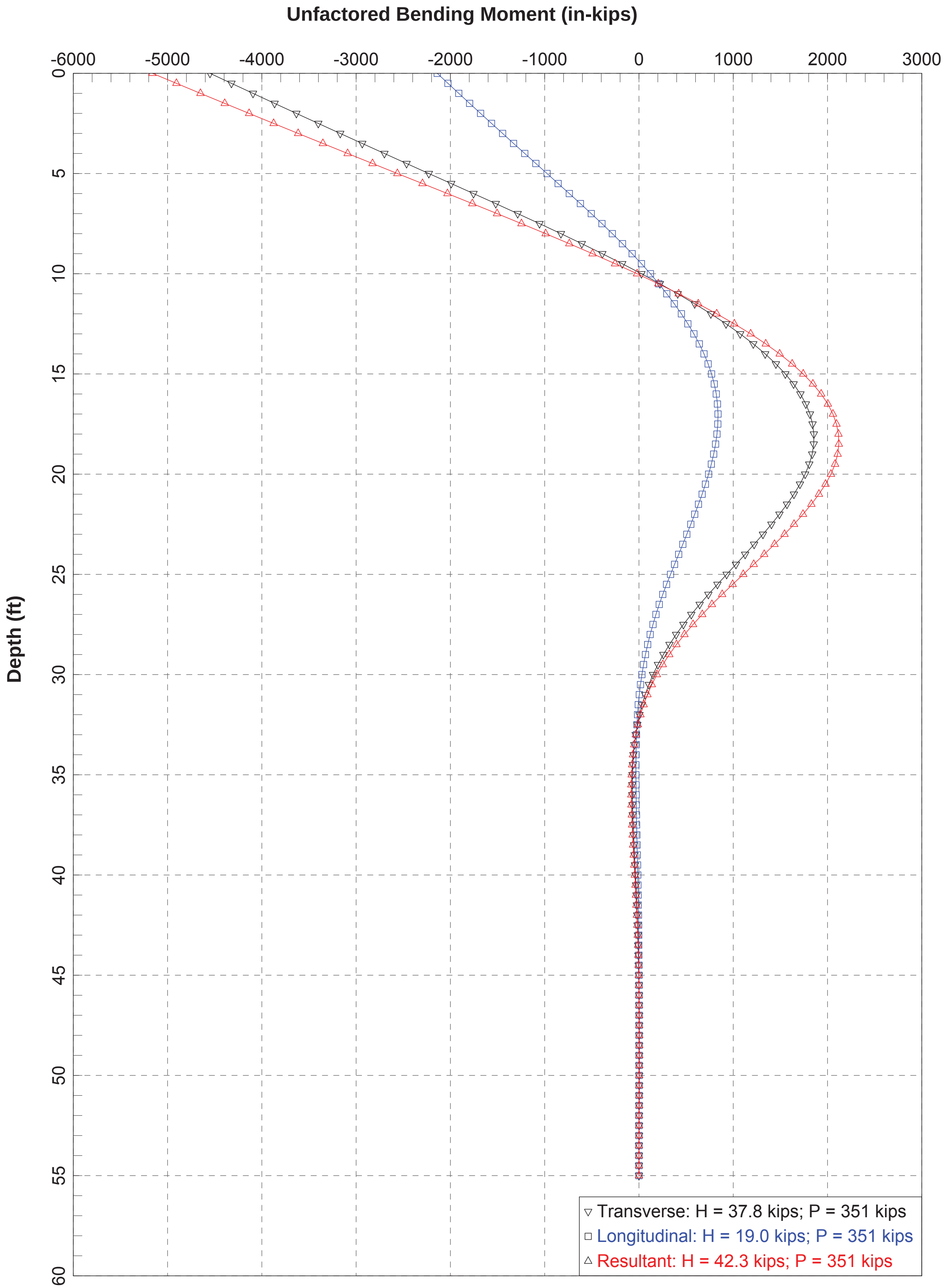
Summary of lateral loads:

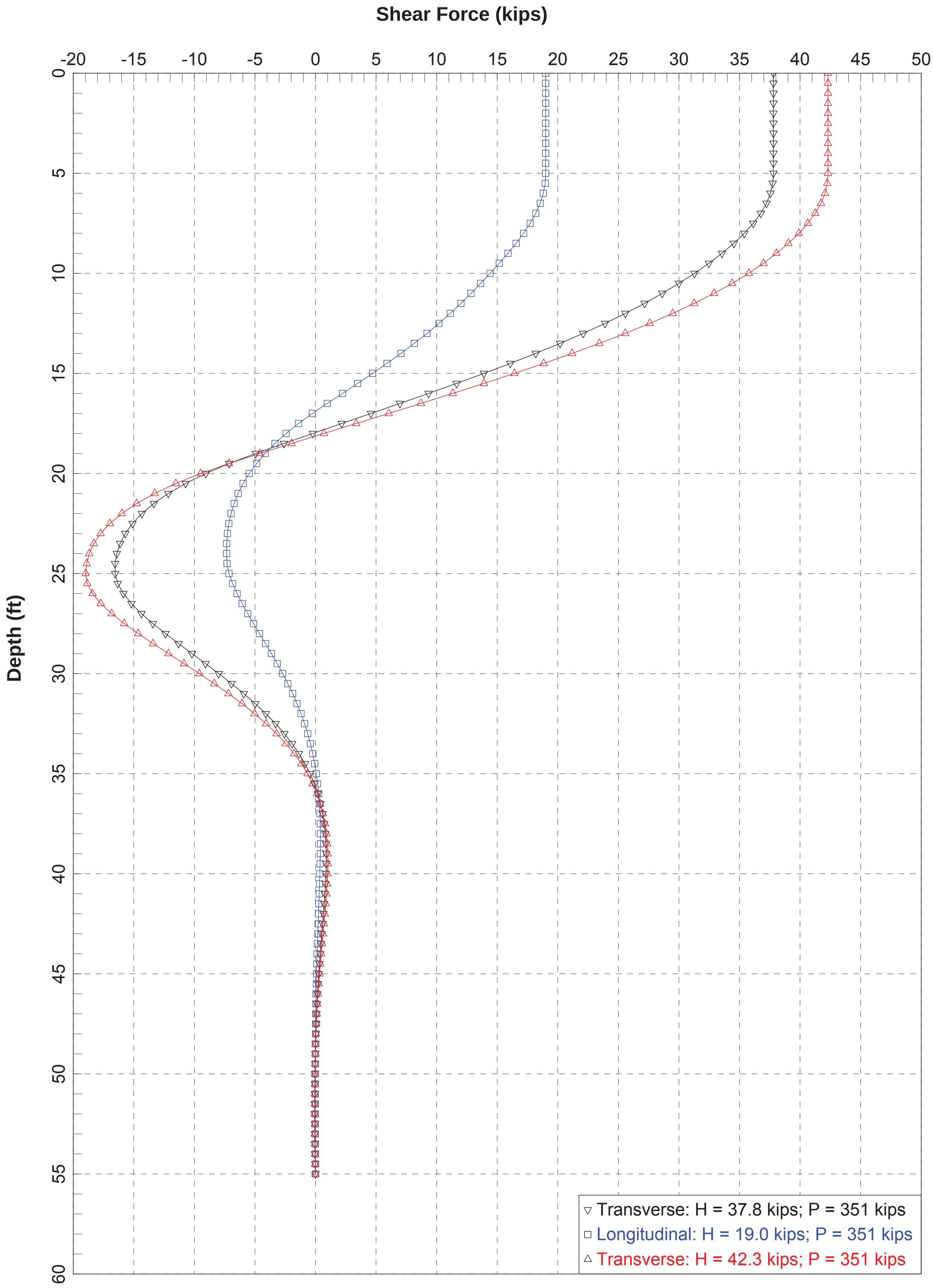
Load Case No.	Direction	H, kips	P, kips
1	Transverse	37.8	351
2	Longitudinal	19.0	351
3	Resultant	42.3	351

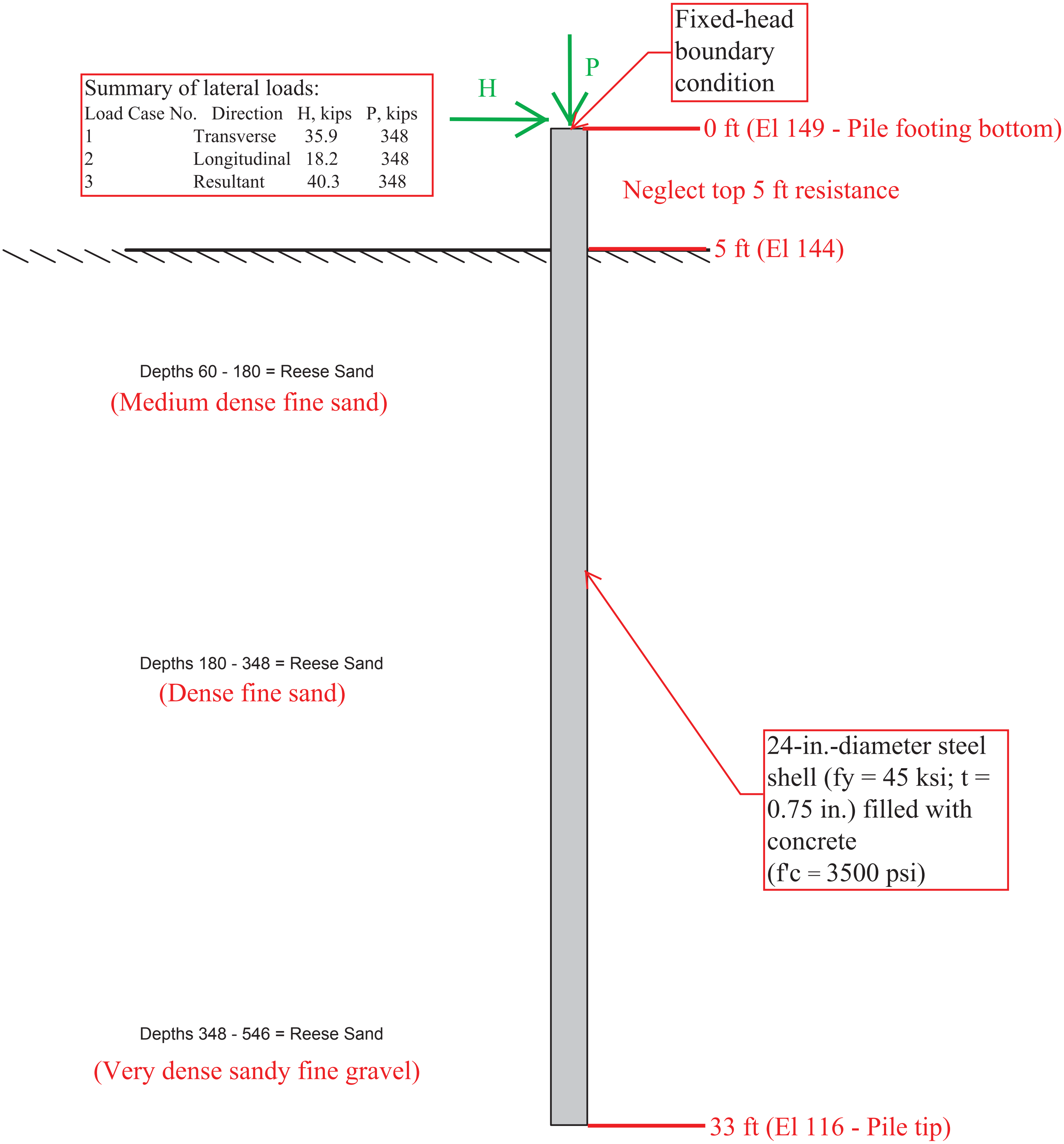
Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 8
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Resultant: H = 42.3 kips; P = 351 kips


Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Resultant: H = 42.3 kips; P = 351 kips


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 37.8 kips; P = 351 kips
□ Longitudinal: H = 19.0 kips; P = 351 kips
△ Transverse: H = 42.3 kips; P = 351 kips

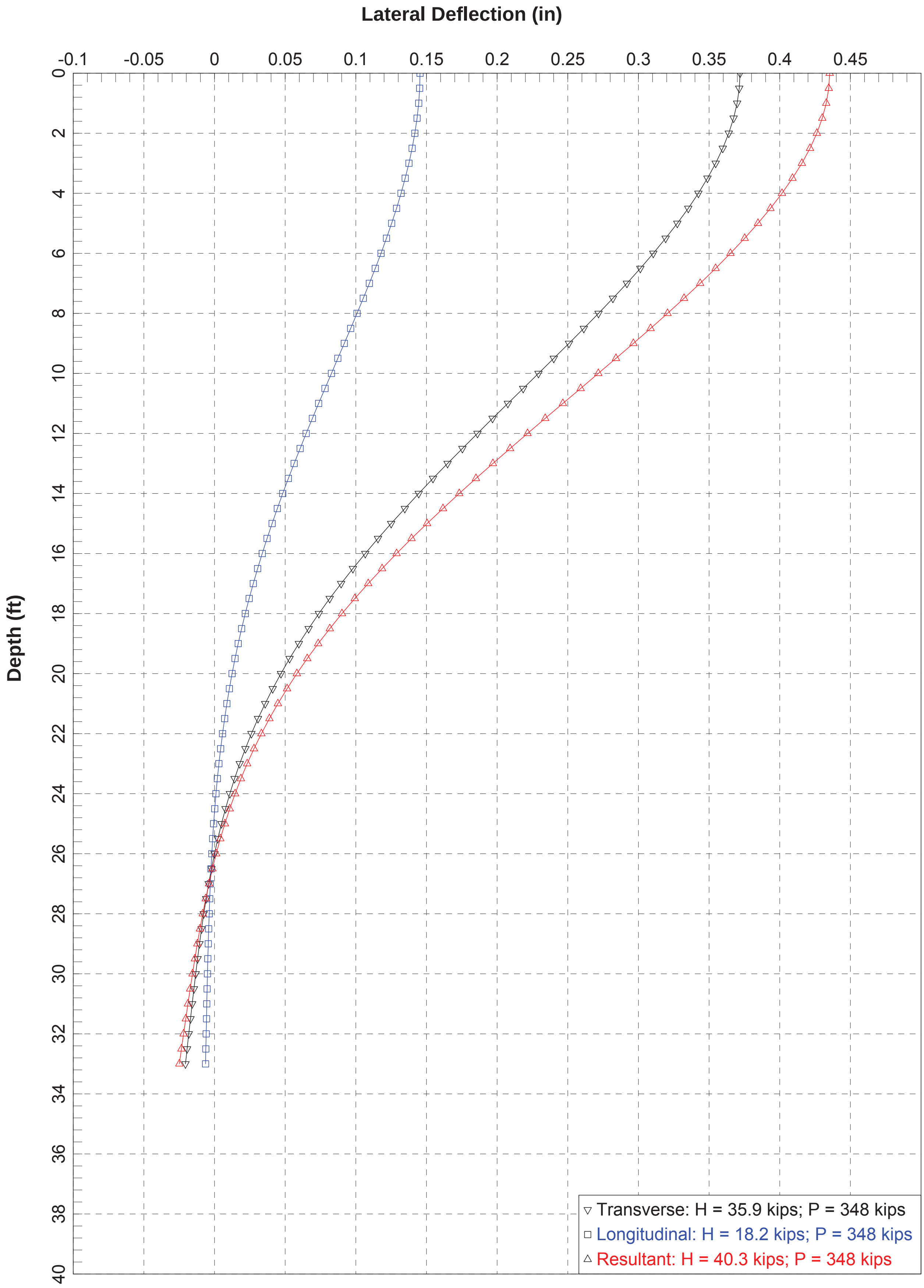


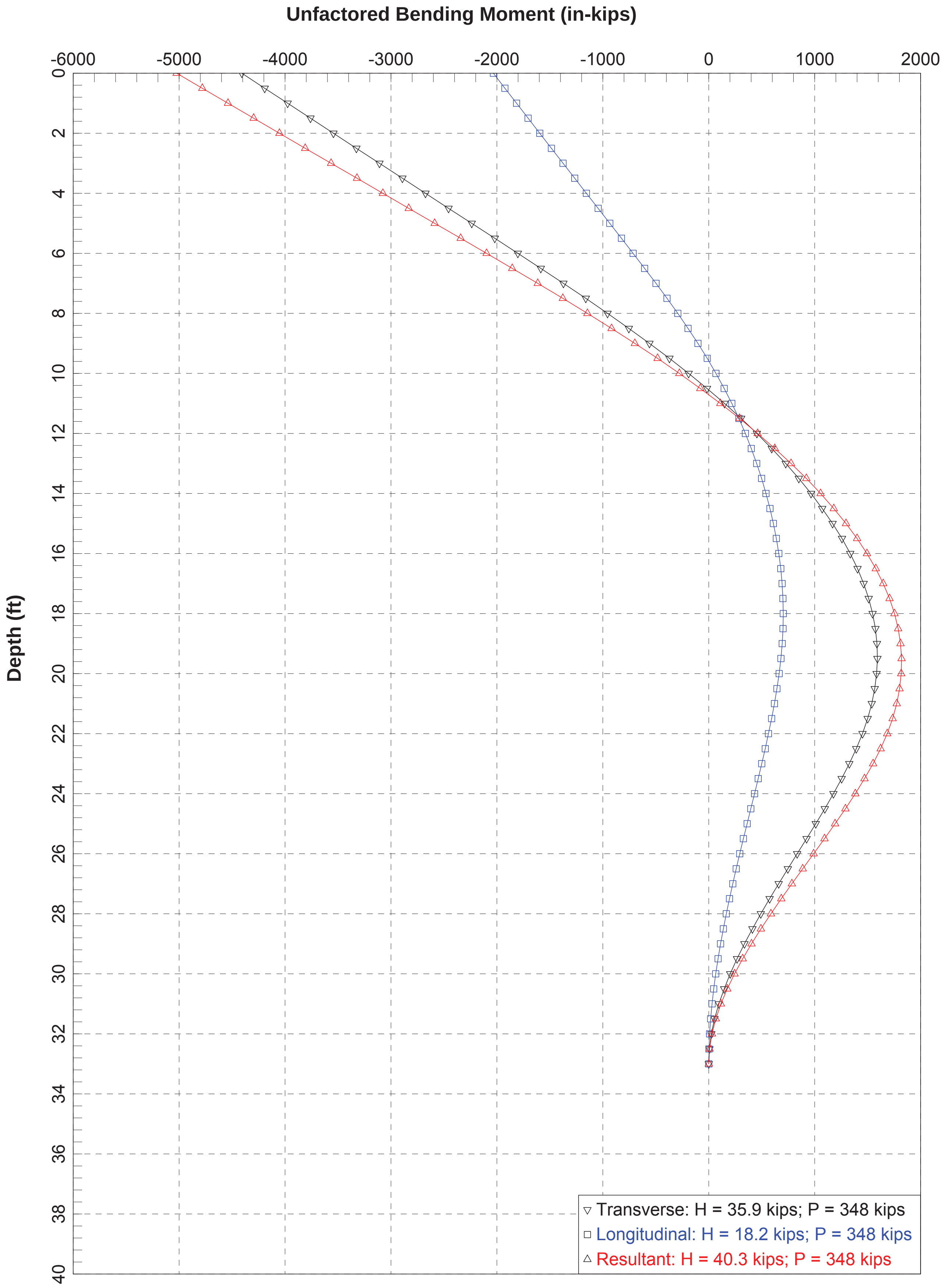
Summary of lateral loads:

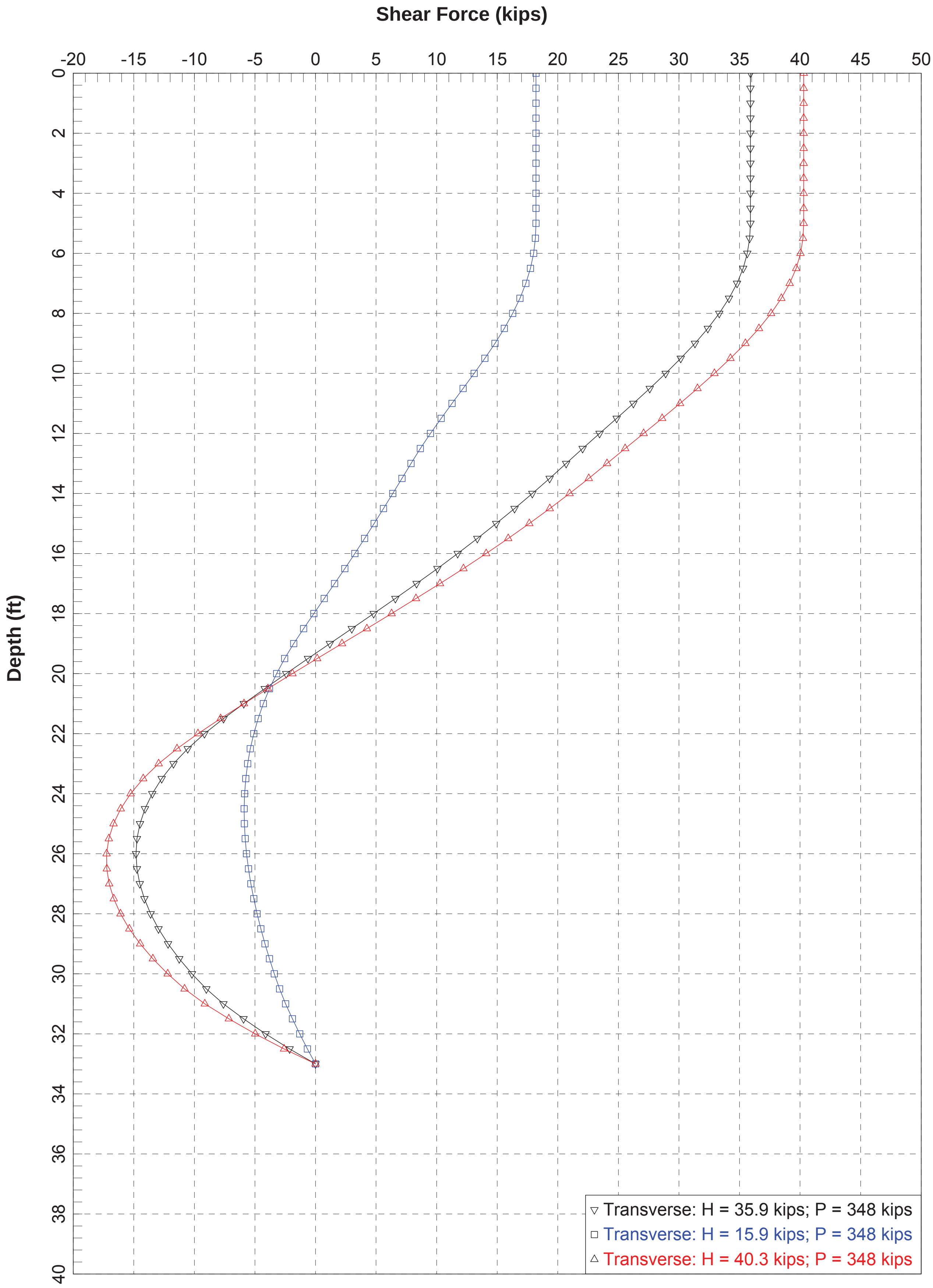
Load Case No.	Direction	H, kips	P, kips
1	Transverse	35.9	348
2	Longitudinal	18.2	348
3	Resultant	40.3	348

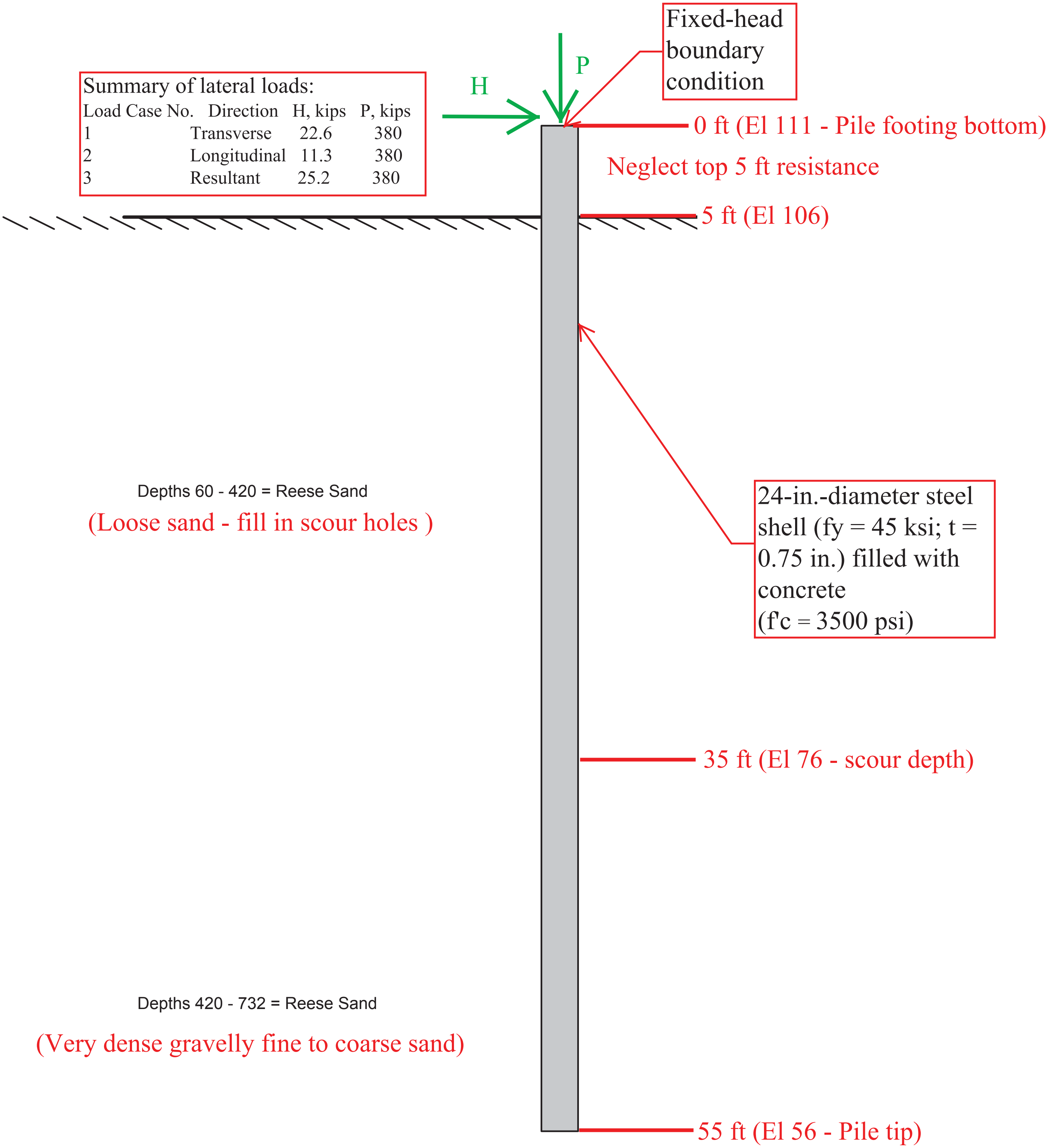
Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 9
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Longitudinal: H = 18.2 kips; P = 348 kips
△ Resultant: H = 40.3 kips; P = 348 kips


Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Longitudinal: H = 18.2 kips; P = 348 kips
△ Resultant: H = 40.3 kips; P = 348 kips


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Transverse: H = 35.9 kips; P = 348 kips
□ Transverse: H = 15.9 kips; P = 348 kips
△ Transverse: H = 40.3 kips; P = 348 kips

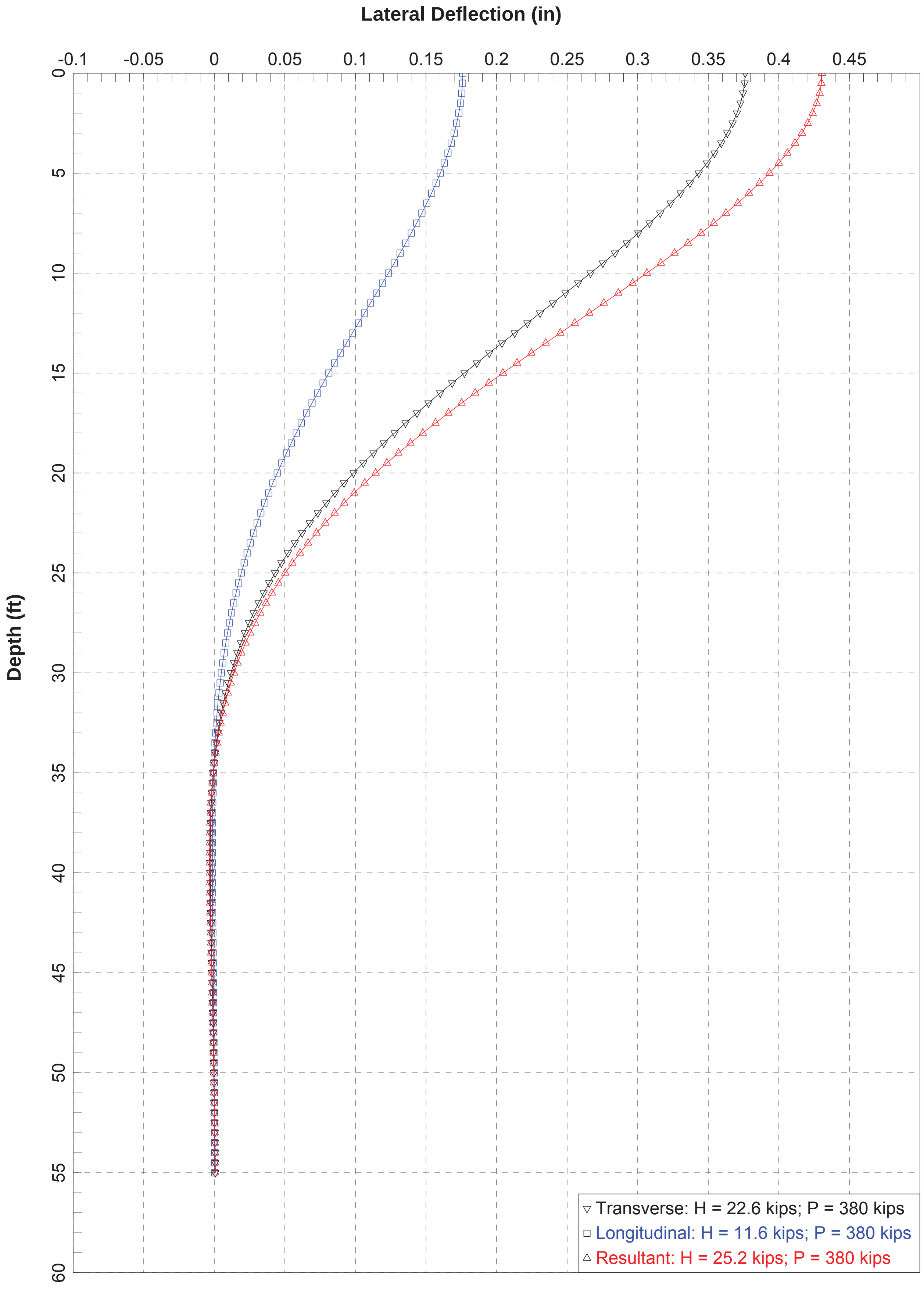


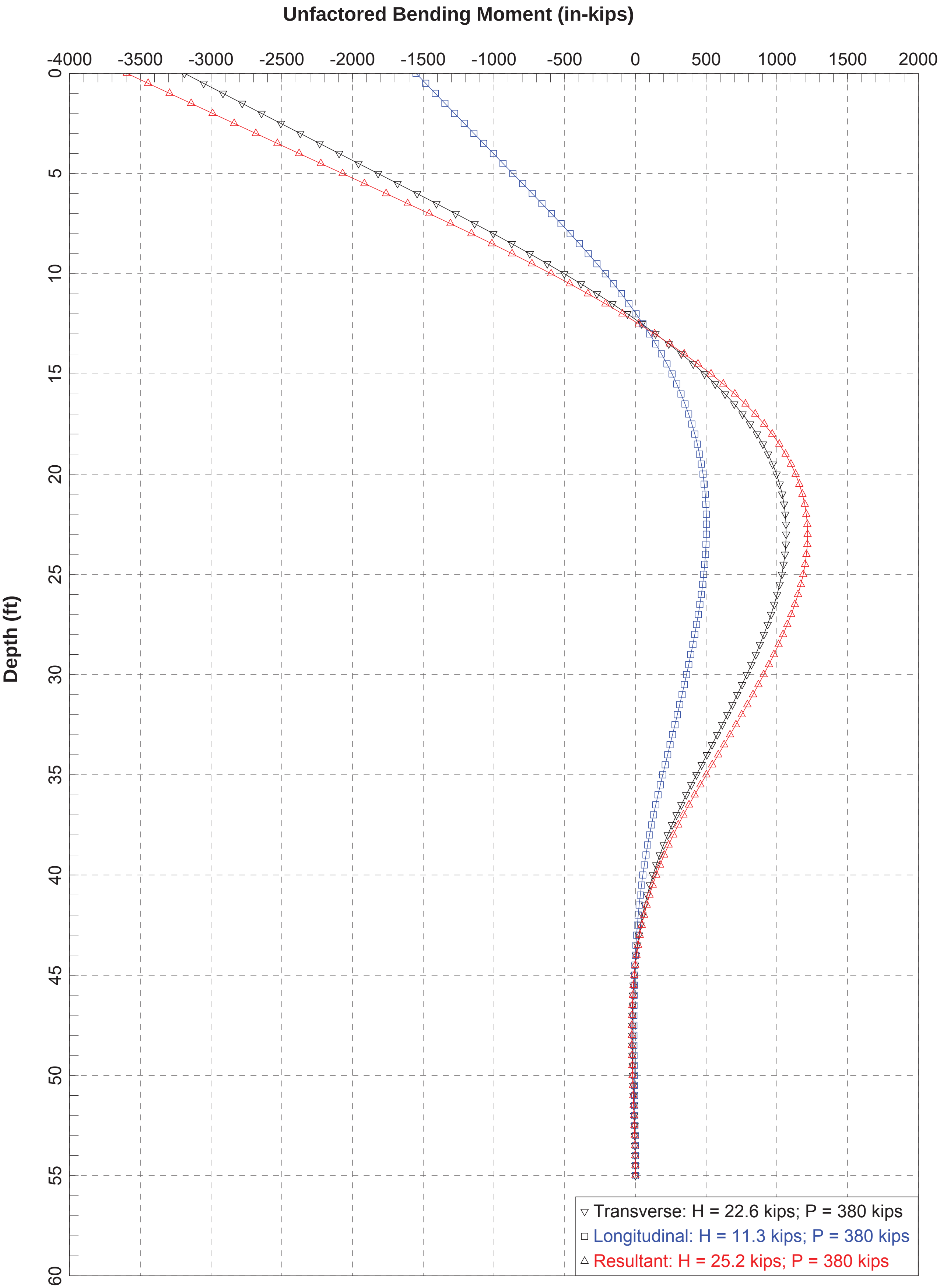
Summary of lateral loads:

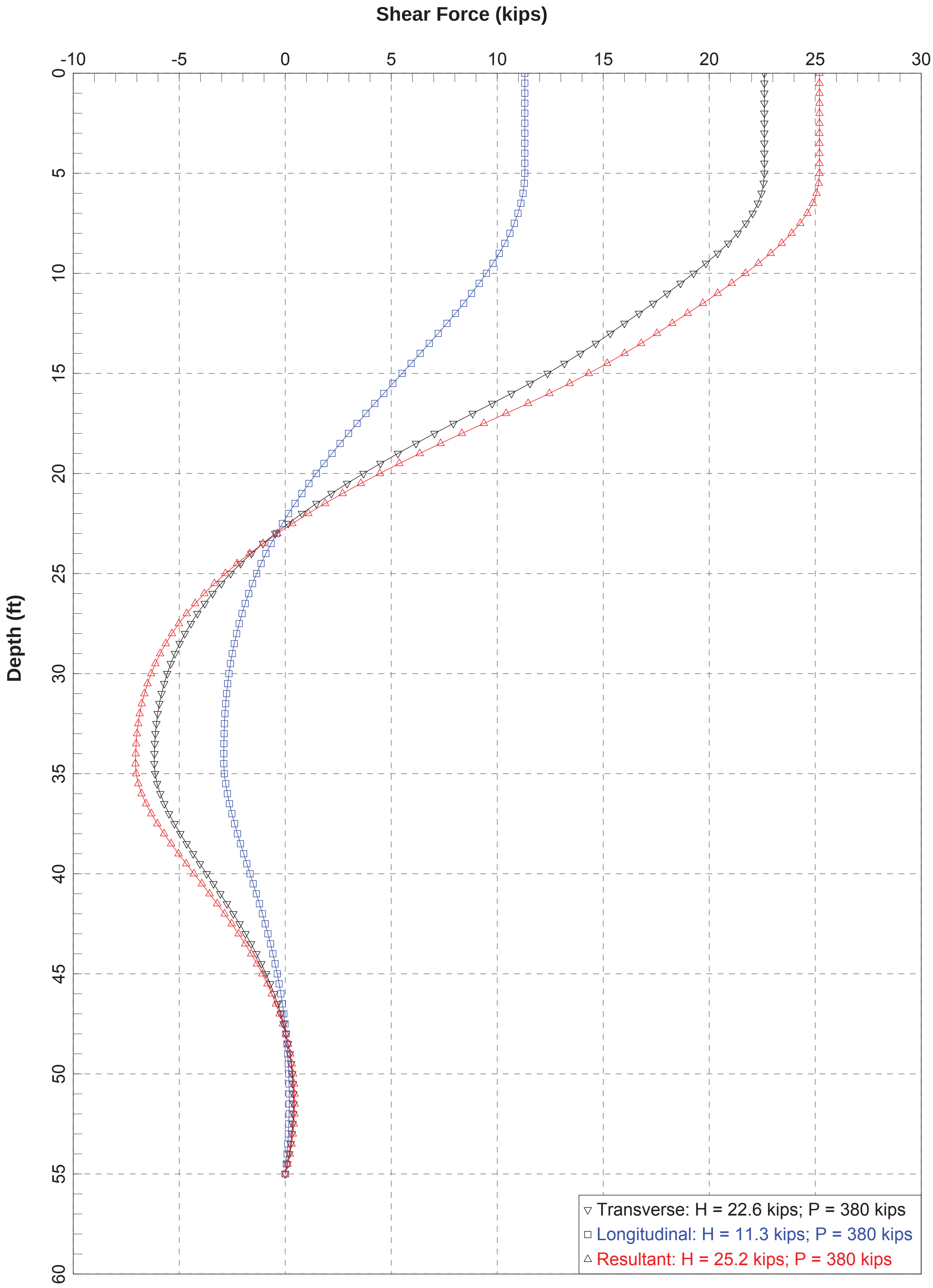
Load Case No.	Direction	H, kips	P, kips
1	Transverse	22.6	380
2	Longitudinal	11.3	380
3	Resultant	25.2	380

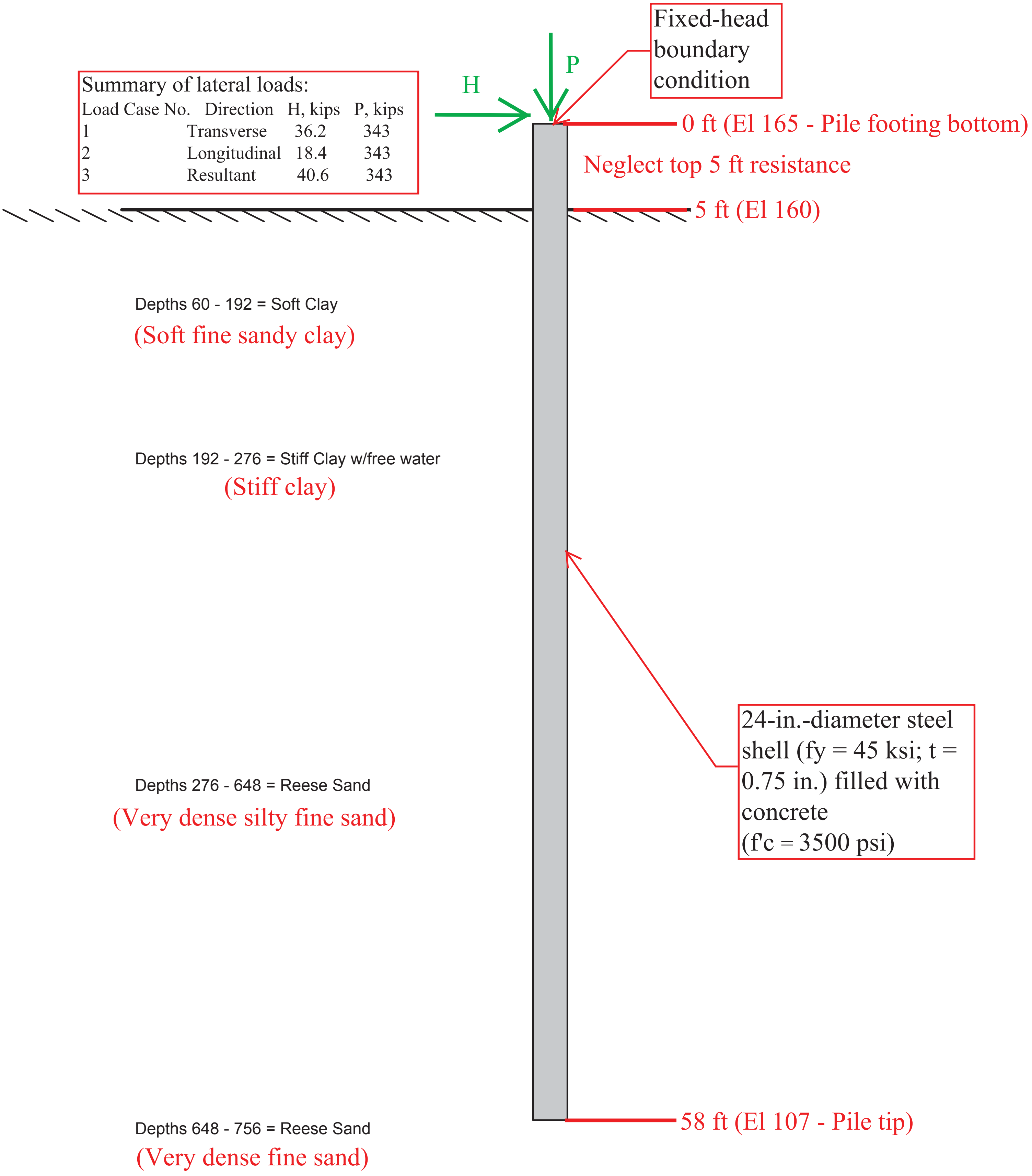
Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Pier 1
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.6 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips


Unfactored Bending Moment (in-kips)
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.3 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips


Shear Force (kips)
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 22.6 kips; P = 380 kips
□ Longitudinal: H = 11.3 kips; P = 380 kips
△ Resultant: H = 25.2 kips; P = 380 kips

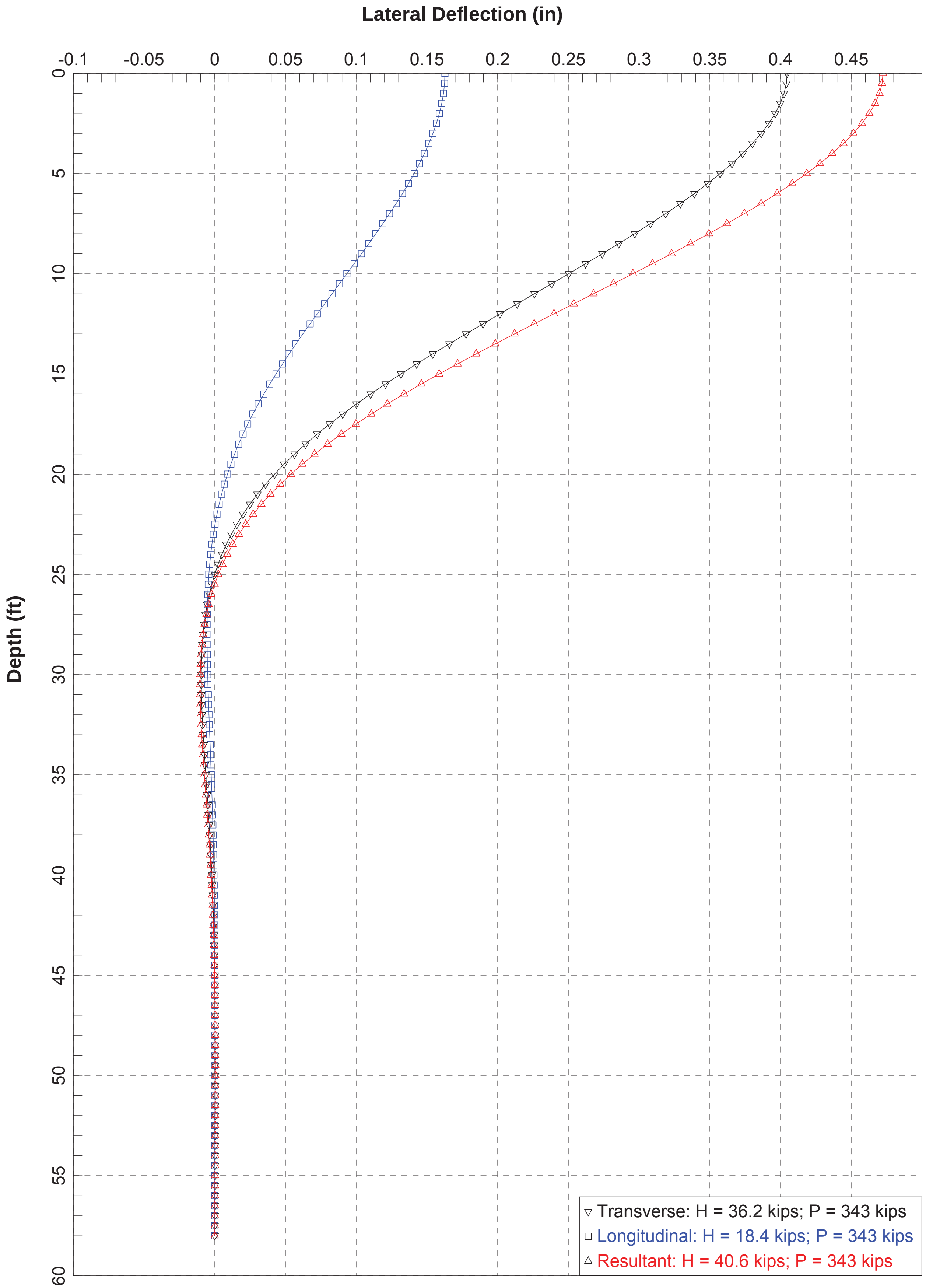


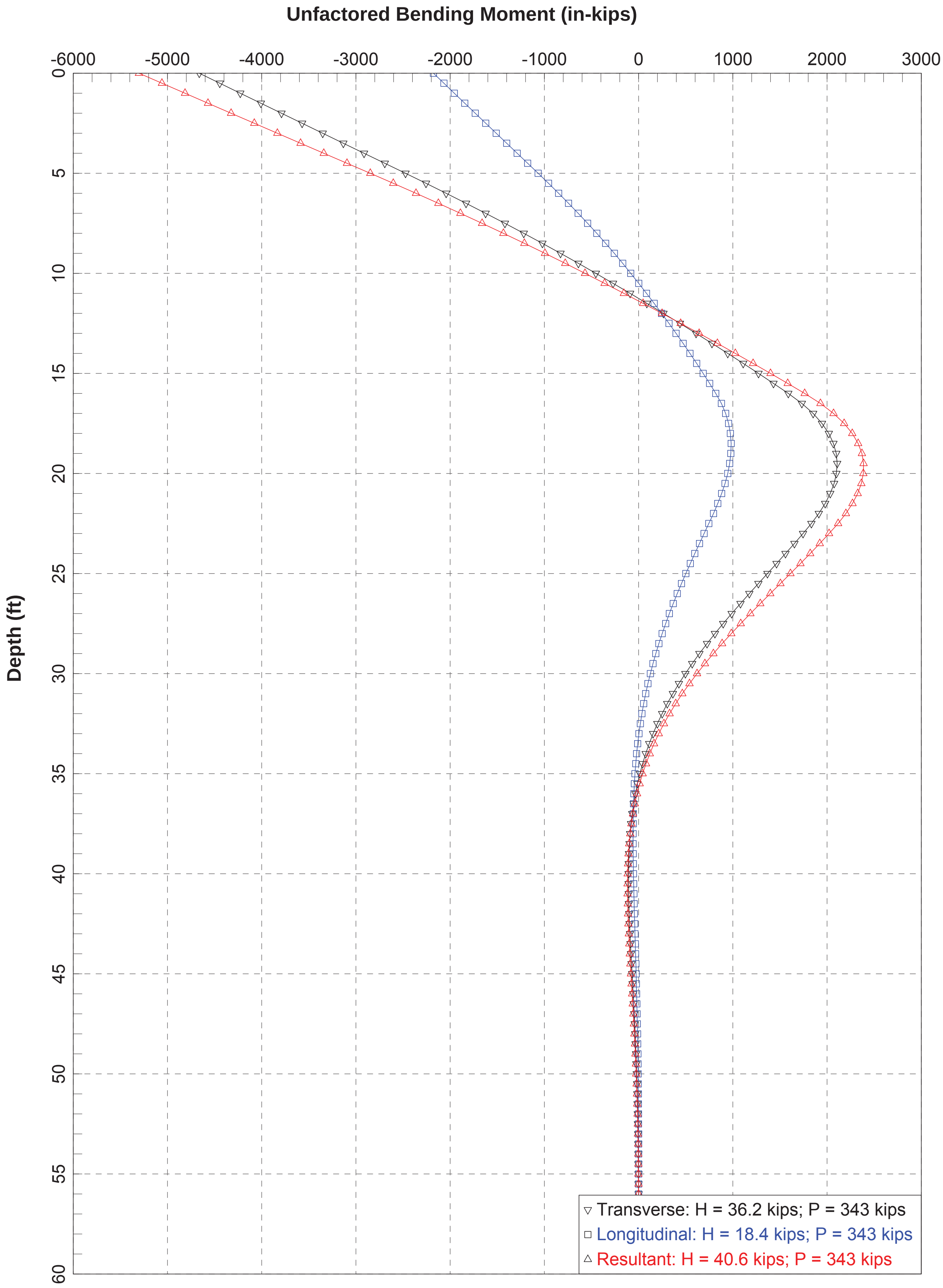
Summary of lateral loads:

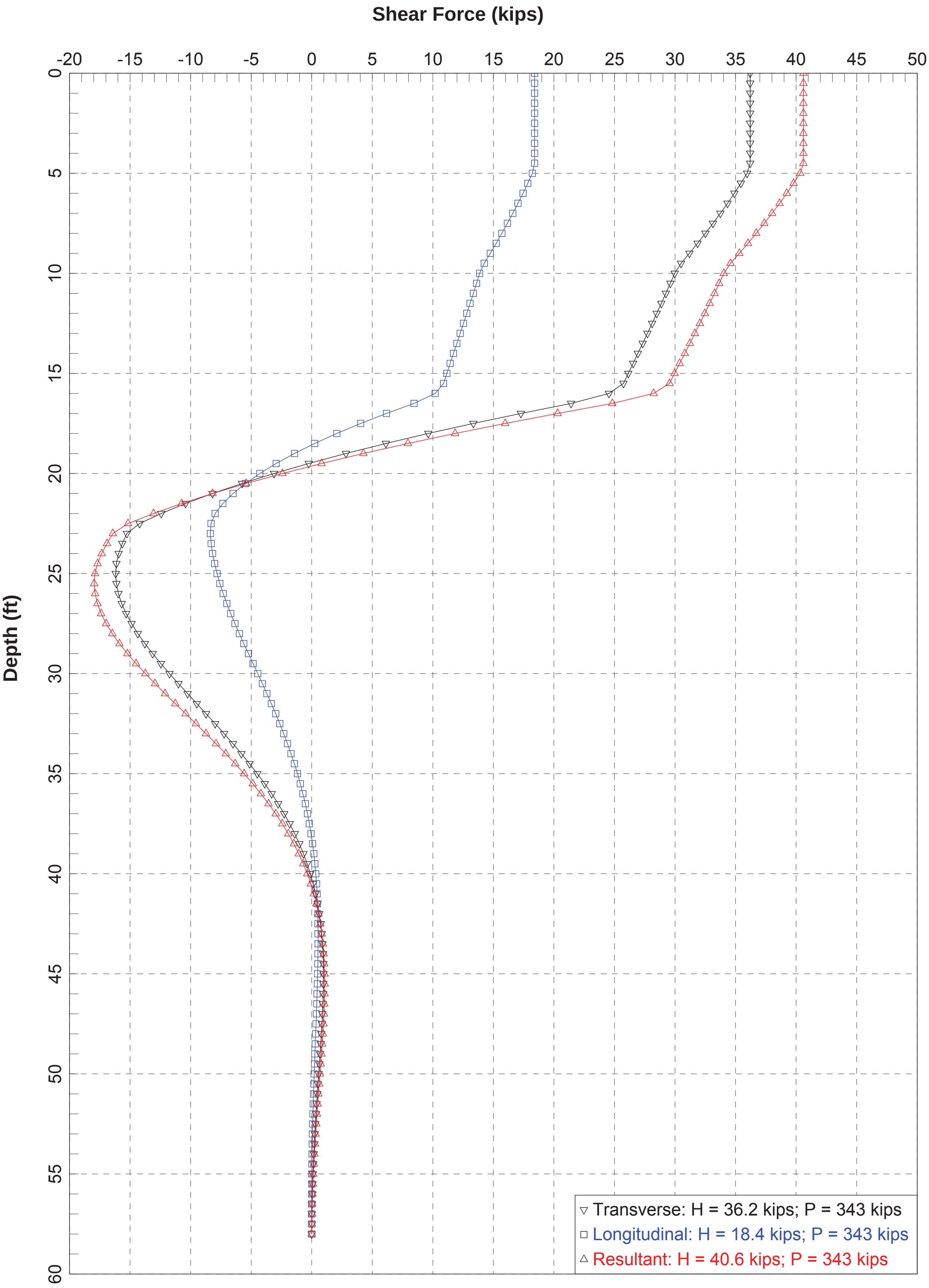
Load Case No.	Direction	H, kips	P, kips
1	Transverse	36.2	343
2	Longitudinal	18.4	343
3	Resultant	40.6	343

Results of Lateral Load Analysis
24-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 10
I-40 over White River

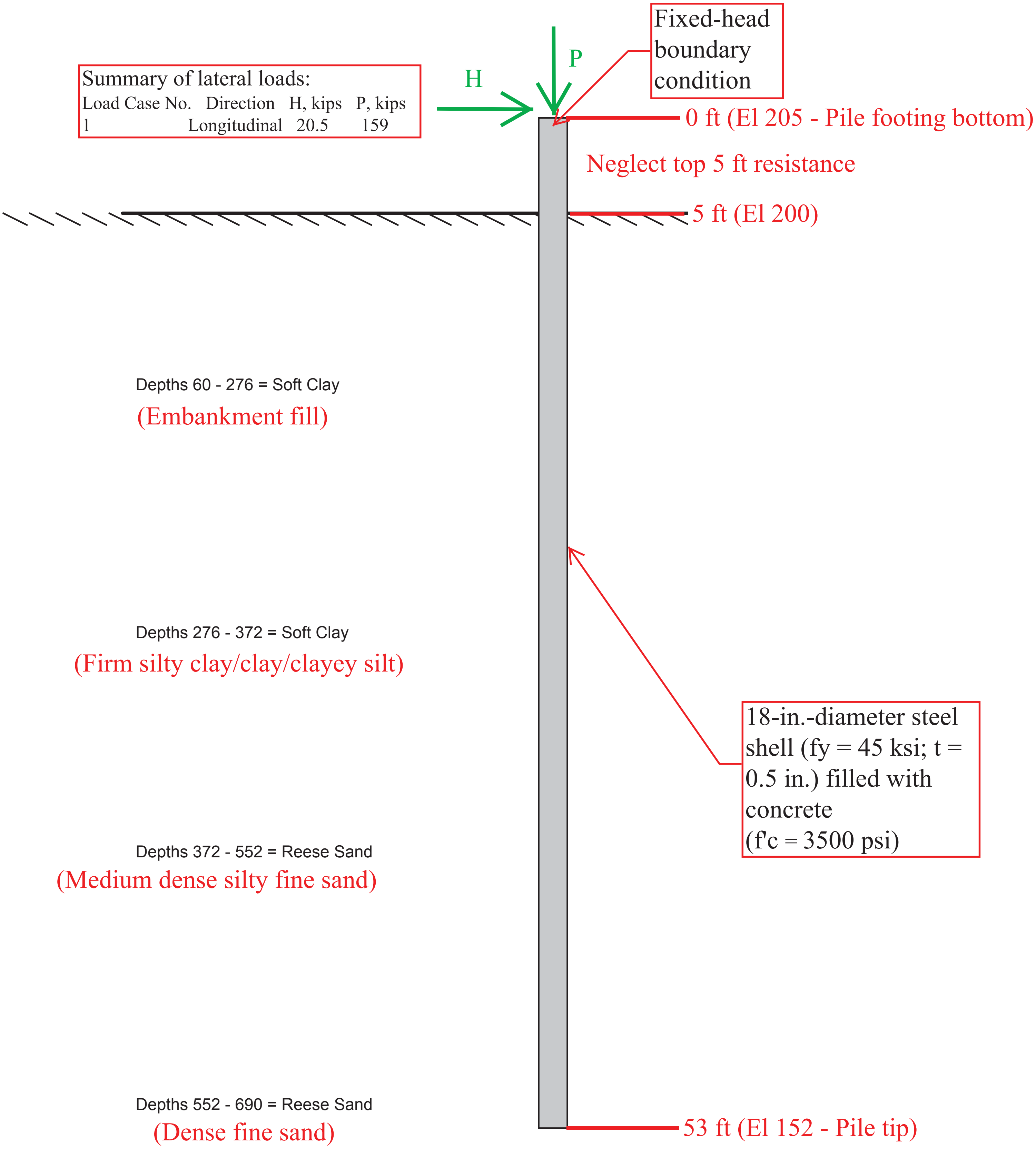
LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips


Unfactored Bending Moment (in-kips)
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips

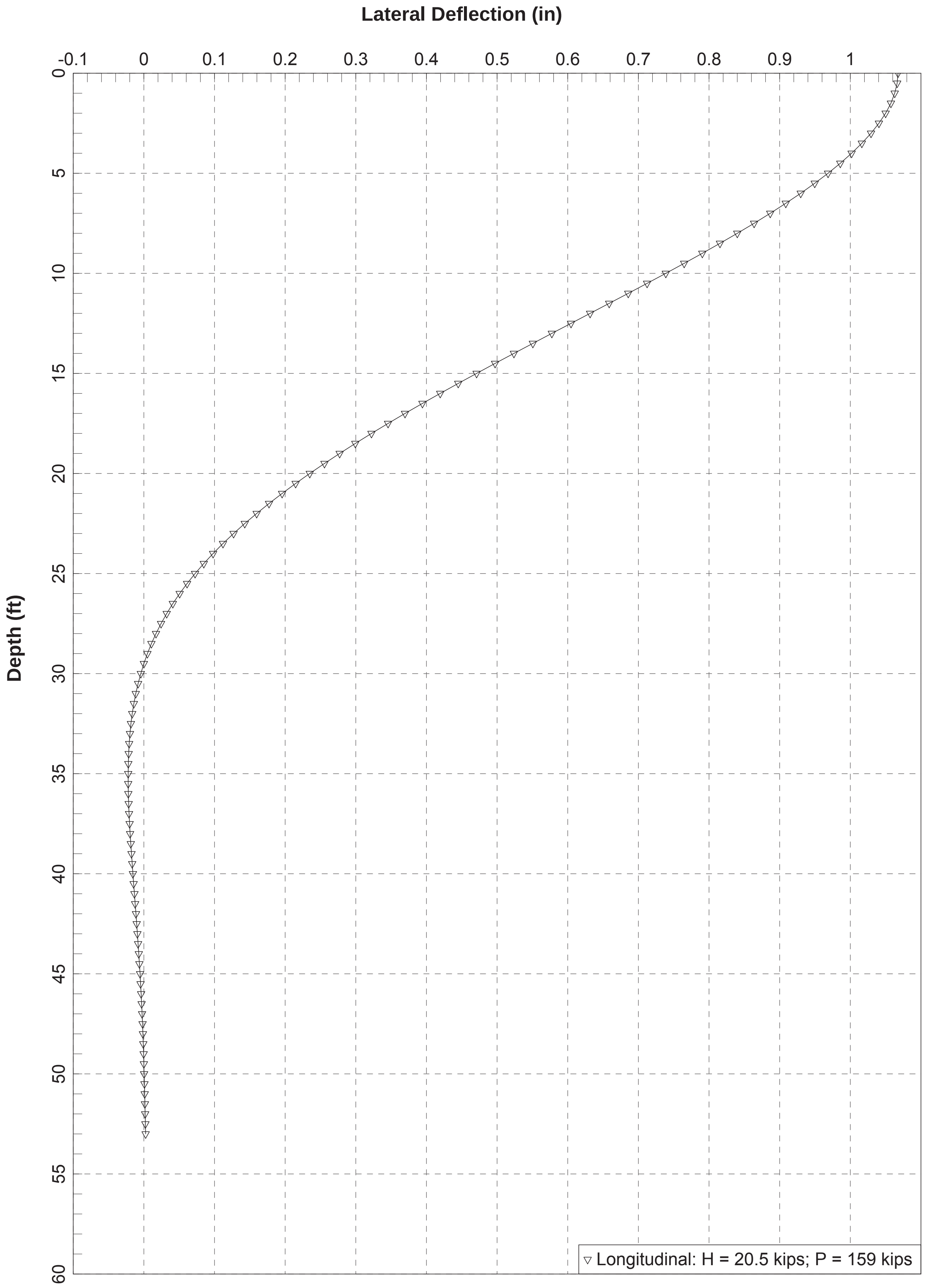


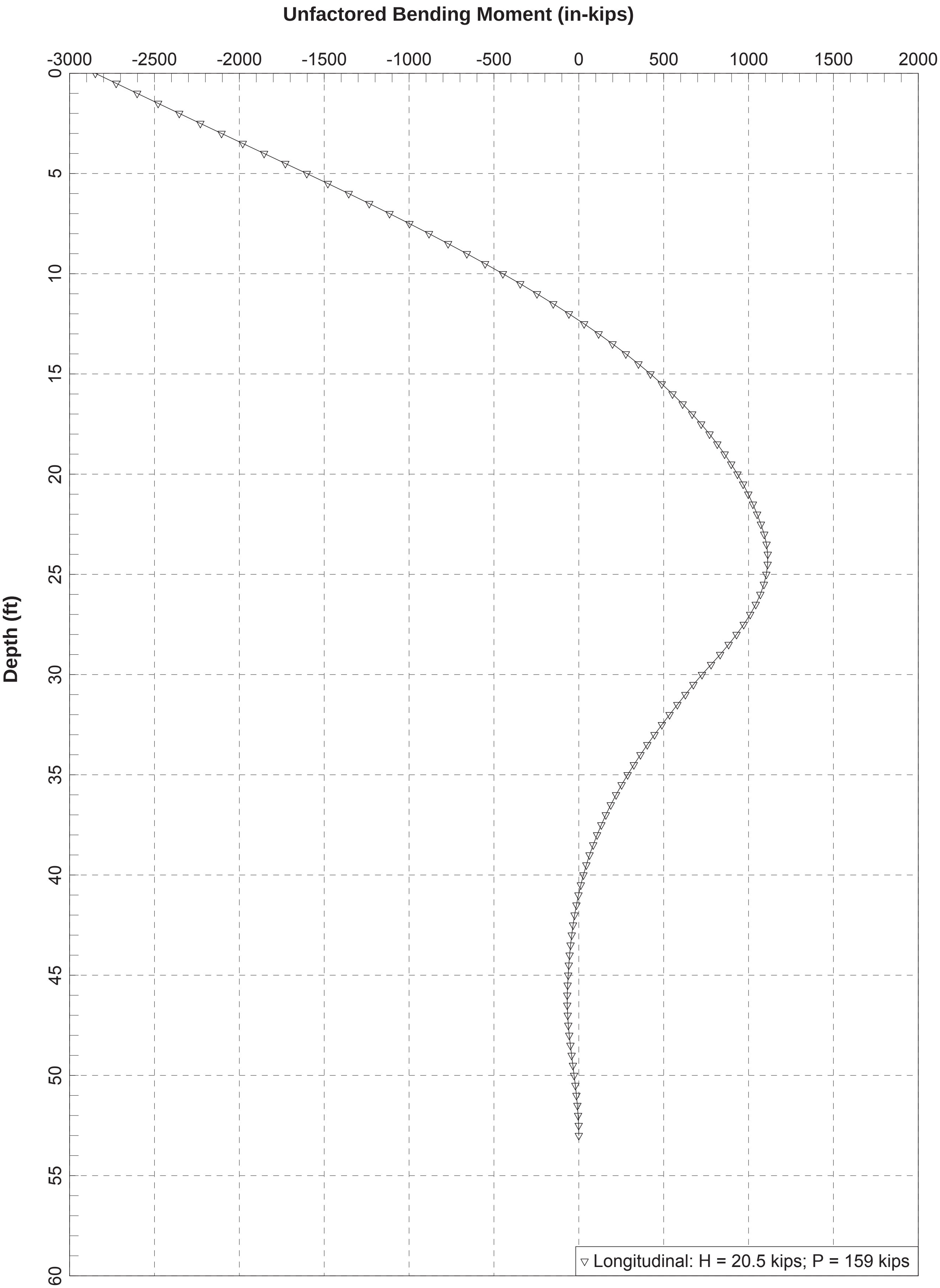
Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Transverse: H = 36.2 kips; P = 343 kips
□ Longitudinal: H = 18.4 kips; P = 343 kips
△ Resultant: H = 40.6 kips; P = 343 kips

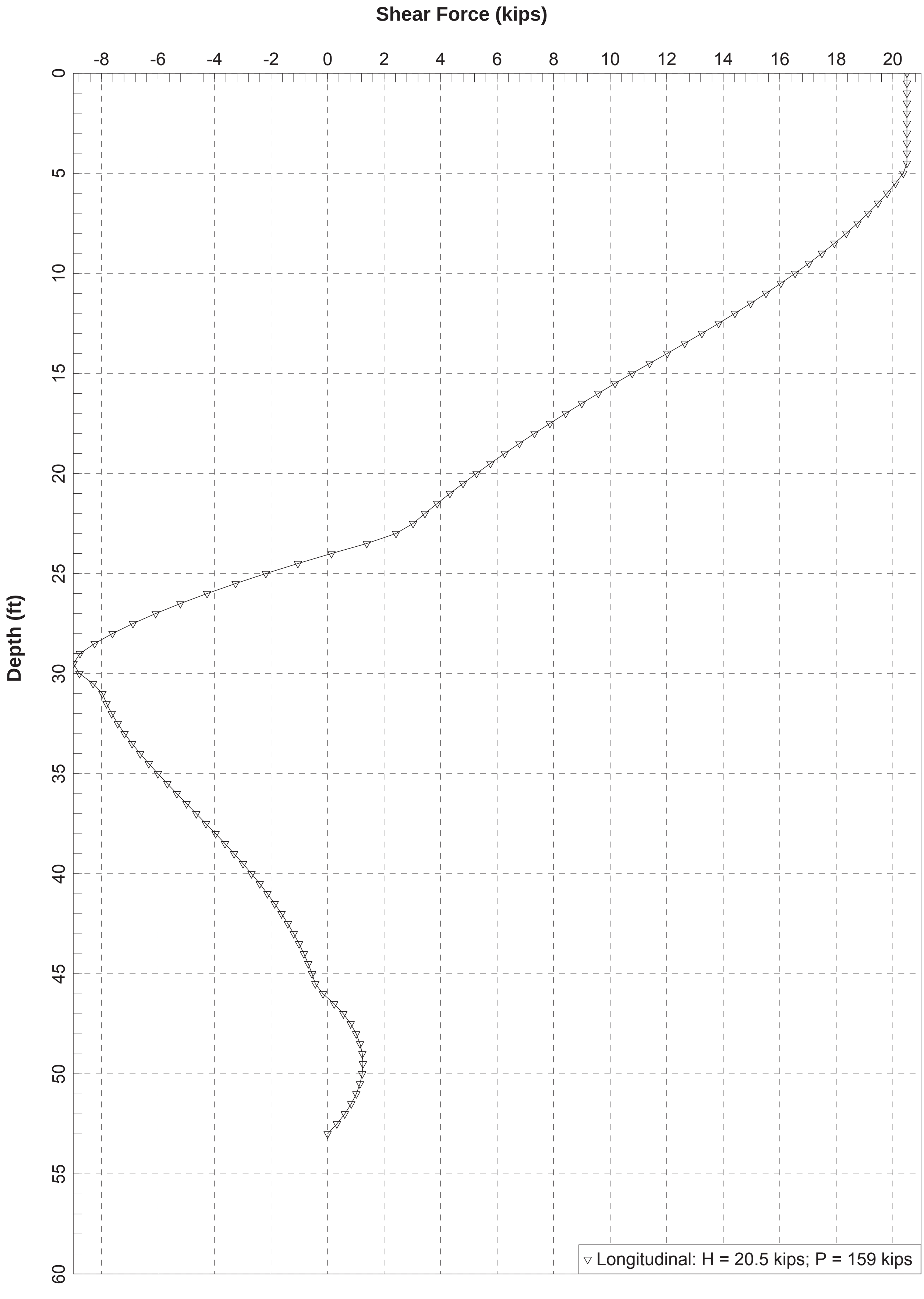


Results of Lateral Load Analysis
18-In.-Diameter Steel Shell Pile Filled w/Concrete
Bent 17 (East Abutment)
I-40 over White River

LPile Plus for Windows 5.0.25 (Single User), (c) 2005 by Ensoft, Inc.


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips


Unfactored Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips

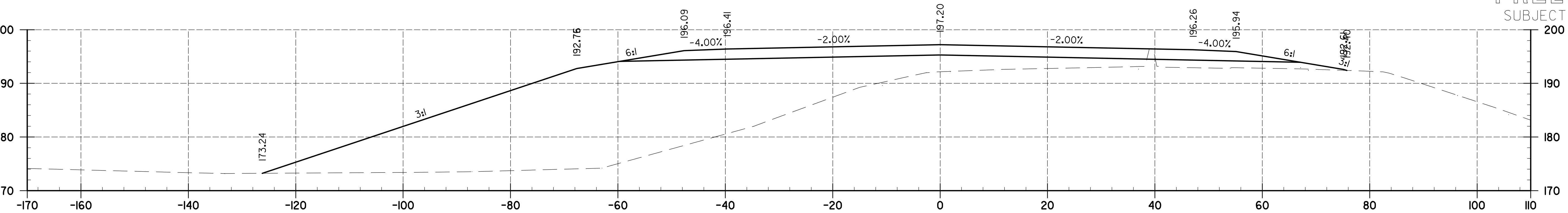

Shear Force (kips)
-8
-6
-4
-2
0
2
4
6
8
10
12
14
16
18
20
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
▽ Longitudinal: H = 20.5 kips; P = 159 kips

APPENDIX I

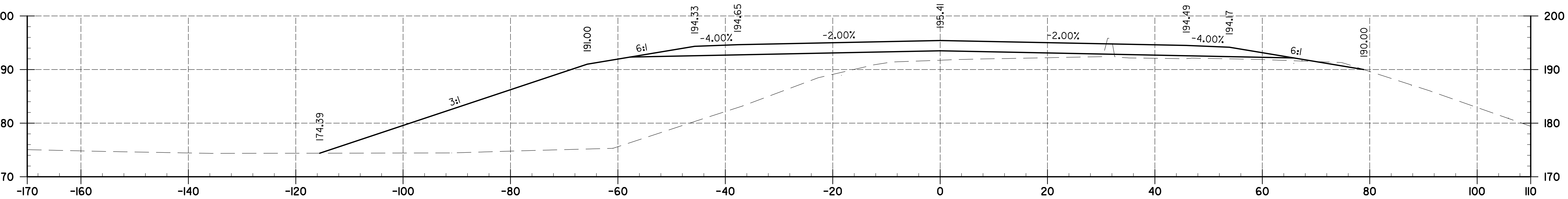
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610		_

②

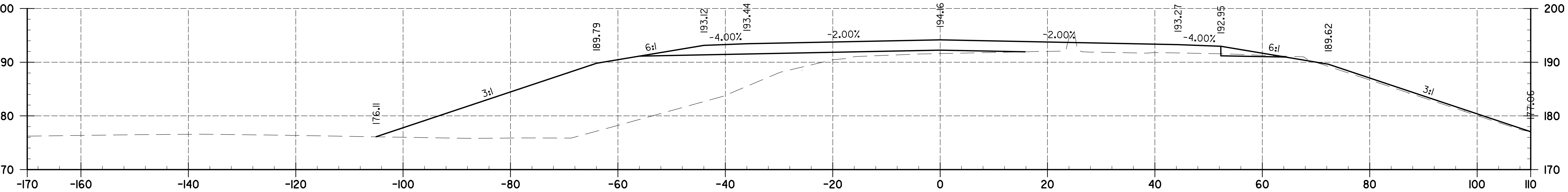
PRELIMINARY
SUBJECT TO REVISION



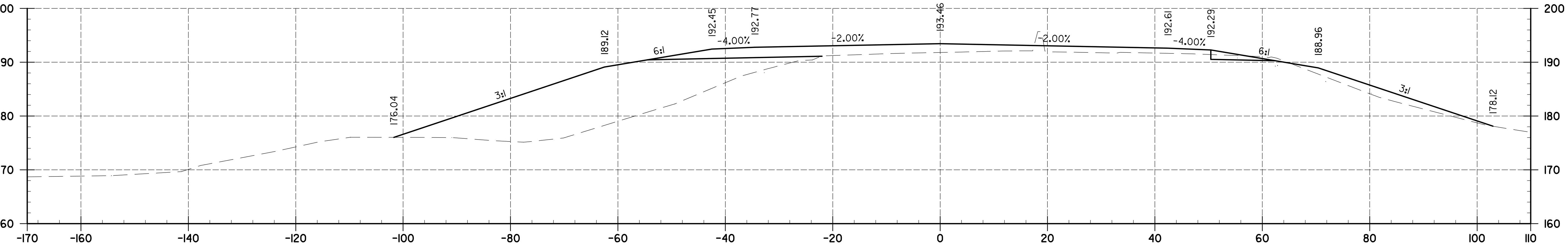
1878+00



1877+00



1876+00



1875+00

c:\pwworking\nal\rayrf\d0111634\WLXM2620-Cross-Section1.dgn

...\WLXM2620-Cross-Section1.dgn

11:40:33 AM

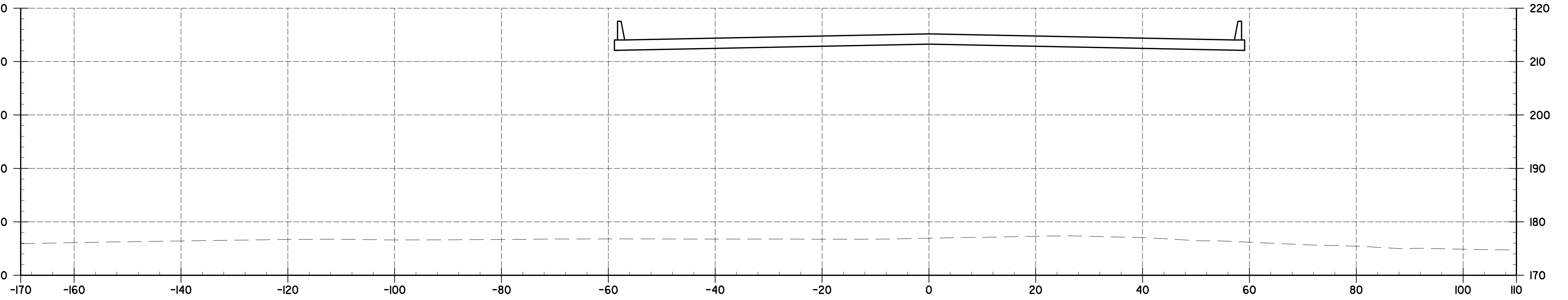
4/25/2014

JACOBS

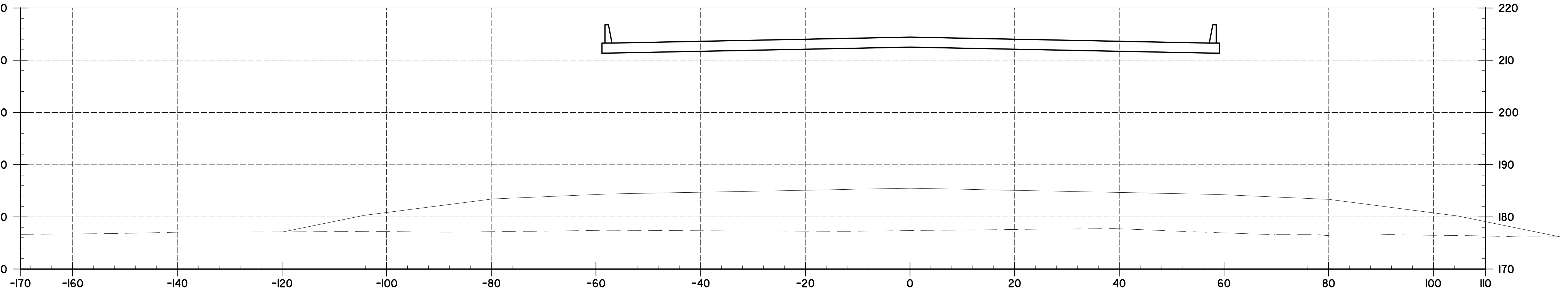
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610		–

②

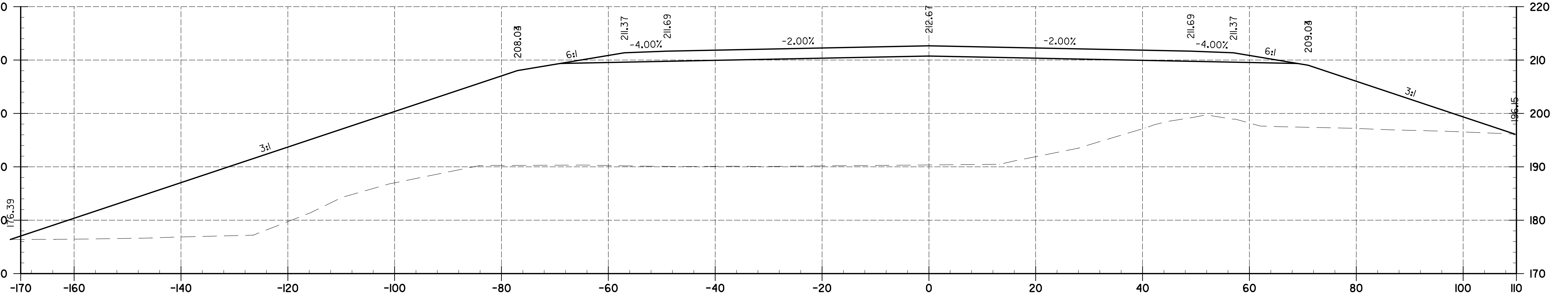
PRELIMINARY
SUBJECT TO REVISION



1884+25.60 - TOE OF FILL SLOPE



1884+00



1883+41.92 - BEGIN BRIDGE

c:\pwworking\nal\rayrf\d0111634\WLXM2620-Cross-Section1.dgn

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11:42:19 AM

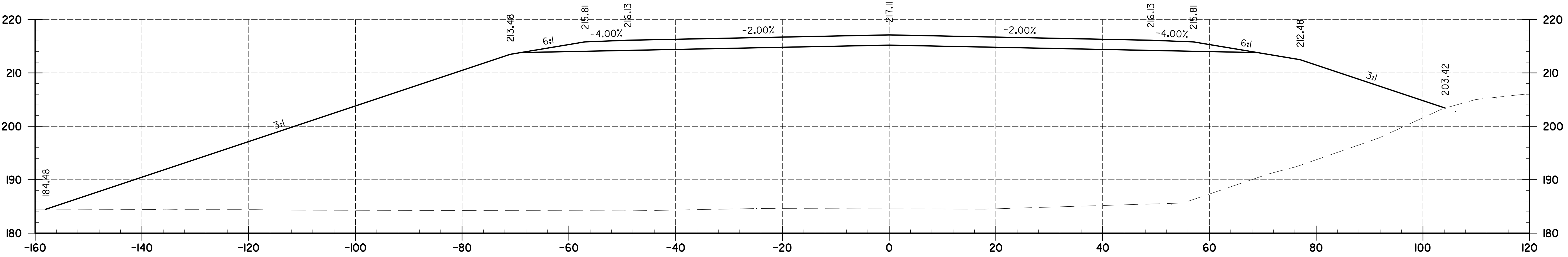
4/25/2014

JACOBS

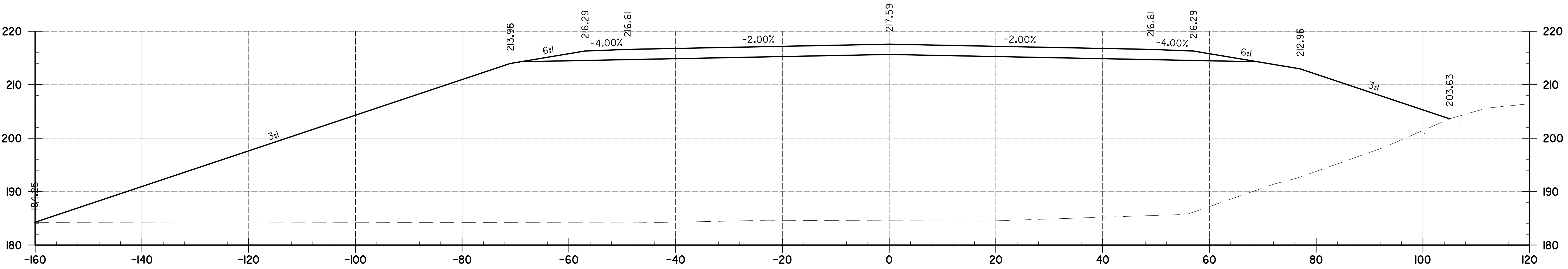
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610		–

②

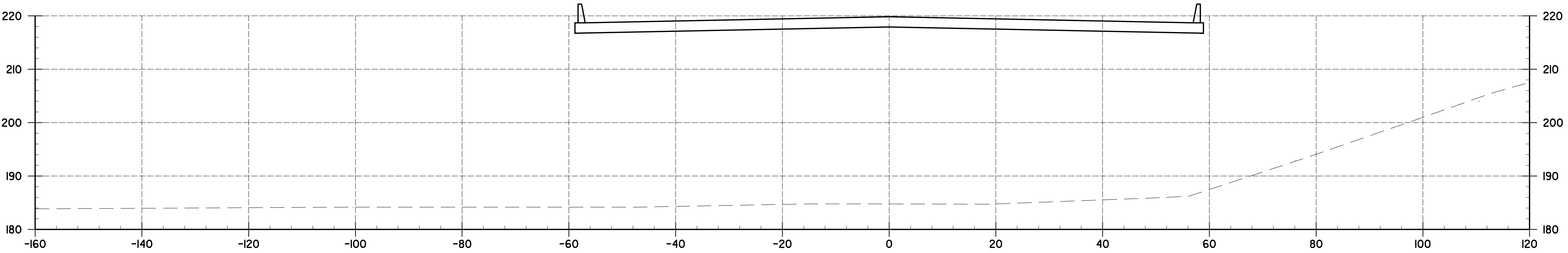
PRELIMINARY
SUBJECT TO REVISION



1912+00



1911+84.08 - END BRIDGE



1911+09.10 - TOE OF FILL SLOPE

c:\pwworking\nal\rayrf\d0111634\WLXM2620-Cross-Section1.dgn

...\WLXM2620-Cross-Section1.dgn

11:43:13 AM

4/25/2014

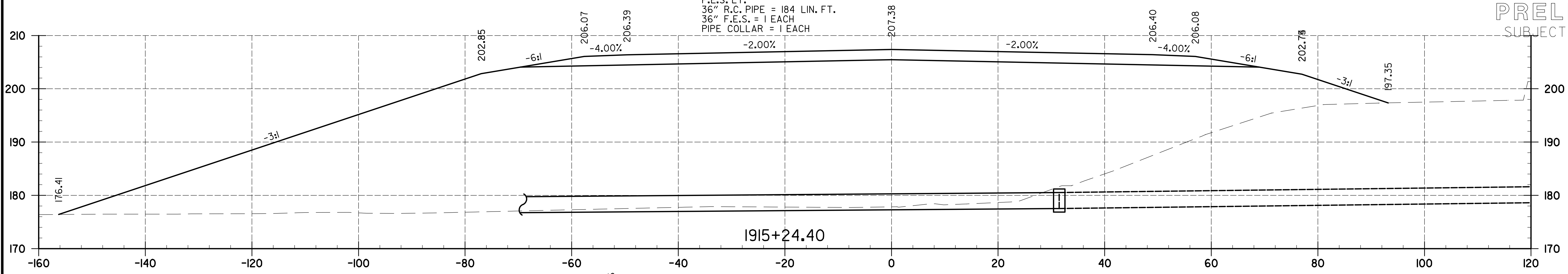
JACOBS

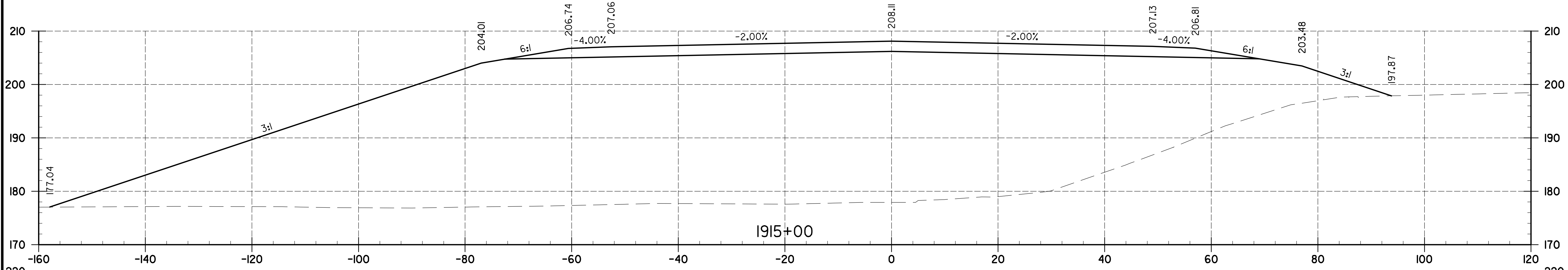
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		BB0610		_

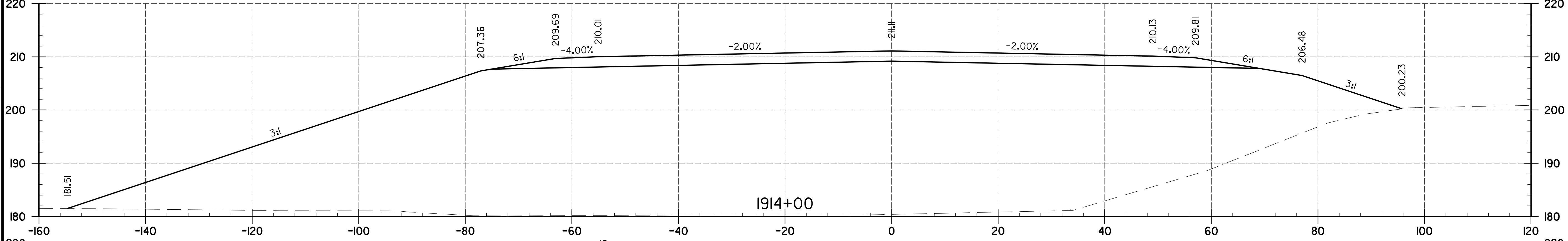
②

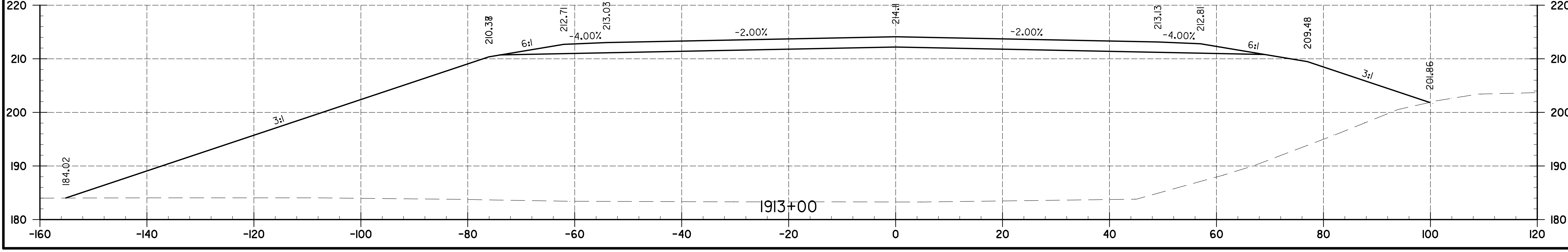
PRELIMINARY
SUBJECT TO REVISION

STA. 1915+24.4 IN PLACE
36"x171' R.C. PIPE CULVERT
WITH HDWLS. LT. & RT.
REMOVE HDWL. LT. AND EXTEND
R.C. PIPE 180' LT. ON 10°50' LT. FWD. SKEW
(CLASS III)(TYPE 3 BEDDING) WITH
F.E.S. LT.
36" R.C. PIPE = 184 LIN. FT.
36" F.E.S. = 1 EACH
PIPE COLLAR = 1 EACH









c:\pwworking\nal\rayrf\d0116634\WLXM2620-Cross-Section1.dgn

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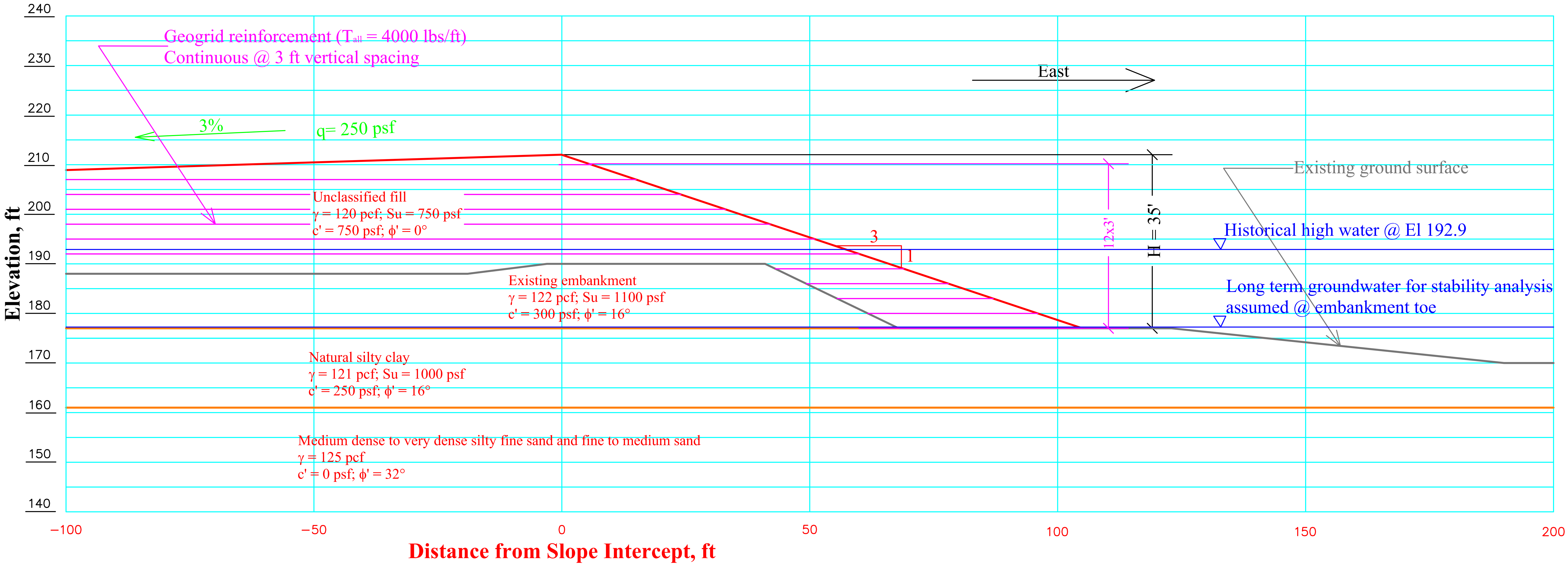
11:43:37 AM

4/25/2014

JACOBS

APPENDIX J



10 ft 0 10 20 ft

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

Section and Material Parameters for Stability Analysis
Reinforced End Slope @ West Abutment
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

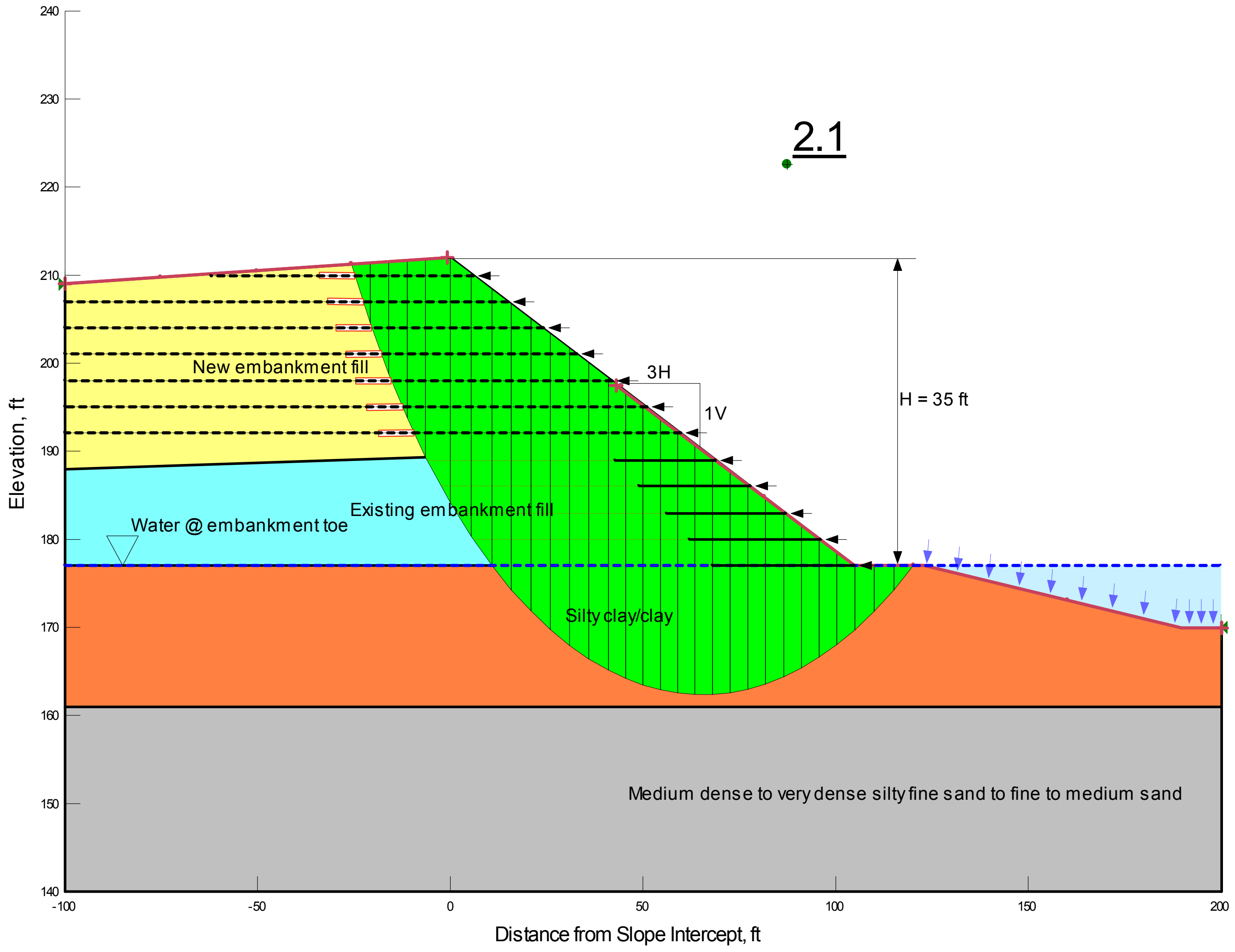
May 8, 2014

Plate

Stability Analysis Results – West Abutment
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

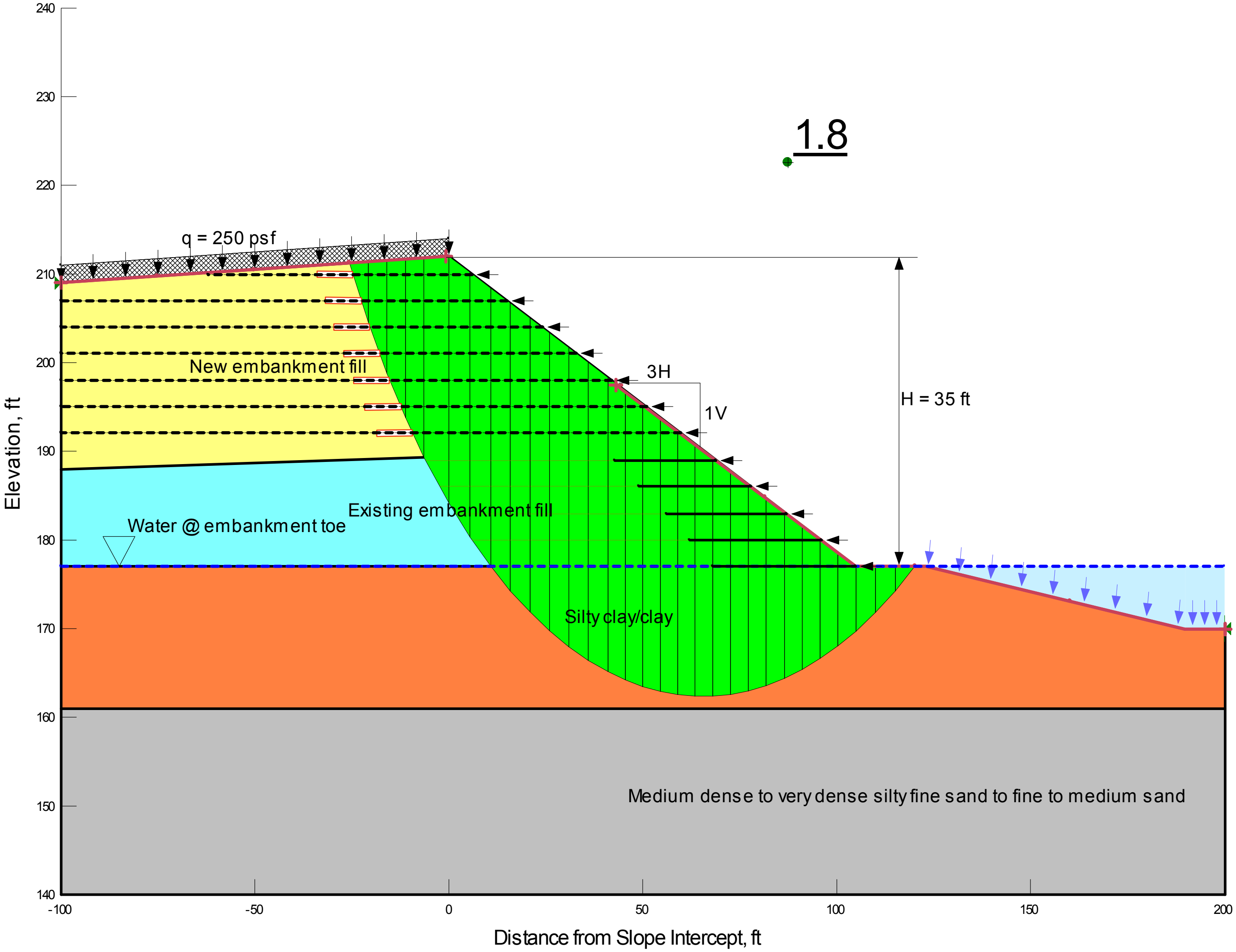
Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Analysis long term groundwater @ embankment toe	2.1
Long Term	Analysis long term groundwater @ embankment toe	1.8
Long Term	Historical high water @ El 192.9	1.9
Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.3
Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

* 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.



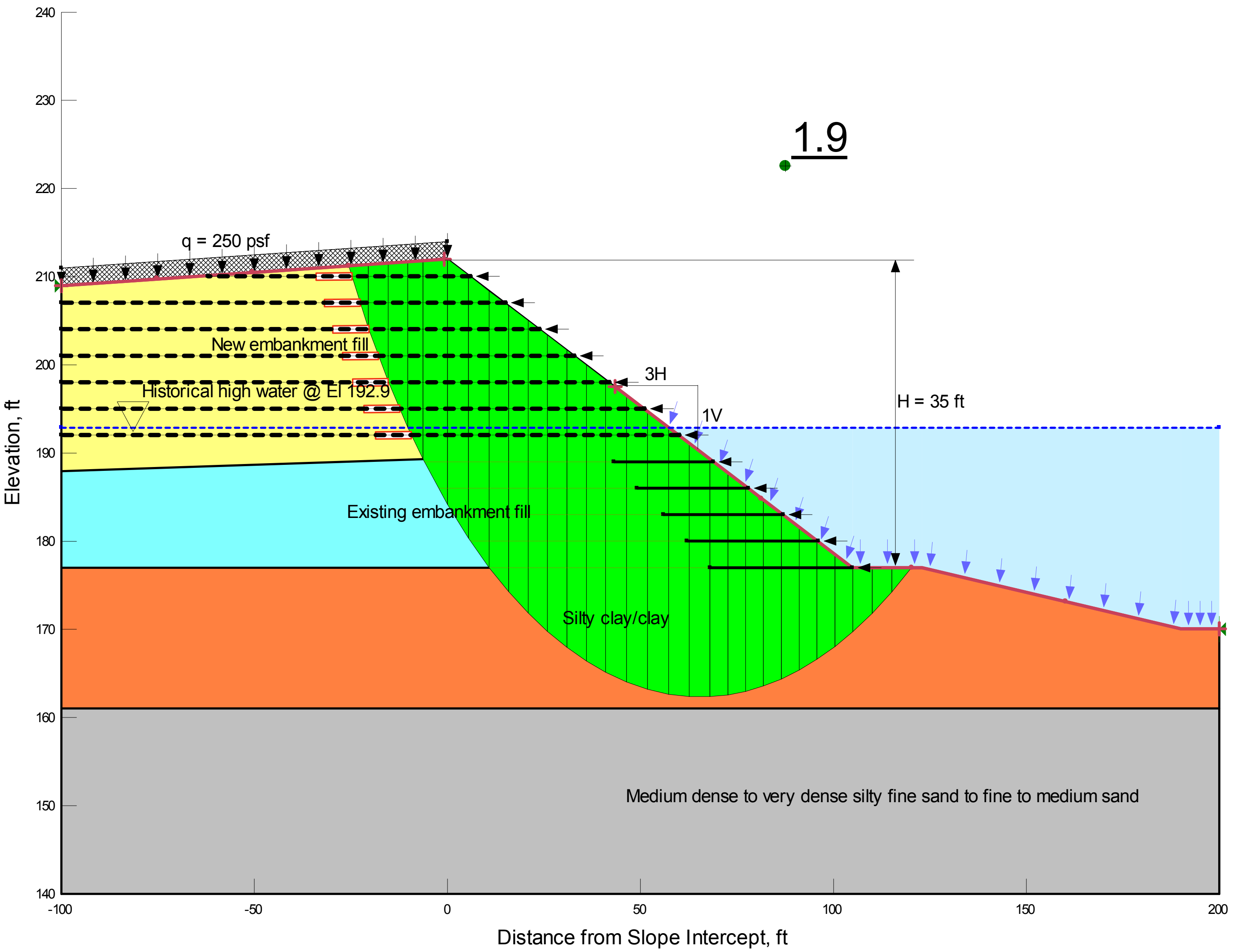
Results of Stability Analyses – End of Construction Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



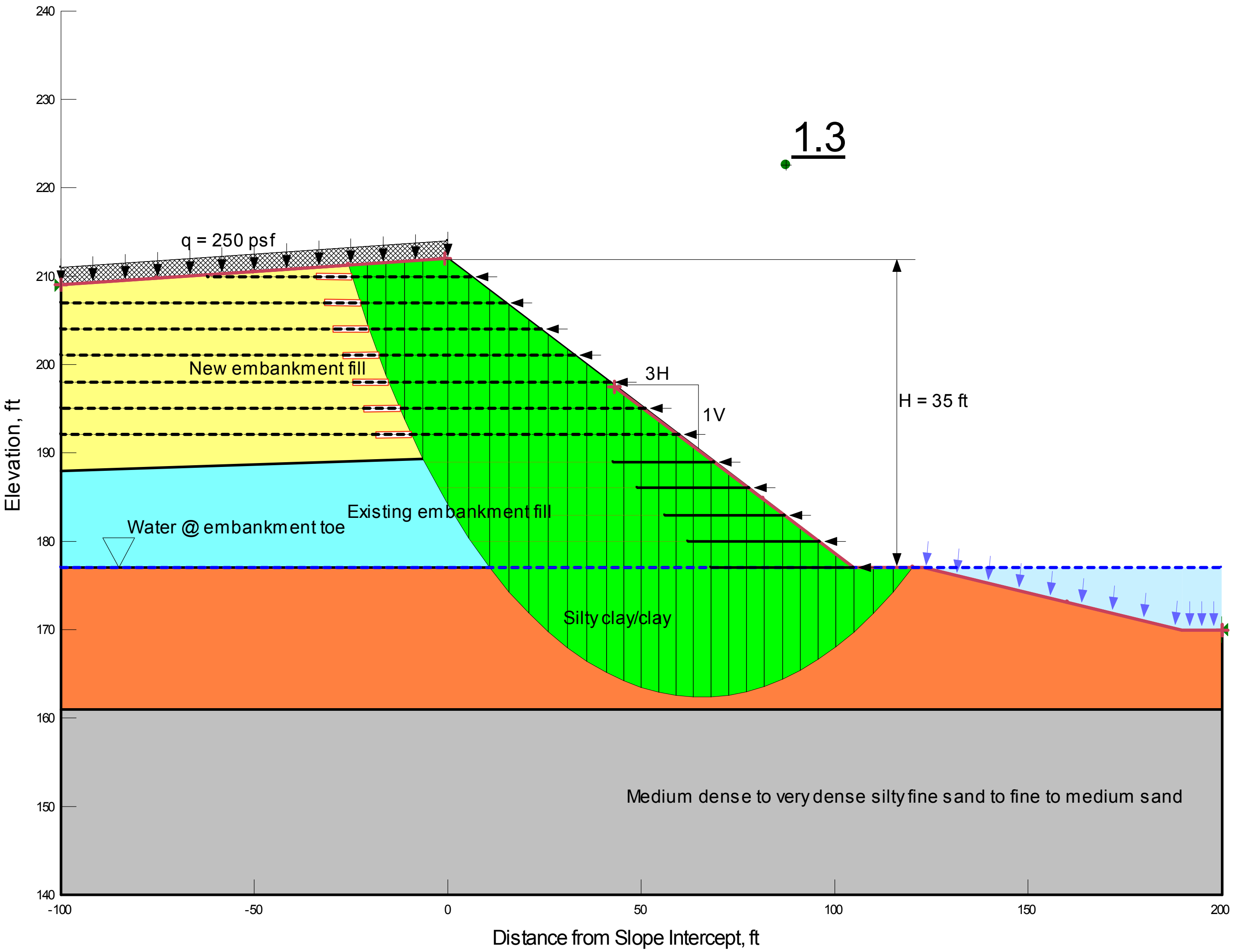
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



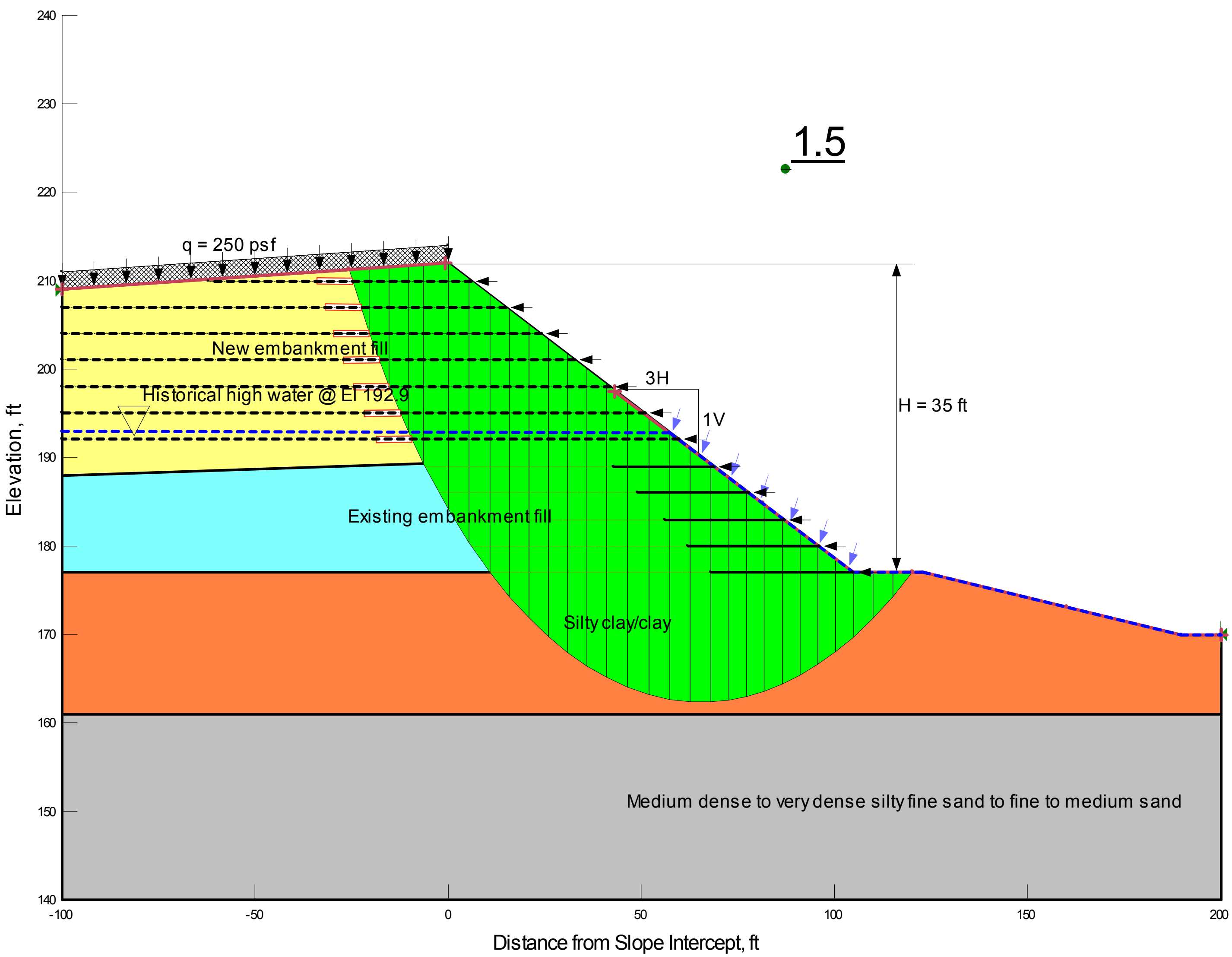
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

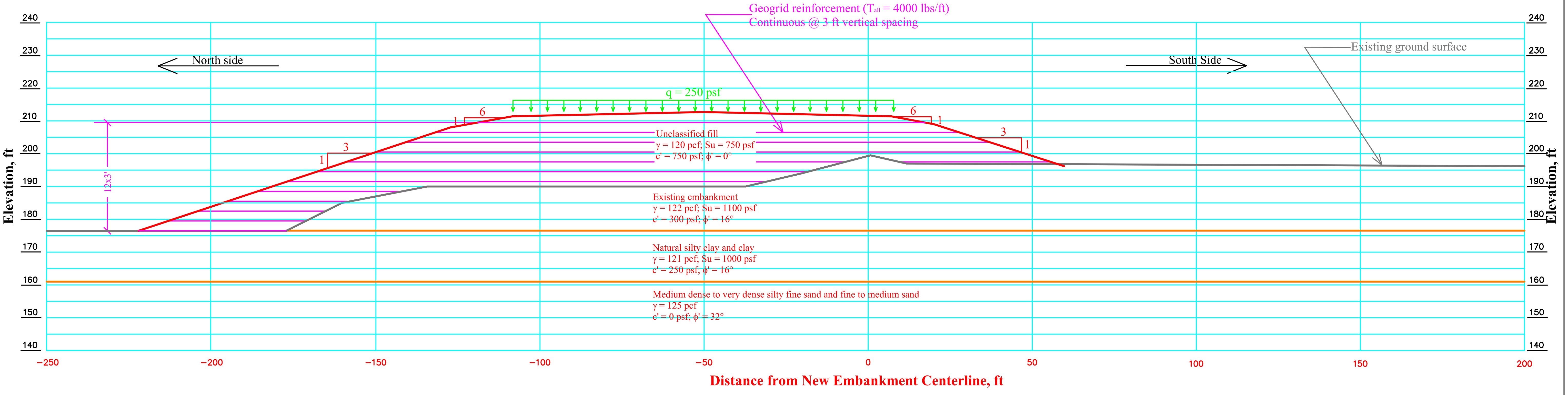
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced 3H:1V End Slope @ West Abutment – H = 35 ft
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX K



15 ft 0 15 30 ft

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

Section and Material Parameters for Stability Analysis
Reinforced Side Slopes @ West Bridge Abutment - Sta 1883+41.92
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

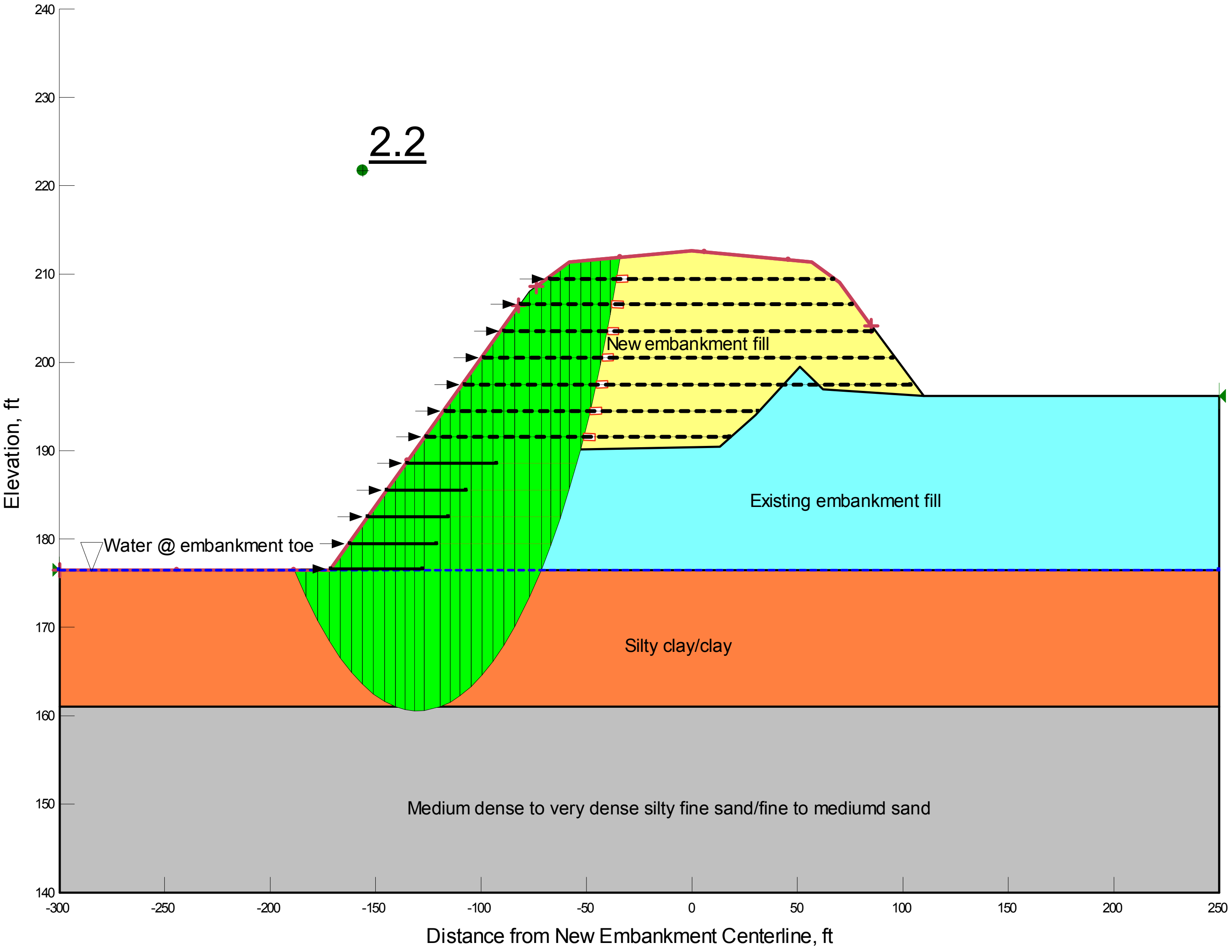
May 4, 2014

Plate

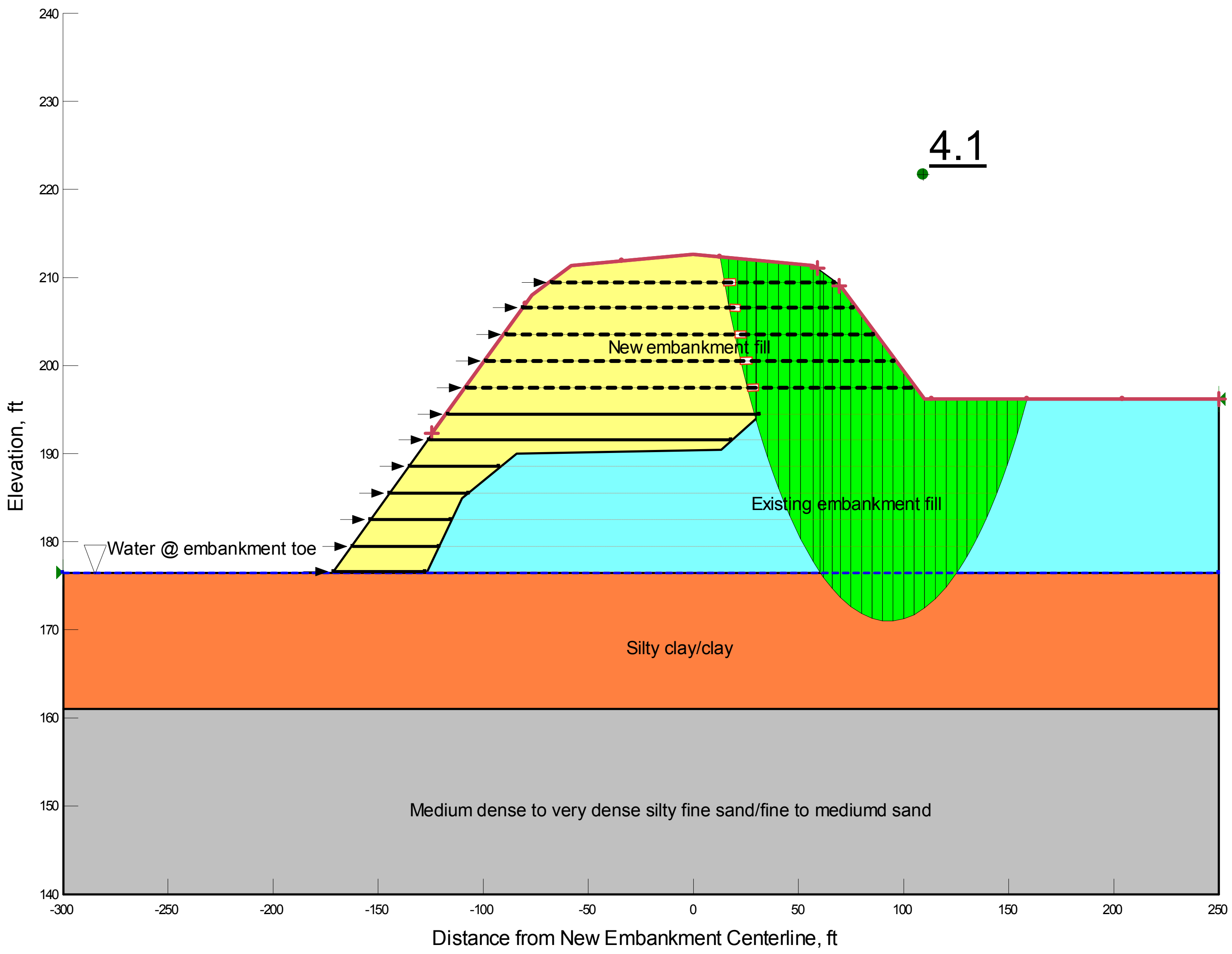
Stability Analysis Results
Reinforced Side Slopes @ West Abutment - Sta 1883+41.92
AHTD JOB BB0610:White River Str. & Apprs. (F)

Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.2
South			4.1
North	Long Term	Analysis long term groundwater @ embankment toe	1.9
South			2.0
North		Historical high water @ El 192.9	1.9
South			2.0
North	Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.4
South			1.5
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.5

* 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.

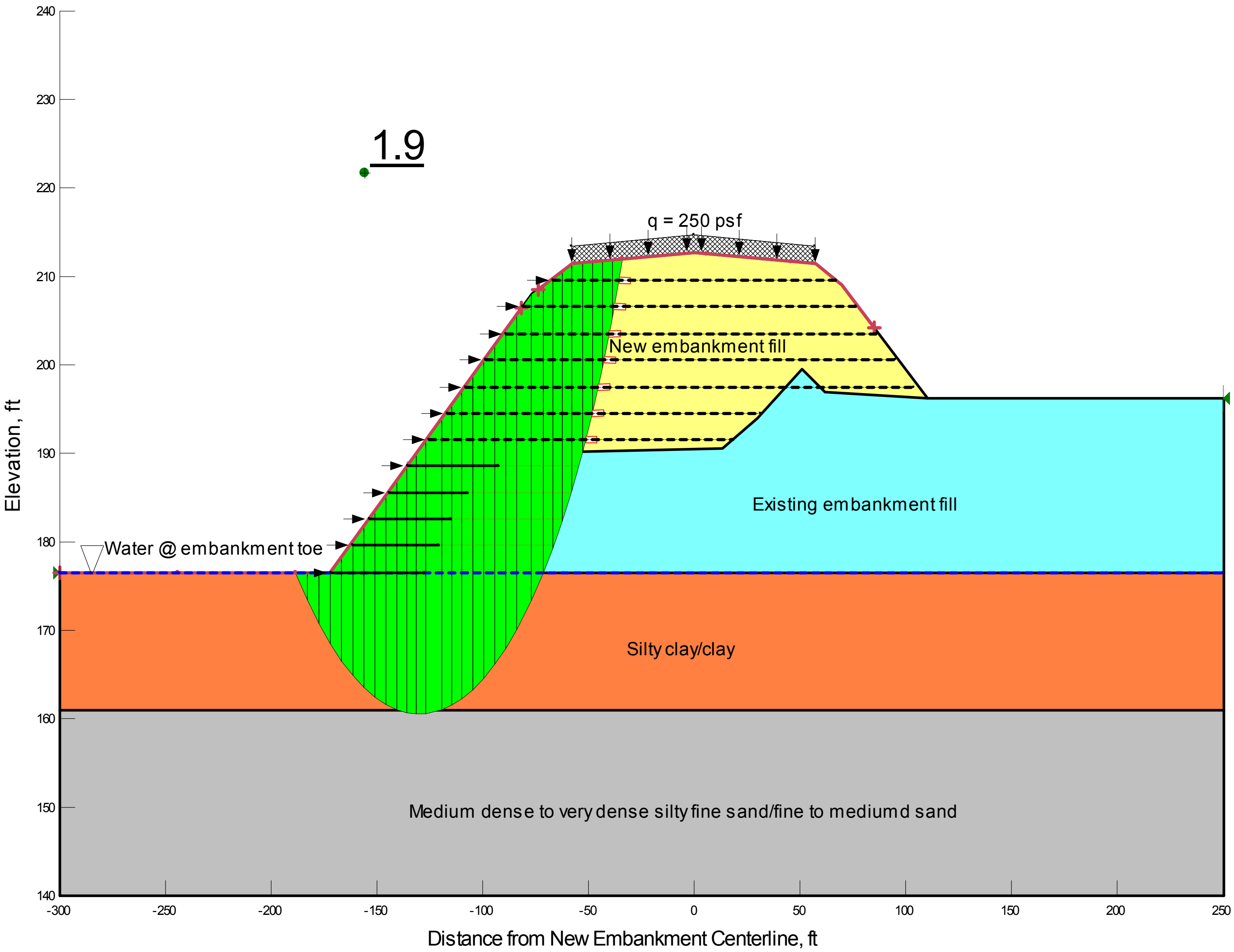


Results of Stability Analyses – End of Construction Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



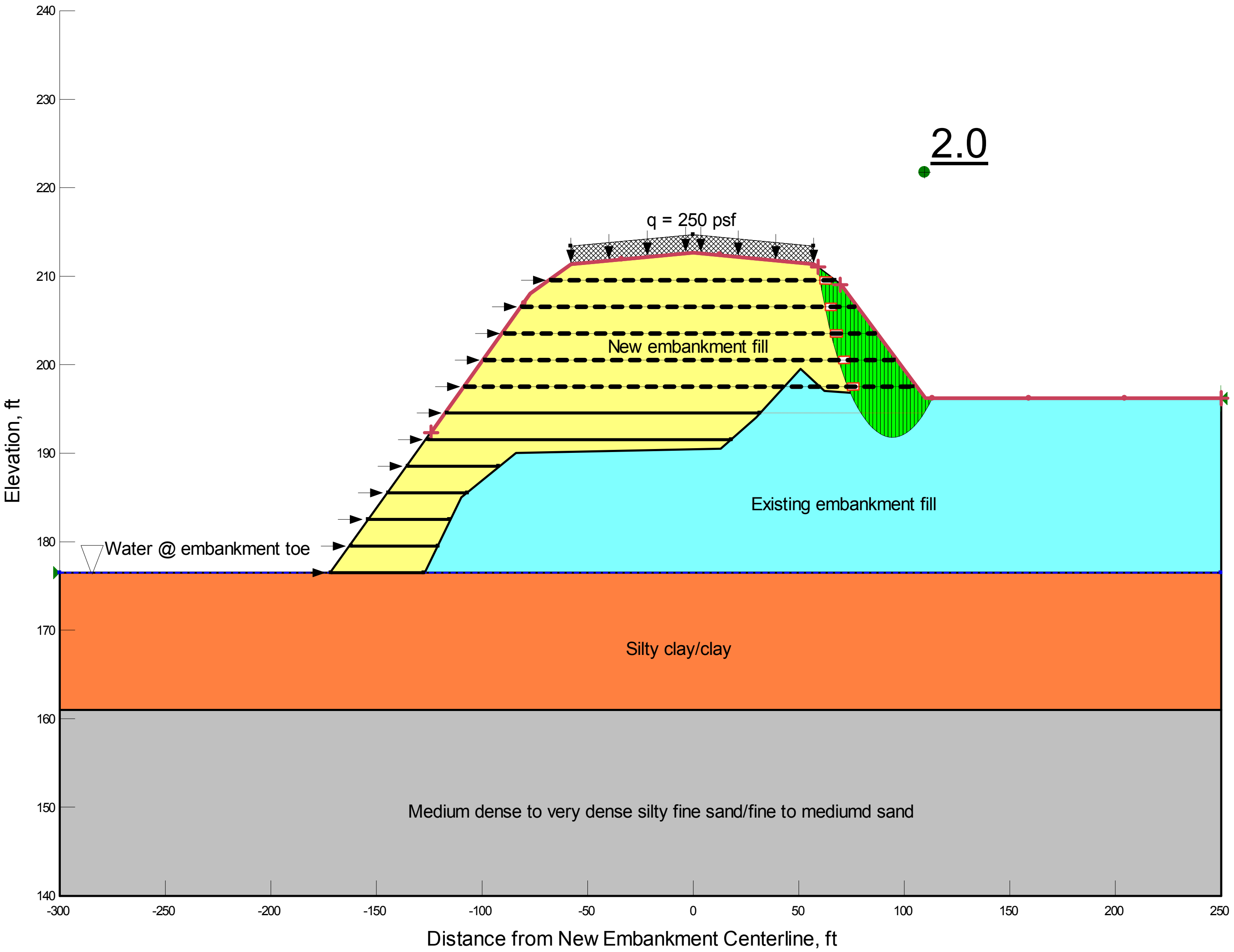
Results of Stability Analyses – End of Construction Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



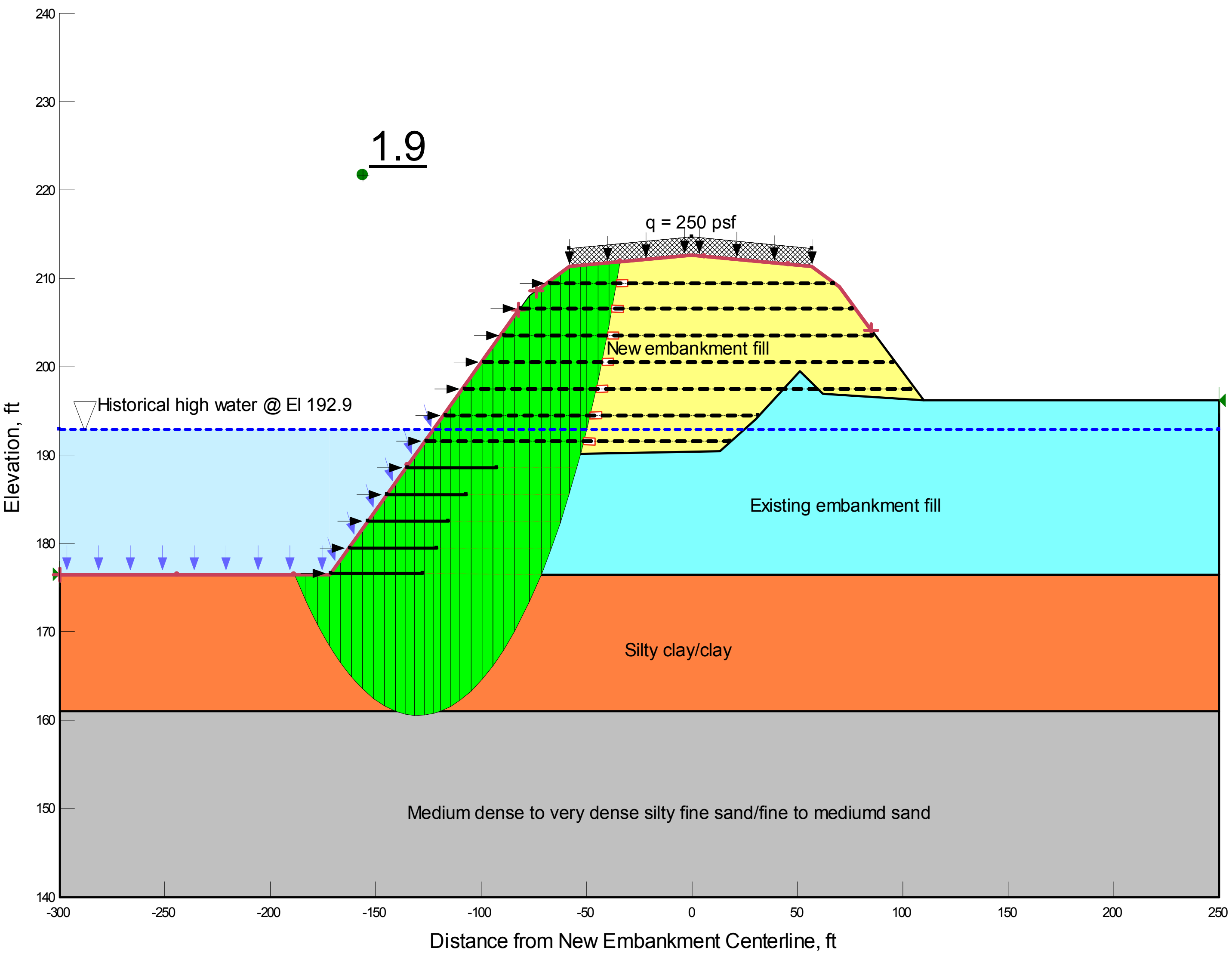
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



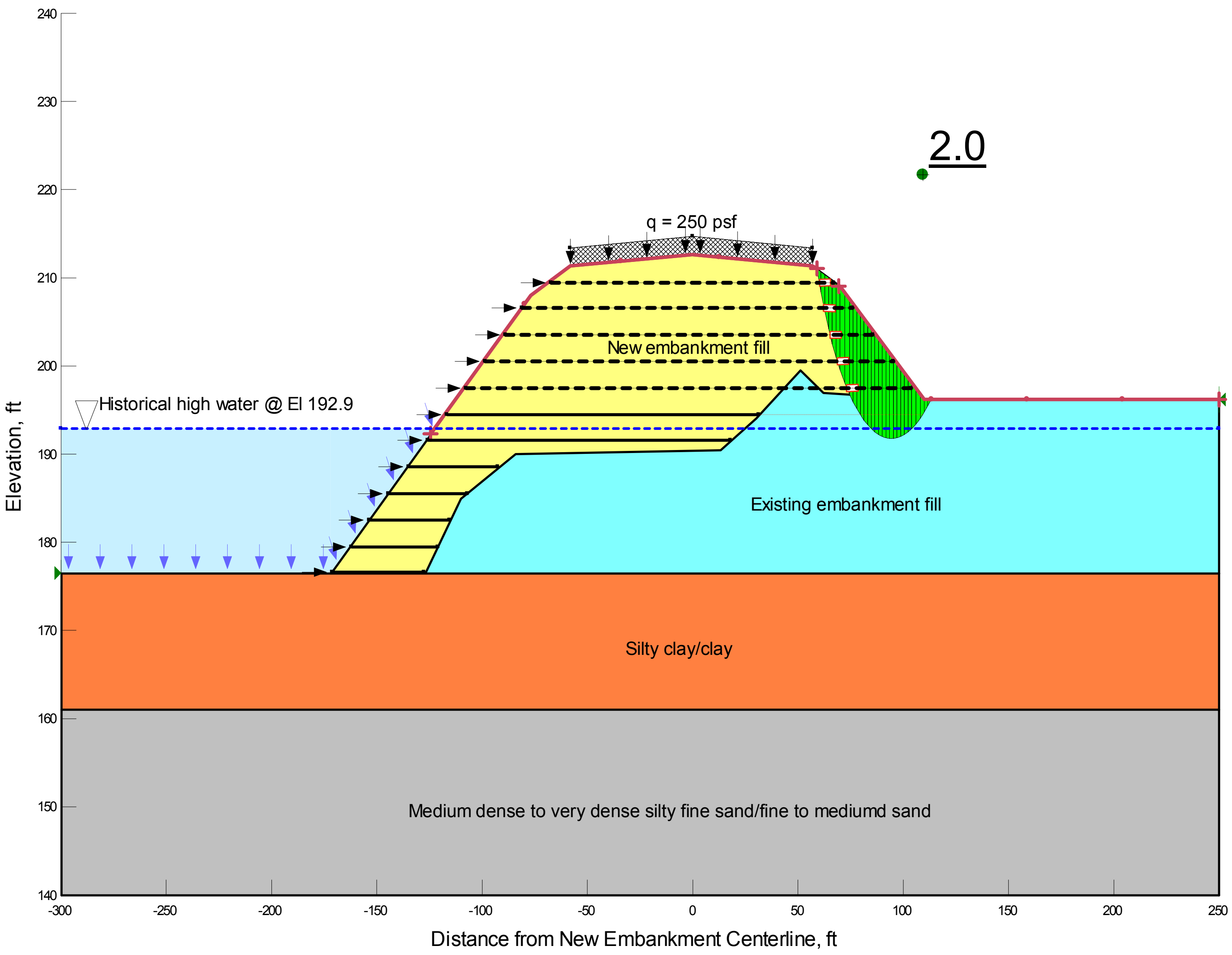
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



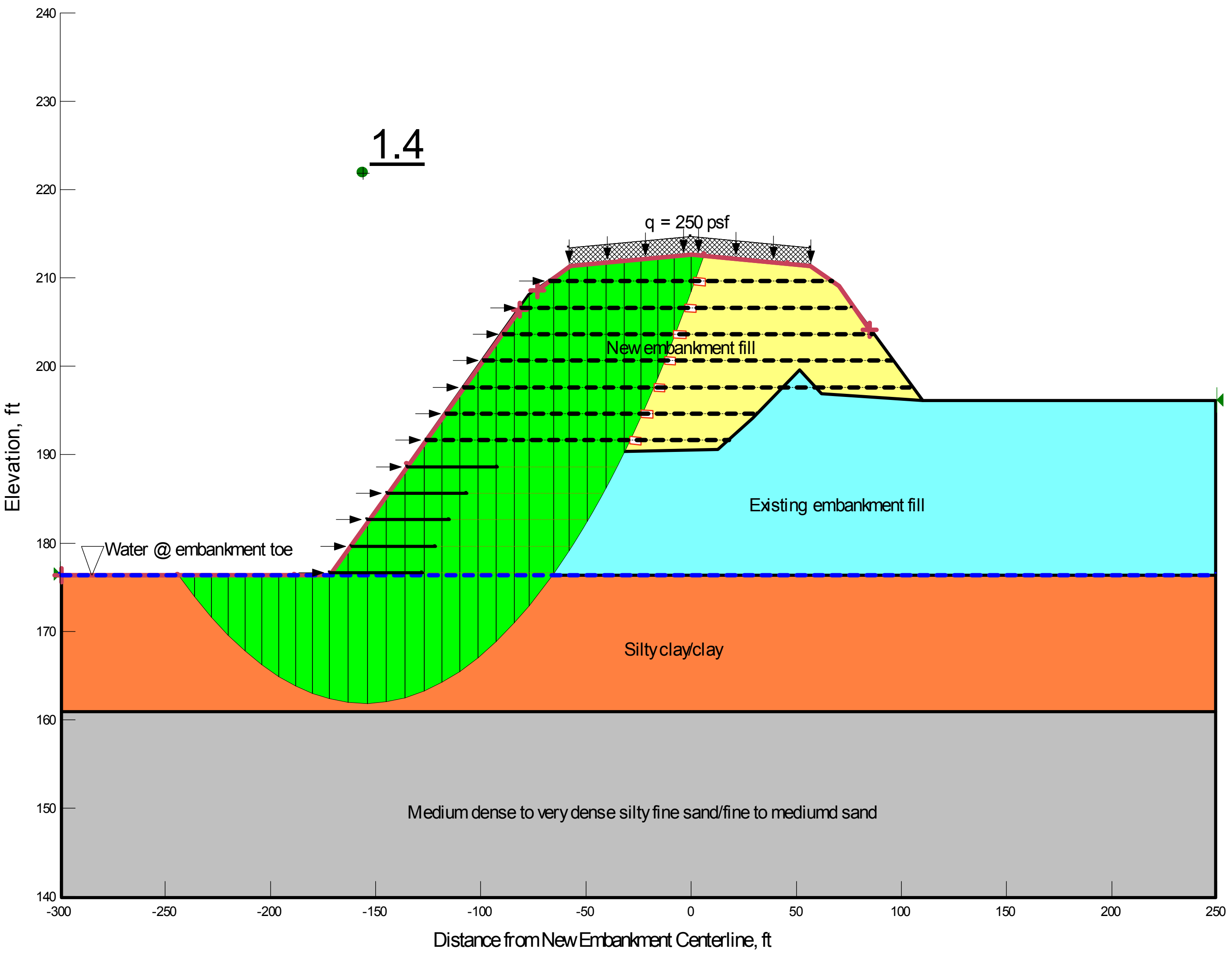
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



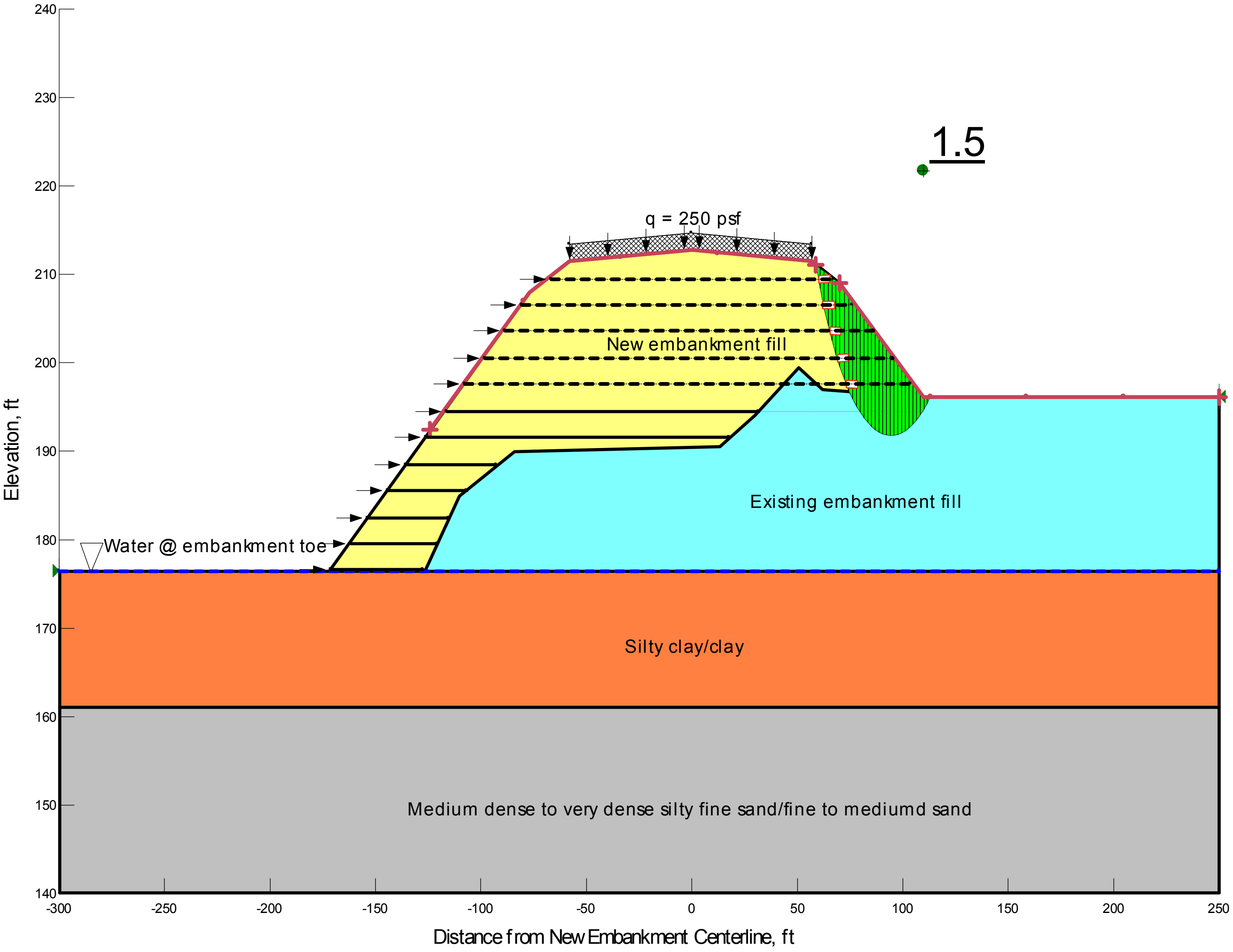
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



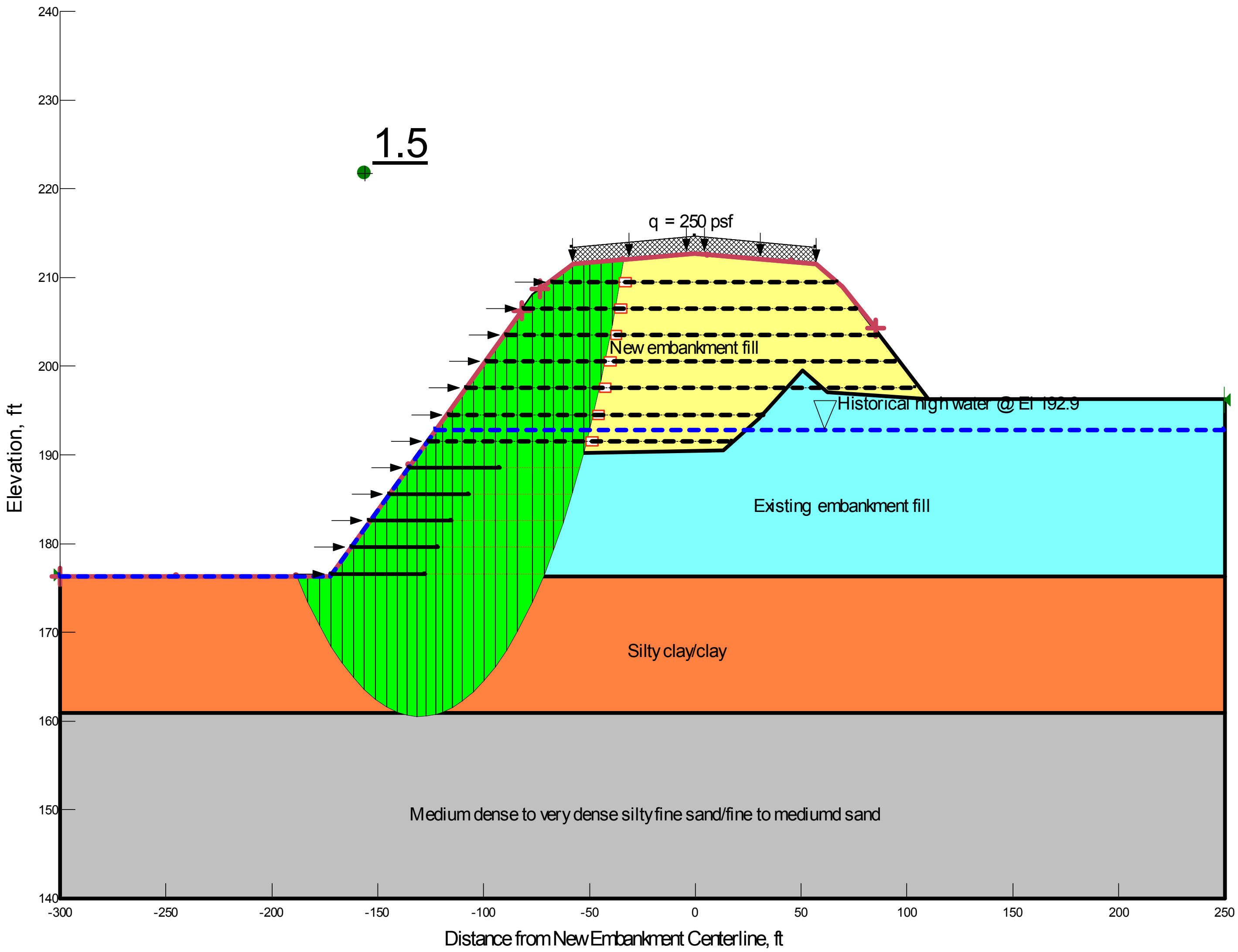
Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced South Side Slope @ West Abutment - Sta 1883+41.92
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

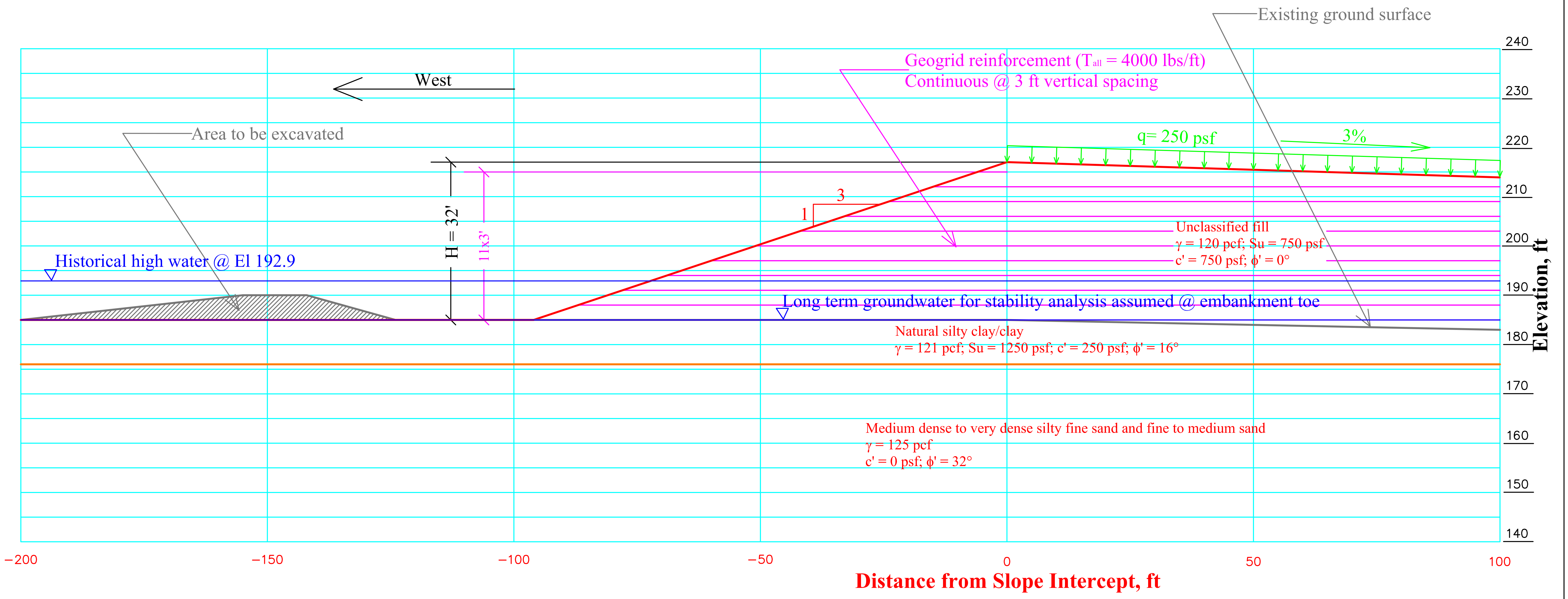
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced North Side Slope @ West Abutment - Sta 1883+41.92
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX L



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

Section and Material Parameters for Stability Analysis
Reinforced End Slope @ East Abutment
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

May 8, 2014

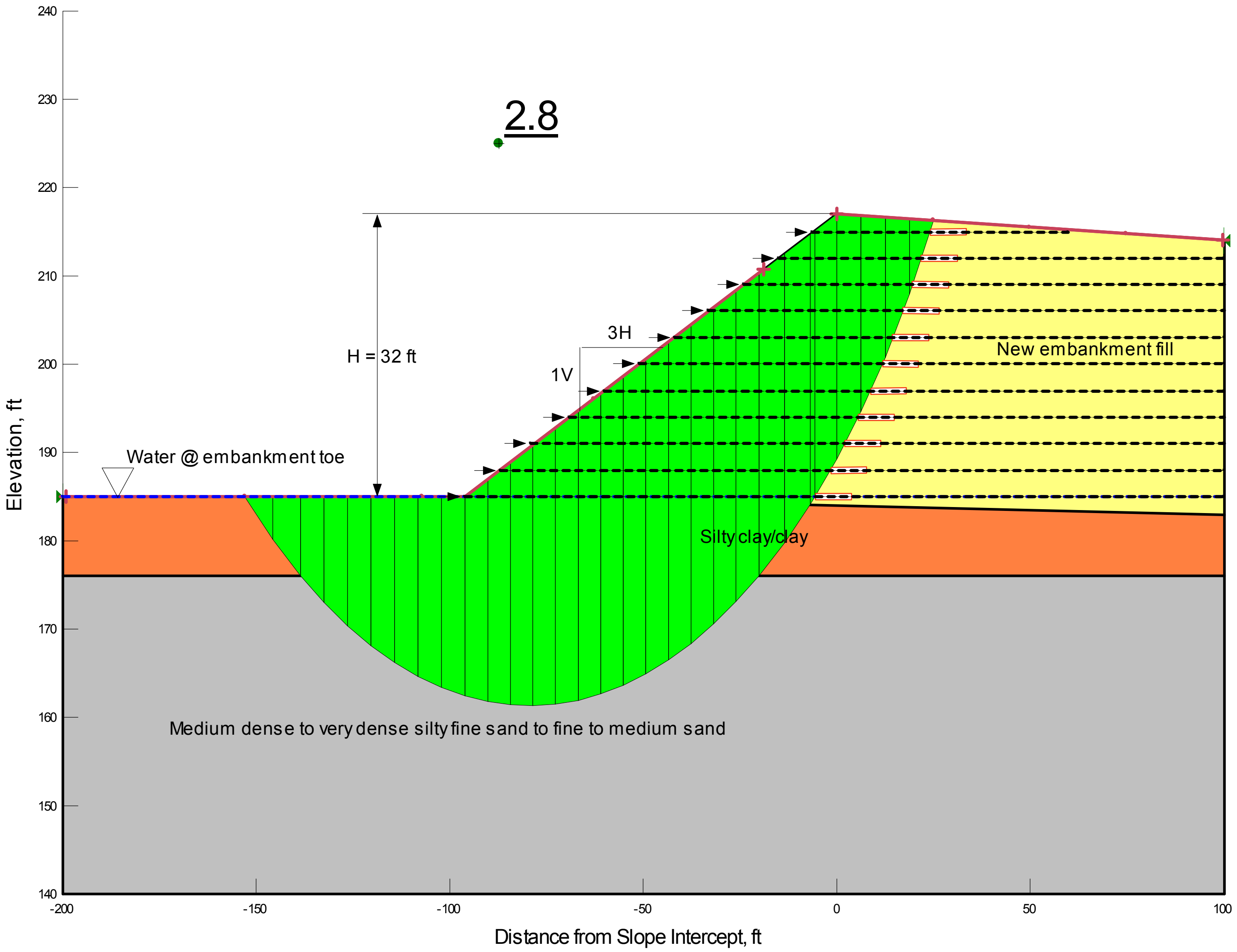
Scale: As Shown

Plate

Stability Analysis Results – East Abutment
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

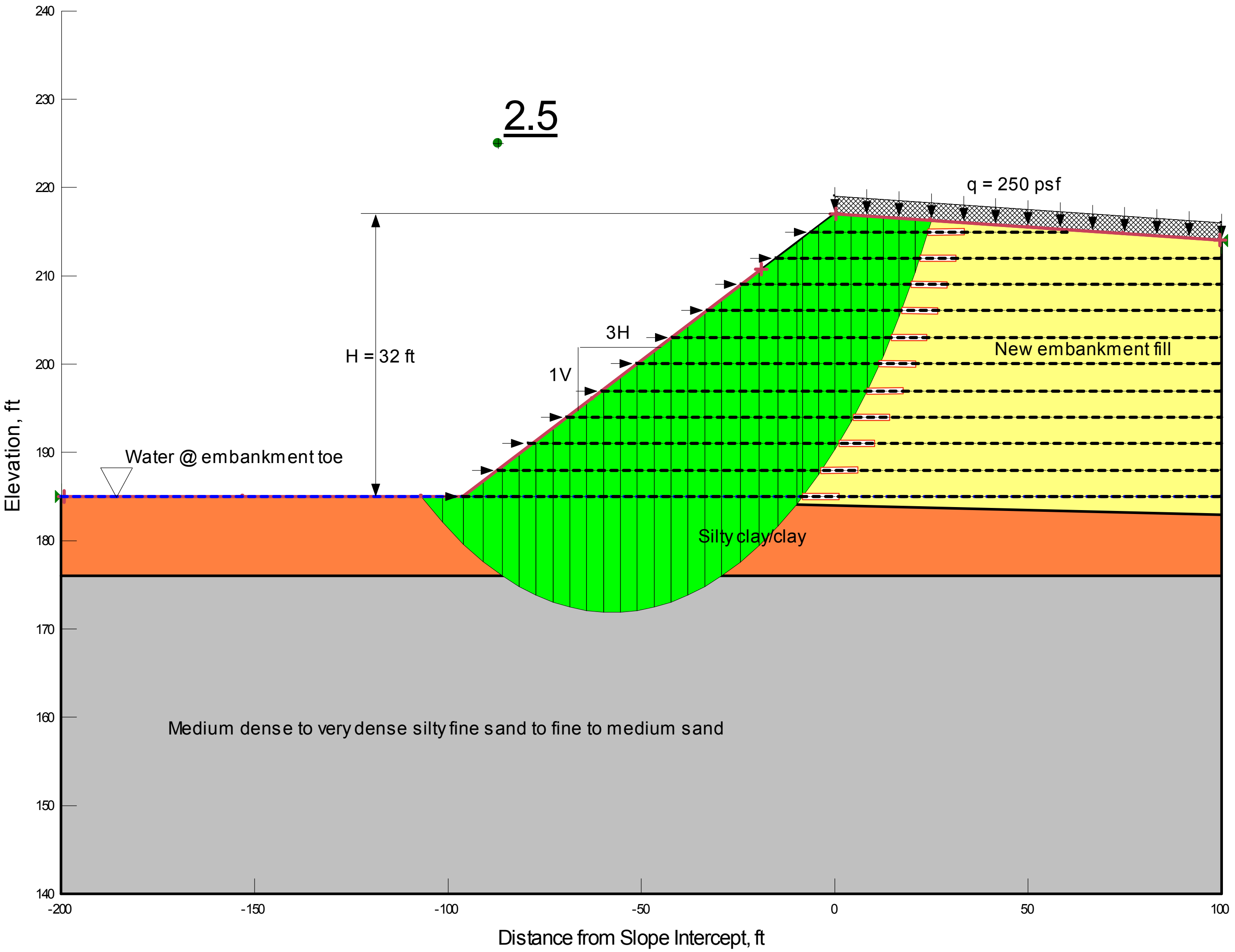
Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
End of Construction	Analysis long term groundwater @ embankment toe	2.8
Long Term	Analysis long term groundwater @ embankment toe	2.5
Long Term	Historical high water @ El 192.9	2.4
Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.8
Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.3

* 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.

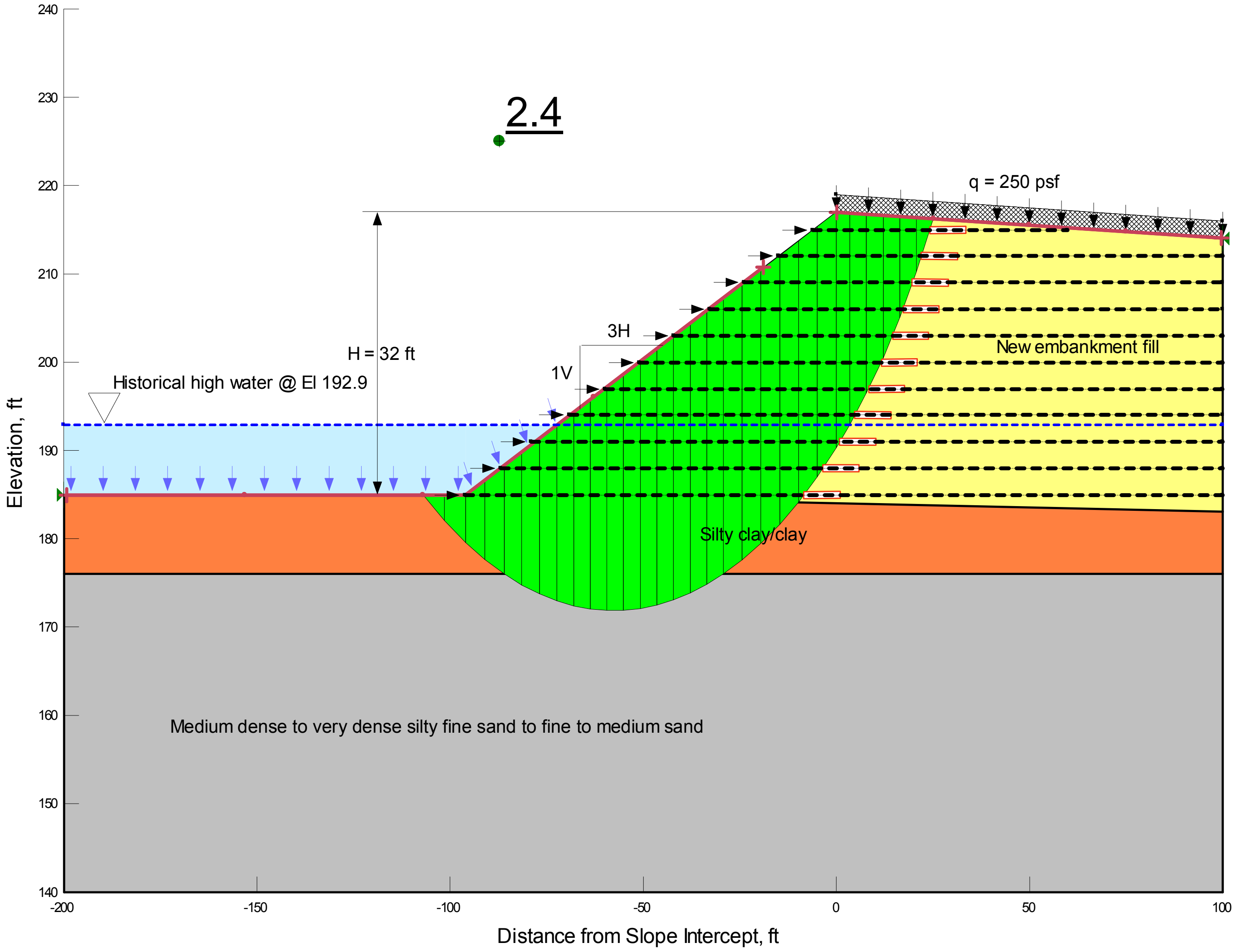


Results of Stability Analyses – End of Construction Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

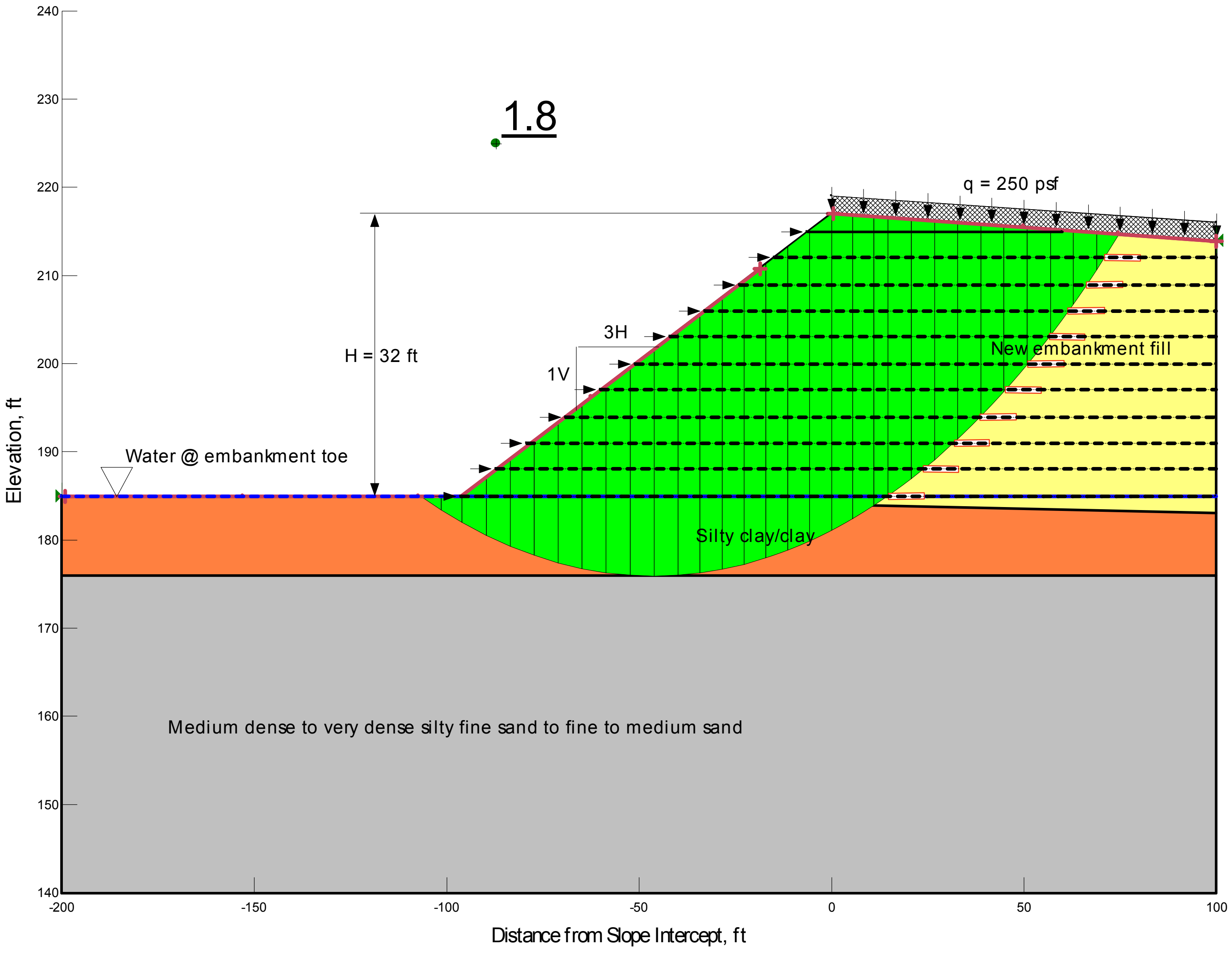


Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



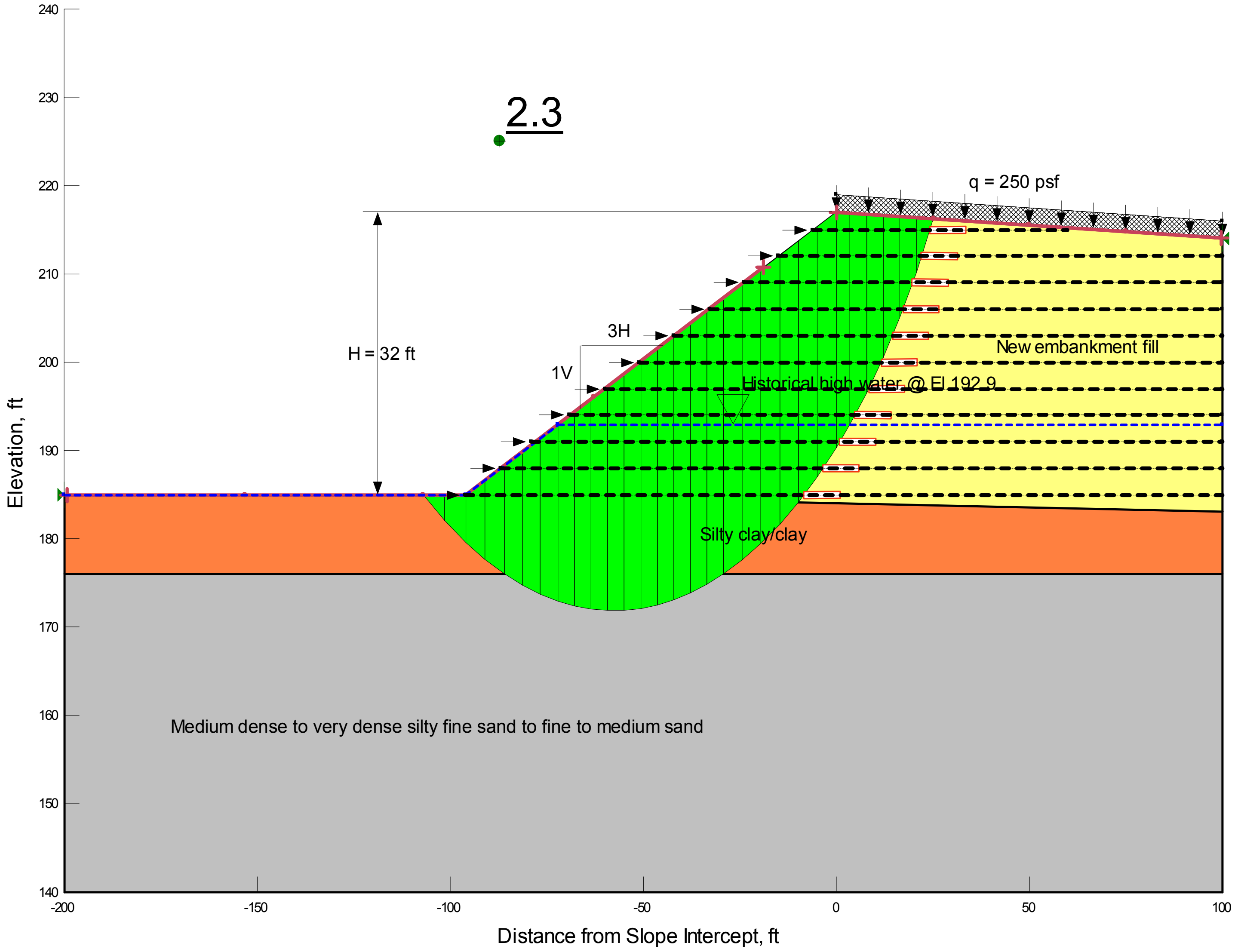
Results of Stability Analyses – Long Term Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

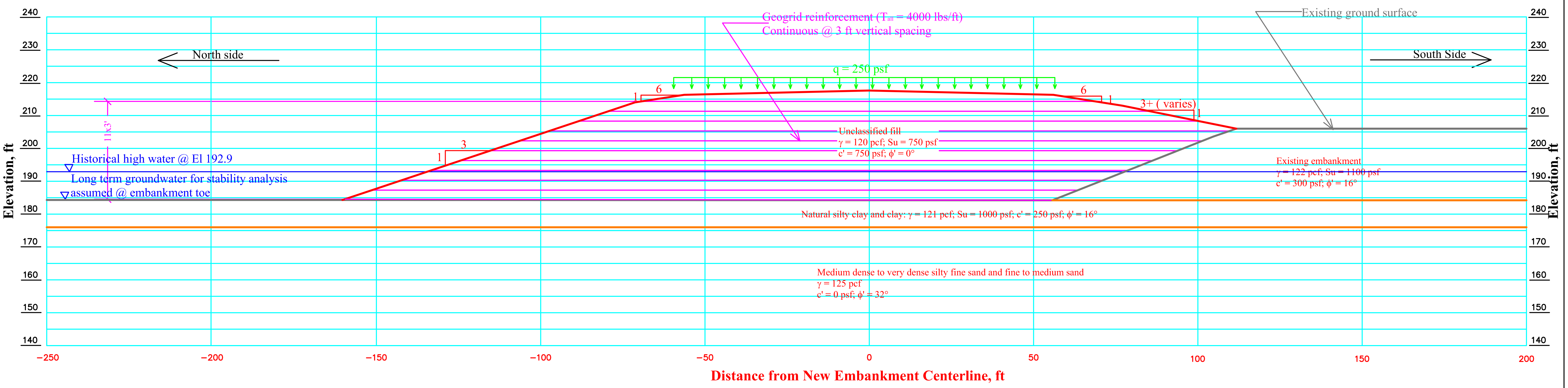
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Reinforced 3H:1V End Slope @ East Abutment – H = 32 ft
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX M



15 ft 0 15 30 ft

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

Section and Material Parameters for Stability Analysis
Reinforced Side Slopes @ East Bridge Abutment - Sta 1911+84.08
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

Scale: As Shown

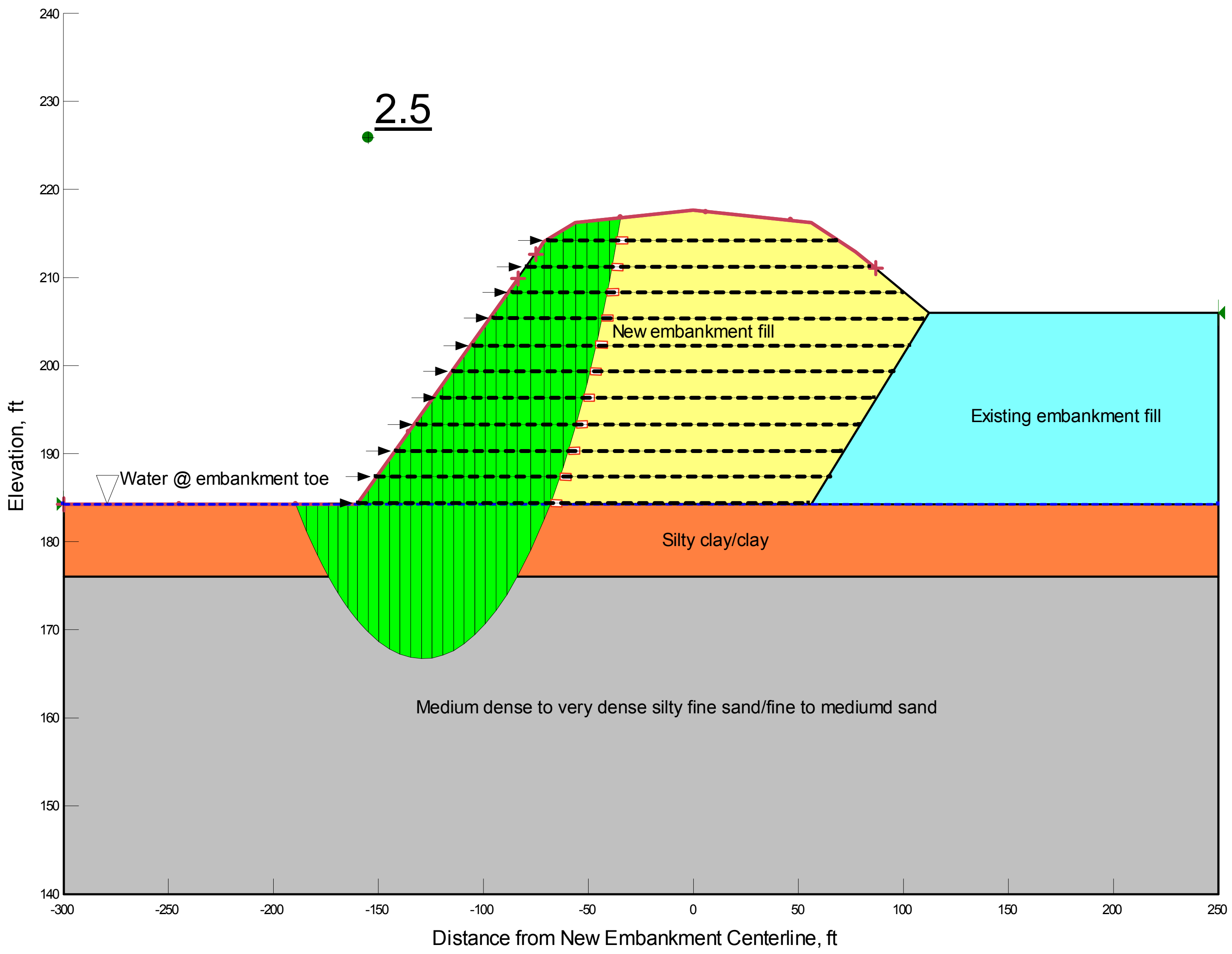
May 7, 2014

Plate

Stability Analysis Results
Reinforced Side Slopes @ East Abutment - Sta 1911+84.08
AHTD JOB BB0610:White River Str. & Apprs. (F)

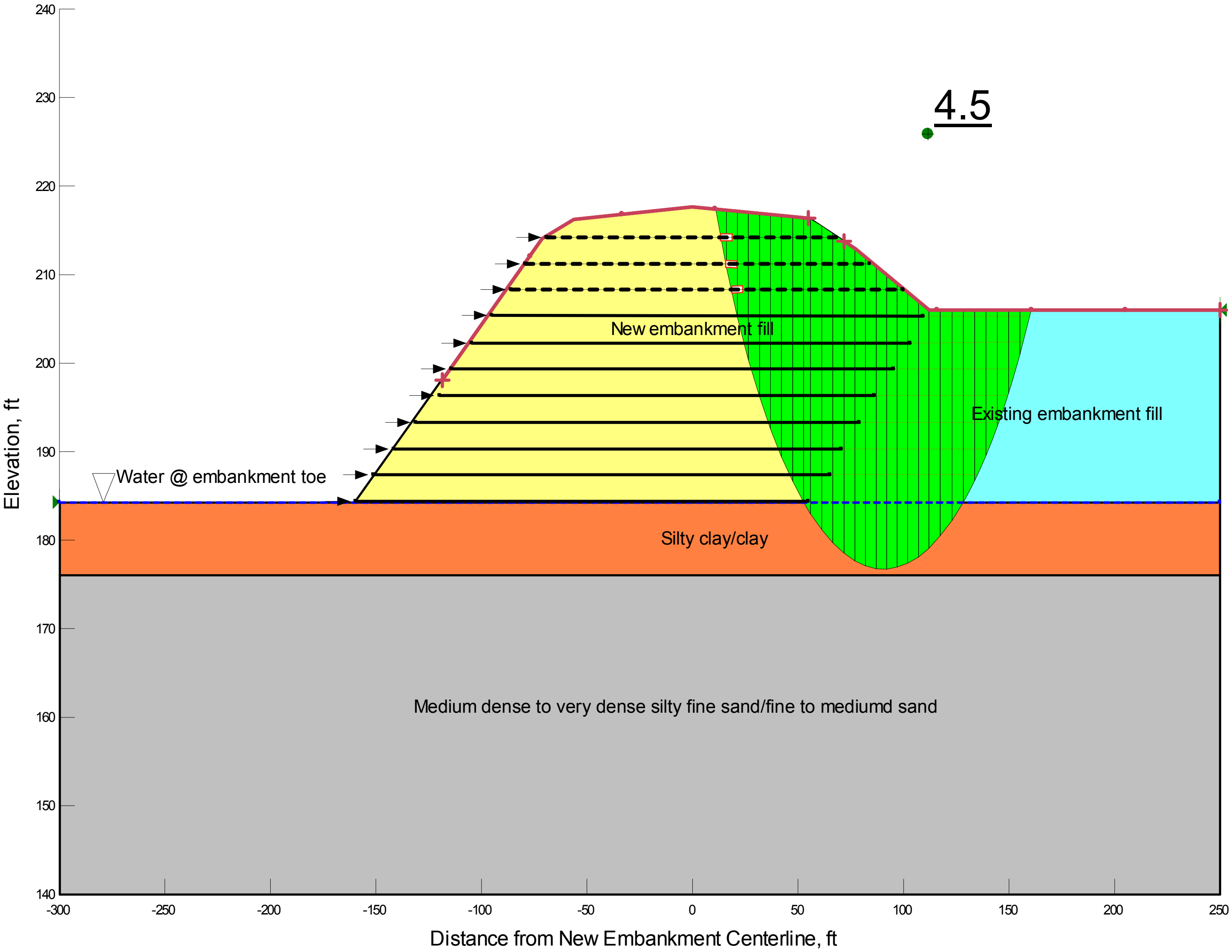
Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.5
South			4.5
North	Long Term	Analysis long term groundwater @ embankment toe	2.3
South			3.4
North		Historical high water @ El 192.9	2.3
South			3.4
North	Seismic ($k_h = 0.5A_S = 0.10$)*	Analysis long term groundwater @ embankment toe	1.7
South			1.9
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	2.1

* 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.



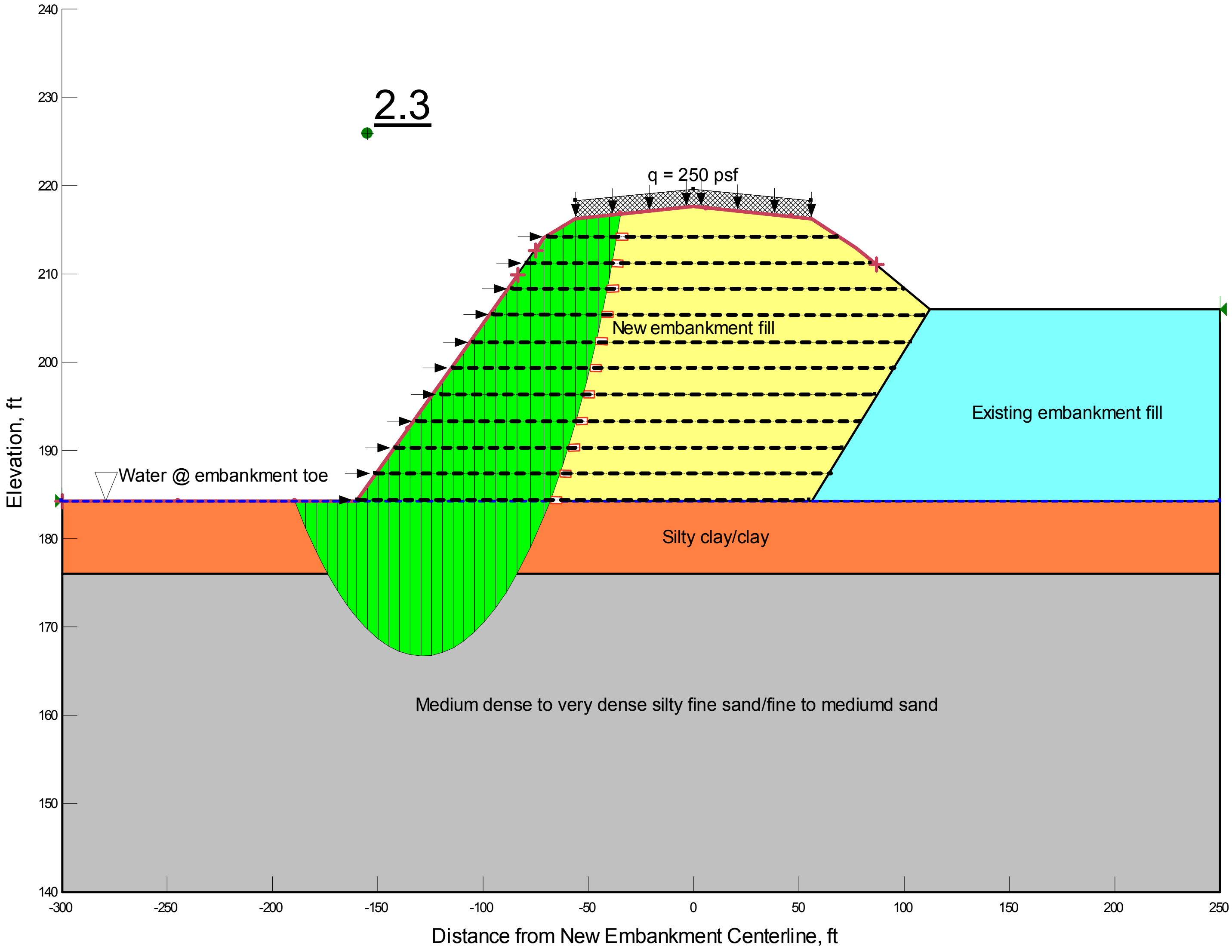
Results of Stability Analyses – End of Construction Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



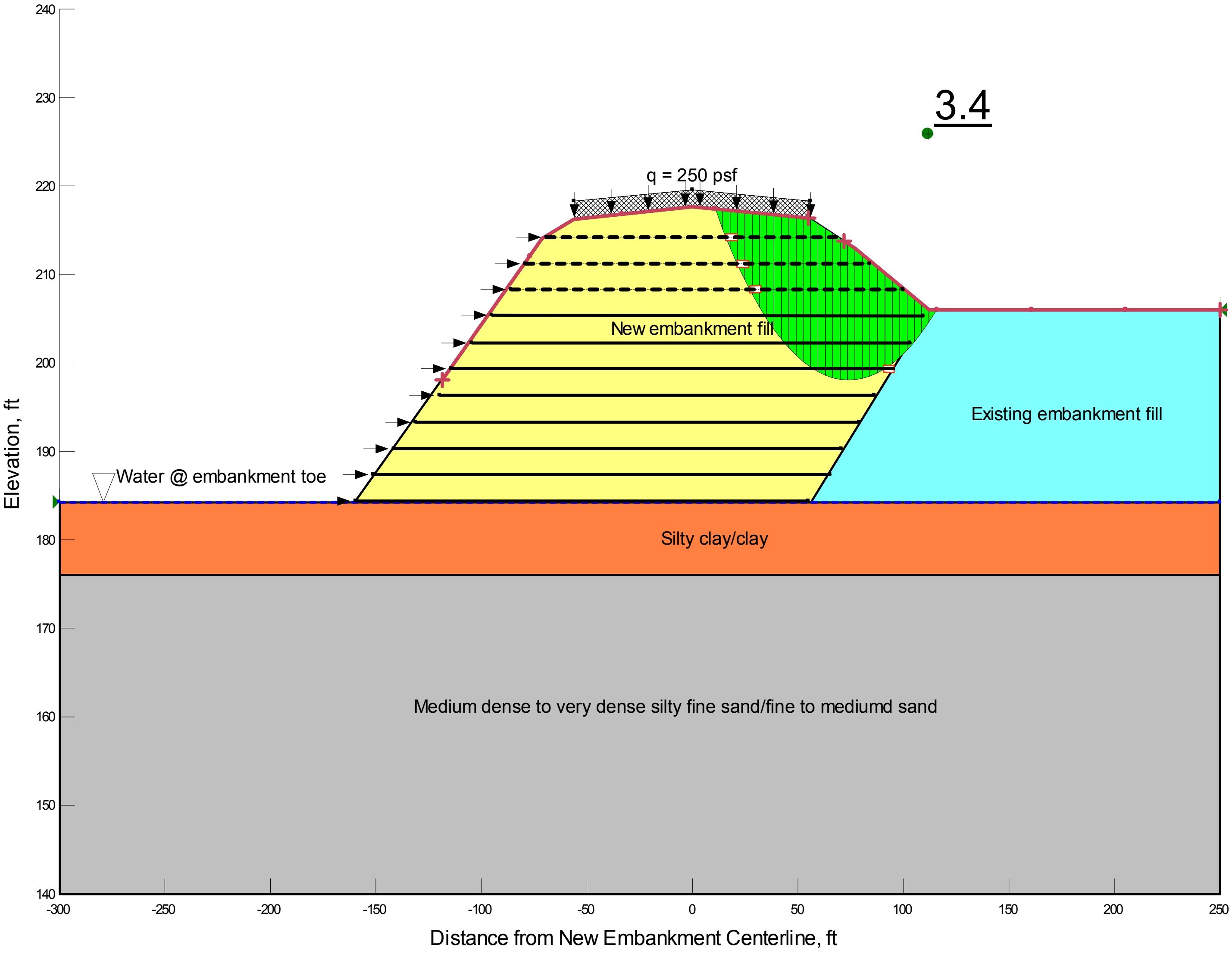
Results of Stability Analyses – End of Construction Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



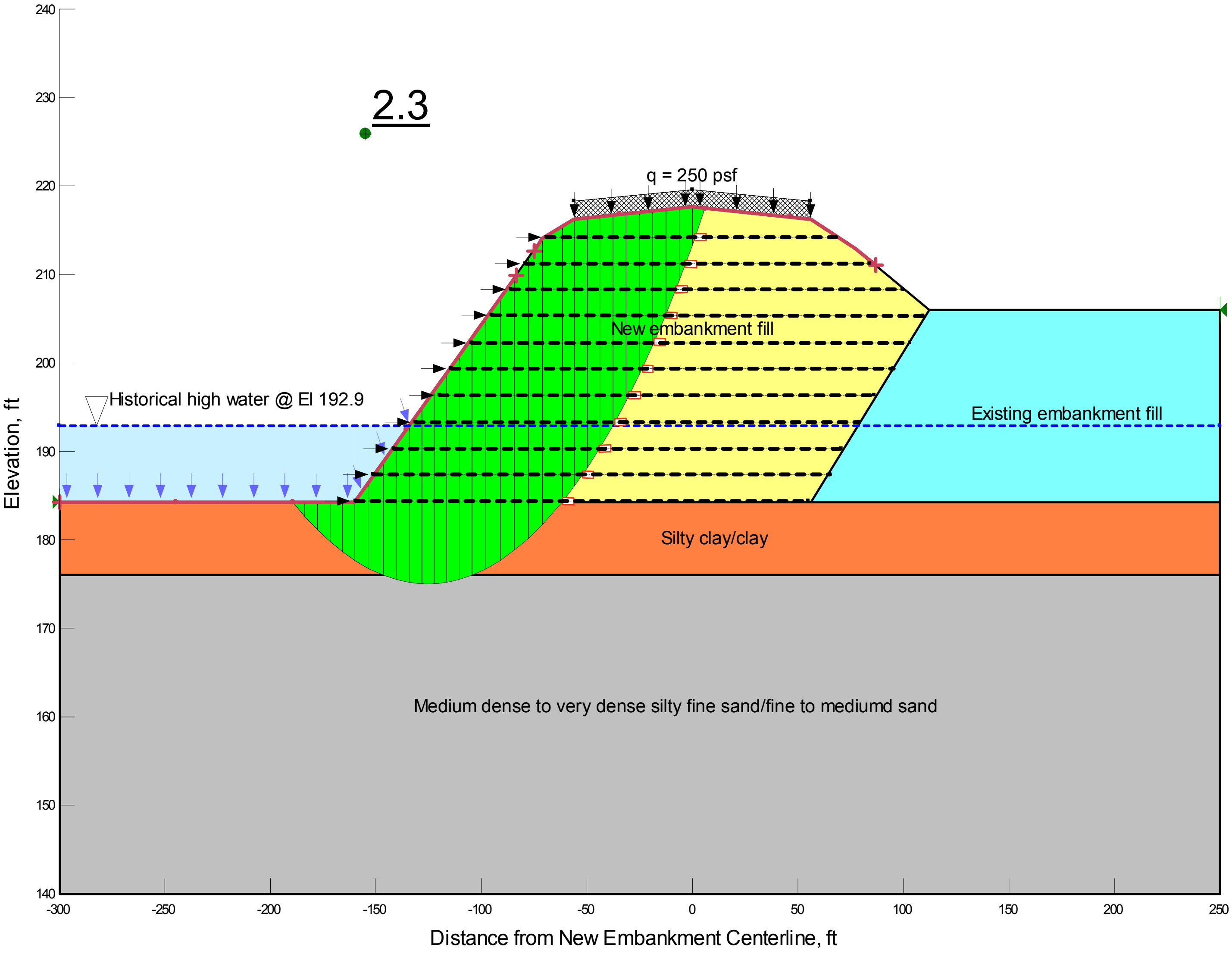
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



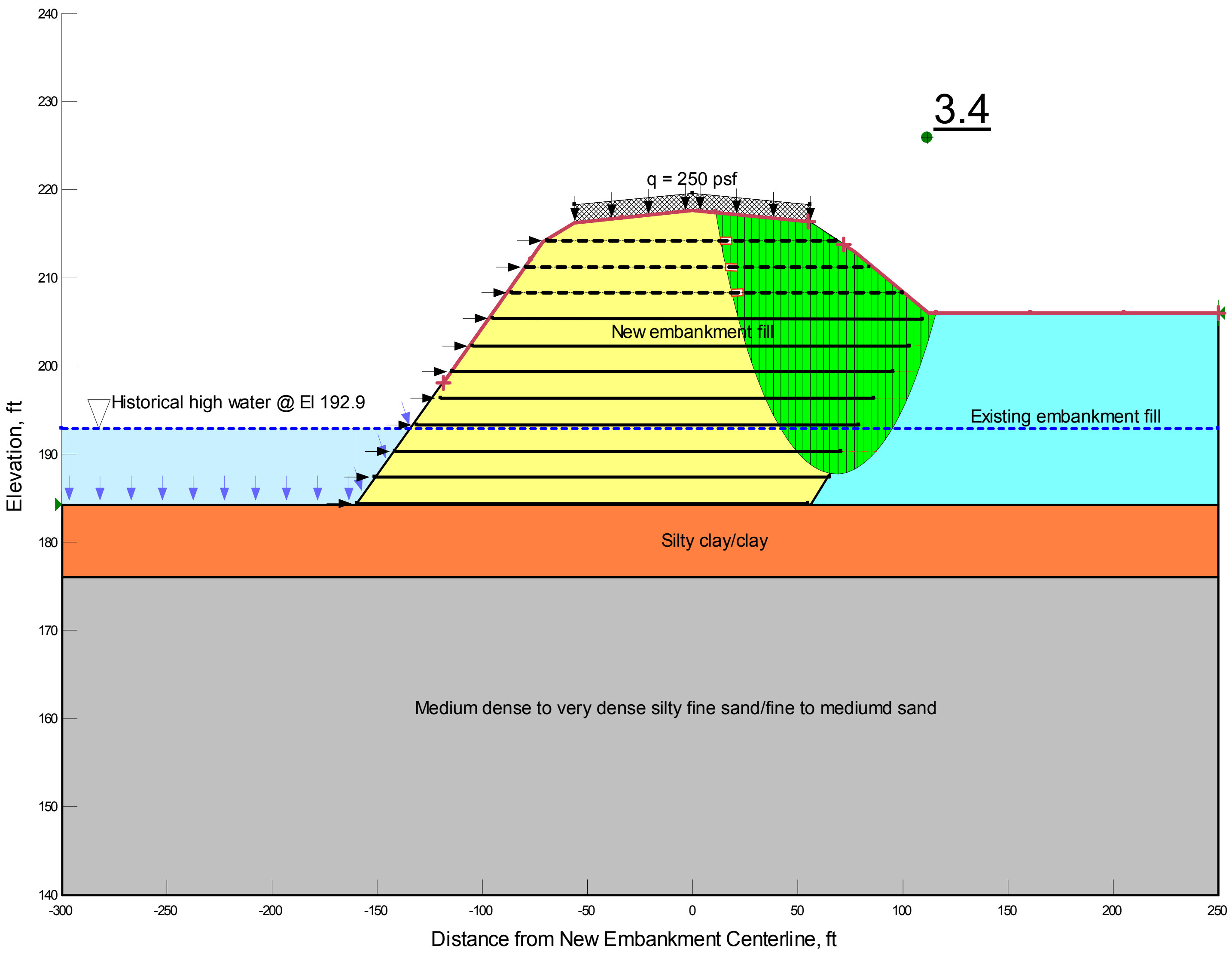
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



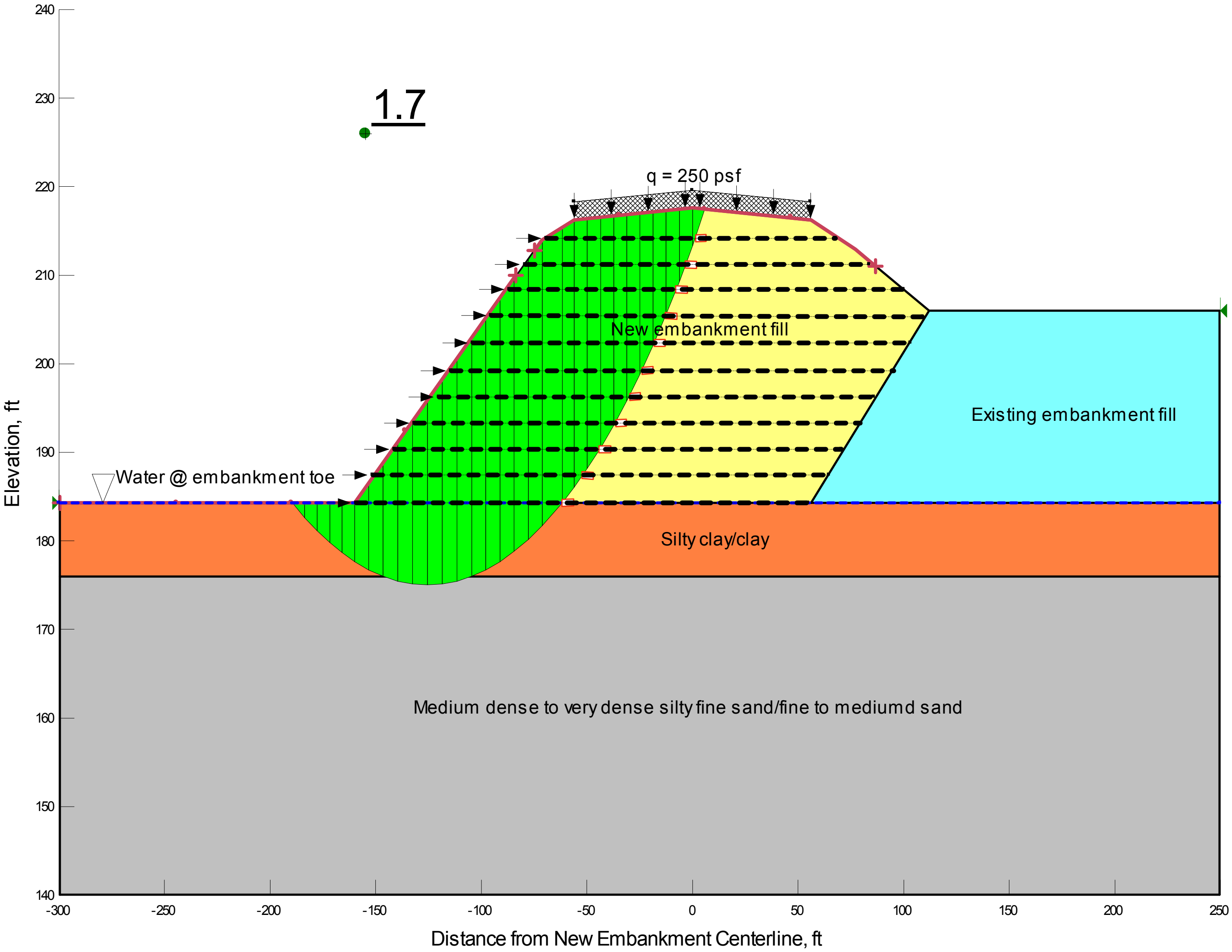
Results of Stability Analyses – Long Term Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



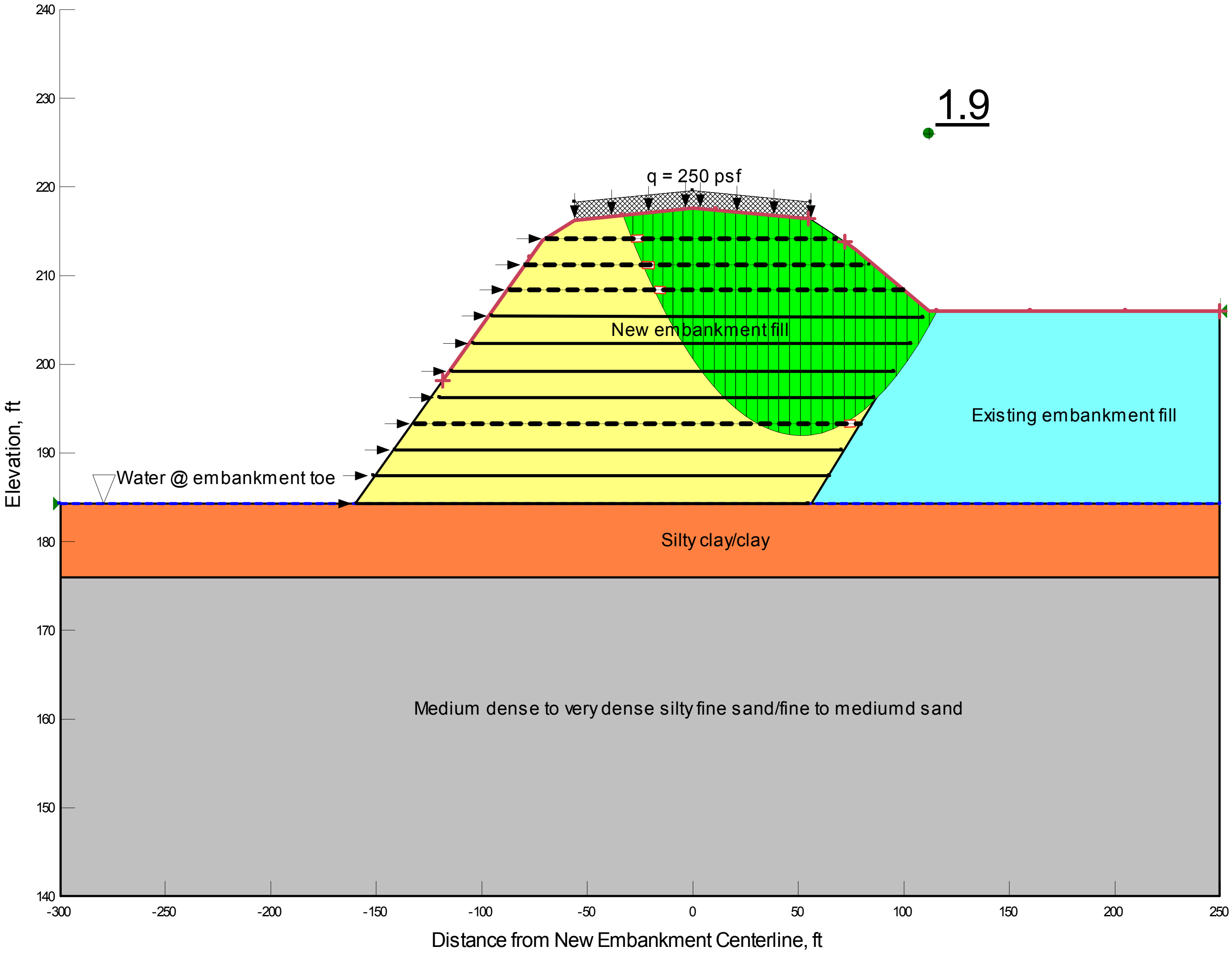
Results of Stability Analyses – Long Term Condition
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



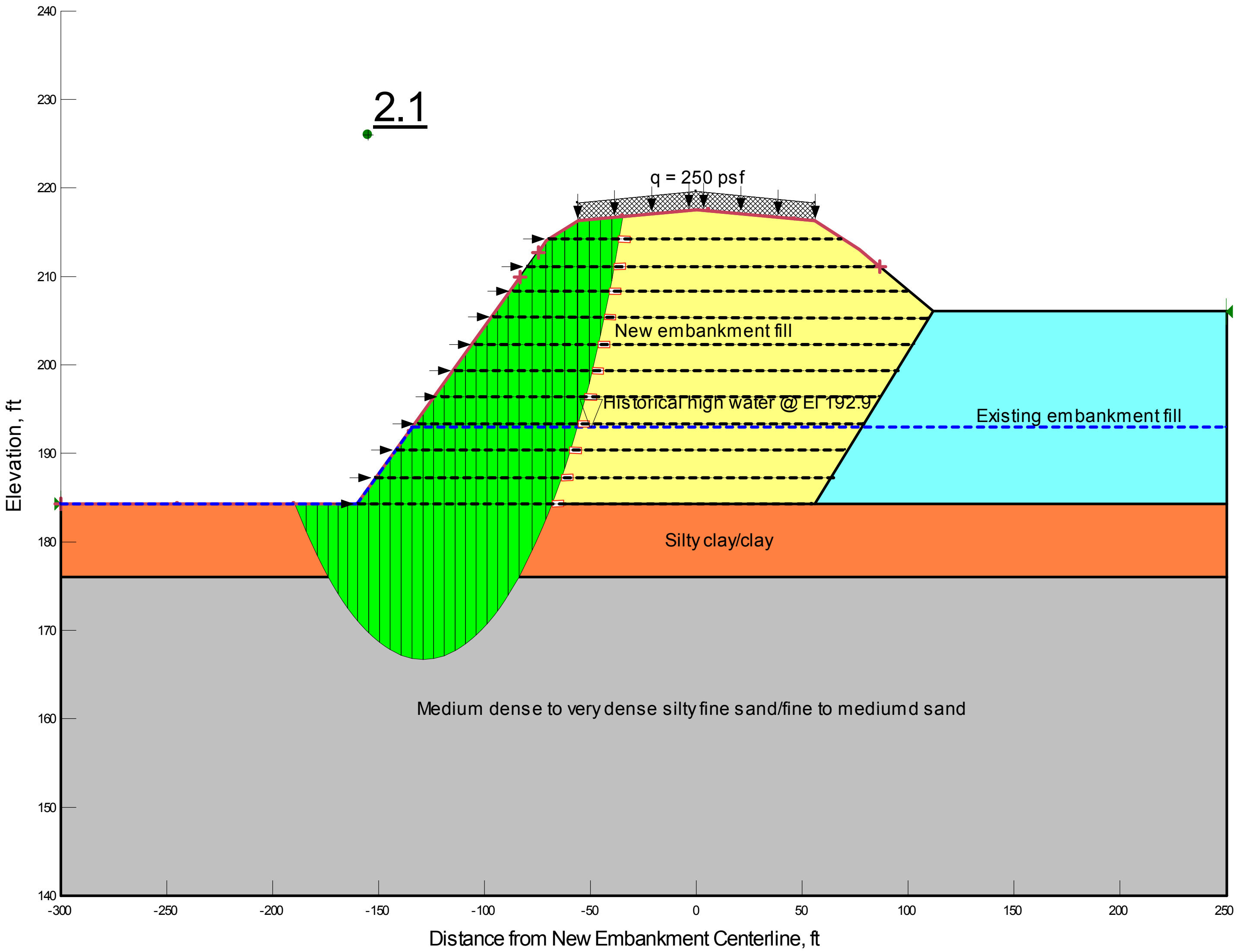
Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 0.5A_S = 0.10$)
Reinforced South Side Slope @ East Abutment - Sta 1911+84.08
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

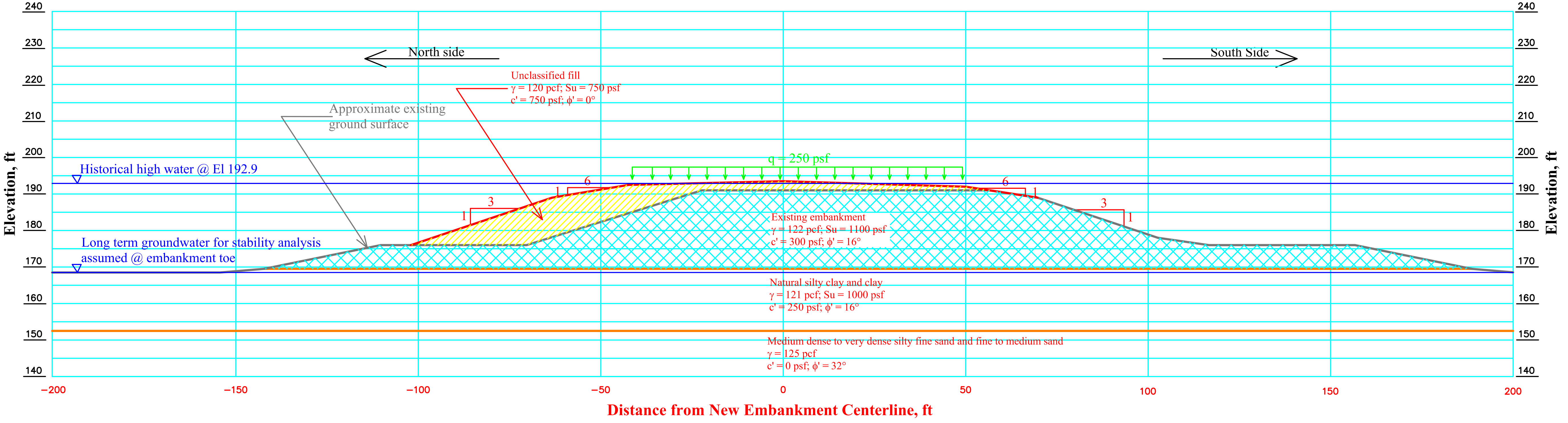
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers

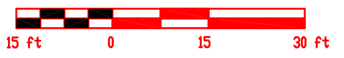


Results of Stability Analyses – Rapid Drawdown Condition
Reinforced North Side Slope @ East Abutment - Sta 1911+84.08
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX N





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

Section and Material Parameters for Stability Analysis
Unreinforced Side Slopes @ Sta 1875+00
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

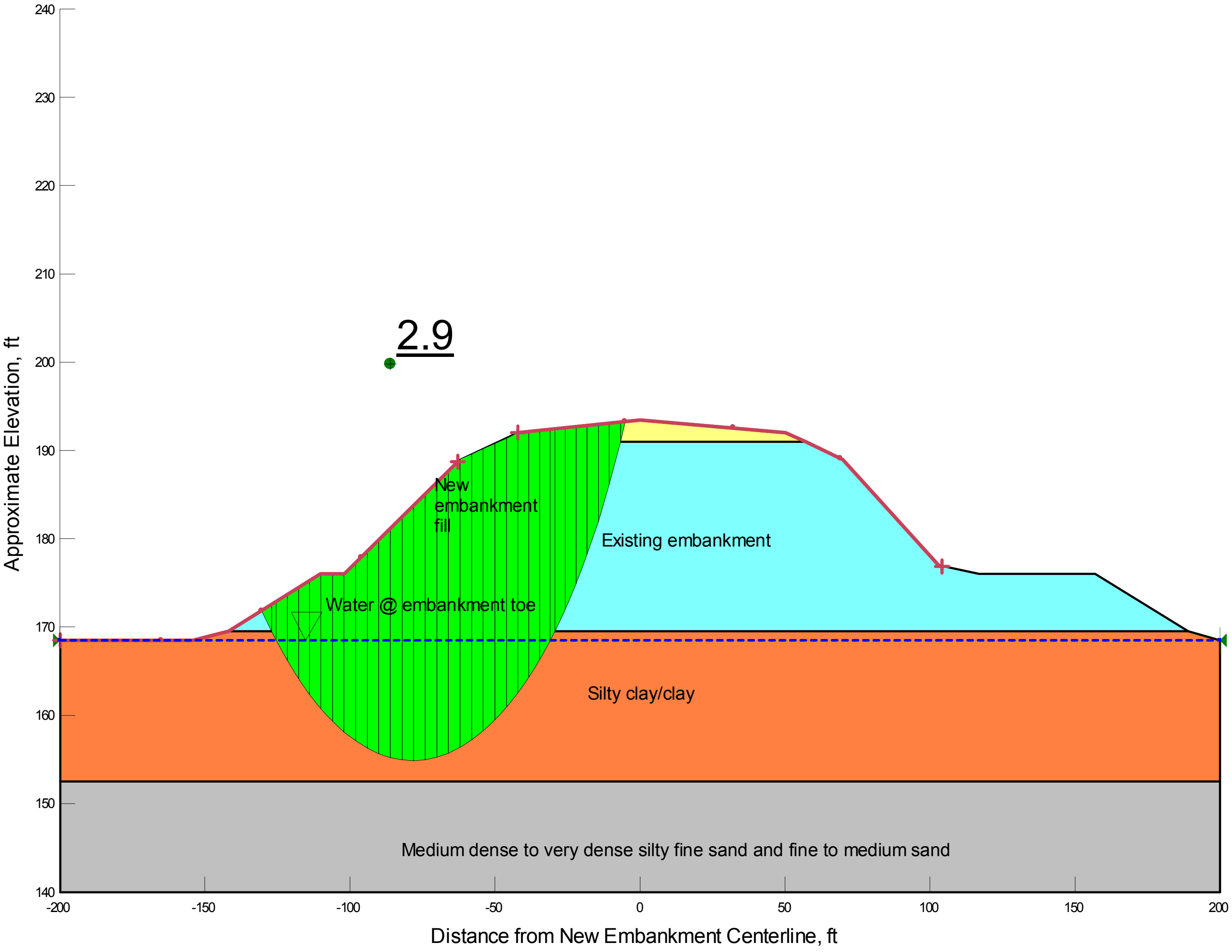
Scale: As Shown

May 2, 2014

Plate

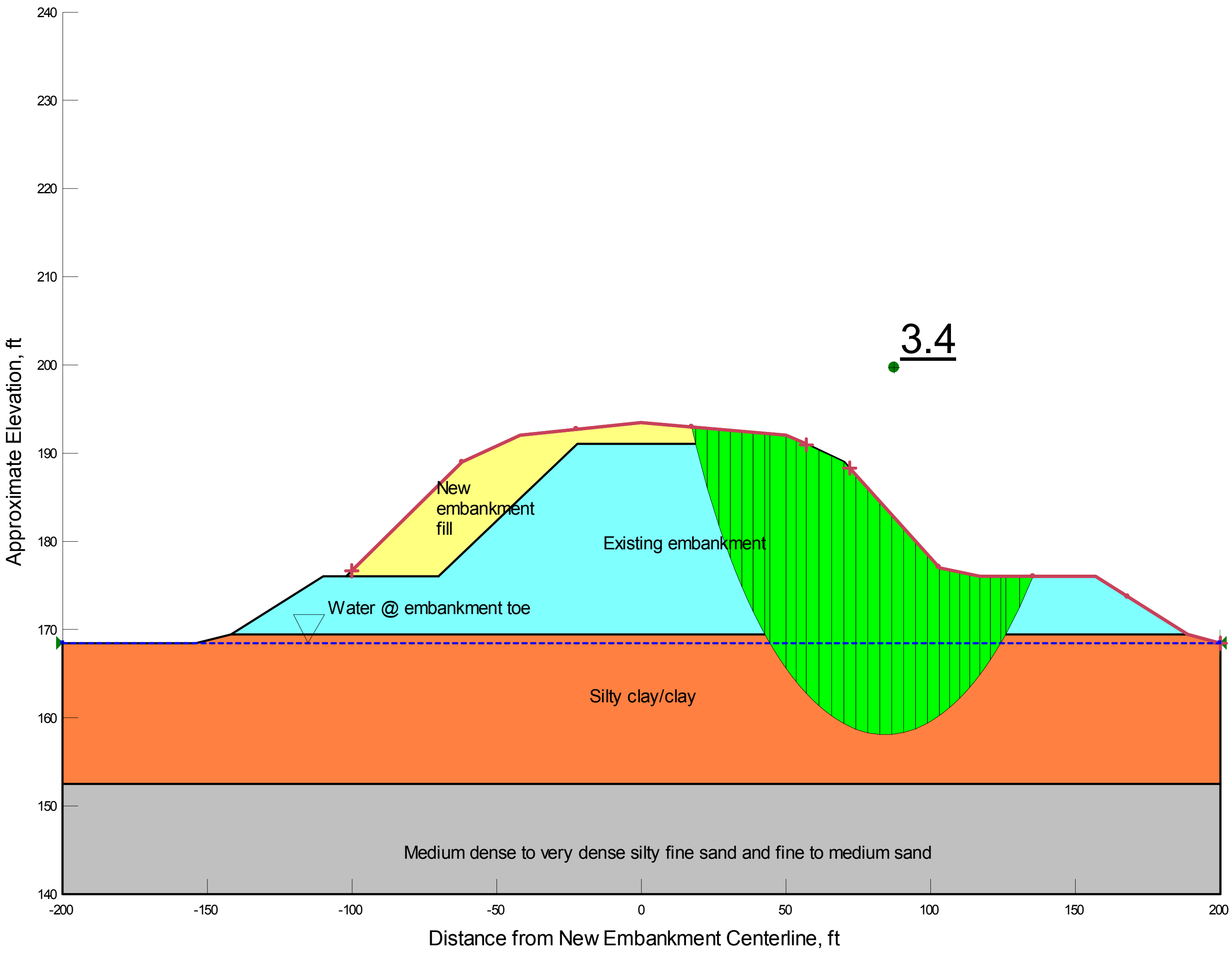
Stability Analysis Results
Unreinforced Side Slopes @ Sta 1875+00
AHTD JOB BB0610:White River Str. & Apprs. (F)

Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.9
South			3.4
North	Long Term	Analysis long term groundwater @ embankment toe	2.2
South			2.4
North		Historical high water @ El 192.9	3.0
South			3.2
North	Seismic ($k_h = 1.0A_S = 0.19$)	Analysis long term groundwater @ embankment toe	1.1
South			1.2
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.6
South			1.7



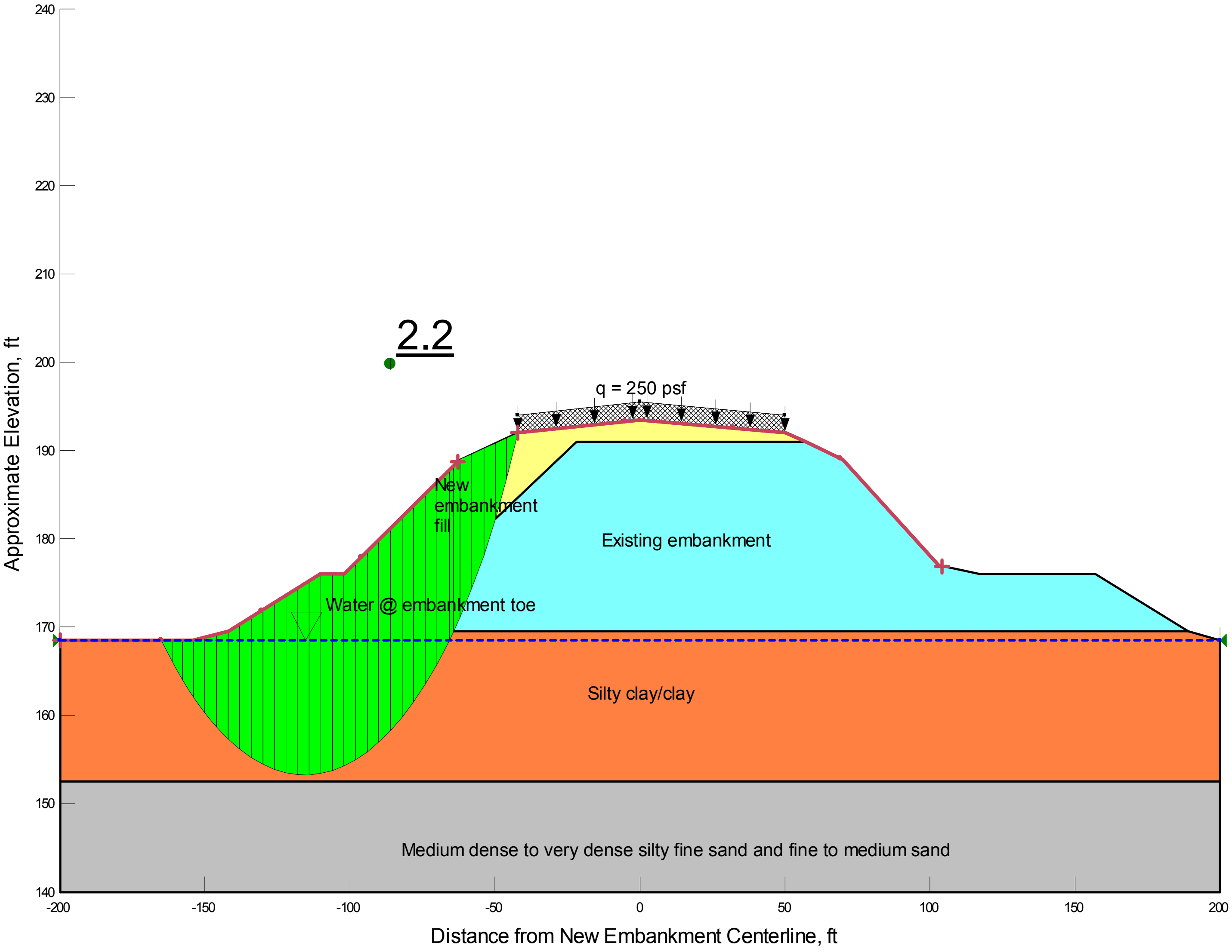
Results of Stability Analyses – End of Construction Condition
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



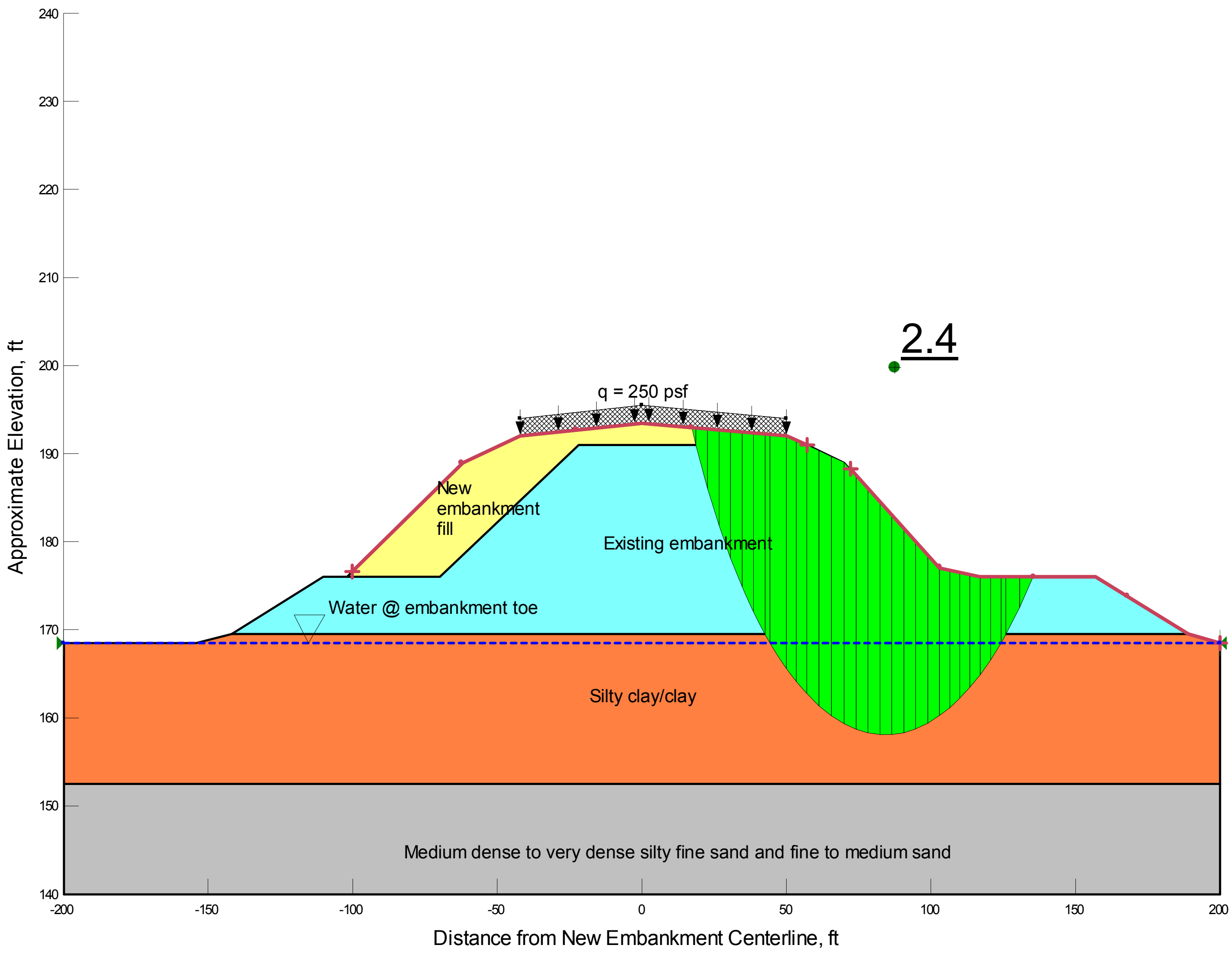
Results of Stability Analyses – End of Construction Condition
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



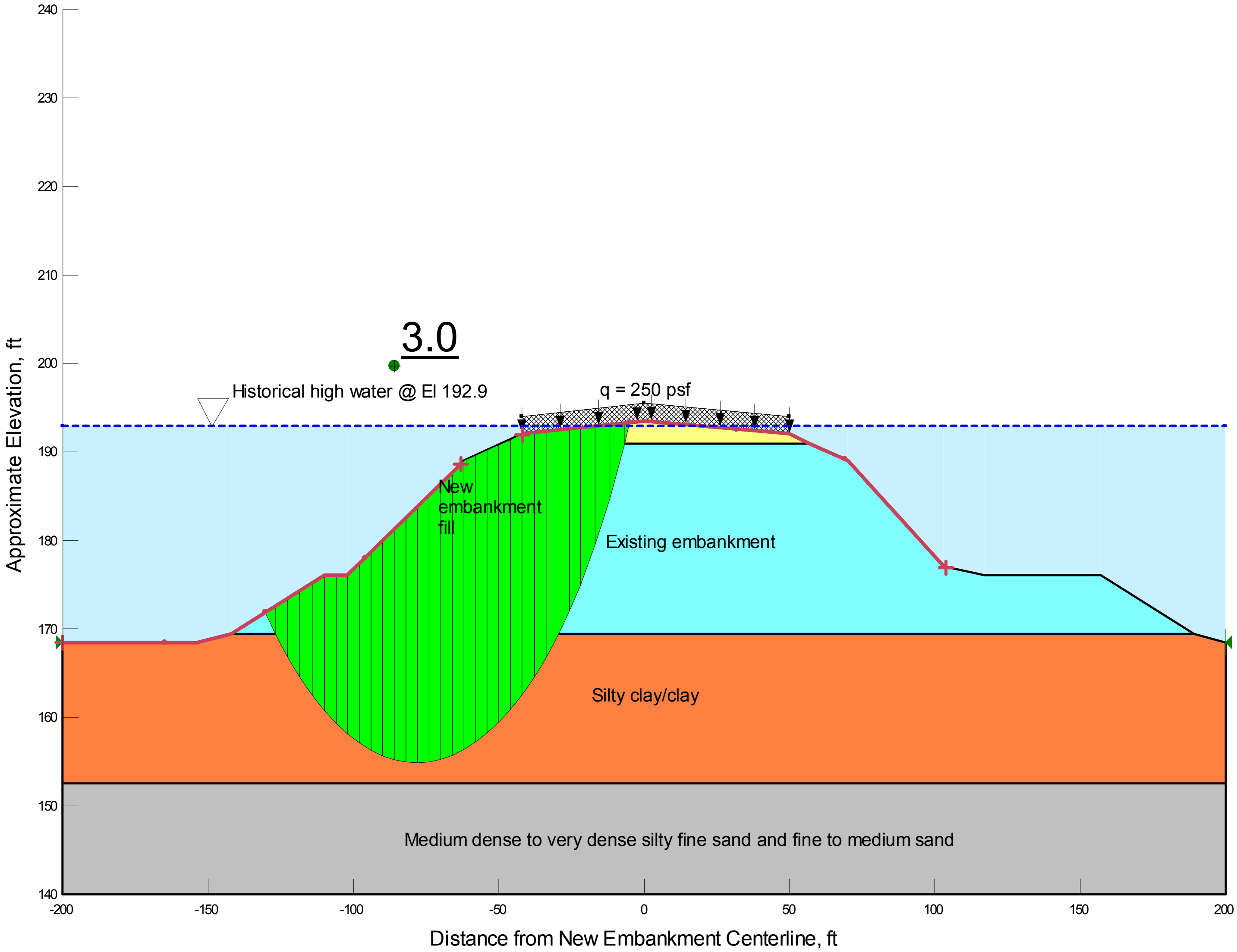
Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



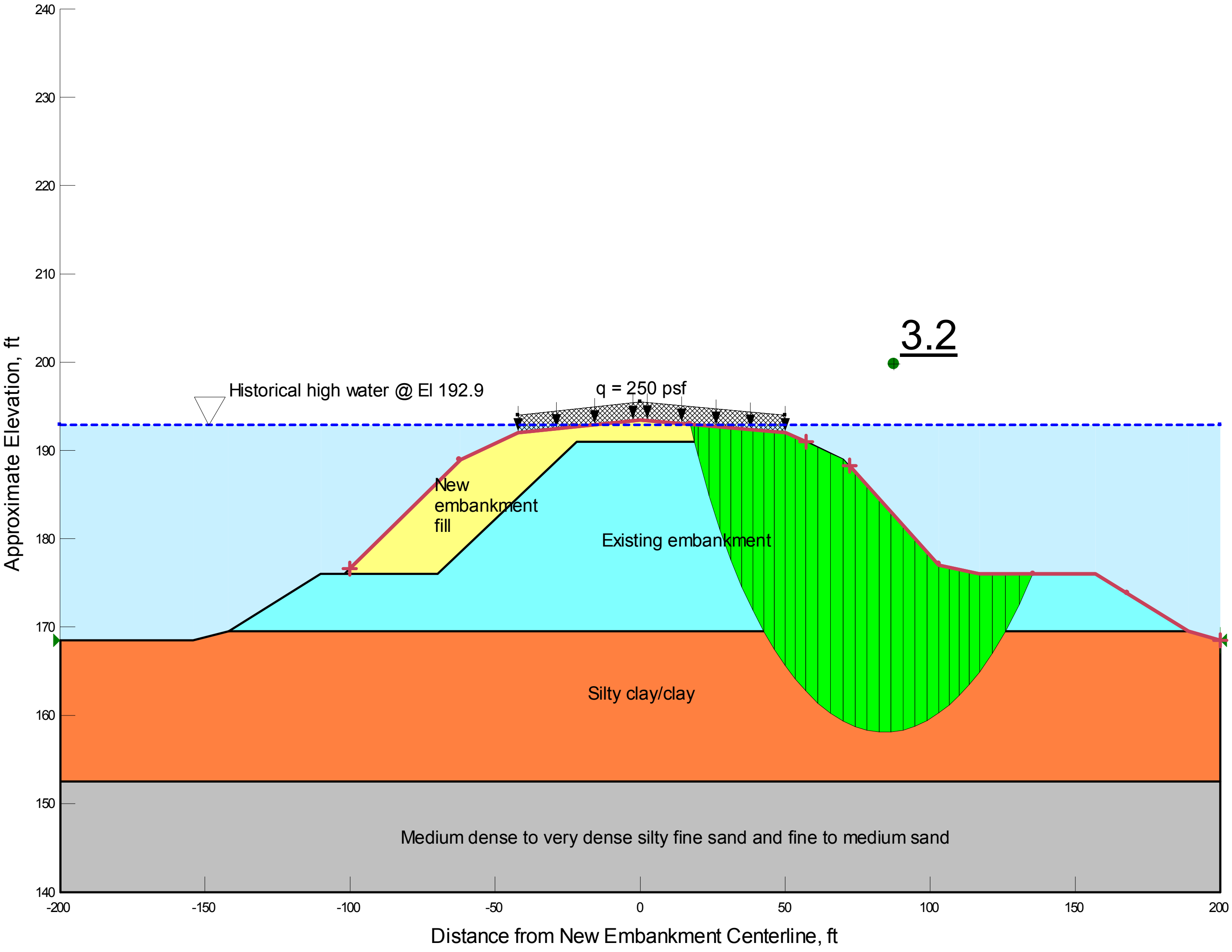
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



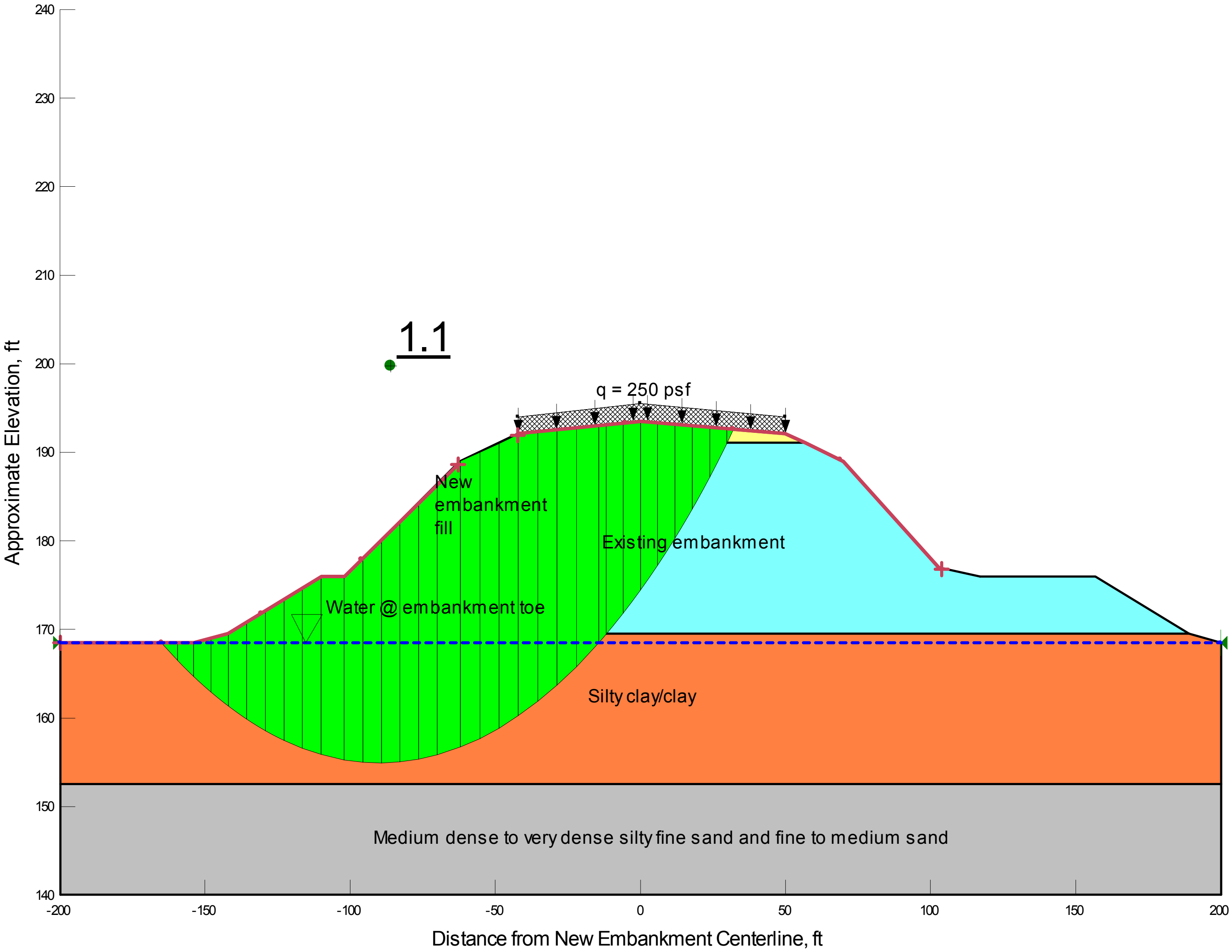
Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1875+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

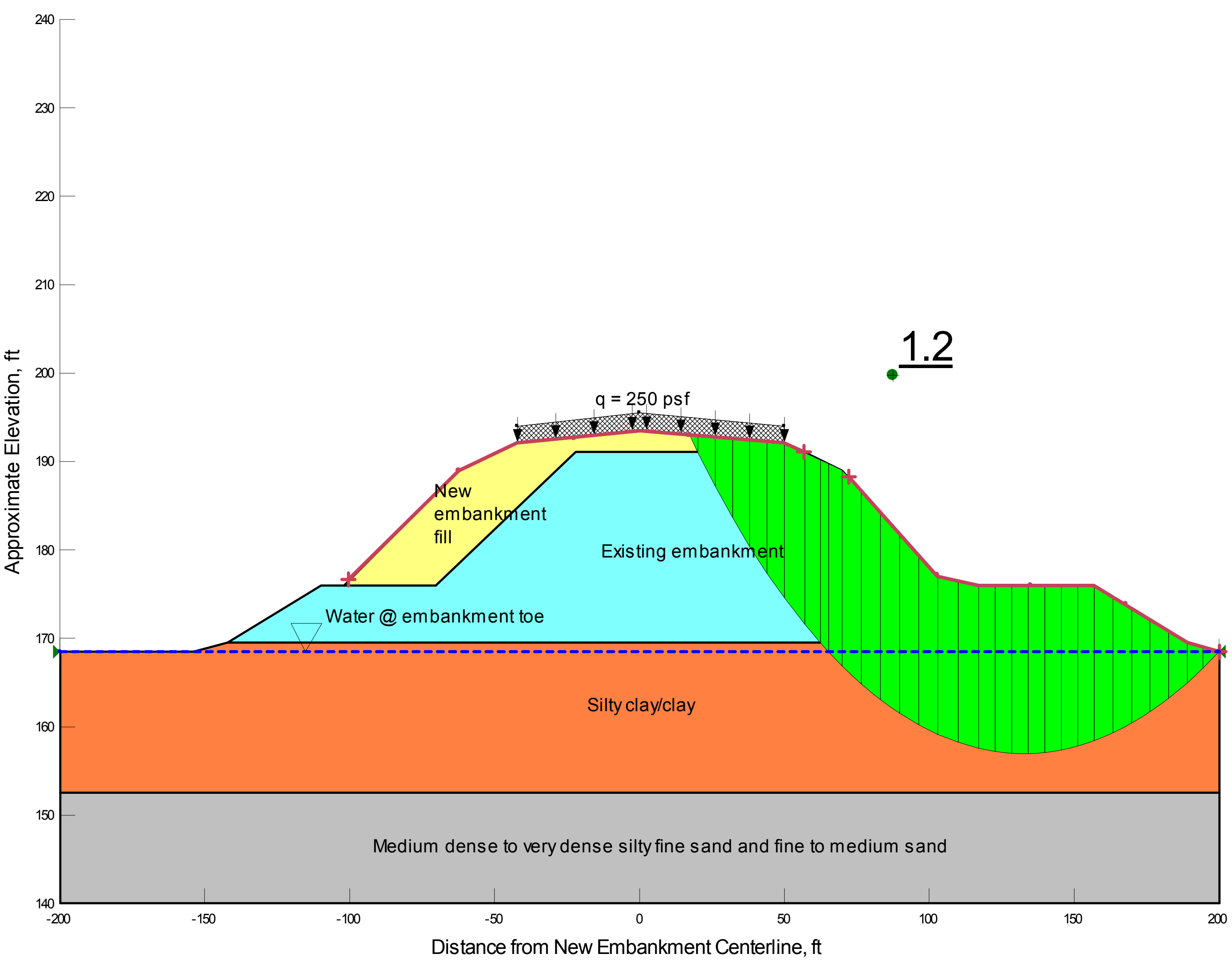


Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1875+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

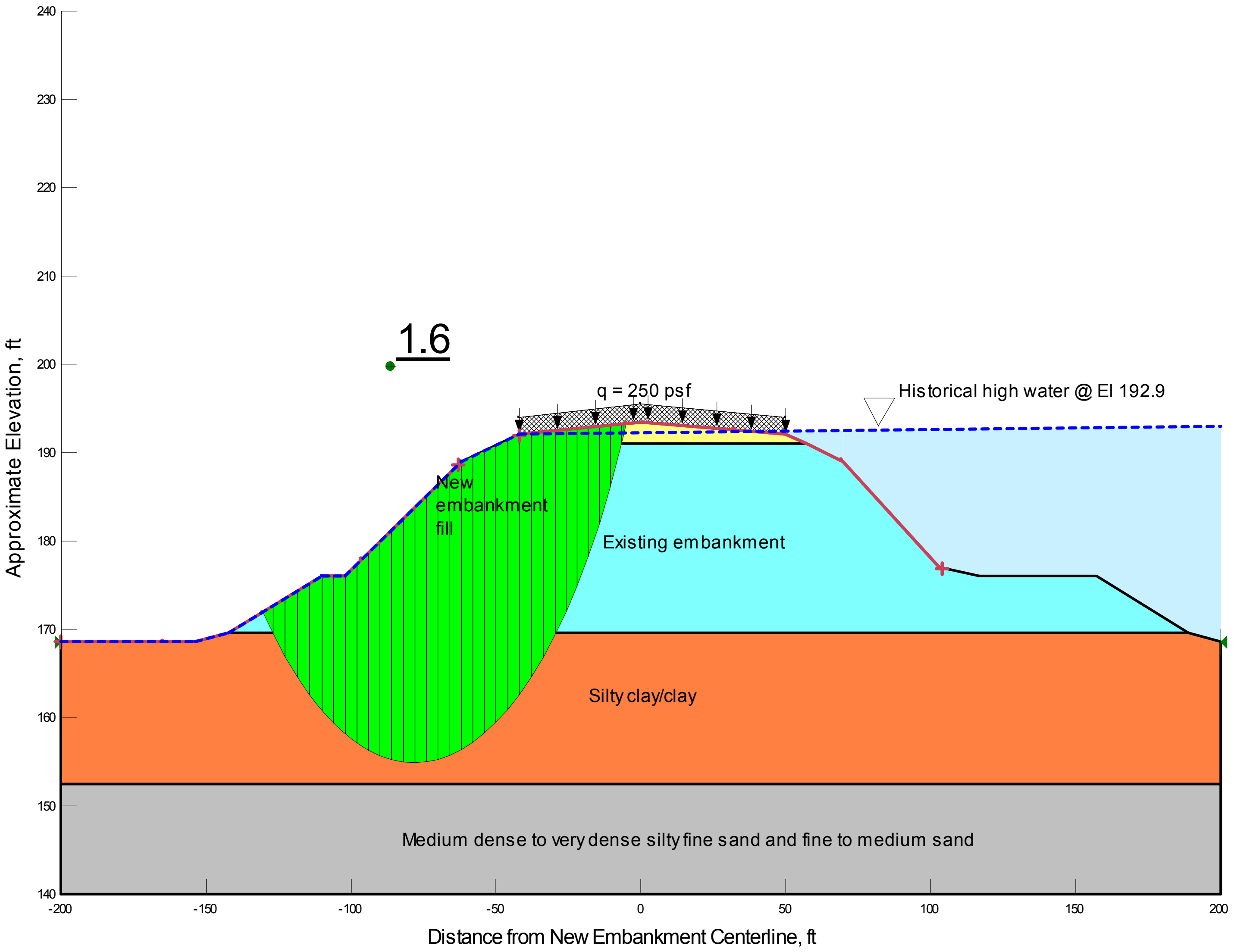


Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced North Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

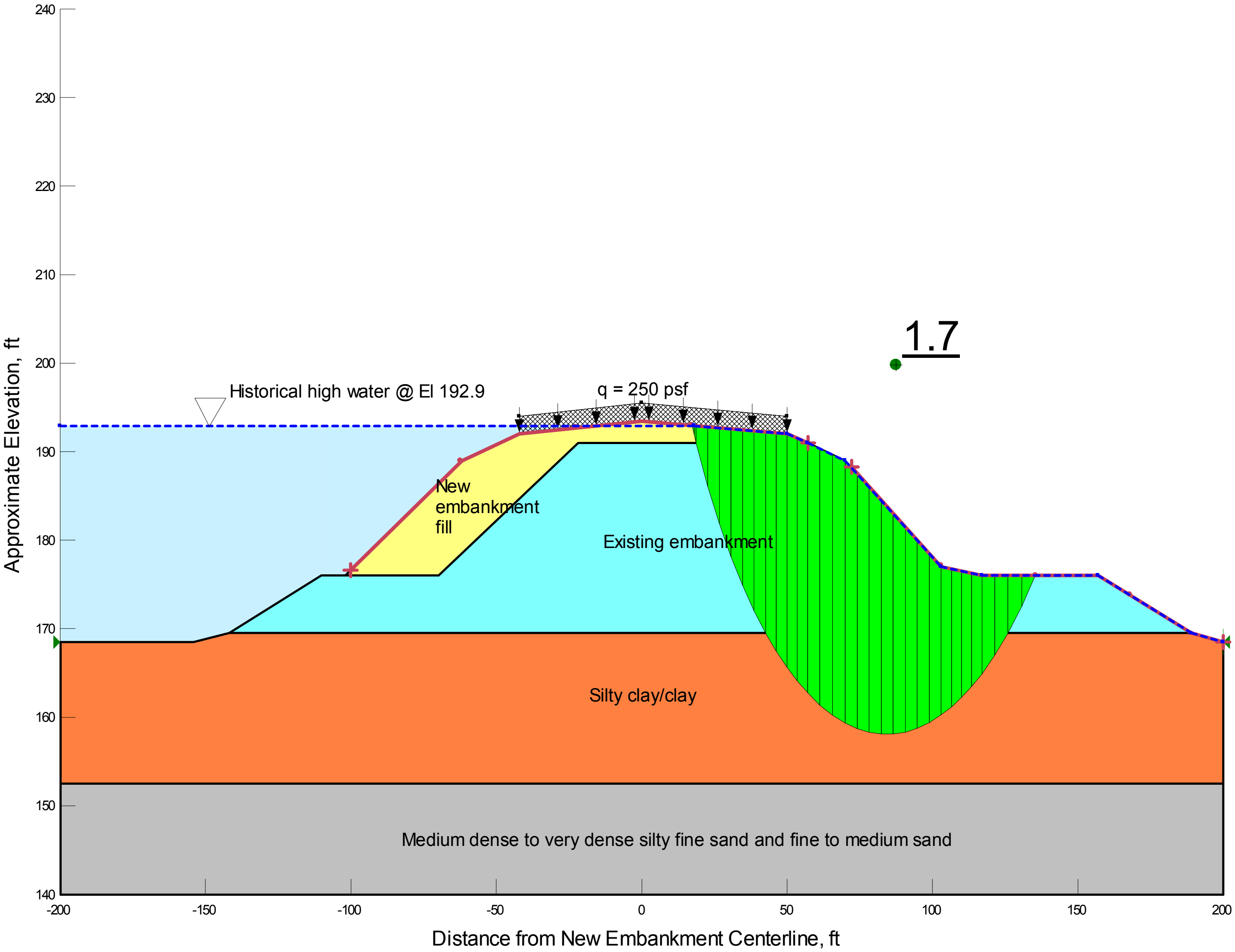


Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced South Side Slope @ Sta 1875+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

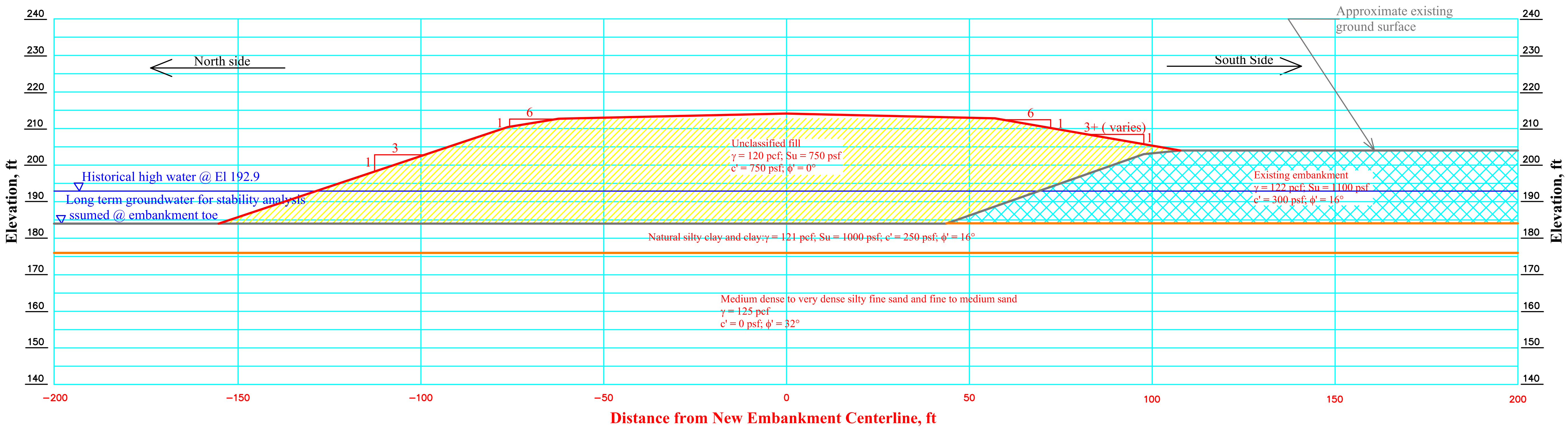


Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced North Side Slope @ Sta 1875+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)



Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced South Side Slope @ Sta 1875+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

APPENDIX O



15 ft 0 15 30 ft

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

Section and Material Parameters for Stability Analysis
Unreinforced Side Slopes @ Sta 1913+00
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

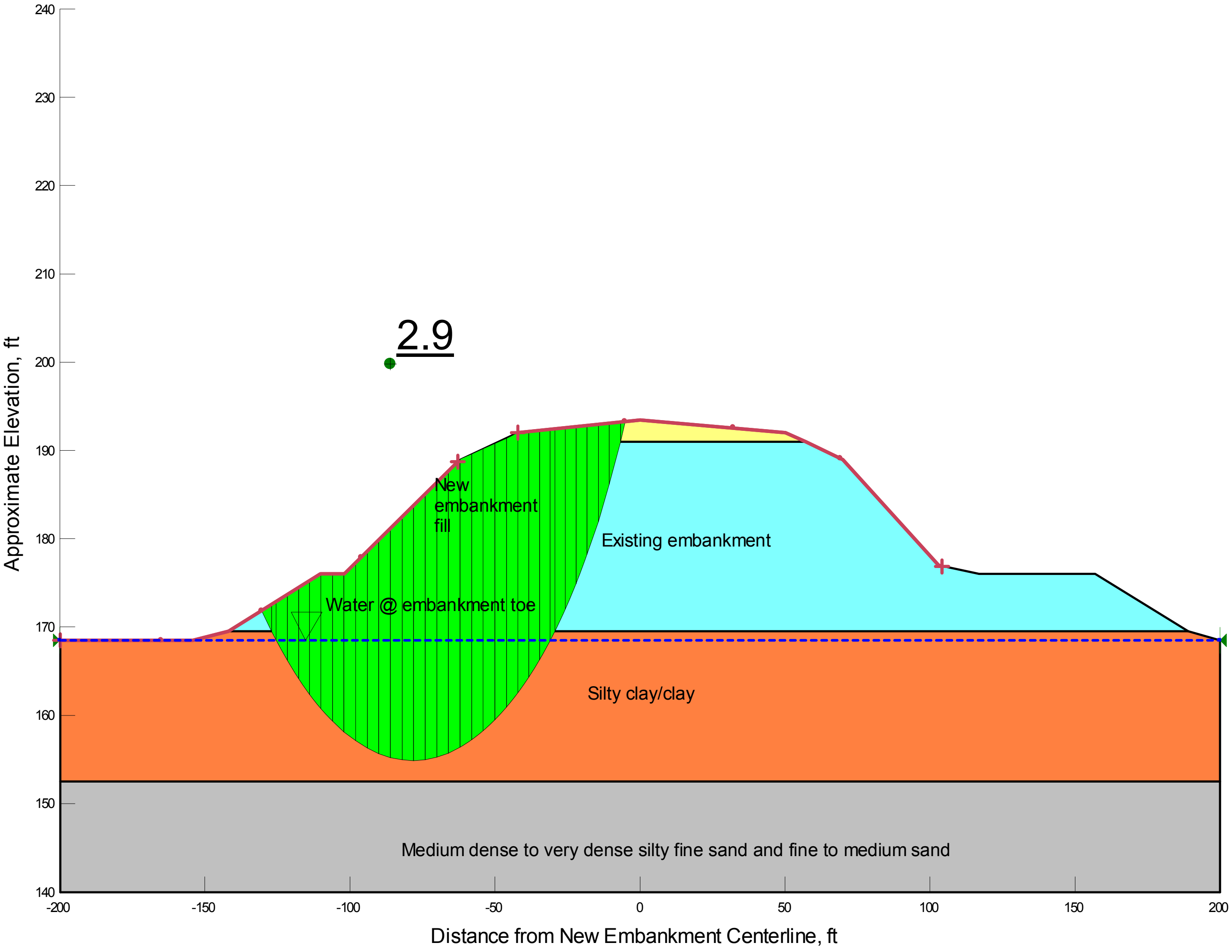
Scale: As Shown

May 5, 2014

Plate

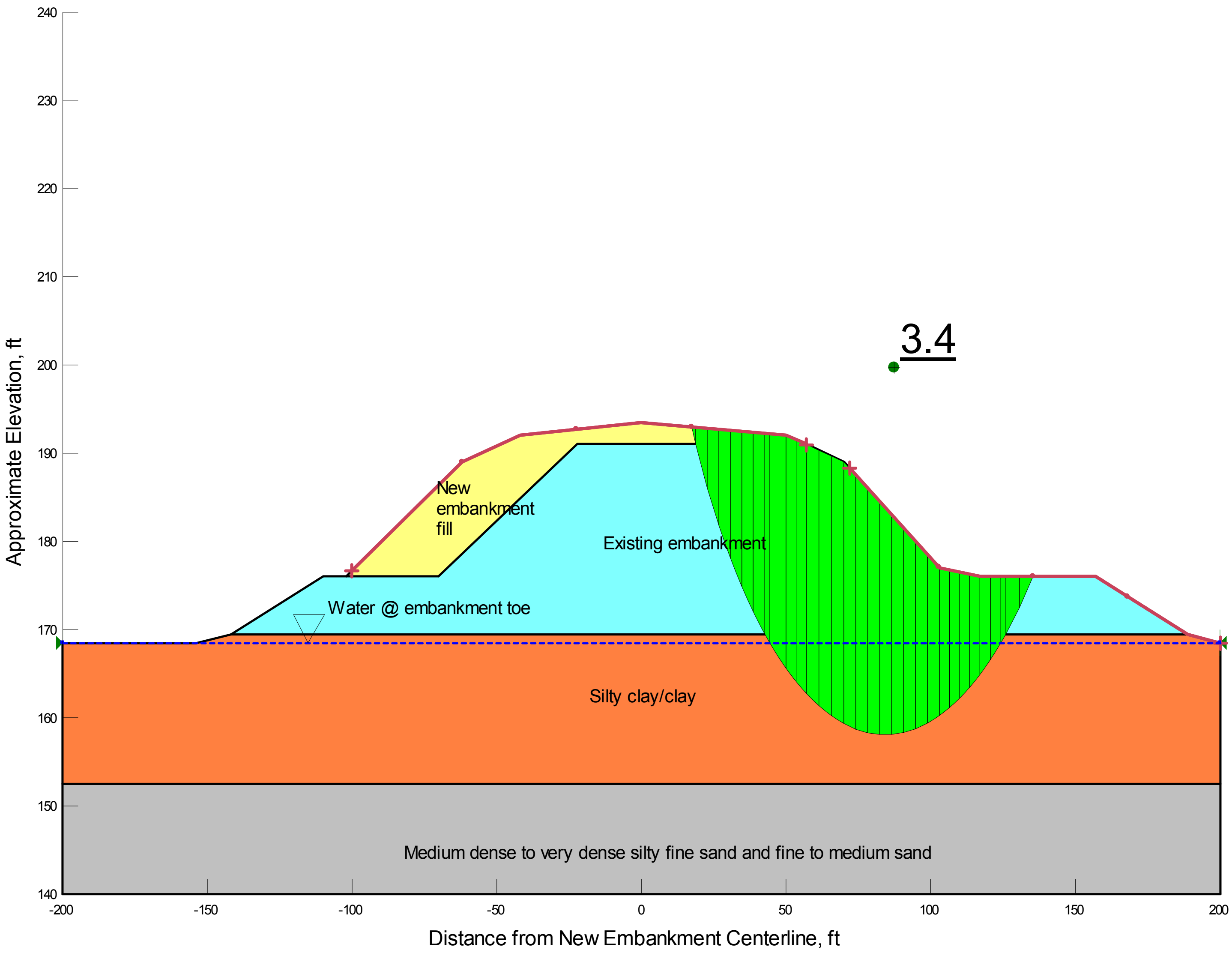
Stability Analysis Results
Unreinforced Side Slopes @ Sta 1913+00
AHTD JOB BB0610:White River Str. & Apprs. (F)

Embankment Side	Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
North	End of Construction	Analysis long term groundwater @ embankment toe	2.9
South			3.4
North	Long Term	Analysis long term groundwater @ embankment toe	1.8
South			4.3
North		Historical high water @ El 192.9	1.8
South			3.9
North	Seismic ($k_h = 1.0A_S = 0.19$)	Analysis long term groundwater @ embankment toe	1.1
South			1.6
North	Rapid Drawdown	Drawdown from 192.9 to embankment toe	1.6

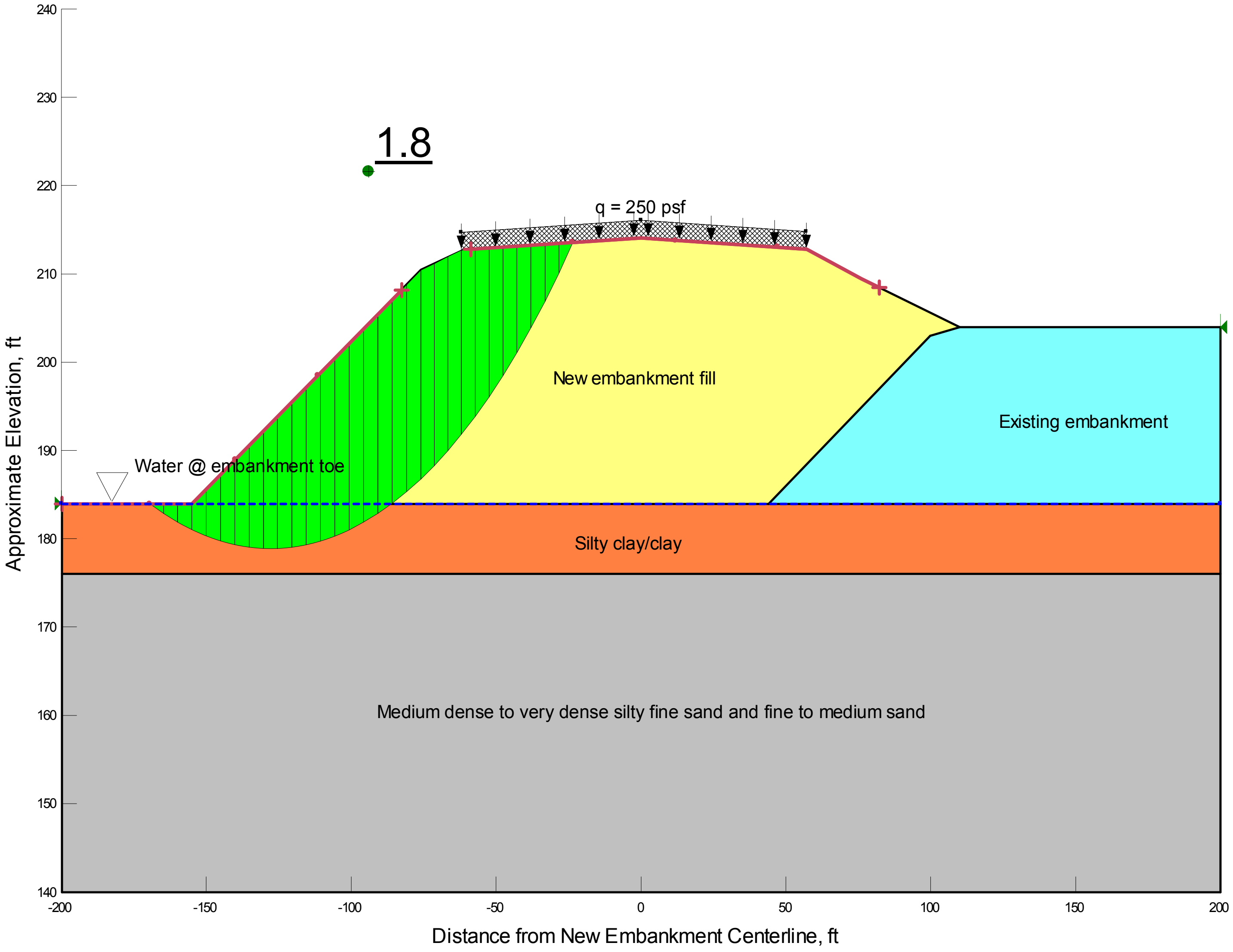


Results of Stability Analyses – End of Construction Condition
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

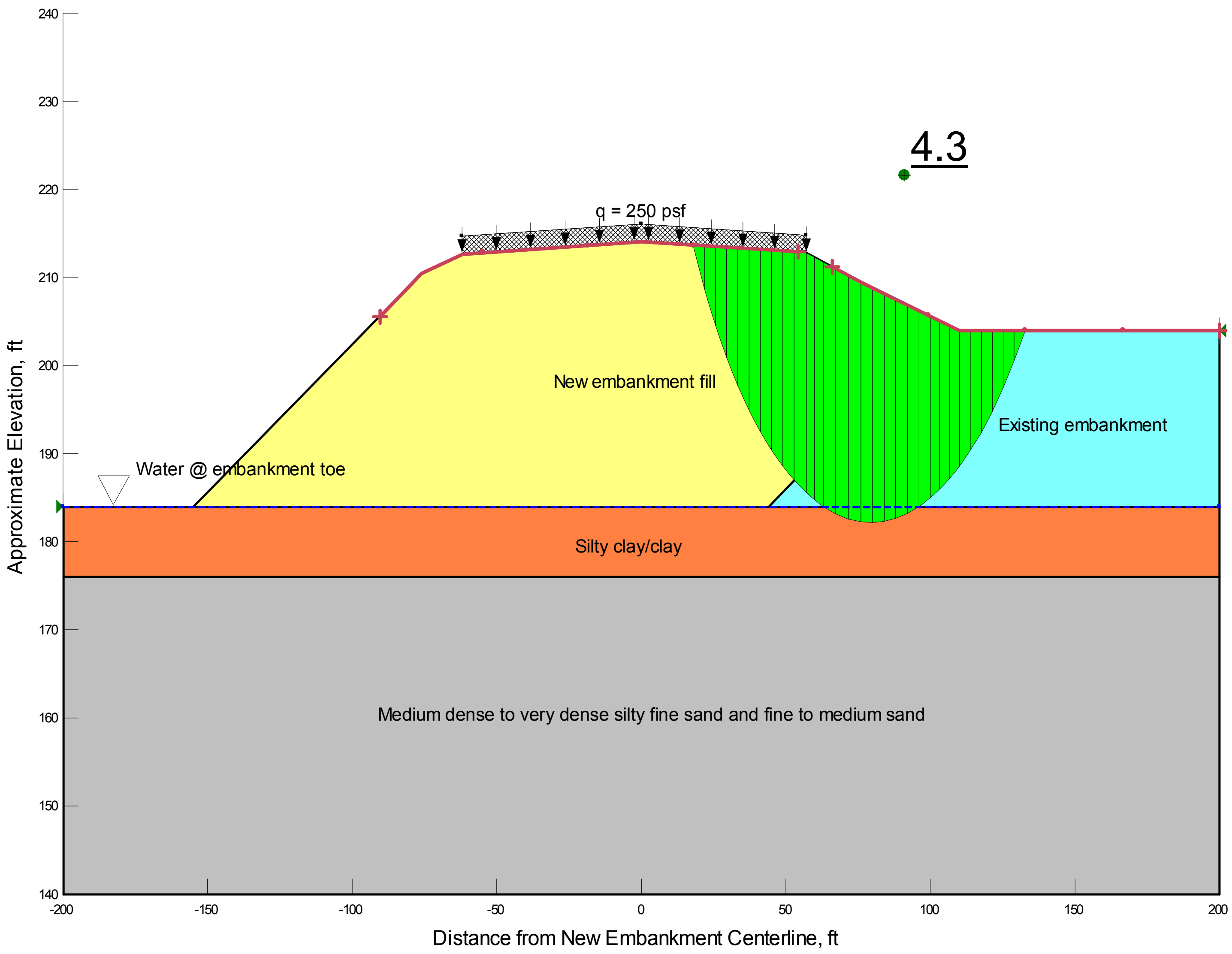


Results of Stability Analyses – End of Construction Condition
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

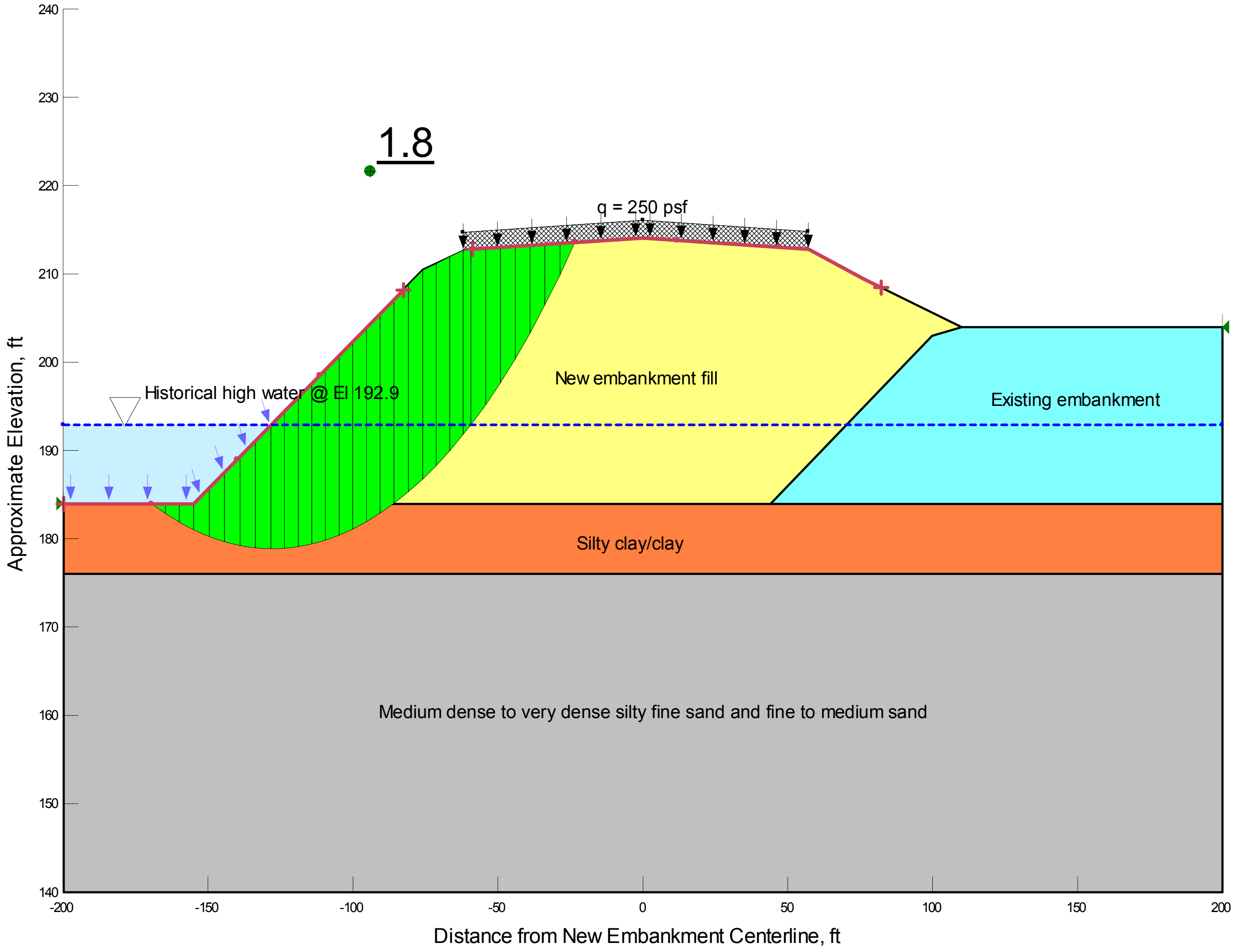


Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

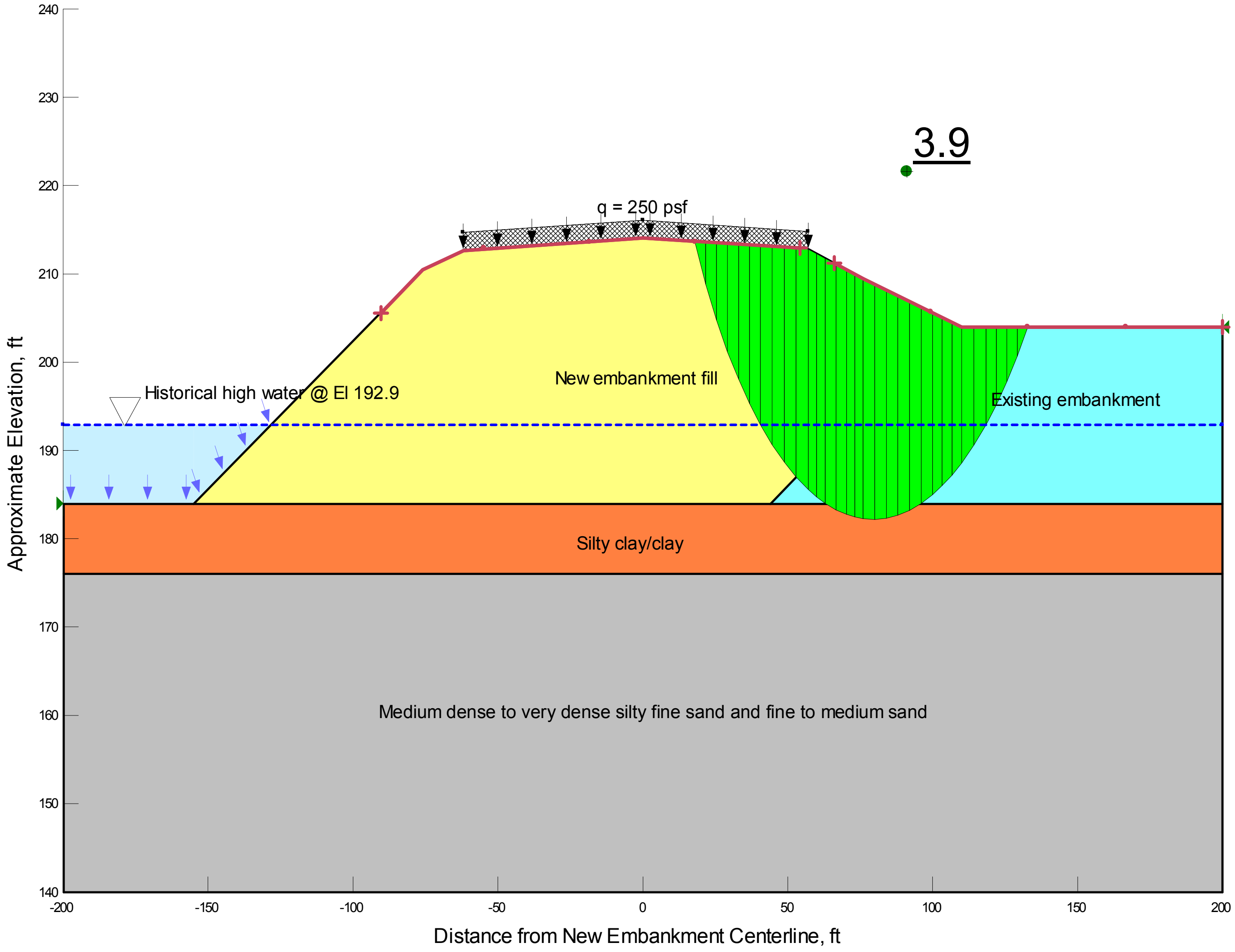
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

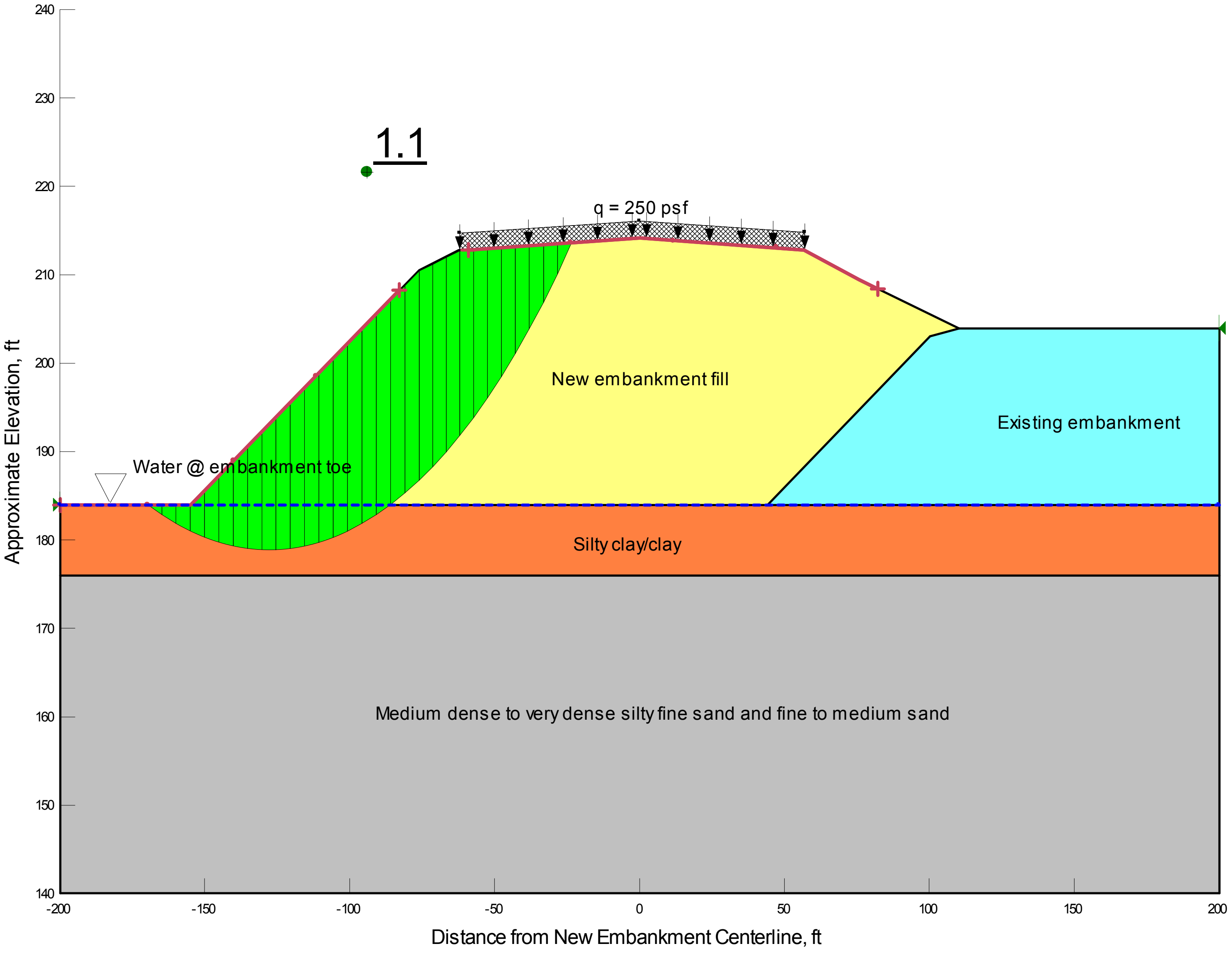


Results of Stability Analyses – Long Term Condition
Unreinforced North Side Slope @ Sta 1913+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)



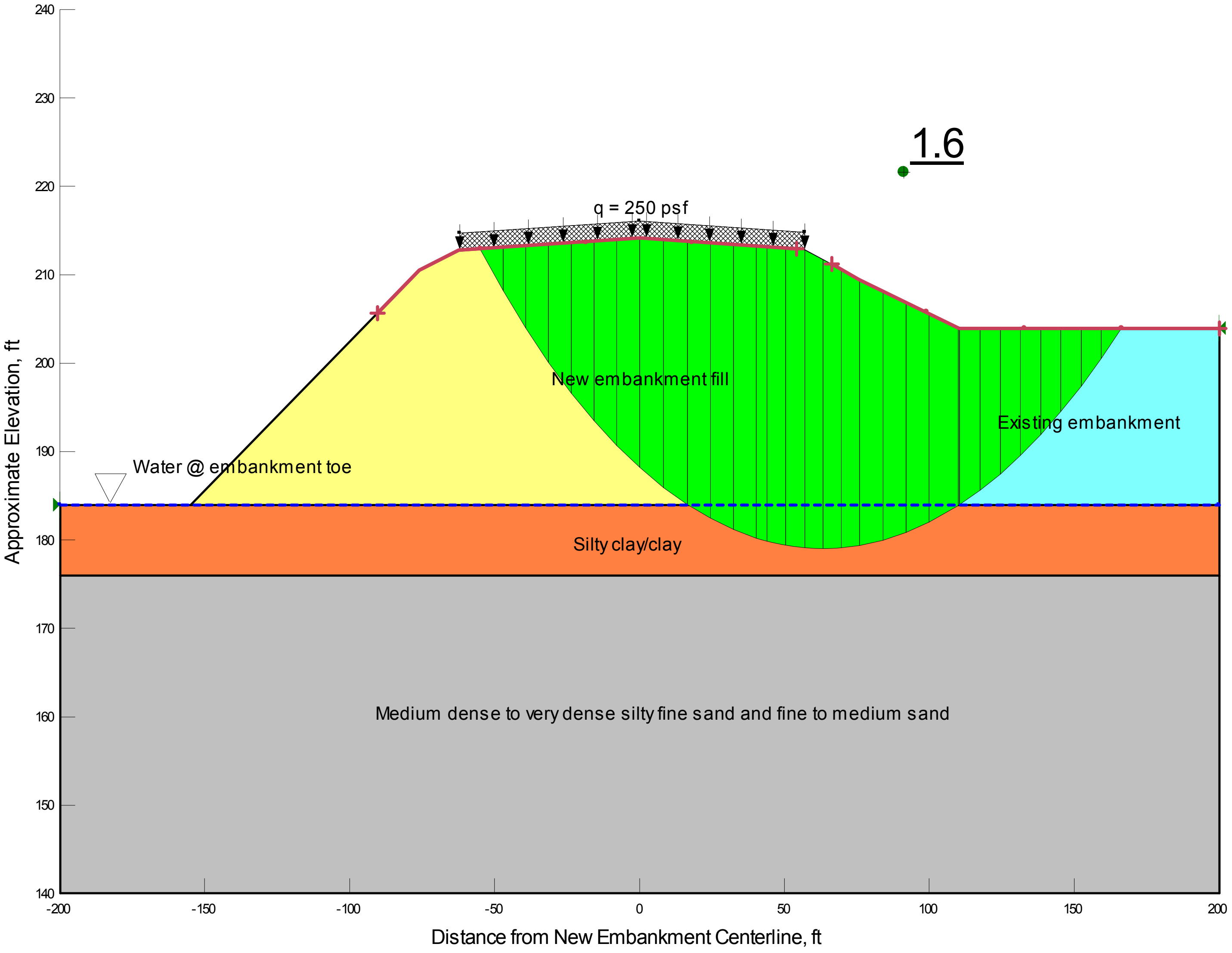
Results of Stability Analyses – Long Term Condition
Unreinforced South Side Slope @ Sta 1913+00
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



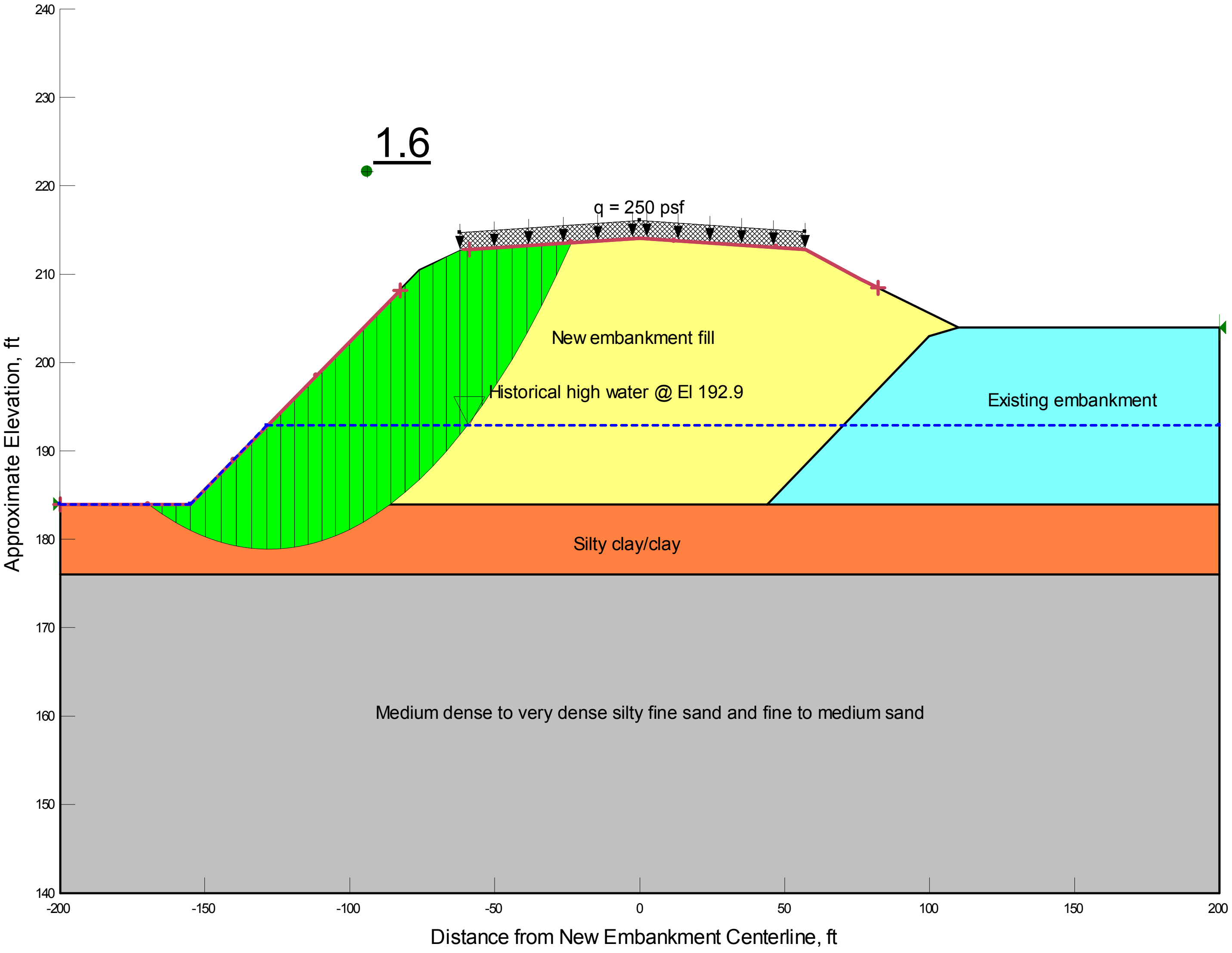
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced North Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Unreinforced South Side Slope @ Sta 1913+00
Analysis Long Term Groundwater @ Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

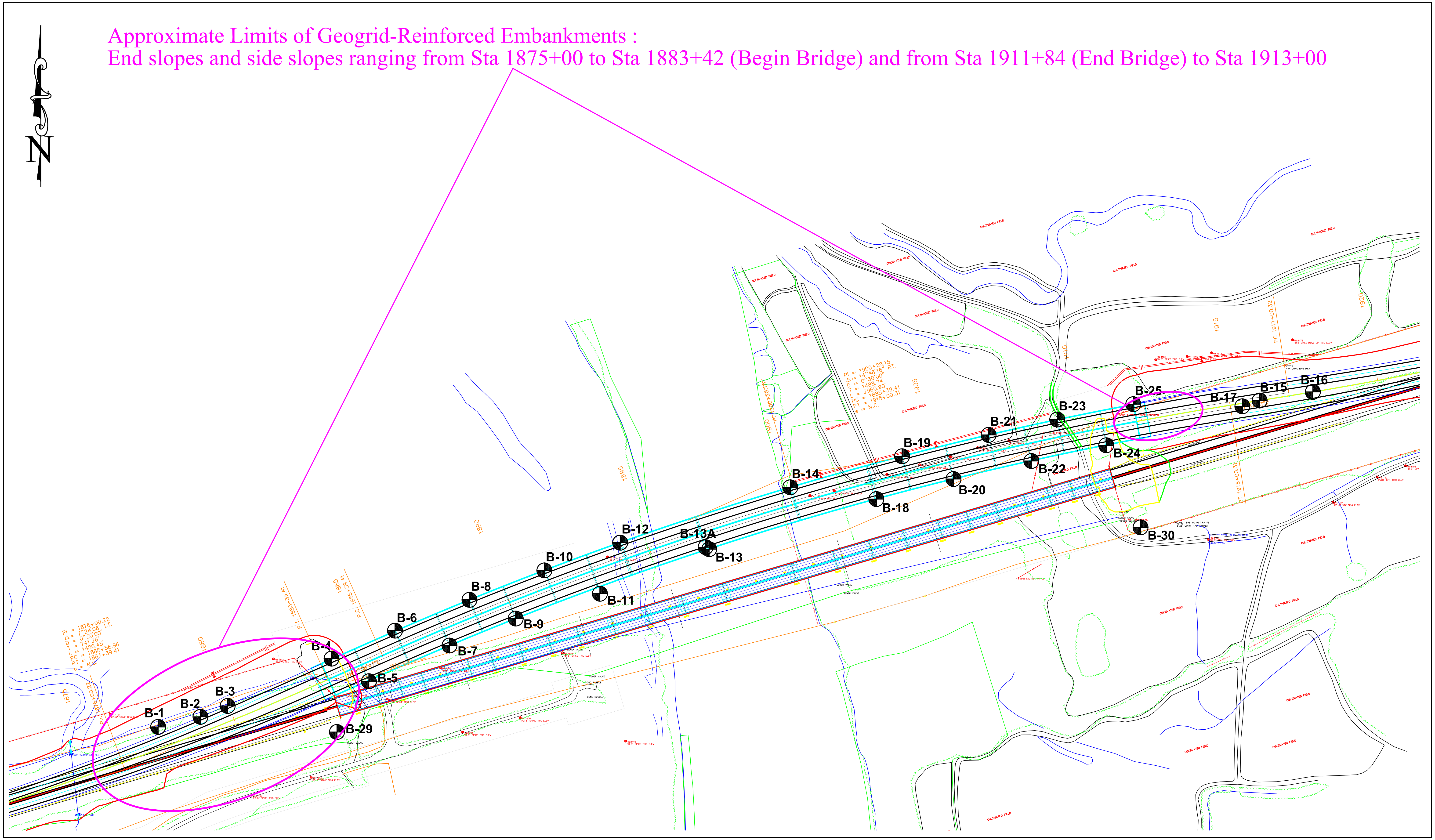
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Unreinforced North Side Slope @ Sta 1913+00
Drawdown from El 192.9 to Embankment Toe
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX P



Approximate Limits of Geogrid-Reinforced Embankments :
End slopes and side slopes ranging from Sta 1875+00 to Sta 1883+42 (Begin Bridge) and from Sta 1911+84 (End Bridge) to Sta 1913+00

Note: Plan provided by Jacobs.

150 ft 0 150 300 ft

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

Recommended Limits of Geogrid-Reinforced Embankments
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

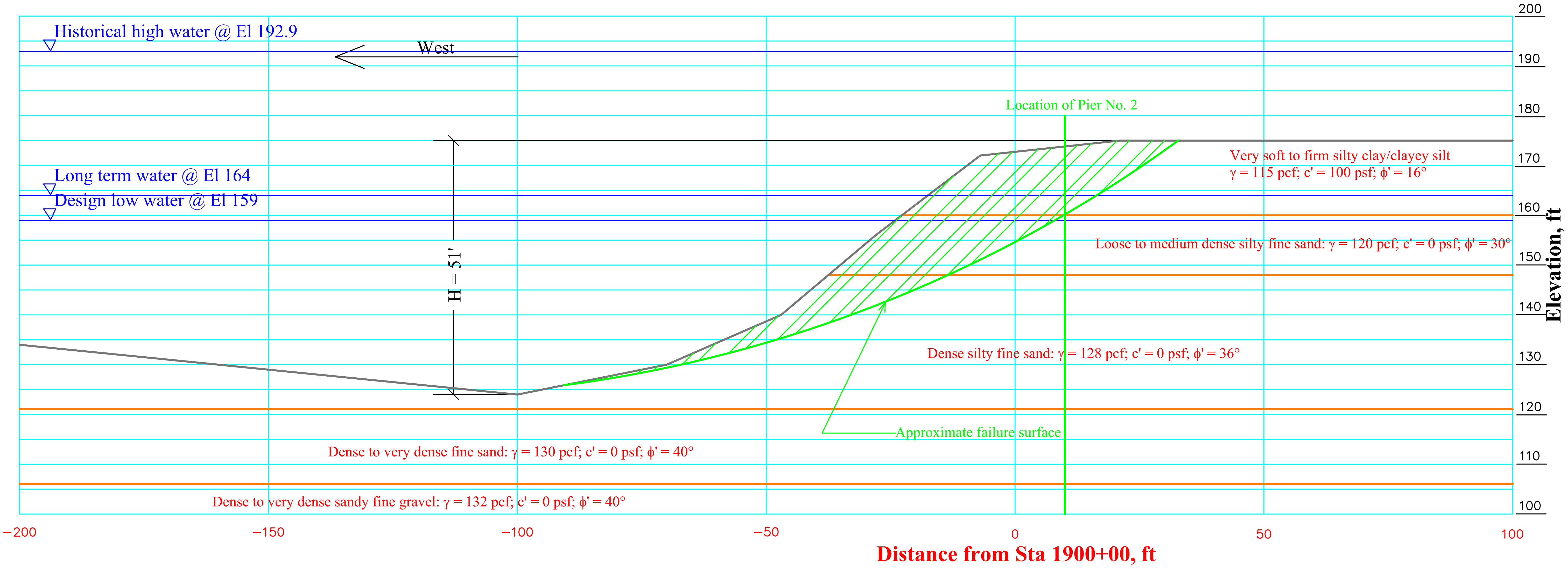
Job No.: 13-017

Scale: As Shown

April 29, 2014

Plate 2a

APPENDIX Q



10 ft 0 10 20 ft

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

Section and Material Parameters for Stability Analysis
Existing East River Bank near Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

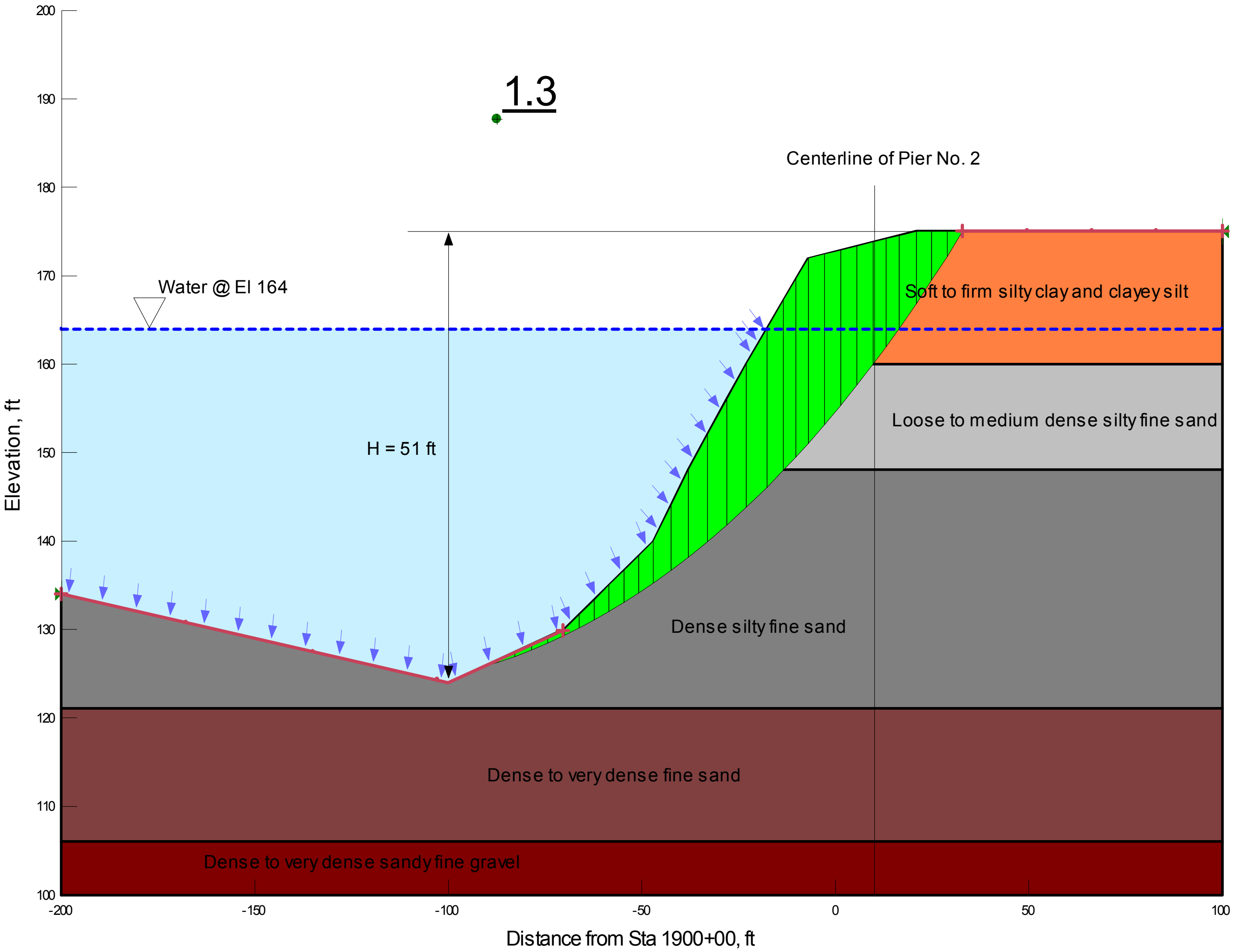
Scale: As Shown

May 8, 2014

Plate

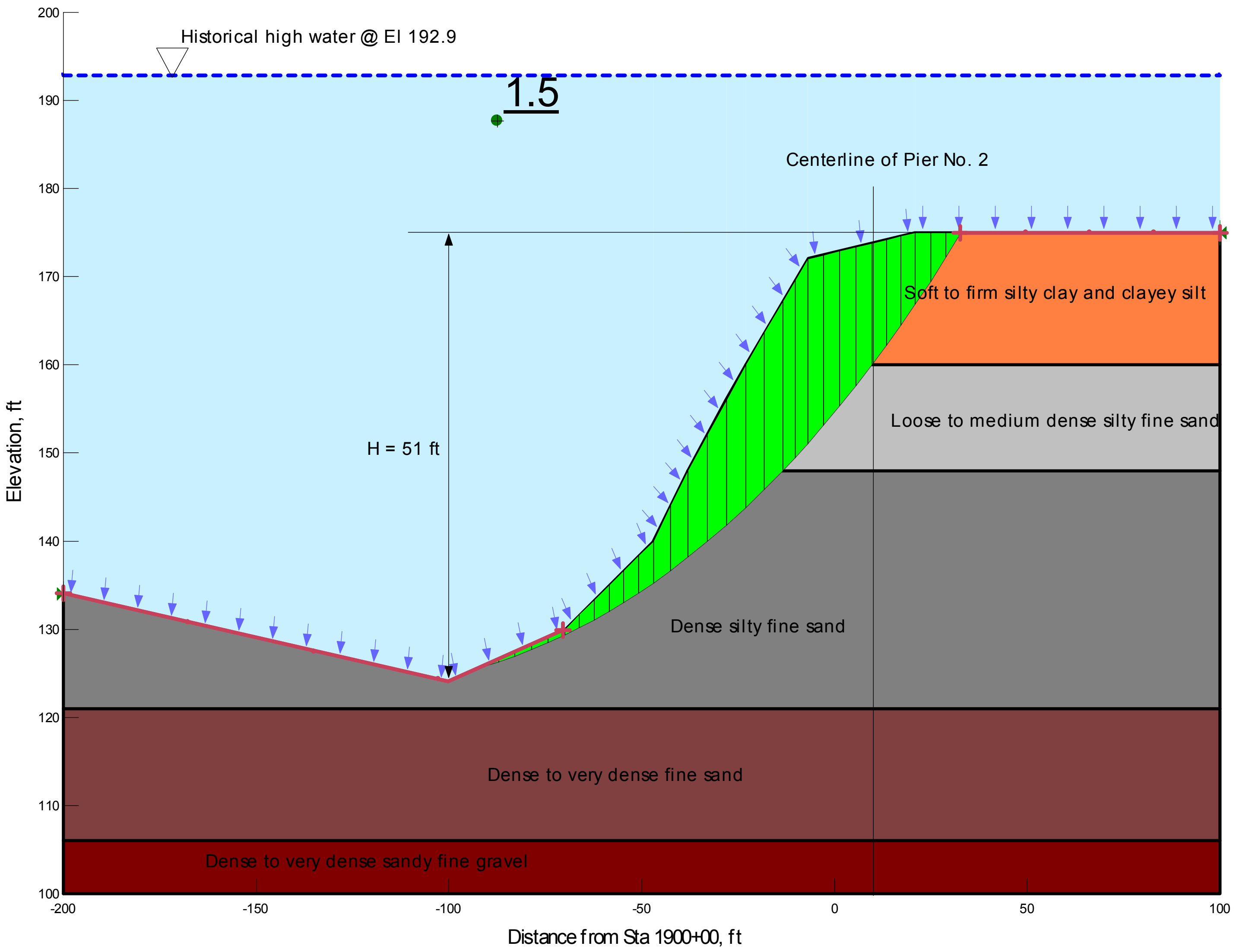
Stability Analysis Results – East Abutment
Existing East River Bank near Pier No. 2 – H = 51 ft
AHTD Job BB0610: White River Str. & Apprs. (F)

Design Loading Condition	**Design Water Condition**	**Calculated Minimum Factor of Safety**
Long Term	Water @ El 164	1.3
Long Term	Historical high water @ El 192.9	1.5
Seismic (k_h = 1.0A_S = 0.19)*	Water @ El 164	0.7
Rapid Drawdown	Drawdown to El 159 (design low water)	0.8



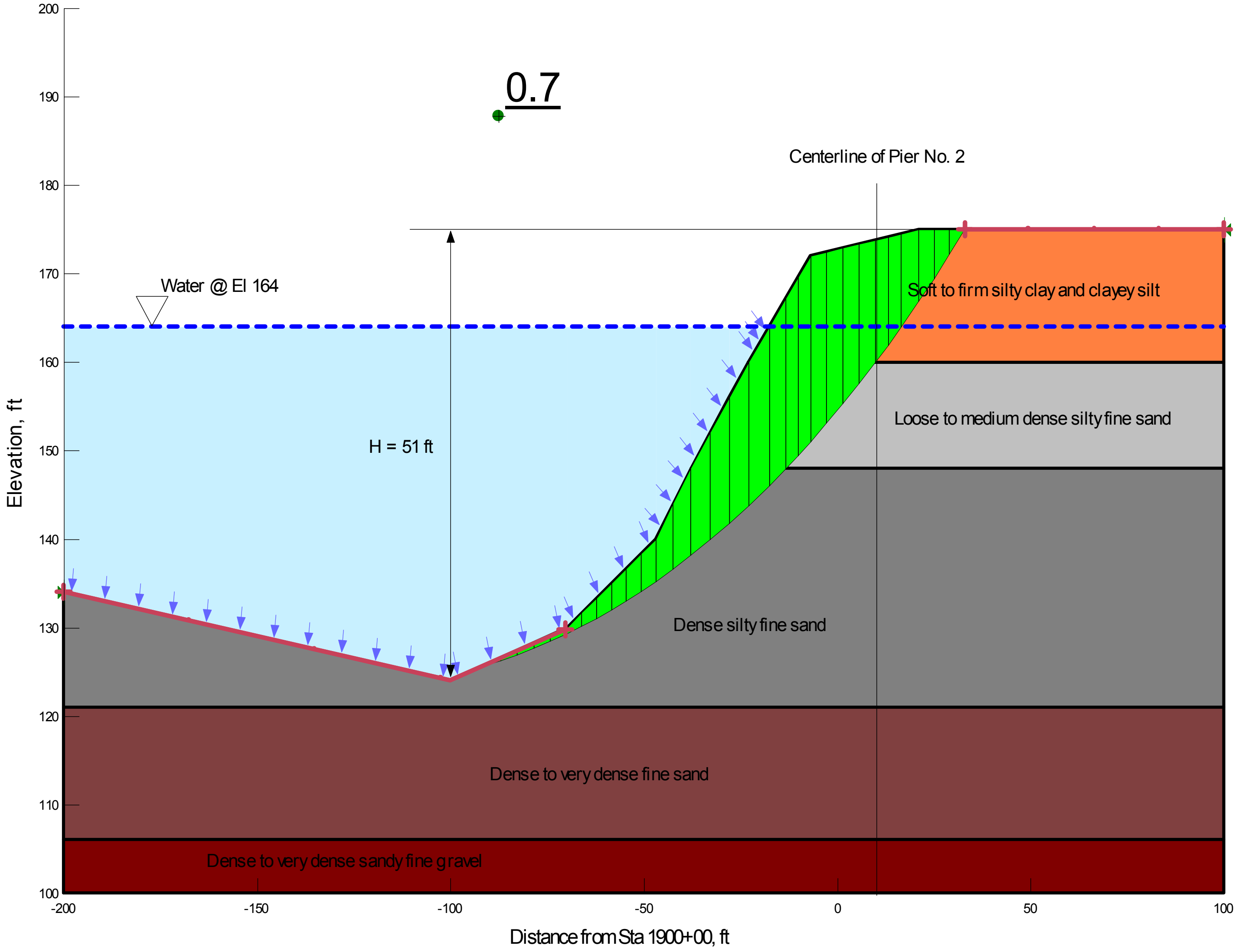
Results of Stability Analyses – Long Term Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



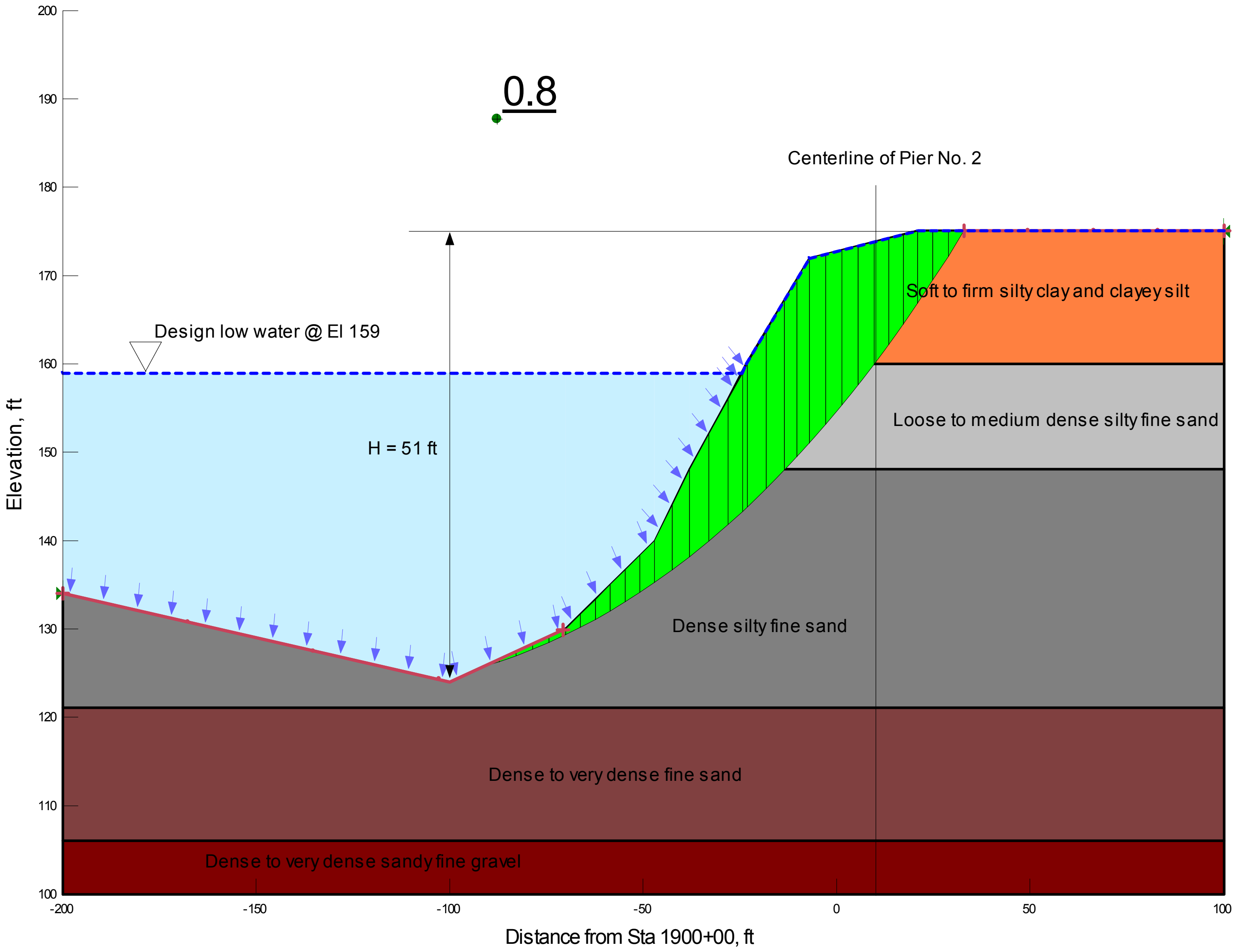
Results of Stability Analyses – Long Term Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Historical High Water @ El 192.9
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.19$)
Existing East River Bank near Pier No. 2 – H = 51 ft
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

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



Results of Stability Analyses – Rapid Drawdown Condition
Existing East River Bank near Pier No. 2 – H = 51 ft
Drawdown to El 159 (Design Low Water)
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

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

Section and Material Parameters for Stability Analysis
Retaining Structure near Pier No. 2
AHTD Job No. BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

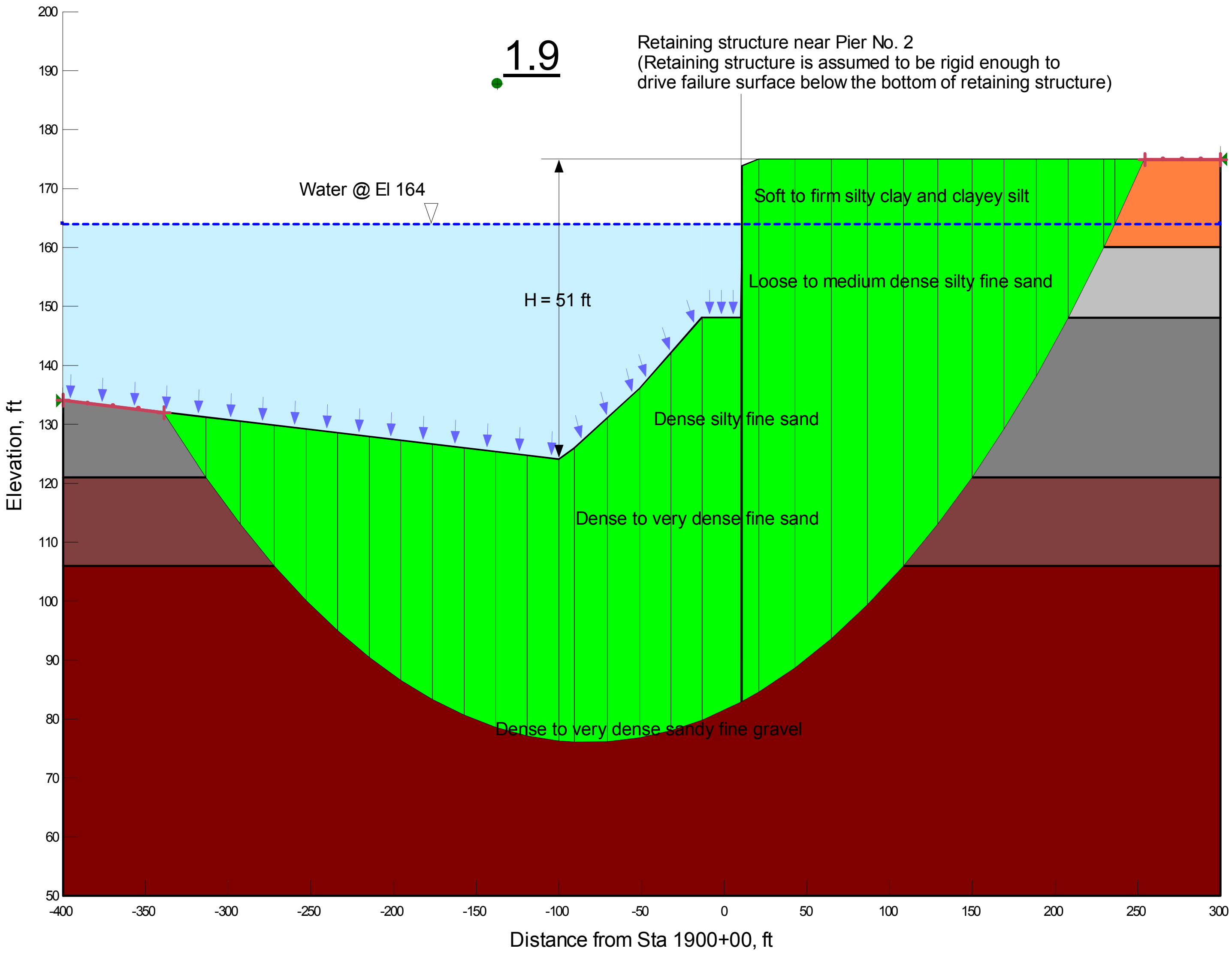
Scale: As Shown

September 18, 2014

Plate

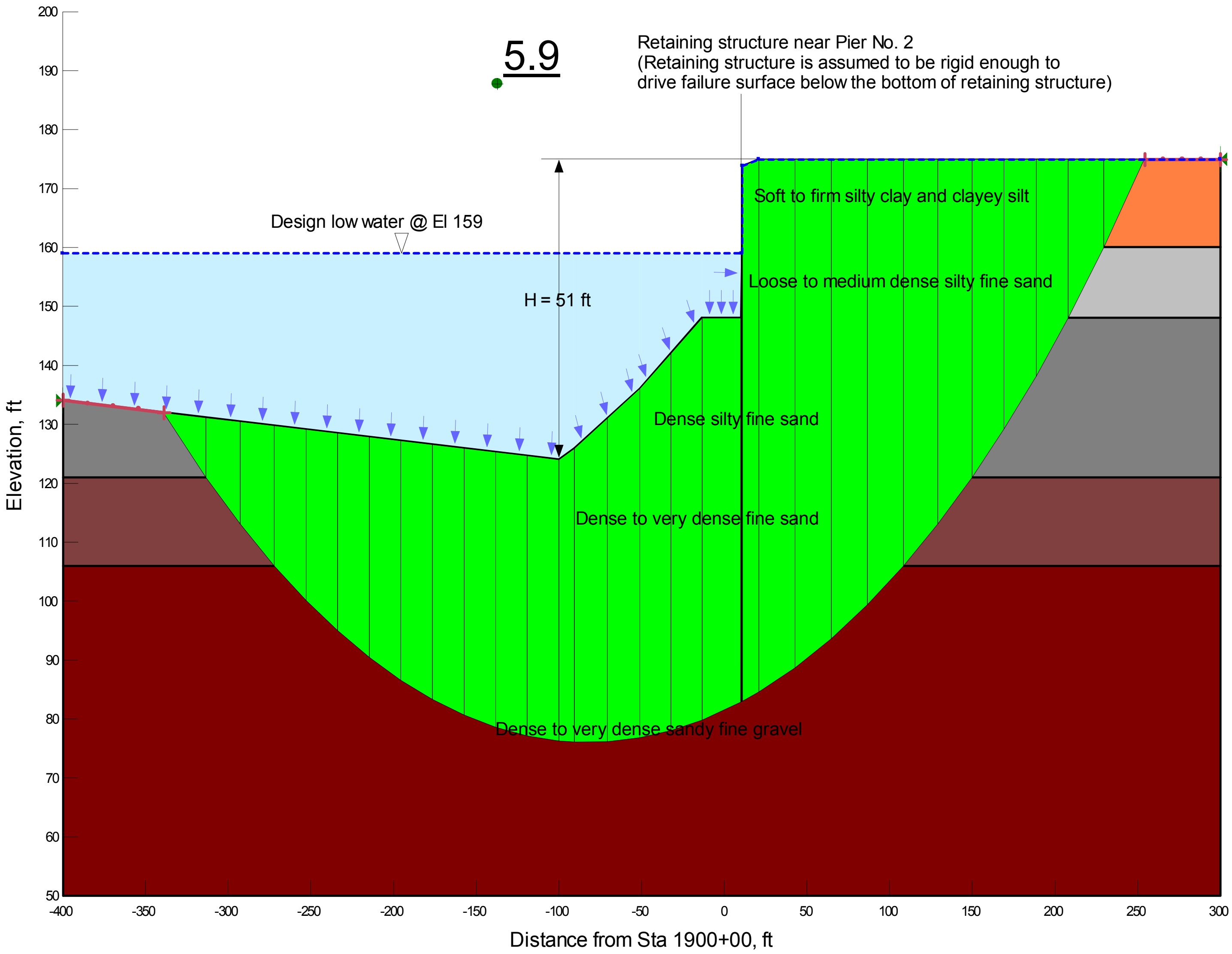
Stability Analysis Results
East River Bank Remedied by Retaining Structure
AHTD Job BB0610: White River Str. & Apprs. (F)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
Seismic (k_h = 1.0A_S = 0.19)*	Water @ El 164	1.9
Rapid Drawdown	Drawdown to El 159 (design low water)	5.9



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.23$)
East River Bank Remedied by Retaining Structure
Water @ El 164
AHTD JOB BB0610:White River Str. & Apprs. (F)

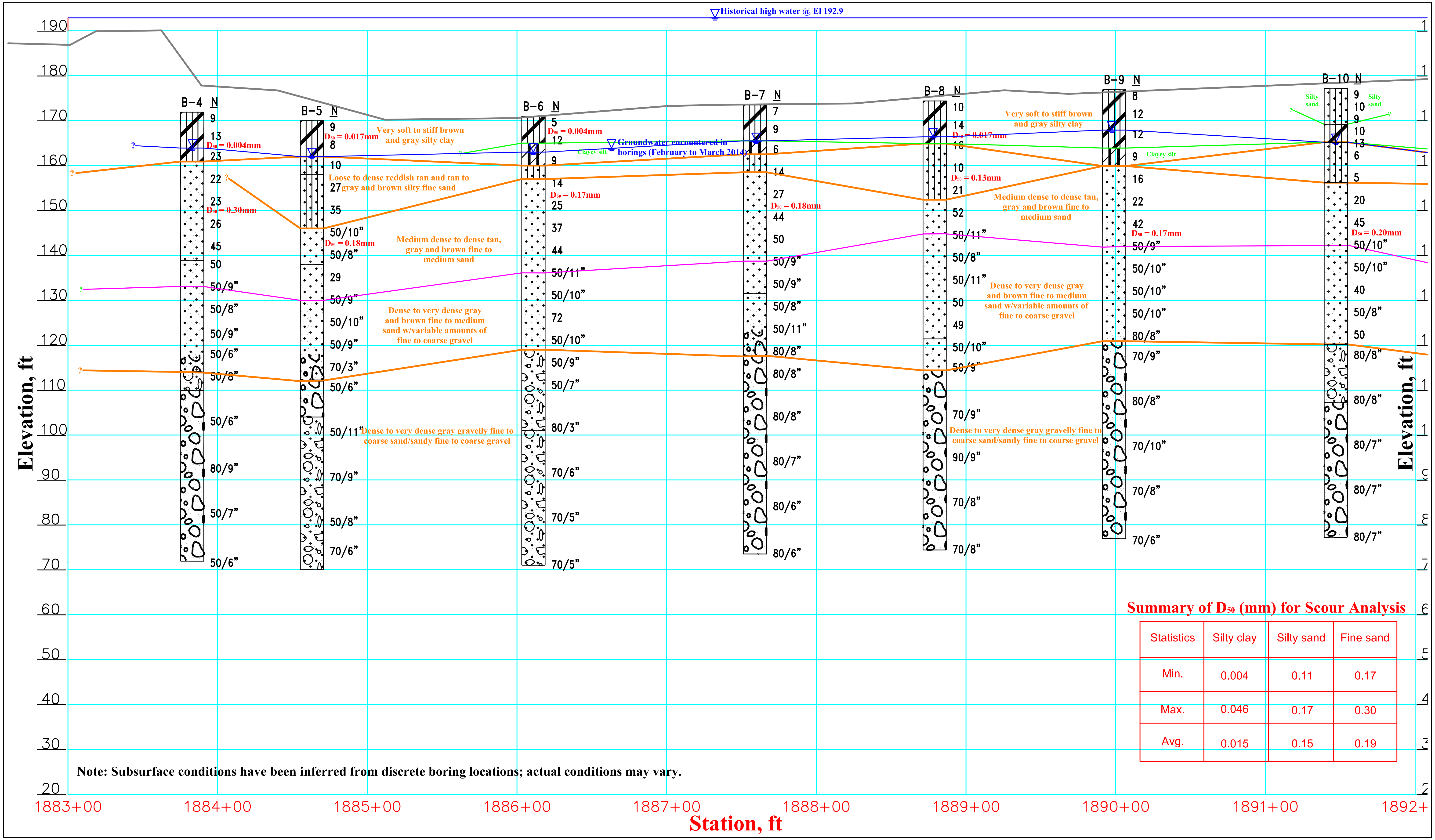
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
East River Bank Remedied by Retaining Structure
Drawdown to El 159 (Design Low Water)
AHTD JOB BB0610:White River Str. & Apprs. (F)

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

APPENDIX R



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Note: Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.

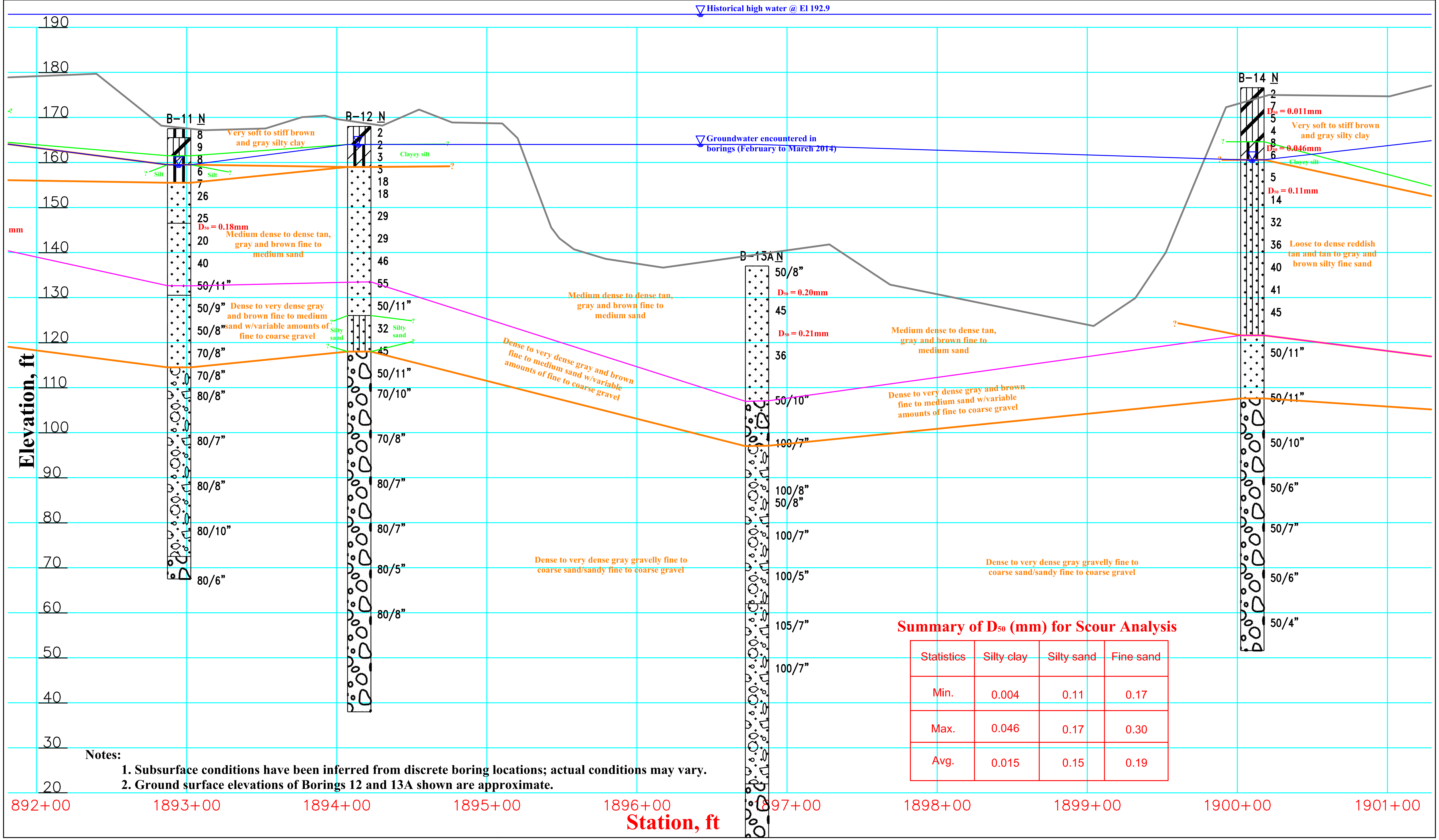
Horizontal Scale: 30 ft 0 30 60 ft

Vertical Scale: 10 ft 0 10 20 ft

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

D_{50} for Scour Analysis - Page 1 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	Scale: As Shown
April 8, 2014	Plate



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Notes:

1. Subsurface conditions have been inferred from discrete boring locations; actual conditions may vary.
2. Ground surface elevations of Borings 12 and 13A shown are approximate.

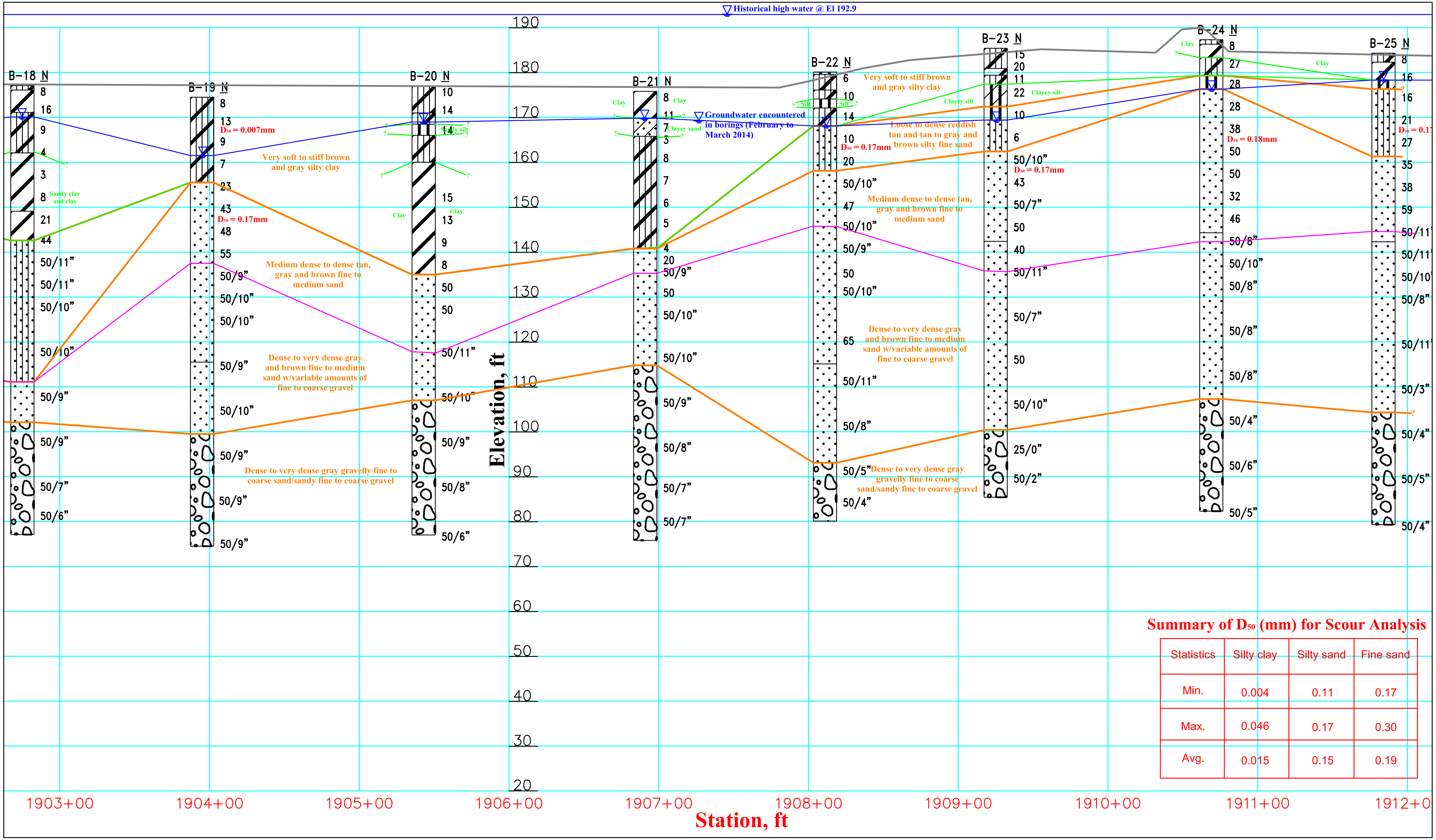
Horizontal Scale: 30 ft 0 30 60 ft

Vertical Scale: 10 ft 0 10 20 ft

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

D_{50} for Scour Analysis - Page 2 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017	Scale: As Shown
April 8, 2014	Plate



Summary of D_{50} (mm) for Scour Analysis

Statistics	Silty clay	Silty sand	Fine sand
Min.	0.004	0.11	0.17
Max.	0.046	0.17	0.30
Avg.	0.015	0.15	0.19

Horizontal Scale:
30 ft 0 30 60 ft

Vertical Scale:
10 ft 0 10 20 ft

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

D_{50} for Scour Analysis - Page 3 of 3
I-40 Replacement Bridge over White River
AHTD JOB BB0610: White River Str. & Apprs. (F)
Prairie County, Arkansas

GHBW Job No.: 13-017

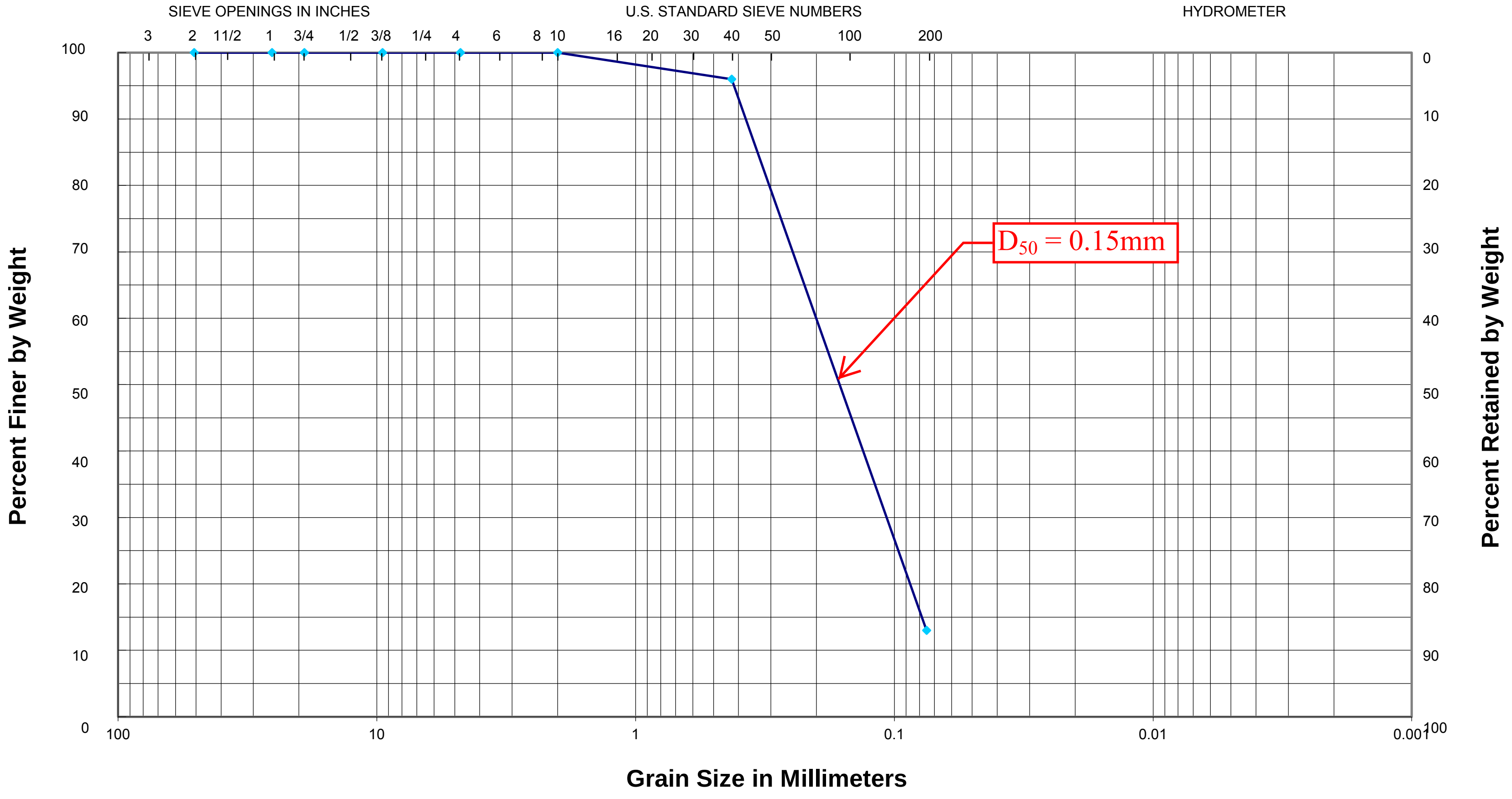
Scale: As Shown

April 8, 2014

Plate

13-017

GRAIN SIZE CURVE



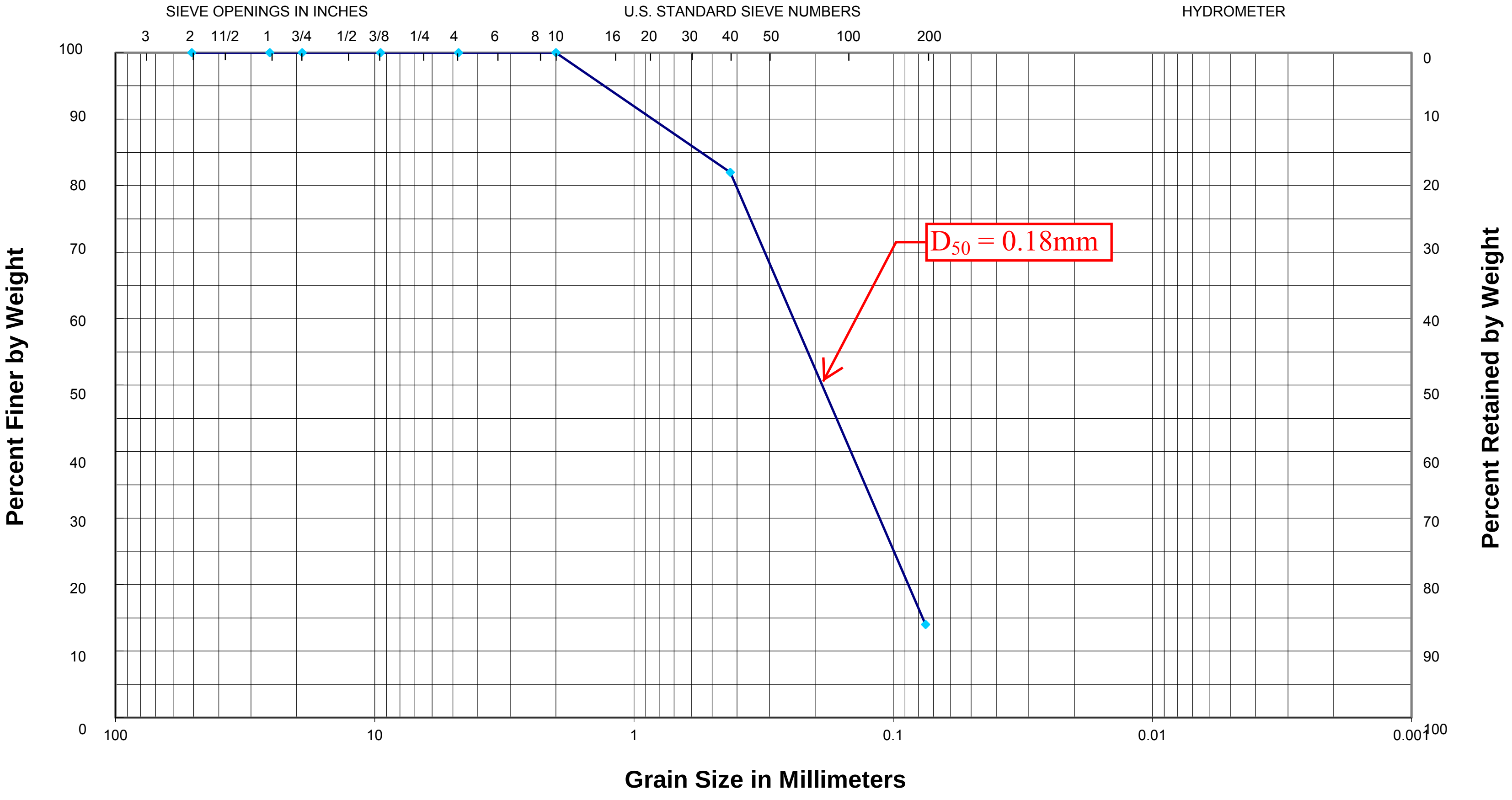
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 1, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



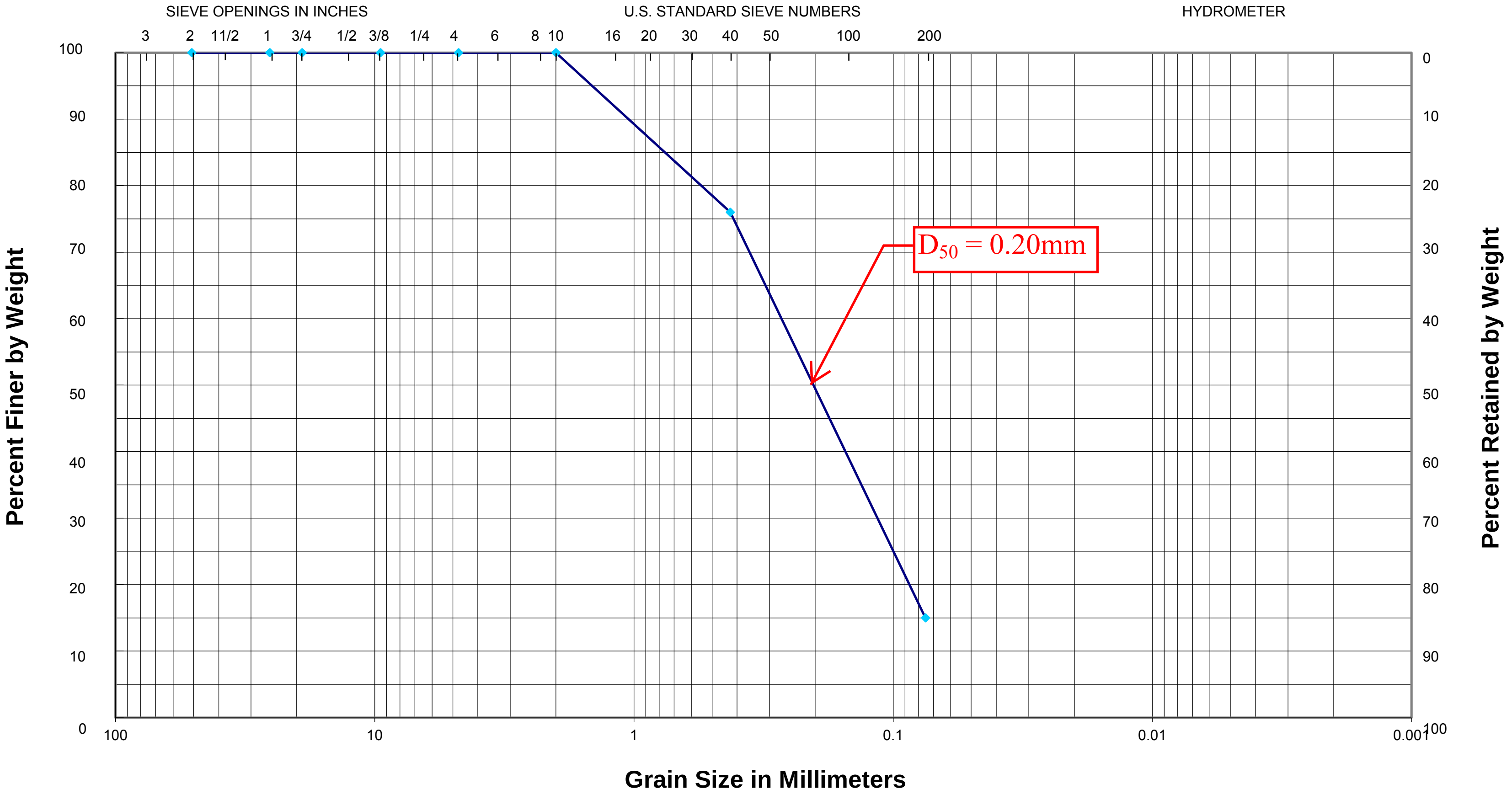
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 2, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

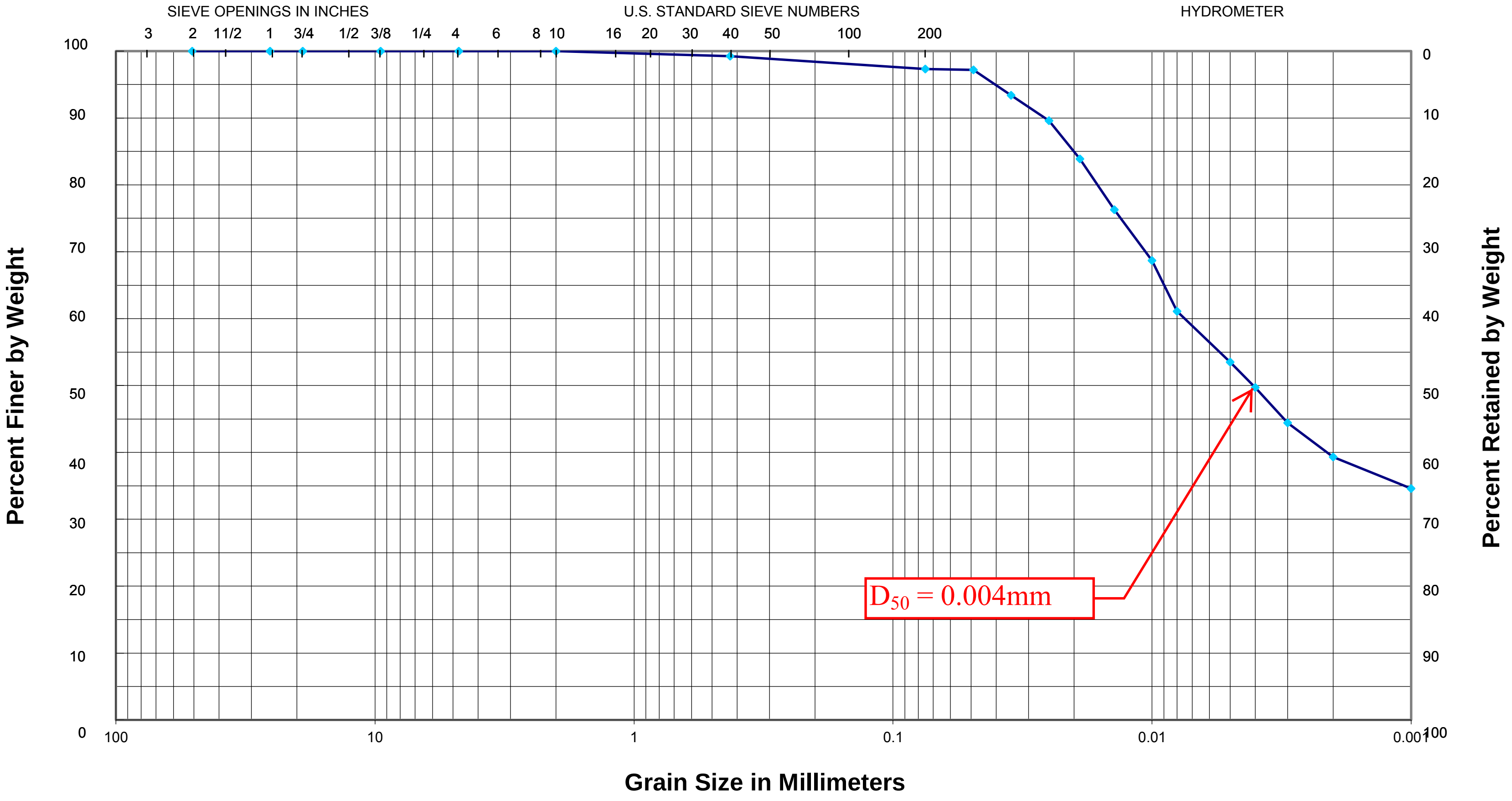
Sample: Boring 3, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

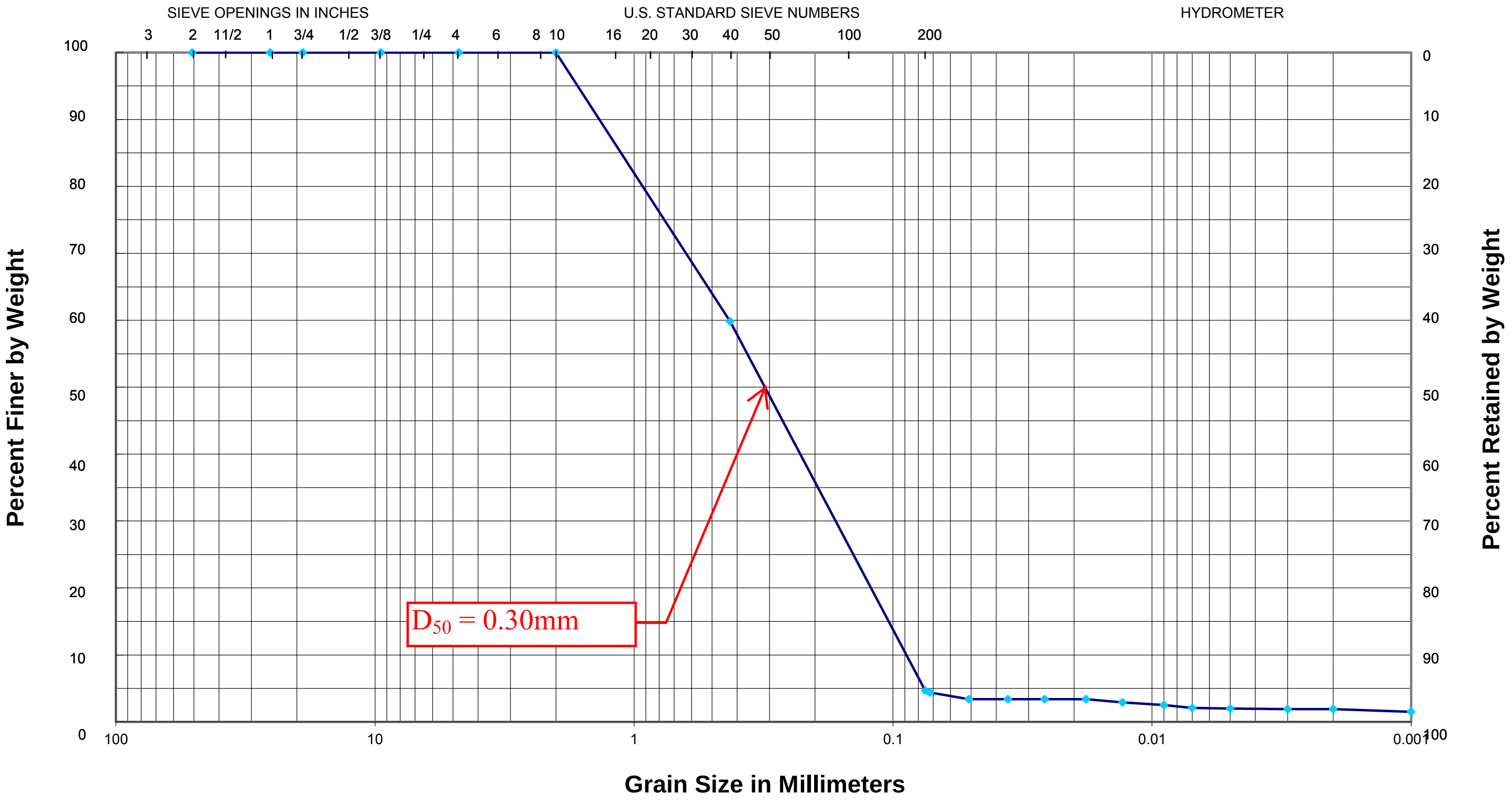
Sample: Boring 4, 7-7.5 ft
Properties: G_s = 2.700; LL = 46, PL = 19, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE

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



D_{50} = 0.30mm

GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

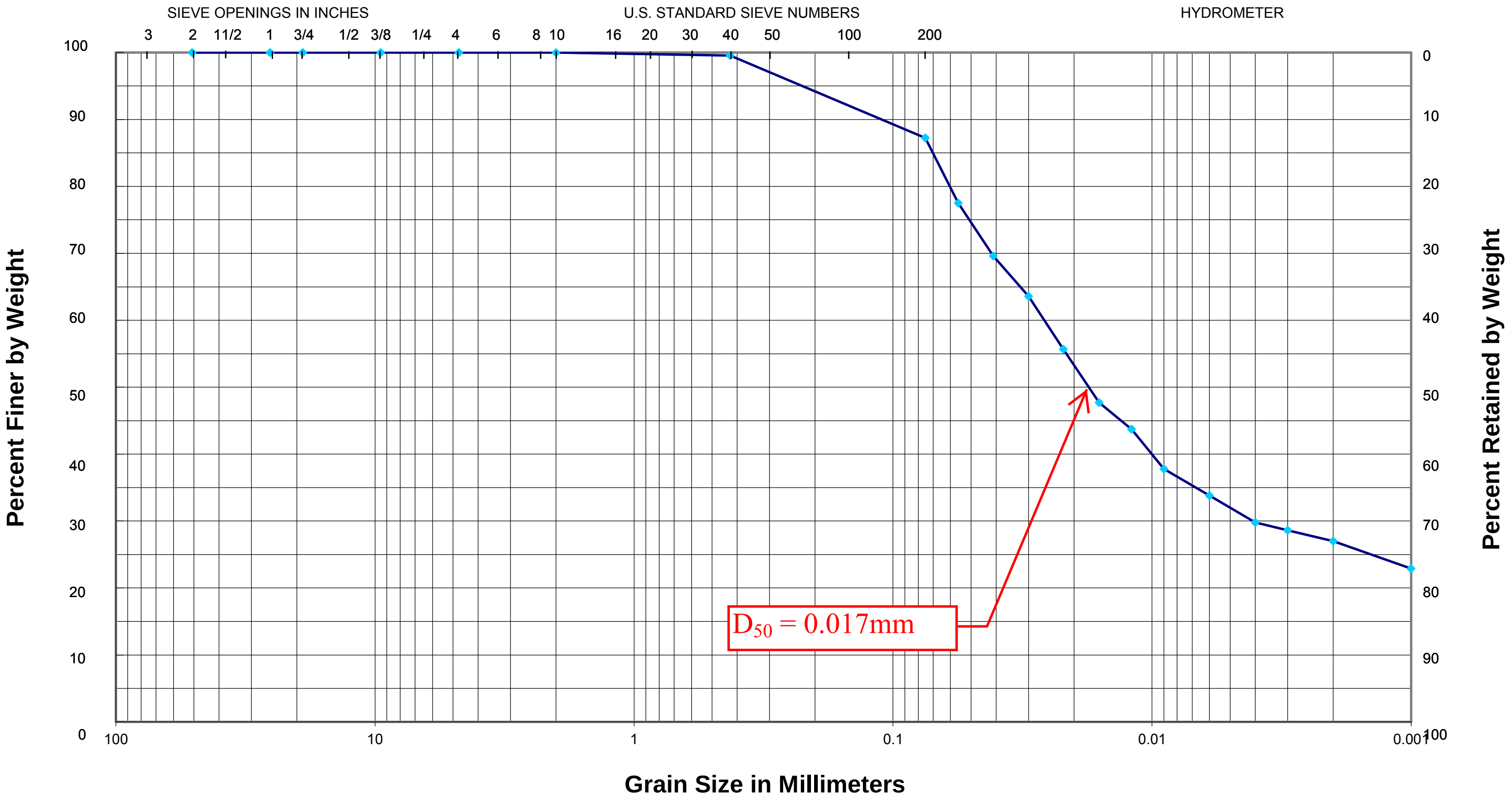
Sample: Boring 4, 14-15 ft
Properties: G_s = 2.653; Non-plastic

Description: Gray and brown fine to medium sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



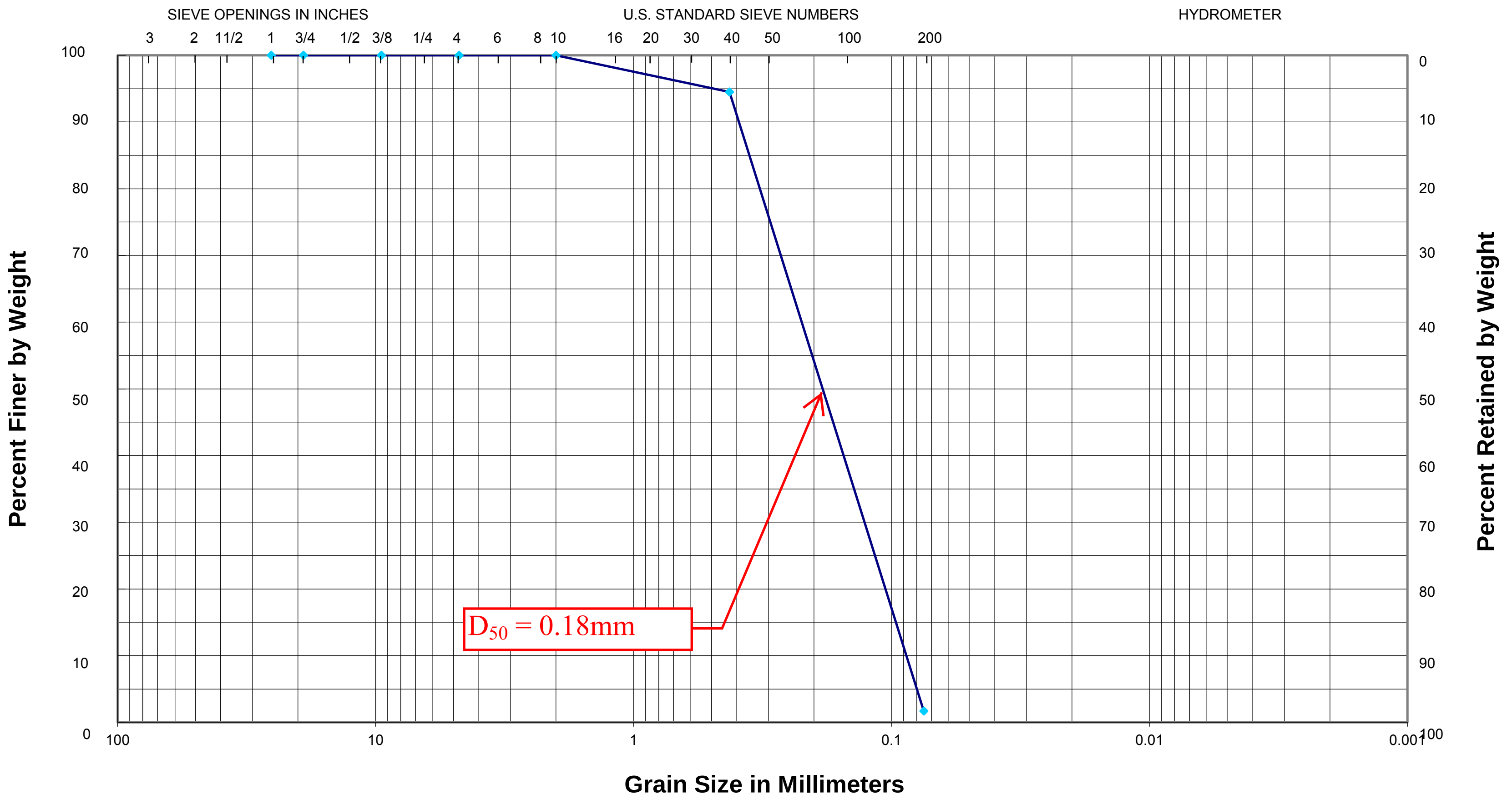
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 5, 3-3.5 ft
Properties: G_s = 2.637; LL = 33, PL = 17, PI = 16

Description: Dark gray and brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



D_{50} = 0.18mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

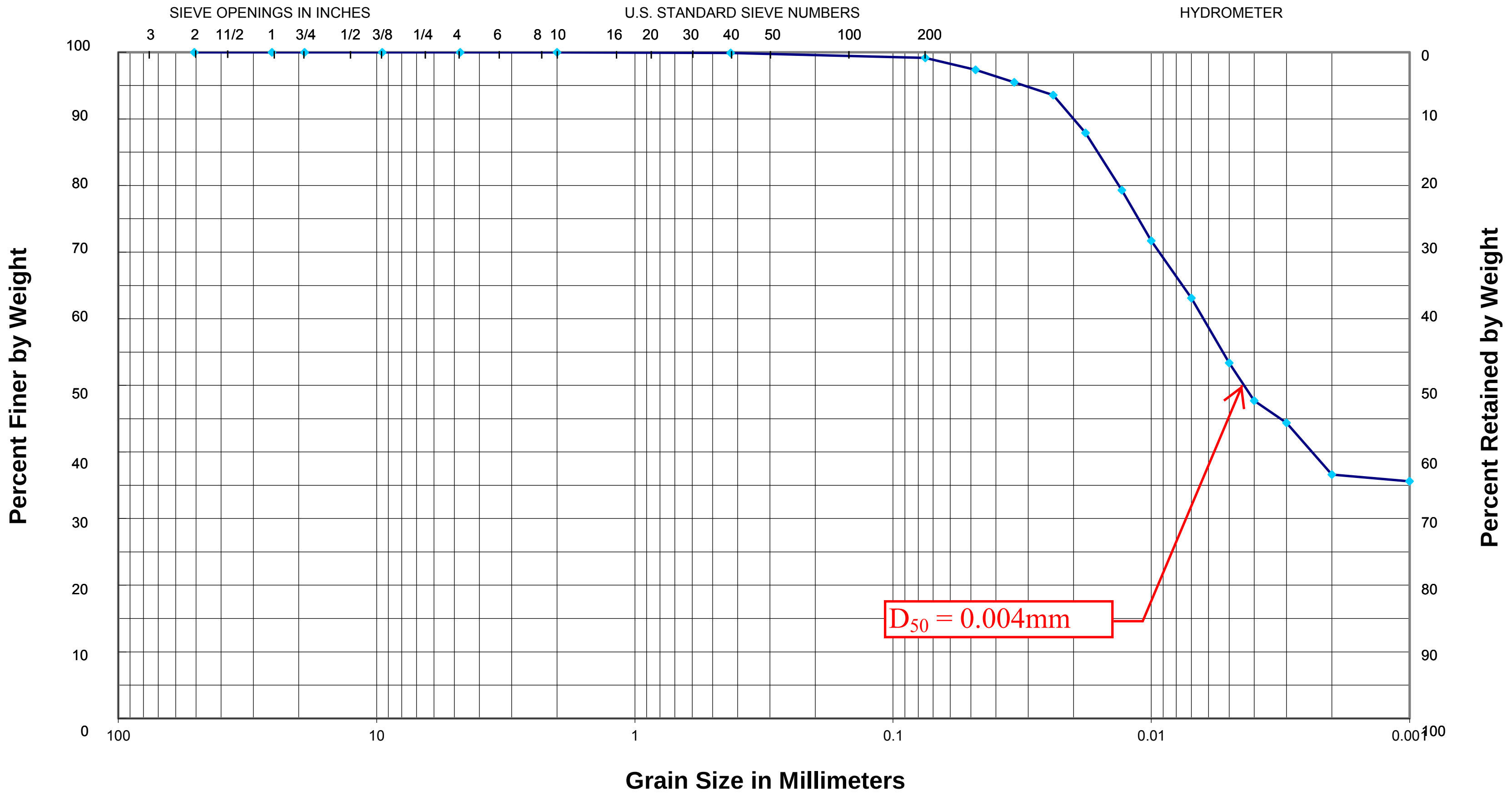
Sample: Boring 5, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray and tan fine sand w/trace medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



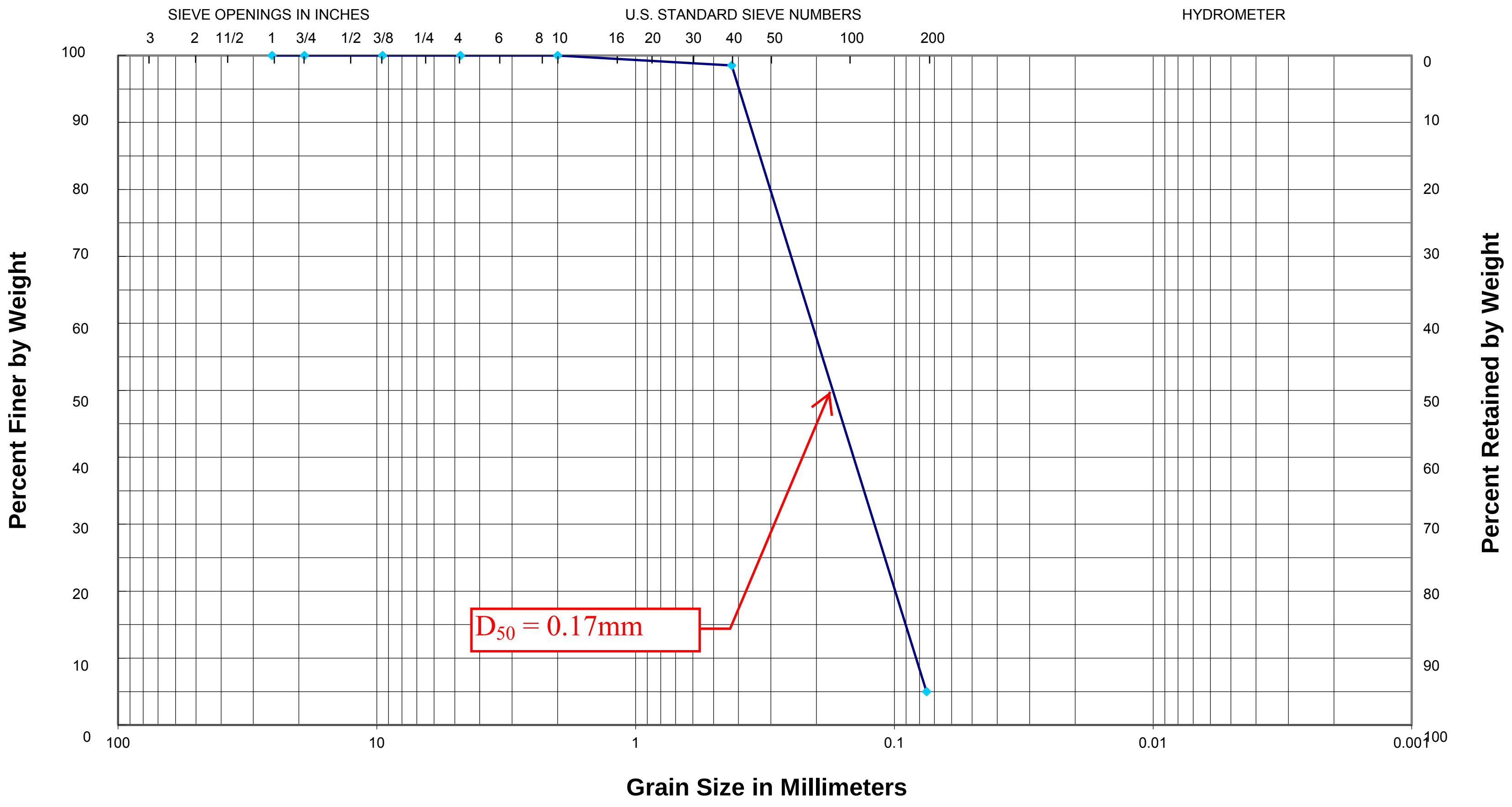
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 6, 3-3.5 ft
Properties: G_s = 2.707; LL = 48, PL = 21, PI = 27

Description: Gray and brown silty clay
Classification: USCS = CL; AASHTO = A-7-6

13-017

GRAIN SIZE CURVE



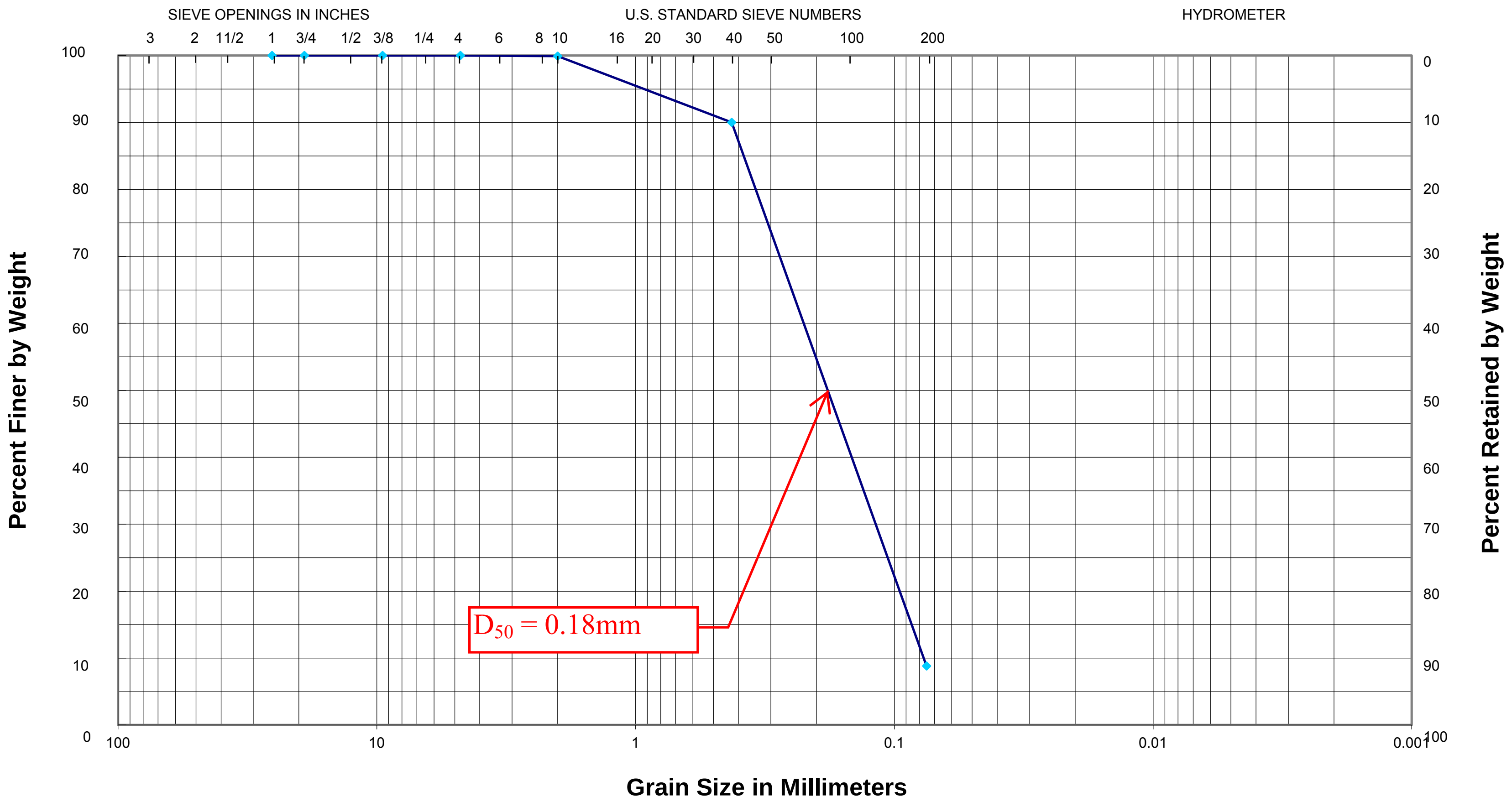
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 6, 19-20 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

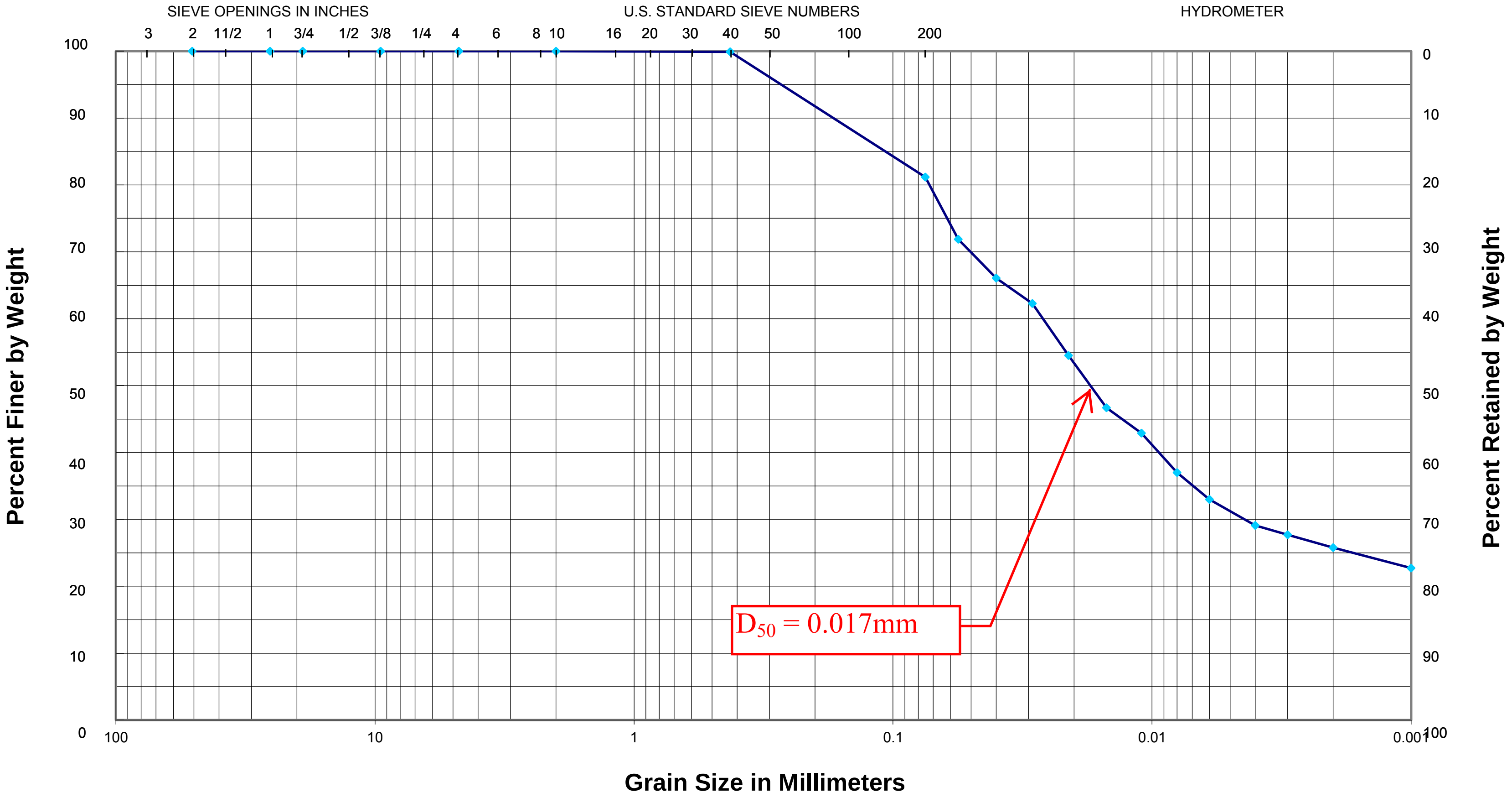
Sample: Boring 7, 24-25 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



D_{50} = 0.017mm

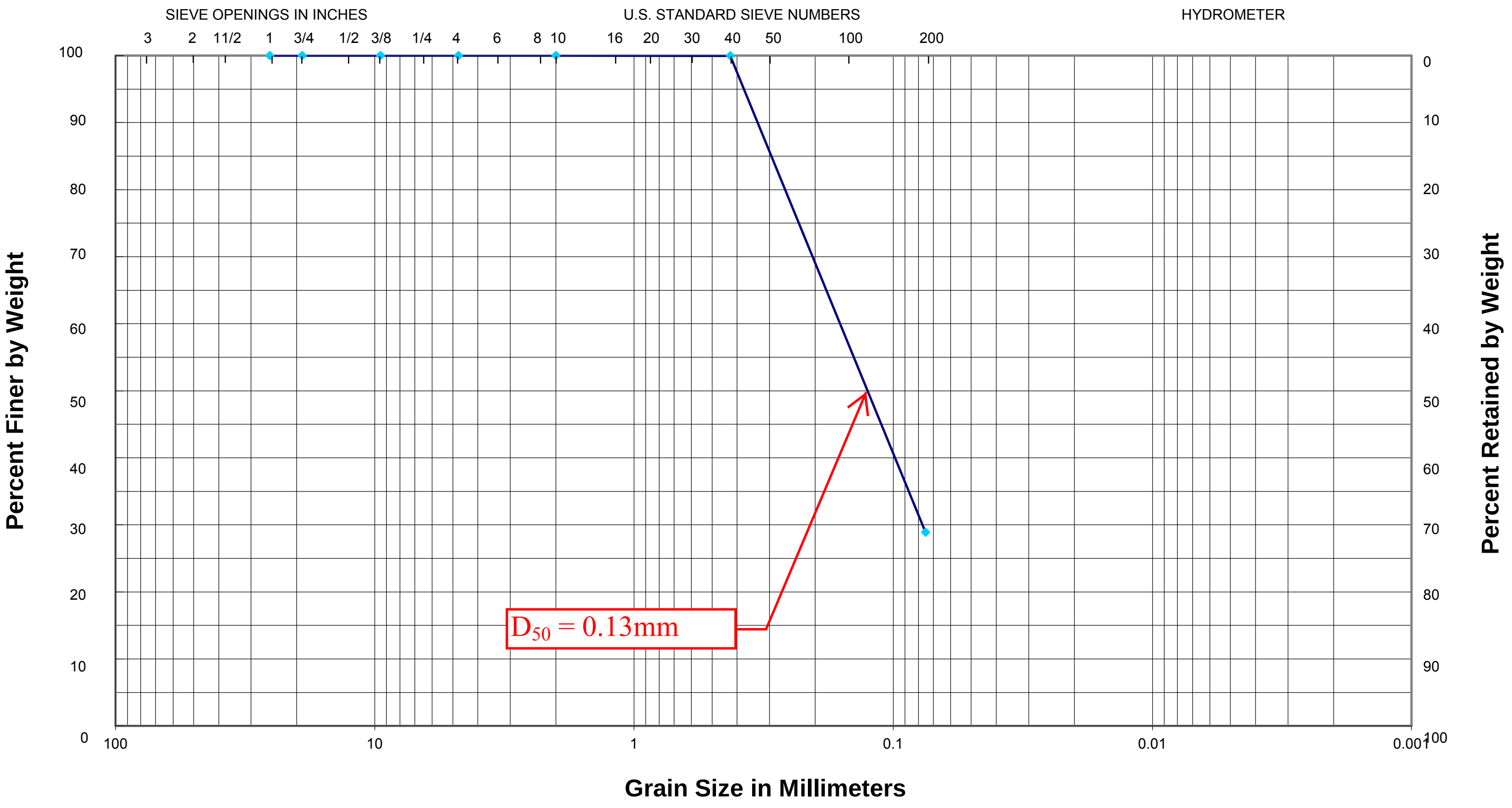
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 8, 7-7.5 ft
Properties: G_s = 2.697; LL = 37, PL = 15, PI = 12

Description: Gray and brown silty clay, sandy
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE



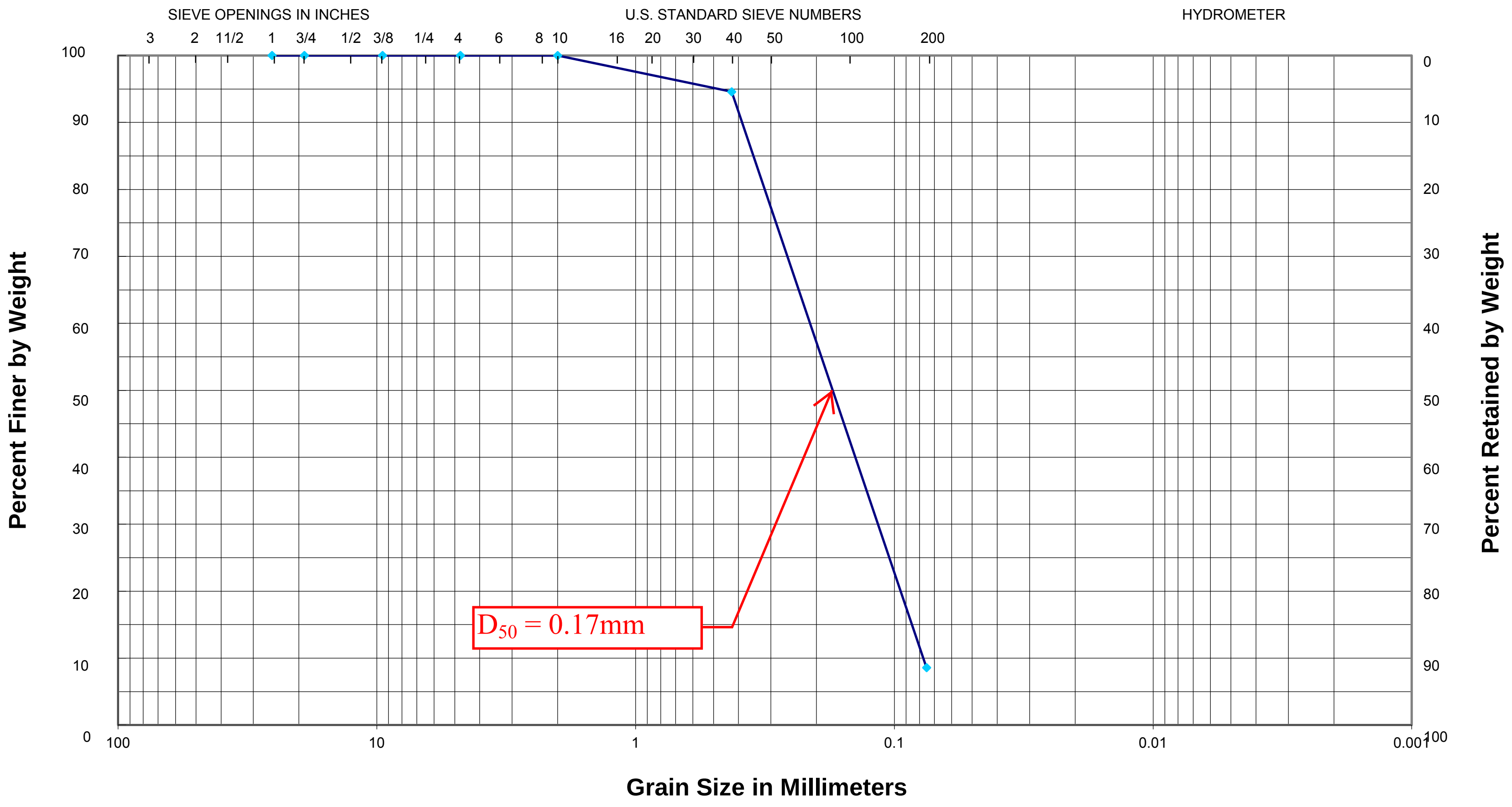
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 8, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



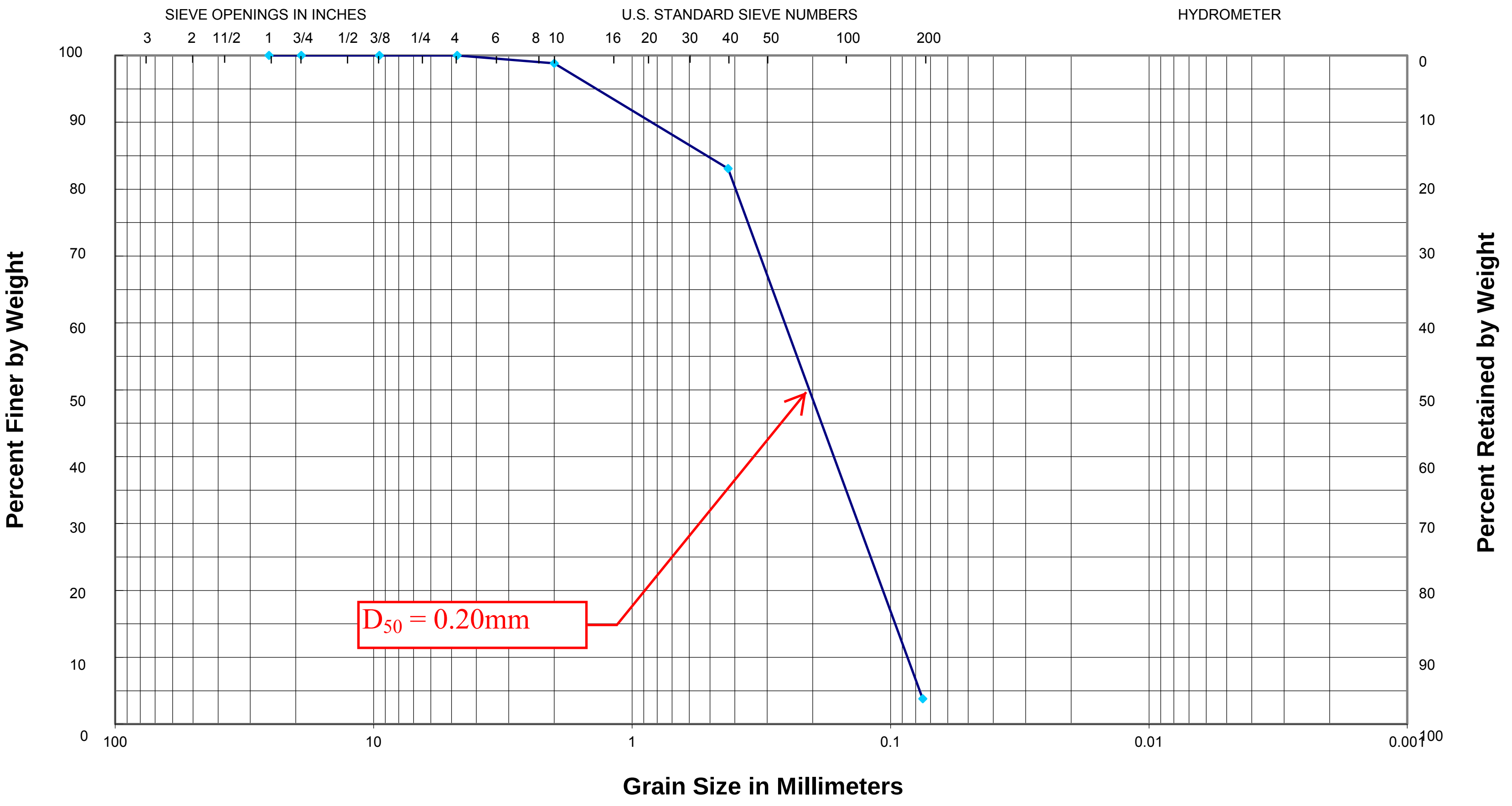
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 9, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



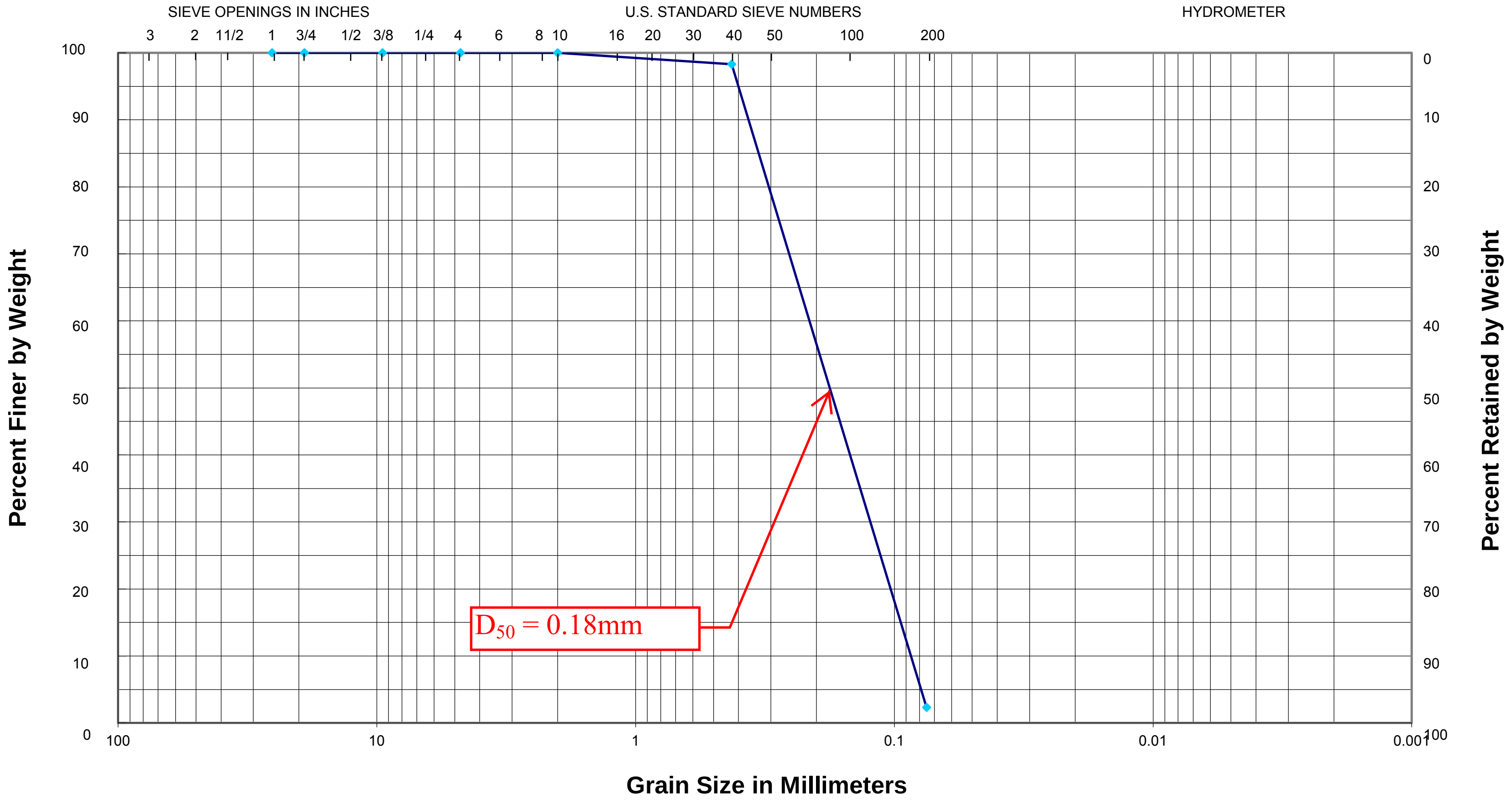
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 10, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

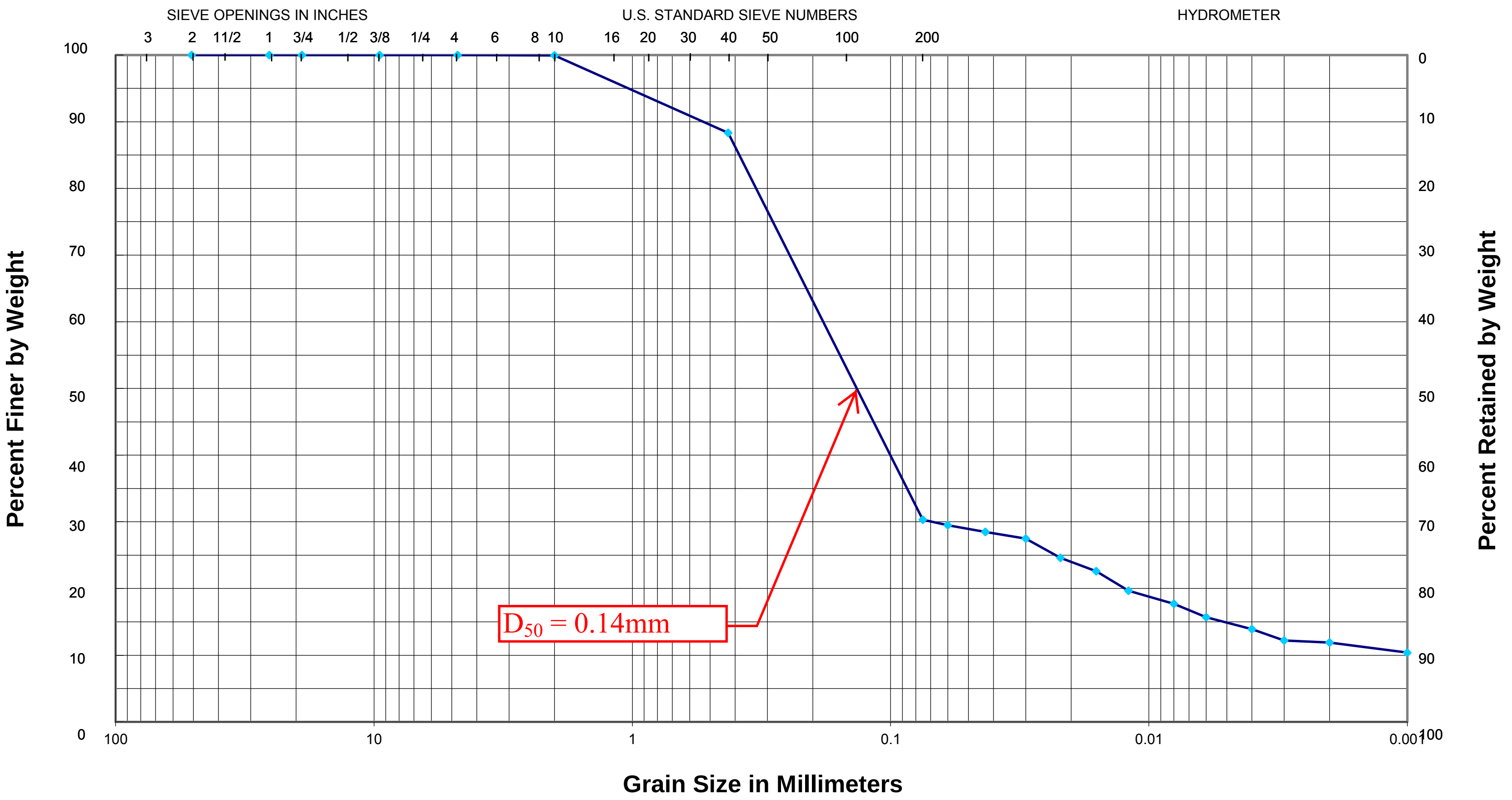
Sample: Boring 11, 29-30 ft
Atterberg Limits: Non-plastic

Description: Gray fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



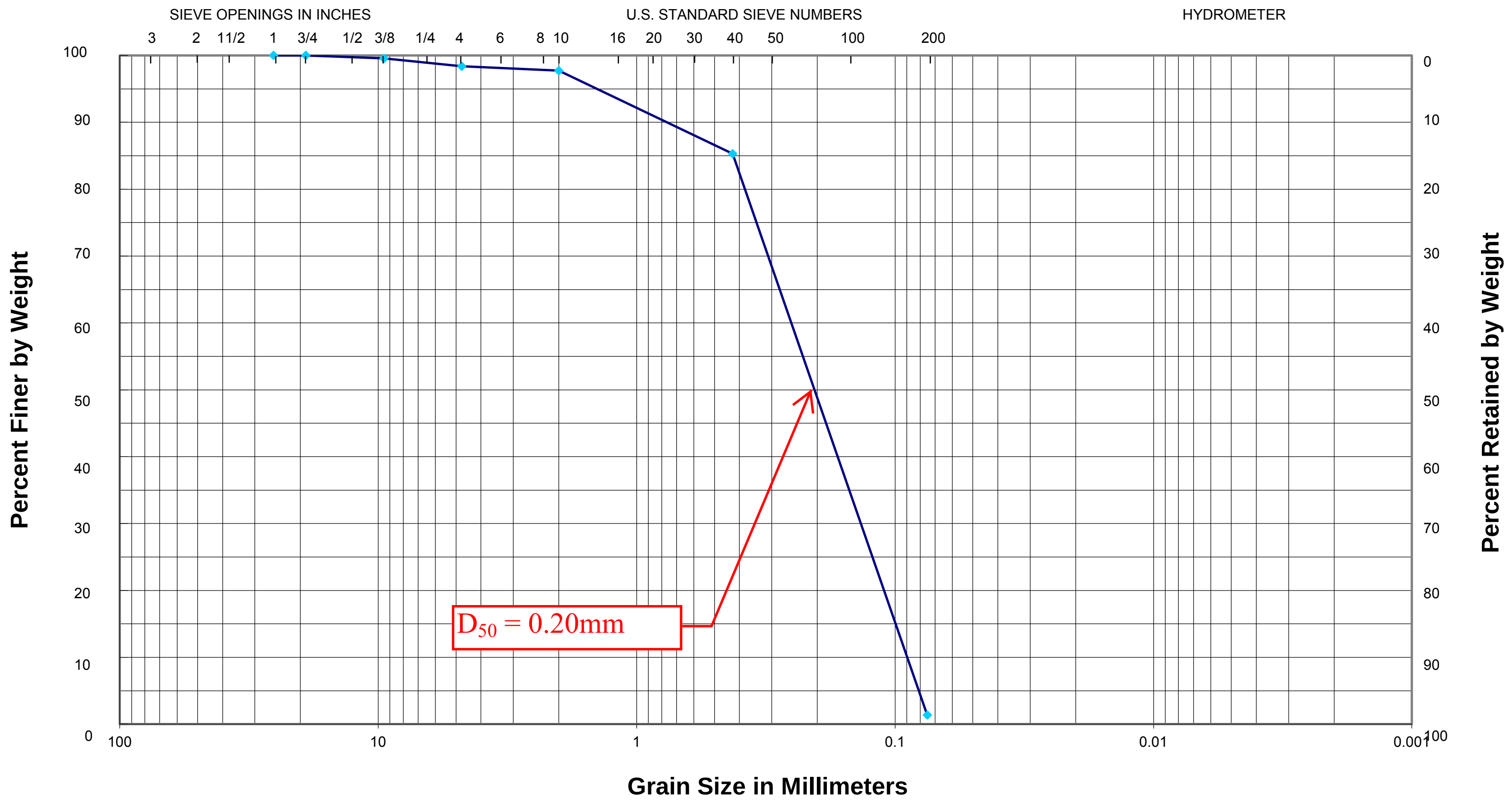
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 13, 6.5-7.5 ft
Properties: G_s = 2.661; Non-plastic

Description: Brown and gray silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

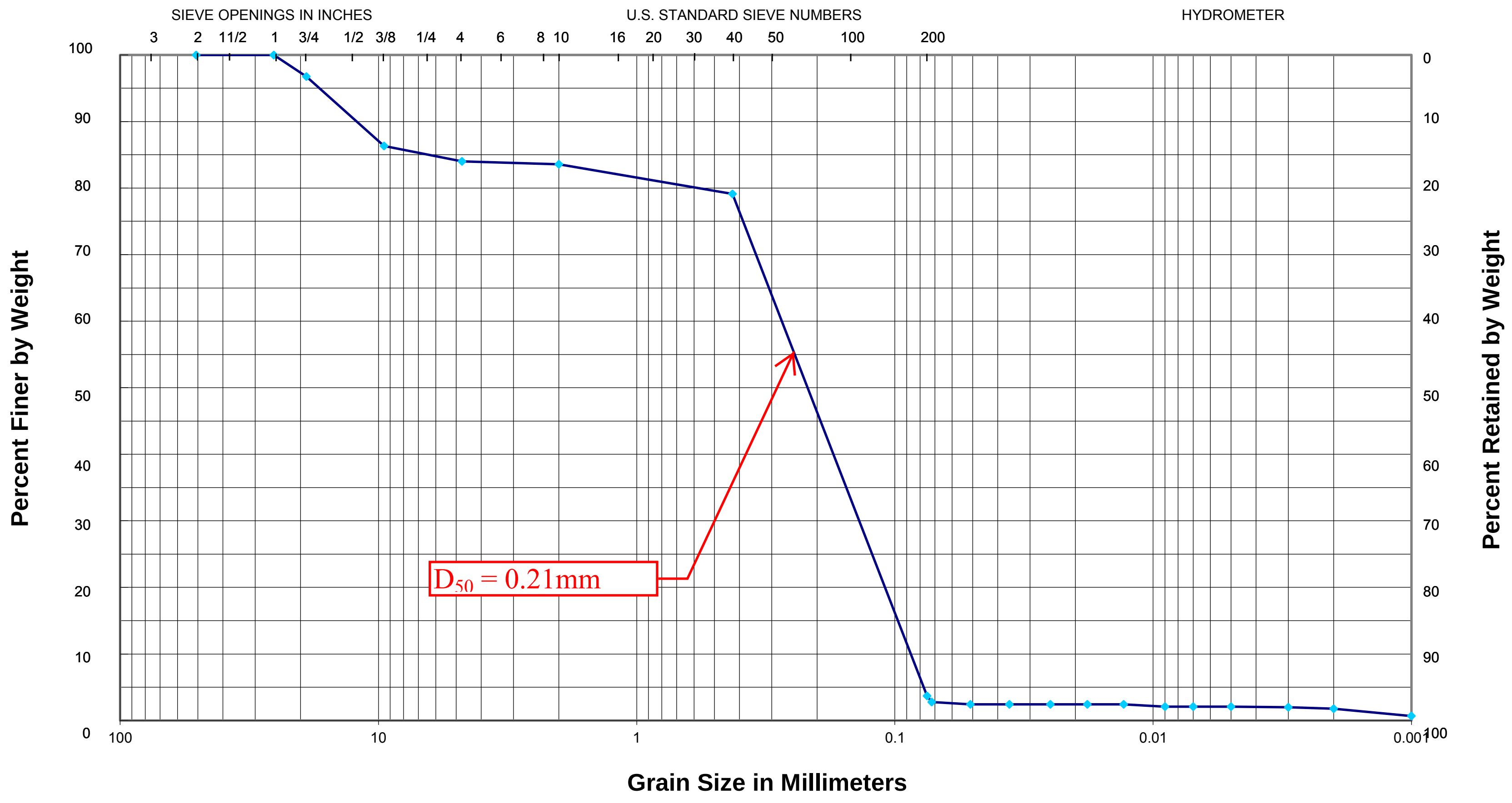
Sample: Boring 13A, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand w/a little medium sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

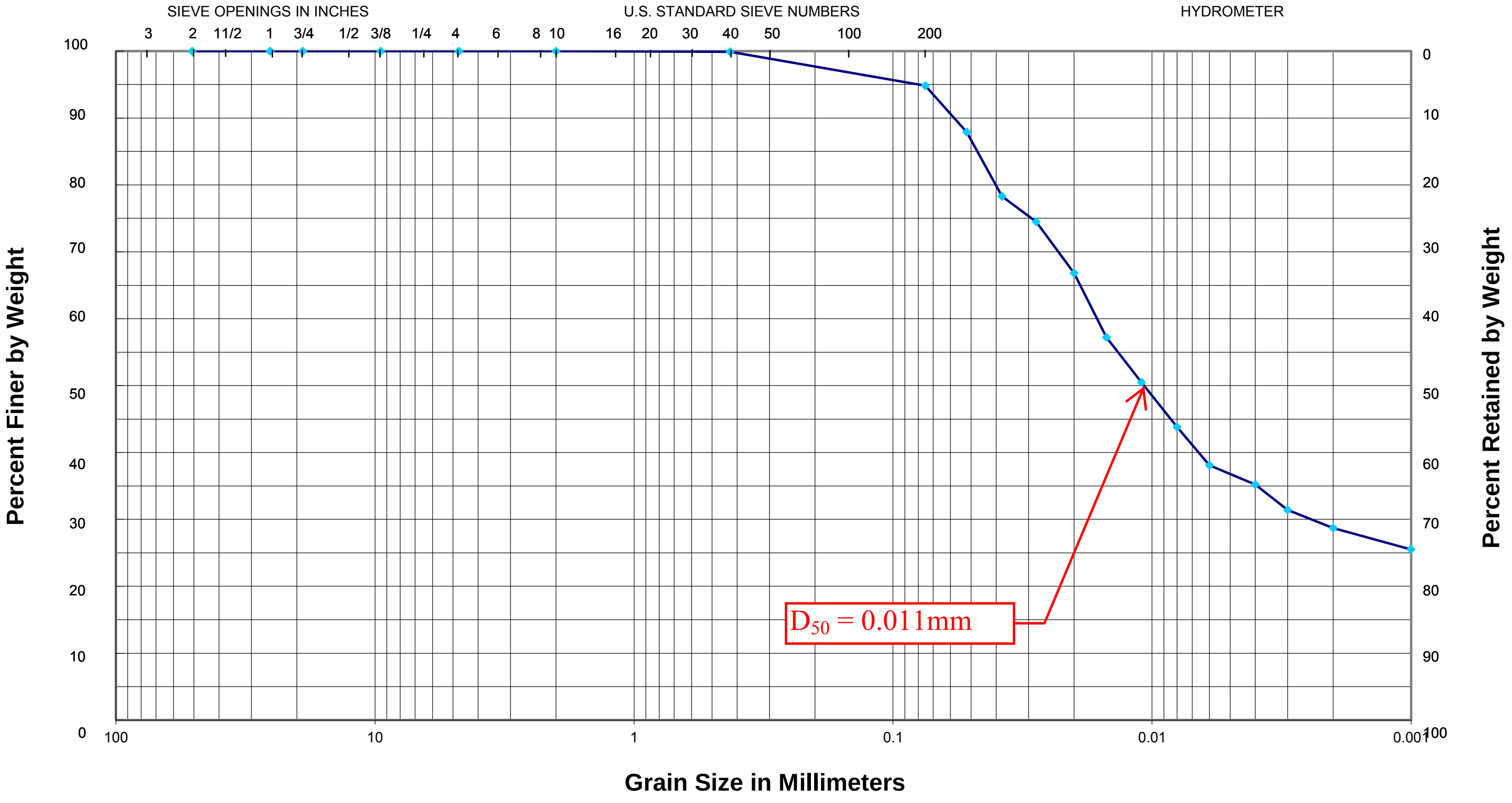
Sample: Boring 13A, 19-20 ft
Properties: G_s = 2.654; Non-plastic

Description: Brown and tan fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

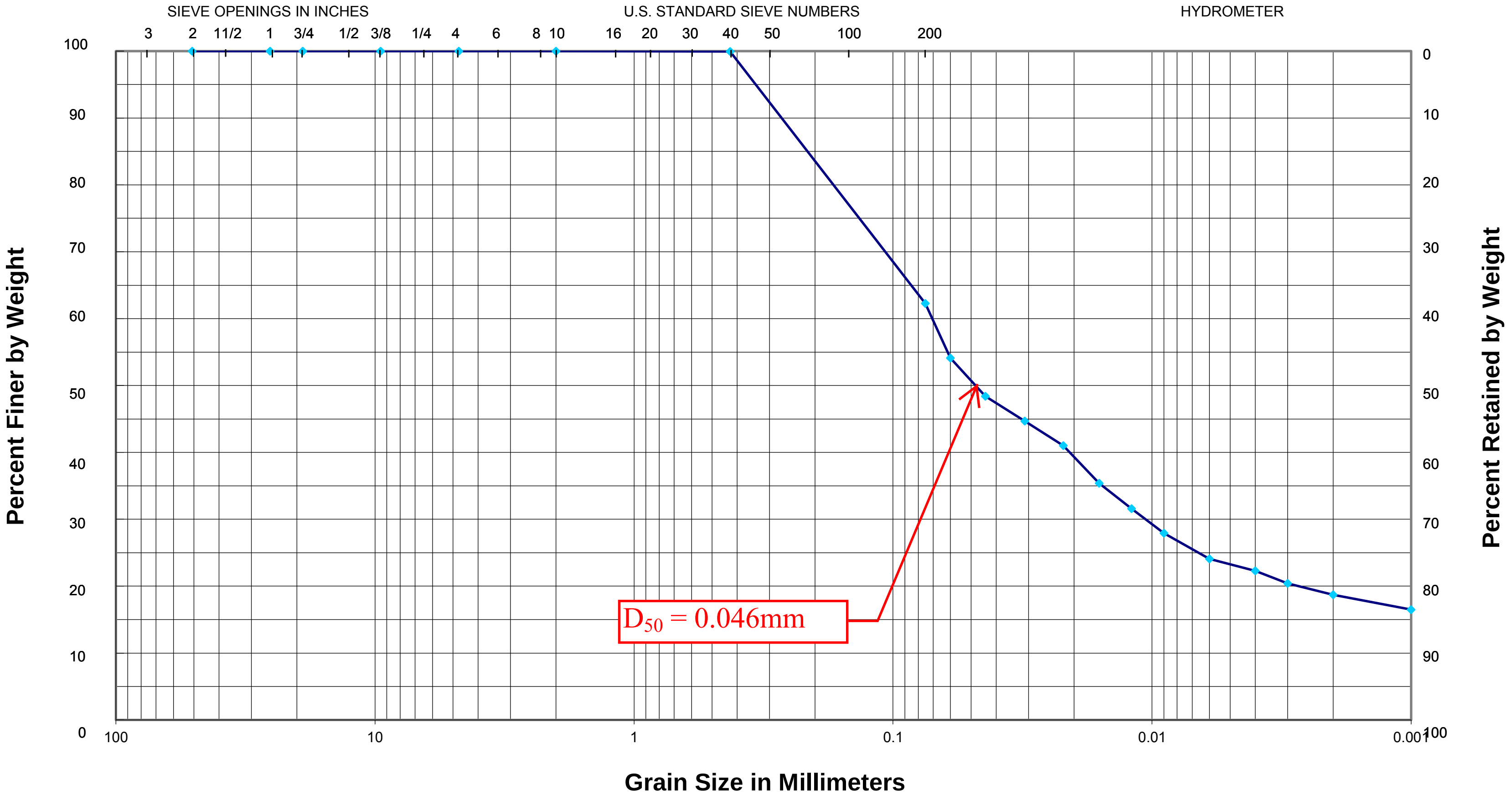
Sample: Boring 14, 4.5-5.5 ft
Properties: G_s = 2.669; LL = 34, PL = 19, PI = 15

Description: Brown silty clay
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

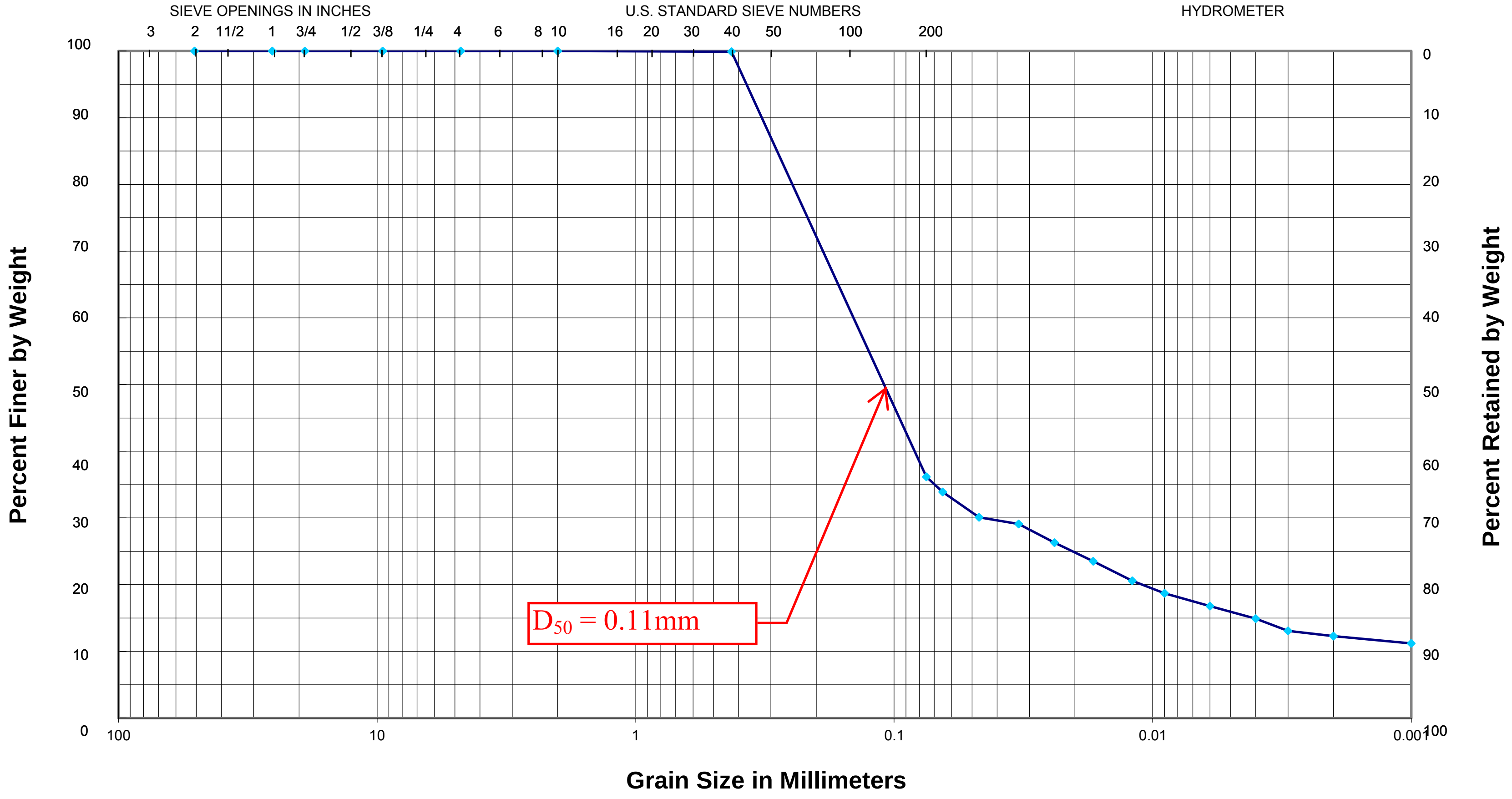
Sample: Boring 14, 14-15 ft
Properties: G_s = 2.668; LL = 22, PL = 17, PI = 5

Description: Gray clayey silt
Classification: USCS = CL-ML; AASHTO = A-4

13-017

GRAIN SIZE CURVE

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



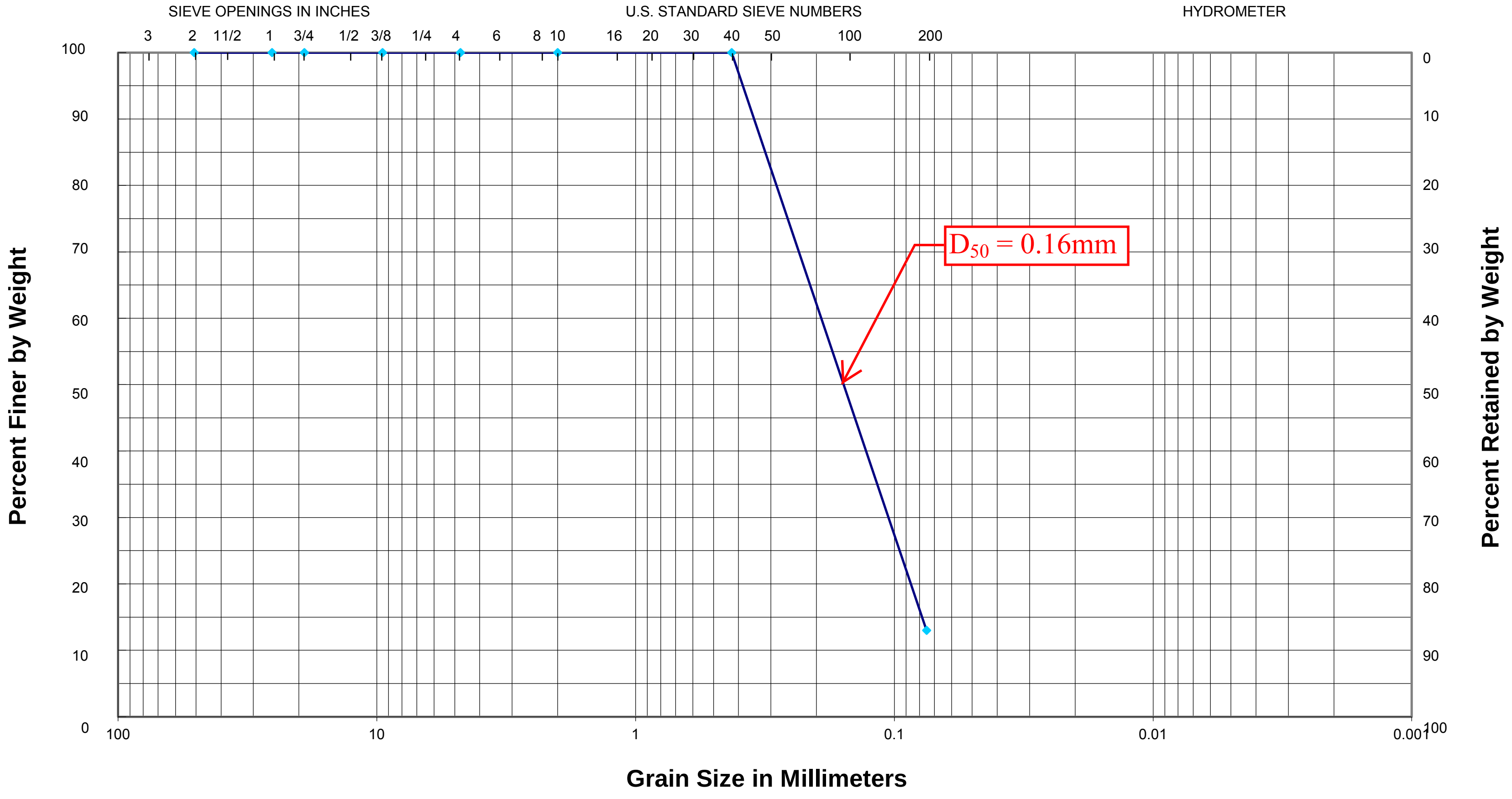
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 14, 24-24.5 ft
Properties: G_s = 2.668; Non-plastic

Description: Gray silty fine sand
Classification: USCS = SM; AASHTO = A-4

13-017

GRAIN SIZE CURVE



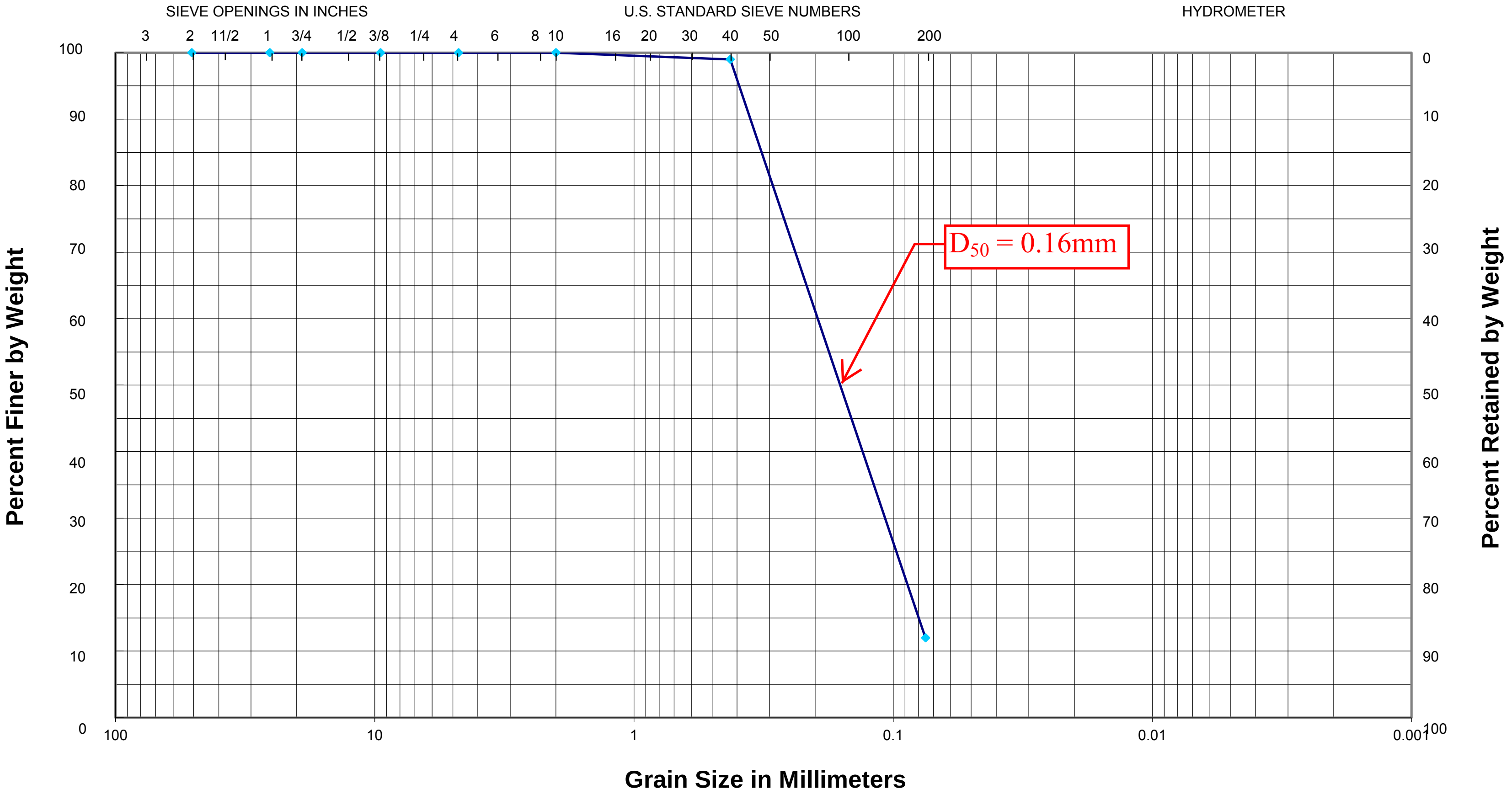
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 6.5-7.5 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



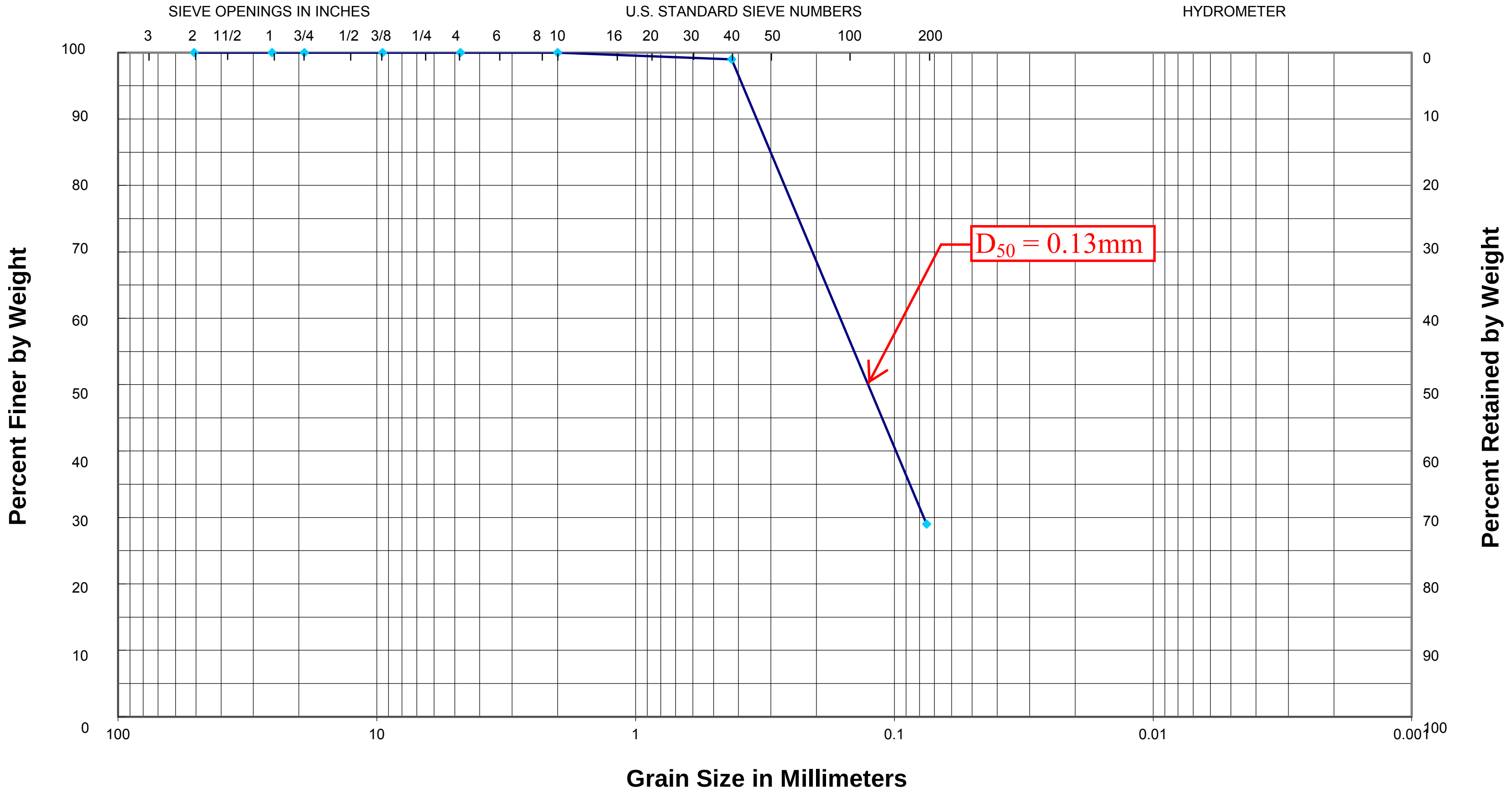
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 15, 24-25 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SP-SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



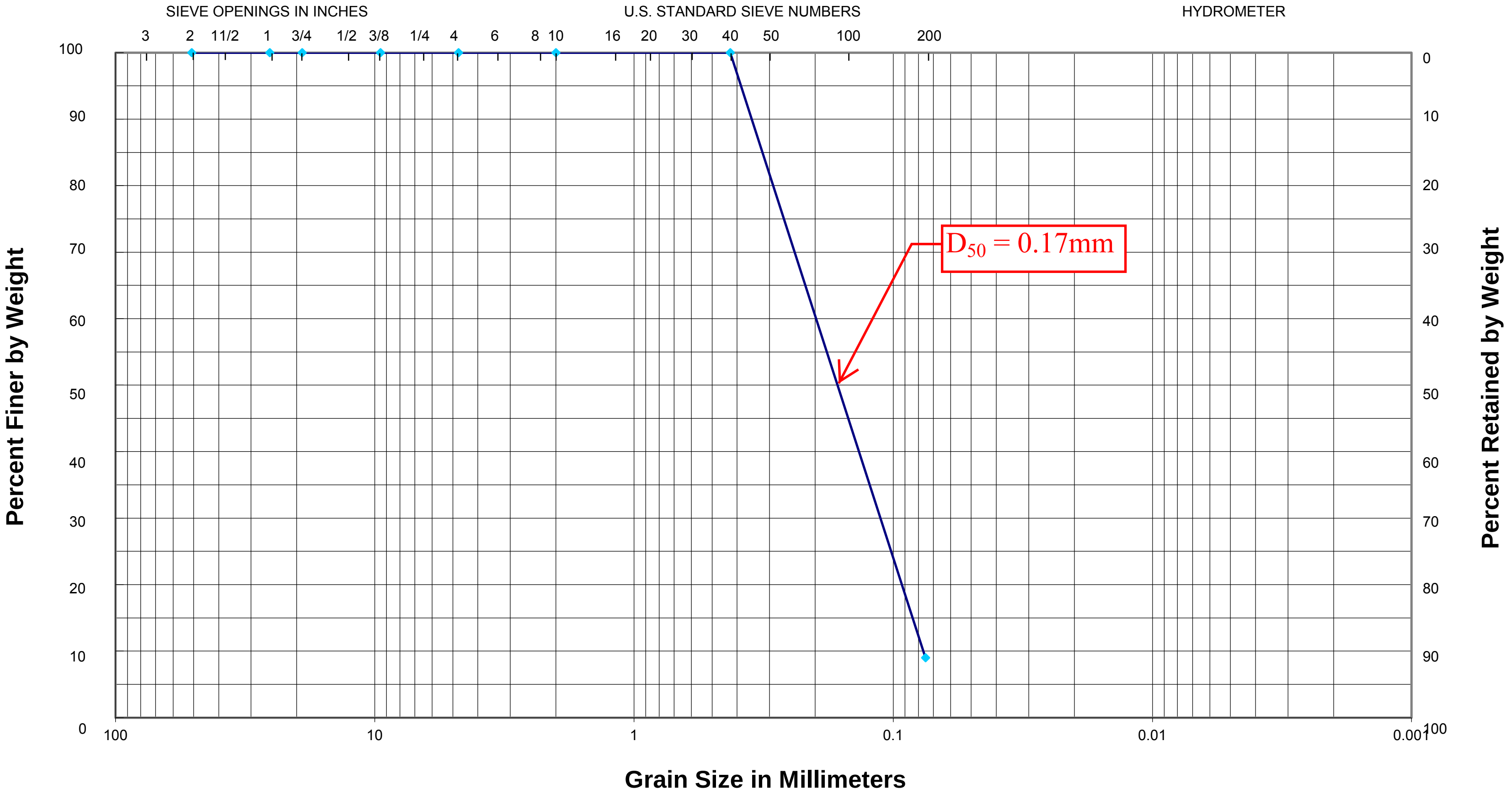
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 9-10 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



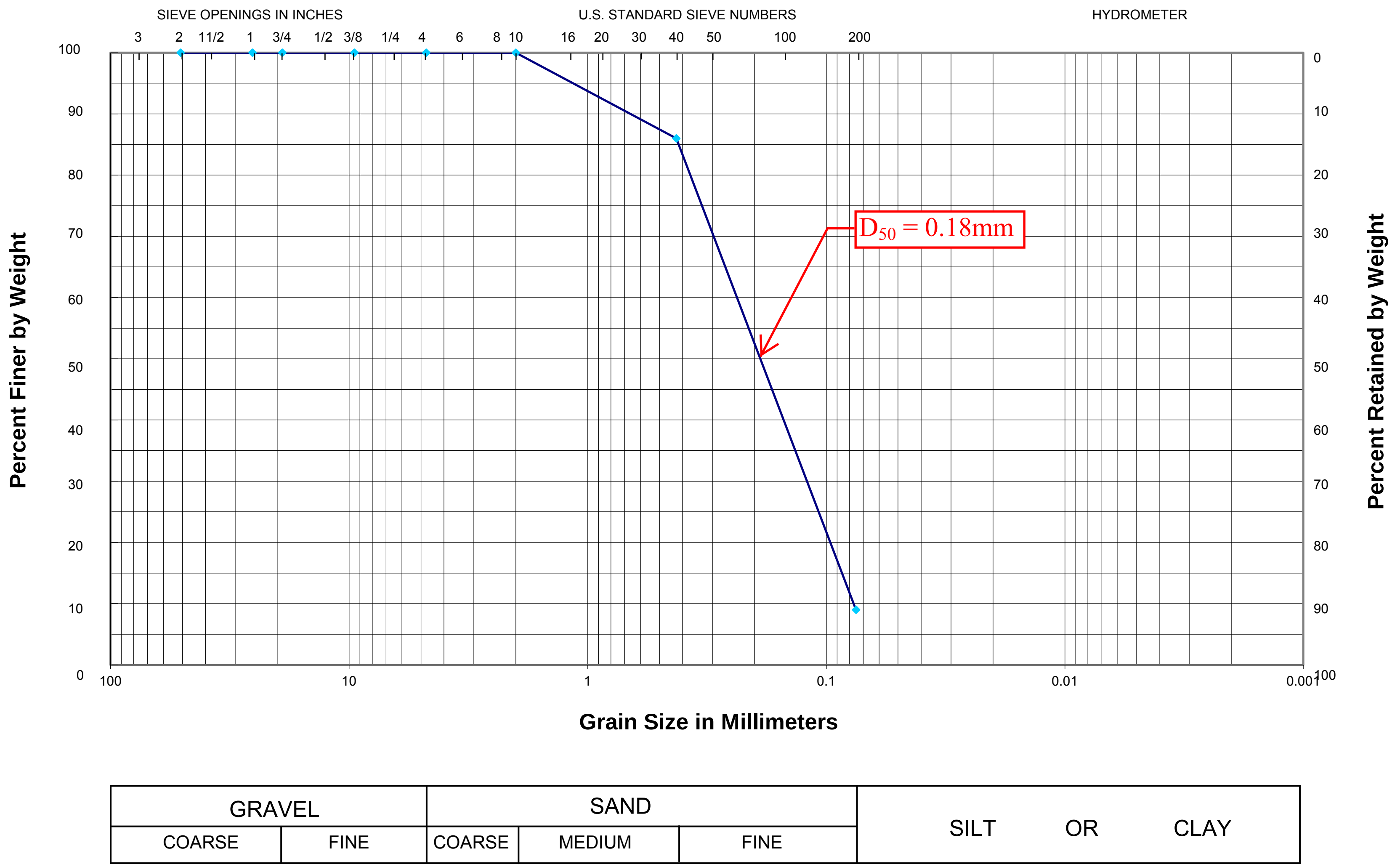
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 16, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



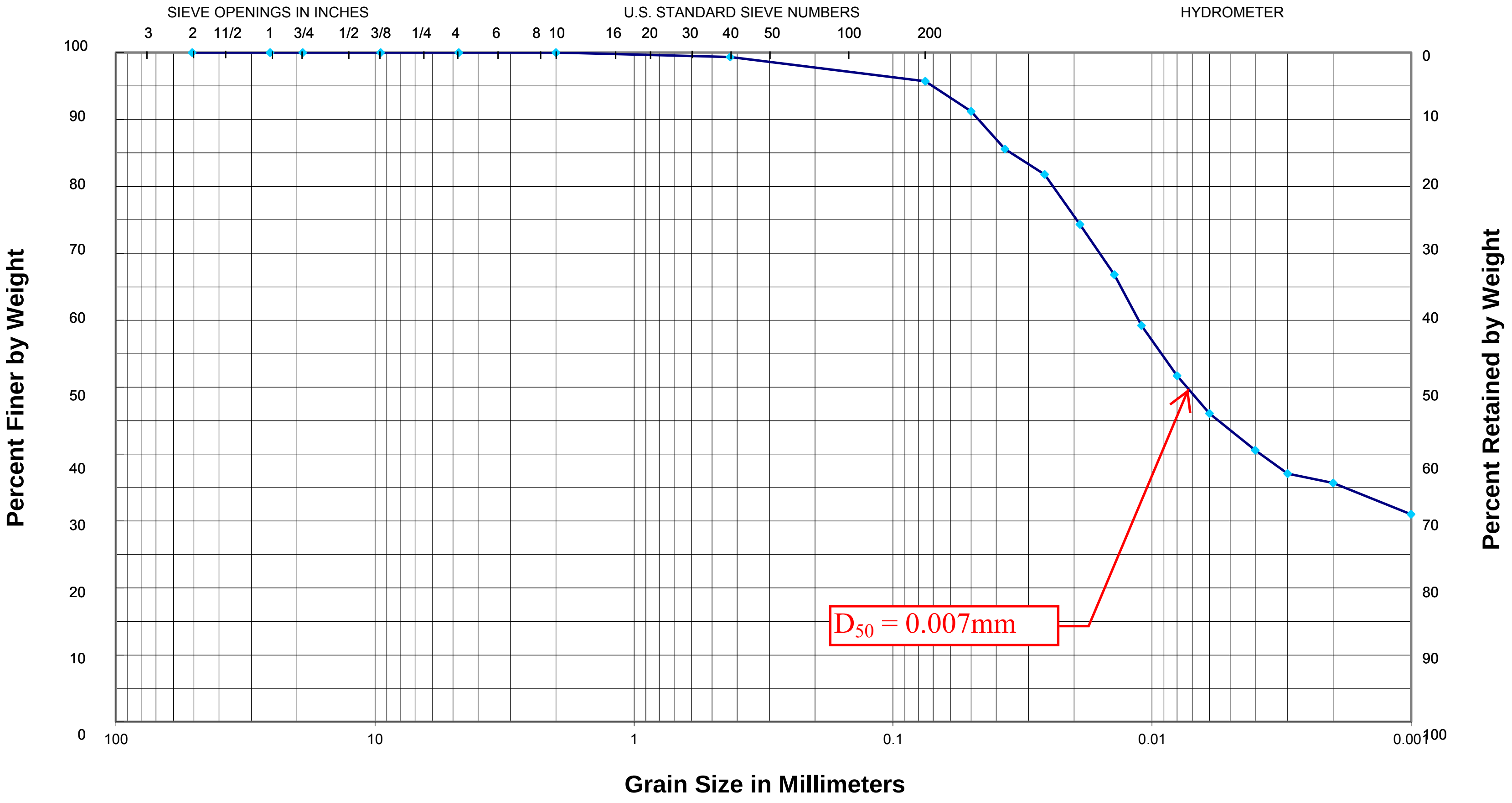
Sample: Boring 17, 29-30 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE

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



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

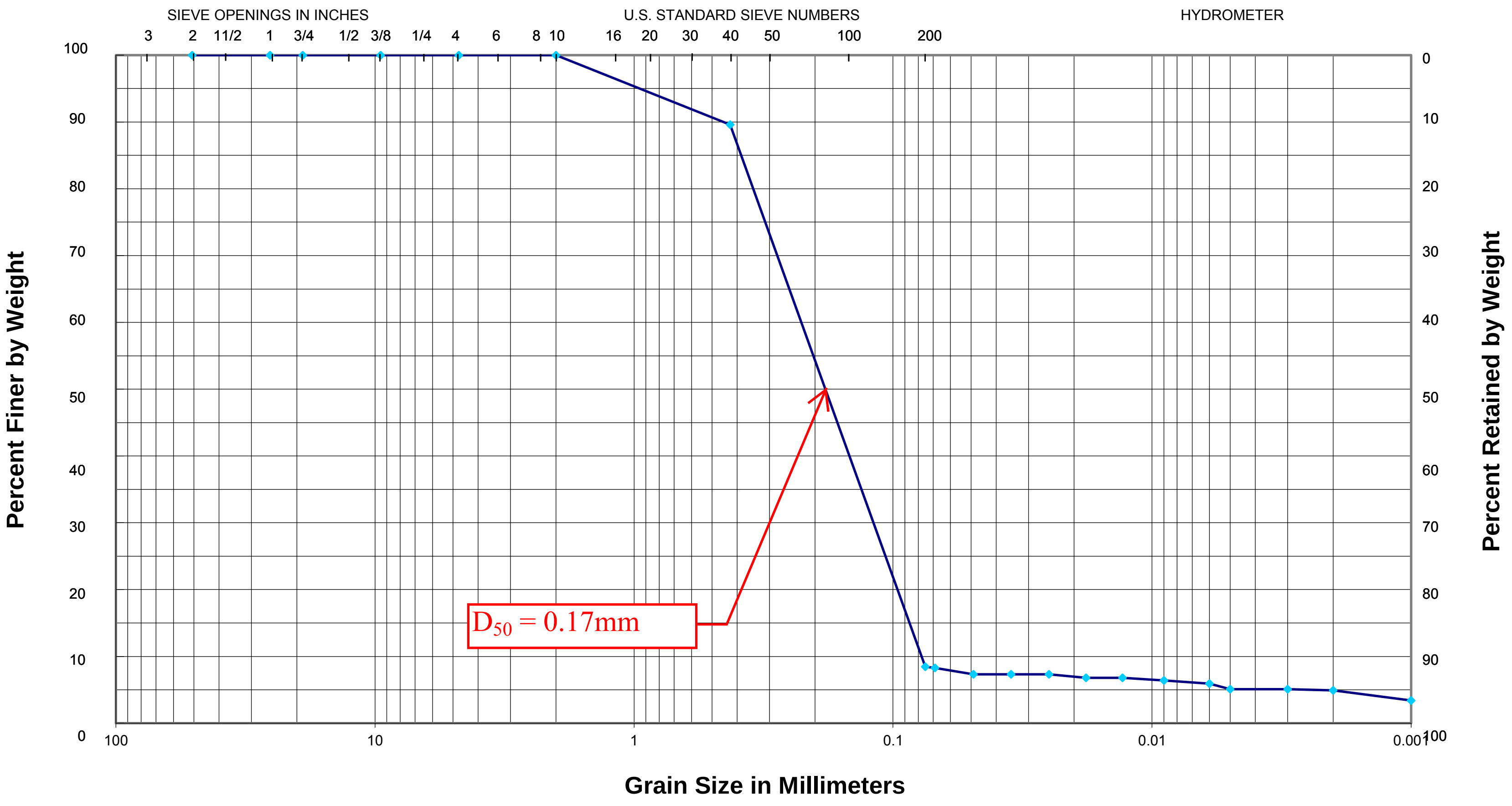
Sample: Boring 19, 6.5-7 ft
Properties: G_s = 2.696; LL = 40, PL = 18, PI = 22

Description: Gray and brown silty clay w/ferrous stains and nodules
Classification: USCS = CL; AASHTO = A-6

13-017

GRAIN SIZE CURVE

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



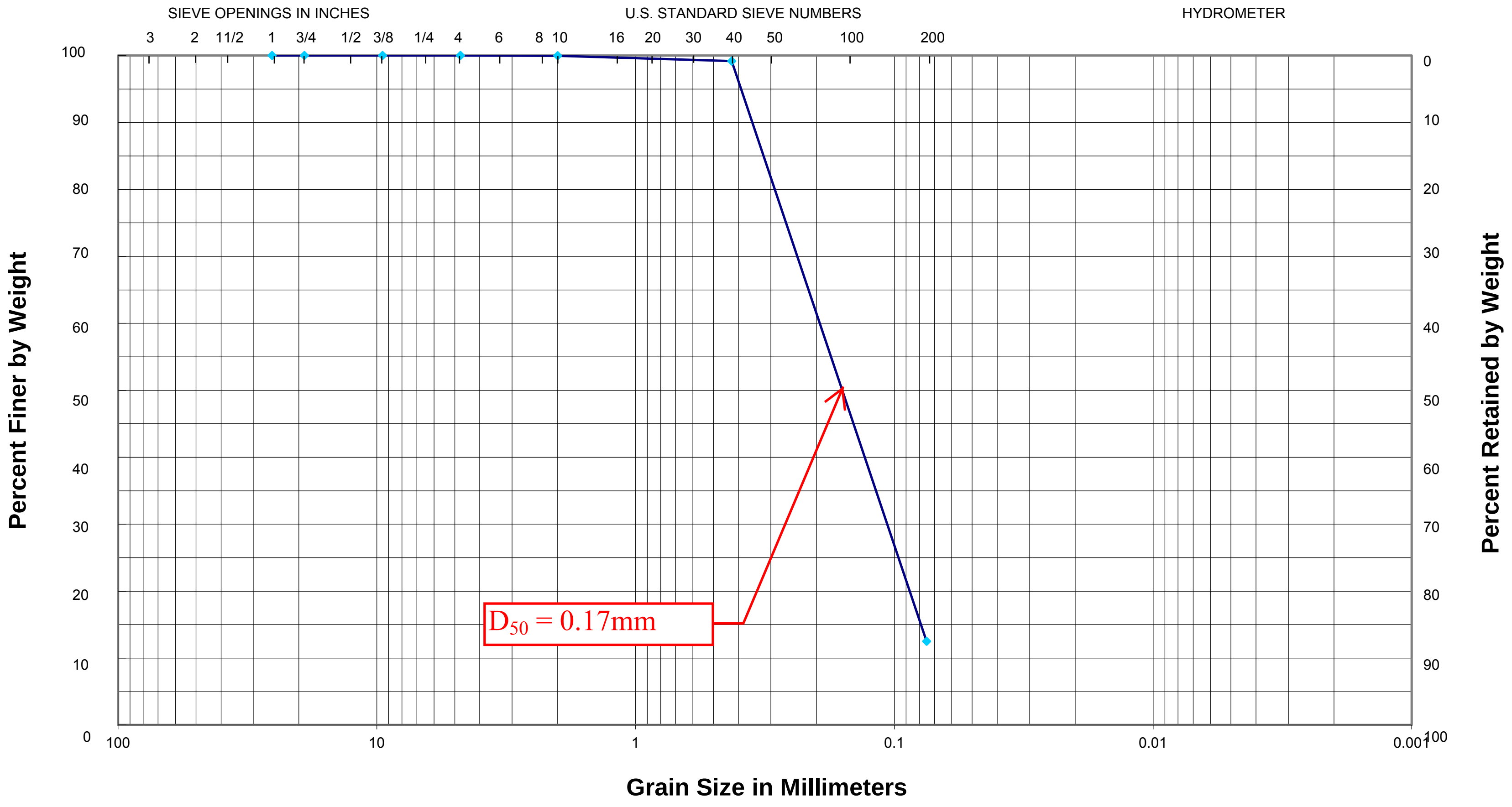
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring 19, 24-25 ft
Properties: G_s = 2.678; Non-plastic

Description: Gray fine sand, slightly silty
Classification: USCS = sp-sm; AASHTO = A-3

13-017

GRAIN SIZE CURVE



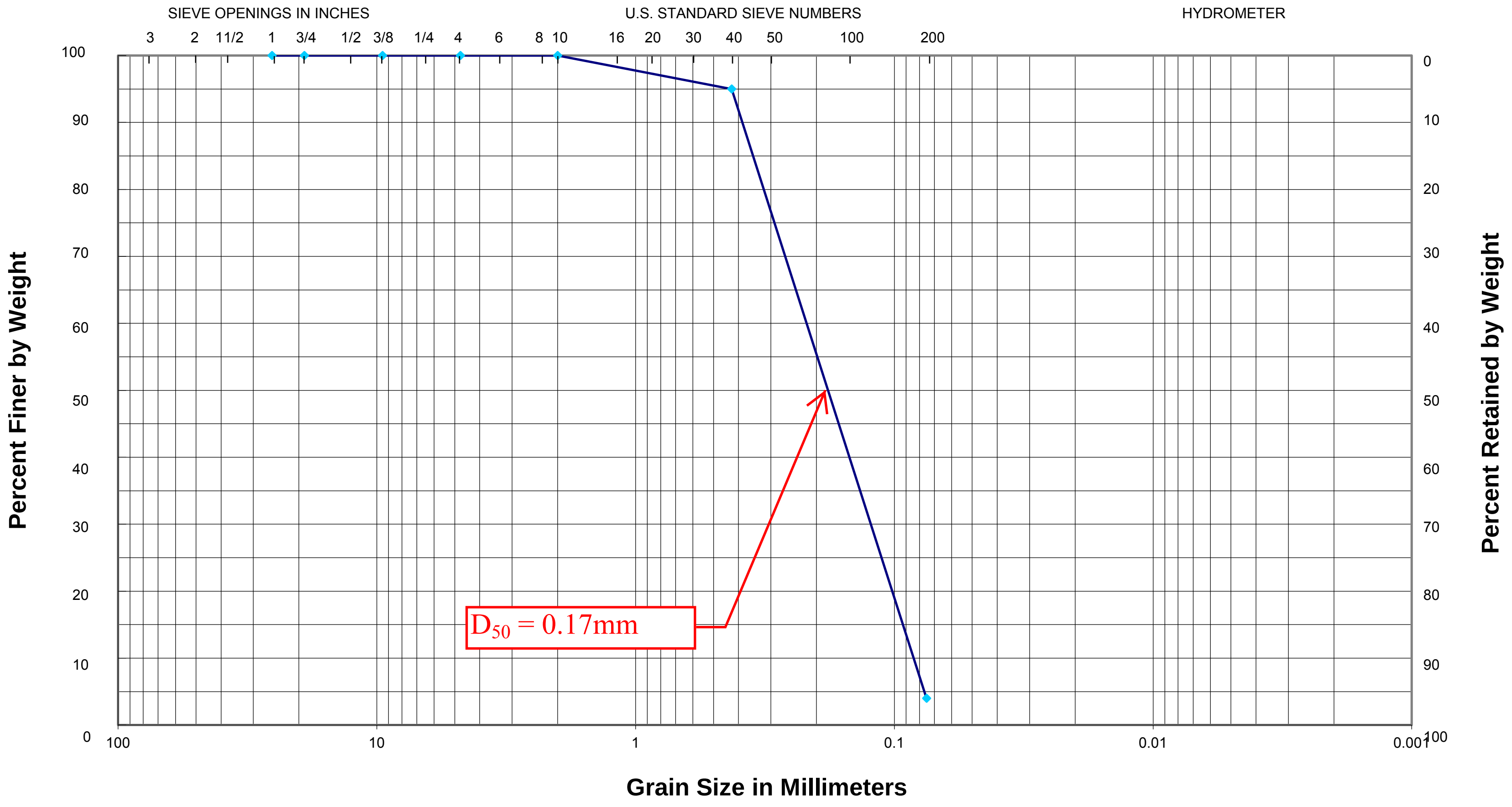
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 22, 14-15 ft
Atterberg Limits: Non-plastic

Description: Brown silty fine sand
Classification: USCS = SM; AASHTO = A-2-4

13-017

GRAIN SIZE CURVE



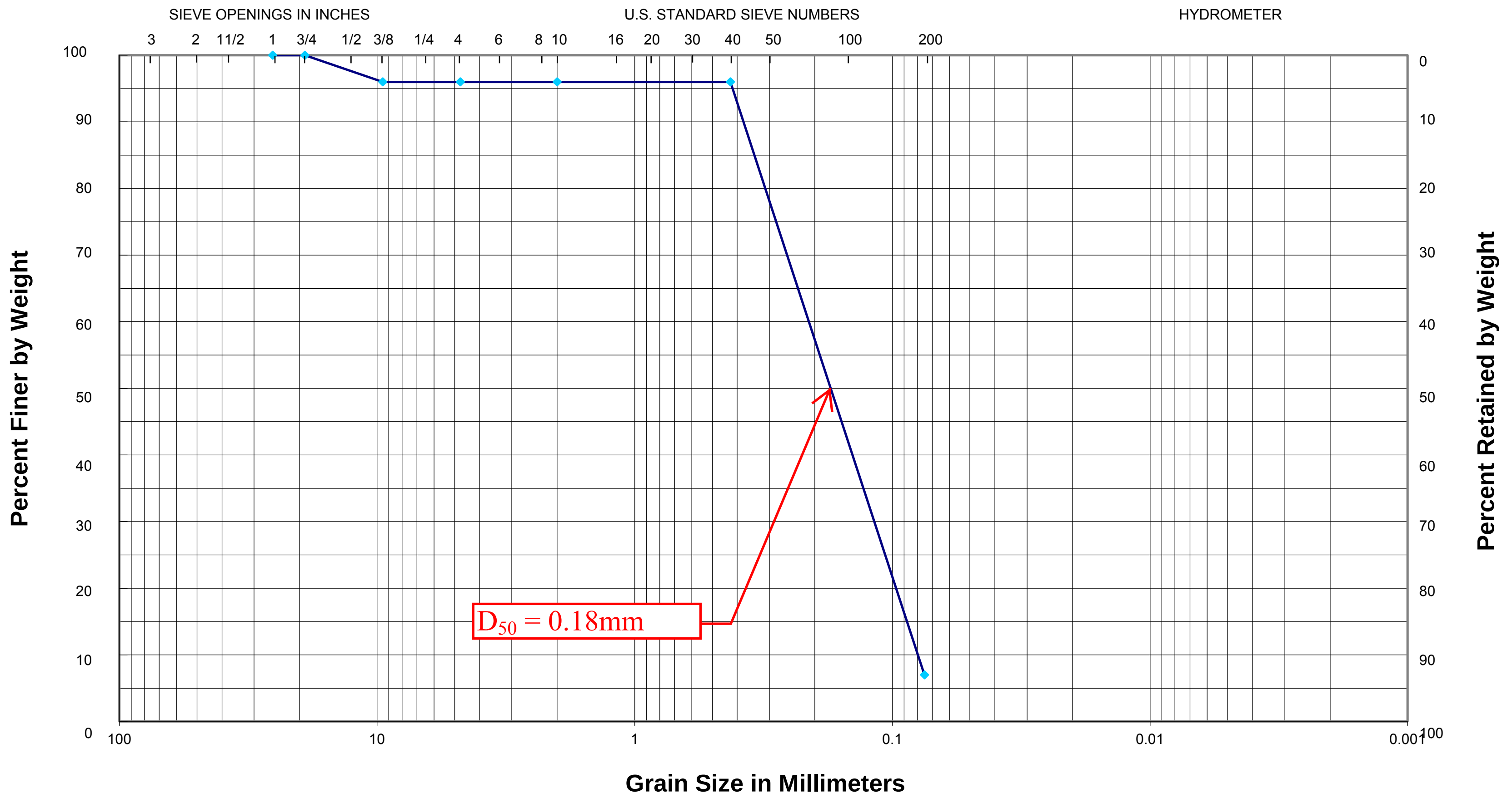
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 23, 24-25 ft
Atterberg Limits: Non-plastic

Description: Grayish brown fine sand
Classification: USCS = SP; AASHTO = A-3

13-017

GRAIN SIZE CURVE



D_{50} = 0.18mm

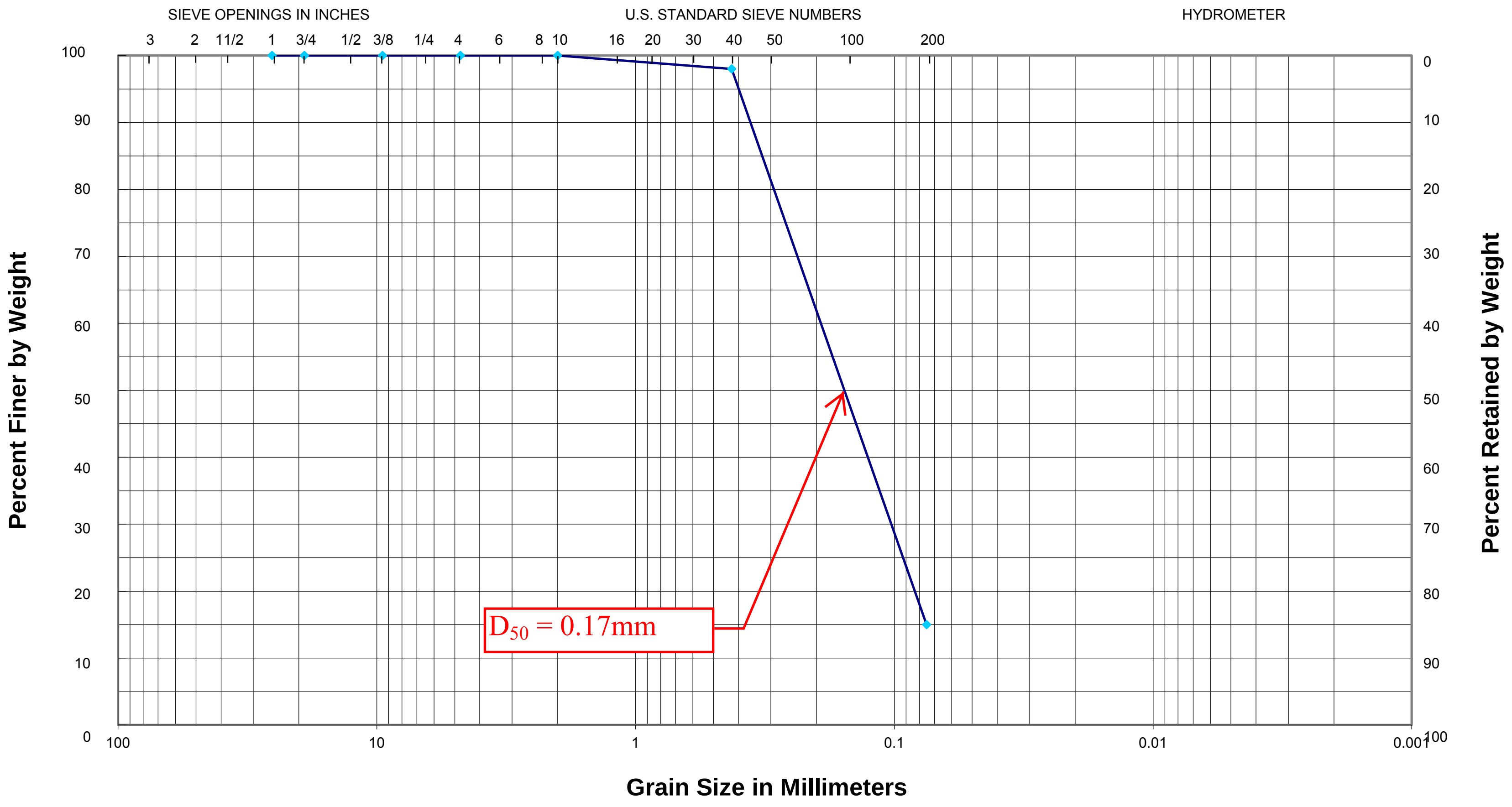
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 24, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan fine sand, slightly silty
Classification: USCS = SP-SM; AASHTO = A-3

13-017

GRAIN SIZE CURVE



D_{50} = 0.17mm

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring 25, 19-20 ft
Atterberg Limits: Non-plastic

Description: Brown and tan silty fine sand
Classification: USCS = SM; AASHTO = A-2-4