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GEOTECHNICAL INVESTIGATION
BRIDGE FOUNDATIONS and EMBANKMENTS
AHTD JOB CA0608
BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING)(S)
LITTLE ROCK, PULASKI COUNTY, ARKANSAS

INTRODUCTION

Submitted herein are foundation recommendations for bridge structures included in the I-630 widening project in Little Rock, Pulaski County, Arkansas. This project is designated as AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S). Results of stability analyses performed for the plan embankments at the Rock Creek Bridge location are also included in this submittal. Interim foundation recommendations for the bridges were provided on February 4 and February 7, 2015. Recommendations related to the roadway and retaining walls are provided in other report submittals under separate covers.

The AHTD Job CA0608 project consists of widening of the existing I-630 alignment between Sta 1062+91.02 (Log Mile 6.75) and Sta 1186+68.52 (Log Mile 4.41) in Little Rock, Pulaski County, Arkansas. The total project alignment is about 2.3 miles. The approximate bridge locations are indicated on the map provided in Attachment 1. Current layout drawings for the bridge structures, as provided by the Engineer on March 16, 2015, are also provided in Attachment 1.

The widening project includes four (4) widening/relocation/replacement bridges. These bridge structures are:

1. I-630 over Rock Creek Bridge (Bridges A and B5582). The Rock Creek Bridge widening project phase consists of the addition of one (1) lane to both the westbound and eastbound directional lanes. The widened bridge will have five (5) bents with a total bridge length of about 260 feet. Earthen embankments with simple

slopes will be utilized at the bridge abutments, with new embankment fill in the widened sections and incorporated into the existing embankments.

2. The replacement Pedestrian Bridge over Rock Creek. The replacement Pedestrian Bridge is planned on the north side (upstream) of the existing I-630 bridge over Rock Creek. The pedestrian bridge will be a 16-ft-wide and 254-ft-long structure, also with five (5) bents. Like the Rock Creek Bridge, the Pedestrian Bridge will also utilize earthen embankments at the bridge ends.

3. I-630 over Rodney Parham Road Replacement Bridge (Bridges A and B5583). The I-630 over Rodney Parham Road bridge project phase includes replacement of the existing four-span bridges. The replacement bridge will also be a four-span structure with five (5) bents and a total length of about 430 feet. The widened I-630 roadway in this location will include four (4), 12-ft-wide traffic lanes and 10-ft-wide shoulders in both the eastbound and westbound directions. New retaining walls are planned at the bridge ends. The channel of Briarwood Creek will be contained by a 6 ft by 5 ft, five (5) barrel reinforced concrete box culvert with a total length of 88 feet.

4. Hughes Street over I-630 Replacement Bridge (Bridge 05584). The existing Hughes Street Bridge over I-630 will be replaced with a new two-lane bridge. The replacement bridge will accommodate a 34-ft-wide roadway and a 6.5-ft-wide sidewalk on each side of the bridge. The new bridge will be a continuous composite W-Beam structure with two (2) spans and a total length of about 185 feet. I-630 at the Hughes location will be widened to include five, 12-ft-wide travel lanes and 10-ft-wide shoulders in each direction. Preliminary plans are to accommodate the widening by excavating the existing bridge end slopes and incorporating cut slopes or new retaining walls at the bridge ends. With the exception of cutting the current bridge end slopes to accommodate the widening, site grading at this location is expected to be minor.

Foundation loads of the widening / relocation / replacement bridges are expected to be moderate. Preliminary plans are to support the foundation loads at each of the bridge ends on steel piles. The current bridge layout drawings indicate drilled shafts will be utilized at the interior bents of the Rodney Parham Bridge. We understand that footings are planned at the interior bents of the Rock Creek and Hughes Street bridges.

The purposes of this study phase have been to explore subsurface conditions at the I-630 widening alignment and to develop recommendations to guide design and construction of foundations, embankments, and earthwork. These purposes have been achieved by a multi-phased study that included:

- Drilling sample and core borings to evaluate subsurface conditions and obtain samples for laboratory testing;

- Performing laboratory tests to evaluate pertinent engineering properties of the foundation and subgrade strata; and
- Analyzing field and laboratory data to develop recommendations for foundation design, embankment configurations, and construction considerations.

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

SUBSURFACE EXPLORATION

Subsurface conditions in the bridge alignments were evaluated by drilling sample and core borings at representative and accessible locations. The subsurface exploration program was developed based on the preliminary bridge layouts available at the time of field studies (June through September 2014 and December 2014). At that time, the bridge layout for the Rodney Parham Bridge was planned as a single-span structure. Consequently, relevant borings performed for retaining walls (Borings W3 through W6) have been utilized in conjunction with the bridge borings (Borings S1 through S4) to develop foundation recommendations for the current four-span bridge planned at Rodney Parham.

The subsurface exploration program is summarized by project facet in Tables 1 through Table 3.

Table 1: Summary of Subsurface Exploration Program – Rock Creek Bridge

Boring No.	Approx. Station, ft	Approx. Offset, ft	Approx. Surface El, ft	Completion Depth, ft
S7	1111+15	95 Lt	334	70
S8	1112+35	105 Rt	322	55
S9	1109+45	50 Rt	330	35
S10	1108+15	75 Lt	333	10
S10A	1105+18	78 Lt	333	50
S11	1110+70	35 Rt	308	2
S12	1109+55	45 Lt	306	3

Table 2: Summary of Subsurface Exploration Program – Pedestrian Bridge

Boring No.	Approx. Station, ft	Approx. Offset, ft	Approx. Surface El, ft	Completion Depth, ft
S13	109+05	5 Rt	323	60
S15	107+80	5' Rt	306	2
S16	107+60	5 Rt	305	2
S17	106+50	5 Rt	326	50

Table 3: Summary of Subsurface Exploration Program – Rodney Parham Bridge

Boring No.	Approx. Station, ft	Approx. Offset, ft	Approx. Surface El, ft	Completion Depth, ft
S1	1127+10	70 Rt	318	40
S2	1128+55	110 Rt	322	45
S3	1126+80	80 Lt	320	50
S4	1128+10	75 Lt	321	50
W3	1124+10	75 Rt	315	20
W4	1125+55	70 Rt	315	19
W5	1125+25	90 Lt	320	20
W6	1124+00	80 Lt	320	12

Table 4: Summary of Subsurface Exploration Program – Hughes Street Bridge

Boring No.	Approx. Station, ft	Approx. Offset, ft	Approx. Surface El, ft	Completion Depth, ft
S5	98+70	30 Rt	398	65
S6	101+50	35 Rt	408	80
S18	99+85	40 Rt	383	55

The results of the borings for the various bridge structures are provided in Attachment 2 for the Rock Creek bridges, Attachment 3 for the Rodney Parham Bridge, and Attachment 4 for the Hughes Street Bridge. The site vicinity of each bridge is shown on Plate 1 of each attachment. The approximate boring locations are shown on the Plan of Borings included as Plate 2 in each attachment. The subsurface conditions encountered in the borings, and the results of field and laboratory tests, are shown on the boring logs in each attachment. The approximate centerline station and offset of the boring locations are noted on the logs. In addition, the approximate ground surface elevation is shown on each log. It must be noted that the ground surface elevations shown <u>are approximate</u> and have been inferred from the available topographic and survey information provided by the Engineer. Keys to the terms and symbols used on the logs are provided in Attachment 5. Generalized subsurface profiles at the bridge locations are provided in Attachment 6.

The sample and core borings were drilled with a truck-mounted Mobile B-53 rotary-drilling rig using a combination of dry-auger and rotary wash procedures or continuous-flight auger drilling methods. Soil and weathered rock 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. Where rock hardness precluded recovery via the SPT, cuttings were obtained for use in visual classification.

Representative rock cores were obtained using a 5-ft-long, NQ_{WL}-size core barrel fitted with a diamond bit and using a wireline system. For each core run, the percent recovery was determined as the ratio of recovery to total length of core run. Rock Quality Designation (RQD) was also determined for each core run as the sum of sound rock core greater than 4-inch length divided by the total length of the run and expressed in percent. Both these values are presented in the right hand column of the log forms, opposite the corresponding core run.

Photographs of the rock cores recovered at the Rodney Parham Bridge location are provided in Attachment 7 (30 to 40 ft of Boring S1 and 35 to 45 ft of Boring S2). Rock core recovery at the Rock Creek Bridge location was limited by the poor rock conditions and the difficulty in advancing the boreholes through the rocky overburden soil (see Attachment 2, Boring S13, 20 to 30 ft). Likewise, coring at this bridge location was limited in the majority of the borings for the same reasons.

The Rock Creek channel was not accessible to drilling equipment. Consequently, rock conditions in the creek channel were evaluated by soundings using a hand probe and visual observations and mapping of the rock exposures in the channel bottom at plan bent locations (see Attachment 2, Borings S11, S12, S15, and S16).

All soil and rock samples were removed from sampling tools in the field, examined, and visually classified by the field geologist or geotechnical technician. Representative 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 soil and rock encountered in the borings, laboratory tests consisting of natural water content determinations,

classification tests, and rock compressive strength measurements 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)
- Unconfined compressive strength of rock cores (ASTM D-7012, Method A)

The test results are shown on the boring logs. Summaries of classification test results, along with classification by the Unified Soil Classification System and AASHTO classification system, are presented in Attachment 8. The rock compression test results are also summarized in Attachment 8.

GENERAL SITE AND SUBSURFACE CONDITIONS

Site Conditions

The CA0608 project includes four (4) bridge structures. The site conditions at these bridge locations are described in the following paragraphs.

Rock Creek Bridges. The existing I-630 bridges over Rock Creek (Bridge Nos. A5582 and B5582) are located between I-630 Sta 1109+15 and Sta 1111+74, approximately I-630 log mile 5.80, in Little Rock, Pulaski County, Arkansas. These bridges over Rock Creek are about 0.4 mile east of the John Barrow Road and I-630 interchange. The roadway over Rock Creek is a three-lane interstate roadway with a Portland cement concrete pavement section. The eastbound and westbound directional lanes are presently separated by concrete median barrier. The existing pedestrian bridge over Rock Creek is connected to the north side of the I-630 bridge outside lane.

The embankment on the west bridge end is a simple slope, with a roughly 3-horizontal to 1-vertical (3H:1V) configuration shown on the layout drawings. The end slope is armored with concrete riprap and dumped riprap protects the lower portions of the slope. Some riprap and boulder-sized concrete fragments are exposed in the bridge end embankment fill on each side of the existing bridges. The Bents 2 and 3 piers are armored at the bottoms with continuous concrete extending along the width of the bent.

The east bridge end embankment is a compound slope with a lower 2H:1V configuration, an upper 2.5H:1V configuration section and an approximately 10-ft-wide horizontal bench. The lower slope and horizontal bench are armored with concrete riprap. The Bent 4 piers are located in the horizontal bench. The upper slope and the embankment flanks are armored with dumped riprap.

The creek channel at this location is relatively broad. Normally, stream flow is relatively slow from north to south. However, flow becomes heavy during and following rain events. Shale is exposed in the channel. Some scouring is apparent around the existing concrete footing armor. There is some accumulation of sand and gravel downstream (south) of the bridge. Numerous utilities are in and around the creek channel at this location, with sanitary sewer and water extending under the bridge, oriented with the channel. We understand that these utilities are encased in concrete. An existing utility line also crosses the channel just downstream of the bridge location.

Rodney Parham Bridge. The I-630 bridges over Rodney Parham Road (Bridge Nos. A5583 and B5583) are located between I-630 Sta 1123+95 and Sta 1128+25, approximately log mile 5.48, in Little Rock, Pulaski County, Arkansas. The existing interstate over Rodney Parham Road is a three-lane roadway (each direction) with a Portland cement concrete pavement section. The eastbound and westbound directional lanes are presently separated by a concrete median barrier. The existing bridge is a four-span structure, with one (1) span over Rodney Parham Road and three (3) spans west of Rodney Parham Road. The space below the two (2) spans west of Rodney Parham Road is utilized for a recreation area.

The channel of Briarwood Creek flows north-south through the fourth span. Retaining walls transition grades on the north side of the interstate embankment, parallel to Rodney Parham Road. A concrete retaining wall with decreasing height also extends from the south side of the west bridge end wall. Most of the channel of Briarwood Creek is lined with concrete. The area on the south side of the interstate is primarily parkland of Little Rock's Kanis Park. The north side of the bridge and interstate alignment is primarily residential. With the exception of the interstate roadway embankment, the terrain is generally flat.

Hughes Street Bridge. The Hughes Street Bridge over I-630 (Bridge 05584) is located at approximately I-630 Sta 1154+25, log mile 5.0, in Little Rock, Pulaski County, Arkansas. The existing bridge is a two-span structure with simple slopes at each bridge end. The terrain on each end of the bridge is significantly higher than the grade of the existing interstate roadway, indicating that this I-630 roadway section was constructed in a cut section. Weathered shale is locally exposed at the bridge ends.

Site Geology

The Geologic Map of Arkansas[1] indicates that the CA0608 bridge alignments are located in the mapped exposure of the Pennsylvanian Period Jackfork Sandstone Formation. Shale is predominant in the Jackfork with a variable content of fine- to coarse-grained sandstone. The shale is typically argillaceous, though some carbonaceous shale units can be present. This shale is often tilted and folded with numerous fractures and jointing planes, which subsequently subjects the shale to some weathering along the bedding planes. The weathering can be differential and locally extends relatively deep. The subordinate sandstone units encountered in the Jackfork are locally discontinuous. Due to the folded and faulted nature of the formation, prominent cleavage and minor folding and faulting are found locally within the shale and sandstone units. The Jackfork rests conformably on the Stanley Shale and thickness varies from 3500 to 6000 feet.

At the Rodney Parham Bridge location, the Jackfork Sandstone is overlain by alluvial deposits of the Rock Creek and Briarwood Creek flood plains. The alluvial deposits are comprised of sand, silt, and gravel units with variable content and depth. The alluvial deposits overly the predominant shale and subordinate sandstone of the Jackfork Sandstone.

Seismic Conditions

Seismic Site Class. In light of the results of the borings performed for this study, the surface geology of the alignment locale, and our understanding of the project, a Seismic Site Class **B** (rock profile) is considered applicable to the Rock Creek Bridge, the Pedestrian Bridge, and the Rodney Parham Bridge locations. At the Hughes Street Bridge, a Seismic Site Class **C** (very dense soil and soft rock profile) is considered suitable. These seismic site classes have been determined 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]. For the abutment bents, the seismic site class was determined at the base of the earth approach embankments, as recommended in Section C3.4.2.2 of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design. For the interior bents, the ground surface was conservatively utilized in seismic site class evaluating.

Seismic Performance Zone / Seismic Design Category. Based on the bridge locations and

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1 Geologic Map of Arkansas, Arkansas Geologic Commission and U.S. Geologic Survey; 1993
2 AASHTO LRFD Bridge Design Specifications, AASHTO, 2012.
3 AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition; AASHTO; 2011.

utilizing the General Procedure (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 0.089 at the bridge locations. The mapped S_1 value is based on a 7 percent chance of exceedance in 75 years (i.e., a mean return period of approximately 1000 years).

At the Rock Creek Bridge, the Pedestrian Bridge, and the Rodney Parham Bridge locations, the site coefficient (F_v) for S_1 adjusted for Seismic Site Class B is 1.00. Accordingly, the calculated design 1.0-sec period spectral acceleration coefficient (S_{D1}) for Site Seismic Class B is 0.089 at all these bridge locations. Table 3.10.6-1 of the 2012 AASHTO LRFD Bridge Design Specifications indicates that a **Seismic Performance Zone (SPZ) 1** is fitting for the Rock Creek Bridge, the Pedestrian Bridge, and the Rodney Parham Bridge. Per Table 3.5-1 of the 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design, a **Seismic Design Category (SDC) A** is considered appropriate for these bridges.

At the Hughes Street Bridge location, a Seismic Site Class C has been determined. The site coefficient (F_v) for S_1 adjusted for Seismic Site Class C is 1.70. Consequently, A S_{D1} value of 0.15 is calculated for this bridge location in light of the Site Seismic Class C. As a result, a **Seismic Performance Zone (SPZ) 1** and a **Seismic Design Category (SDC) A** are also considered suitable for design of the Hughes Street Bridge.

Design Peak Ground Acceleration (A_S). 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 0.129 for all the bridges. For a Seismic Site Class B, the Site Coefficient for the PGA, F_{PGA} is determined to be 1.00. Consequently, a design PGA (A_s) value of 0.129 is considered appropriate for the Rock Creek Bridge, the Pedestrian Bridge, and the Rodney Parham Bridge locations. At the Hughes Street Bridge location where a Seismic Site Class C was determined, an F_{PGA} value of 1.20 and an A_s value of 0.16 are considered suitable.

Summary of Seismic Conditions. The seismic conditions and parameters developed and recommended for design of the bridges of the I-630 widening project are summarized in Table 5 below.

Table 5: Summary of Seismic Conditions and Parameters

Structure	Rock Creek Bridge and Pedestrian Bridge	Rodney Parham Bridge	Hughes Street Bridge
Seismic Site Class	B	B	C
Design 1.0-Second Spectral Acceleration, S_{D1}	0.09	0.09	0.15
Seismic Performance Zone	1	1	1
Seismic Design Category	A	A	A
Design Peak Ground Acceleration, A_S	0.13	0.13	0.16

Subsurface Conditions

With the bridges in different locations along the alignment, the subsurface conditions would be expected to vary somewhat. The generalized subsurface conditions at the various bridge locations are described in the following paragraphs.

Rock Creek Bridge and Pedestrian Bridge Locations. The existing bridge end embankments are constructed of fill. The embankment fill is predominantly stiff to very stiff tan, reddish tan, reddish brown, brown, gray, to dark gray silty clay with shale and sandstone fragments. Some cobble-sized (i.e., 3 in. to 12 in.) to boulder-sized (i.e., 12 in. or larger) shale, sandstone, syenite, and concrete fragments were also encountered in the existing embankment fill. The embankment fill to El 315± to El 309±, and an average of El 312±. The silty clay and shale/sandstone fragment embankment fill has low plasticity and exhibits overall fair to good compaction, moderate shear strength, and low compressibility. SPT N-values in the embankment fill range from 10 blows per ft to in excess of 50 blows per ft and average 38 blows per foot. It should be noted that these SPT N-values could be somewhat misrepresentative due to the presence of shale, sandstone and debris fragments, particularly the larger fragments. Fill content, depth, and compaction will likely vary throughout the bridge alignments.

The natural overburden soils below the embankment fill are typically comprised of medium dense to dense brown, gray, tan, and reddish brown sandy fine to coarse gravel and medium dense gray and tan clayey fine sand with sandstone fragments. The granular overburden soils are thought to represent alluvial soils deposited by the nearby stream. These predominantly soil units have an

average thickness of about 3 ft and extend to approximately El 311 to El 305 (average approximate El 309). This stratum exhibits medium relative density and low compressibility.

Low hardness to moderately hard tan, gray, to dark gray moderately weathered shale is below El 311± to El 305± (average El 309±) and extends to El 310± to El 304± (average El 306±). Moderately hard tan and gray weathered fine-grained sandstone seams, layers and strata are also interbedded in the predominant weathered shale stratum (see Boring S7). The sandstone partings and seams are typically medium-spaced (i.e., 8- to 24-in. spacing). The upper zones of the weathered shale can be highly weathered and have very poor rock quality and low hardness, though with moderate to high shear strength and low compressibility. With the exception of the upper highly weathered zones, the moderately hard moderately weathered shale is considered competent with SPT N-values typically in excess of 50 blows per foot. The weathered shale is steeply bedded with a dip observed to be on the order of 75° down to the north. Because of folding, the bedding orientation can vary widely.

The moderately weathered shale grades to moderately hard to hard slightly weathered to fresh dark gray shale below El 310± to El 304± (average El 306±). This stratum also contains some medium-spaced sandstone partings and seams and is also steeply bedded. The basal shale is strong and competent. The shale extends below the maximum 70-ft exploration depth (El 264±) of the borings.

Outside the creek channel, groundwater was locally encountered at 15-ft depth (El 307±) in August to September 2014 and December 2014 (see Boring S8). Groundwater was not encountered elsewhere prior to the introduction of drilling fluids. Groundwater levels will vary with seasonal precipitation, surface infiltration, and stream levels of Rock Creek. In addition, shallow perched water could be present at shallow depths in the more pervious on-site fill or in the natural granular.

Rodney Parham Bridge Location. On-site fill extends to 2 to 12.5 ft below existing grades, ranging from approximately El 317± to El 303± and an average El 312±). Locally the fill extends to the shale or sandstone bedrock. The on-site fill content is highly variable and includes stiff to very stiff brown clayey silt to brown, gray, tan, reddish tan, to reddish brown silty clay with a variable content of shale and/or sandstone fragments as well as fine to coarse gravel. The on-site fill also contains occasional cobble-sized (i.e., 3 in. to 12 in.) sandstone fragments. The predominant silty clay/clayey silt with shale/sandstone fragments has low plasticity. The fill exhibits overall fair to good compaction, moderate shear strength, and low compressibility. SPT

N-values in the on-site fill range from 10 blows per ft to in excess of 50 blows per ft and average 31 blows per foot. Fill content, depth, and compaction will likely vary in the replacement bridge alignment.

The natural overburden soils include stiff to very stiff reddish tan, tan and gray silty clay, fine sandy clay with variable amounts of shale and sandstone fragments and medium dense tan and gray sandy, clayey fine to coarse gravel and gravelly, clayey fine sand (see Borings S4 and W3). The silty clay, fine sandy clay and gravelly, clayey sand units exhibit low plasticity, moderate shear strength / medium relative density and low compressibility.

In the bridge alignment, the fill and overburden soils are locally underlain by moderately hard to hard tan, reddish tan, gray, to brown weathered fine-grained sandstone at variable depths of 5 to 8 ft (i.e., below El 314± to El 311± and an average El 313±) (see Borings S1, S2, and S4). Though weakly cemented and exhibiting poor to fair rock quality, the weathered sandstone stratum is competent and relatively strong. SPT N-values in the weathered sandstone typically exceed 50 blows per foot. The thickness of the weathered sandstone units range from about 5 to 7 ft and average approximately 6 feet.

Low hardness to moderately hard tan, gray, to dark gray highly weathered to moderately weathered shale is below the overburden soils and/or the weathered sandstone and extends to El 304± to El 293± (average El 299±). The weathered shale is steeply bedded and exhibits poor rock quality but has high shear strength and low compressibility. SPT N-values in the weathered shale typically exceed 50 blows per foot. Competence of the highly weathered to weathered shale generally increases with depth.

The moderately weathered shale grades to moderately hard to hard dark gray slightly weathered to fresh shale below El 304± to El 293± (average El 299±). The basal dark gray shale is also steeply bedded and contains close to very close sandstone seams and layers. The basal shale is strong and competent. Laboratory uniaxial compression tests on rock cores indicate the compressive shear strength of the dark gray shale ranges from 740 to 1260 lbs per sq in. with an average compressive strength of 990 lbs per sq inch. Rock quality designation (RQD) ranges from 75 to 88 percent and averages 81 percent, indicating good rock quality.

Groundwater was locally encountered at 7.5- to 9-ft depth (El 308± to El 306±) in June to September 2014 (see Borings W3 and W4). Groundwater was not encountered elsewhere prior to the introduction of drilling fluids. It is our opinion the water locally encountered in these borings is

perched water in the more pervious on-site fill and natural granular overburden soil. Groundwater levels will vary with seasonal precipitation, surface infiltration and runoff, and stream levels of the nearby Briarwood Creek and other surface water features.

Hughes Street Bridge Location. Based on the results of the borings drilled for the Hughes Street Bridge, the surface soil stratum is typically variable on-site fill. The on-site fill includes about 12 in. of medium dense brown fine sandy silt with organics (see Boring S5). The on-site fill is predominantly comprised of stiff to very stiff tan and reddish tan silty clay with shale and sandstone fragments and some sandstone cobbles (i.e., 3 in. to 12 in.) extending to 3- to 4-ft depth. The silty clay and shale/sandstone fragment fill is relatively compact with moderate shear strength and low compressibility. On-site fill was not encountered at the north abutment (Boring S6). Fill content, depth, and compaction will likely vary with location along the alignment.

A localized stratum of natural overburden very stiff reddish tan and tan silty clay with sandstone fragments locally extends to about 4.5-ft depth (see Boring S5). The thin stratum of natural overburden soil exhibits moderate shear strength and low compressibility.

The on-site fill and natural silty clay overburden soils are underlain by low hardness to moderately hard gray, tan, reddish tan, and/or maroon highly weathered shale. The highly weathered shale extends to variable depths of 4 to 33 ft below existing grades (approximately El 404 to El 350). The variations in the depth of the highly weathered shale are apparently associated with the bridge location in a cut area and the site grading of the initial interstate construction. The highly weathered shale is steeply dipping and contains some silty clay laminations and seams as well as discontinuous sandstone partings and seams. This stratum exhibits very poor rock quality but moderate to high shear strength and low compressibility.

The highly weathered shale grades to low hardness to moderately hard gray, tan, reddish tan, maroon, to dark gray weathered shale below 4- to 33-ft depth (approximately El 404 to El 350). The lower weathered shale units are also steeply bedded with a dip on the order of 70° to 80°, generally down to the northeast. However, the dip is likely to vary locally due to localized folds and faulting. The lower weathered shale units contain some sandstone partings and have ferrous stains in the bedding planes. Localized sandstone beds are also interbedded in the predominant weathered shale. The lower weathered shale exhibits poor rock quality, is competent, and has high shear strength and low compressibility. Competence and rock quality generally improve with depth.

The basal stratum encountered in the borings is moderately hard to hard dark gray shale found at 37 to 68 ft below existing grades (approximately El 346 to El 340). The basal shale units are steeply bedded, slightly weathered to fresh and contain discontinuous interbedded sandstone seams. The basal shale has poor rock quality but is strong and competent.

Shallow groundwater was not encountered in the Hughes Street borings drilled over the period of June to July 2014. Drilling fluids were typically introduced into the boreholes at 10-ft depth, so groundwater conditions at depth could not be determined. Though not encountered in the borings, shallow perched groundwater could be present. In addition, seasonal seeps could develop as infiltrated surface water migrates to the exposed cut slopes at the bridge ends and in the shallow, steeply-bedded weathered shale in the interstate roadway grade. Groundwater conditions will vary with seasonal precipitation and surface runoff and infiltration.

Generalized Subsurface Profiles. To aid in visualizing the subsurface conditions in the bridge alignments, Generalized Subsurface Profiles are included in Attachment 6. It should be recognized that the stratigraphy illustrated by the profiles 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.

ANALYSES and RECOMMENDATIONS

General Foundation Design Considerations

Foundations for the CA0608 bridges must satisfy two (2) basic and independent design criteria: a) foundations must have an acceptable factor of safety against bearing failure under maximum design loads, and b) foundation movement due to consolidation or swelling of the underlying strata should not exceed tolerable limits for the structure. Construction factors, such as installation of foundations, excavation procedures and surface and groundwater conditions, must also be considered.

In light of the results of the borings, the anticipated moderate bridge foundation loads, and our understanding of the project, driven steel piles are recommended for the abutment bents at each of the bridge locations. For the interior bents, footings founded in competent weathered shale or weathered sandstone are suitable. For the Rodney Parham Bridge, drilled shafts

extending to the competent shale and/or sandstone are recommended for the interior bent foundations. Foundation recommendations are discussed in the following report sections.

Steel Piles – Abutment Bents

Axial Pile Capacities. Steel piles are recommended for support of the foundation loads at the bridge ends. HP12x53 and HP14x73 steel piles are considered suitable sections. Other pile sizes or types may be evaluated if desired. For the Rock Creek and Rodney Parham bridges, it is expected that point-bearing steel piles will be driven to refusal in the competent moderately hard to hard weathered shale/weathered sandstone or shale. It is expected that at the Hughes Street over I-630 bridge steel piles will be driven to capacity in the weathered shale. Piles should extend through the embankment fill, the natural overburden soils, and zones of low hardness highly weathered shale into the competent moderately hard to hard weathered shale/weathered sandstone or shale. We recommend that all the steel piles be fitted with rock points.

Bearing capacities of piles driven to refusal should be determined using the LRFD structural design procedure[4]. We recommend that nominal (ultimate) resistance (P_n) of steel piles be determined based on the yield strength of steel piles (f_y) and the net end area (A_{net}) of the section. An effective resistance factor (φ_c) of 0.50 is recommended for structural determination of factored bearing capacities in accordance with AASHTO LRFD Bridge Design Specifications regarding design of steel structures. This effective resistance factor for steel piles has been based on the assumption of severe driving conditions.

It has been our experience that allowable compression pile capacities of 70 tons and 96 tons are common for 36 kip per sq inch, HP12x53 and HP14x73 piles, respectively. The 70-ton and 90-ton capacities are based on allowable stress design (ASD). However, the appropriate factored bearing capacity should be confirmed by the Engineer and the Department. Post-construction settlement of piles driven to refusal will be negligible. Given the plan to incorporate the existing embankments into the new embankments, the age of the existing embankments, and the height and content anticipated for the future fill, downdrag loads due to long-term embankment settlement are considered negligible.

We recommend a minimum pile penetration of 10 ft below the pile cap bottom. We also recommend that piles be driven to an elevation at or below the adjacent lowest grade. Piling

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4 AASHTO LRFD Bridge Design Specifications, AASHTO, 2012.

adjacent to creek channels or other surface water features potentially susceptible to scour, piles should be driven below the anticipated scour depth. Estimated pile tip elevations for the CA0608 bridges are summarized in Tables 6 to 9 below.

Table 6: Estimated Tip Elevations of Steel Piles Driven to Refusal – Rock Creek

Bent No.	Estimated Pile Tip El, ft	Comments
1 (West Abutment)	308 to 306	Estimated 17 to 19 ft below plan pile cap bottom (El 325)
5 (East Abutment)	304	Estimated 22 ft below plan pile cap bottom (El 326)

Table 7: Estimated Tip Elevations of Steel H Piles Driven to Refusal – Pedestrian Bridge

Bent No.	Estimated Pile Tip El, ft	Comments
1 (West Abutment)	306	Estimated 21 ft below plan pile cap bottom (El 327)
5 (East Abutment)	307 to 306	Estimated 20 to 21 ft below plan pile cap bottom (El 327)

Table 8: Estimated Tip Elevations of Steel Piles Driven to Refusal – Rodney Parham

Bent No.	Estimated Pile Tip El, ft	Comments
1 (West Abutment)	308 to 304	Estimated 23 to 27 ft below plan pile cap bottom (El 331)
5 (East Abutment)	314 to 313	Estimated 20 to 21 ft below plan pile cap bottom (El 334)

Table 9: Estimated Tip Elevations of Steel Piles Driven to Refusal – Hughes Street

Bent No.	Estimated Pile Tip El, ft	Comments
1 (South Abutment)	375	Estimated 21 ft below plan pile cap bottom (El 396)
3 (North Abutment)	375	Estimated 27 ft below plan pile cap bottom (El 402)

It should be noted that tip elevations shown in the above table are estimates only based on the results of the borings and the inferred surface elevations at particular boring locations. As-built pile tip elevations may vary. Pile capacity and final depth must be field verified.

The results of the borings indicate as-built pile lengths will range from 17 ft to 27 ft, more or less. Longer piles may be warranted by uplift and/or lateral resistance considerations. Pre-boring will be required for some piles to develop the needed penetration into the competent weathered shale/weathered sandstone or shale.

Piling Construction. Piles should be installed in compliance with AHTD Standard Specifications Section 805. Specific driveability analyses will be performed to evaluate suitable

driving equipment. To develop an estimate or required hammer size, drivability analyses were performed for steel HP12x53 and HP14×73 piles at representative bents. Wave equation analyses (WEAP) methods were used with the computer program GRLWEAP 2010[5]. The results of the driveability analysis are summarized in Table 10. The drivability analyses results are provided graphically in Attachments 9, 10, and 11 for the Rock Creek Bridge/Pedestrian Bridge, Rodney Parham Bridge, and Hughes Street Bridge, respectively.

Table 10: Results of Driveability Analyses

Structure	Bent	Pile Size	Pile Penetration (Pile Tip El), ft	Hammer Energy, ft-kips	Max Blow Count Prior to Refusal, Blows/ft	Max Comp Stress Prior to Refusal, ksi
Rock Creek	5 (East Abutment)	HP12×53	22 (El 304)	20.1	62	21.0
		HP14×73	22 (El 304)	20.1	94	20.0
Rodney Parham	1 (West Abutment)	HP12×53	27 (El 304)	20.1	70	21.3
		HP14×73	27 (El 304)	20.1	104	20.2
Hughes Street	3 (North Abutment)	HP12×53	27 (El 375)	22.6	154	23.3
		HP14×73	27 (El 375)	27.1	166	23.7

Based on the results of WEAP analysis, we recommend a minimum hammer energy of 20 ft-kips per blow for the HP12x53 piles and HP14×73 piles at the Rock Creek Bridge, Pedestrian Bridge, and Rodney Parham Bridge locations. For the steel piles at the Hughes Street Bridge location, a minimum hammer energy of 22.6 ft-kips per blow for HP12×53 piles and a minimum hammer energy of 27.1 ft-kips per blow for HP14×73 piles are recommended.

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 H 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 36 ksi yield strength of the steel H piles (i.e., 32.4 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 prior to hammer acceptance and start of driving. We have recommended that all piles be fitted with rock points.

As a minimum, safe bearing capacity of production piles should be determined by AHTD Standard Specifications Section 805.09, Method A. Blow counts on steel piles should be limited to

[5] GRLWEAP 2010; Pile Dynamics, Inc.

about 20 blows per inch. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows. Driving records should be available for review by the Engineer during pile installation.

In light of the presence of cobble- to boulder-sized shale, sandstone and concrete fragments in the embankment fill and, possibly, in the natural overburden sand and gravel, pre-boring could be required for installation of some piles. A boulder breaker or similar tools or pre-excavation could be required to advance piles at locations where large debris or rock fragments are buried in the on-site fill. The void space remaining around piles after pre-boring should be backfilled with grout or other approved material.

Drilled Shafts – Rodney Parham Bridge

The foundation loads at the interior bents of the Rodney Parham Bridge may be supported on drilled shafts. Consideration could also be given to using drilled shafts at bridge ends. Drilled shafts should be founded at least one-and-one-half (1) shaft diameters or a minimum of 6 ft, whichever is greater, into the moderately hard to hard dark gray slightly weathered to fresh shale. For drilled shafts founded as recommended, a maximum nominal/ultimate bearing capacity of 175 kips per sq ft is recommended. A resistance factor (φ_{stat}) of 0.50 is recommended for drilled shaft end bearing. We recommend that drilled shafts be sized for compression loads based on the factored unit end bearing resistance only. Total and differential settlement of properly installed drilled shafts founded in the competent slightly weathered to fresh shale should be less than 0.5 inch.

Uplift loads will be resisted by circumferential shaft friction. We recommend that shaft penetration through the variable overburden soils be neglected. For that portion of shaft penetration extending through the moderately hard weathered shale or weathered sandstone a maximum nominal/ultimate skin resistance value of 4400 lbs per sq ft is recommended. For skin resistance in the moderately hard to hard dark gray slightly weathered to fresh shale bearing stratum a maximum nominal/ultimate skin resistance value of 7500 lbs sq ft is recommended. A resistance factor (φ_{stat}) of 0.40 is recommended for uplift capacity, a resistance factor (φ_{up}) of 0.40 is recommended.

In light of the results of the borings, drilled shaft excavations are expected to extend to minimum depths of 18 to 34 ft below existing grades. Minimum shaft bottom elevations are estimated to range from about El 299 to El 288 and average about El 295. As-built drilled shaft

lengths will vary with the required penetration into the bearing stratum and specific subsurface conditions. Depending on specific subsurface conditions and rock quality, localized deepening or shortening of shaft depths will be warranted. All drilled shaft excavations should be observed by the Engineer to verify suitable bearing and adequate penetration.

A minimum shaft diameter of 36 in. is recommended for drilled shafts. A minimum shaft length of three (3) shaft diameters is also recommended. Heavy-duty drilling equipment will be required for drilled shaft excavation. Rock drilling methods could be required to advance shaft excavations through hard sandstone units and more resistant shale zones. Minor seepage into shaft excavations can often be controlled by expedient shaft construction. However, we recommend that temporary casing be on site in the event it is required to control caving or seepage inflow into shaft excavations.

Footings – Interior Bridge Bents

Foundation loads at the interior bents of the CA0608 bridges may be supported on footings bearing in the competent moderately hard tan, gray, brown to dark gray weathered shale, the moderately hard to hard tan, reddish tan, gray, to brown weathered fine-grained sandstone, or moderately hard to hard dark gray shale. Footings should be founded with a minimum embedment of 3 ft into the competent rock bearing stratum, i.e., the weathered shale, weathered sandstone, and/or dark gray shale. At the creek locations, footings bottoms should be located to bear below the maximum anticipated scour depth. Geotechnical input parameters recommended for use in scour analyses for the Rock Creek Bridge are provided in Attachment 12.

Footings founded in the competent moderately hard weathered shale, the moderately hard to hard fine-grained sandstone, or moderately hard to hard shale as recommended may be sized based on a maximum nominal/ultimate bearing pressure (q_n) of 20 kips per sq foot. A bearing resistance factor (φ_b) of 0.45 is recommended for footings bearing in rock. Accordingly, a factored unit bearing resistance (q_R) of 9000 lbs per sq ft is considered appropriate. Post-construction settlement of foundations supported in the competent weathered shale/weathered sandstone or shale is expected to be less than 0.5 inch.

Uplift resistance of footings will be developed by the weight of the structure and the foundation units. Resistance to sliding may be determined using a nominal/ultimate friction value ($\tan\delta$) of 0.60 for concrete on the competent bearing rock. A resistance factor for sliding (φ_τ) of 0.85 is recommended for evaluation of sliding resistance from friction. At the Rock Creek Bridge,

Pedestrian Bridge, and Rodney Parham Bridge locations, a nominal/ultimate passive resistance value (f_L) of 1250 lbs per sq ft may be assumed for the competent weathered shale/weathered sandstone or shale bearing stratum below the maximum anticipated scour depth. A reduced nominal/ultimate passive resistance value (f_L) of 1000 lbs per sq ft is recommended for the competent weathered shale bearing stratum at the Hughes Street Bridge location. The embedment in the overburden soils or the top 2 ft embedment, whichever is deeper, should be neglected from passive resistance evaluation. A resistance factor (φ_{ep}) of 0.50 is recommended for evaluation passive resistance.

New footings must extend through the on-site fill, the overburden soils and any highly weathered zone to bear fully in the competent moderately hard to hard weathered shale/weathered sandstone or fresh shale. A minimum embedment of 3 ft into the competent weathered shale/weathered sandstone has been recommended. Based on the results of Boring S18, a minimum footing bottom at El 376 has been estimated for the Hughes Street Bridge. Estimated footing bottom elevations for the other I-630 bridges are summarized in Tables 11 to 13.

Table 11: Estimated Footing Bottom Elevations – Rock Creek Bridge

Bent No.	Plan Footing Bottom El on Preliminary Bridge Layouts, ft	Estimated Footing Bottom El, ft	Comments
2	304.5	301	Min 3 ft into competent weathered shale (rockline at El 304±)
3	303	303	Competent weathered shale line estimated at El 308±
4	305	305	Competent weathered shale / weathered sandstone line estimated at El 308±

Table 12: Estimated Footing Bottom Elevations – Rock Creek Pedestrian Bridge

Bent No.	Plan Footing Bottom El on Preliminary Bridge Layouts, ft	Estimated Footing Bottom El, ft	Comments
2	303	302	Competent weathered shale line estimated at El 305±

Bent No.	Plan Footing Bottom El on Preliminary Bridge Layouts, ft	Estimated Footing Bottom El, ft	Comments
3	301	301	Competent weathered shale line estimated at El 306±
4	306	303	Min 3 ft into competent weathered shale (rockline at El 306±)

Table 13: Estimated Footing Bottom Elevations – Rodney Parham Bridge

Bent No.	Plan Footing Bottom El on Preliminary Bridge Layouts, ft	Estimated Footing Bottom El, ft	Comments
2	299	304 to 300	Approximately 15 to 16 ft below existing grades
3	302	300	Approximately 15 ft below existing grades
4	305	307 to 305	Approximately 8 to 13 ft below existing grades

It must be noted that the estimated footing bottom elevations shown in preceding tables are approximations only which have been based on the results of the borings and the inferred surface elevations at particular locations. In addition, these elevations do not take into account the effect of potential scour. As noted, the footing bottom must be fully embedded below the maximum anticipated scour depth. Final footing bottom elevations and suitable bearing must be field verified.

All footing excavations should be observed by the Engineer or Department to verify suitable bearing. Highly-weathered zones, silty clay seams and layers, or otherwise unsuitable material should be removed from footing excavations prior to concrete placement. Any overexcavation of footings must be backfilled with concrete. The use of dental concrete is acceptable where weathered zones or silty clay layers are excavated.

Care must be exercised not to undermine the existing bridge foundations, underground utilities, or other existing features with deep excavations for new footings. In addition, footings should be located as far as possible from existing footings to avoid stress overlap between the

existing and new bridge foundations. New footings planned at locations adjacent to the existing foundations should be evaluated for adverse effects of stress overlap.

Approach Embankments – Rock Creek Bridge and Pedestrian Bridge

General. Stability of the approach embankments at Rock Creek Bridge and Pedestrian Bridge end locations were analyzed. Stability analyses related to retaining walls have been deferred until a later study phase and will be submitted under separate cover.

We understand that the existing embankments of the Rock Creek Bridge and Pedestrian Bridge will be utilized to the extent possible with some fill placed on the widened roadway sections of the embankments. Stability analyses of embankment side slopes have been performed utilizing the currently-available cross sections. Relevant cross sections at the Rock Creek Bridge and Pedestrian Bridge locations are included in Attachment 13.

Composite slope configurations with a primary 3-horizontal to 1-vertical (3H:1V) slope will typically be utilized for the side slopes. A roughly 3-horizontal to 1-vertical (3H:1V) slope configuration has been assumed for stability analysis of the west end slopes. Stability analysis of the east end slope at the Rock Creek Bridge location assume a composite slope configuration which is comprised of a lower 2H:1V slope and an upper to 2.25H:1V slope with a 10-ft-wide horizontal bench. The composite east end slope at the Pedestrian Bridge will be comprised of a lower 2.5H:1V slope and an upper retaining wall (Wall BB) with a 2H:1V backslope behind the wall.

The purposes of the stability analyses are to verify overall stability of the design embankment configuration with respect to shear strength of embankment fill, retaining wall, and foundation soils. This submittal provides the results of stability analyses performed on the overall embankments only. Localized stability of the Wall BB will be analyzed and the results of analyses performed on the retaining walls will be submitted under separate report cover. 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. The *in-situ* soil properties have been modeled for use in stability analyses based on the results of laboratory tests and our experience with similar soils. The retaining wall is assumed to be rigid enough to drive shear failure below the wall bottom. For the purposes of stability analyses, a uniform surcharge of 275 lbs per sq ft has been included to accommodate vehicle traffic loads.

Stability analyses have been performed using the computer program SLOPE/W 2007[6] and a Morgenstern-Price analysis. The loading conditions evaluated for the approach embankments include the following.

- End of construction with total stresses.
- Long term with effective stresses and groundwater at the El 307±.
- Long term with effective stresses and the embankment saturated to a water level approximately equal to the design 50-year flood at El 321.5.
- Seismic condition with effective stresses and groundwater at El 307±. A horizontal acceleration coefficient (k_h) value of 0.13, which is the peak ground acceleration value, was utilized.
- Rapid drawdown with effective stresses and a saturated embankment and drawdown from the design 50-year flood of El 321.5 to the embankment toe elevation.

Stability analyses have been performed to verify the suitability of the plan approach embankment sections. Results of the stability analyses performed for the end slopes, the side slope at the west abutment, and the side slope at the east abutment of the Rock Creek Bridge are provided in Attachment 14. The results of stability analyses performed for the Pedestrian Bridge embankments are provided in Attachment 15 for the end slopes, west side slope, and east side slope. 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 attachments containing the results of stability analyses.

Results of Stability Analyses. The results of stability analyses are summarized in Tables 14 through 17 for embankment slopes of the I-630 over the Rock Creek Bridge location.

Table 14: Stability Analysis Results – End Slope at West Abutment (I-630)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	3.8
Long Term	Groundwater @ El 307±	1.6
	Design flood @ El 321.5	1.5
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.3
Rapid Drawdown	Drawdown from design flood to embankment toe	1.4

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

Table 15: Stability Analysis Results – End Slope at East Abutment (I-630)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.1
	Design flood @ El 321.5	2.0
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.5
Rapid Drawdown	Drawdown from design flood to embankment toe	1.9

Table 16: Stability Analysis Results – Side Slope at West Abutment (I-630)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	2.6
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.7
Rapid Drawdown	Drawdown from design flood to embankment toe	2.1

Table 17: Stability Analysis Results – Side Slope at East Abutment (I-630)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.4
	Design flood @ El 321.5	2.3
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.6
Rapid Drawdown	Drawdown from design flood to embankment toe	2.1

The results of stability analyses for the embankment slopes at the Pedestrian Bridge over the Rock Creek location are summarized below in Tables 18 through 21.

Table 18: Stability Analysis Results – End Slope at West Abutment (Pedestrian Bridge)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	4.2
Long Term	Groundwater @ El 307±	1.9
	Design flood @ El 321.5	1.7
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.4
Rapid Drawdown	Drawdown from design flood to embankment toe	1.5

Table 19: Stability Analysis Results – End Slope at East Abutment (Pedestrian Bridge)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	3.4
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	1.9
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.8
Rapid Drawdown	Drawdown from design flood to embankment toe	1.9

Table 20: Stability Analysis Results – Side Slope at West Abutment (Pedestrian Bridge)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	6.2
Long Term	Groundwater @ El 307±	2.9
	Design flood @ El 321.5	2.6
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.8
Rapid Drawdown	Drawdown from design flood to embankment toe	2.6

Table 21: Stability Analysis Results – Side Slope at East Abutment (Pedestrian Bridge)

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.2
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	2.4
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.6
Rapid Drawdown	Drawdown from design flood to embankment toe	2.2

The results of slope stability analyses summarized in Tables 14 through 21 indicate acceptable factors of safety against sliding for the end and side slopes with the plan configuration for all loading conditions analyzed and for both end and side slopes evaluated at the bridge ends. Consequently, the design slope configuration for the Rock Creek Bridge is considered to be adequate and suitable.

Embankment Construction Considerations. Unclassified fill is suitable for the new embankment sections. We recommend that the top 24 in. of embankment fill in slopes have a maximum liquid limit of 40 and a plasticity index (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 bonding of horizontal fill lifts. Maximum bench height should be limited to 3 feet. A typical bench width of 8 to 12 ft is recommended. Detailed benching pattern during construction must be based on specific site and construction conditions. The results of the borings indicate presence of some cobble-sized (i.e., 3 in. to 12 in.) to boulder-sized (i.e., 12 in. or larger) shale, sandstone, syenite, and concrete fragments. Where exposed during site grading, these debris and fragments should be removed and properly backfill with suitable embankment fill.

Site Grading and Earthwork

We understand that the bridges over Rock Creek will incorporate the existing embankments to the extent possible. Some site grading/reshaping of the existing embankments elsewhere is also likely to be required. After any required 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, undercut potential is expected to be low. However, 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 all work is completed.

Groundwater was locally encountered at 15-ft depth (El 307±) at the Rock Creek Bridge locations and at 7.5- to 9-ft depth (El 308± to El 306±) at the Rodney Parham Bridge location (June through September 2014). In addition, shallow perched water was locally encountered in the on-site fill and in granular soils at shallower depths. Groundwater levels will vary with seasonal precipitation, surface infiltration, and stream level of nearby creeks and waterways.

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 fully encapsulated by a filter fabric complying with AHTD Standard Specifications Subsection 625.02, Type 2 and vented to positive discharge. Where surface seeps or springs are encountered during site grading, we recommend the seepage be directed via French drains or blanket drains to positive discharge at daylight or to storm drainage lines.

Rock Excavation

Rock excavation methods could be required for some site grading cuts. Some rock excavation could also be required for hard sandstone and more resistant weathered shale encountered in footing excavations and piling pre-bore excavations. Some overbreak of excavations advanced into the sandstone and weathered shale should be anticipated. Any overbreak or overexcavation of footings must be backfilled with concrete.

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. Any splices in steel piles should develop the full cross-sectional capacity of un-spliced

piles. Pre-boring will be required for pile installation. We have recommended that all steel piles be fitted with rock points. Blow counts on steel piles should be limited to about 20 blows per inch. Practical pile refusal may be defined as a penetration of 0.5 in. or less for the final 10 blows.

As a minimum, safe bearing capacity of production piles should be determined by AHTD Standard Specifications Section 805.09, Method A. Driving records should be available for review by the Engineer during pile installation.

Drilled Shafts

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

All drilled shaft excavations should be observed by the Engineer or Department to verify suitable bearing and adequate penetration. Drilled shafts will be advanced through the overburden soils and weathered shale and weathered sandstone to the moderately hard to hard shale bearing stratum. The more resistant shale and sandstone units may require the use of rock coring tools in order to obtain the required penetration. The potential for hard rock drilling should be anticipated.

CLOSURE

The Engineer or a designated representative thereof should monitor site preparation, grading work and 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 report.

Attachment 1	Layout Drawings – All Bridges
Attachment 2	Boring Logs – Rock Creek
Attachment 3	Boring Logs – Rodney Parham Road
Attachment 4	Boring Logs – Hughes Street
Attachment 5	Keys to Terms and Symbols on Boring Logs
Attachment 6	Generalized Subsurface Profiles – All Bridges
Attachment 7	Photographs of Rock Cores – Rodney Parham Road
Attachment 8	Laboratory Test Results
Attachment 9	Steel Pile Drivability Analysis – Rock Creek
Attachment 10	Steel Pile Drivability Analysis – Rodney Parham Road
Attachment 11	Steel Pile Drivability Analysis – Hughes Street
Attachment 12	Geotechnical Parameters for Scour Analysis – Rock Creek
Attachment 13	Relevant Cross Sections – Rock Creek
Attachment 14	Results of Stability Analysis –Rock Creek Bridge
Attachment 15	Results of Stability Analysis – Rock Creek Pedestrian Bridge

* * * * *

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, please call on us.

Sincerely,

GRUBBS, HOSKYN,
BARTON &WYATT, INC.

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

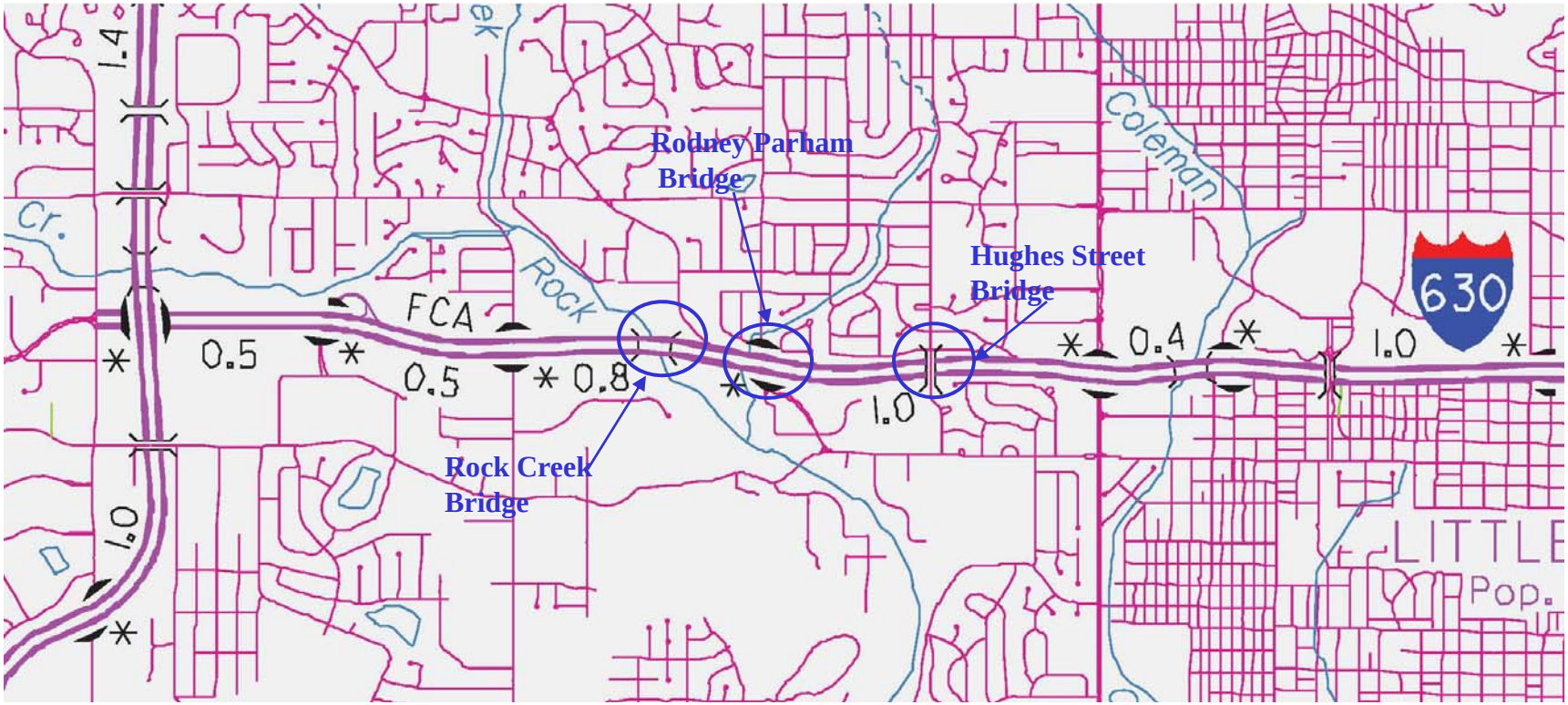
Mark E. Wyatt, P.E.
President

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

YZ/MEW:jw

Copies Submitted: Bridgefarmer & Associates, Inc.
Attn: Mr. Shahriar Azad, P.E. (2+email)
Attn: Mr. Stephen Smiley, P.E. (1-email)

ATTACHMENT 1



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

Bridge Locations
CA0608 – I-630
Little Rock, Pulaski County, Arkansas

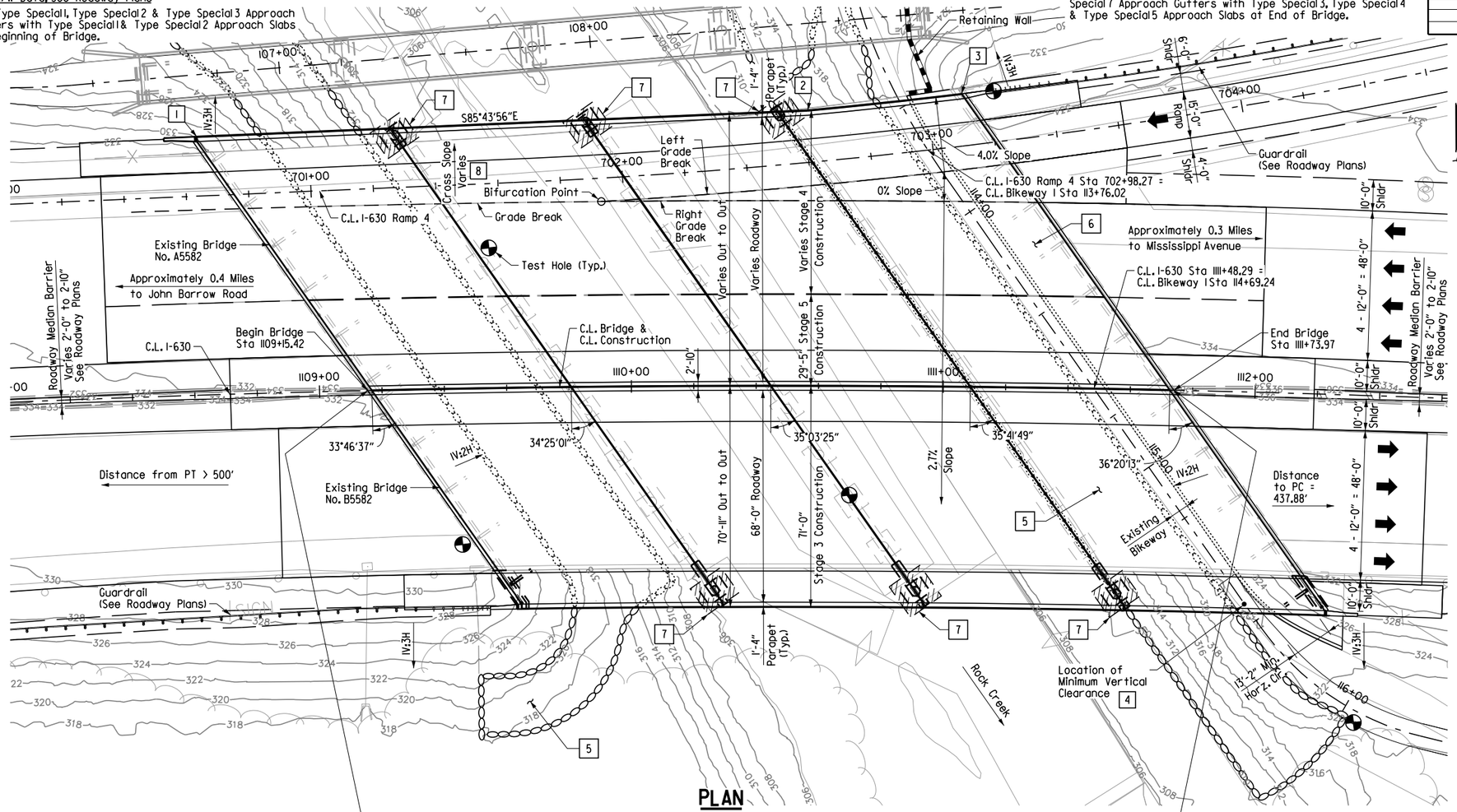
Job No. 14-030

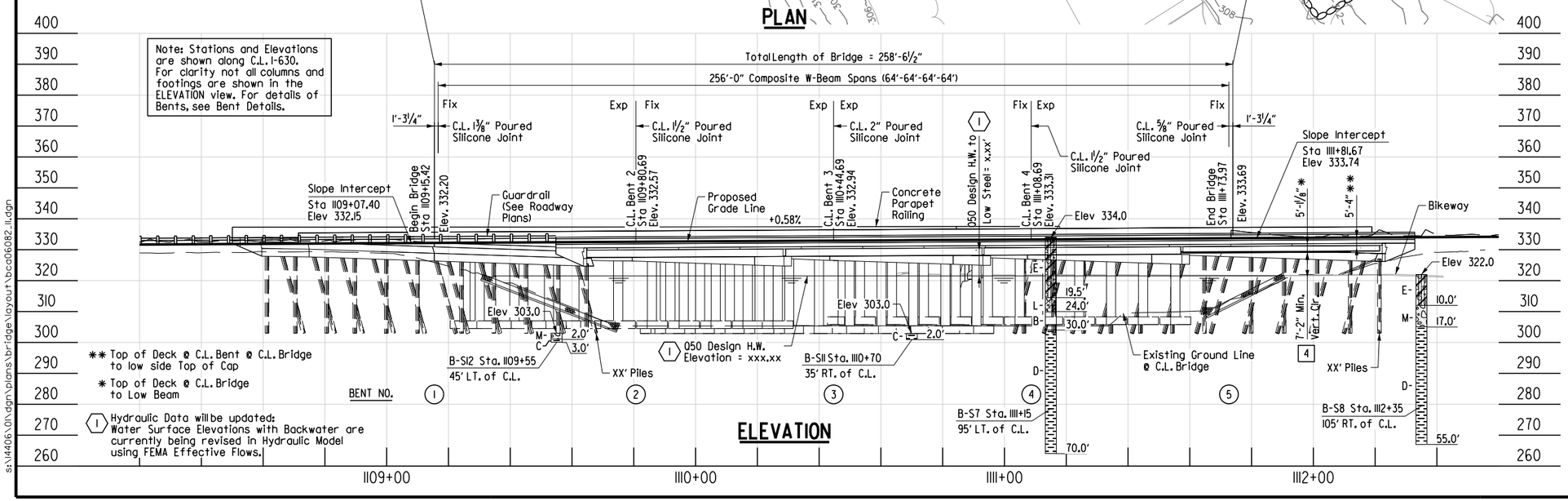
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		CA0608	170	XXXX
				1	A&B5582	BRIDGE LAYOUT		[Dwg#]

For R/W Data, See Roadway Plans

Use Type Special 1, Type Special 2 & Type Special 3 Approach Gutters with Type Special 1 & Type Special 2 Approach Slabs at Beginning of Bridge.

Use Type Special 4, Type Special 5, Type Special 6 & Type Special 7 Approach Gutters with Type Special 3, Type Special 4 & Type Special 5 Approach Slabs at End of Bridge.





BRIDGE A
WB Main Lanes and RAMP 4 are on Bridge A. Stationing and Profile Grade Line for Bridge A and WB approaches are along C.L. I-630 and is on a +0.58% grade. RAMP 4 control elevations prior to Sta. 703+61.67 on Bridge A and WB approaches are are derived from C.L. I-630 Profile Grade Line. Grade Break Line exists and bifurcates at 59.42' Left C.L. I-630 Sta. 1109+94.26=7.5' Right RAMP 4 Sta 701+92.78.

Prior to the bifurcation of Grade Break Line, the Grade Break Line follows along a concentric curve 59.42' Lt. of C.L. I-630. Cross Slope left of this concentric curve is radial to C.L. RAMP 4 and rotates about the Grade Break Line. Cross Slope right of this concentric curve, is 2.7% upward to the left radial to C.L. I-630.

After the bifurcation of Grade Break Line, the Left Grade Break Line follows along a concentric curve 7.5' Rt. of C.L. RAMP 4. Cross Slope left of this concentric curve is radial to C.L. RAMP 4 and rotates about the Left Grade Break Line. The Right Grade Break Line follows along a concentric curve 59.42' Lt. of C.L. I-630. Cross Slope right of this concentric curve, is 2.7% upward to the left radial to C.L. I-630.

The Cross Slope transitions uniformly from 2.7% downward to Rt. of C.L. RAMP 4 at Sta. 700+02.78 (C.L. I-630 Sta. 1108+02.75) to 4.0% downward to the left of C.L. RAMP 4 at Sta. 703+02.78 (C.L. I-630 Sta. 1110+98.81). The surface of bridge and end approach between Left and Right Grade Break Line has constant 0.0% Cross Slope radial to C.L. I-630 and is on a +0.58% grade.

For BRIDGE A Cross Slope Transition Sketch see Dwg. No. XXXX2

BRIDGE B
EB Main Lanes are on Bridge B. Stationing and Profile Grade Line for Bridge B and EB approaches are along C.L. I-630 and is on a +0.58% grade. Cross slope is constant 2.7% downward to the Rt.

For BRIDGE A & B Vertical Curve Data see Dwg. No. XXXX1

Horizontal Curve RAMP 4		Horizontal Curve I-630	
PI Sta	= 703+81.53	PI Sta	= 1109+89.35
Delta	= 12°56'05"	Delta	= 12°30'00"
Degree	= 3°0'00.00"	Degree	= 1°0'00.00"
Tangent	= 216.50'	Tangent	= 627.49'
Length	= 431.16'	Length	= 1250.00'
Radius	= 1909.86'	Radius	= 5729.57'
PC Sta	= 701+65.03	PC Sta	= 1103+61.86
PT Sta	= 705+96.15	PT Sta	= 1116+11.85
PC Brg	= S 85°43'56"E	PC Brg	= N 89°53'11"E
PT Brg	= N 81°19'59"E	PT Brg	= S 77°36'50"E

1. C.L. I-630 Sta 1108+63.24, 82.94' LT
2. C.L. I-630 Sta 1110+56.40, 88.41' LT
3. C.L. I-630 Sta 1111+04.94, 93.98' LT
4. Existing Vertical Clearance being maintained by use of shallow beams.
5. Dumped Riprap 1'-6" thick placed on "Filter Blanket"
6. Existing Concrete Riprap at Bridge End
7. Remove existing concrete riprap and replace with new concrete riprap, as necessary for Bent Construction (See Std. Dwg. No. 55002). Payment for this work shall be considered subsidiary to the pay item "Modifications of Existing Bridge Structure (Br. No. A5582)" and "Modification of Existing Bridge Structure (B5582)".
8. See Cross Slope Transition Sketch on Dwg. No. XXXX2

60% SUBMITTAL

PRELIMINARY FOR REVIEW ONLY

STEPHEN T. SMILEY, P.E., 13072

FEBRUARY-2015

BRIDGEFARMER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHEET 1 OF 4
LAYOUT OF I-630 BRIDGE OVER ROCK CREEK

BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING) (S)
PULASKI COUNTY
ROUTE 630, SECTION 21
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY:	AKH	DATE:	4/25/2014	FILENAME:	bca06082_11
CHECKED BY:	MT	DATE:	4/30/2014	SCALE:	1" = 20'
DESIGNED BY:	STS	DATE:	4/21/2014		

BRIDGE NO. A&B5582 DRAWING NO. XXXXX

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For R/W Data, See Roadway Plans

Horizontal Curve Data

PI Sta	= 106+03.72	PC Sta	= 109+72.63
PC Sta	= 105+73.88	PCC Sta	= 111+82.56
PT Sta	= 106+33.23	Δ	= 199°25'43"
Δ	= 14°50'22"	D	= 95°00'00"
D	= 25°00'00"	L	= 209.92'
T	= 29.85'	R	= 60.31'
L	= 59.36'		
R	= 229.18'		
PC Brg	= S 73°14'04"E		
PT Brg	= S 88°04'26"E		

60% SUBMITTAL

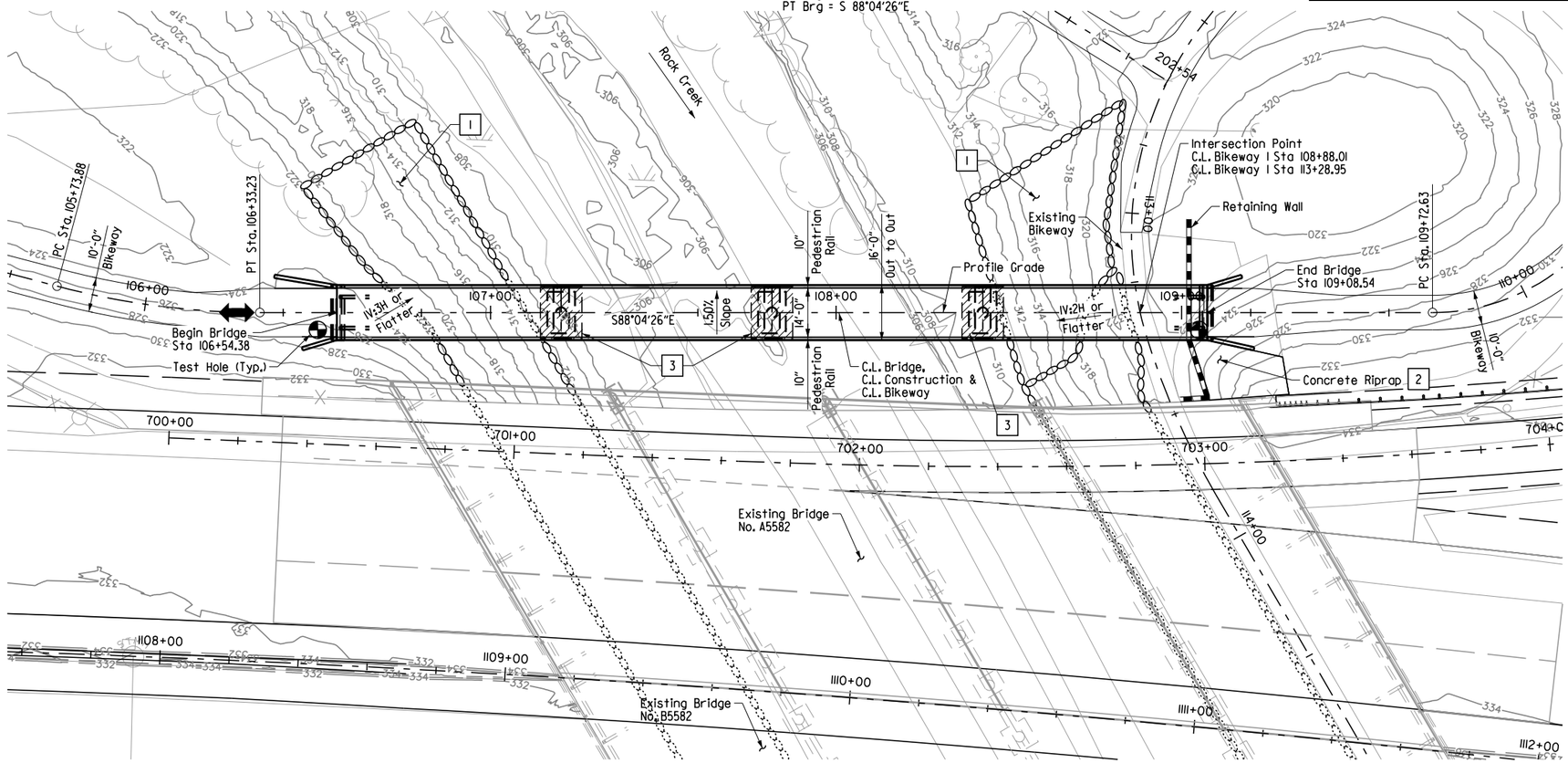
PRELIMINARY
FOR REVIEW ONLY

TZU-JUI TANG, P.E., 12896

FEBRUARY-2015

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.		CA0608	168	XXXX

(1) [Brdg#] **BRIDGE LAYOUT** [Dwg#]



PLAN

1 Dumped riprap 1'-6" thick placed on filter blanket.

2 Concrete Riprap at Bridge End.
(See Std. Dwg. No. 55002)

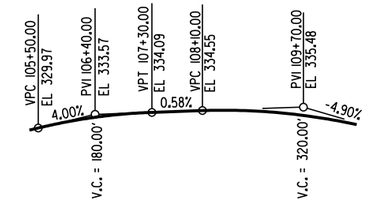
3 Remove existing concrete riprap and replace with new concrete riprap, as necessary for Bent Construction (See Std. Dwg. No. 55002). Payment for this work shall be considered subsidiary to the pay item "Modifications of Existing Bridge Structure (Br. No. A5582)".

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE (2)	NATURAL WATER SURFACE ELEVATION (1&3)	WATER SURFACE ELEV. WITH BACKWATER (4)
	YEARS	CFS	FEET	FEET
Design	50	22,700	321.50	xxx.xx
Base	100	25,000	322.51	xxx.xx
Extreme	500	30,250	324.77	xxx.xx
Overtopping	N/A	N/A	N/A	N/A

1. Existing water surface without Proposed Structure or Roadway approaches.
2. DISCHARGE FLOWS ARE FEMA EFFECTIVE FLOWS
3. WSEL's ARE FEMA EFFECTIVE WSEL's
4. HYDRAULIC WSEL's ARE UNDER REVISION (1)

Proposed Low Bridge Chord elevation = 329.38 ft.
Drainage area = 15.93 square miles.
Historical H.W. Elev. = 321.0 ft. (1969)

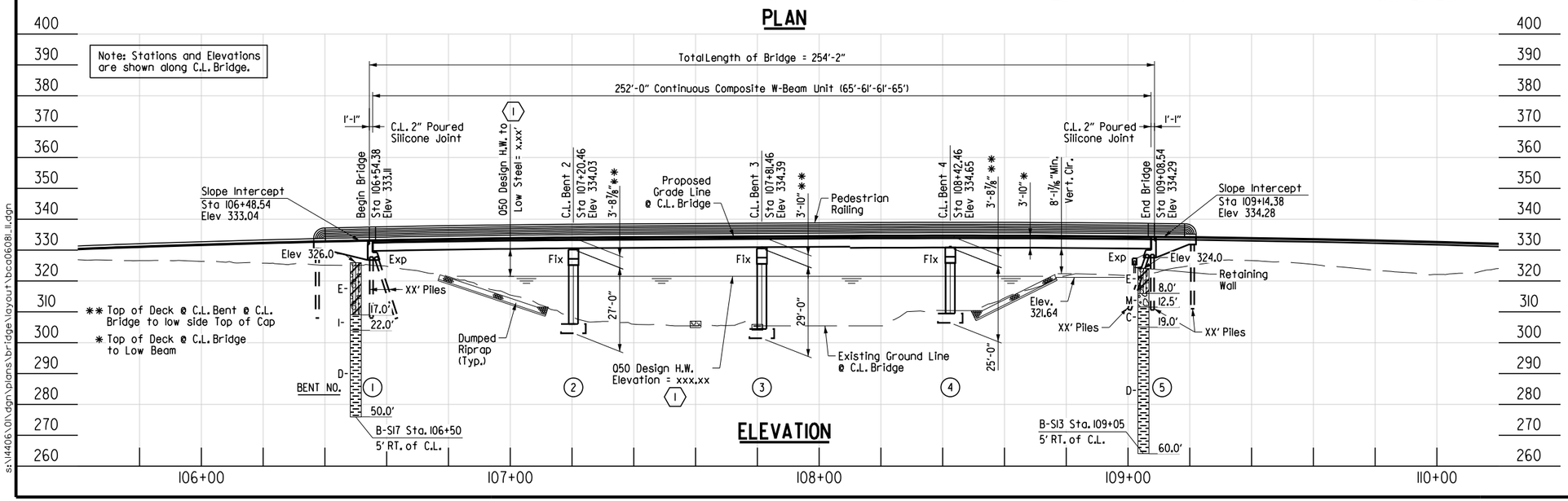


VERTICAL CURVE DATA

(Along Profile Grade)

(1) Hydraulic Data will be updated:
Water Surface Elevations with Backwater are currently being revised in Hydraulic Model using FEMA Effective Flows.

Note: Stations and Elevations are shown along C.L. Bridge.



ELEVATION

BRIDGEFARMER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHEET 1 OF 3
LAYOUT OF PEDESTRIAN BRIDGE
I-630 AT ROCK CREEK

BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING) (S)
PULASKI COUNTY
ROUTE 630, SECTION 21
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY:	AKH	DATE:	4/25/2014	FILENAME:	bca06081_11
CHECKED BY:	STS	DATE:	4/30/2014	SCALE:	1" = 20'
DESIGNED BY:	MT	DATE:	4/21/2014		

BRIDGE NO. [Brdg#] DRAWING NO. XXXXX

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For R/W Data, See Roadway Plans

Use Type Special 8, Type Special 9 & Type Special 10 Approach Gutters with Type Special 6 & Type Special 7 Approach Slabs at Beginning of Bridge.

Limits of Noise Wall are not yet finalized. Wall Length will be determined later.

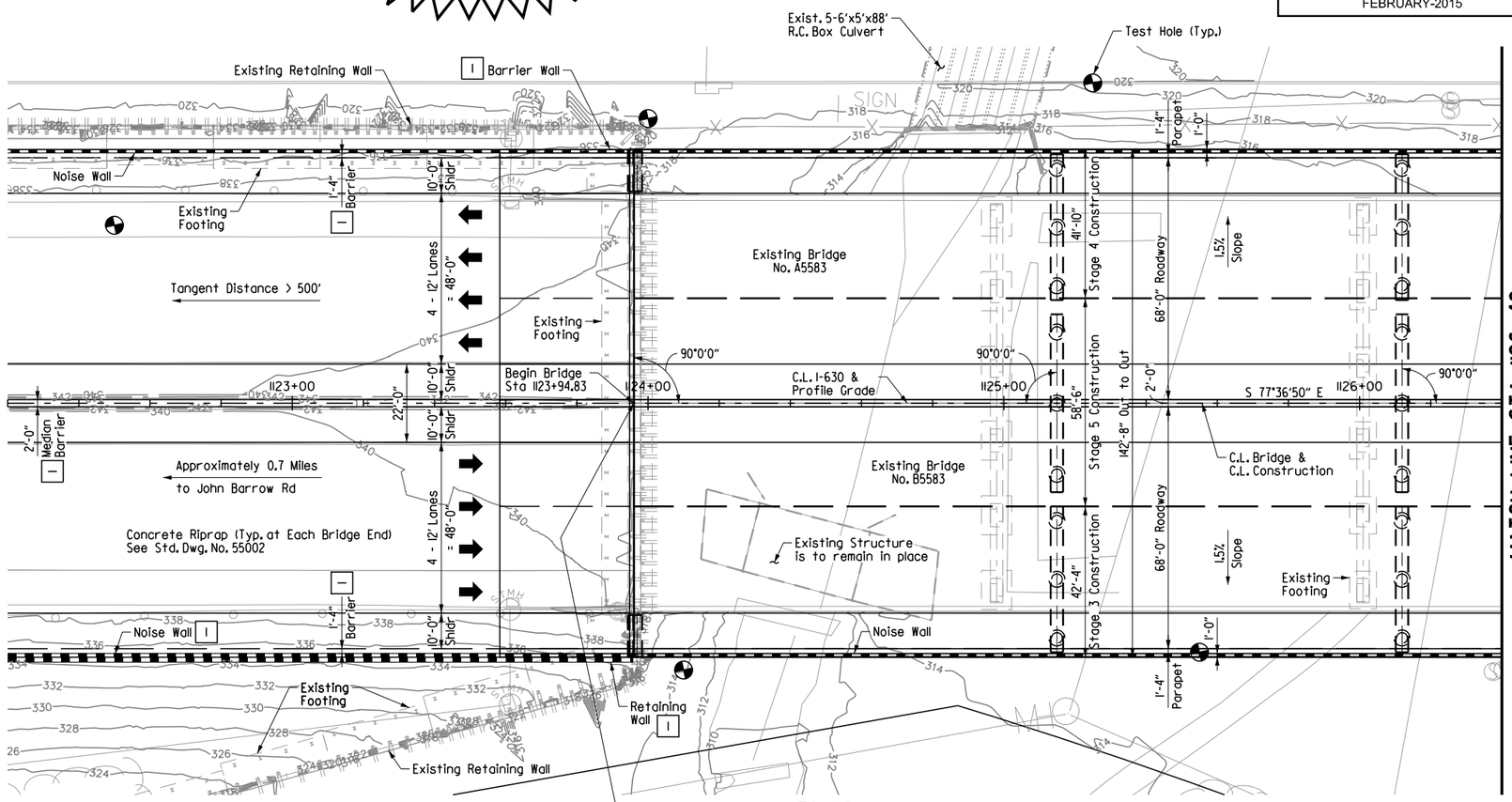
60% SUBMITTAL

PRELIMINARY FOR REVIEW ONLY

STEPHEN T. SMILEY, P.E., 13072

FEBRUARY-2015

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.		CA0608	174	XXXX
				1 A&B5583		BRIDGE LAYOUT		[Dwg#]



PLAN

1 See Rdwy Plans

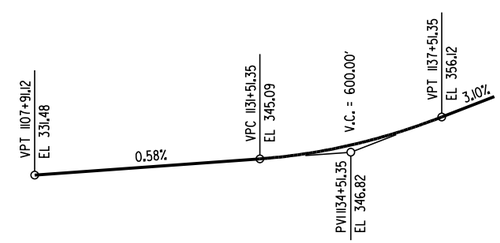
Note:
The proposed bridge is positioned to maximize re-use of Existing Abutment 1 and to minimize interference with the existing substructure. The Contractor shall verify the location of the existing substructure before constructing the new substructure. Any adjustments necessary to fit the proposed bridge shall be submitted to the Engineer for approval.

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE (2)	NATURAL WATER SURFACE ELEVATION (1 & 3)	WATER SURFACE ELEV. WITH BACKWATER (4)
	YEARS	CFS	FEET	FEET
Design	50	2,562	320.50	xxx.xx
Base	100	2,848	321.50	xxx.xx
Extreme	500	3,484	323.80	xxx.xx
Overtopping	N/A	N/A	N/A	N/A

1. Existing water surface without Proposed Structure or Roadway approaches.
2. DISCHARGE FLOWS ARE CALCULATED FLOWS FOR BRIARWOOD CREEK ONLY
3. WSEL's ARE INTERPOLATED FROM FEMA ROCK CREEK MODEL
4. HYDRAULIC WSEL's ARE UNDER REVISION 1

Proposed Low Bridge Chord elevation = 336.29 ft.
Drainage area = 1.08 square miles.
Historical H.W. Elev. = 314.8 ft. (As-Builts)



VERTICAL CURVE DATA

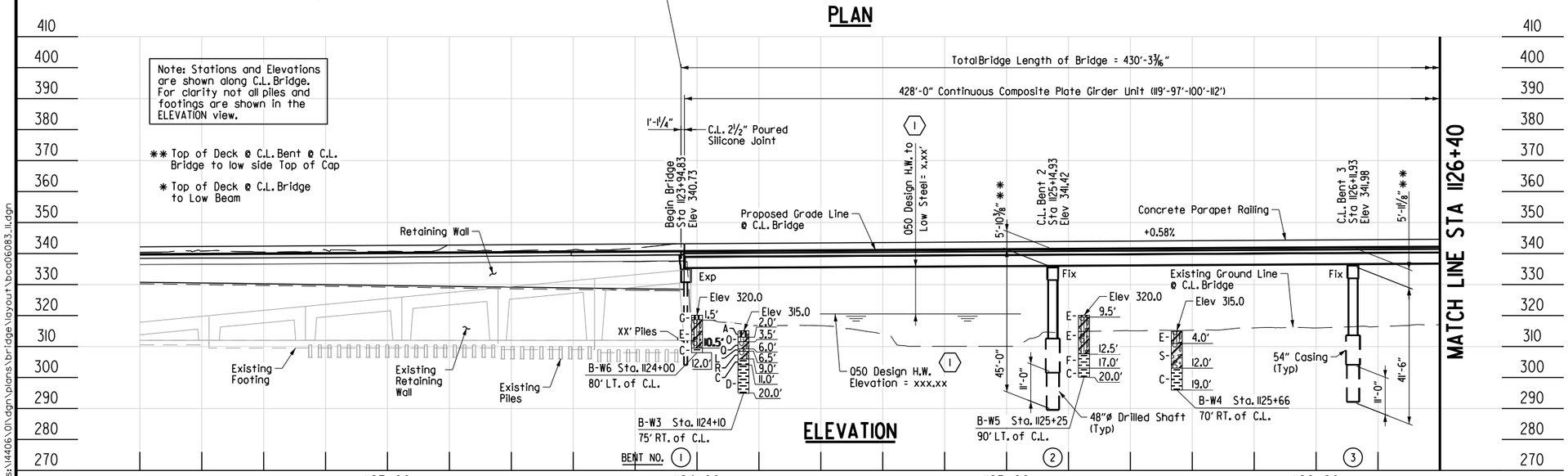
(Along Profile Grade)

1 Hydraulic Data will be updated:
Water Surface Elevations with Backwater are currently being revised in Hydraulic Model using FEMA Effective Flows.

Note: Stations and Elevations are shown along C.L. Bridge. For clarity not all piles and footings are shown in the ELEVATION view.

** Top of Deck @ C.L. Bent @ C.L. Bridge to low side Top of Cap

* Top of Deck @ C.L. Bridge to Low Beam



ELEVATION

BRIDGEFARMER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHEET 1 OF 5
LAYOUT OF I-630 BRIDGE OVER RODNEY PARHAM

BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING) (S)
PULASKI COUNTY
ROUTE 630, SECTION 21

ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY:	AKH	DATE:	4/25/2014	FILENAME:	bca06083_11
CHECKED BY:	MT	DATE:	4/30/2014	SCALE:	1" = 20'
DESIGNED BY:	STS	DATE:	4/21/2014		

BRIDGE NO. A&B5583 DRAWING NO. XXXXX

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For R/W Data, See Roadway Plans

Use Type Special 11, Type Special 12 & Type Special 13 Approach Gutters with Type Special 8 & Type Special 9 Approach Slabs at End of Bridge.

Limits of Noise Wall are not yet finalized. Wall Length will be determined later.

60% SUBMITTAL

PRELIMINARY FOR REVIEW ONLY

STEPHEN T. SMILEY, P.E., 13072

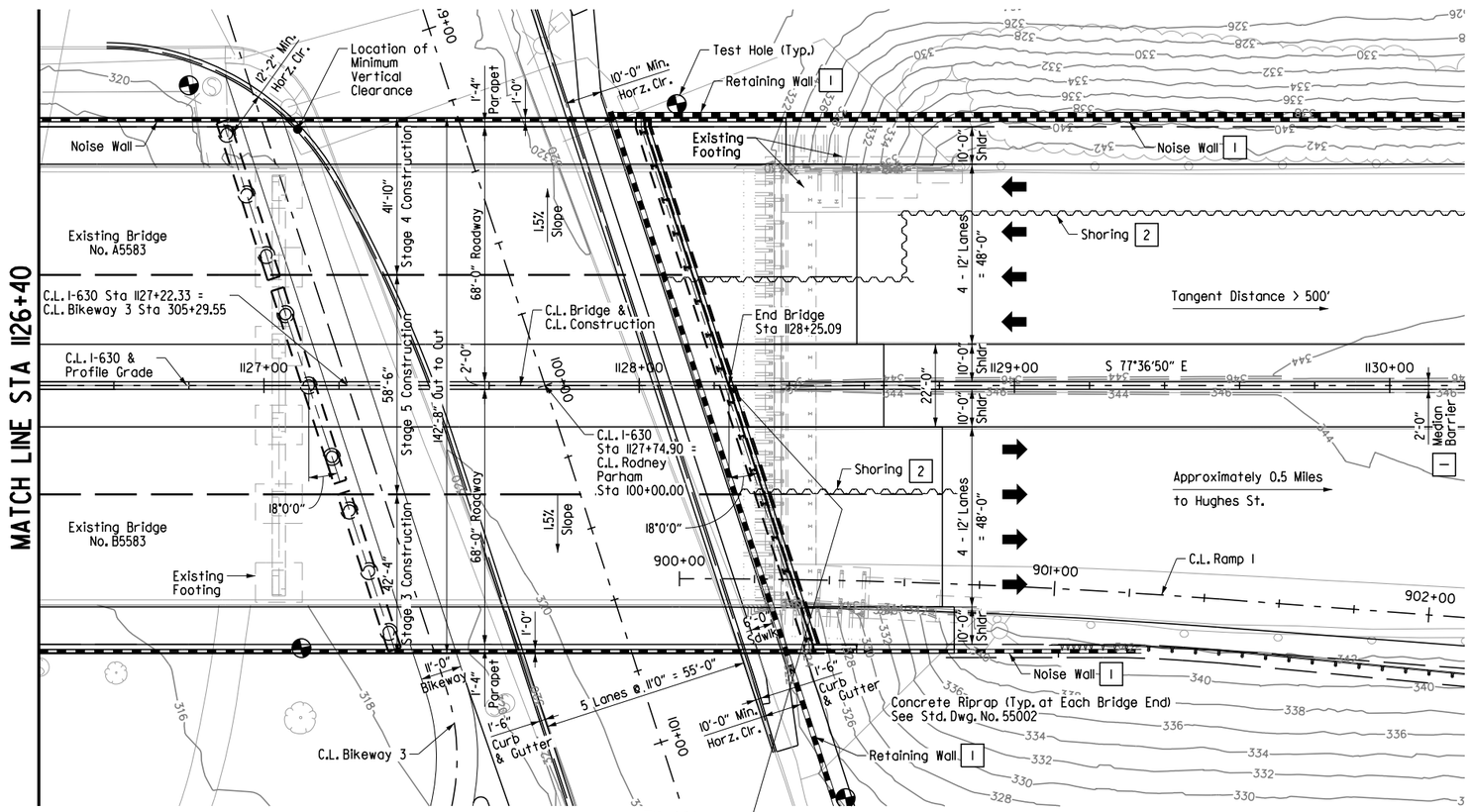
FEBRUARY-2015

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.		CA0608	174A	XXXX
				1	A&B5583	BRIDGE LAYOUT		[Dwg#]

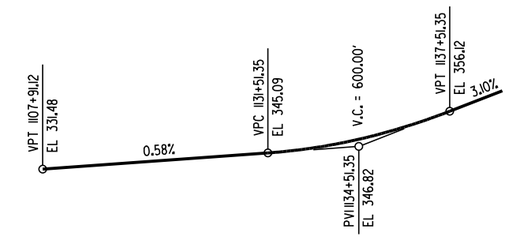
1 See Rdwy Plans

2 Shoring will be required during construction. See SP Job CA0608 "Shoring".

Note:
The proposed bridge is positioned to maximize re-use of Existing Abutment 1 and to minimize interference with the existing substructure. The Contractor shall verify the location of the existing substructure before constructing the new substructure. Any adjustments necessary to fit the proposed bridge shall be submitted to the Engineer for approval.



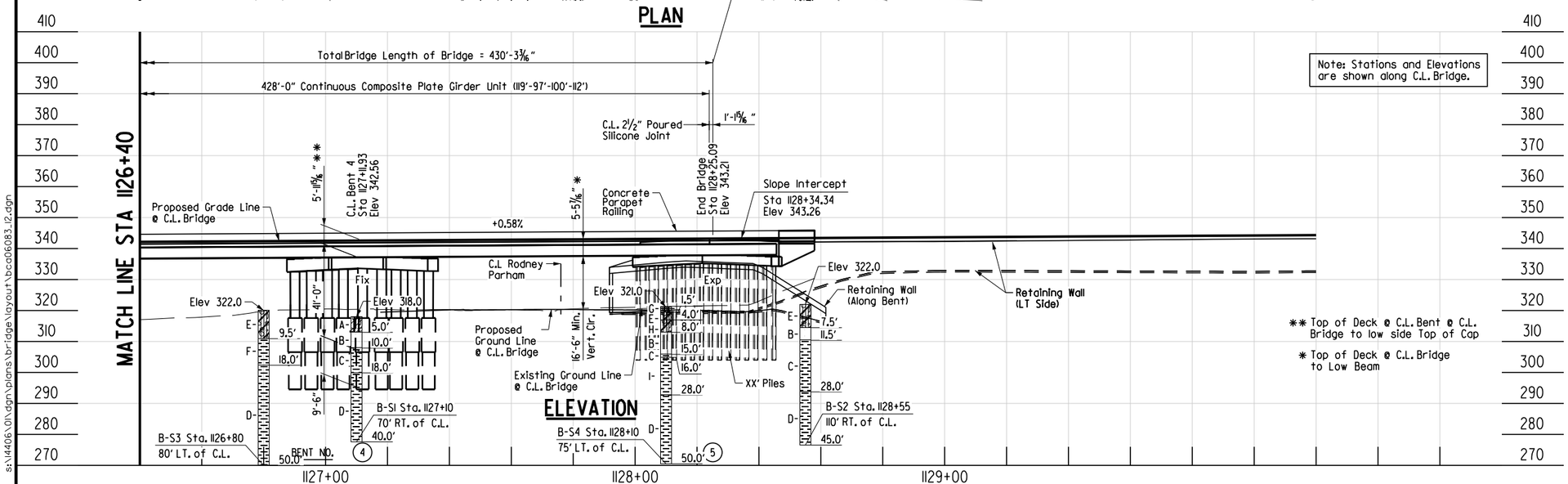
PLAN



VERTICAL CURVE DATA

(Along Profile Grade)

Note: Stations and Elevations are shown along C.L. Bridge.



ELEVATION

** Top of Deck @ C.L. Bent @ C.L. Bridge to low side Top of Cap

* Top of Deck @ C.L. Bridge to Low Beam

BRIDGEFARMER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHEET 2 OF 5
LAYOUT OF I-630 BRIDGE OVER RODNEY PARHAM

BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING) (S)
PULASKI COUNTY
ROUTE 630, SECTION 21

ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY:	AKH	DATE:	4/25/2014	FILENAME:	bca06083_14
CHECKED BY:	MT	DATE:	4/30/2014	SCALE:	1" = 20'
DESIGNED BY:	STS	DATE:	4/21/2014		

BRIDGE NO. A&B5583 DRAWING NO. XXXXX

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For R/W Data, See Roadway Plans

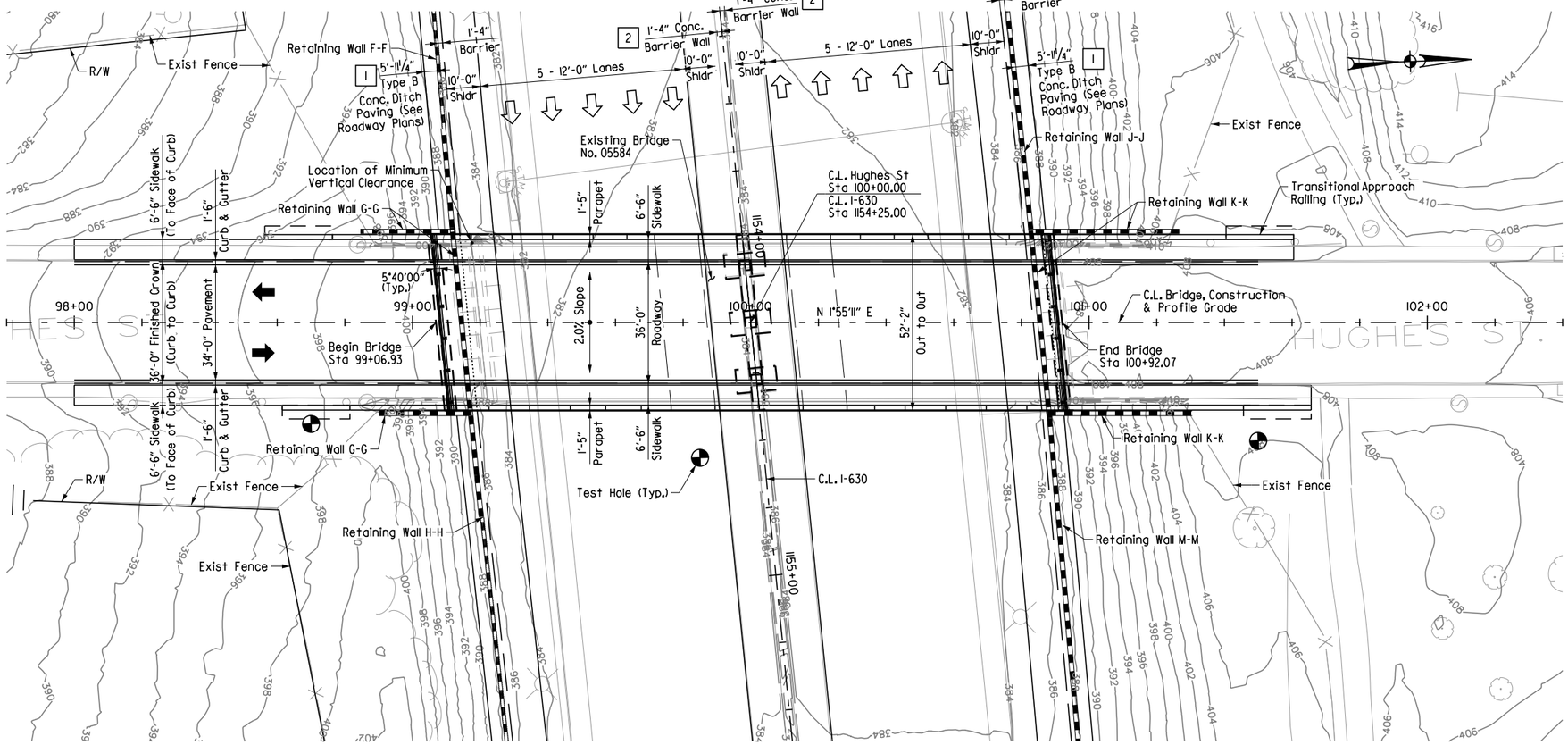
60% SUBMITTAL

PRELIMINARY
FOR REVIEW ONLY

TZU-JUI TANG, P.E., 12896

FEBRUARY-2015

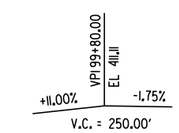
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		CA0608	178	XXXX
				1	05584	BRIDGE LAYOUT		[Dwg#]



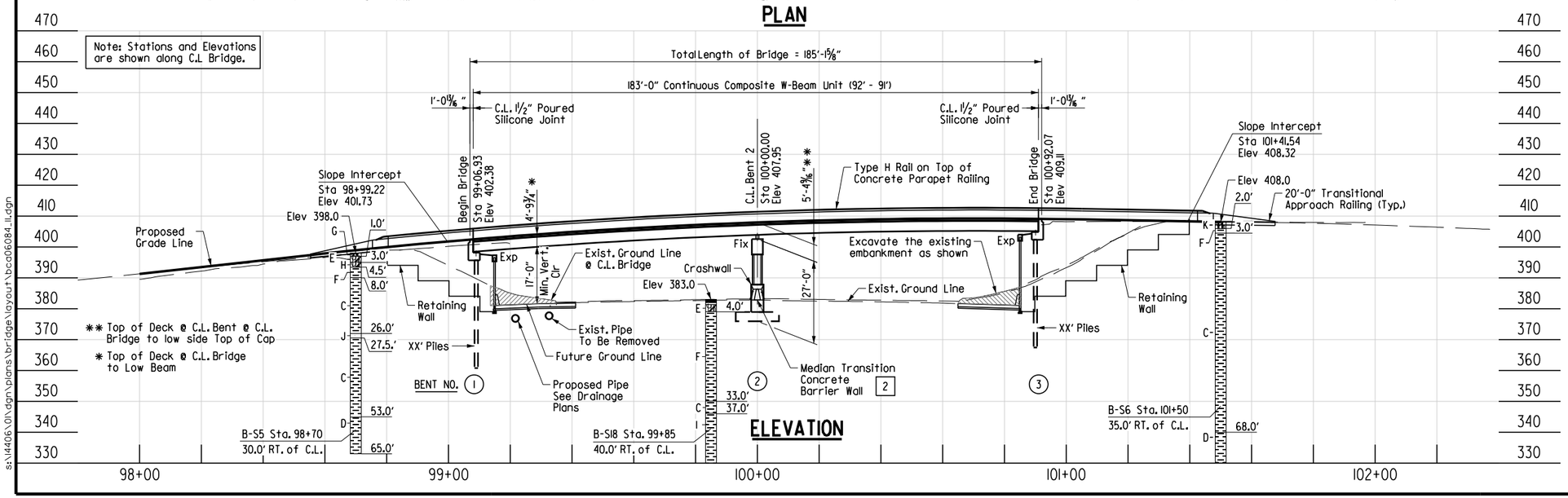
PLAN

1 See Roadway Plans

2 Median barrier width, height, and slope of face transitions to and from each end of Bent Crashwall. See Roadway Plans.



VERTICAL CURVE DATA
(Along Profile Grade)



ELEVATION

BRIDGEFARMER & ASSOCIATES, INC.
CONSULTING ENGINEERS

SHEET 1 OF 3
LAYOUT OF HUGHES STREET BRIDGE
OVER I-630

BAPTIST HOSPITAL-UNIVERSITY AVENUE (WIDENING) (S)
PULASKI COUNTY
ROUTE 630, SECTION 21

ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY:	AKH	DATE:	4/25/2014	FILENAME:	bca06084_11
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DESIGNED BY:	MT	DATE:	4/21/2014		

BRIDGE NO. 05584 DRAWING NO. XXXXX

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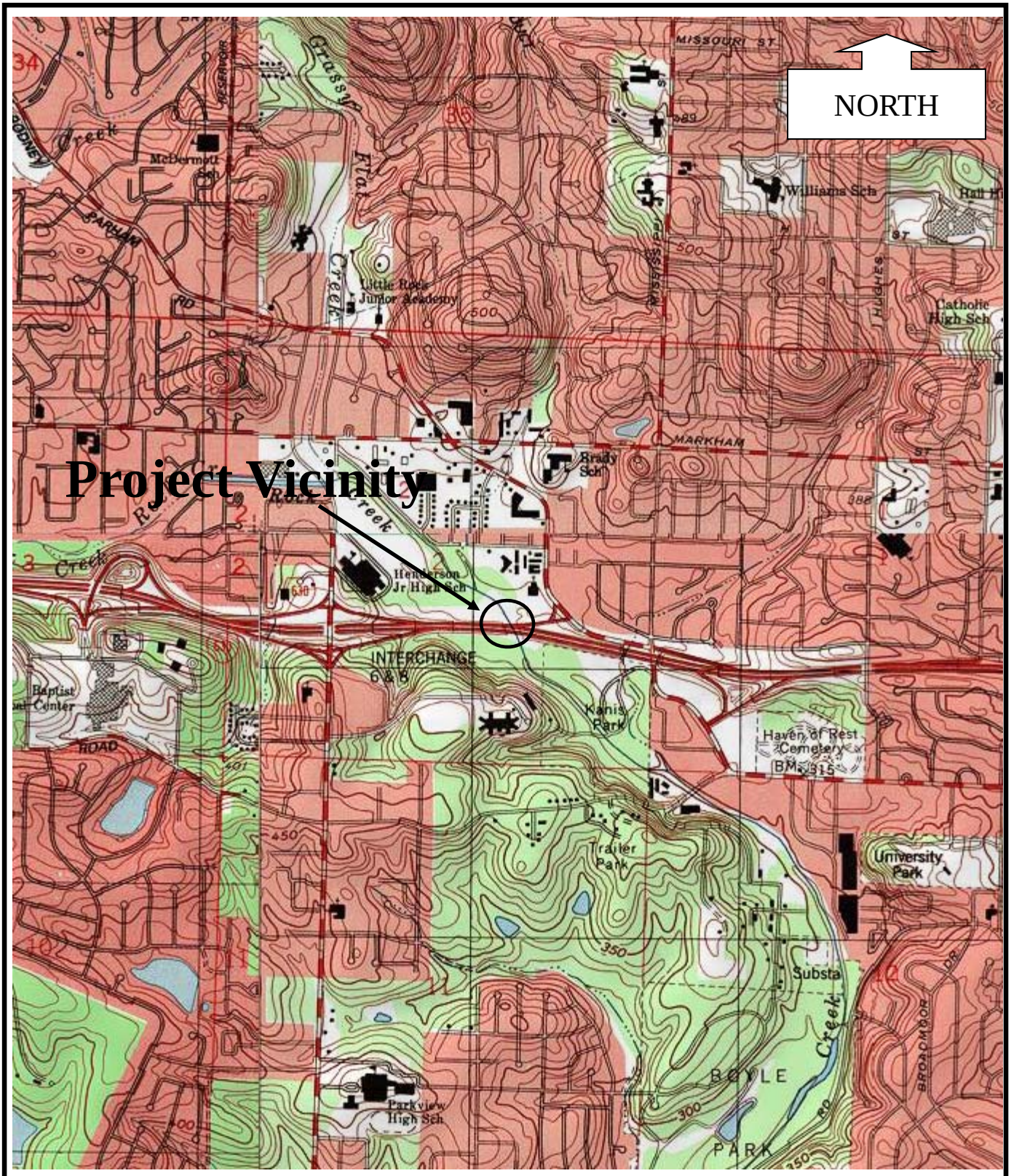
ATTACHMENT 2

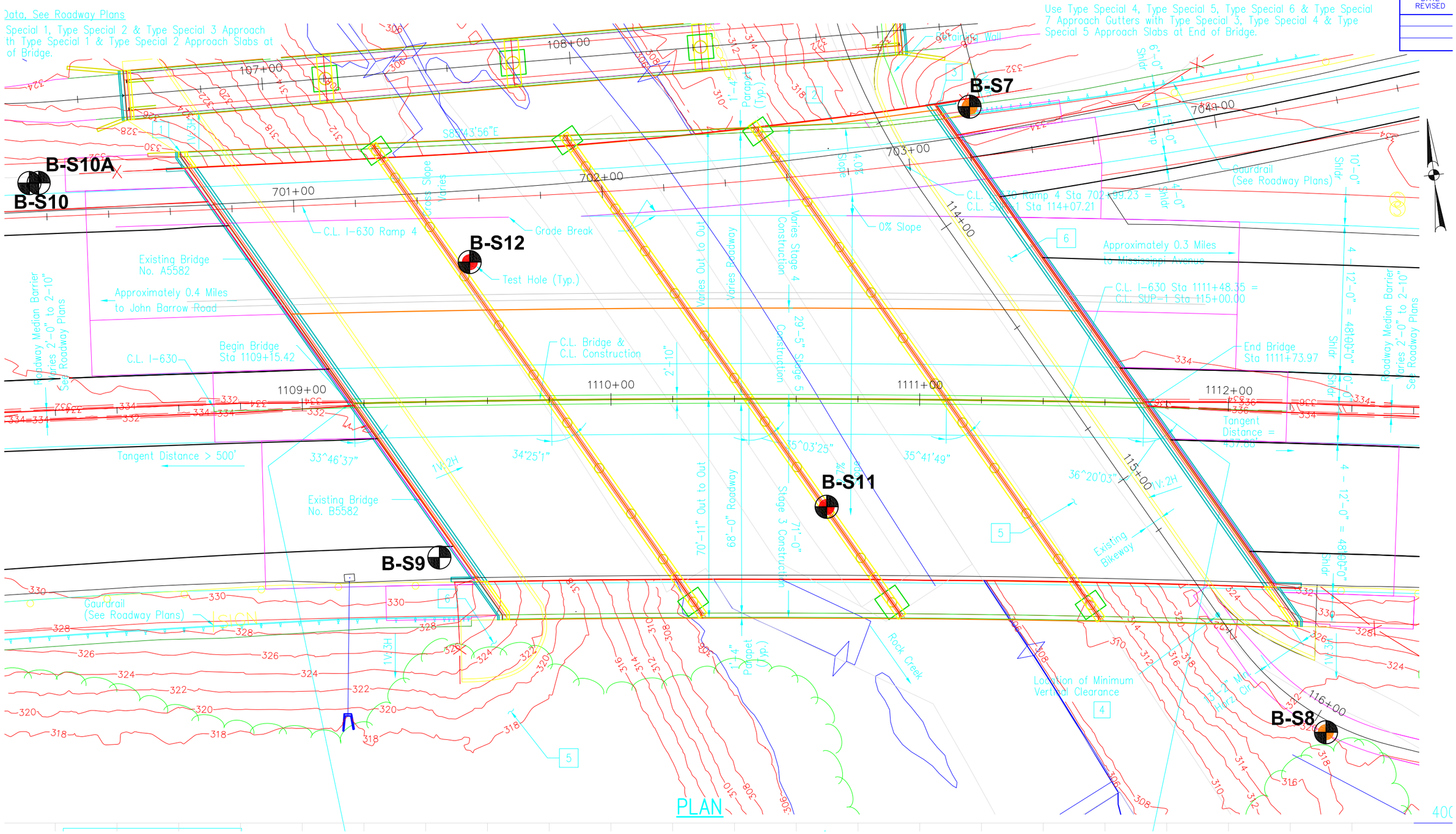


Grubbs, Hoskyn, Barton & Wyatt, INC.
CONSULTING ENGINEERS

SITE VICINITY MAP
I-630 over Rock Creek
AHTD CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Job No. 14-030

Plate 1



Note: Base drawing provided by Bridgefarmer & Associates, Inc.

15 ft 0 15 30 ft

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

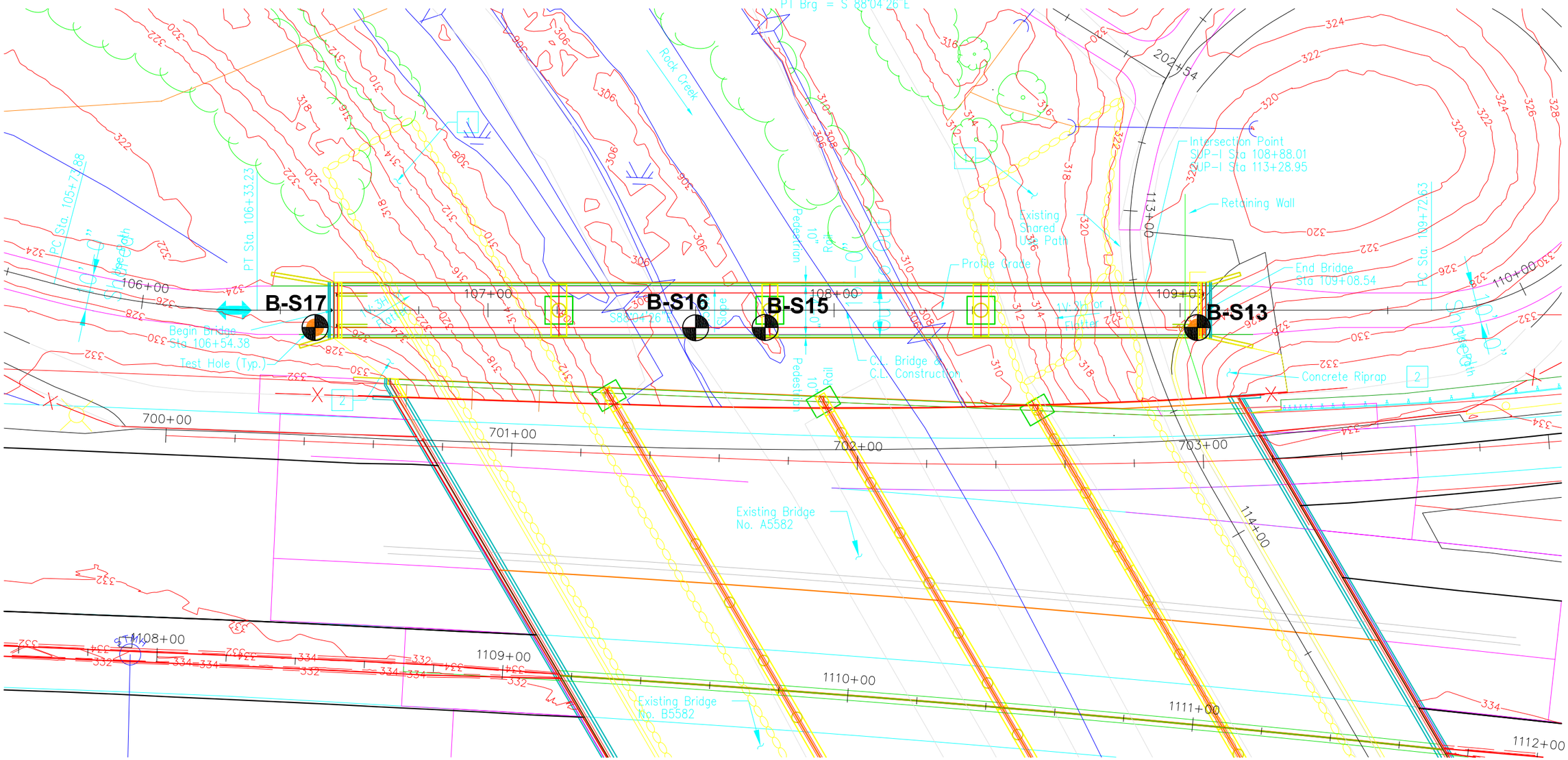
Plan of Borings
I-630 over Rock Creek
AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
December 19, 2014	Plate 2a

PT Sta = 106+33.23
Δ = 14°50'22"
D = 25°00'00"
T = 29.85'
L = 59.36'
R = 229.18'
PC Brg = S 73°14'04"E
PT Brg = S 88°04'26"E

Δ = 199°25'43"
D = 95°00'00"
L = 209.92'
R = 60.31'

PRELIMINARY
FOR REVIEW ONLY
TZU-JUI TANG, P.E., 12896
NOVEMBER-2014



PLAN

Total Length of Bridge = 254'-2"

252'-0" Continuous Composite W-Beam Unit (65'-61'-61'-65')

Note: Stations and Elevations are shown along C.L. Bridge.

Note: Base drawing provided by Bridgefarmer & Associates, Inc.

15 ft 0 15 30 ft

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

Plan of Borings
I-630 over Pedestrian Bridge
AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
December 19, 2014	Plate 2b

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

LOG OF BORING NO. S7

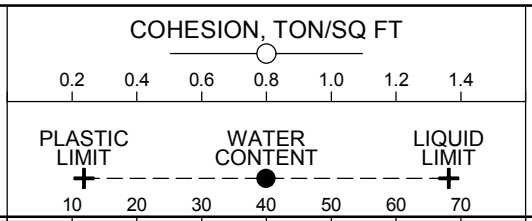
CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1111+15, 95 ft Lt

SURF. EL: 334±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	3 inches: Asphalt Concrete	10		41
	Stiff brown silty clay and shale fragments (fill)	11		
5	- with sandstone fragments below 4.5 ft	20		
		16		
10	- tan and gray with glass debris below 9 ft	21		
15	- with more shale fragments below 14 ft	19		23
20	Medium dense gray and tan clayey fine sand w/sandstone fragments (completely weathered sandstone)	14		
25	Moderately hard tan and gray weathered fine-grained sandstone w/clayey fine sand seams and ferrous stains	50/3"		
30	Moderately hard to hard dark gray shale	30/0"		
35		30/0"		
40	- with medium close sandstone seams below 40 ft			
45		30/0"		
50		30/0"		
55		30/0"		
60		30/0"		
65		30/0"		
70				



COMPLETION DEPTH: 70.0 ft
DATE: 8-22-14

DEPTH TO WATER IN BORING: Dry to 20 ft

DATE: 8/22/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S8

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 1112+35, 105 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 322±	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 %
			2 inches: Asphalt Concrete	25/0"			
			Stiff to very stiff dark gray silty clay w/shale and sandstone fragments (fill)	25/0"			
5			- tan and dark gray below 4 ft	25			
				26			24
				43			
10			Dense reddish brown sandy fine to coarse gravel w/some cobbles				
				25/0"			
15							
			Moderately hard to hard dark gray shale w/medium close sandstone partings and seams	25/0"			
20							
				25/0"			
25							
				25/0"			
30							
				25/0"			
35							
				25/0"			
40							
				25/0"			
45							
				25/0"			
50							
				25/0"			

COMPLETION DEPTH: 55.0 ft
DATE: 8-19-14

DEPTH TO WATER IN BORING: 15 ft

DATE: 8/19/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S9

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1109+45, 50 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 330±	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 %
			18 inches: Portland Cement Concrete 8 inches: Crushed Stone Base				
			Very stiff gray, tan and reddish brown silty clay w/some fine to coarse gravel and shale fragments (fill)	32			
5			- stiff with clay pockets at 4 to 6 ft	14			69
			- firm at 6 to 8 ft	8			
			- stiff with some crushed sandstone and trace wood debris at 8 to 10 ft	14			
10			- stiff to very stiff with more crushed sandstone below 10 ft	24			
15			Very stiff olive gray fine sandy clay w/some crushed sandstone and fine gravel (fill)	35			57
20			Dense to very dense brown and reddish brown sandy fine to coarse gravel w/trace cobbles	50/8"			
25			Moderately hard to hard dark gray shale w/medium close sandstone partings and seams	25/0"			
30				25/0"			
35				25/0"			

COMPLETION DEPTH: 35.0 ft
DATE: 12-6-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 12/6/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S10

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 1108+15, 75 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 333±	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 %
			2 inches: Asphalt Concrete 6 inches: Crushed Stone Base				
			Stiff reddish tan and reddish brown silty clay w/some shale and sandstone fragments (fill)	16			
				19			37
			- with some cobbles and boulders below 4 ft				
5				25			
				11			
				17			
10							
			NOTE: Hole abandoned at refusal on boulder at 10 ft				
15							

COMPLETION DEPTH: 10.0 ft
DATE: 9-15-14

DEPTH TO WATER IN BORING: Dry

DATE: 9/15/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S10A

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 20 ft /Wash

LOCATION: Sta 1108+18, 75 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 333±	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 %
			2 inches: Asphalt Concrete 6 inches: Crushed Stone Base				60
5			Stiff reddish tan and reddish brown silty clay w/some sandstone and shale fragments (fill) - with occasional cobbles and boulders below 4 ft				
10							
15			- stiff to very stiff gray silty clay, slightly sandy with trace fine gravel and ferrous stains - with more cobbles and boulders below 15 ft	51 50/2"			74
20							
25			Moderately hard to hard dark gray shale w/medium close sandstone seams and partings - with occasional quartz veins below 27 ft	50/1" 25/0"			
30							
35				25/0"			
40				25/0"			
45				25/0"			
50				25/0"			

NOTE: Boring offset 3 ft east of S10.

COMPLETION DEPTH: 50.0 ft
DATE: 9-16-14

DEPTH TO WATER
IN BORING: Dry to 20 ft

DATE: 9/16/2014

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

LOG OF BORING NO. S11

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Visual

LOCATION: Sta 1110+70, 35 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 308±	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 %
			Low hardness to moderately hard tan and dark gray shale, dip = ±80° N				
1							
2							
3			NOTE 1: Logged from exposure in creek bed.				
4			NOTE 2: Water depth 5 to 6 ft.				
5							
6							
7							
8							
9							

COMPLETION DEPTH: 2.0 ft
DATE: 9-9-14

DEPTH TO WATER IN BORING: N/A

DATE: 9/9/2014

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

LOG OF BORING NO. S12

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Visual

LOCATION: Sta 1109+55, 45 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 306±	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 %
			Loose brownish gray sandy fine to coarse gravel				
1							
2			Low hardness to moderately hard tan and dark gray moderately weathered shale, dip = ±75° N				
3							
4			NOTE 1: Logged from exposure in creek bed.				
5			NOTE 2: Water depth 5 to 6 ft.				
6							
7							
8							
9							

COMPLETION DEPTH: 3.0 ft
DATE: 9-9-14

DEPTH TO WATER IN BORING: N/A

DATE: 9/9/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S13

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 8 ft /Wash

LOCATION: Sta 109+05, 5 ft Rt

SURF. EL: 323±

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	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 %	% Recovery	% RQD
			Stiff brown silty clay w/shale and sandstone fragments (fill)	58/6"					
				19					
5			- reddish brown below 4 ft	25					
			- tan fine sandy clay w some sandstone fragments below 6 ft	17			39		
10			Dense brown and tan sandy fine to coarse gravel w/some cobbles	25/0"					
15			Moderately hard tan and dark gray weathered shale w/medium close sandstone partings and seams	50/2"					
				25/0"					
20			Moderately hard to hard dark gray shale w/medium close sandstone partings and seams - no recovery on core run at 21 to 26 ft					0	0
25			- no recovery on core run at 26 to 31 ft					0	0
30				25/0"					
				25/0"					
35									
				25/0"					
40									
				25/0"					
45									
				25/0"					
50									
				25/0"					
55									
				25/0"					
60									

COMPLETION DEPTH: 60.0 ft
DATE: 8-21-14

DEPTH TO WATER
IN BORING: Dry to 8 ft

DATE: 8/21/2014

RECRQDN200-2 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S15

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Visual

LOCATION: Sta 107+80, 5 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 306±	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 %
			Low hardness to moderately hard tan and dark gray moderately weathered shale, dip = ±75° N				
1							
2							
3			NOTE 1: Logged from exposure in creek bed.				
4			NOTE 2: Water depth 3 ft.				
5							
6							
7							
8							
9							

COMPLETION DEPTH: 2.0 ft
DATE: 9-9-14

DEPTH TO WATER IN BORING: N/A

DATE: 9/9/2014

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

LOG OF BORING NO. S16

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Visual

LOCATION: Sta 107+60, 5 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 305±	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 %
			Loose brownish gray sandy fine to coarse gravel				
1							
			Low hardness to moderately hard tan and dark gray moderately weathered shale				
2							
3			NOTE 1: Logged from exposure in creek bed.				
4			NOTE 2: Water depth 2 ft.				
5							
6							
7							
8							
9							

COMPLETION DEPTH: 2.0 ft
DATE: 9-9-14

DEPTH TO WATER IN BORING: N/A

DATE: 9/9/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

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

LOG OF BORING NO. S17

CA0608: I-630 over Rock Creek
Little Rock, Arkansas

TYPE: Auger to 15 ft /Wash

LOCATION: Sta 106+50, 5 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 326±	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 %
			Very stiff tan silty clay w/some shale and sandstone fragments, cobbles and boulders (fill)				
5			- stiff below 4 ft				
			- with some wood debris at 7 ft - very stiff with less cobbles and boulders below 8 ft	36			67
10							
			- more cobbles and boulders below 12 ft	50/2"			
15							
			Moderately hard tan and dark gray slightly weathered shale w/medium close sandstone partings	50/2"			
20							
			Moderately hard to hard dark gray shale w/medium close sandstone seams and partings	50/1"			
25							
				30/0"			
30							
				25/0"			
35							
			- with occasional quartz veins below 38 ft	25/0"			
40							
				25/0"			
45							
				25/0"			
50							

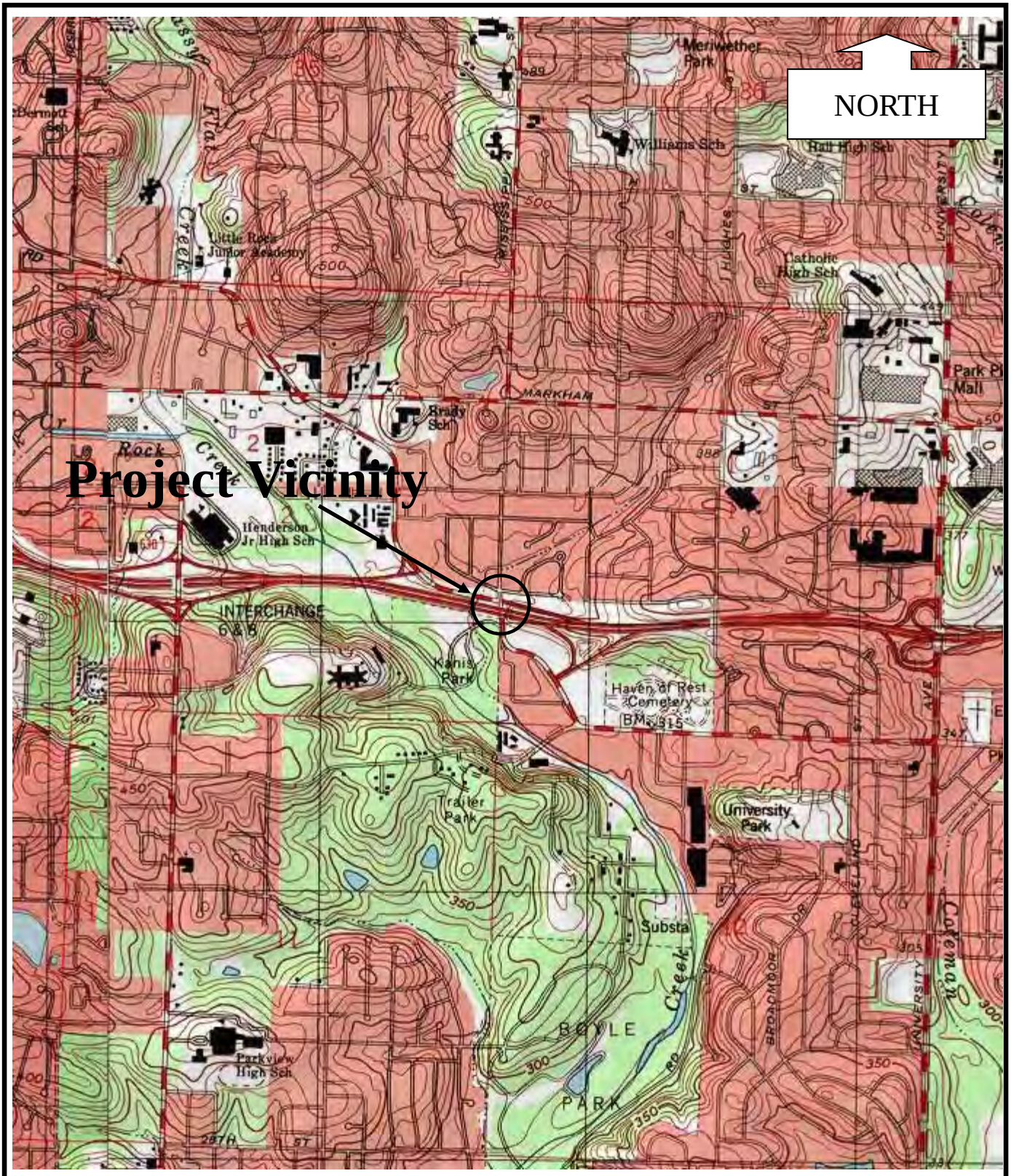
COMPLETION DEPTH: 50.0 ft
DATE: 9-15-14

DEPTH TO WATER
IN BORING: Dry to 15 ft

DATE: 9/15/2014

LGBNEW 14-030_I-630 OVER ROCK.GPJ 2-5-15

ATTACHMENT 3



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

SITE VICINITY MAP
I-630 over Rodney Parham Road
AHTD CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Job No. 14-030

Plate 1

For R/W Data, See Roadway Plans

Use Type Special 8, Type Special 9 & Type Special 10 Approach Gutters with Type Special 6 & Type Special 7 Approach Slabs at Beginning of Bridge.

Limits of Noise Wall are not yet finalized. Wall Length will be determined later.

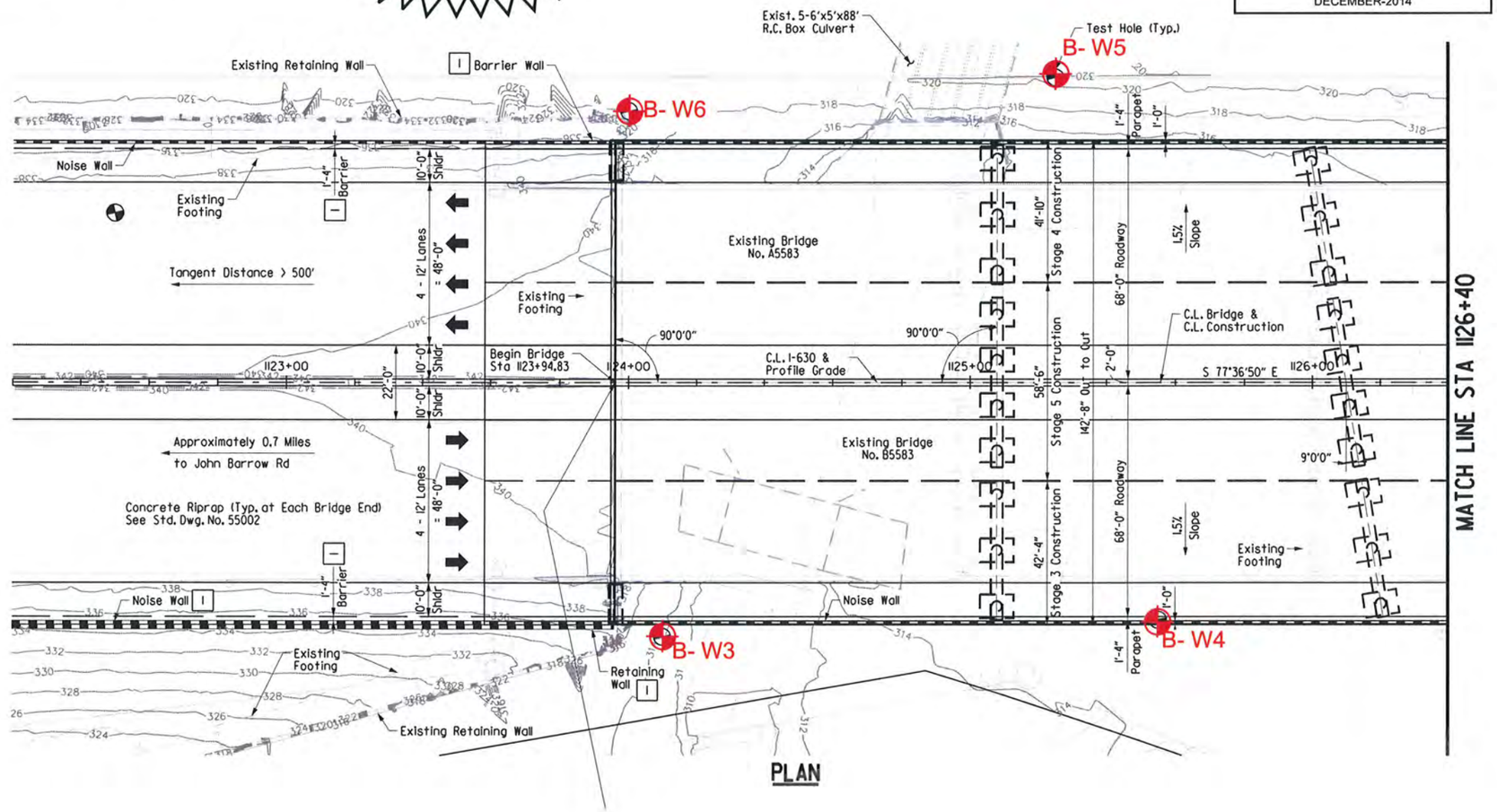
60% SUBMITTAL

PRELIMINARY FOR REVIEW ONLY

STEPHEN T. SMILEY, P.E., 13072

DECEMBER-2014

DATE REVISED	DA FLA



PLAN

Note: Base drawing provided by Bridgefarmer & Associates, Inc.

20 ft 0 20 40 ft

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

Plan of Borings
I-630 over Rodney Parham Road
AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 5, 2015	Plate 2a

Data, See Roadway Plans

e Special 11, Type Special 12 & Type Special 13 Approach with Type Special 8 & Type Special 9 Approach Slabs of Bridge.

Limits of Noise Wall are not yet finalized. Wall Length will be determined later.

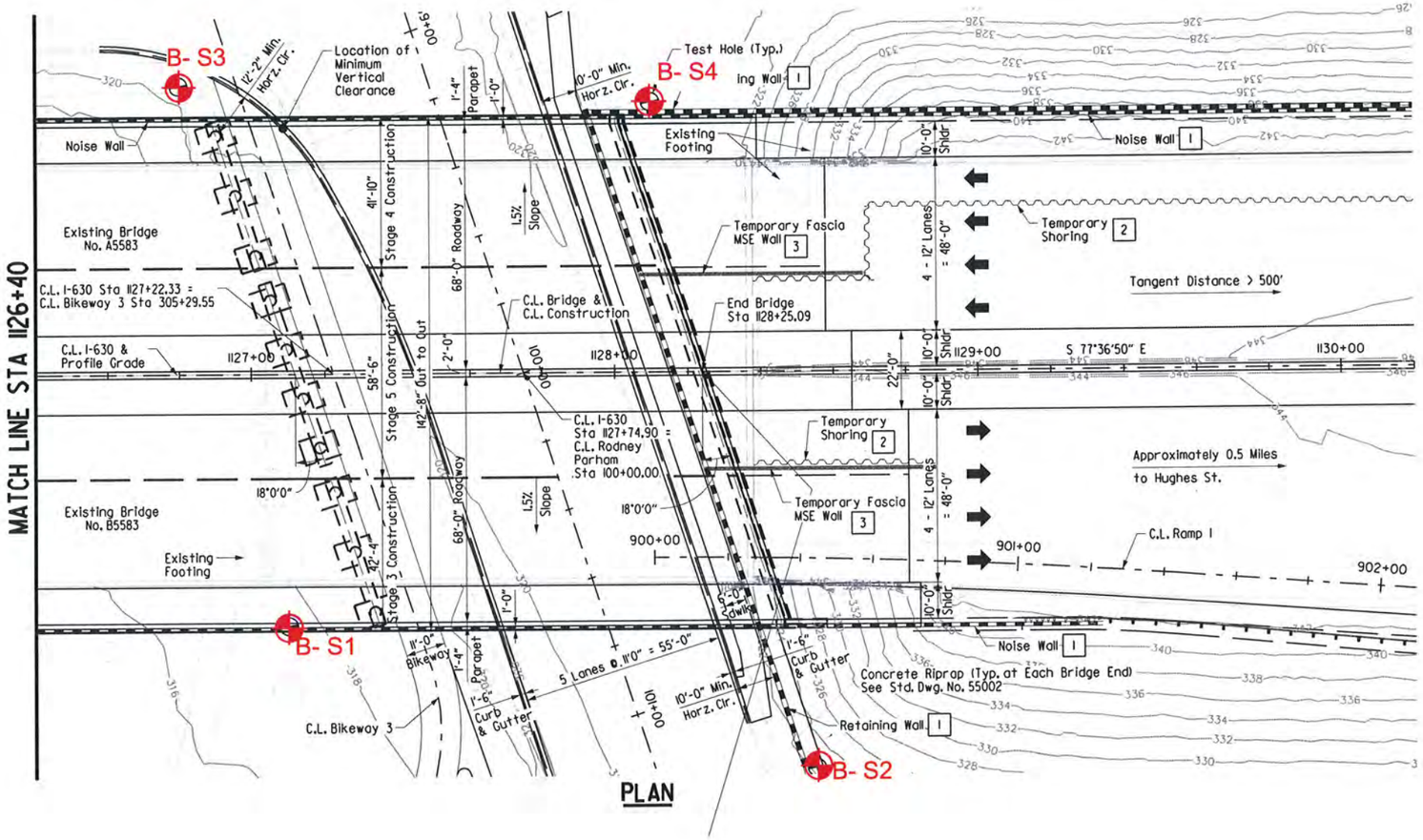
60% SUBMITTAL

PRELIMINARY
FOR REVIEW ONLY

STEPHEN T. SMILEY, P.E., 13072

DECEMBER-2014

DATE REVISED	DAT FILM



Note: Base drawing provided by Bridgefarmer & Associates, Inc.

20 ft 0 20 40 ft

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

Plan of Borings
I-630 over Rodney Parham Road
AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 5, 2015	Plate 2b

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

LOG OF BORING NO. S1

CA0608: I-630 over Rodney Parham Road
Little Rock, Arkansas

TYPE: Auger to 5 ft /Wash

LOCATION: Sta 1127+10, 70 ft Rt

SURF. EL: 318±

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	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 %	% Recovery	% RQD
			Very stiff brown clayey silt, sandy w/crushed stone and some sandstone cobbles (fill)	27 28 50/2"			22		
5			Moderately hard to hard reddish tan and tan weathered fine-grained sandstone w/fine sandy clay seams	30/0"					
10			Moderately hard tan and dark gray weathered shale	50/8"					
15									
20			Moderately hard to hard dark gray shale and carbonaceous shale w/medium close sandstone partings and quartz veins, dip near verticle	50/2" 25/0" 25/0"					
25									
30			- high angle shear at 30.5 - 31.2 ft - with medium close sandstone inclusions at 31.5 and 33 - 34 ft			q_u = 740 psi, TUW = 163 pcf		100	77
35			- with mudstone seam at 34.2 - 34.3 ft - quartz vein at 36.1 ft - with close sandstone inclusions and partings below 38 ft			q_u = 950 psi, TUW = 166 pcf q_u = 1260 psi, TUW = 166 pcf		93	75
40			- with mudstone inclusions below 38 ft						
45									

COMPLETION DEPTH: 40.0 ft
DATE: 7-11-14

DEPTH TO WATER
IN BORING: Dry to 11 ft

DATE: 7/11/2014

RECRQDN200-2 14-030_I-630 OVER RODNEY PARHAM.GPJ 2-6-15

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

LOG OF BORING NO. S2

CA0608: I-630 over Rodney Parham Road
Little Rock, Arkansas

TYPE: Auger to 7.5 ft /Wash

LOCATION: Sta 1128+55, 110 ft Rt

SURF. EL: 322±

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	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 %	% Recovery	% RQD
			Stiff brown silty clay w/some shale fragments (fill)	16			73		
			- very stiff with quartz fragments below 2 ft	38					
5			- with numerous sandstone and shale fragments below 4 ft	50/10"					
				35					
			- auger refusal on sandstone at 7.5 ft	50/1"					
10			Moderately hard to hard tan and gray weathered fine-grained sandstone w/fine sandy clay seams						
			Moderately hard tan, gray and dark gray weathered shale	50/6"					
15									
				50/2"					
20									
				50/1"					
25									
			Moderately hard to hard dark gray shale w/medium close sandstone partings, dip ±80°	25/0"					
30									
				25/0"					
35						q_u = 1190 psi, TUW = 167 pcf q_u = 940 psi, TUW = 168 pcf		88	88
			- with very close sandstone partings below 37 ft - sandstone inclusions at 38 ft						
40			- high angle shear at 40 ft						
			- high angle shear at 41.6 ft			q_u = 1130 psi, TUW = 168 pcf q_u = 750 psi, TUW = 169 pcf		85	85
45									

COMPLETION DEPTH: 45.0 ft
DATE: 7-10-14

DEPTH TO WATER
IN BORING: Dry to 7.5 ft

DATE: 7/10/2014

RECRQDN200-2 14-030_I-630 OVER RODNEY PARHAM.GPJ 2-6-15

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

LOG OF BORING NO. S3

CA0608: I-630 over Rodney Parham Road
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 1126+80, 80 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 320±	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 %
			Loose reddish brown silt w/sandstone fragments to cobble size (fill)	21			33
				10			
5			Stiff tan, reddish tan and gray silty clay w/shale fragments (fill) - very stiff with sandstone fragments below 4 ft	50/6"			
				50/10"			
10			Low hardness tan and gray highly weathered shale w/silty clay seams	31			
			- moderately hard below 13 ft	50/3"			
15							
20			Moderately hard dark gray shale w/medium close sandstone partings and seams	50/2"			
				30/0"			
25			- with close quartz veins from 24 to 26 ft				
				30/0"			
30							
				30/0"			
35							
				30/0"			
40							
				30/0"			
45			- with close quartz veins below 45 ft				
				30/0"			
50							

COMPLETION DEPTH: 50.0 ft
DATE: 8-18-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 8/18/2014

LGBNEW 14-030_I-630 OVER RODNEY PARHAM.GPJ 2-6-15

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

LOG OF BORING NO. S4

CA0608: I-630 over Rodney Parham Road
Little Rock, Arkansas

TYPE: Auger to 7 ft /Wash

LOCATION: Sta 1128+10, 75 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 321±	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 %
			Dense brown fine sandy silt w/sandstone and quartz fragments	50/7"			
			Firm to stiff reddish brown silty clay w/sandstone fragments and clayey silt pockets (fill)	23			64
5				33			49
			Very stiff reddish tan and tan silty clay w/shale fragments and occasional sandstone fragments	46			
			Moderately hard brown weathered fine-grained sandstone w/silty clay seams and quartz veins	25/0"			
10							
				25/0"			
15							
			Low hardness light brown weathered shale				
			Moderately hard dark gray shale, slightly weathered	50/4"			
20							
				50/3"			
25							
			Moderately hard to hard dark gray shale	25/0"			
30							
			- with very close sandstone seams from 33.5 - 34 ft	25/0"			
35							
				25/0"			
40							
				25/0"			
45							
				25/0"			
50							

COMPLETION DEPTH: 50.0 ft
DATE: 8-14-14

DEPTH TO WATER
IN BORING: Dry to 7 ft

DATE: 8/14/2014

LGBNEW 14-030_I-630 OVER RODNEY PARHAM.GPJ 2-6-15

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

LOG OF BORING NO. W3

CA0608: Retaining Walls - I-630 Widening
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 1124+10, 75 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 315±	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 %
			Very stiff brown and tan clayey silt w/sandstone fragments, dry (fill)	50/7"			
			Stiff tan silty clay w/ferrous nodules and stains	11			76
5			Very stiff tan and gray fine sandy clay w/ferrous nodules and sandstone fragments	35			50
			Medium dense tan clayey fine sand w/some fine to coarse gravel				
			Medium dense gray and tan clayey fine to coarse gravel - water at 7.5 ft	21			32
10			Low hardness to moderately hard tan and dark gray weathered shale	50/6"			
			Moderately hard dark gray shale	25/0"			
15							
20							
25							

COMPLETION DEPTH: 20.0 ft
DATE: 9-17-14

DEPTH TO WATER IN BORING: 7.5 ft

DATE: 9/17/2014

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

LOG OF BORING NO. W4

CA0608: Retaining Walls - I-630 Widening
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 1125+55, 70 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 315±	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 %
			Stiff dark brown silty clay w/some sandstone fragments and some crushed limestone, dry (fill)	23			49
				14			
5			Very stiff tan and brown fine sandy clay, silty w/fine to coarse gravel and crushed stone (fill)	27			24
				29			
10			- water at 9 ft	25			
			Moderately hard brown and dark gray weathered shale				
15				50/8"			
				50/4"			
20			NOTE: Water at 6.8 ft at 1 hour.				
25							

COMPLETION DEPTH: 19.0 ft
DATE: 7-7-14

DEPTH TO WATER IN BORING: 9 ft

DATE: 7/7/2014

LGBNEW 14-030_RETAINING WALLS_I-630.GPJ 3-2-15

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

LOG OF BORING NO. W5

CA0608: Retaining Walls - I-630 Widening
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 1125+25, 90 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 320±	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 %
			6 inches: Brown fine sandy silt w/some organics (fill)				
			Stiff brown silty clay w/shale and quartz fragments (fill)	50/9"			
			- with fewer quartz fragments below 2 ft	17			
5			- very stiff from 4 to 6 ft	30			60
			- stiff with quartz and sandstone fragments below 6 ft	11			
			- brown, moist below 8 ft	17			
10							
			Moderately hard red, tan and dark gray highly weathered shale	50/7"			
15							
			Moderately hard tan and dark gray weathered shale	50/2"			
20							
25							

COMPLETION DEPTH: 20.0 ft
DATE: 6-24-14

DEPTH TO WATER IN BORING: Dry

DATE: 6/24/2014

LGBNEW 14-030_RETAINING WALLS_I-630.GPJ 3-2-15

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

LOG OF BORING NO. W6

CA0608: Retaining Walls - I-630 Widening
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 1124+00, 80 ft Lt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 320±	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 fine sandy silt w/some organics	11			
			Very stiff reddish brown and brown silty clay w/some shale and sandstone fragments (fill)	30			
5			- stiff with occasional silt pockets from 4 - 6 ft	18			
			- firm, light brown silty clay with trace organics and fine quartz fragments from 6 to 8 ft	7			75
10			- very stiff with more shale fragments below 8 ft	50/10"			
			Moderately hard reddish tan and dark gray weathered shale	50/2"			
			- auger refusal at 12 ft				
15							
20							
25							

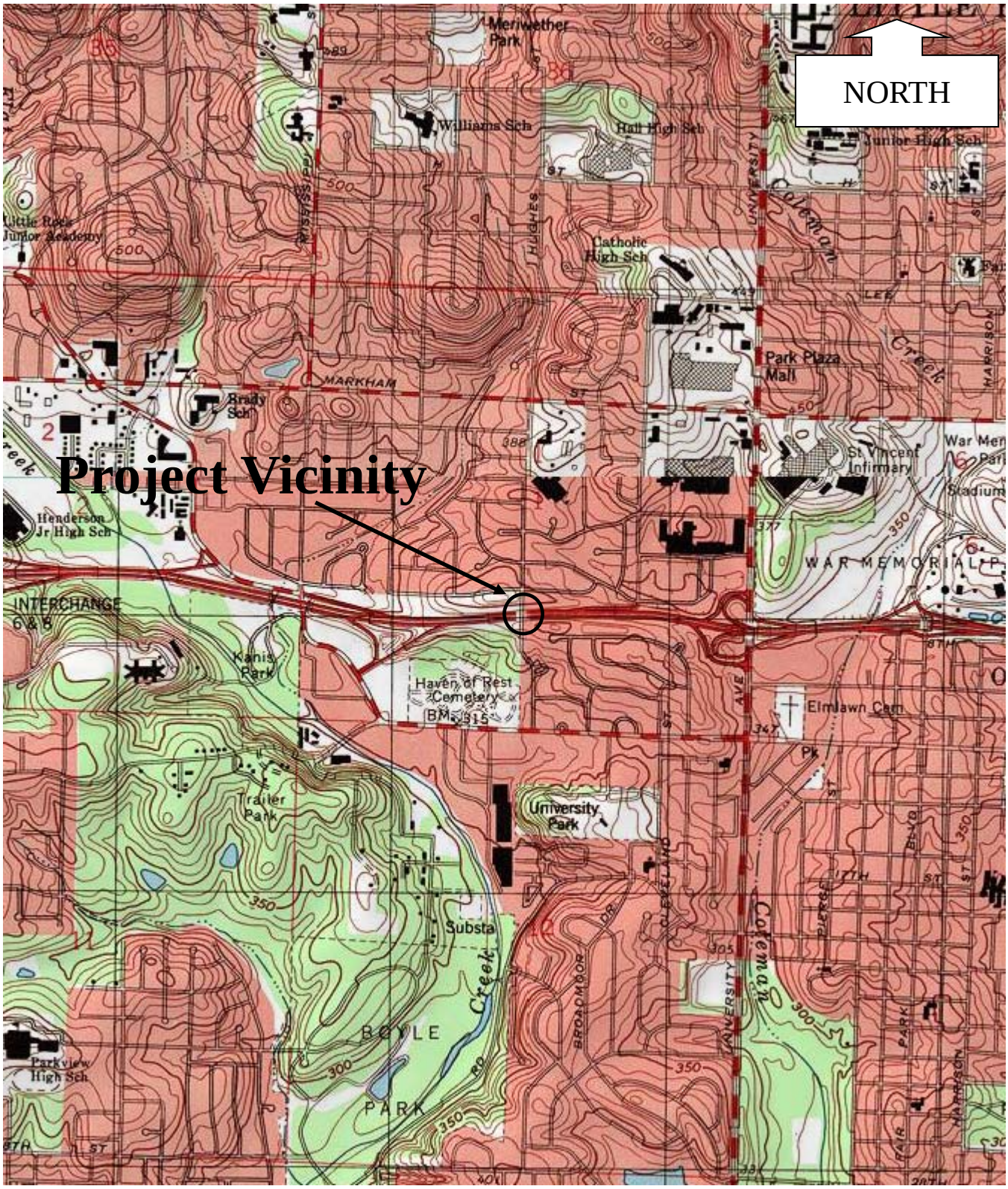
COMPLETION DEPTH: 12.0 ft
DATE: 6-24-14

DEPTH TO WATER IN BORING: Dry

DATE: 6/24/2014

LGBNEW 14-030_RETAINING WALLS_I-630.GPJ 3-2-15

ATTACHMENT 4



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

SITE VICINITY MAP
Hughes Street over I-630
AHTD CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

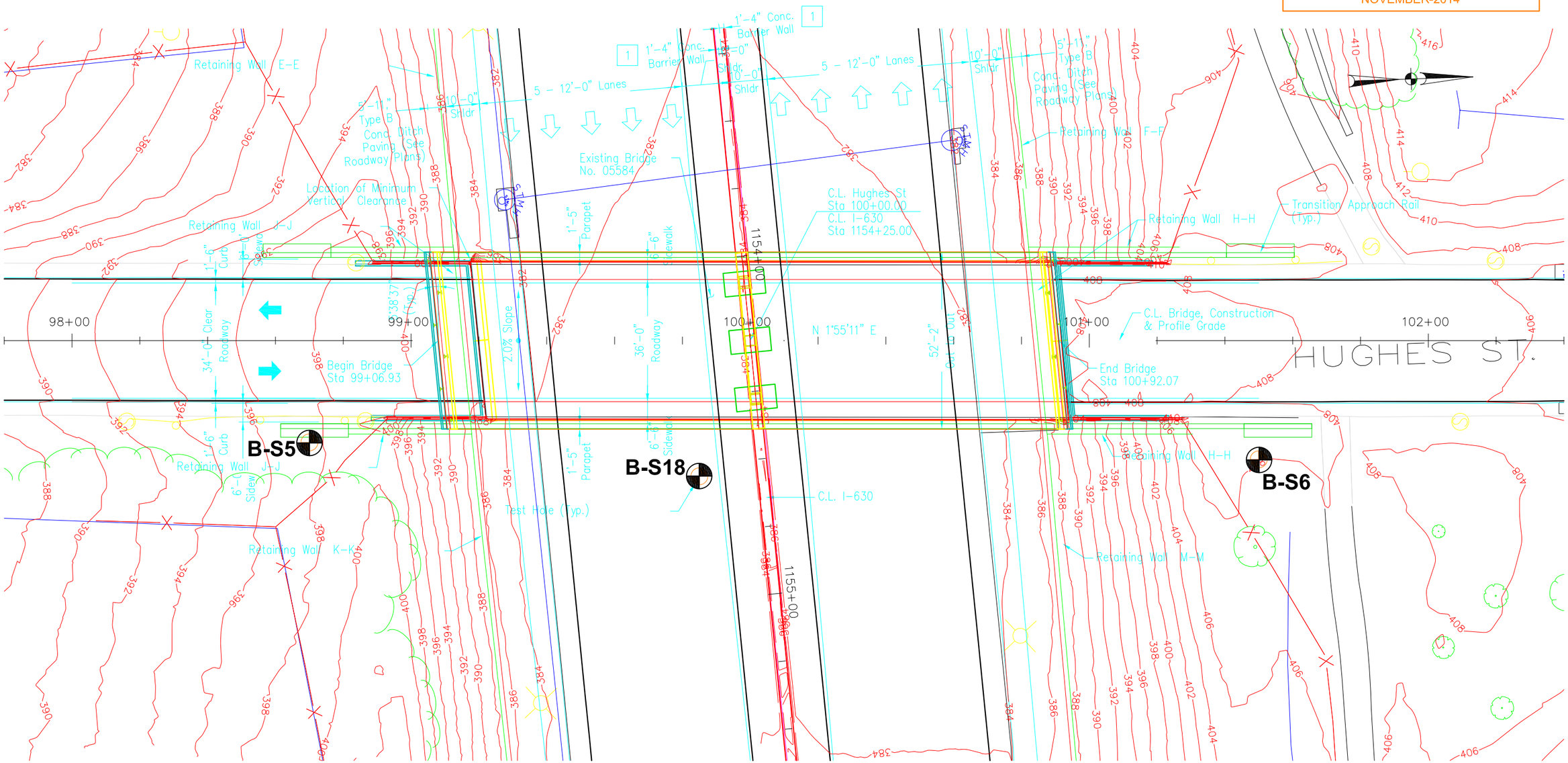
Job No. 14-030

Plate 1

PRELIMINARY
FOR REVIEW ONLY

TZU-JUI TANG, P.E., 12896

NOVEMBER-2014



PLAN

Note: Stations and Elevations are shown along C.L. Bridge.

Total Length of Bridge = 185'-5/8"

183'-0" Continuous Composite W-Beam Unit (92' - 91')

Note: Base drawing provided by Bridgefarmer & Associates, Inc.

15 ft 0 15 30 ft

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

Plan of Borings
Hughes Street over I-630
AHTD Job CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
December 19, 2014	Plate 2

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

LOG OF BORING NO. S5

CA0608: Hughes Street over I-630
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 98+70, 30 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURF. EL: 398±	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 fine sandy silt w/organics and sandstone fragments (fill)	24 33			48
			Very stiff reddish tan and tan silty clay w/sandstone and shale fragments and some cobbles (fill)				
5				50/6"			
			Very stiff reddish tan and tan silty clay w/sandstone fragments				
			Low hardness to moderately hard gray, tan and maroon highly weathered shale w/silty clay laminations and ferrous stains	38			
10			- low hardness, gray and tan with more silty clay seams below 6 ft	50/4"			
			Moderately hard gray and tan weathered shale w/ferrous stains				
15			- gray, tan and maroon below 10 ft	50/5"			
			- moderately hard to hard below 18 ft	25/0"			
20							
				25/0"			
25							
			Hard gray fine-grained sandstone w/very close quartz veins				
			Moderately hard to hard gray and tan weathered shale				
30			- with interbedded sandstone seams below 29 ft				
35			- with fewer sandstone seams below 36 ft				
40							

COMPLETION DEPTH: 65.0 ft
DATE: 7-8-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 7/8/2014

LGBNEW 14-030_HUGHES OVER I-630.GPJ 11-12-14

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

LOG OF BORING NO. S5

CA0608: Hughes Street over I-630
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 98+70, 30 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 %
			- with very close sandstone seams and quartz veins below 46 ft				
50							
			Moderately hard dark gray shale w/very close, very thin fine-grained sandstone partings				
55							
60							
65							
70							
75							
80							
85							

COMPLETION DEPTH: 65.0 ft
DATE: 7-8-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 7/8/2014

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

LOG OF BORING NO. S6

CA0608: Hughes Street over I-630
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 101+50, 35 ft Rt

SURF. EL: 408±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT
	Medium dense brown fine sandy silt w/fine to coarse gravel and organics	11
	Low hardness reddish tan, gray and tan highly weathered shale w/silty clay seams and ferrous stains, approx dip ~ 70° NE	35
5		50/11"
	Low hardness to moderately hard tan, gray and reddish tan weathered shale w/ferrous stains, approx dip ~ 70° NE	50/8"
	- moderately hard below 6 ft	50/9"
10	- auger refusal at 8.5 ft	
		50/6"
15	- maroon, gray and tan below 15 ft	
		50/5"
20		
		50/4"
25		
		50/4"
30		
35		50/4"
	- with very close, very thin sandstone partings below 38 ft	50/1"
40		
		25/0"

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 %

COMPLETION DEPTH: 80.0 ft
DATE: 6-30-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 6/30/2014

LGBNEW 14-030_HUGHES OVER I-630.GPJ 11-12-14

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

LOG OF BORING NO. S6

CA0608: Hughes Street over I-630
Little Rock, Arkansas

TYPE: Auger to 10 ft /Wash

LOCATION: Sta 101+50, 35 ft Rt

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (continued)	BLOWS PER FT	UNIT DRY WT LB/CU FT	COHESION, TON/SQ FT (0.2 0.4 0.6 0.8 1.0 1.2 1.4) / PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT (10 20 30 40 50 60 70)	- No. 200 %
				25/0"			
50							
				25/0"			
55							
60			- with more sandstone partings below 62 ft				
				25/0"			
65							
			Moderately hard dark gray shale w/close sandstone partings and seams				
70							
75							
80							
85							

COMPLETION DEPTH: 80.0 ft
DATE: 6-30-14

DEPTH TO WATER
IN BORING: Dry to 10 ft

DATE: 6/30/2014

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

LOG OF BORING NO. S18

CA0608: Hughes Street over I-630
Little Rock, Arkansas

TYPE: Auger

LOCATION: Sta 99+85, 40 ft Rt

SURF. EL: 383±

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WT LB/CU FT	- No. 200 %
	14 inches: Asphalt Concrete			
	6 inches: Crushed Stone Base			
	Very stiff reddish tan silty clay w/some shale and sandstone fragments (fill)	41		29
5		50/8"		
	Moderately hard reddish tan and gray highly weathered shale w/medium close sandstone seams and partings and close silty clay laminations and seams	50/6"		
10		50/9"		
15		50/9"		
20	- tan, gray and dark gray below 20 ft	50/7"		
25		50/6"		
30		50/5"		
35	Moderately hard tan and dark gray moderately weathered shale	50/3"		
40	Moderately hard to hard dark gray slightly weathered shale	25/0"		
45		25/0"		
50		25/0"		
55		25/0"		

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: 55.0 ft
DATE: 9-6-14

DEPTH TO WATER IN BORING: Dry to 10 ft

DATE: 9/6/2014

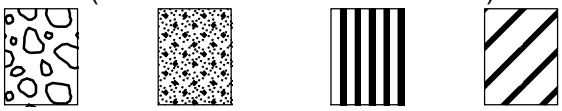
ATTACHMENT 5

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

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

(SHOWN IN SYMBOLS COLUMN)

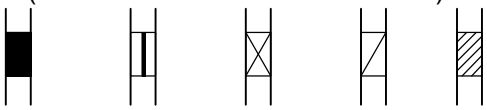


Gravel Sand Silt Clay

Predominant type shown heavy

SAMPLER TYPES

(SHOWN ON SAMPLES COLUMN)



Shelby Tube | Rock Core | Split Spoon | No Recovery | Cutting

TERMS DESCRIBING CONSISTENCY OR CONDITION

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

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

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

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

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

TERMS CHARACTERIZING SOIL STRUCTURE

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

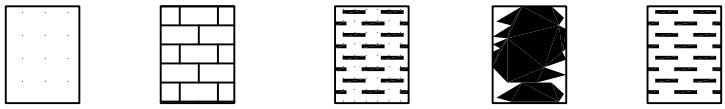
Terms used on this report for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in Technical Memorandum No.3-357, Waterways Experiment Station, March 1953

KEY 3-14-12

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

BORING LOG TERMS – ROCK

ROCK TYPES
(SHOWN IN SYMBOLS COLUMN)



Sandstone Limestone Siltstone Coal Shale

Joint Characteristics –

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

Bedding Characteristics –

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

Lithologic Characteristics –

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

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

Hardness–

Soft (S) – Reserved for plastic material alone.

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

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

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

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

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

Texture –

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

Structure –

Bedding
- Flat – 0° – 5°
- Gently Dipping – 5° – 35°
- Moderately Dipping – 55° – 85°
- Steeply Dipping – 55° – 85°

Fractures, scattered
- Open
- Cemented or Tight

Fractures, closely spaced
- Open
- Cemented or Tight

Brecciated (Sheared and Fragmented)
- Open
- Cemented or Tight

Joints
- Faulted
- Slickensides

Degree of Weathering –

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

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

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

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

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

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

Solution and Void Conditions –

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

Swelling Properties –

Nonswelling
Swelling

Slaking Properties –

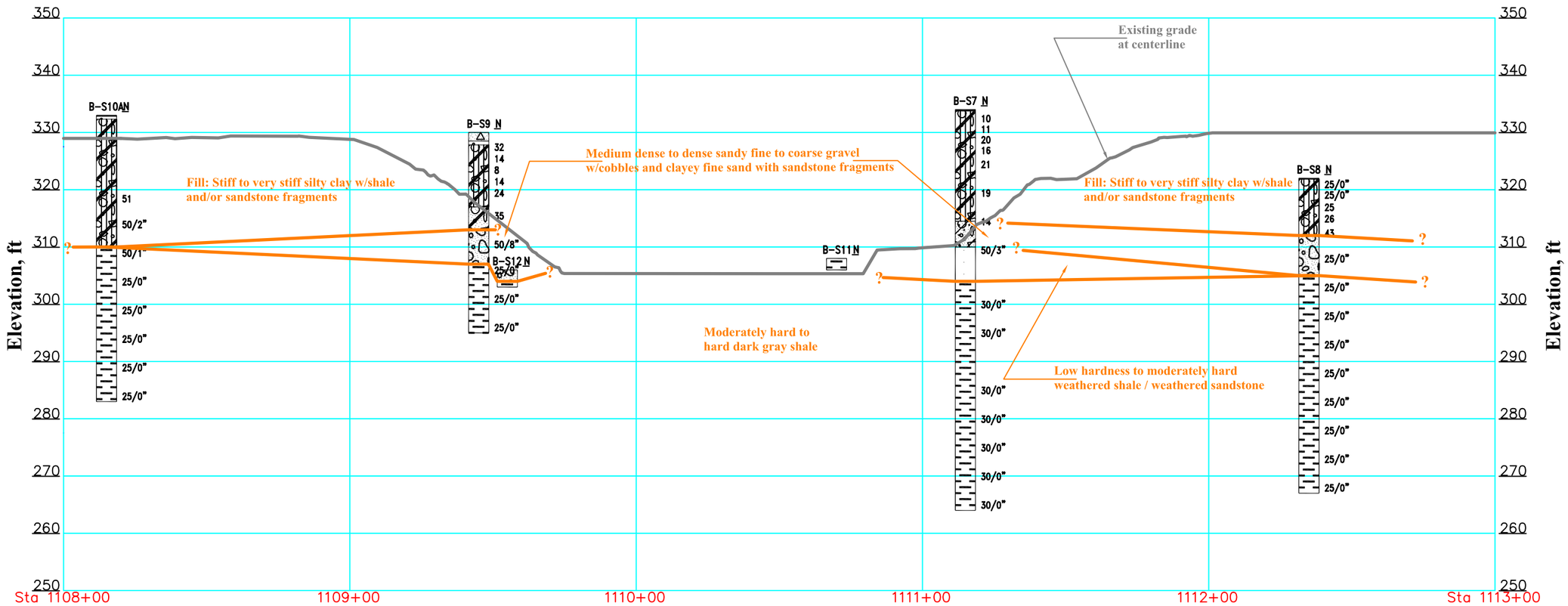
Nonslaking
Slakes slowly on exposure
Slakes readily on exposure

Rock Quality Designation (RQD) –

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

KEYROCK FHWA 3-2-12

ATTACHMENT 6



Notes: 1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface elevation at boring locations are approximate.

Horizontal Scale: 20 ft 0 20 40 ft

Vertical Scale: 10 ft 0 10 20 ft

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

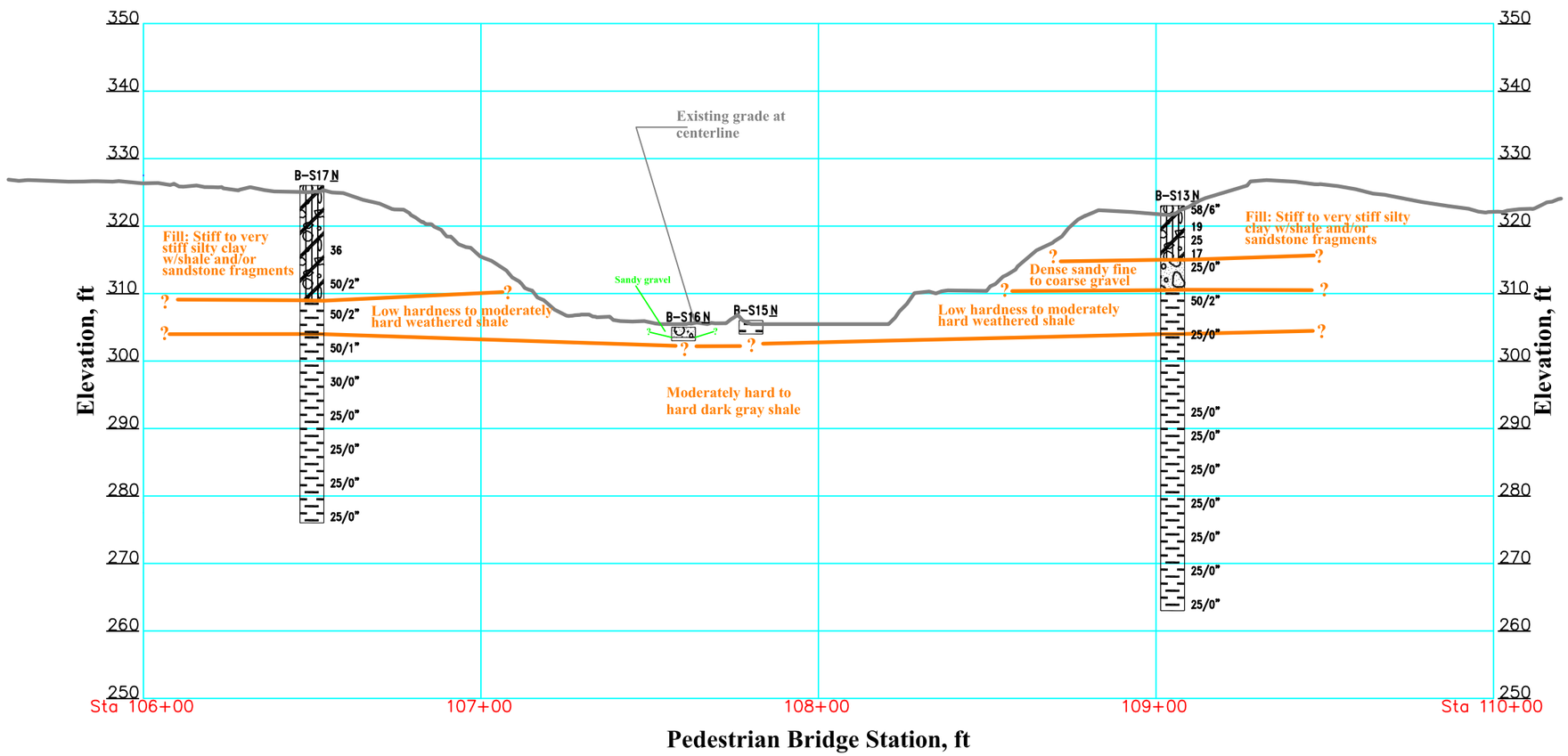
Generalized Subsurface Profile A-A'
I-630 Bridge over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030

Scale: As Shown

February 13, 2015

Plate



Notes: 1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface elevation at boring locations are approximate.

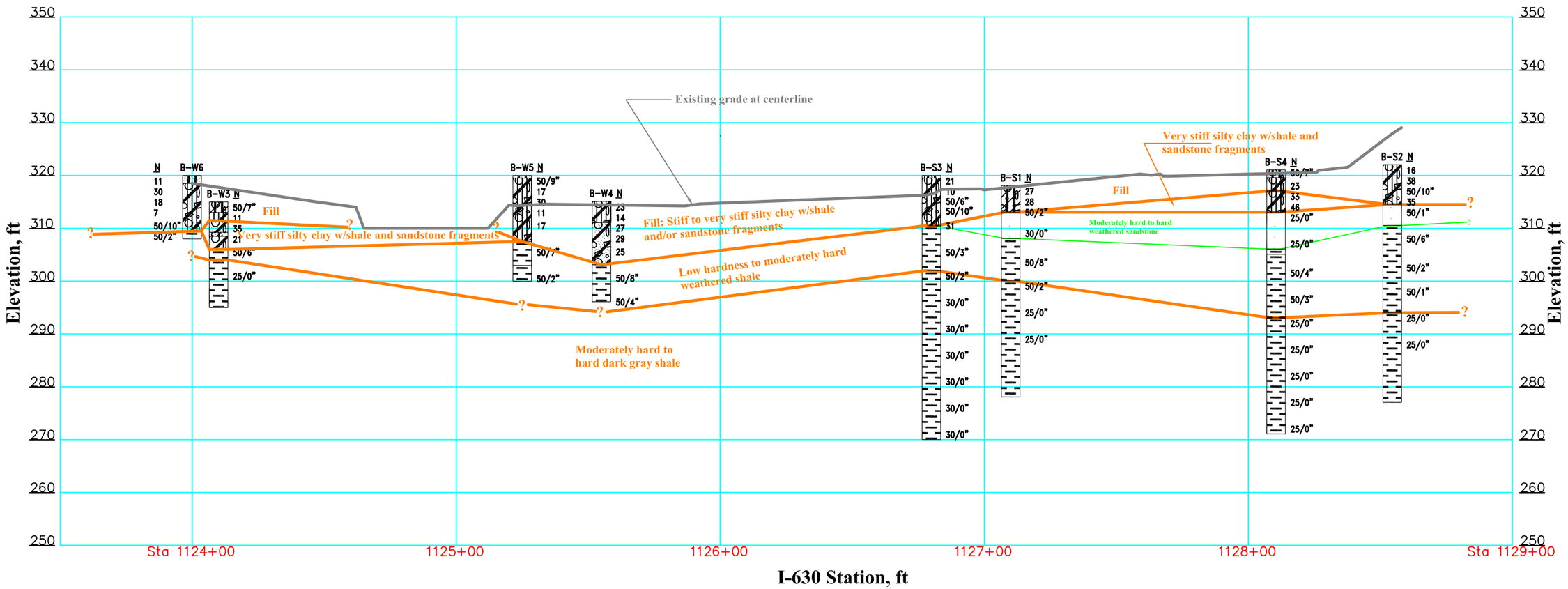
Horizontal Scale: 20 ft 0 20 40 ft

Vertical Scale: 10 ft 0 10 20 ft

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

Generalized Subsurface Profile B-B'
Pedestrian Bridge over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 13, 2015	Plate



Notes: 1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface elevation at boring locations are approximate.

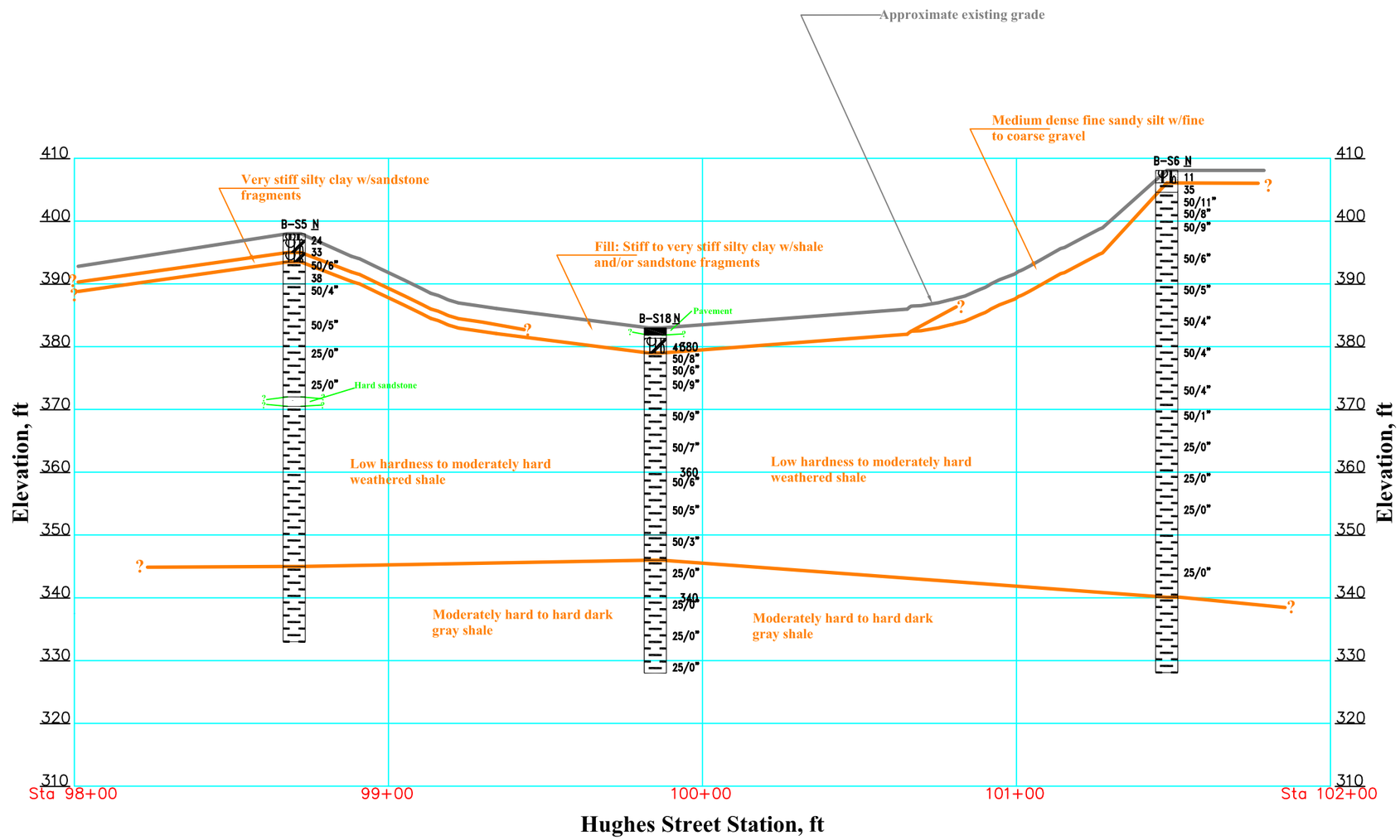
Horizontal Scale: 20 ft 0 20 40 ft

Vertical Scale: 10 ft 0 10 20 ft

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

Generalized Subsurface Profile C-C'
I-630 over Rodney Parham Road
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 13, 2015	Plate



Notes: 1. Subsurface conditions have been inferred between discrete boring locations. Actual conditions may vary.
2. Ground surface elevation at boring locations are approximate.

Horizontal Scale: 20 ft 0 20 40 ft

Vertical Scale: 10 ft 0 10 20 ft

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

Generalized Subsurface Profile D-D'
Hughes Street over I-630
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Pulaski County, Arkansas

GHBW Job No.: 14-030

Scale: As Shown

February 13, 2015

Plate 10

ATTACHMENT 7

Rock Core - Boring S1 (Sta 1127+10, 70' Rt), 30 to 40 ft
AHTD JOB CA0608
I-630 over Rodney Parham Road

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

Rock Core - Boring S2 (Sta 1128+55, 110' Rt), 35 to 45 ft
AHTD JOB CA0608
I-630 over Rodney Parham Road

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

ATTACHMENT 8

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-630 over Rock Creek - AHTD Job CA0608
LOCATION: Little Rock, Pulaski County, Arkansas
JOB NUMBER: 14-030

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		
S7	1-2	19	31	21	10	100	100	95	80	68	54	41	SC	A-4
S7	14-15	7	27	19	8	----	----	----	----	----	----	23	SC	A-2-4
S7	34-40	----	29	19	10	100	100	100	90	64	31	22	SC	A-2-4
S8	6.5-7.5	11	28	19	9	----	----	----	----	----	----	24	SC	A-2-4
S9	4-5	21	49	25	24	100	100	88	85	80	76	69	CL	A-7-6
S9	14-15	15	25	19	6	100	100	91	88	84	79	57	CL-ML	A-4

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: I-630 over Rock Creek - AHTD Job CA0608
LOCATION: Little Rock, Pulaski County, Arkansas
JOB NUMBER: 14-030

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		
S10	2.5-3.5	23	34	19	15	----	----	----	----	----	----	37	SC	A-6
S10A	1-3	16	44	21	23	----	----	----	----	----	----	60	SC	A-7-6
S10A	14-15	15	24	17	7	----	----	----	----	----	----	74	CL-ML	A-4
S10A	29-35	----	28	19	9	100	100	100	99	73	27	17	SC	A-2-4
S13	6.5-7.5	13	23	17	6	----	----	----	----	----	----	39	SC-SM	A-4
S13	28.5-29	----	29	20	9	100	100	100	88	70	38	27	SC	A-2-4
S17	7-7.5	15	34	15	19	----	----	----	----	----	----	67	CL	A-6

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

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: AHTD Job CA0608 - I-630 over Rodney Parham Road
LOCATION: Little Rock, Pulaski County, Arakansas
GHBW JOB NUMBER: 14-030

BORING NO.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING						UNIFIED CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	3/4 in.	3/8 in.	#4	#10	#40	#200		
S1	2.5-3.5	7	23	18	5	----	----	----	----	----	22	GC-GM	A-4
S2	0.5-1.5	18	27	19	8	----	----	----	----	----	73	CL	A-4
S3	0.5-1.5	12	34	23	8	----	----	----	----	----	33	SC	A-2-4
S4	2.5-3.5	16	38	18	20	----	----	----	----	----	64	CL	A-6
S4	4.5-5.5	16	38	24	14	----	----	----	----	----	49	SC	A-6
S4	18.5-19	11	35	22	13	----	----	----	----	----	----	Shale	
W3	2.5-3.5	17	30	20	10	----	----	----	----	----	76	CL	A-4
W3	4.5-5.5	14	21	17	4	----	----	----	----	----	50	CL-ML	A-4
W3	6.5-7.5	13	18	16	2	----	----	----	----	----	32	GM	A-2-4

SUMMARY OF CLASSIFICATION TEST RESULTS

PROJECT: AHTD Job CA0608 - I-630 over Rodney Parham Road
LOCATION: Little Rock, Pulaski County, Arakansas
GHBW JOB NUMBER: 14-030

BORING NO.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING						UNIFIED CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	3/4 in.	3/8 in.	#4	#10	#40	#200		
W4	0.5-1.5	11	25	18	7	100	87	80	75	68	49	SC-SM	A-4
W4	4.5-5.5	10	29	19	7	----	----	----	----	----	24	SC-SM	A-2-4
W4	14-14.7	7	28	20	8	----	----	----	----	----	----	Shale	
W5	4.5-5.5	13	33	19	14	----	----	----	----	----	60	CL	A-6
W6	6.5-7.5	19	29	18	11	----	----	----	----	----	75	CL	A-6

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

SUMMARY of ROCK STRENGTH TEST RESULTS

PROJECT: AHTD JOB CA0608 - I-630 over Rodney Parham Road
LOCATION: Little Rock, Pulasi County, Arkansas
GHBW JOB NO.: 14-030

Boring No.	Boring Location	Core Depth, ft	Rock Type	Total Unit Wt, lb/ft^3	Compressive Strength (ASTM D-7012), psi
S1	Sta 1127+10, 70' Rt	32-33	Shale	163	740
S1	Sta 1127+10, 70' Rt	37-38	Shale	166	950
S1	Sta 1127+10, 70' Rt	38-39	Shale	166	1260
S2	Sta 1128+55, 110' Rt	37-38	Shale	167	1190
S2	Sta 1128+55, 110' Rt	38-39	Shale	168	940
S2	Sta 1128+55, 110' Rt	42-43	Shale	168	1130
S2	Sta 1128+55, 110' Rt	43-44	Shale	169	750

SUMMARY OF CLASSIFICATION TEST RESULTS

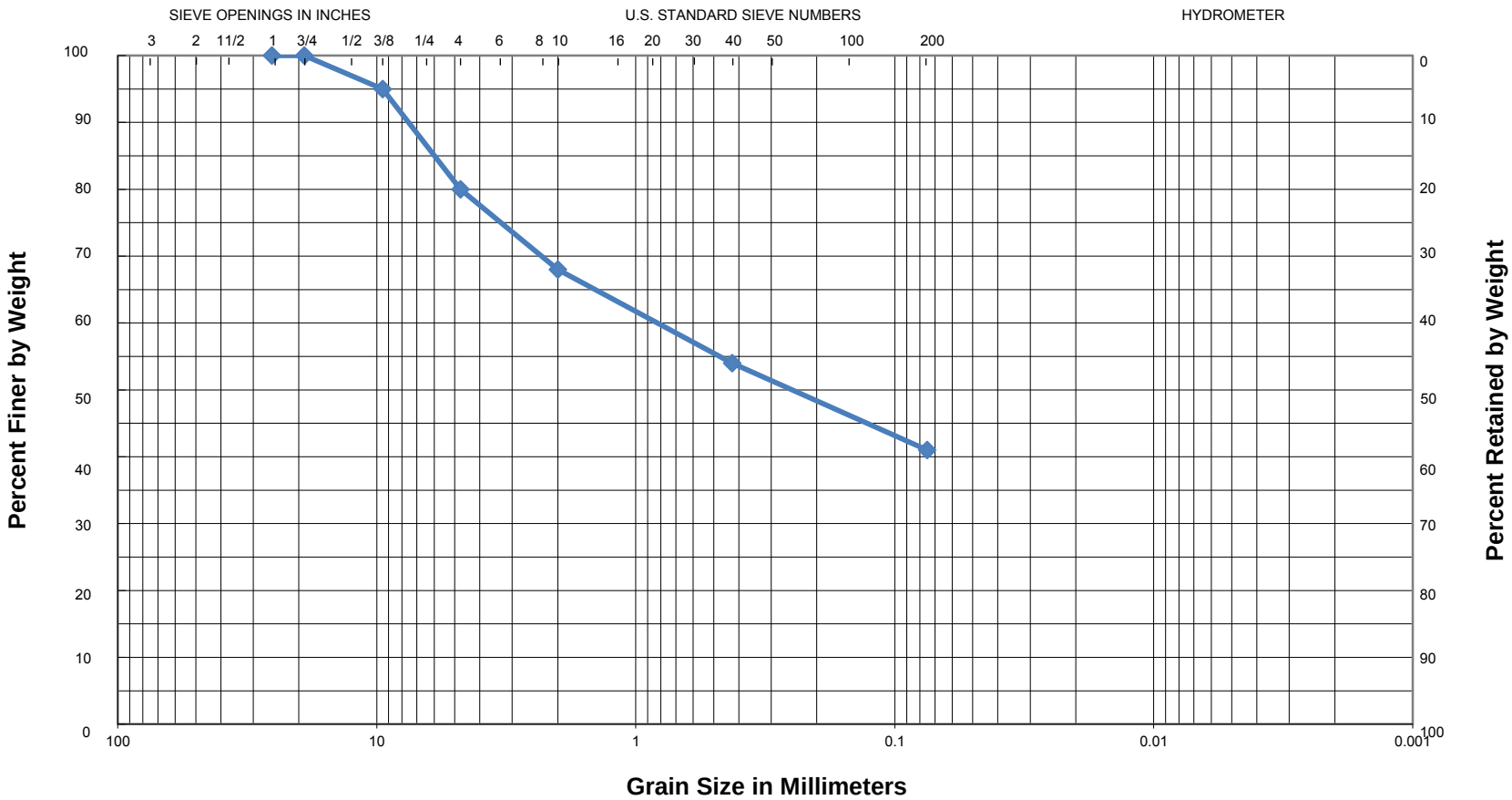
PROJECT: AHTD Job No. CA0608 - Hughes Street over I-630
LOCATION: Little Rock, Pulaski County, Arkansas
JOB NUMBER: 14-030

BORING NO.	SAMPLE DEPTH (ft)	WATER CONTENT (%)	ATTERBERG LIMITS			SIEVE ANALYSIS PERCENT PASSING						UNIFIED CLASS.	AASHTO CLASS.
			LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	3/4 in.	3/8 in.	#4	#10	#40	#200		
S5	2.5-3.5	10	33	20	13	100	90	79	60	52	48	SC	A-6
S5	4.5-5.5	14	42	23	19	----	----	----	----	----	----	Shale	
S6	2.5-3.5	14	39	26	13	----	----	----	----	----	----	Shale	
S6	8.5-9	12	38	23	15	----	----	----	----	----	----	Shale	
S18	2.5-3.5	11	31	22	9	----	----	----	----	----	29	SC	A-2-4
S18	6-6.5	9	30	21	9	----	----	----	----	----	----	Shale	

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

14-030

GRAIN SIZE CURVE



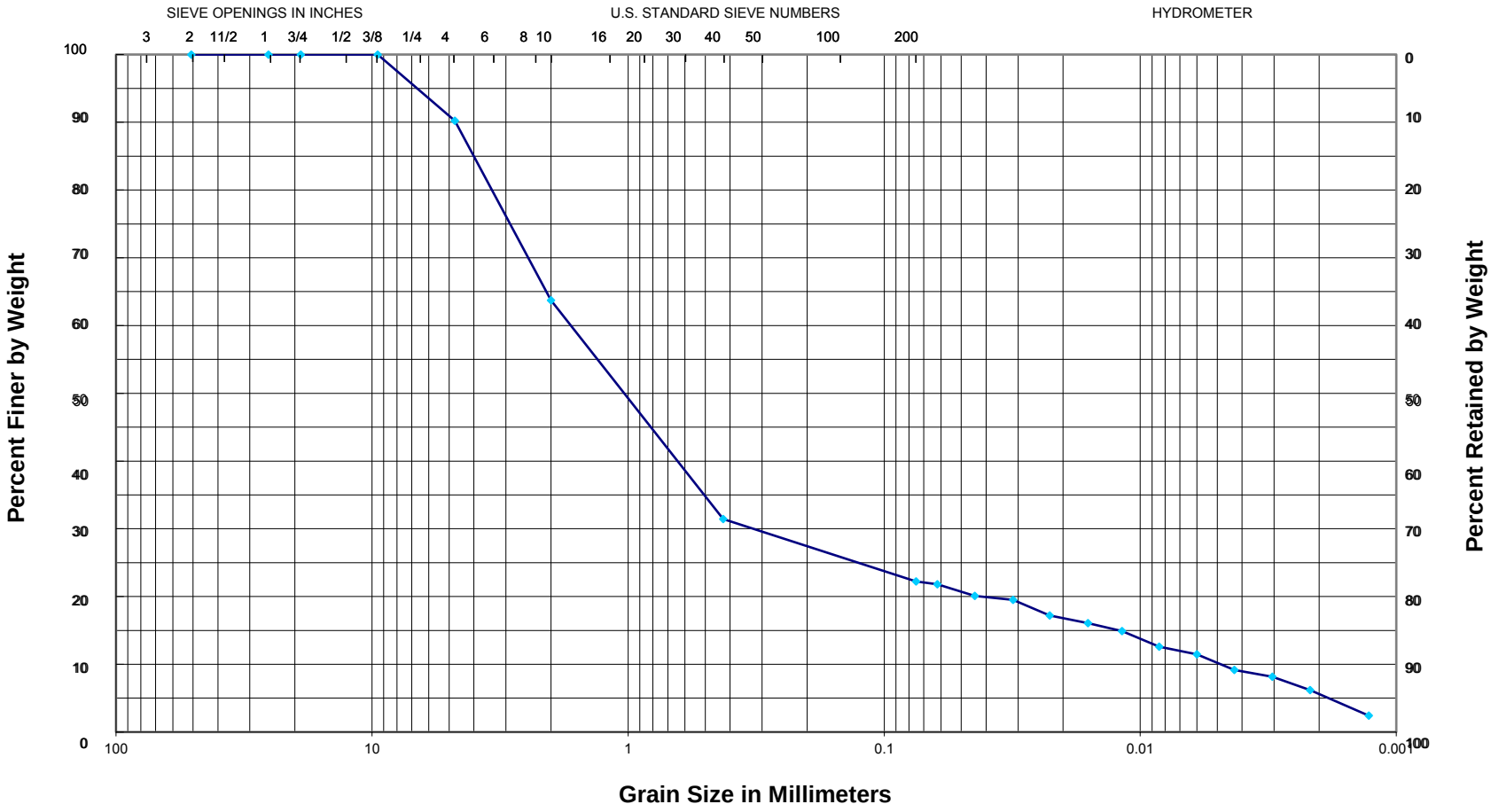
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring S7, 1-2 ft
Atterberg Limits: LL = 31, PL =21, PI = 10

Description: Reddish brown silty clay with shale and sandstone fragments
Classification: **USCS = SC; AASHTO = A-4**

14-030

GRAIN SIZE CURVE



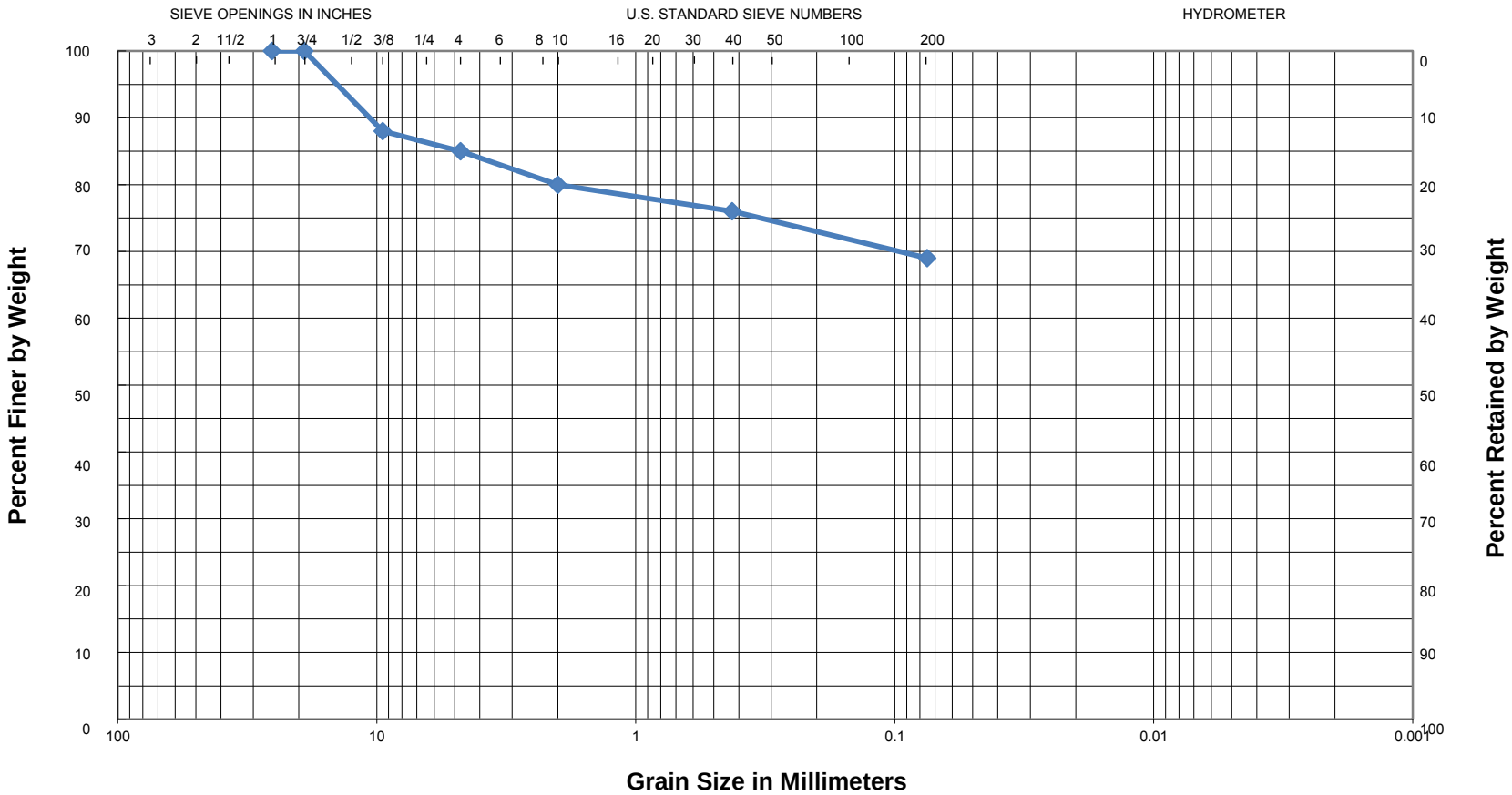
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring S7; 34-40 ft
Properties: G_s = 2.814; LL = 29, PI = 19, PI = 10

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

14-030

GRAIN SIZE CURVE



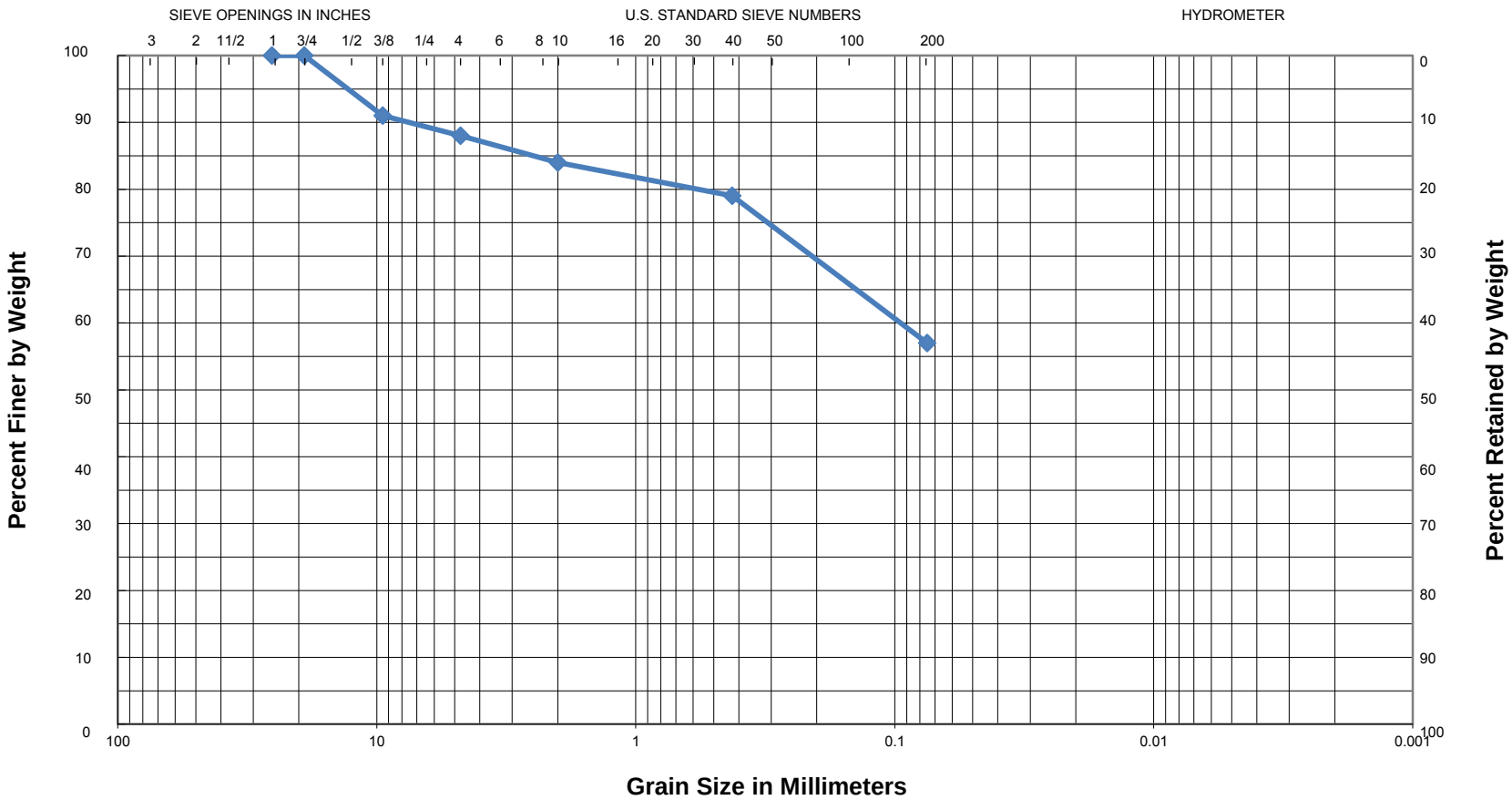
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring S9, 4-5 ft
Atterberg Limits: LL = 49, PL = 25, PI = 24

Description: Gray, tan, and reddish brown silty clay with fine to coarse gravel and shale fragments
Classification: **USCS = CL; AASHTO = A-7-6**

14-030

GRAIN SIZE CURVE



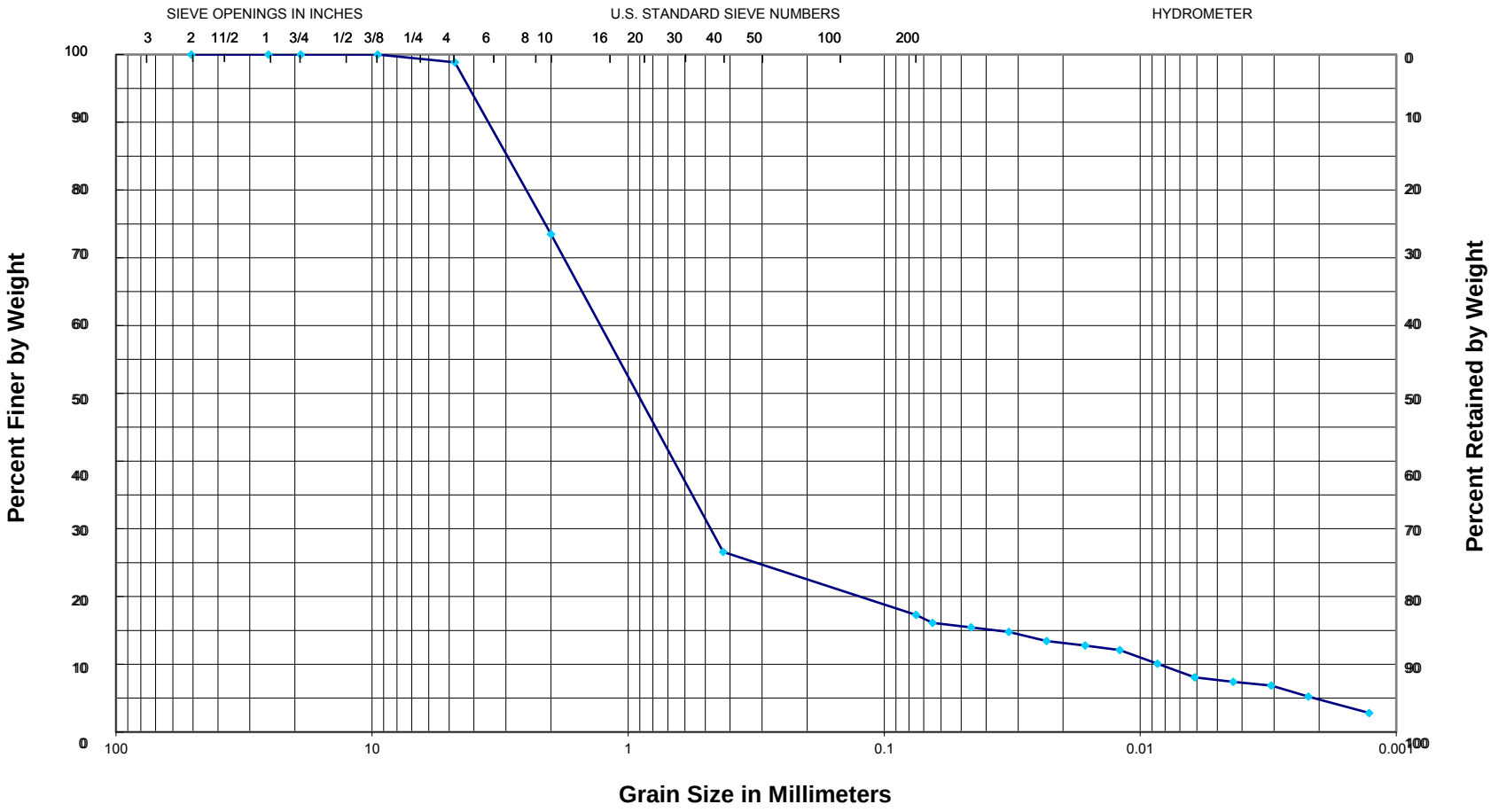
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring S9, 14-15 ft
Atterberg Limits: LL = 25, PL = 19, PI = 6

Description: Olive gray fine sandy clay w/crushed stone and fine gravel
Classification: **USCS = CL-ML; AASHTO = A-4**

14-030

GRAIN SIZE CURVE



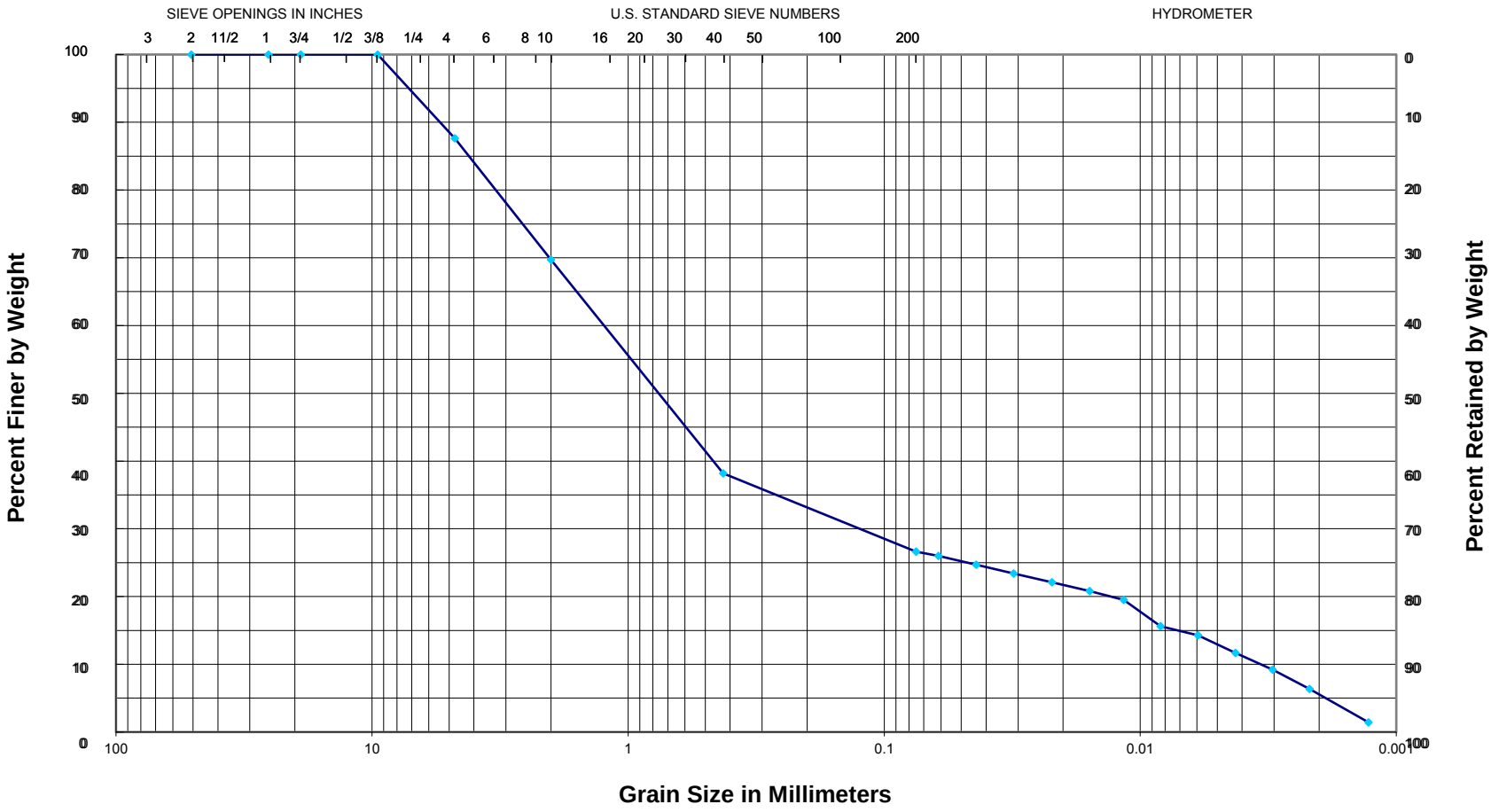
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring S10A; 29-35 ft
Properties: G_s = 2.818; LL = 28, PI = 19, PI = 9

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

14-030

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring S13; 28.5-29 ft
Properties: G_s = 2.825; LL = 29, PI = 20, PI = 9

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

14-030

GRAIN SIZE CURVE

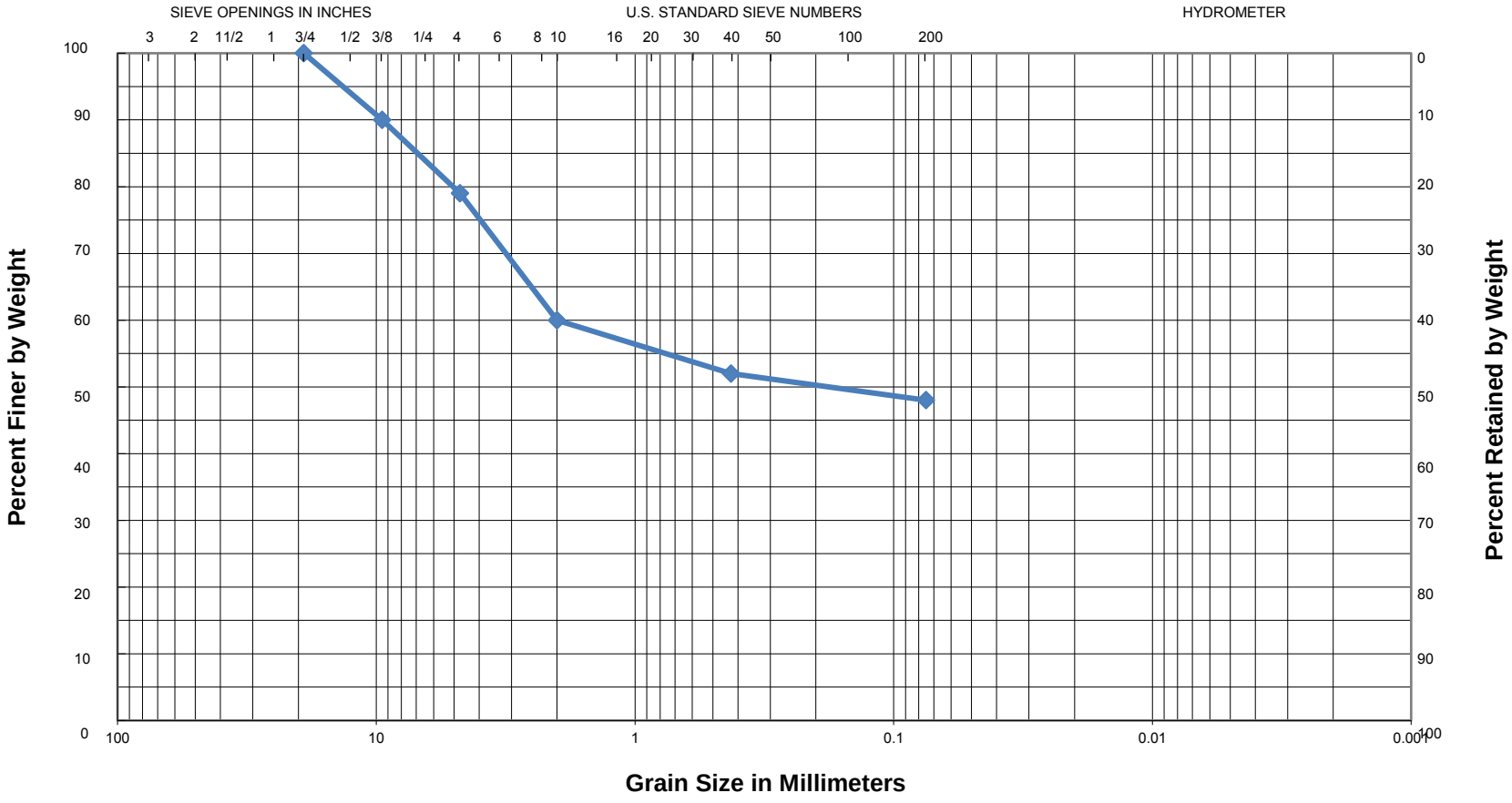
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring W4, 0.5-1.5 ft
Atterberg Limits: LL = 25, PL = 18, PI =7

Description: Dark brown silty clay with some sandstone fragments and crushed lim stone
Classification: **USCS = SC-SM; AASHTO = A-**4

14-030

GRAIN SIZE CURVE



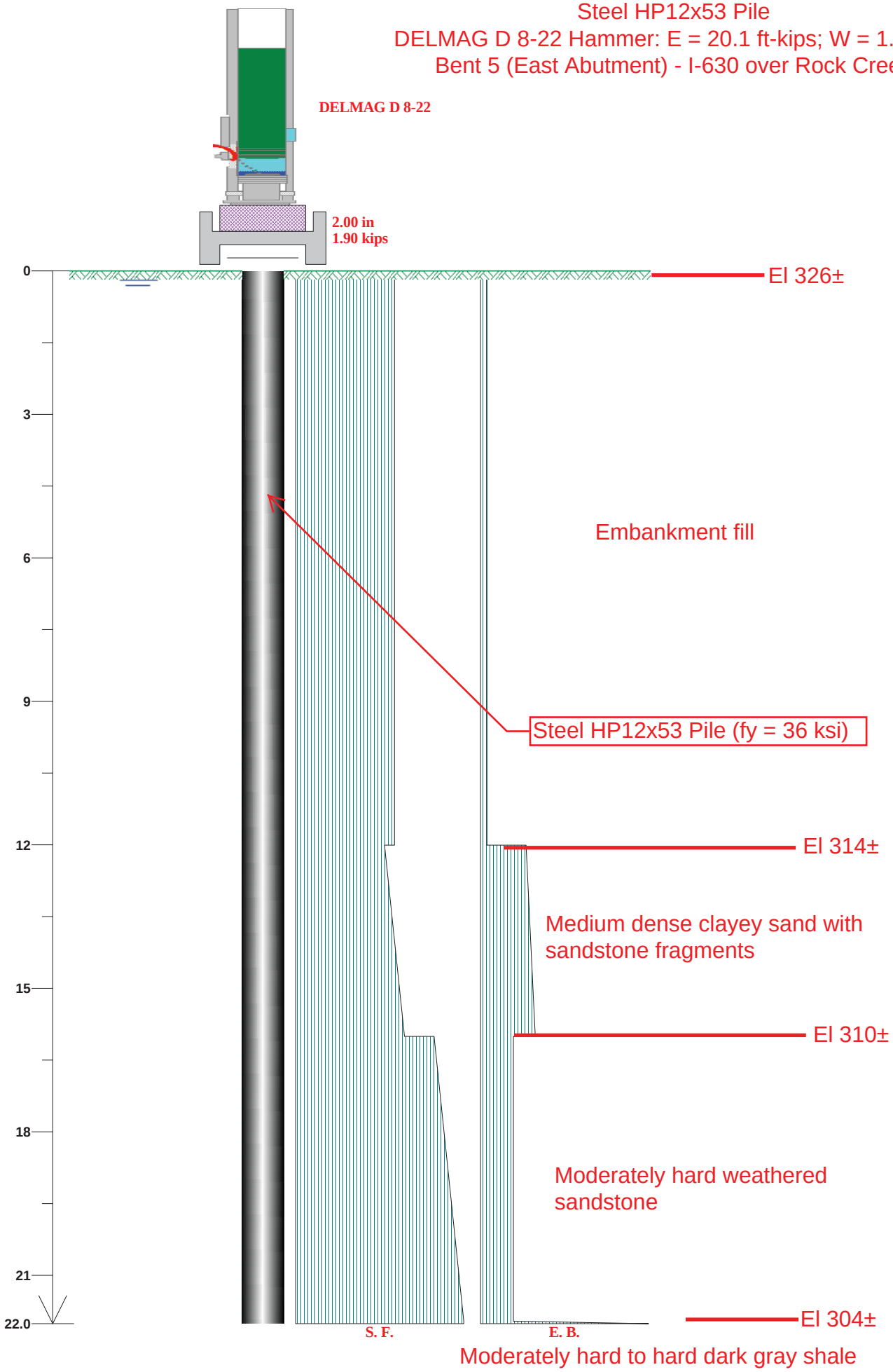
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

Sample: Boring S5, 2.5-3.5 ft
Atterberg Limits: LL = 33, PL = 20, PI =13

Description: Reddish tan and tan silty clay with sandstone & shale fragments
Classification: **USCS = SC; AASHTO = A-6**

ATTACHMENT 9

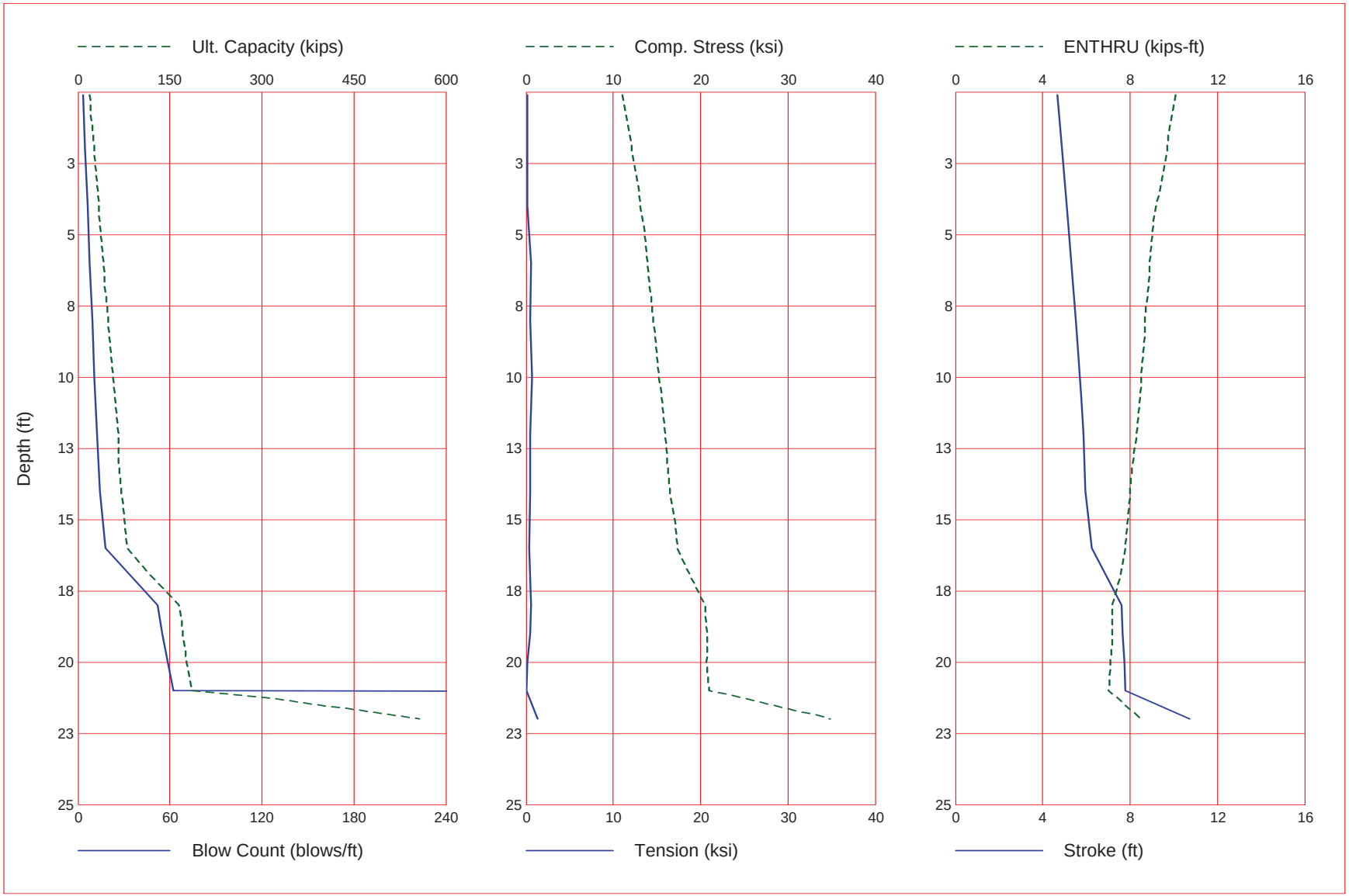
Model for Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek



Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_HP12x53_Rock Creek

Feb 09 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 09 2015
14-030_Driveability_HP12x53_Rock Creek GRLWEAP Version 2010

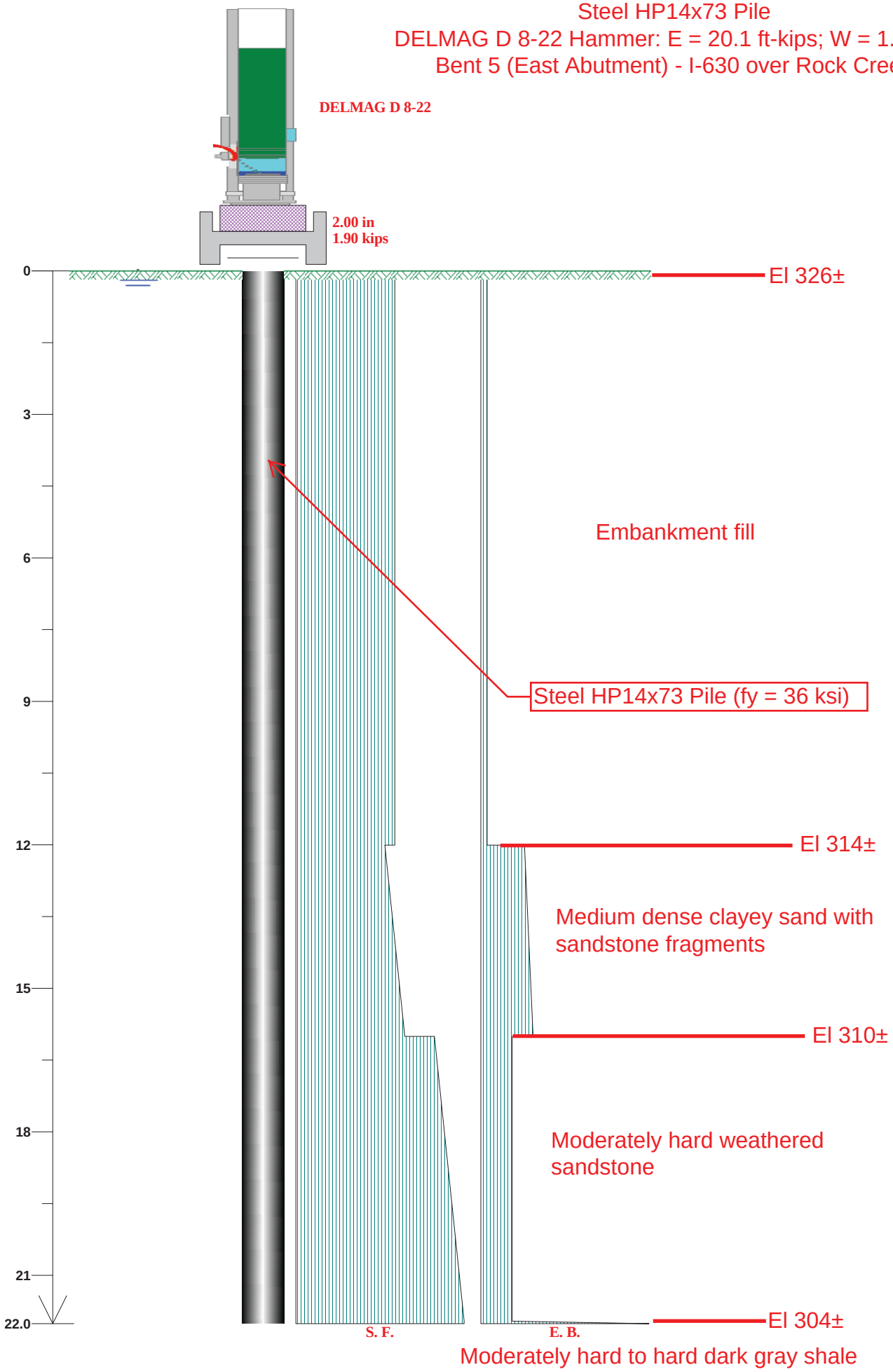
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	18.4	0.4	18.0	3.6	11.061	-0.164	4.66	10.1
2.0	25.9	7.9	18.0	4.8	12.127	-0.121	4.88	9.7
4.0	33.9	15.9	18.0	6.2	13.112	-0.131	5.11	9.2
6.0	41.8	23.8	18.0	7.7	13.877	-0.577	5.31	8.9
8.0	49.8	31.8	18.0	9.3	14.538	-0.457	5.51	8.7
10.0	57.7	39.7	18.0	10.8	15.184	-0.714	5.68	8.5
12.0	65.7	47.7	18.0	12.6	15.945	-0.512	5.86	8.3
14.0	70.0	55.2	14.8	14.4	16.495	-0.451	5.97	8.0
16.0	79.7	63.5	16.1	17.8	17.340	-0.390	6.24	7.8
18.0	165.1	75.1	90.0	51.7	20.518	-0.550	7.60	7.2
19.0	171.1	81.1	90.0	54.8	20.764	-0.451	7.67	7.2
20.0	177.4	87.4	90.0	58.4	20.686	-0.192	7.73	7.1
21.0	183.8	93.8	90.0	62.2	20.962	0.000	7.79	7.0
22.0	556.8	100.5	456.3	9999.0	34.868	-1.280	10.73	8.5

Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek

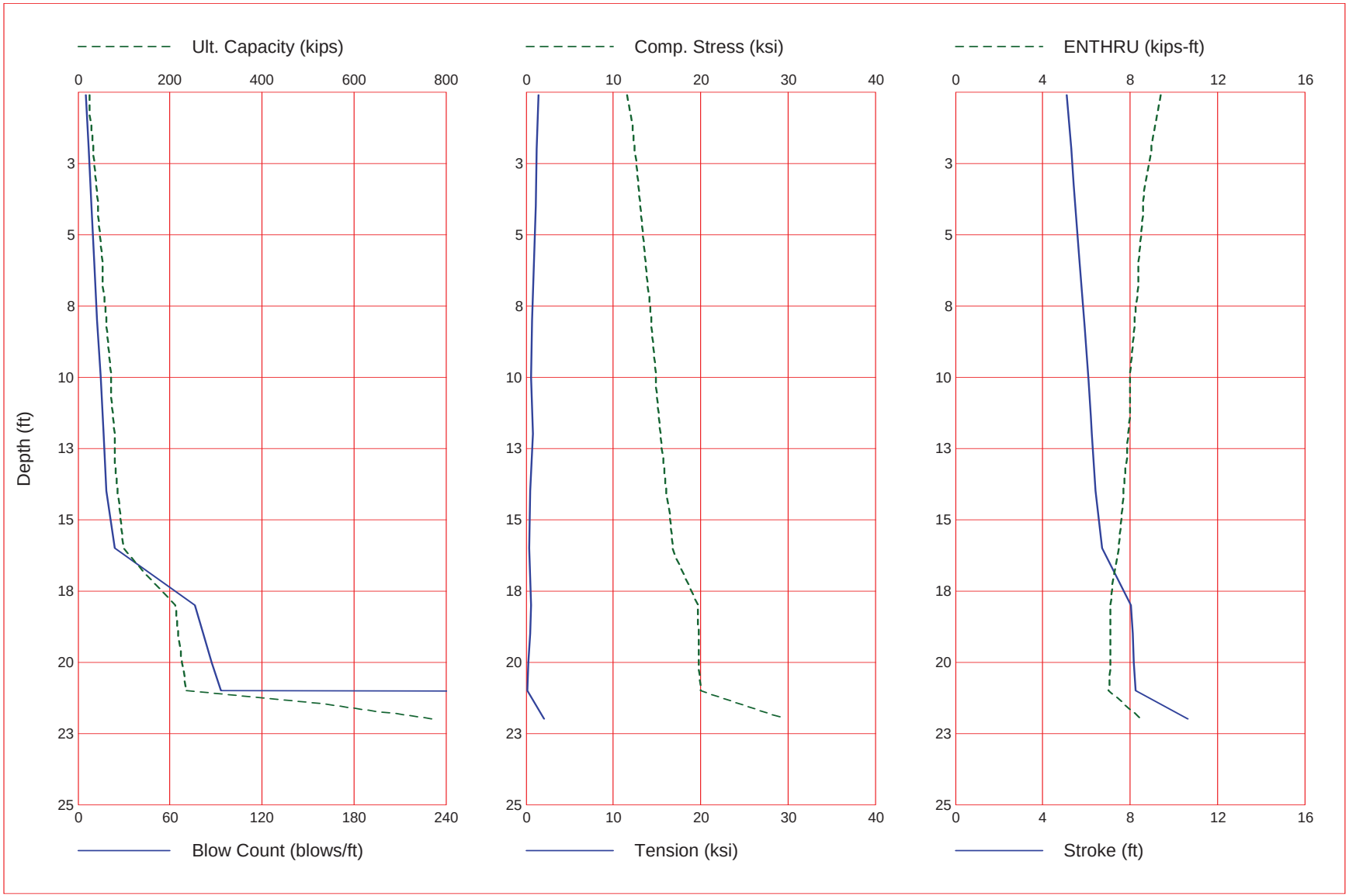
Model for Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek



Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_HP14x73_Rock Creek

Feb 09 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 09 2015
14-030_Driveability_HP14x73_Rock Creek 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	25.0	0.5	24.5	5.1	11.533	-1.412	5.10	9.4
2.0	33.9	9.4	24.5	6.7	12.440	-1.201	5.32	9.0
4.0	43.3	18.8	24.5	8.6	13.067	-1.140	5.49	8.6
6.0	52.7	28.2	24.5	10.4	13.717	-0.946	5.70	8.4
8.0	62.1	37.6	24.5	12.5	14.314	-0.740	5.91	8.2
10.0	71.5	47.0	24.5	14.6	14.843	-0.596	6.09	8.0
12.0	80.9	56.4	24.5	16.6	15.425	-0.842	6.23	7.9
14.0	85.8	65.3	20.4	18.6	16.006	-0.502	6.41	7.7
16.0	97.5	75.2	22.3	23.7	16.782	-0.376	6.71	7.5
18.0	211.3	88.8	122.5	76.3	19.655	-0.554	8.06	7.1
19.0	218.5	96.0	122.5	82.0	19.837	-0.529	8.12	7.1
20.0	225.9	103.4	122.5	87.2	19.813	-0.315	8.19	7.1
21.0	233.6	111.1	122.5	93.5	20.049	-0.128	8.26	7.0
22.0	769.6	118.9	650.7	9999.0	29.850	-2.073	10.63	8.5

Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 5 (East Abutment) - I-630 over Rock Creek

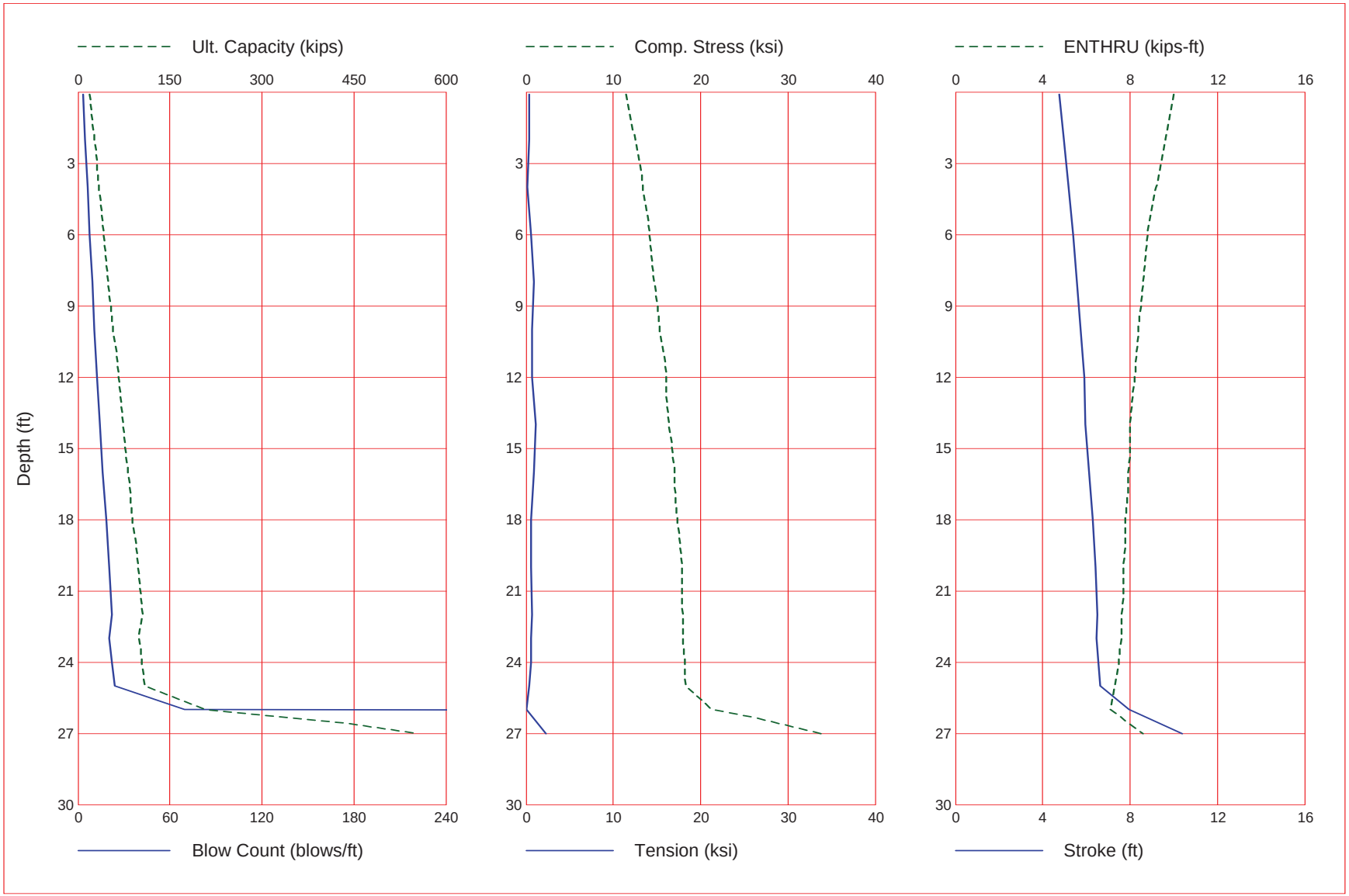
ATTACHMENT 10

Model for Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham Road

Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_Bent 1_HP12x53

Feb 09 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 09 2015
14-030_Driveability_Bent 1_HP12x53 GRLWEAP Version 2010

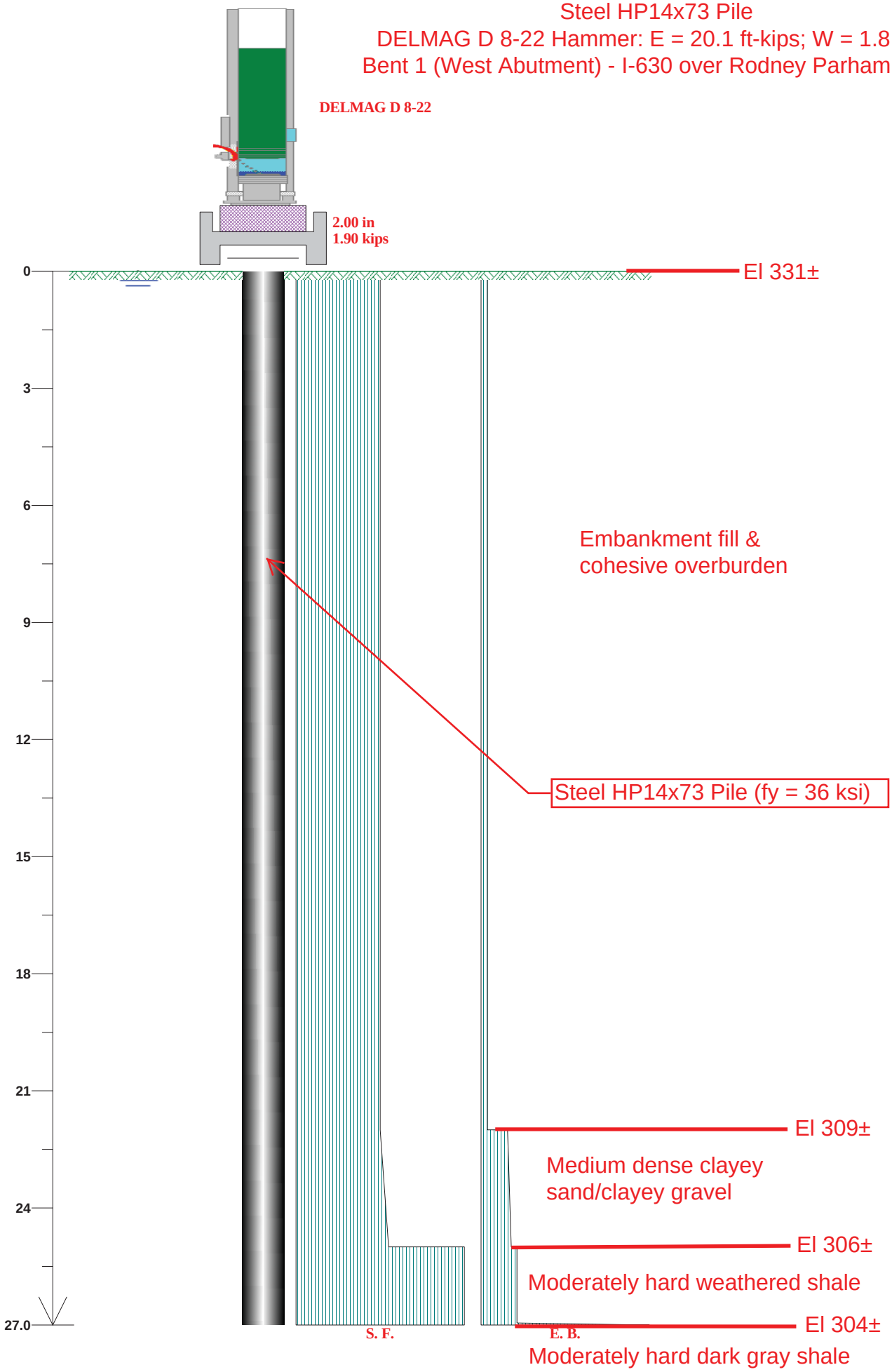
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	18.4	0.4	18.0	3.6	11.492	-0.420	4.76	10.0
2.0	25.9	7.9	18.0	4.8	12.536	-0.393	4.97	9.6
4.0	33.9	15.9	18.0	6.2	13.377	-0.172	5.18	9.2
6.0	41.8	23.8	18.0	7.7	14.098	-0.536	5.38	8.8
8.0	49.8	31.8	18.0	9.2	14.689	-0.884	5.55	8.6
10.0	57.7	39.7	18.0	10.9	15.322	-0.732	5.74	8.4
12.0	65.7	47.7	18.0	12.5	16.023	-0.677	5.89	8.2
14.0	73.6	55.6	18.0	14.3	16.352	-1.101	5.97	8.0
16.0	81.5	63.5	18.0	16.3	17.001	-0.889	6.13	7.9
18.0	89.5	71.5	18.0	18.4	17.290	-0.559	6.29	7.8
20.0	97.4	79.4	18.0	20.5	17.887	-0.534	6.42	7.7
22.0	105.4	87.4	18.0	22.1	17.977	-0.743	6.51	7.6
23.0	99.7	91.4	8.3	20.5	17.973	-0.566	6.45	7.6
24.0	104.3	95.6	8.7	22.1	18.199	-0.631	6.53	7.5
25.0	108.9	99.9	9.0	23.9	18.284	-0.415	6.63	7.3
26.0	207.8	107.8	100.0	69.5	21.275	-0.013	7.94	7.1
27.0	557.3	115.8	441.5	9999.0	33.865	-2.321	10.37	8.6

Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham

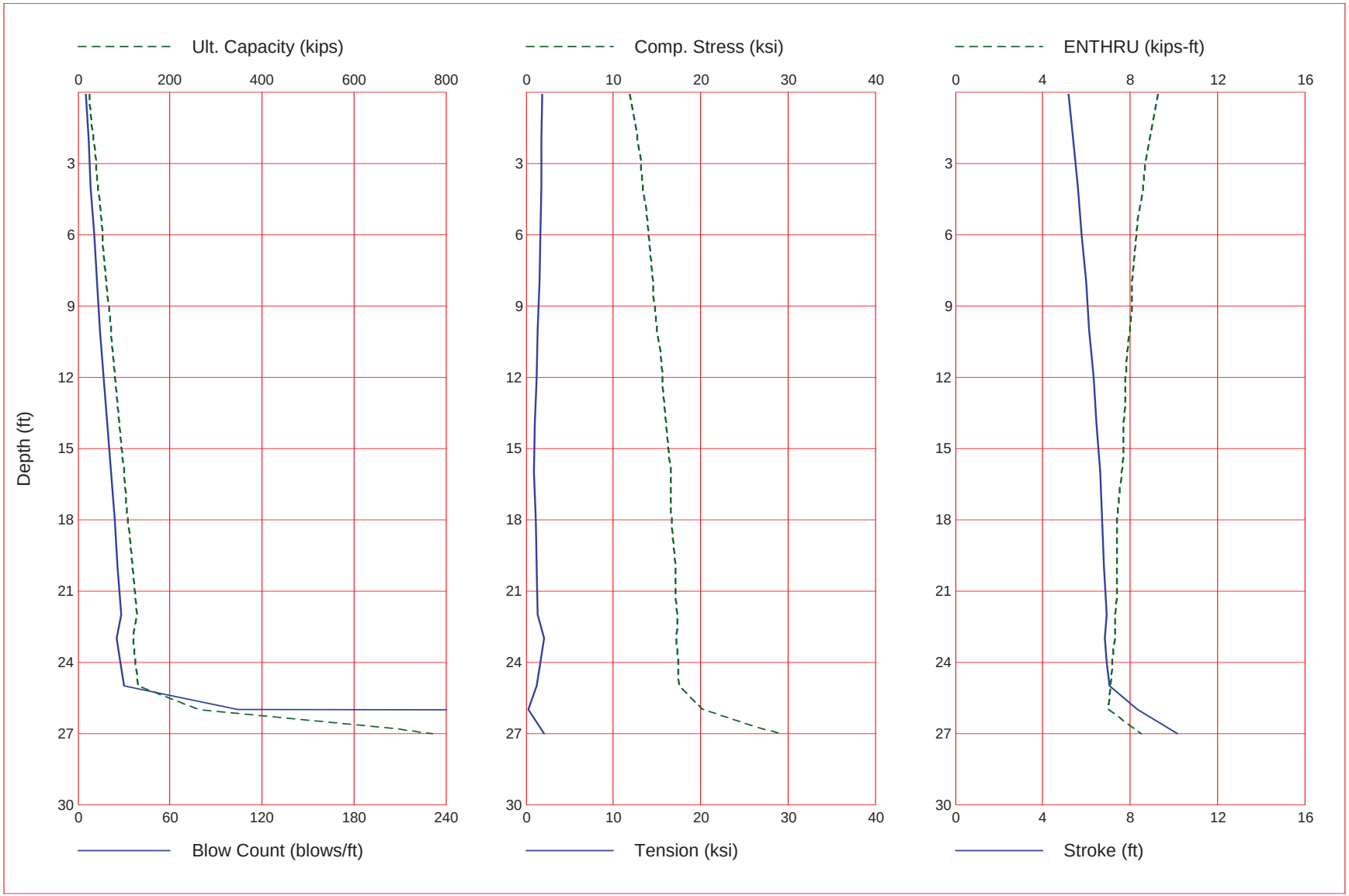
Model for Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham Road



Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_Bent 1_HP14x73

Feb 09 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 09 2015
14-030_Driveability_Bent 1_HP14x73 GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

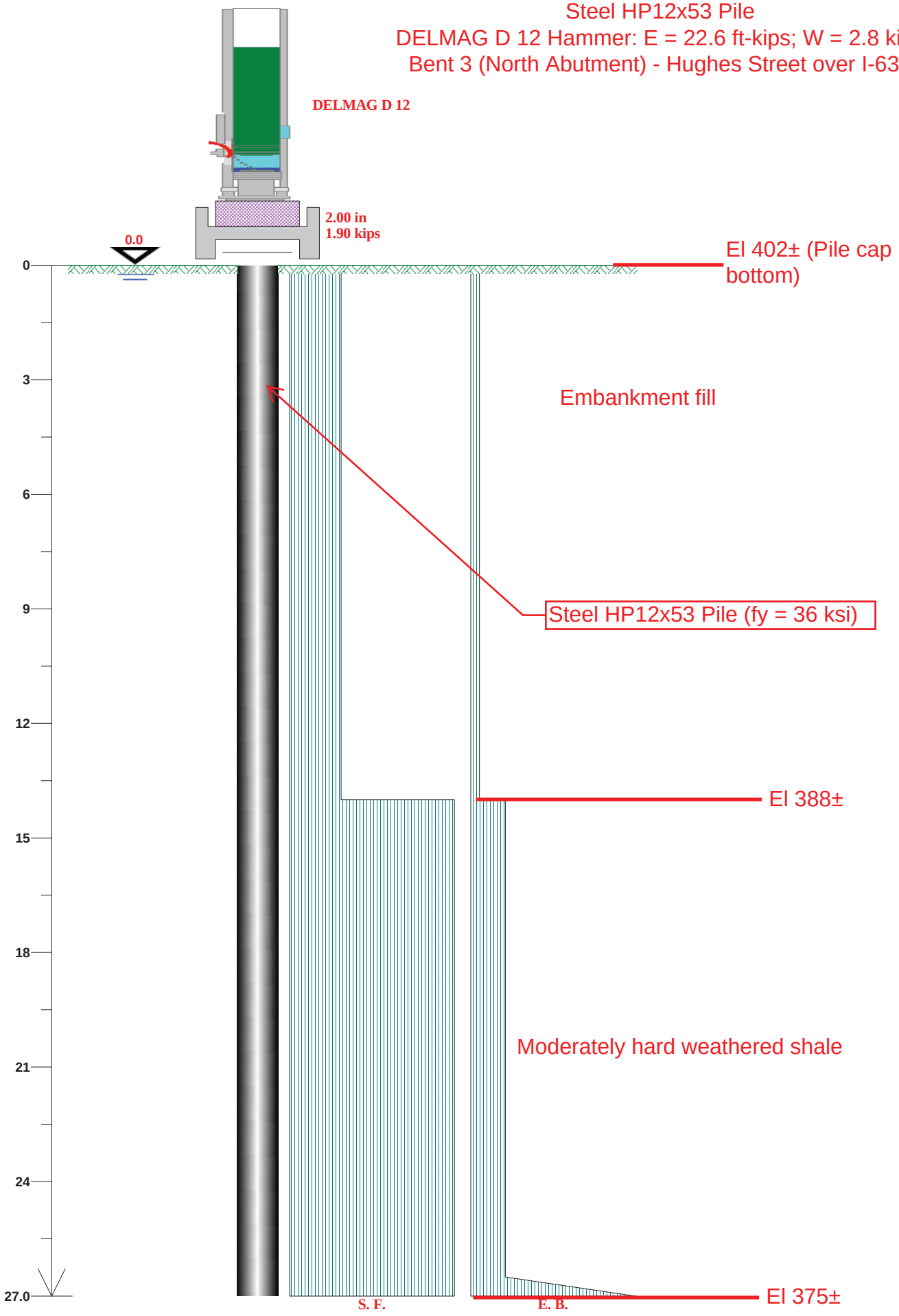
Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
0.1	25.0	0.5	24.5	5.0	11.859	-1.831	5.17	9.3
2.0	33.9	9.4	24.5	6.7	12.702	-1.757	5.39	8.9
4.0	43.3	18.8	24.5	8.5	13.385	-1.767	5.60	8.6
6.0	52.7	28.2	24.5	10.5	14.001	-1.697	5.80	8.3
8.0	62.1	37.6	24.5	12.4	14.517	-1.497	5.98	8.1
10.0	71.5	47.0	24.5	14.5	14.965	-1.293	6.14	8.0
12.0	80.9	56.4	24.5	16.8	15.624	-1.183	6.32	7.8
14.0	90.3	65.8	24.5	19.2	16.083	-1.049	6.48	7.7
16.0	99.7	75.2	24.5	21.6	16.543	-0.901	6.62	7.6
18.0	109.1	84.6	24.5	23.7	16.678	-1.153	6.71	7.4
20.0	118.5	94.0	24.5	25.9	17.101	-1.259	6.81	7.4
22.0	127.9	103.4	24.5	28.2	17.291	-1.318	6.93	7.3
23.0	119.7	108.2	11.5	25.4	17.193	-2.069	6.84	7.3
24.0	125.1	113.1	12.0	27.7	17.430	-1.647	6.95	7.2
25.0	130.7	118.2	12.5	30.3	17.547	-1.258	7.05	7.1
26.0	263.7	127.6	136.1	104.0	20.248	-0.313	8.34	7.0
27.0	770.3	137.0	633.3	9999.0	29.148	-2.131	10.18	8.5

Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 8-22 Hammer: E = 20.1 ft-kips; W = 1.8 kips
Bent 1 (West Abutment) - I-630 over Rodney Parham

ATTACHMENT 11

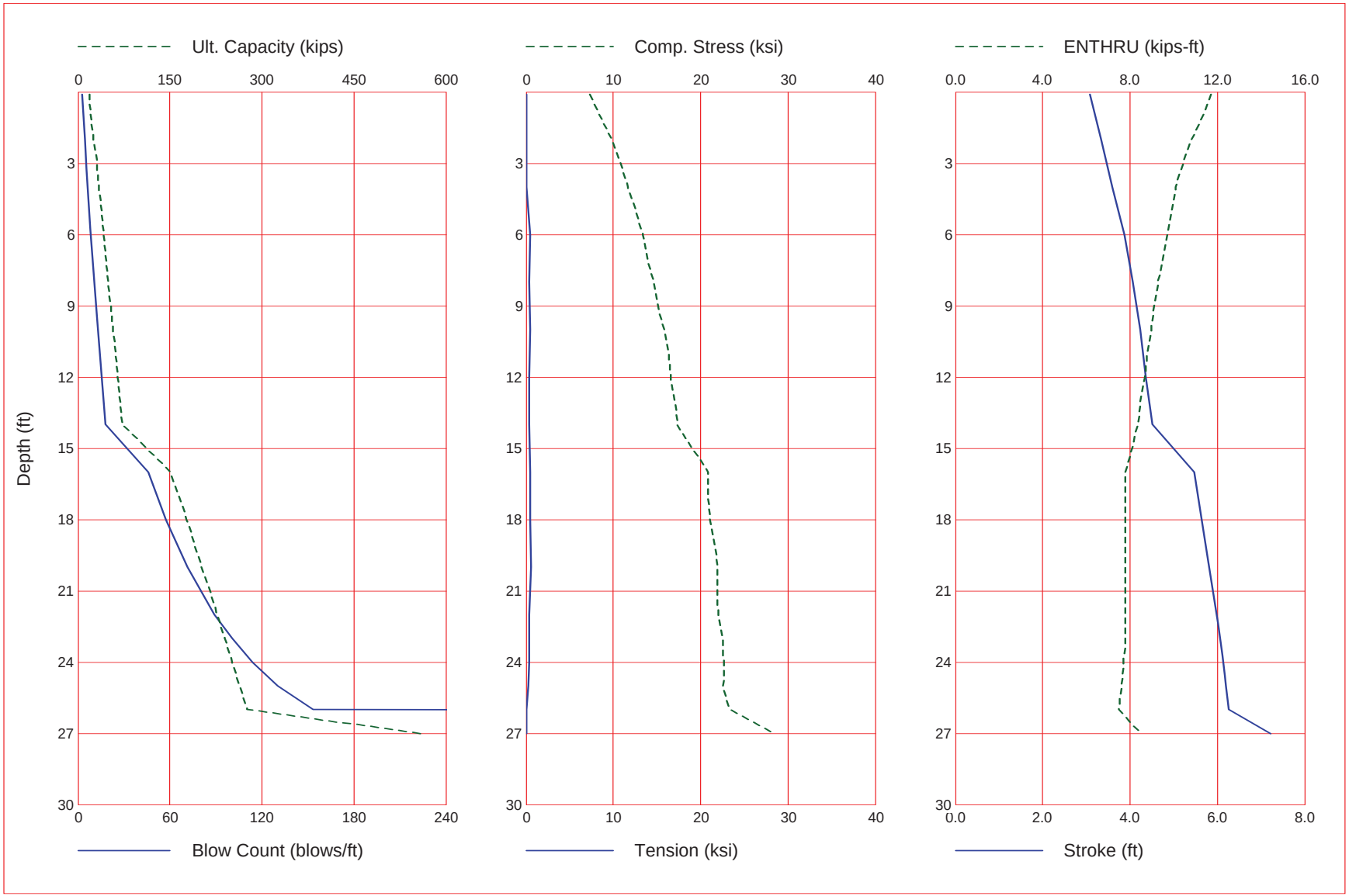
Model for Driveability Analysis
Steel HP12x53 Pile
DELMAG D 12 Hammer: E = 22.6 ft-kips; W = 2.8 kips
Bent 3 (North Abutment) - Hughes Street over I-630



Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_Bent 3_HP12x53

Feb 18 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 12 Hammer: E = 22.6 ft-kips; W = 2.8 kips
Bent 3 (North Abutment) - Hughes Street over I-630

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 18 2015
14-030_Driveability_Bent 3_HP12x53 GRLWEAP Version 2010

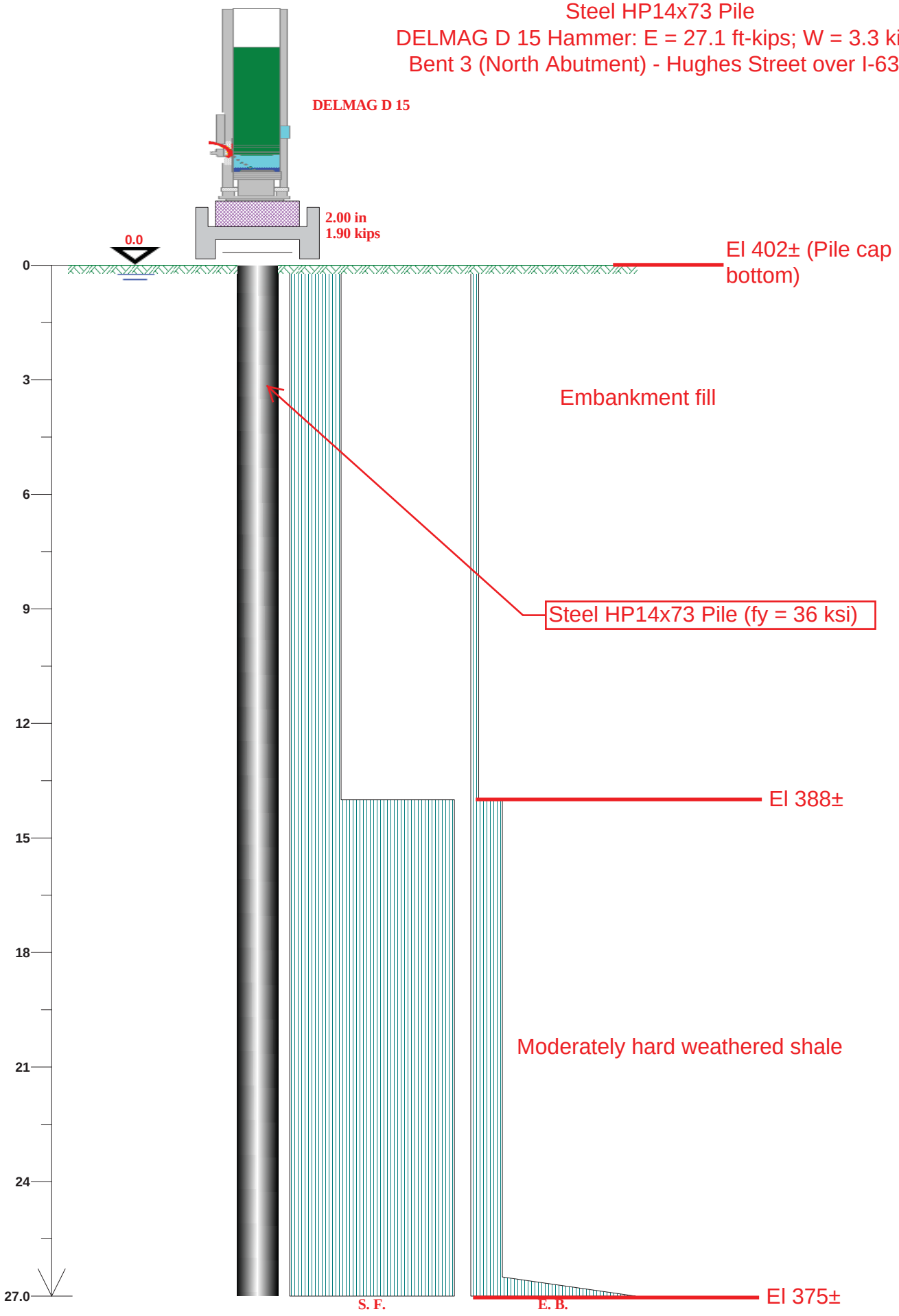
Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

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	18.4	0.4	18.0	2.9	7.264	0.000	3.08	11.7
2.0	25.8	7.8	18.0	4.6	9.814	0.000	3.34	10.8
4.0	33.6	15.6	18.0	6.5	11.645	0.000	3.60	10.1
6.0	41.4	23.4	18.0	8.5	13.401	-0.456	3.87	9.7
8.0	49.2	31.2	18.0	10.7	14.637	-0.385	4.07	9.3
10.0	57.0	39.0	18.0	13.0	15.799	-0.500	4.24	9.0
12.0	64.8	46.8	18.0	15.6	16.602	-0.404	4.36	8.7
14.0	72.6	54.6	18.0	18.2	17.361	-0.395	4.51	8.4
16.0	151.6	79.6	72.0	46.0	20.839	-0.513	5.47	7.8
18.0	176.5	104.5	72.0	57.4	21.098	-0.479	5.65	7.8
20.0	201.5	129.5	72.0	71.1	21.972	-0.623	5.81	7.8
22.0	226.4	154.4	72.0	89.3	22.030	-0.404	5.98	7.8
23.0	238.9	166.9	72.0	100.7	22.507	-0.423	6.06	7.8
24.0	251.4	179.4	72.0	114.0	22.706	-0.413	6.13	7.7
25.0	263.9	191.9	72.0	130.2	22.528	-0.265	6.20	7.6
26.0	276.4	204.4	72.0	153.5	23.288	-0.083	6.27	7.5
27.0	559.6	216.8	342.7	9999.0	28.332	0.000	7.23	8.5

Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP12x53 Pile
DELMAG D 12 Hammer: E = 22.6 ft-kips; W = 2.8 kips
Bent 3 (North Abutment) - Hughes Street over I-630

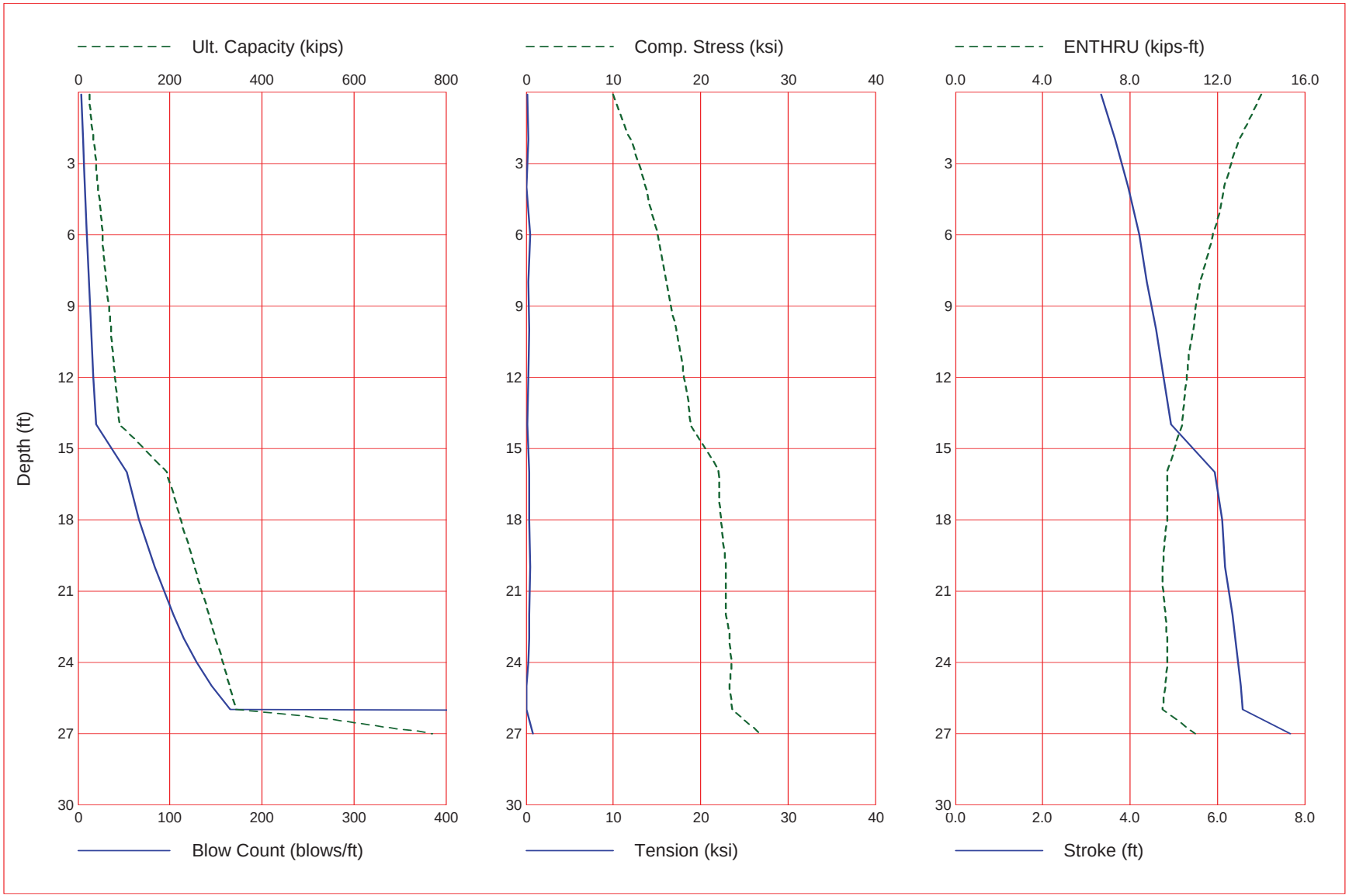
Model for Driveability Analysis
Steel HP14x73 Pile
DELMAG D 15 Hammer: E = 27.1 ft-kips; W = 3.3 kips
Bent 3 (North Abutment) - Hughes Street over I-630



Grubbs, Hoskyn, Barton & Wyatt, Inc.
14-030_Driveability_Bent 3_HP14x73

Feb 07 2015
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000



Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 15 Hammer: E = 27.1 ft-kips; W = 3.3 kips
Bent 3 (North Abutment) - Hughes Street over I-630

Grubbs, Hoskyn, Barton & Wyatt, Inc. Feb 07 2015
14-030_Driveability_Bent 3_HP14x73 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	25.0	0.5	24.5	3.3	9.994	-0.199	3.33	14.0
2.0	33.9	9.4	24.5	5.1	11.998	-0.224	3.66	13.0
4.0	43.3	18.8	24.5	7.1	13.759	0.000	3.96	12.3
6.0	52.7	28.2	24.5	9.2	15.088	-0.439	4.21	11.8
8.0	62.1	37.6	24.5	11.6	16.035	-0.270	4.39	11.2
10.0	71.5	47.0	24.5	14.2	17.192	-0.414	4.59	10.9
12.0	80.9	56.4	24.5	16.8	18.058	-0.244	4.77	10.6
14.0	90.3	65.8	24.5	19.6	18.811	-0.132	4.94	10.4
16.0	193.9	95.9	98.0	53.3	22.129	-0.425	5.95	9.7
18.0	224.0	126.0	98.0	66.2	22.357	-0.393	6.11	9.7
20.0	254.0	156.0	98.0	83.2	22.867	-0.526	6.18	9.5
22.0	284.1	186.1	98.0	103.4	22.922	-0.322	6.35	9.6
23.0	299.2	201.2	98.0	114.9	23.329	-0.355	6.42	9.7
24.0	314.2	216.2	98.0	128.9	23.511	-0.248	6.48	9.7
25.0	329.2	231.2	98.0	145.3	23.281	-0.007	6.54	9.6
26.0	344.3	246.3	98.0	165.8	23.664	0.000	6.58	9.5
27.0	771.3	261.3	510.0	9999.0	26.854	-0.791	7.66	11.0

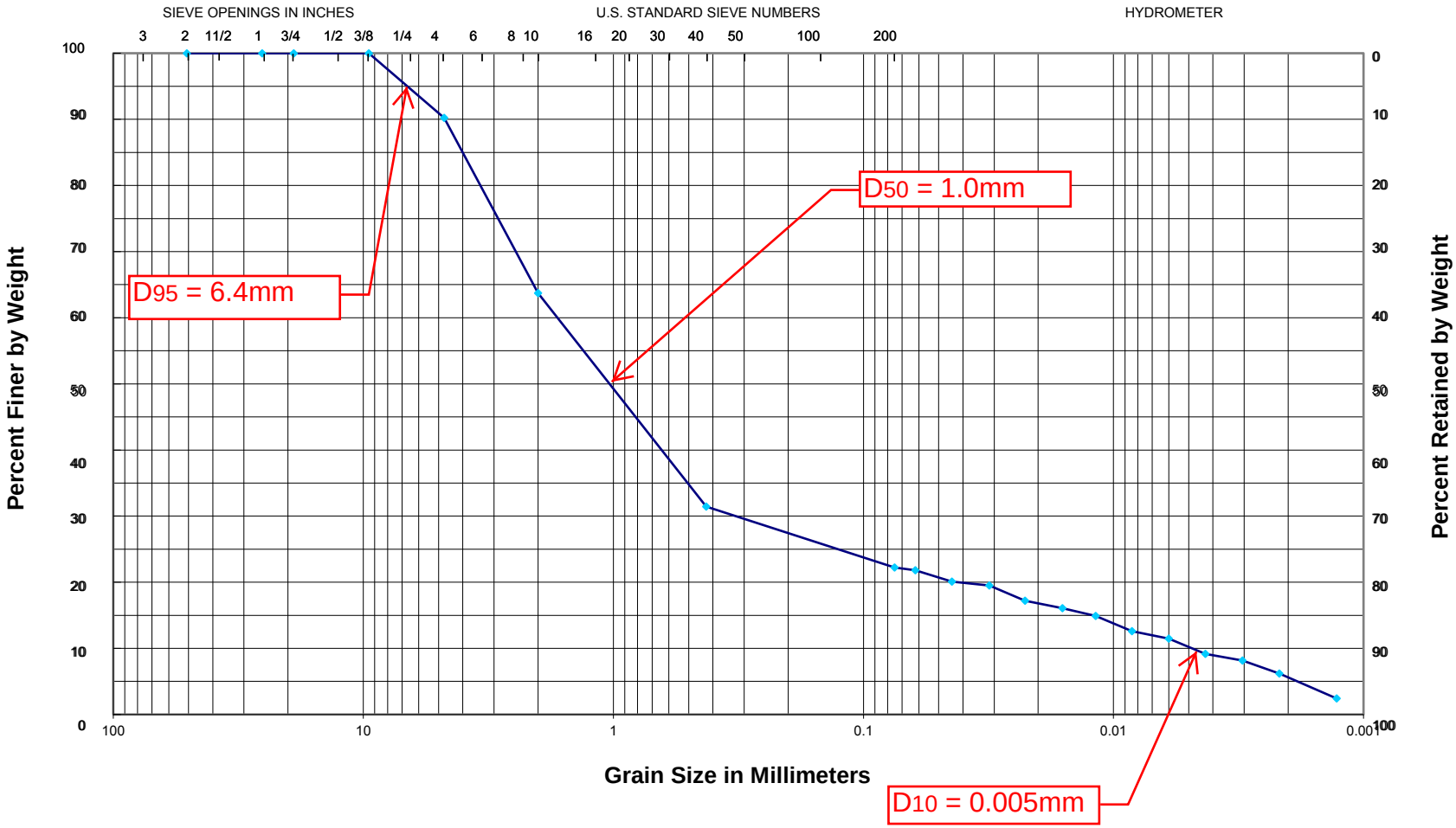
Refusal occurred; no driving time output possible

Results of Driveability Analysis
Steel HP14x73 Pile
DELMAG D 15 Hammer: E = 27.1 ft-kips; W = 3.3 kips
Bent 3 (North Abutment) - Hughes Street over I-630

ATTACHMENT 12

14-030

GRAIN SIZE CURVE



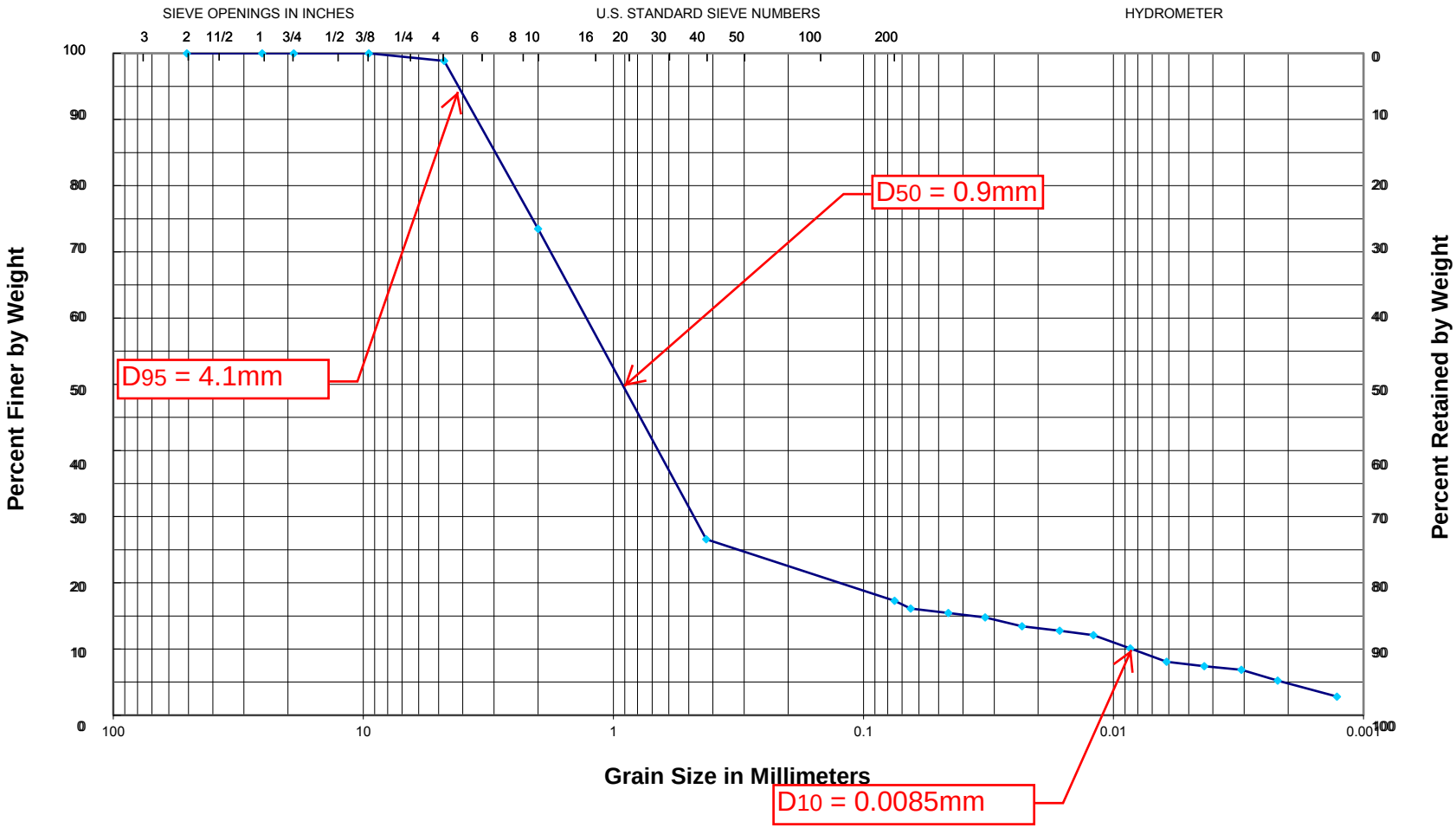
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring S7; 34-40 ft
Properties: G_s = 2.814; LL = 29, PI = 19, PI = 10

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

14-030

GRAIN SIZE CURVE



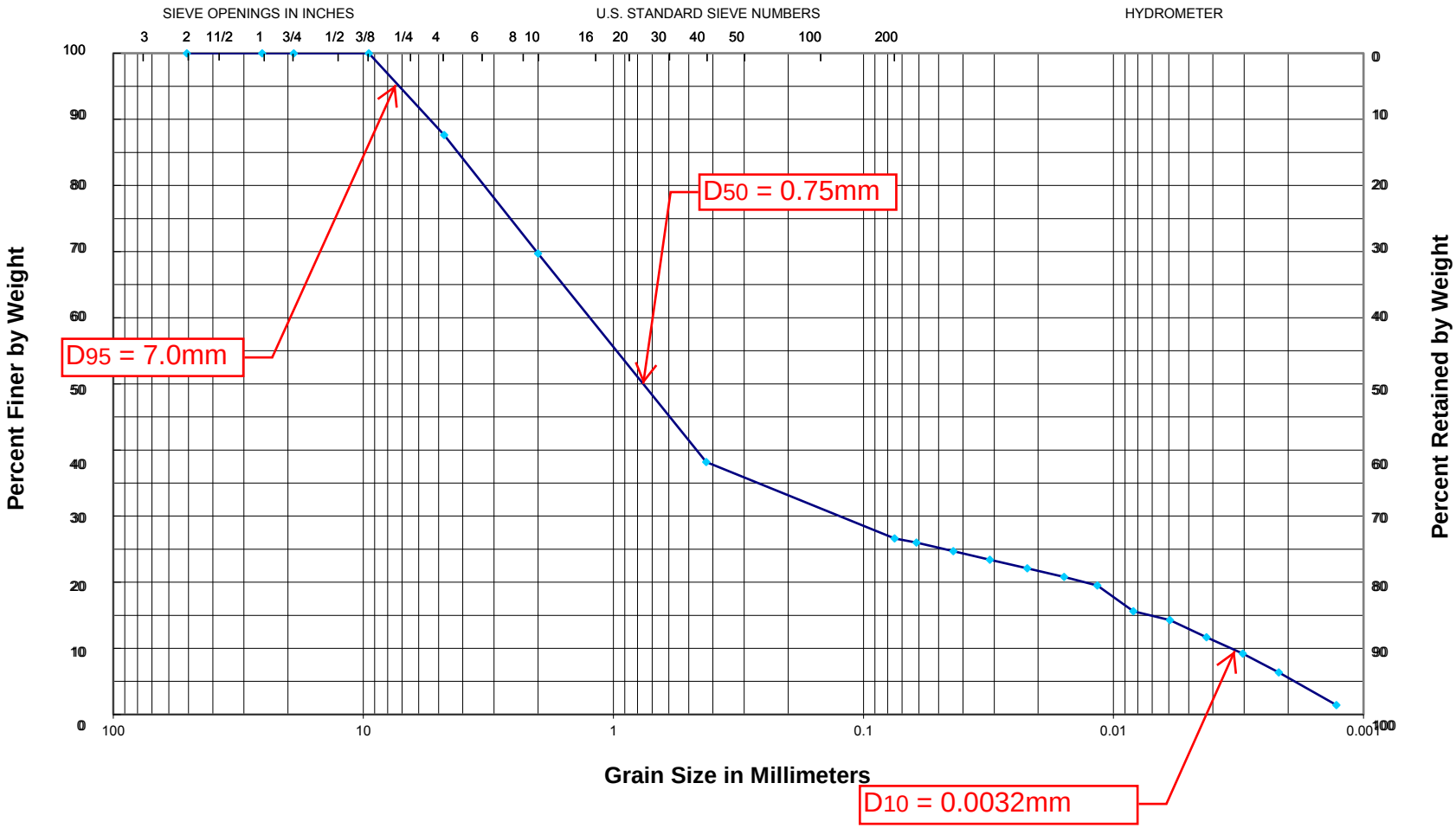
GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

Sample: Boring S10A; 29-35 ft
Properties: G_s = 2.818; LL = 28, PI = 19, PI = 9

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

14-030

GRAIN SIZE CURVE



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		

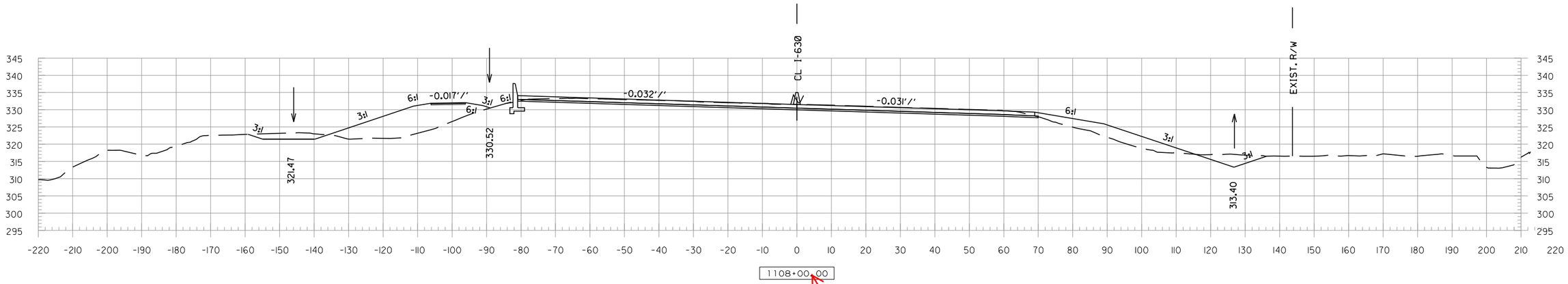
Sample: Boring S13; 28.5-29 ft
Properties: G_s = 2.825; LL = 29, PI = 20, PI = 9

Description: Dark gray shale fragments (cuttings)
Classification: USCS = SC; AASHTO = A-2-4

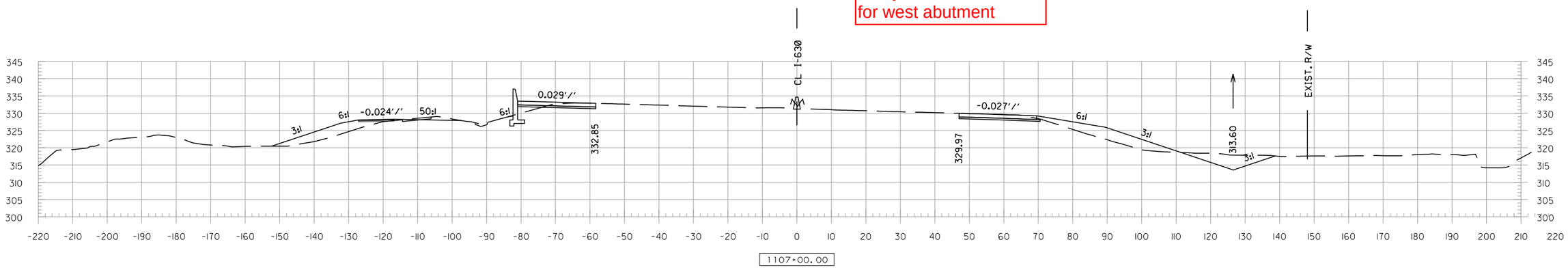
ATTACHMENT 13

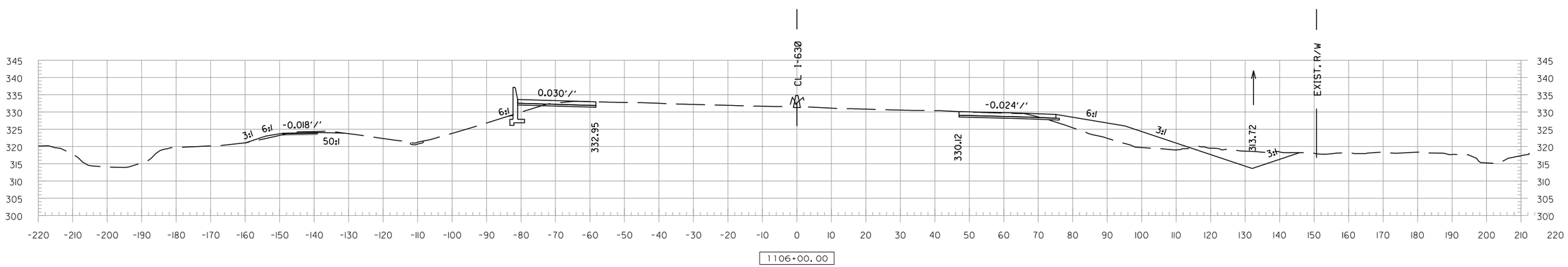
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				X	ARK.			
				JOB NO.		CA0608	16	46

(2) CROSS SECTIONS I-630



Analyzed this section for west abutment





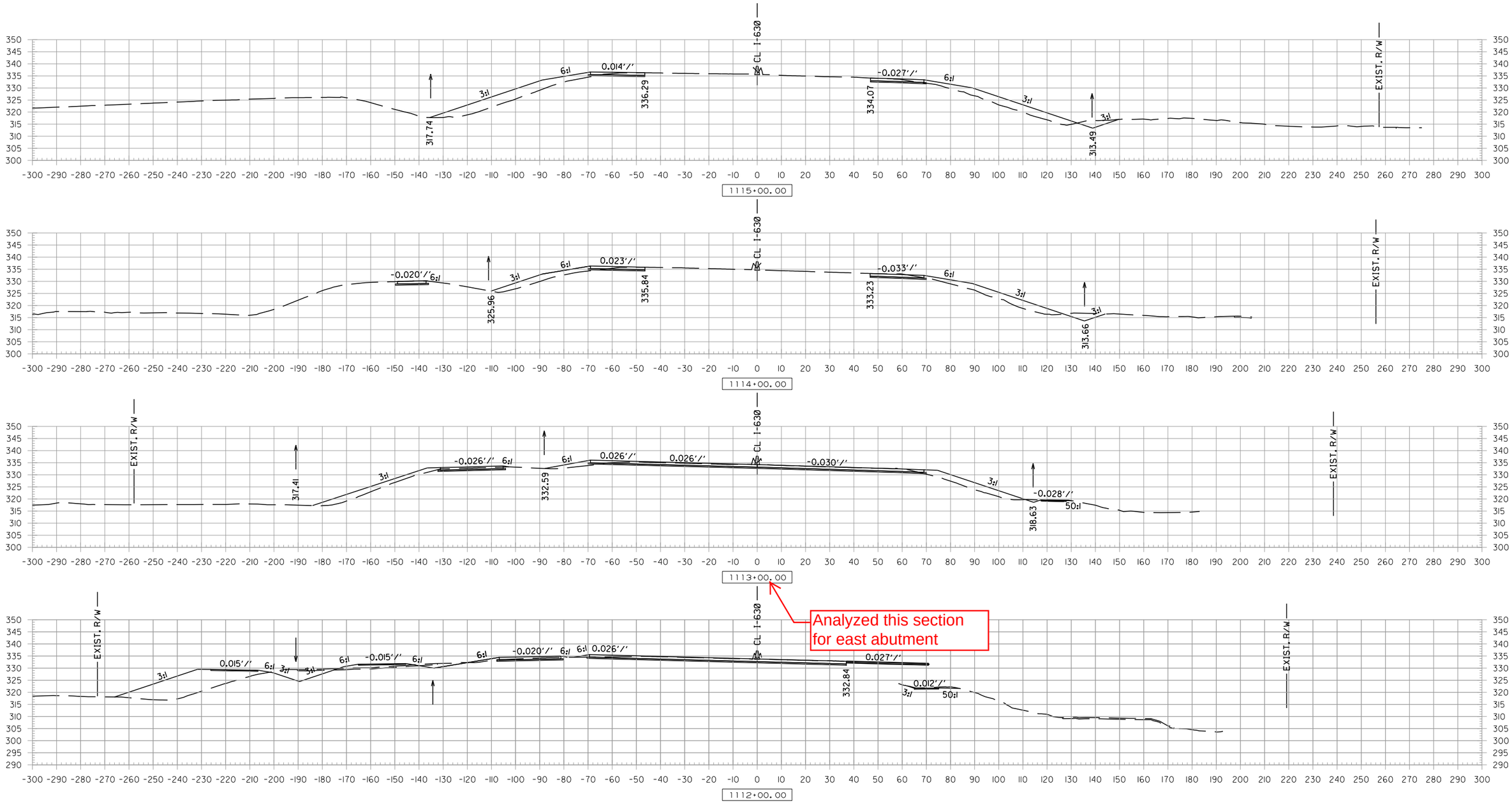
STA. 1106+00 TO STA. 1108+00

12/10/2014 3:53:11 PM

S:\14406\DI\DGN\Reference\STATIC\14406_XS30_30_Sc.dgn

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED.RD. DIST.NO.	STATE	FED.AID PROJ.NO.	SHEET NO.	TOTAL SHEETS
				X	ARK.			
				JOB NO.		CA0608	18	46

2 CROSS SECTIONS I-630



STA. 1112+00 TO STA. 1115+00

12/10/2014 3:53:11 PM

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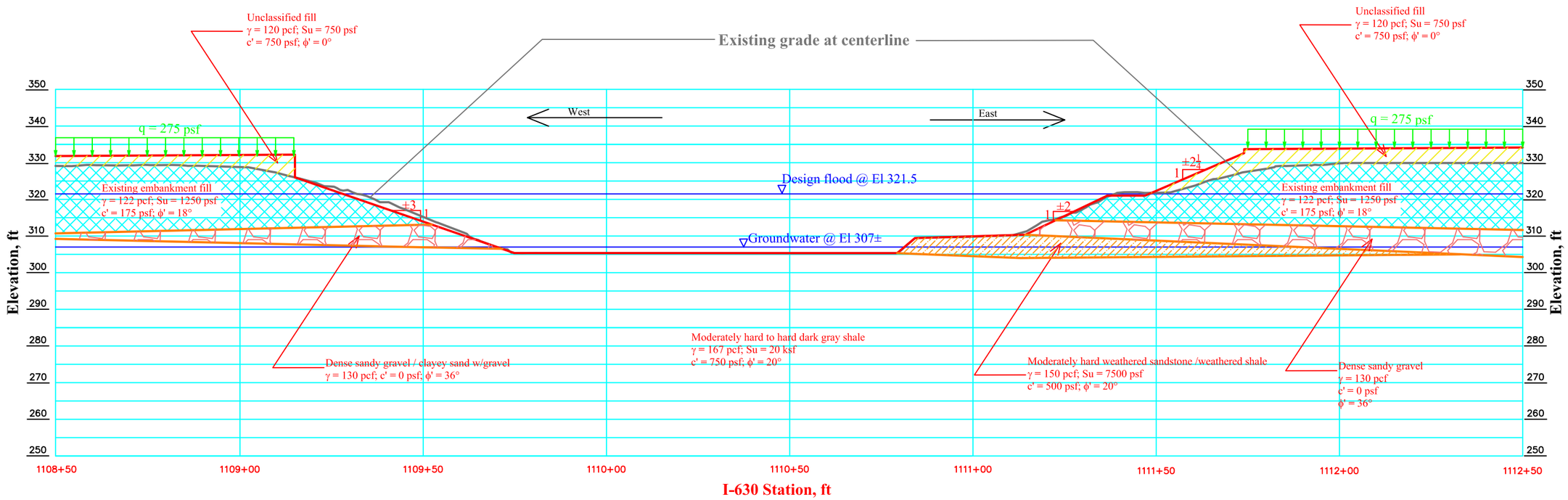
ATTACHMENT 14



Note: Section developed for purpose of stability analysis only, not for construction.

15 ft 0 15 30 ft

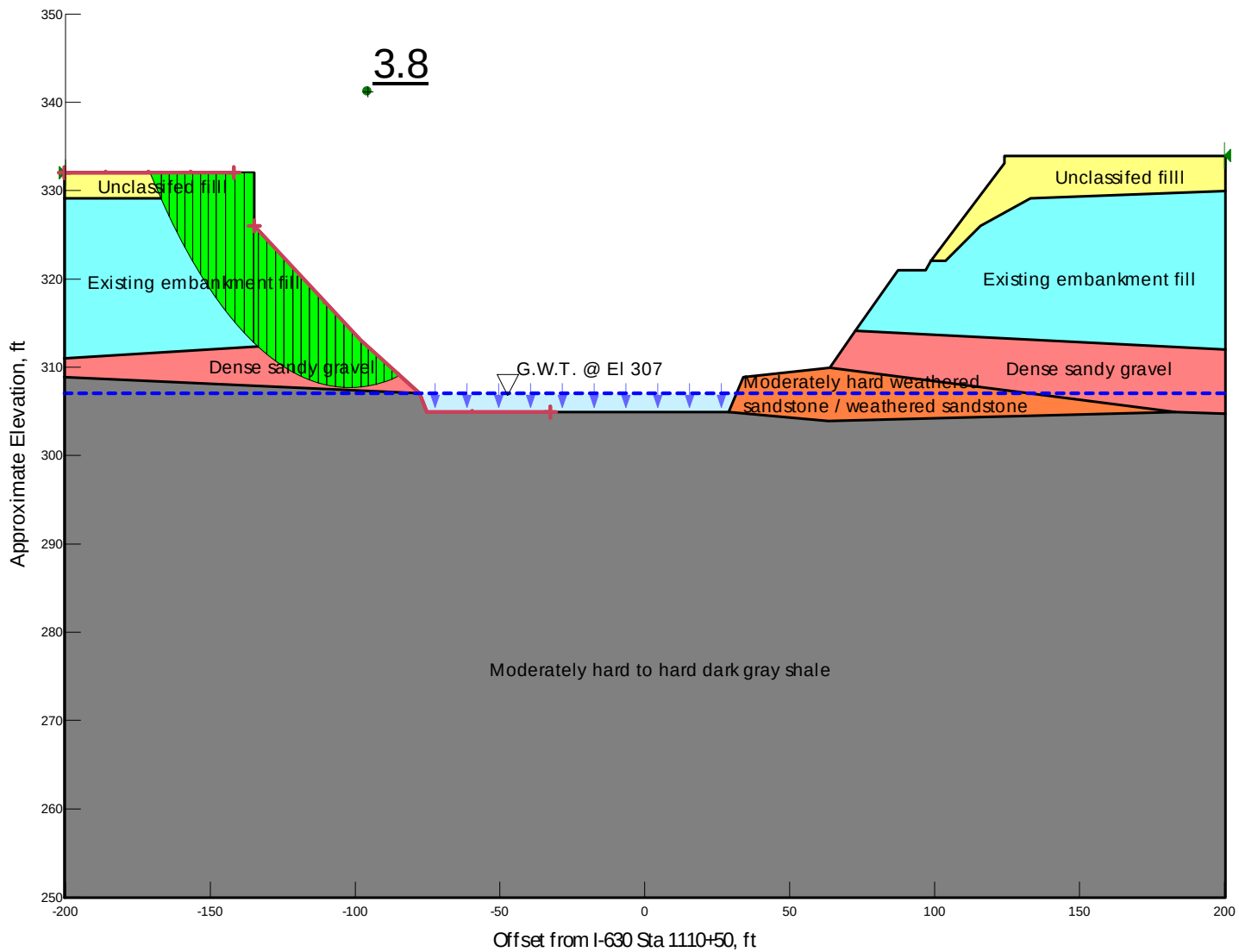
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
End Slopes @ Bridge Abutments - I-630 over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 12, 2015	Plate

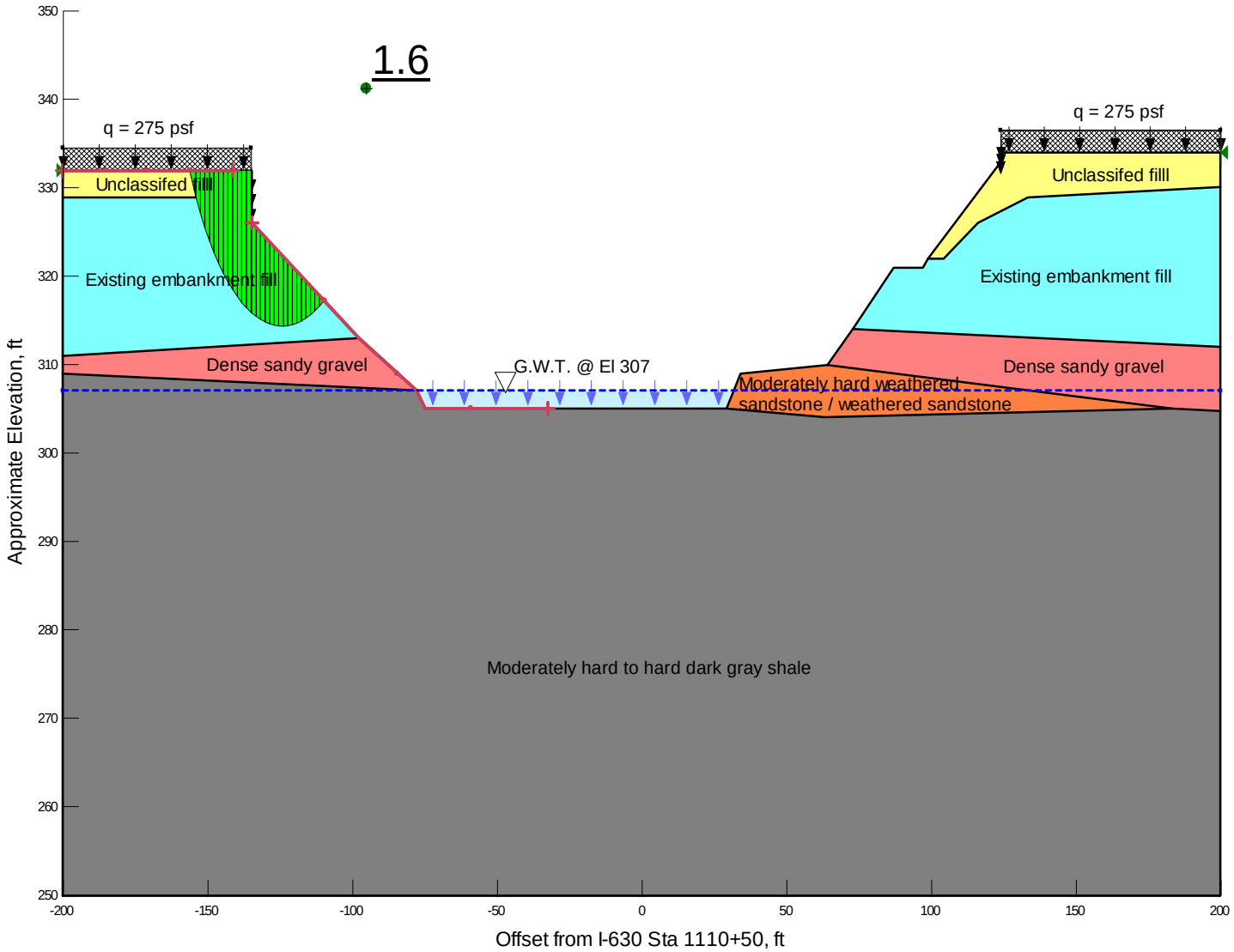
Results of Stability Analyses
End Slope at West Bridge Abutment – I-630 over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	3.8
Long Term	Groundwater @ El 307±	1.6
	Design flood @ El 321.5	1.5
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.3
Rapid Drawdown	Drawdown from design flood to embankment toe	1.4



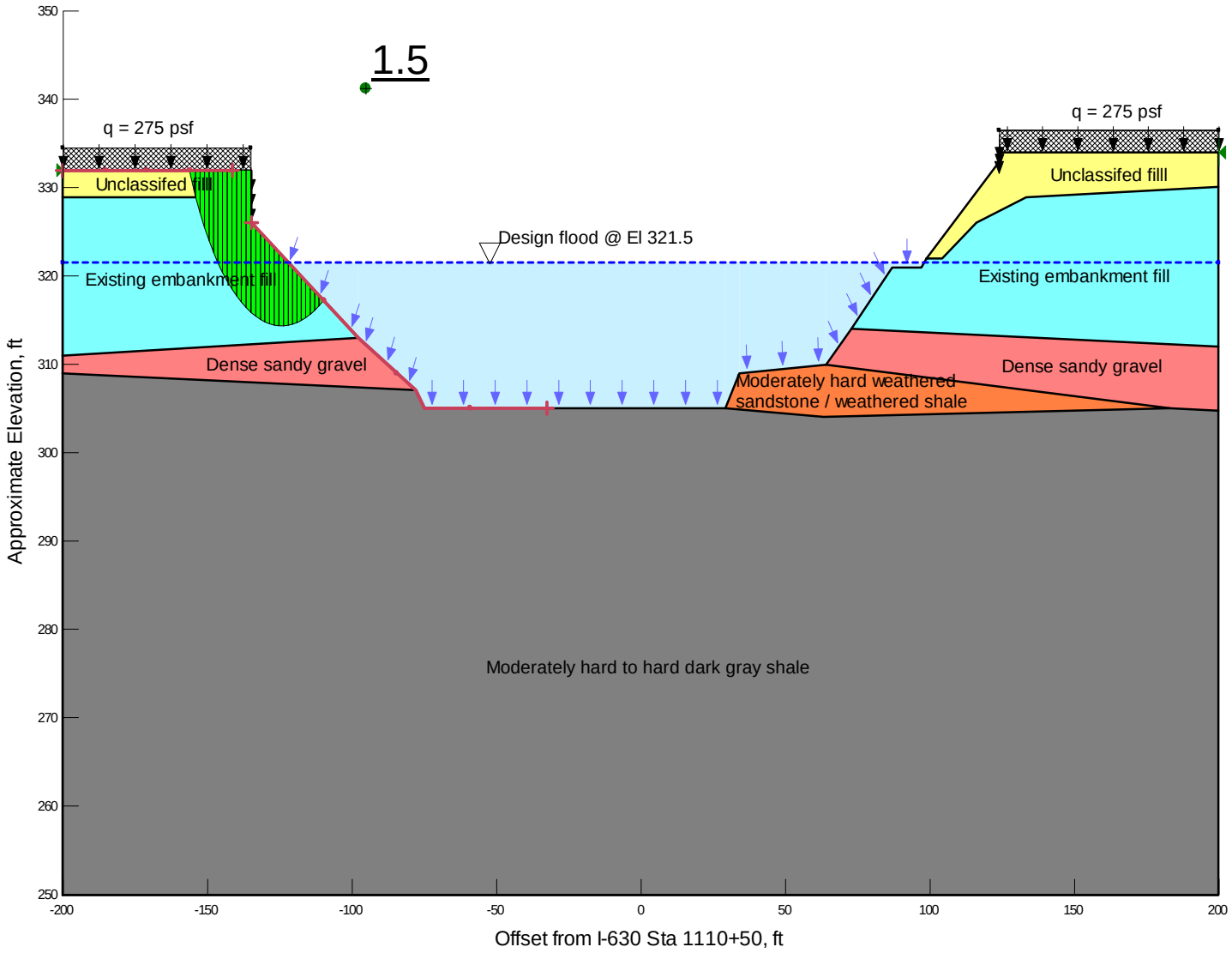
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
End Slope @ West Bridge Abutment – I-630 over Rock Creek

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



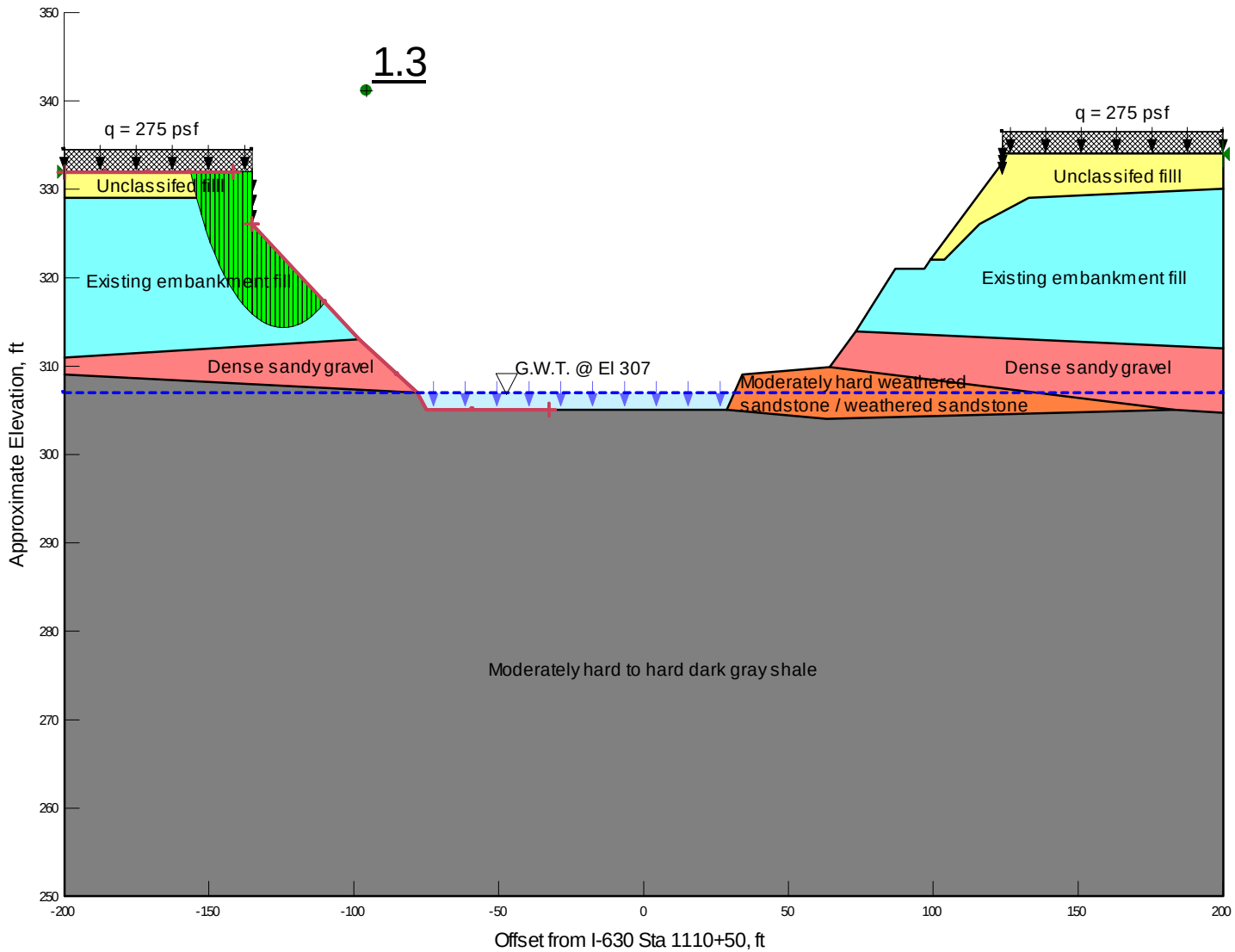
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
End Slope @ West Bridge Abutment – I-630 over Rock Creek

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



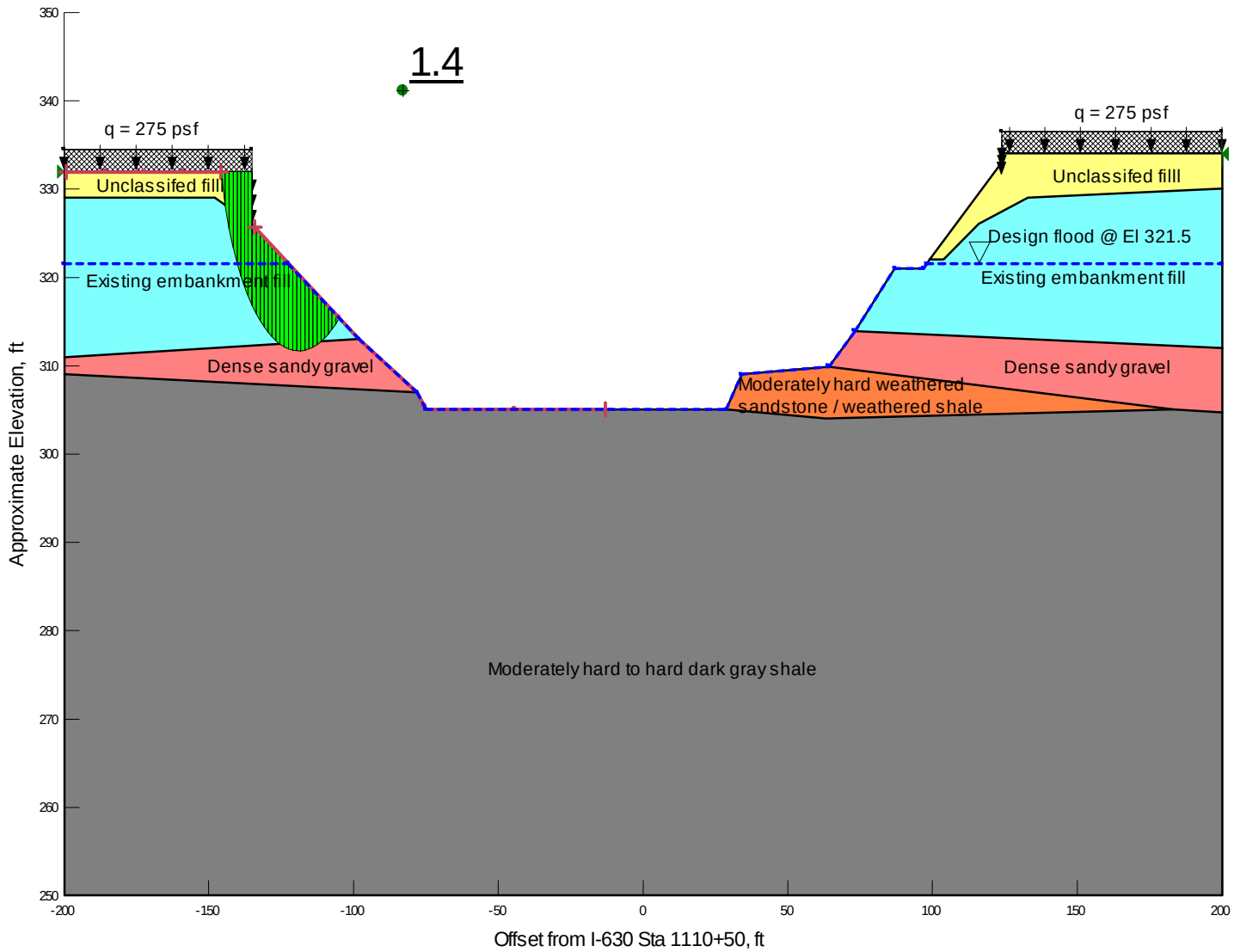
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
End Slope @ West Bridge Abutment – I-630 over Rock Creek

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
End Slope @ West Bridge Abutment – I-630 over Rock Creek

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

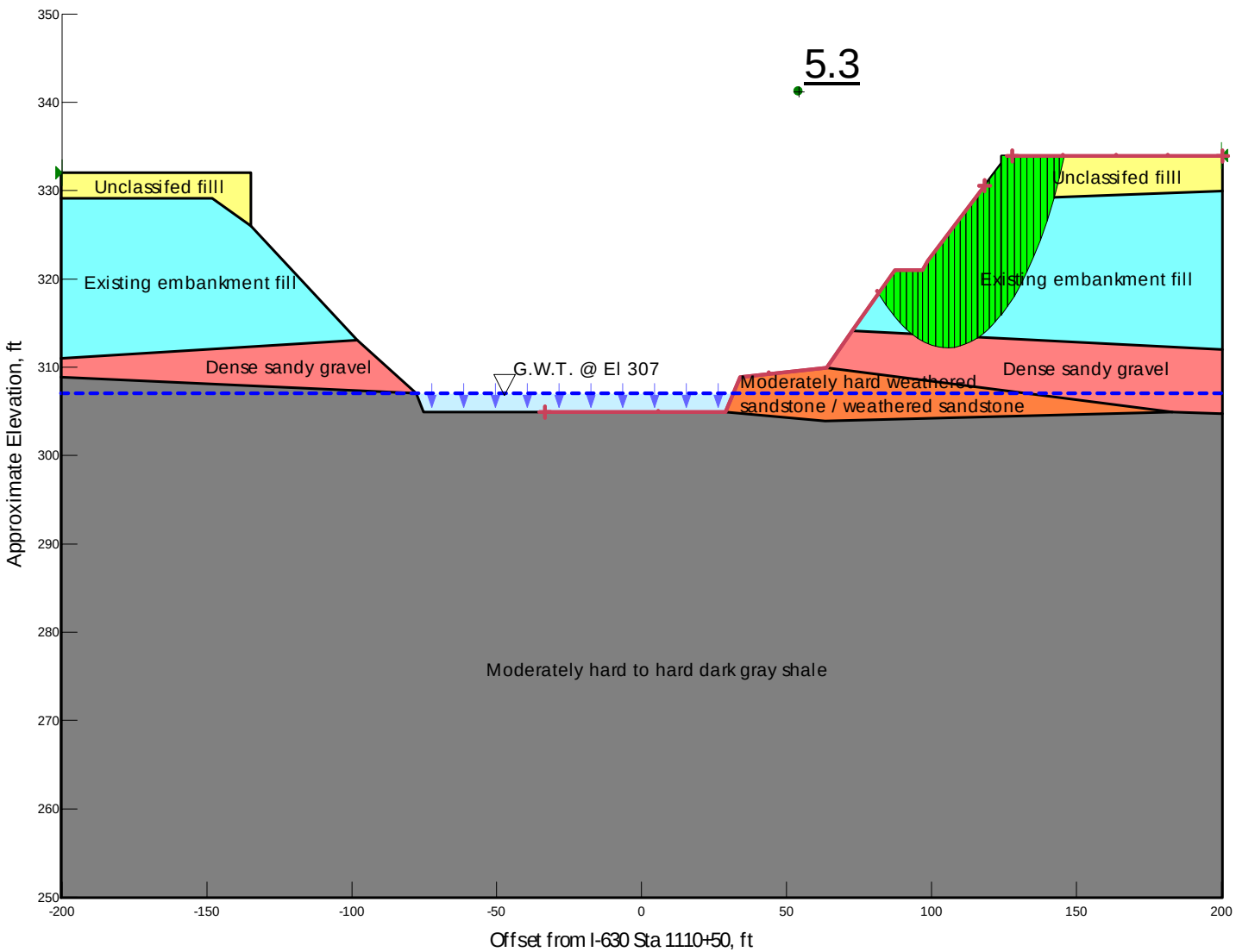


Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
End Slope @ West Bridge Abutment – I-630 over Rock Creek

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

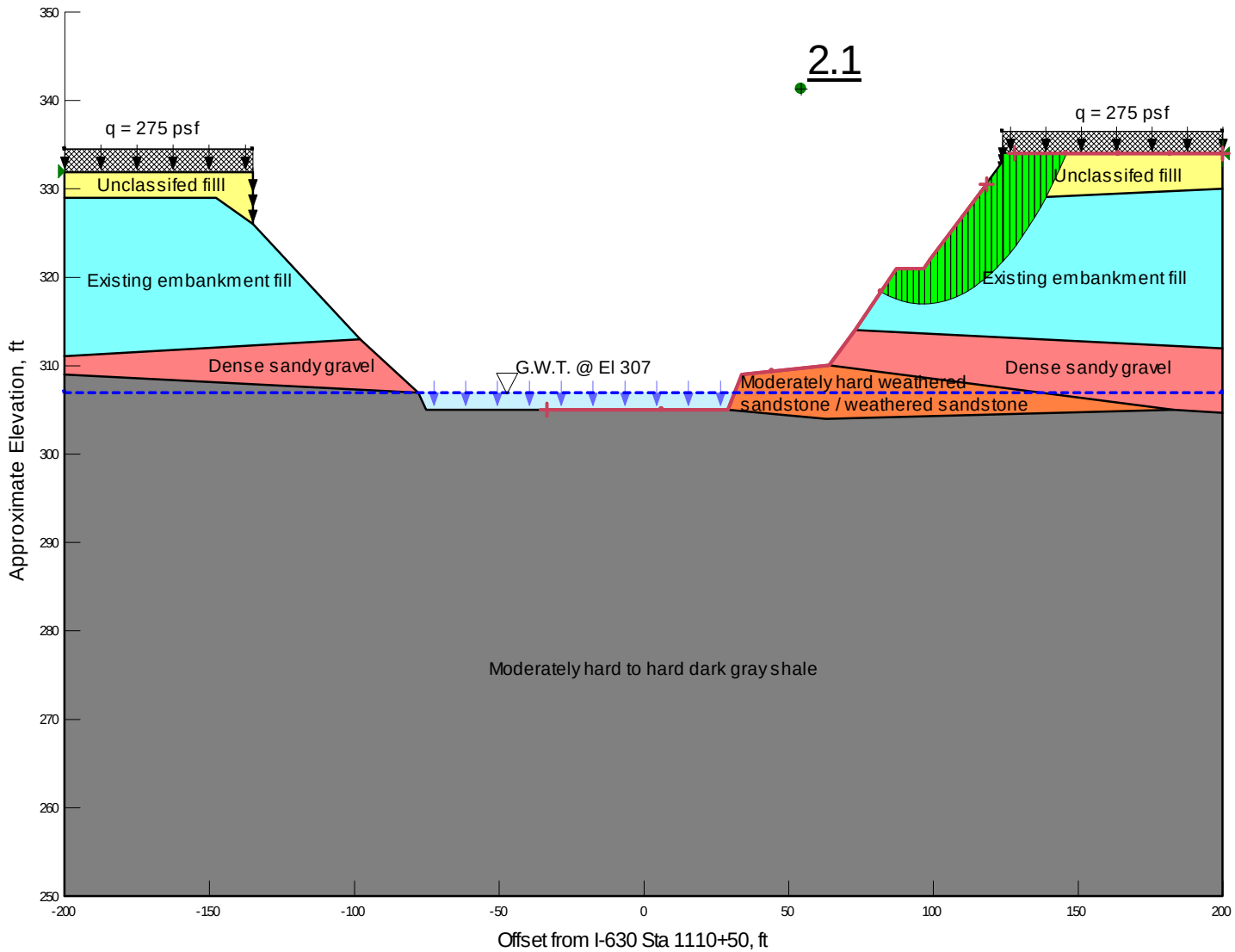
Results of Stability Analyses
End Slope at East Bridge Abutment – I-630 over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.1
	Design flood @ El 321.5	2.0
Seismic (k_h = 1.0A_S = 0.13)	Groundwater @ El 307±	1.5
Rapid Drawdown	Drawdown from design flood to embankment toe	1.9



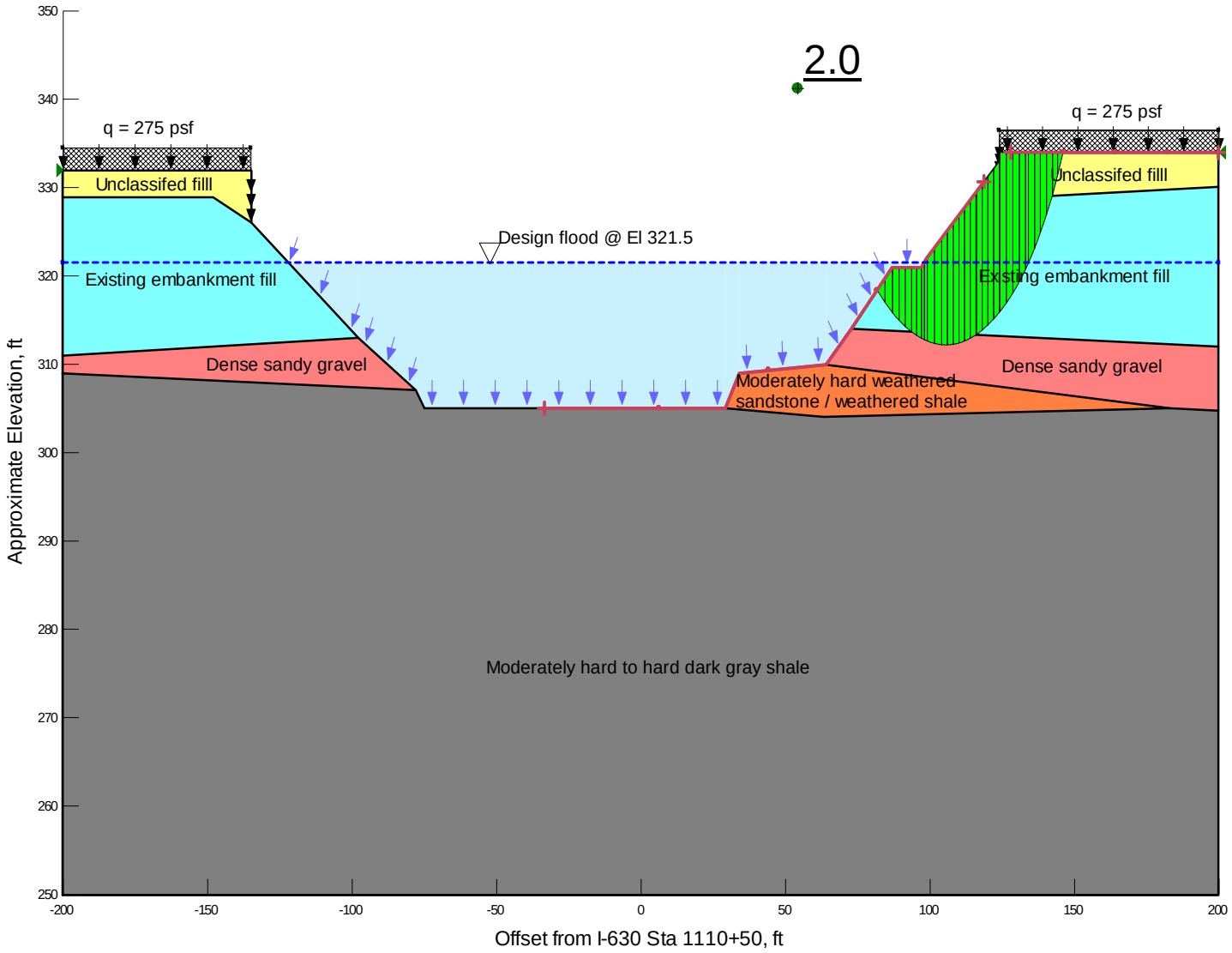
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
End Slope @ East Bridge Abutment – I-630 over Rock Creek

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



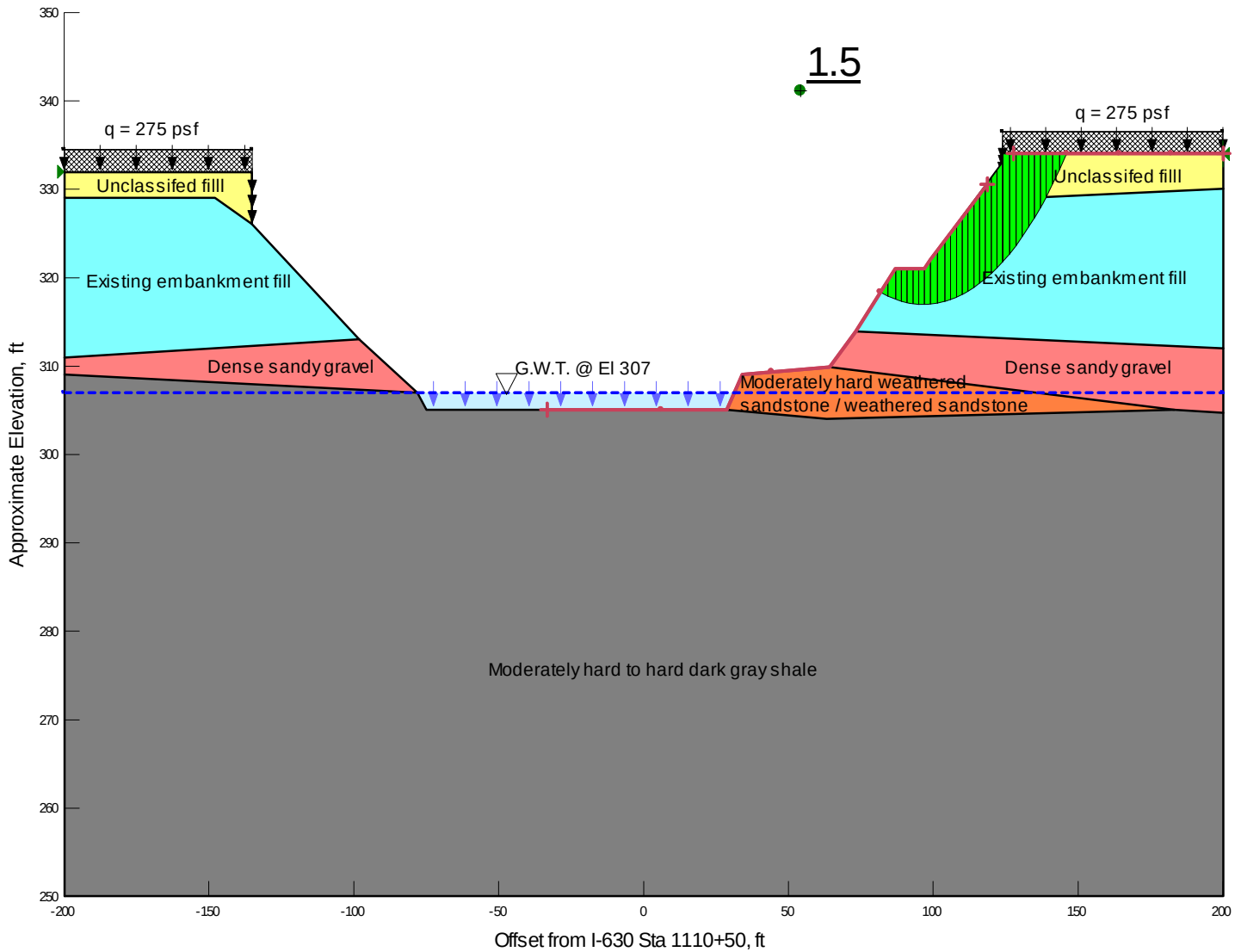
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
End Slope @ East Bridge Abutment – I-630 over Rock Creek

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



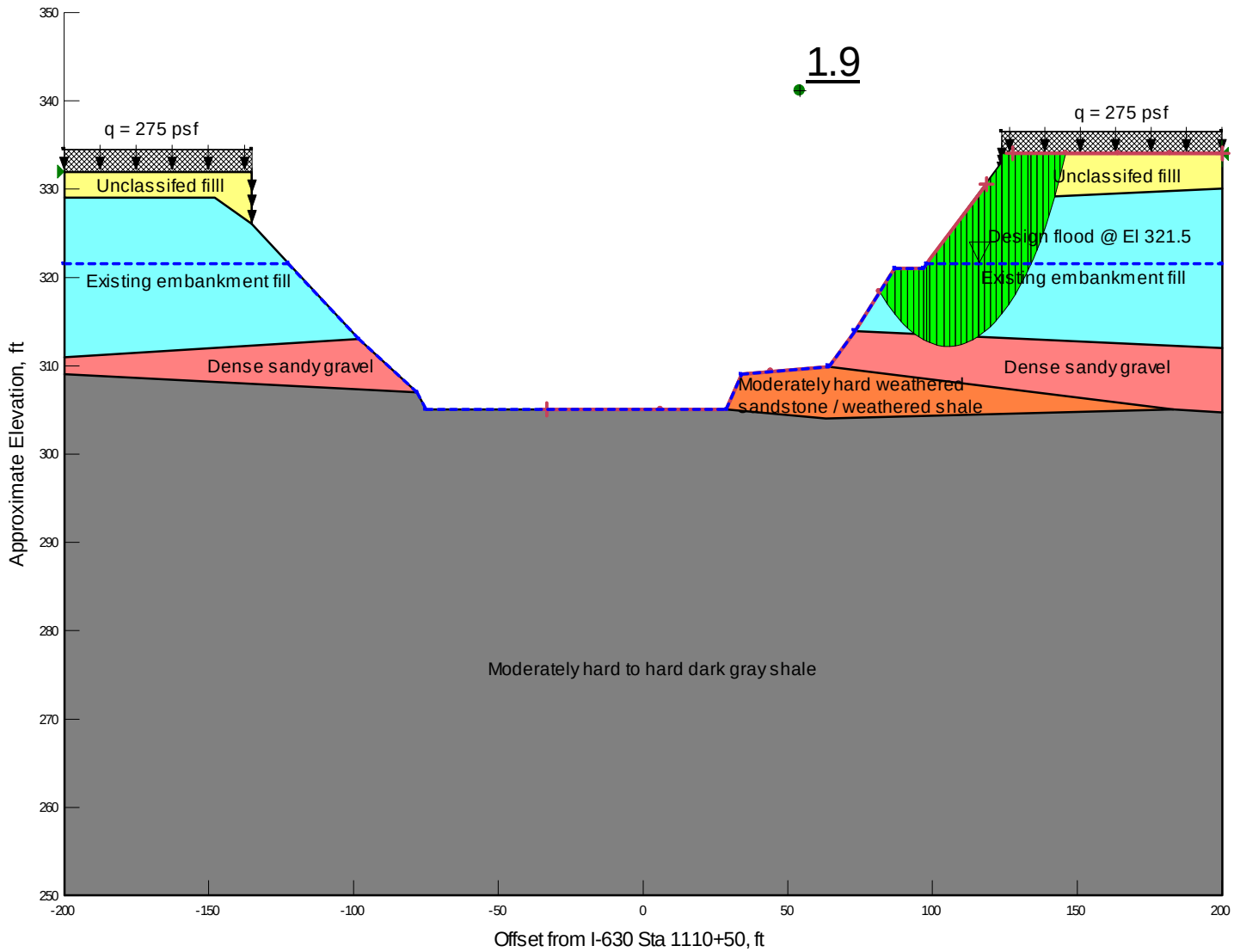
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
End Slope @ East Bridge Abutment – I-630 over Rock Creek

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



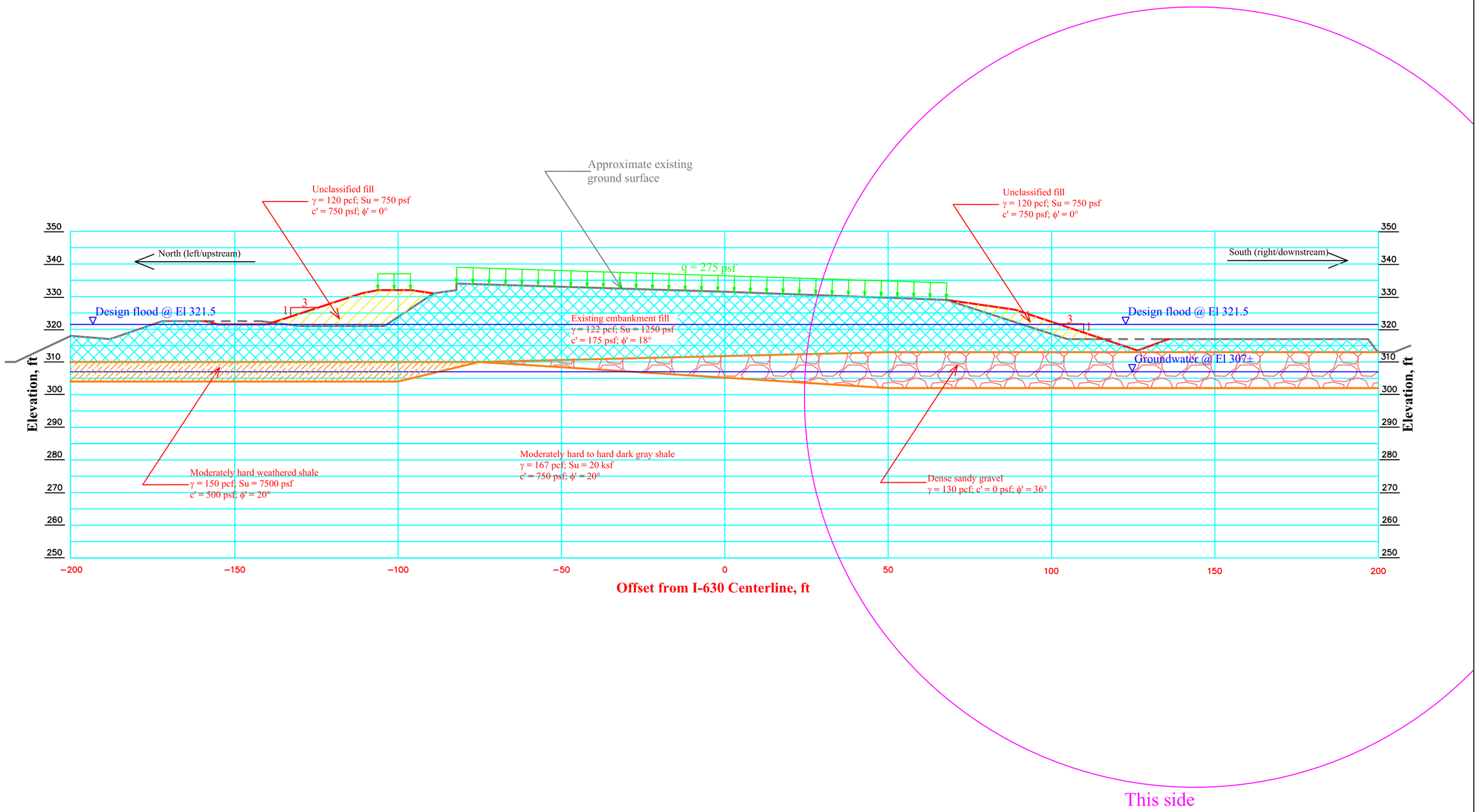
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
End Slope @ East Bridge Abutment – I-630 over Rock Creek

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



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
End Slope @ East Bridge Abutment – I-630 over Rock Creek

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



15 ft 0 15 30 ft

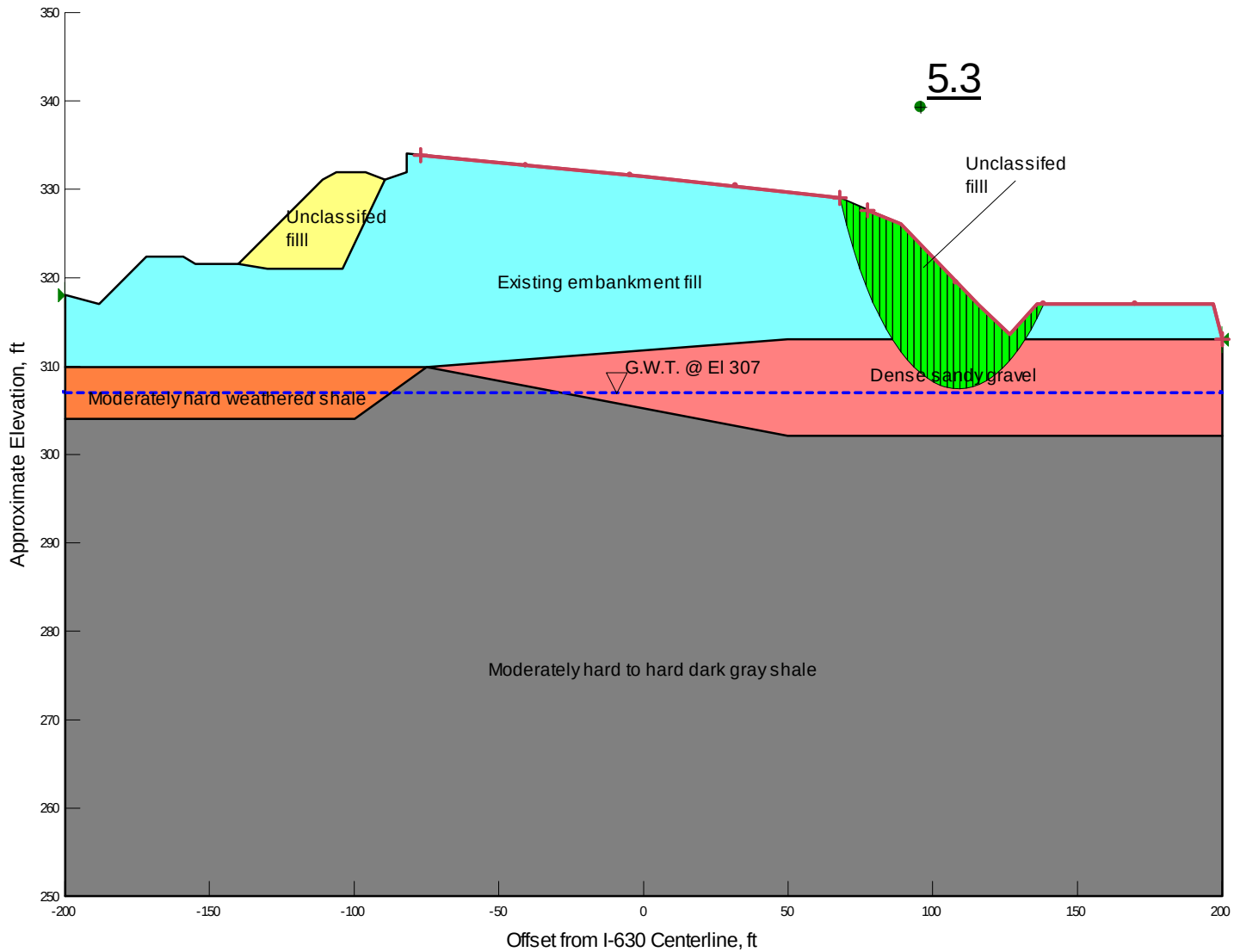
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
Side Slope at West Bridge Abutment - I-630 over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 10, 2015	Plate

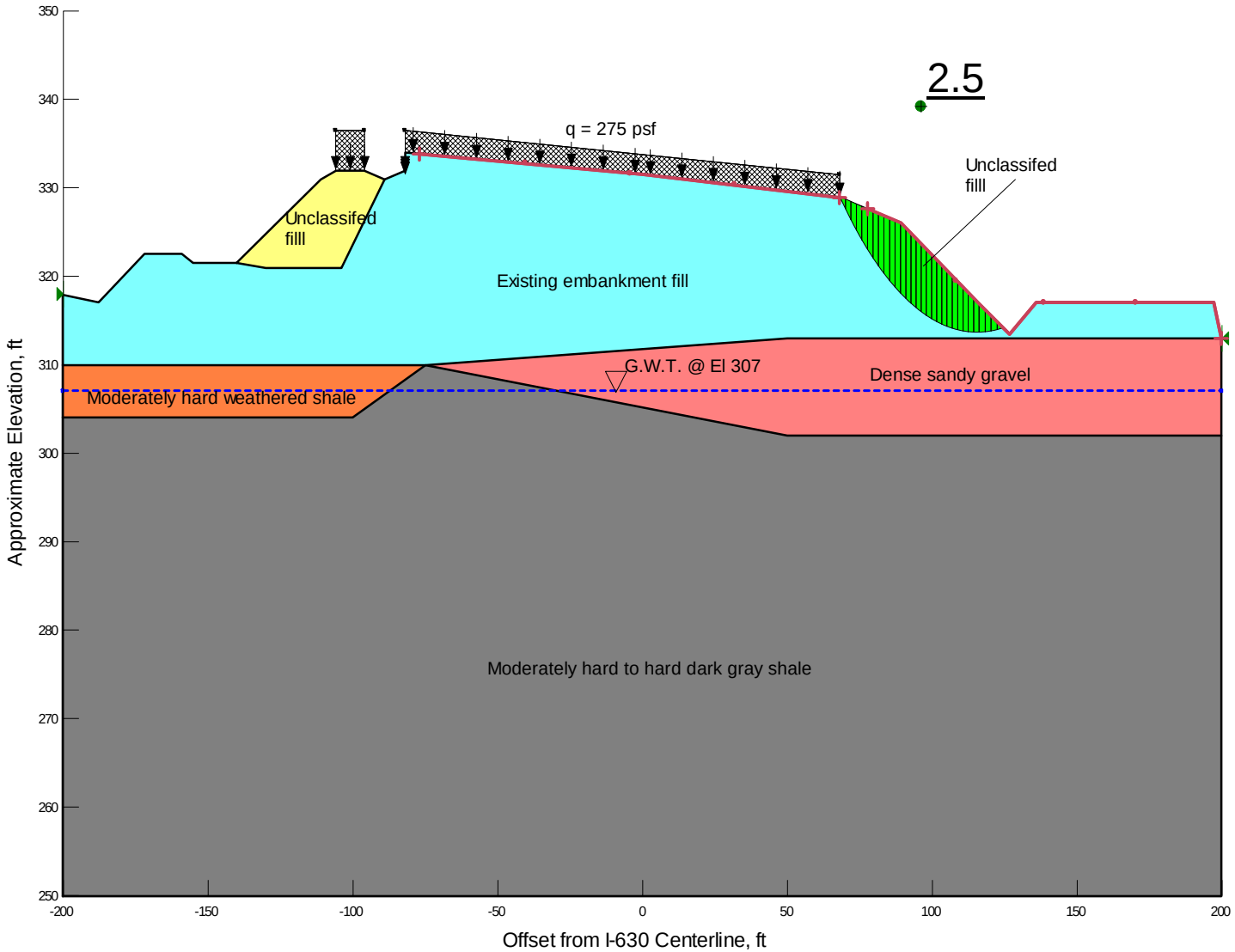
Results of Stability Analyses
Side Slope at West Bridge Abutment – I-630 over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	2.6
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.7
Rapid Drawdown	Drawdown from design flood to embankment toe	2.1



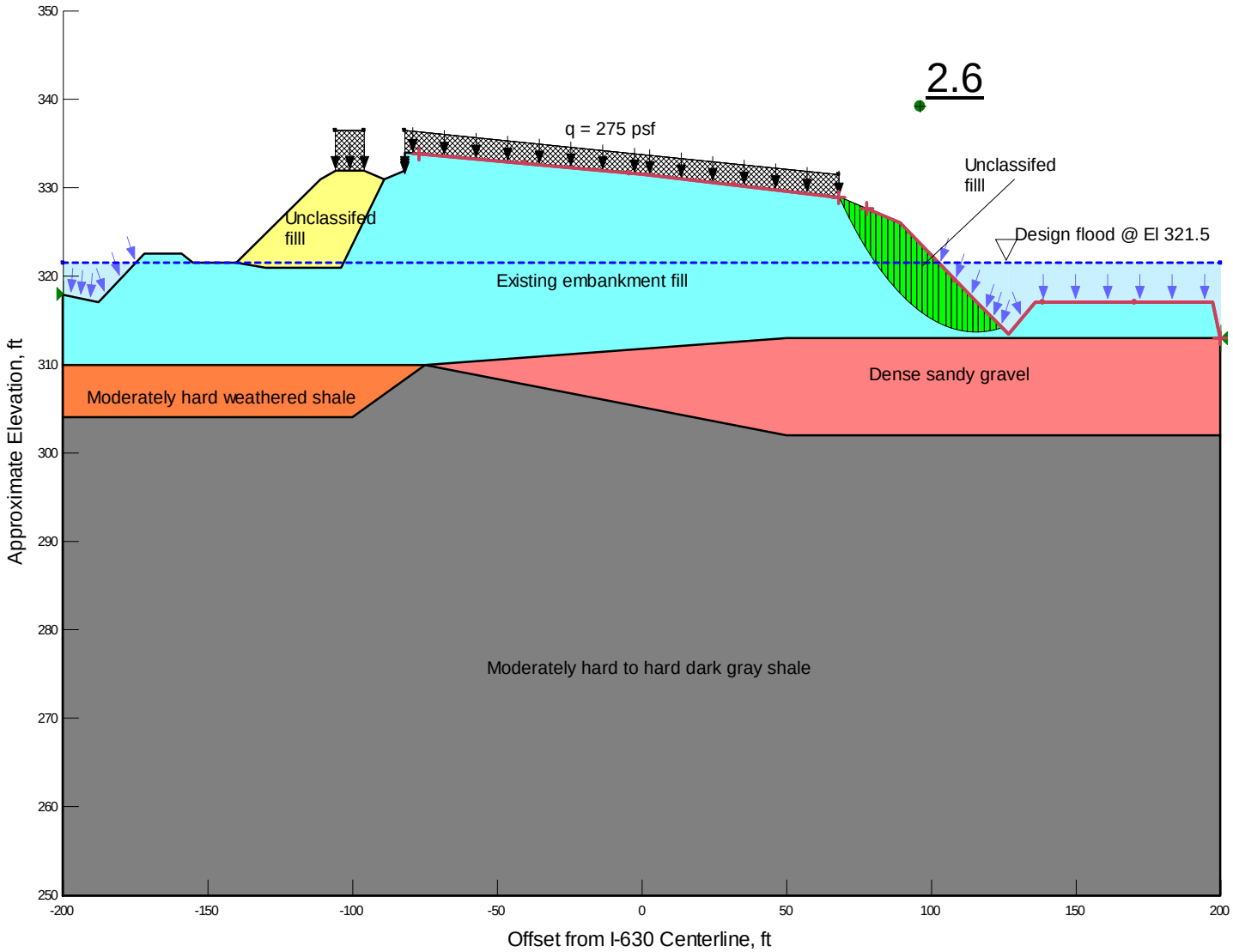
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
Side Slope @ West Bridge Abutment – I-630 over Rock Creek

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



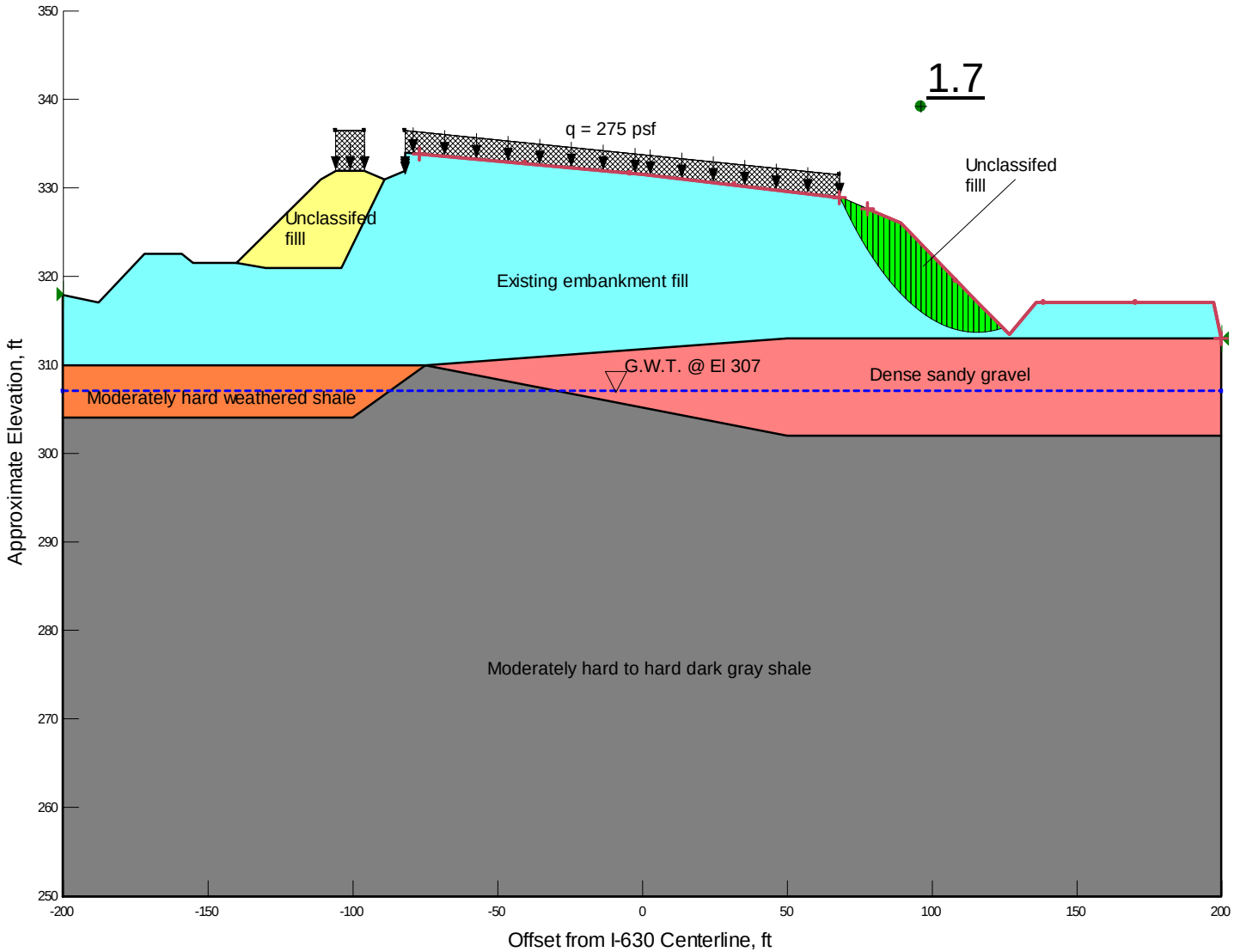
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
Side Slope @ West Bridge Abutment – I-630 over Rock Creek

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



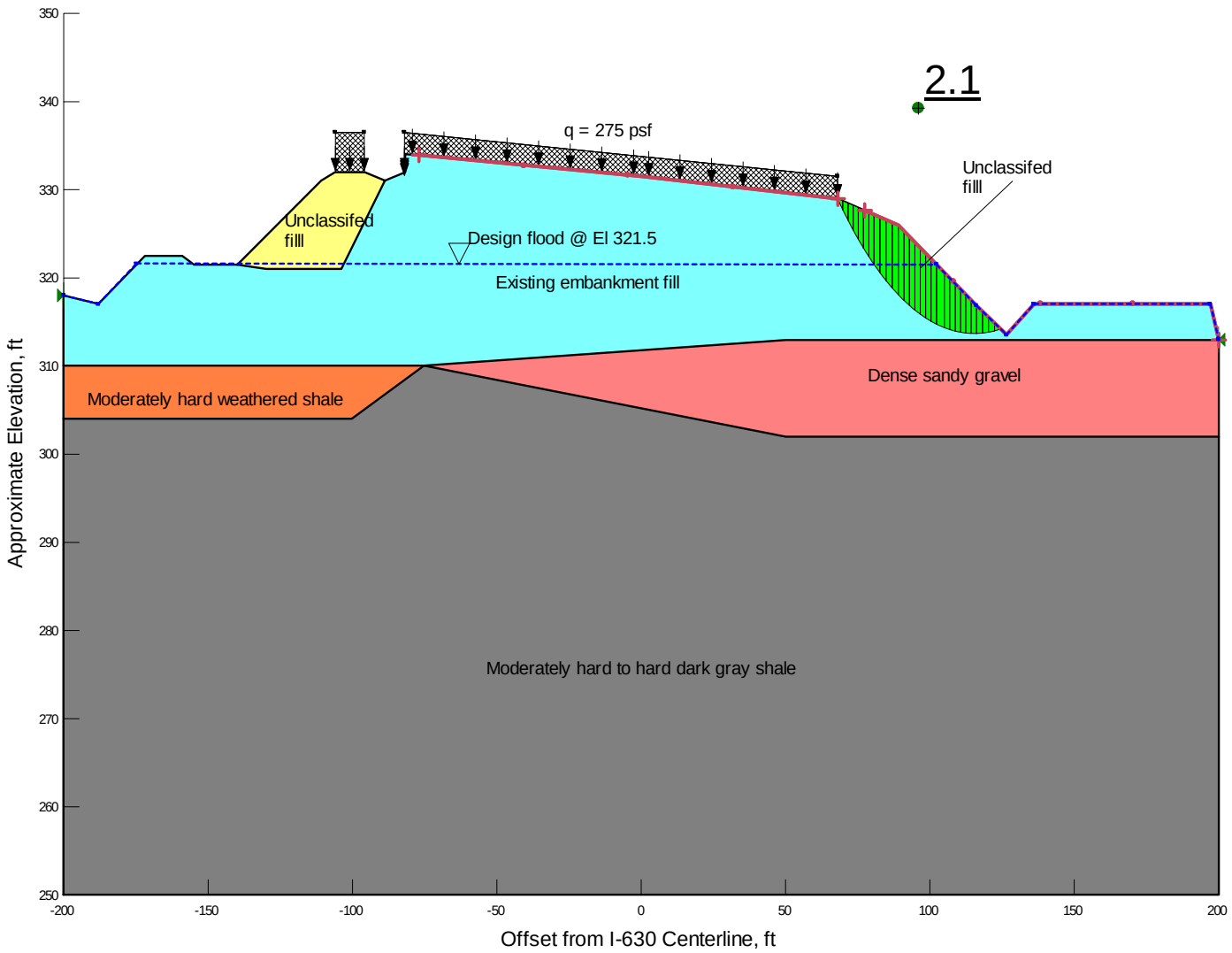
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
Side Slope @ West Bridge Abutment – I-630 over Rock Creek

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



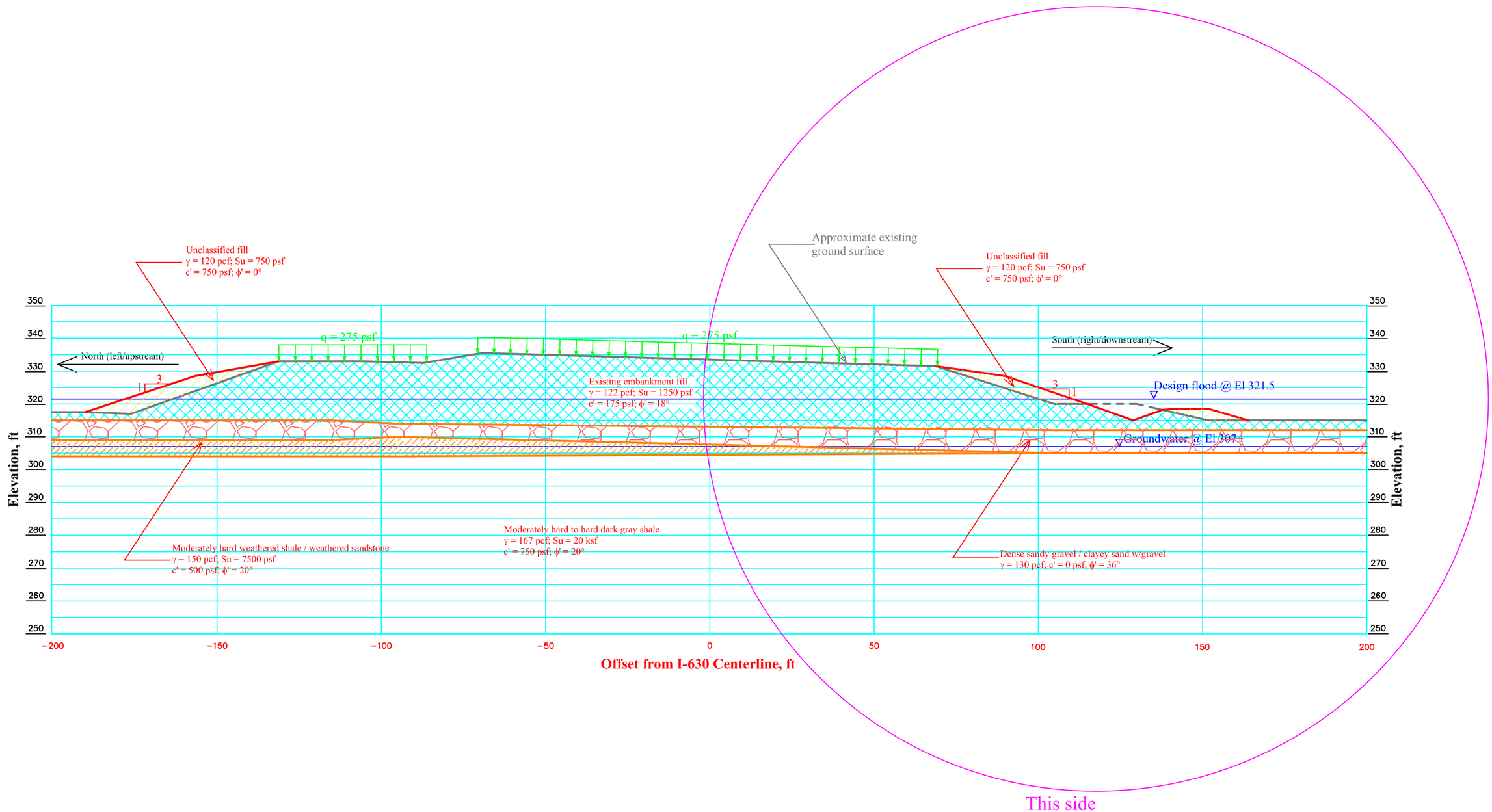
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
Side Slope @ West Bridge Abutment – I-630 over Rock Creek

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



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
Side Slope @ West Bridge Abutment – I-630 over Rock Creek

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



15 ft 0 15 30 ft

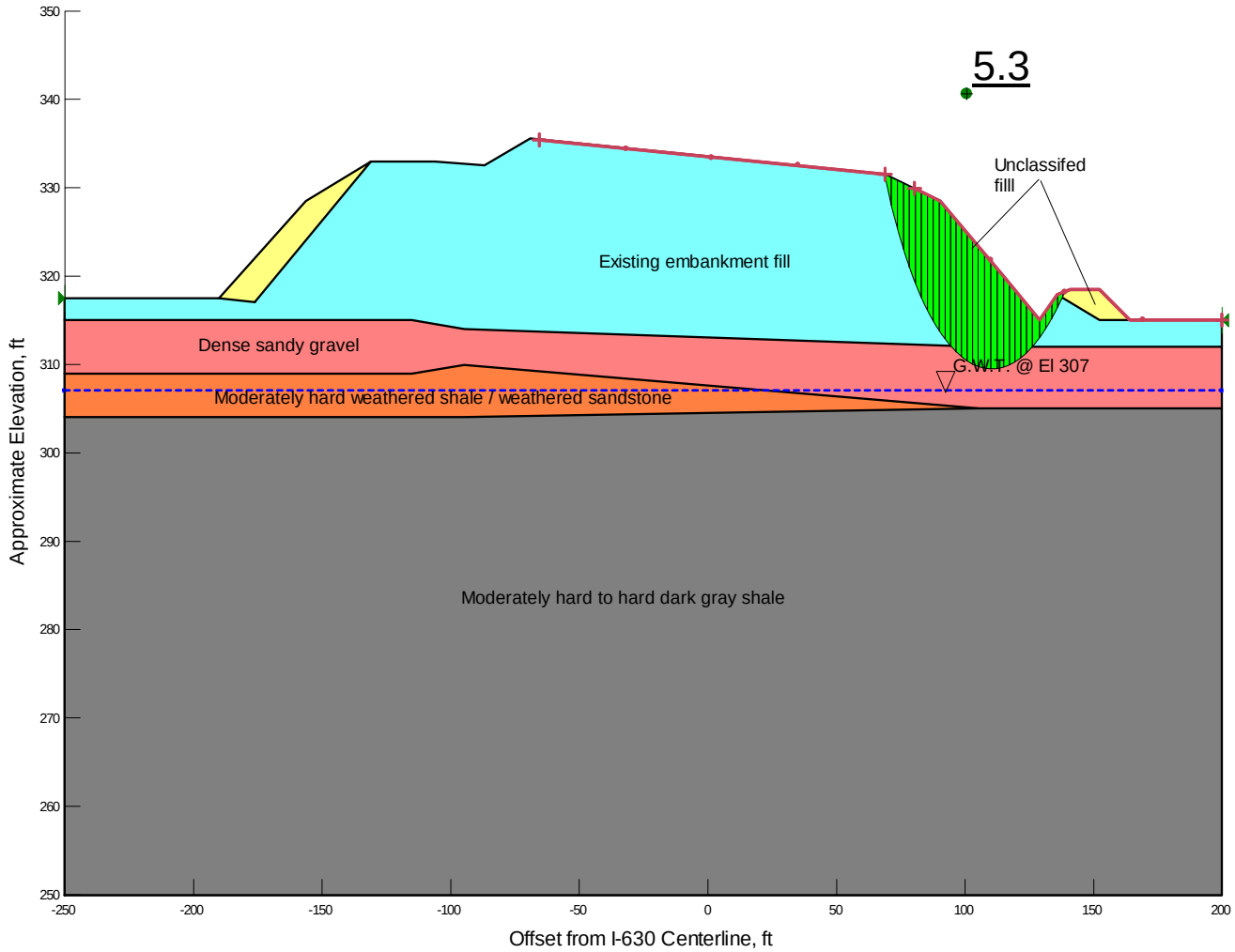
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
Side Slope at East Bridge Abutment - I-630 over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
February 11, 2015	Plate

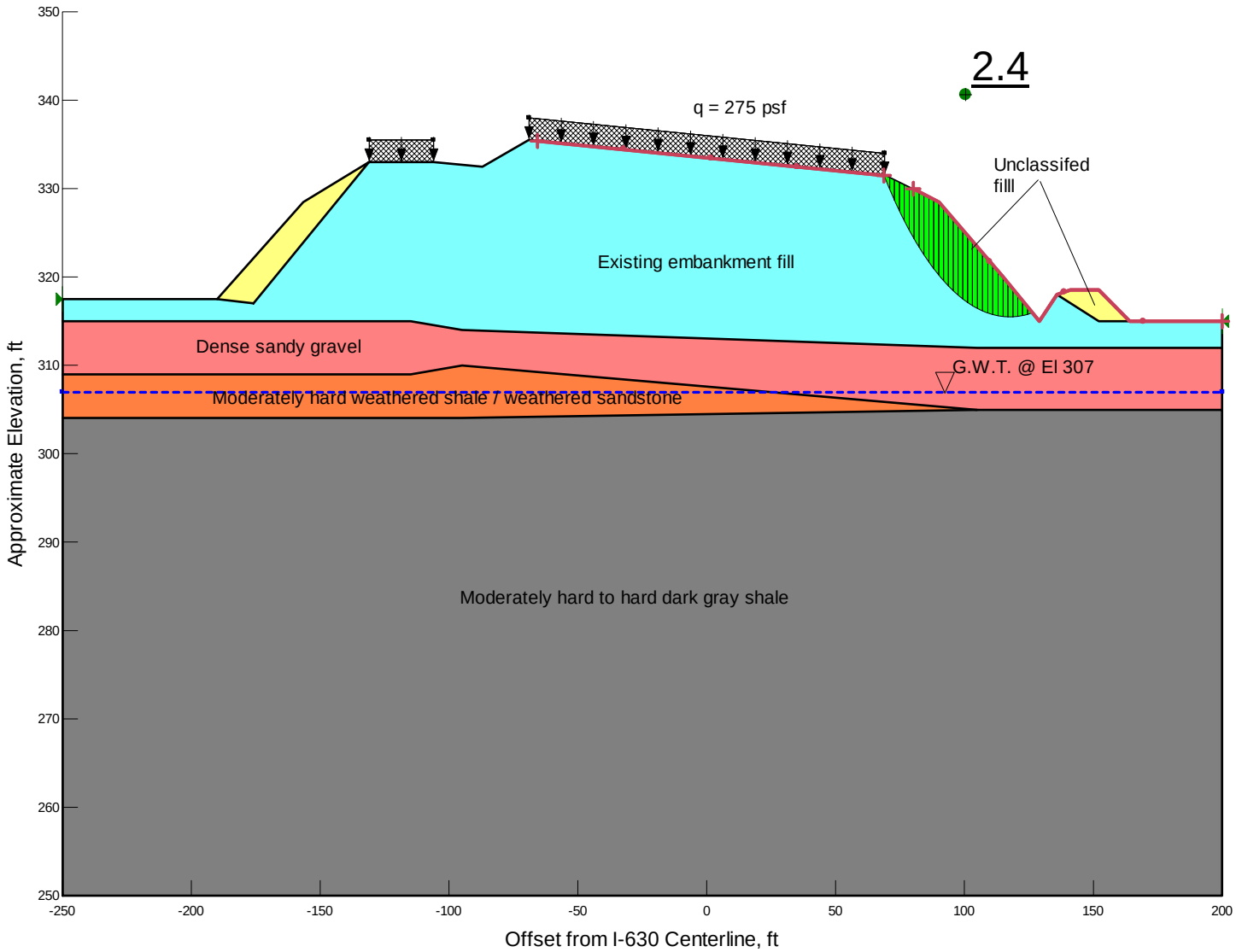
Results of Stability Analyses
Side Slope at East Bridge Abutment – I-630 over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.3
Long Term	Groundwater @ El 307±	2.4
	Design flood @ El 321.5	2.3
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.6
Rapid Drawdown	Drawdown from design flood to embankment toe	2.1



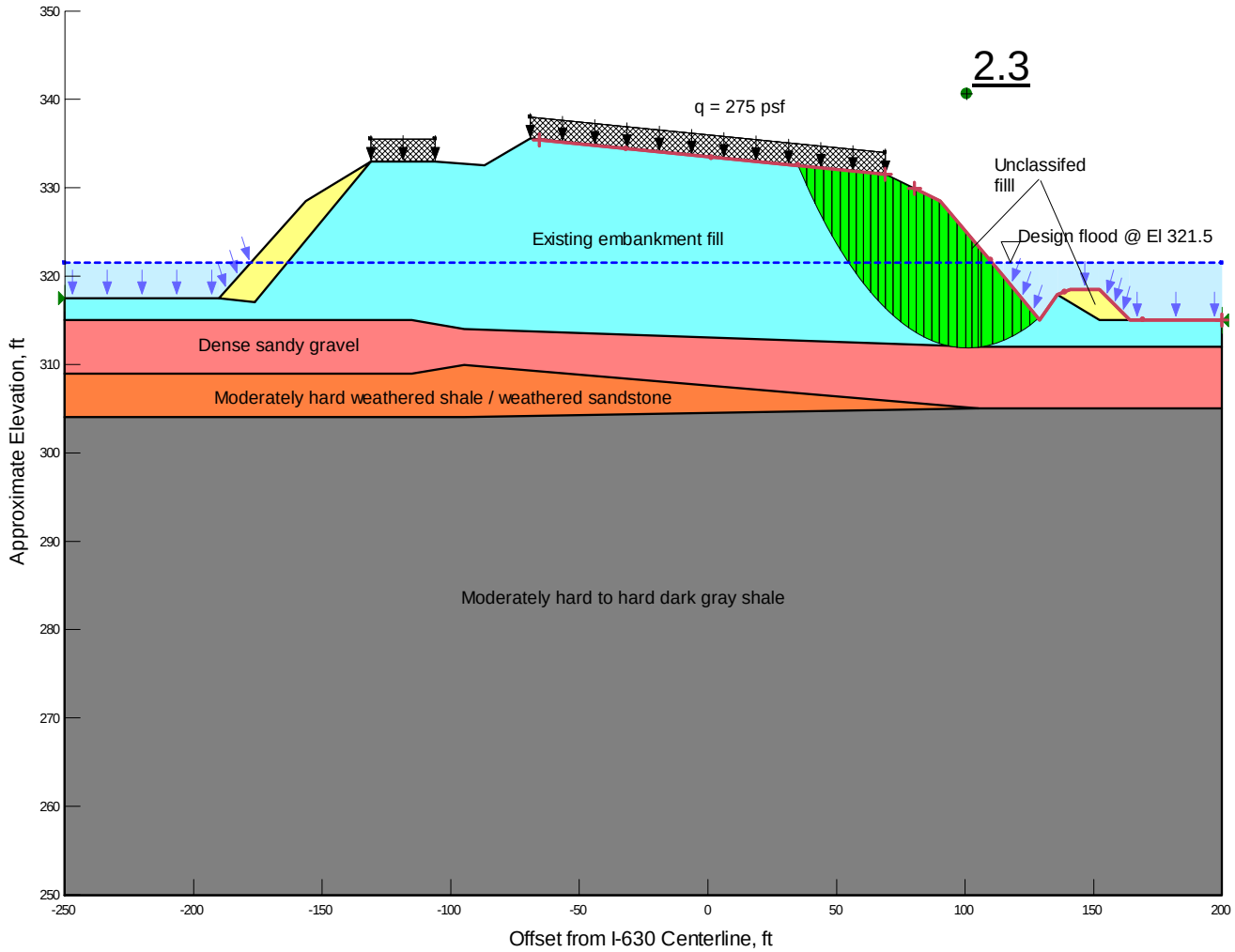
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – I-630 over Rock Creek

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



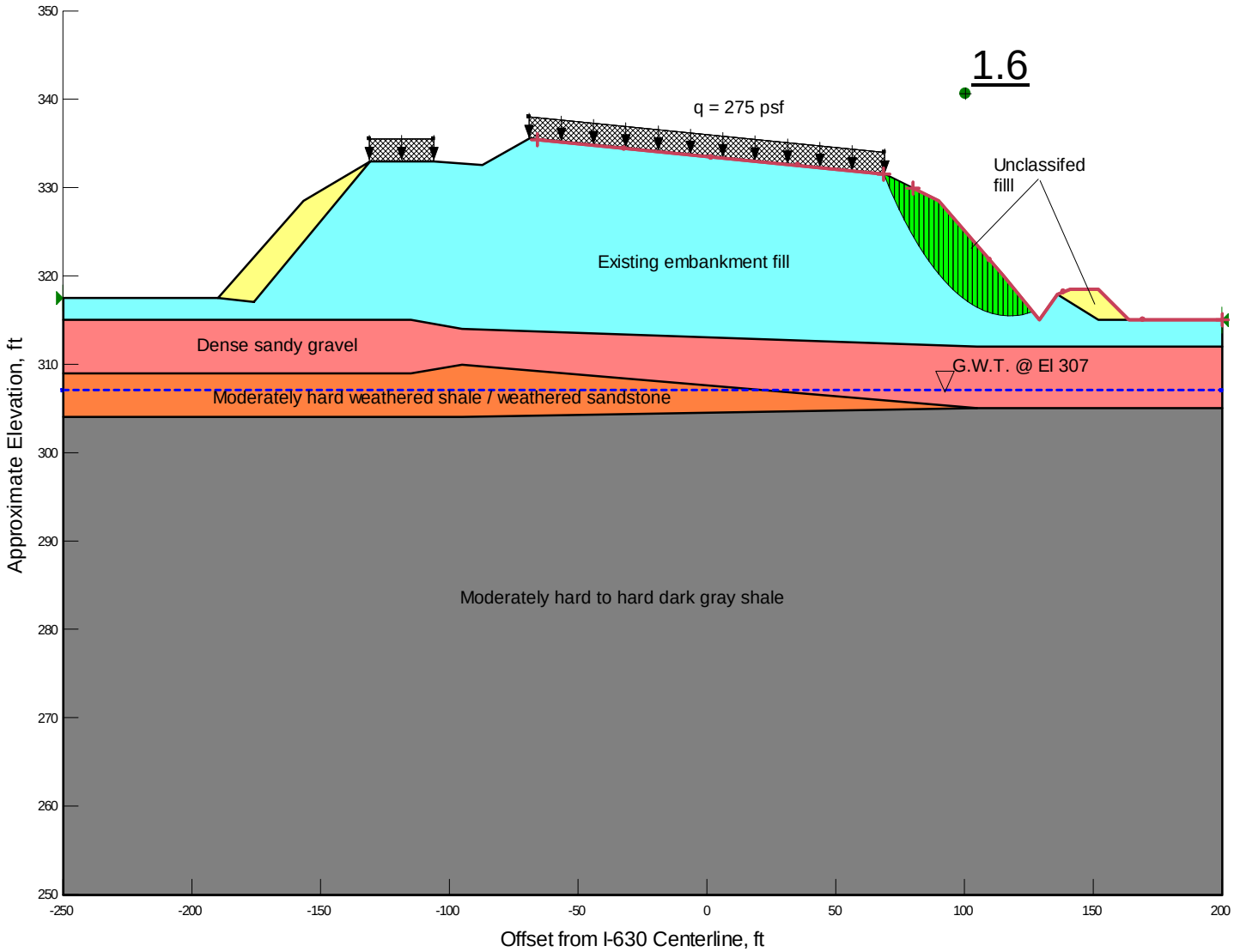
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – I-630 over Rock Creek

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



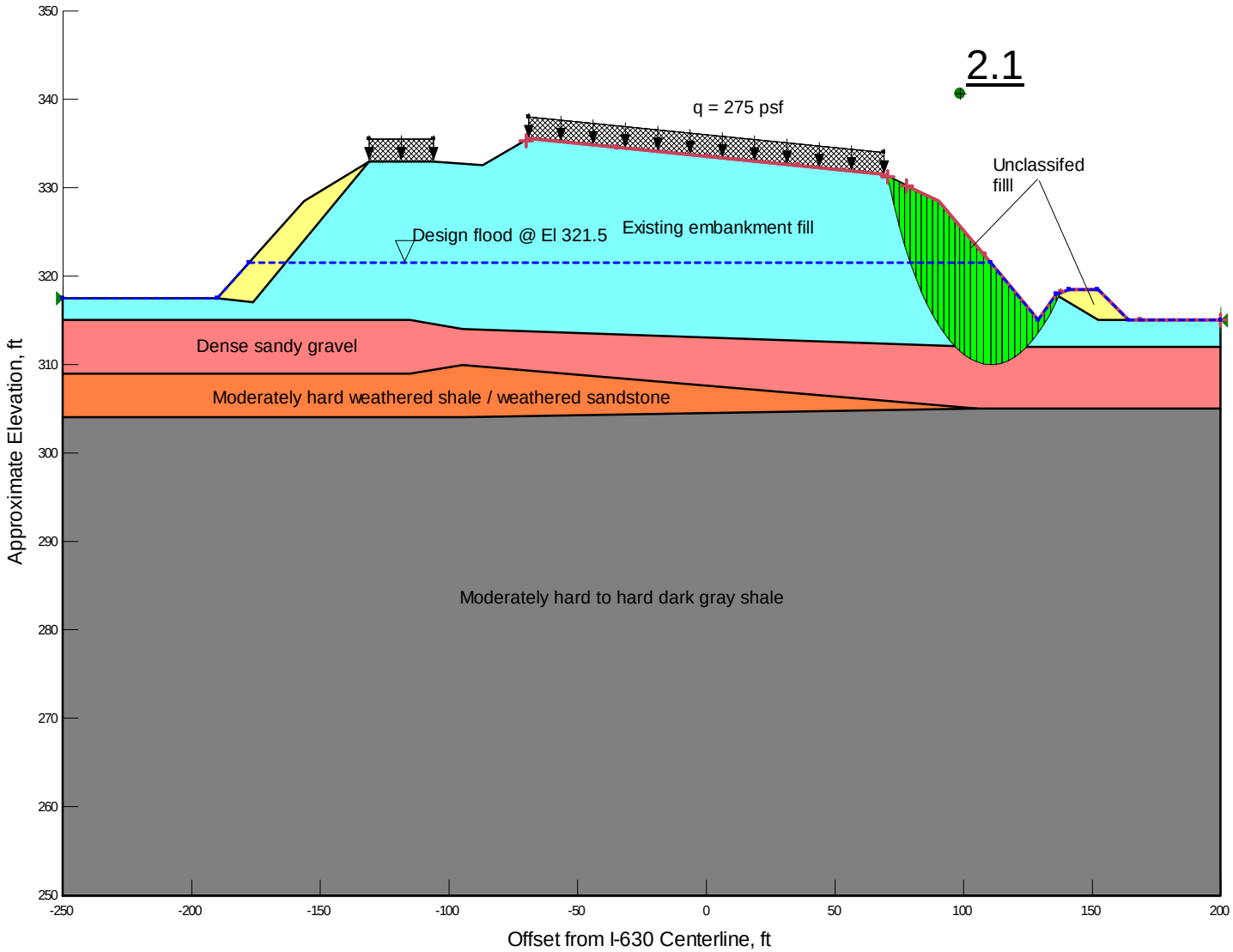
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
Side Slope @ East Bridge Abutment – I-630 over Rock Creek

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – I-630 over Rock Creek

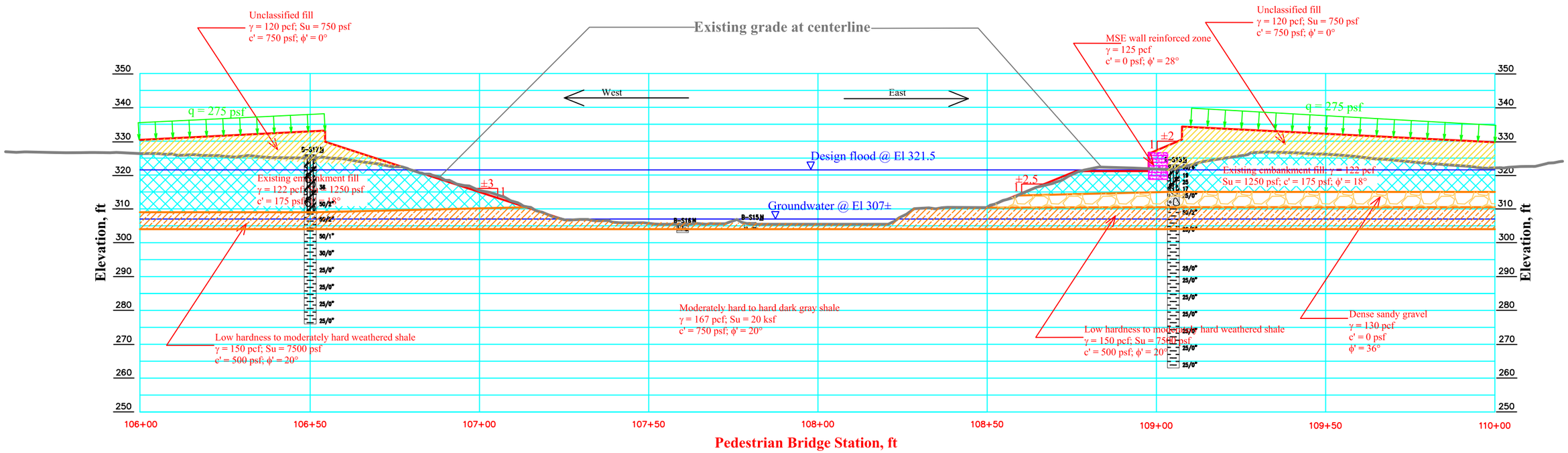
GRUBBS, HOSKYN, BARTON & WYATT, INC.
Consulting Engineers



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
Side Slope @ East Bridge Abutment – I-630 over Rock Creek

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

ATTACHMENT 15



Note: Section developed for purpose of stability analysis only, not for construction.

15 ft 0 15 30 ft

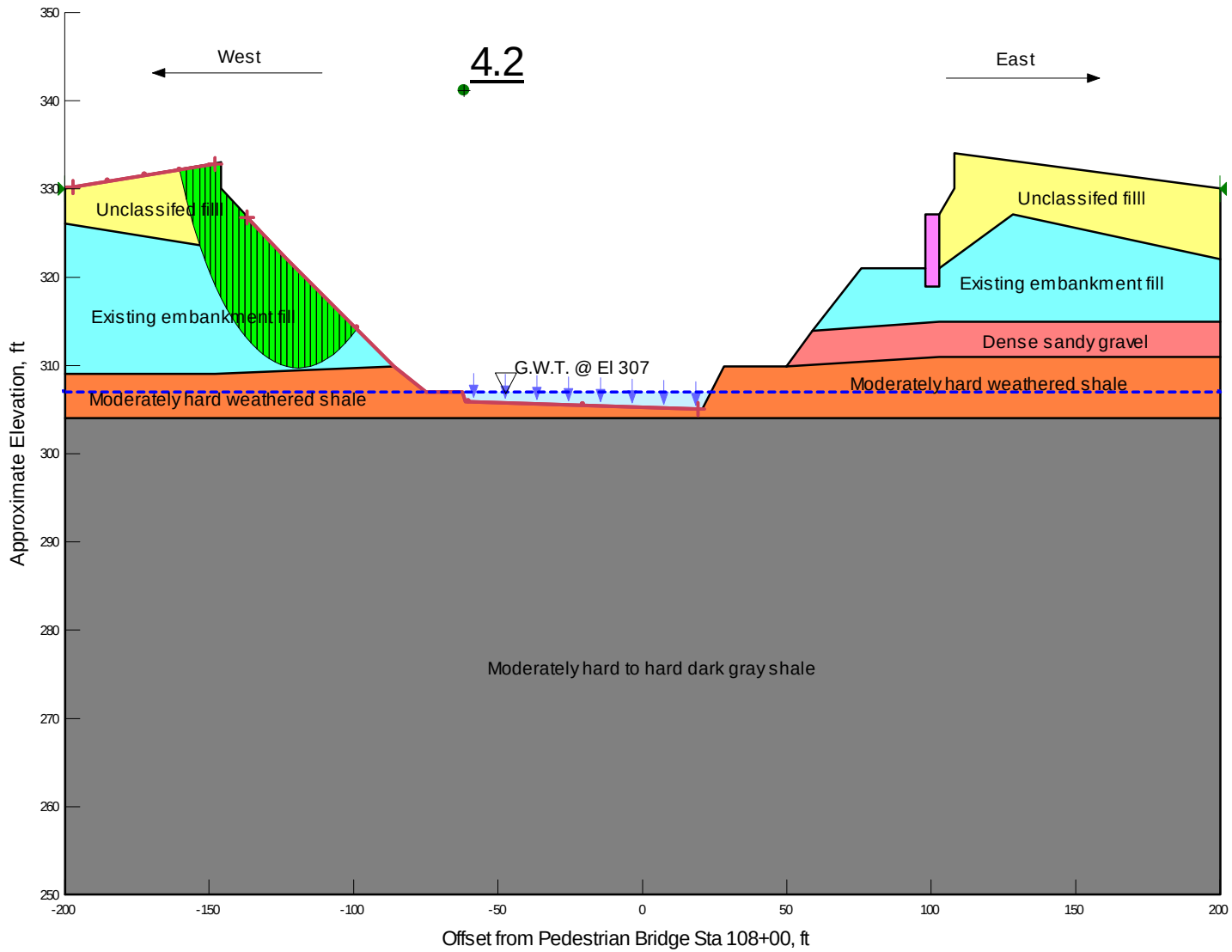
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
End Slopes @ Bridge Abutments - Pedestrian Bridge over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
March 2, 2015	Plate

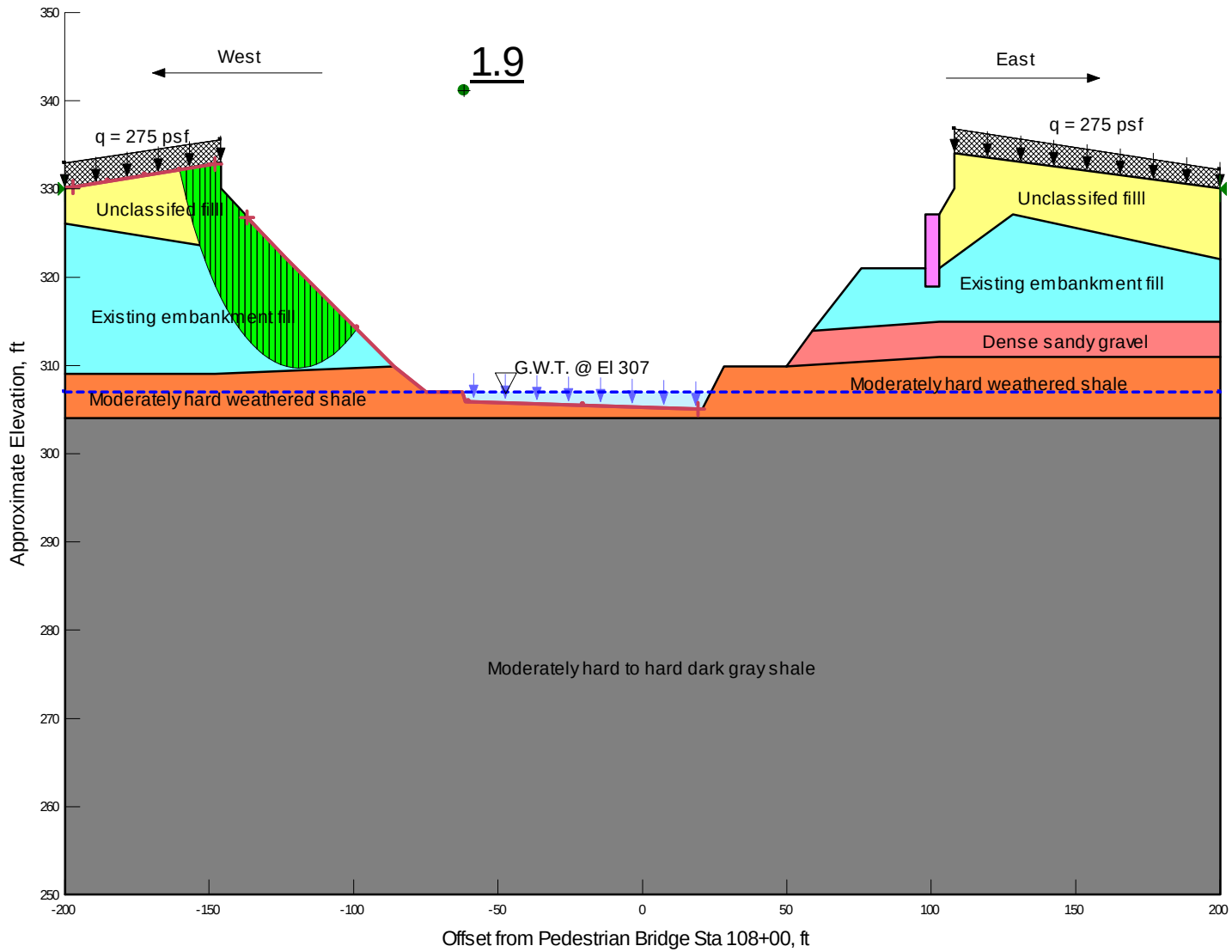
Results of Stability Analyses
End Slope at West Bridge Abutment – Pedestrian Bridge over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	4.2
Long Term	Groundwater @ El 307±	1.9
	Design flood @ El 321.5	1.7
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.4
Rapid Drawdown	Drawdown from design flood to embankment toe	1.5



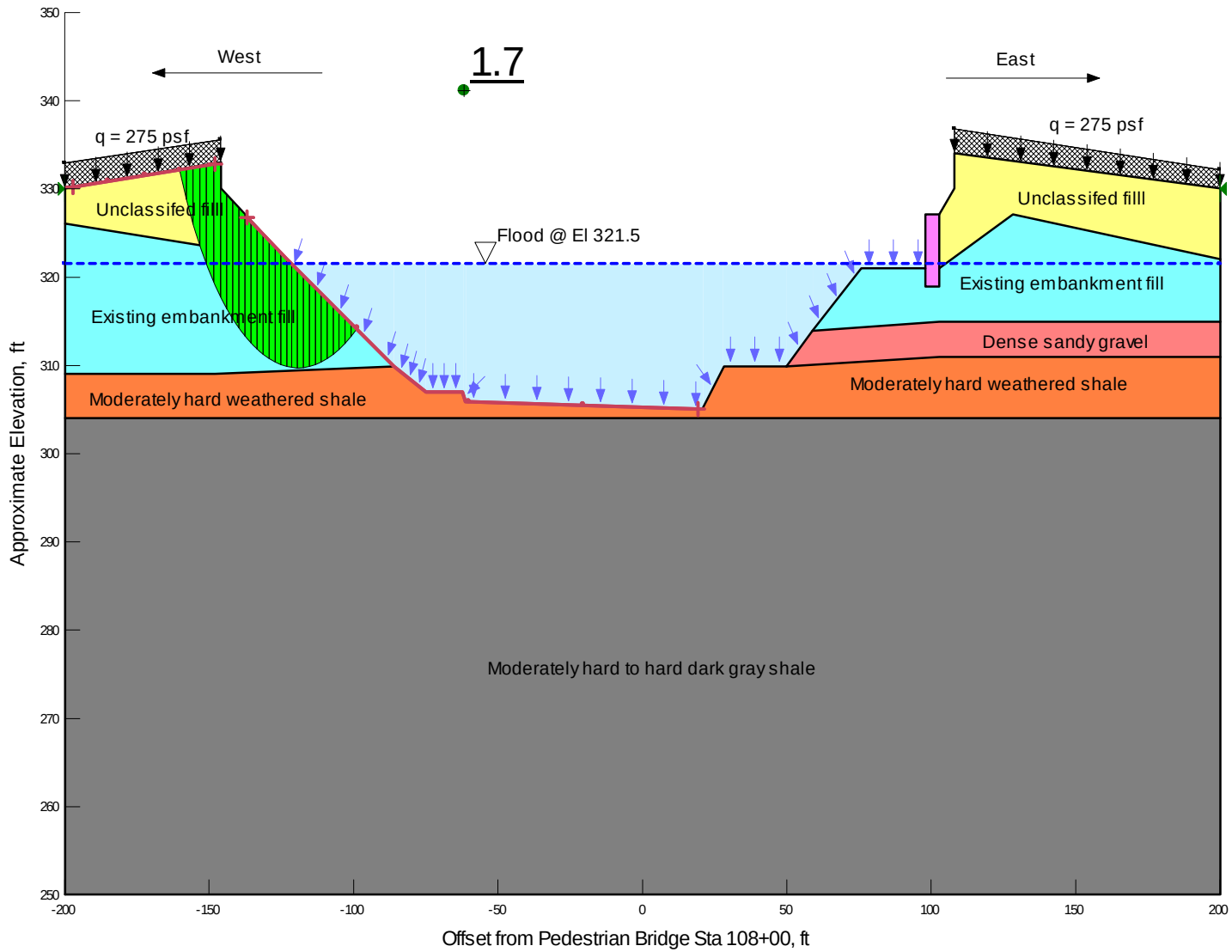
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
End Slope @ West Bridge Abutment – Pedestrain Bridge over Rock Creek

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



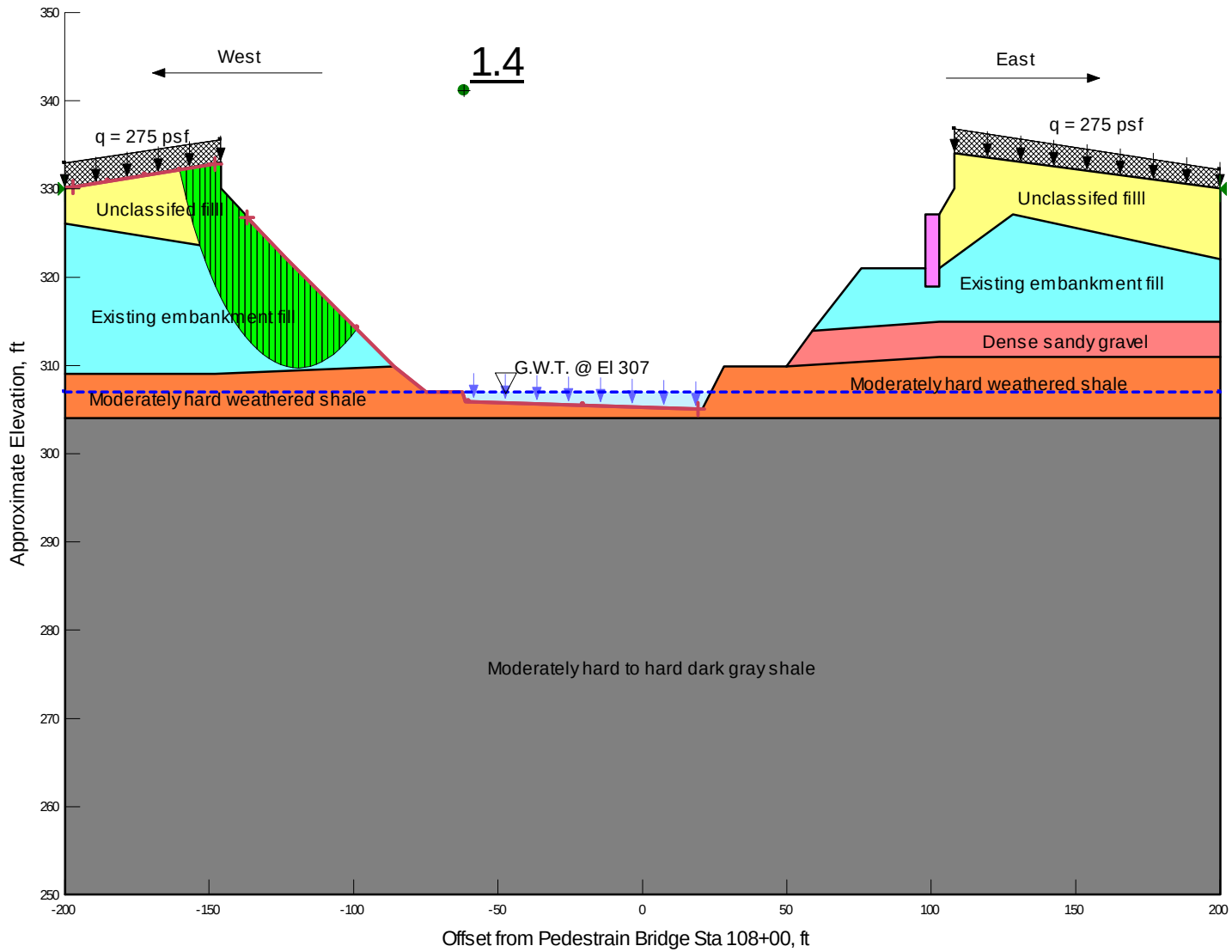
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
End Slope @ West Bridge Abutment – Pedestrain Bridge over Rock Creek

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



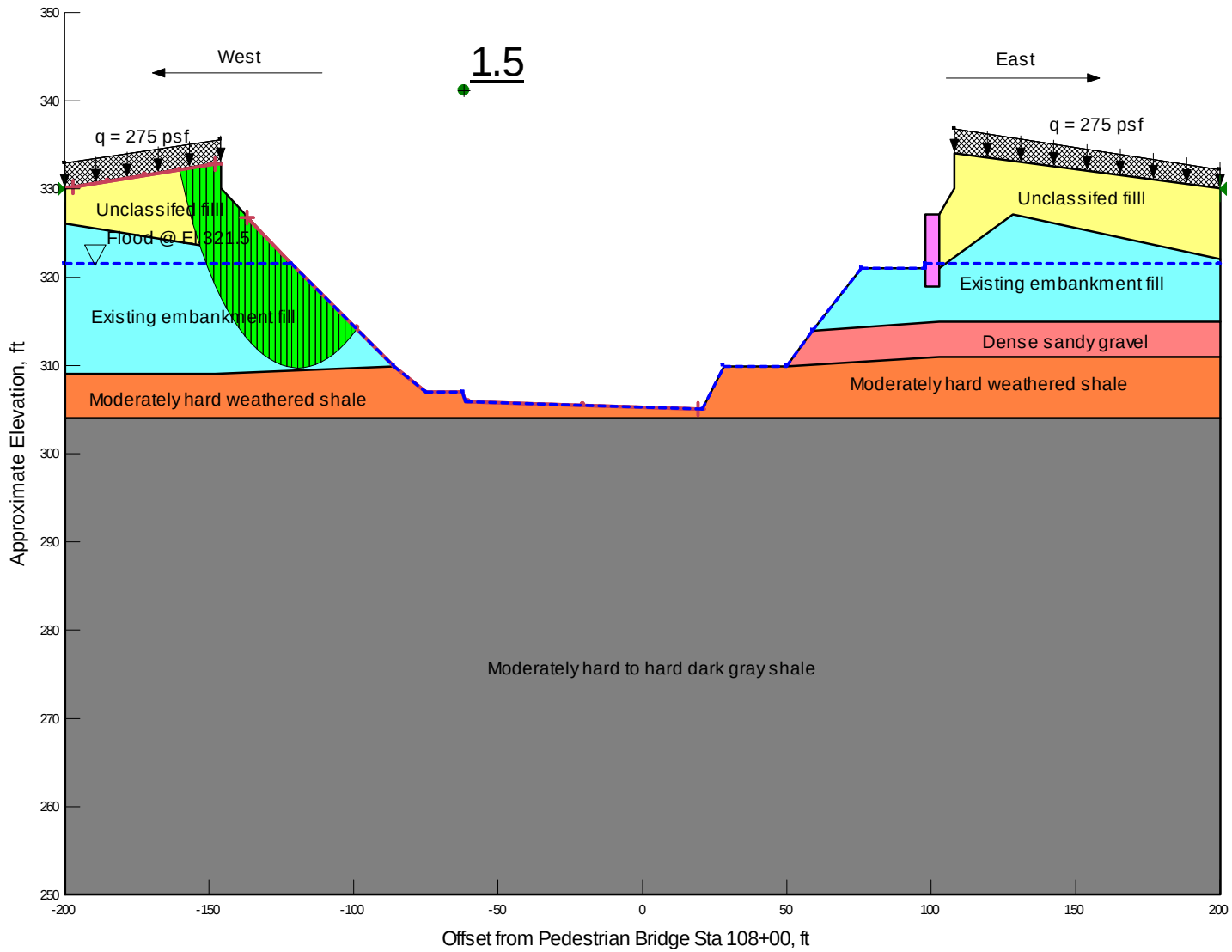
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
End Slope @ West Bridge Abutment – Pedestrain Bridge over Rock Creek

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
End Slope @ West Bridge Abutment – Pedestrain Bridge over Rock Creek

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

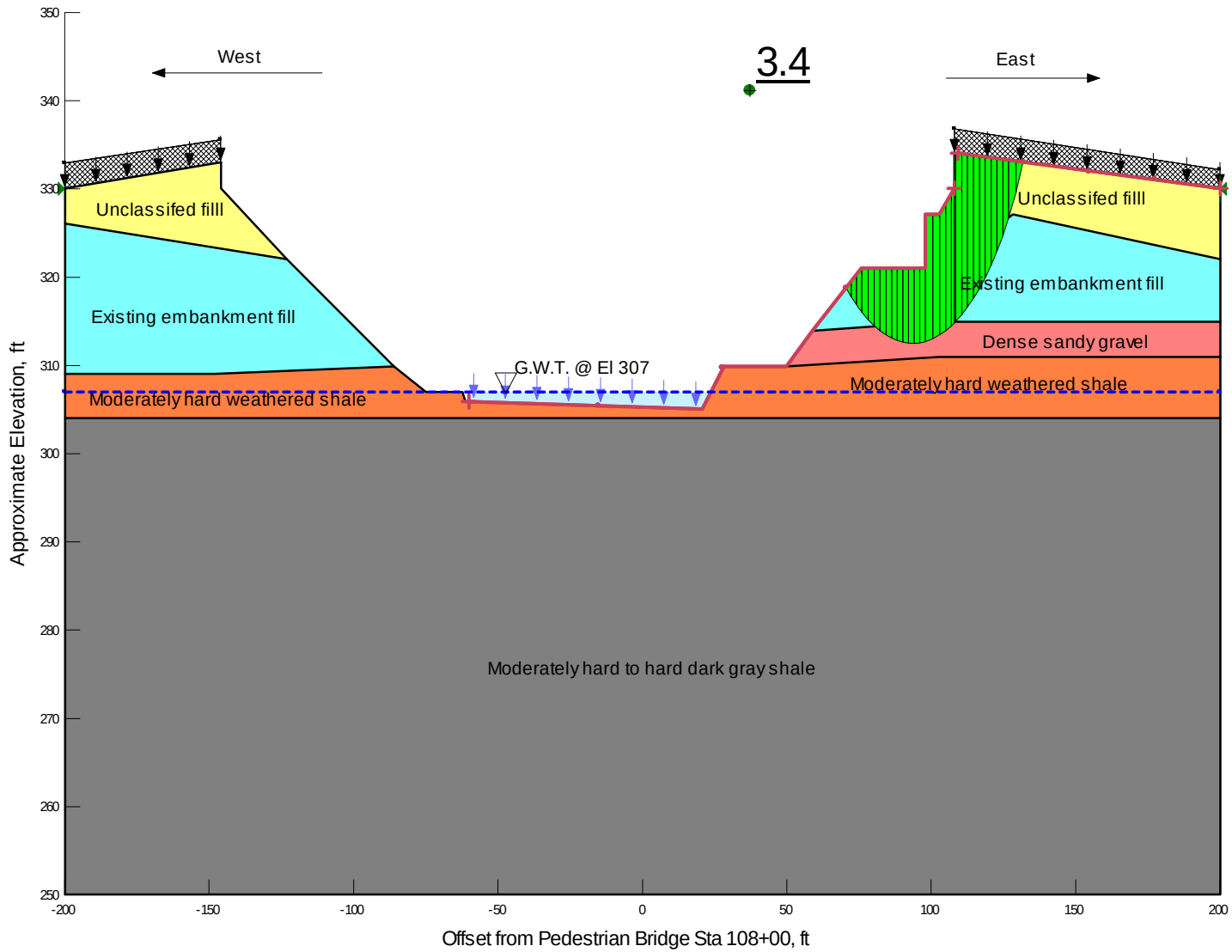


Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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

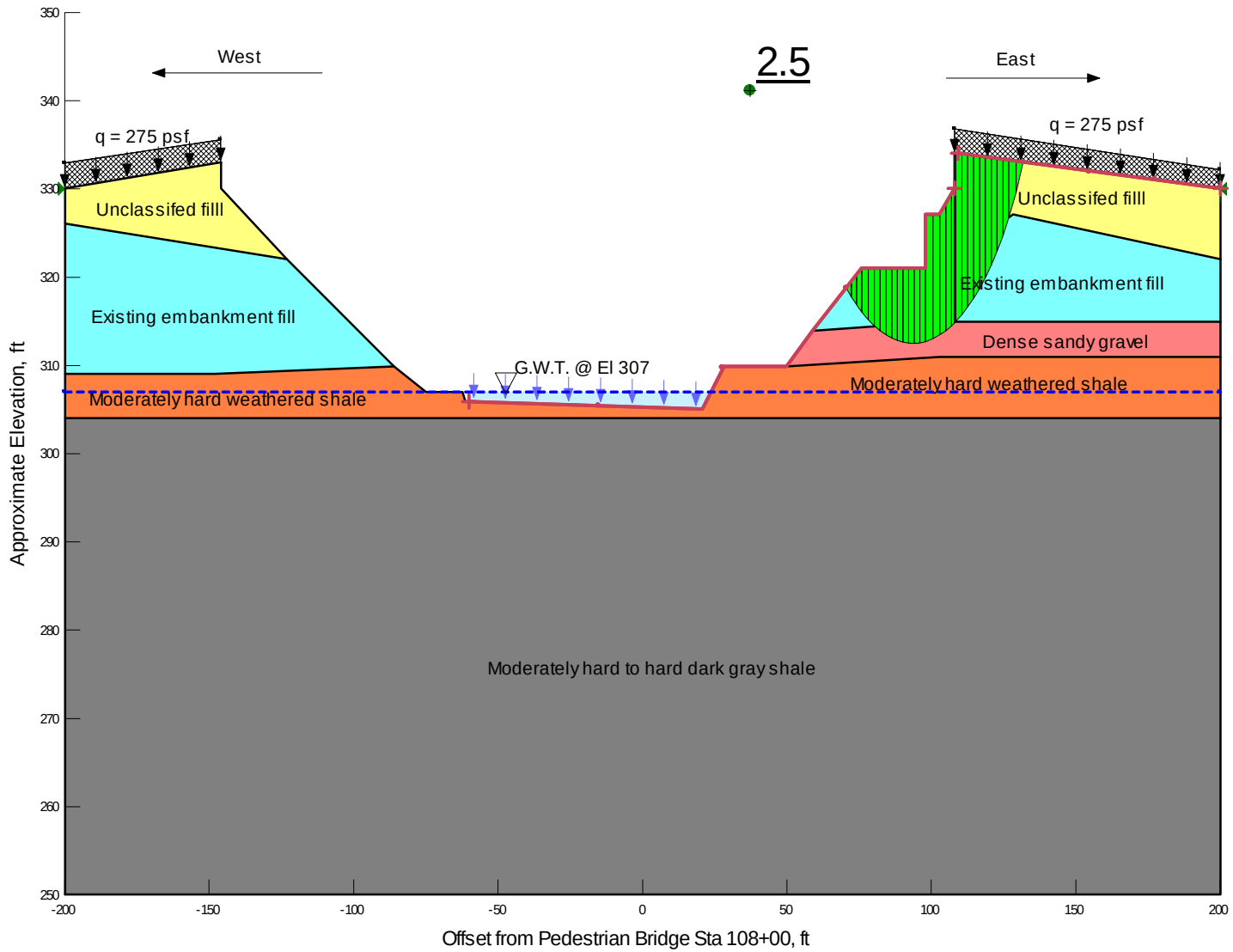
Results of Stability Analyses
End Slope at East Bridge Abutment – Pedestrian Bridge over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	3.4
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	1.9
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.8
Rapid Drawdown	Drawdown from design flood to embankment toe	1.9



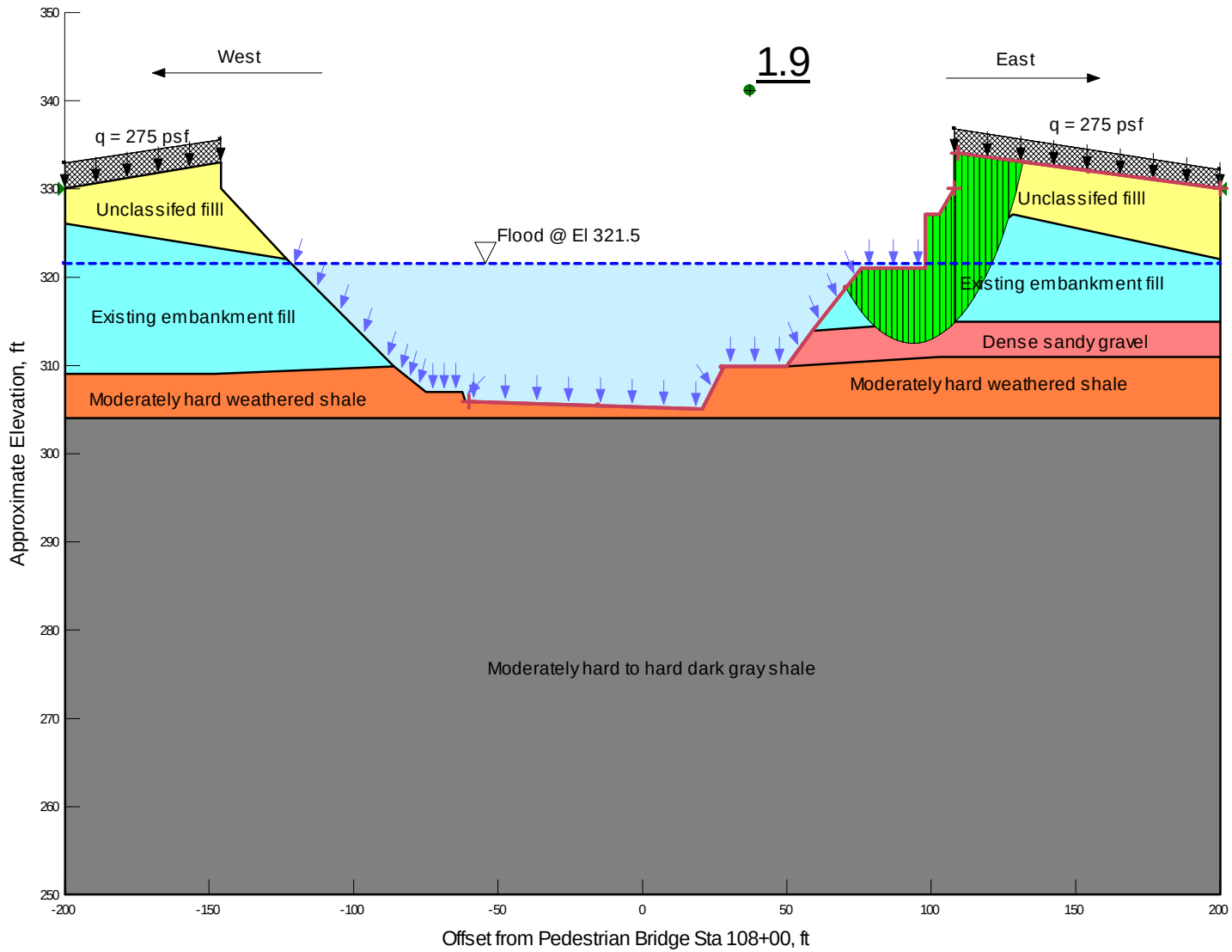
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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



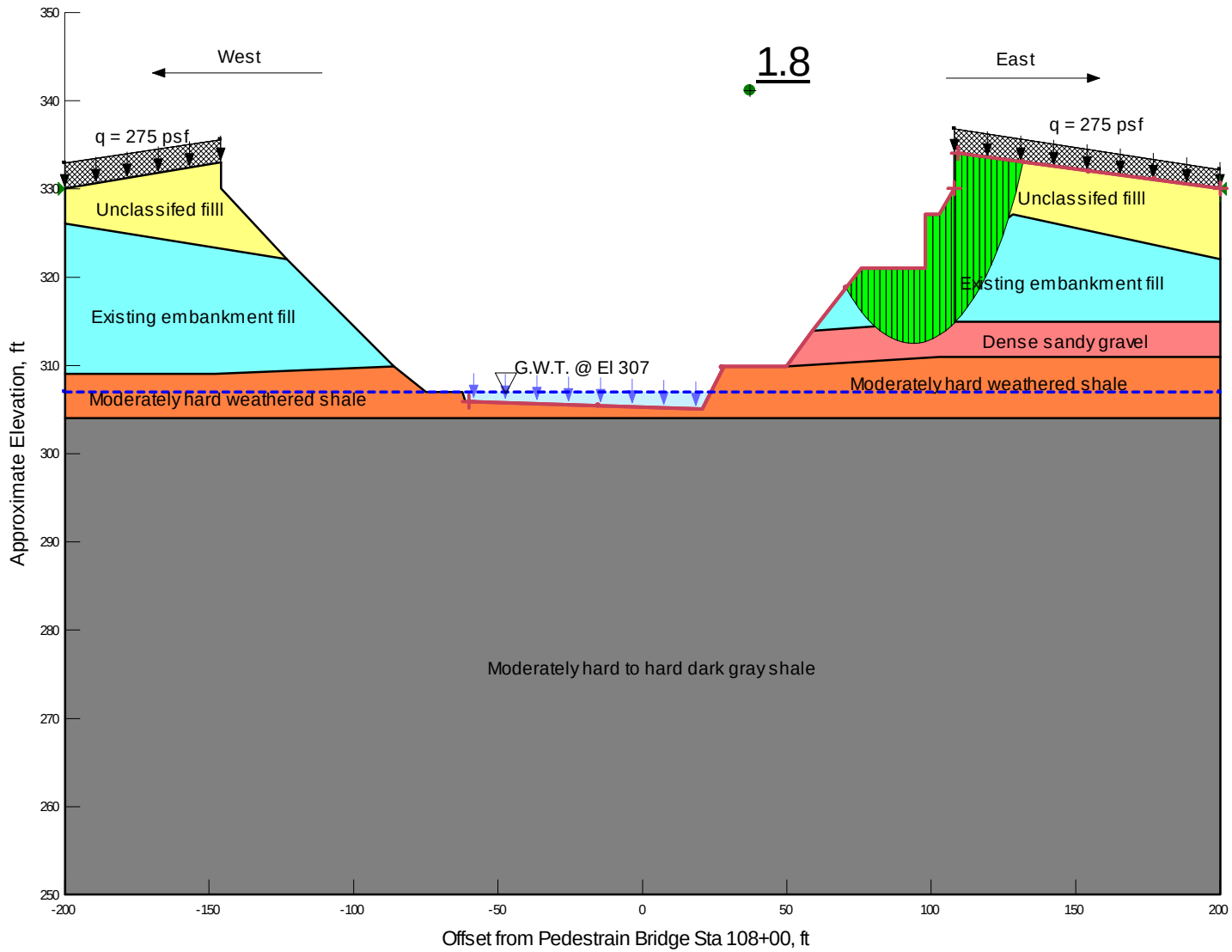
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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



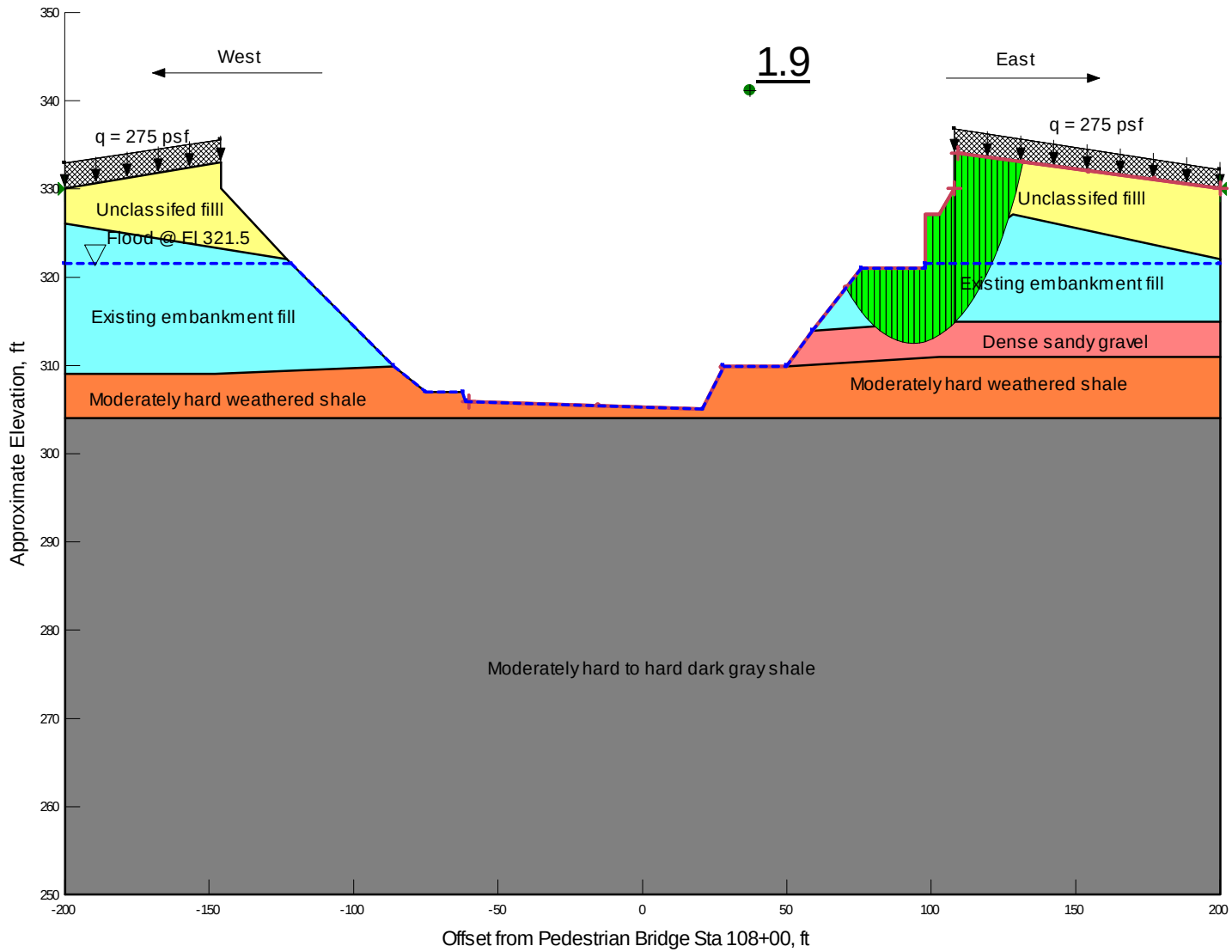
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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



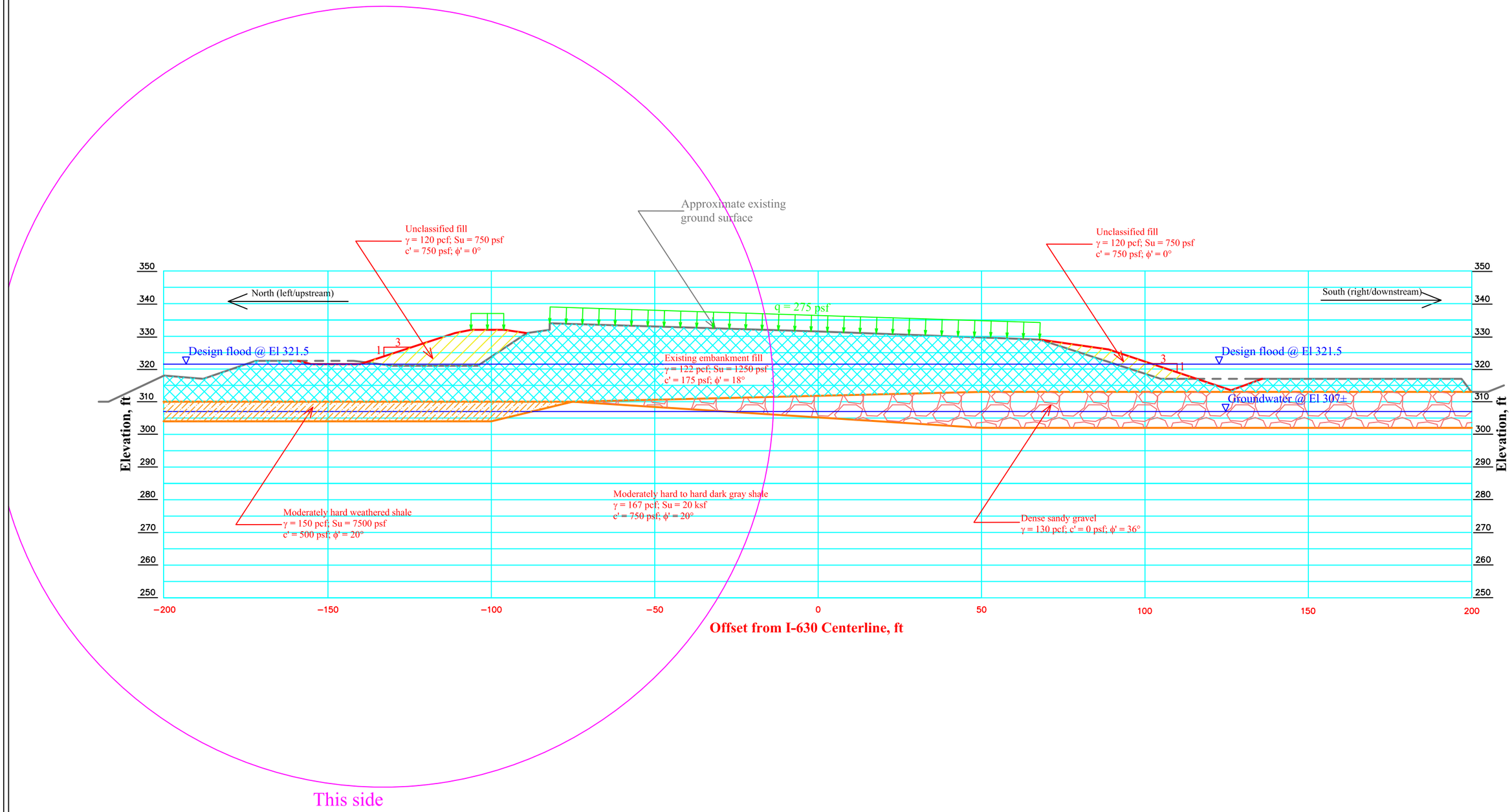
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
End Slope @ East Bridge Abutment – Pedestrain Bridge over Rock Creek

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



15 ft 0 15 30 ft

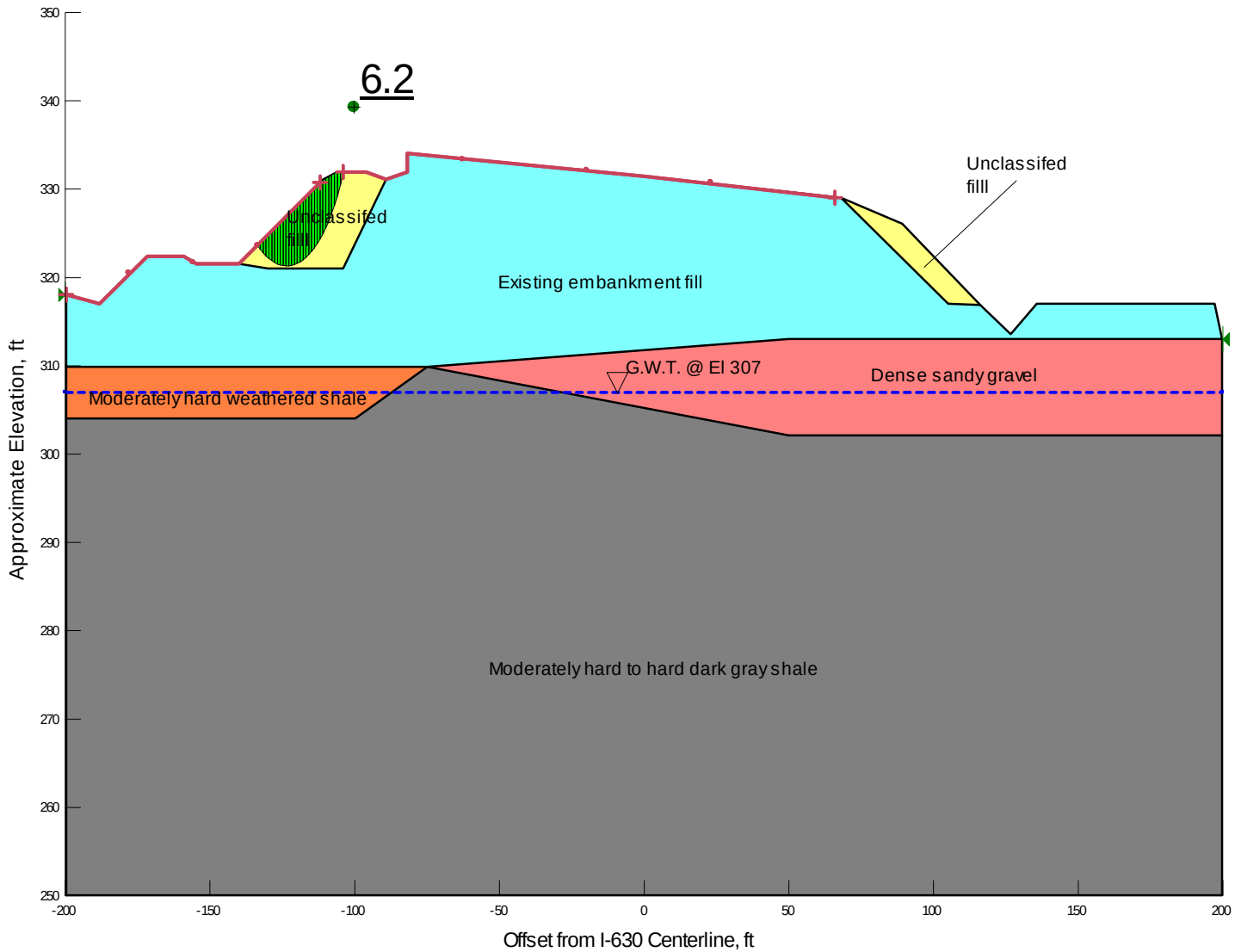
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
Side Slope at West Bridge Abutment - Pedestrian Bridge over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
March 2, 2015	Plate

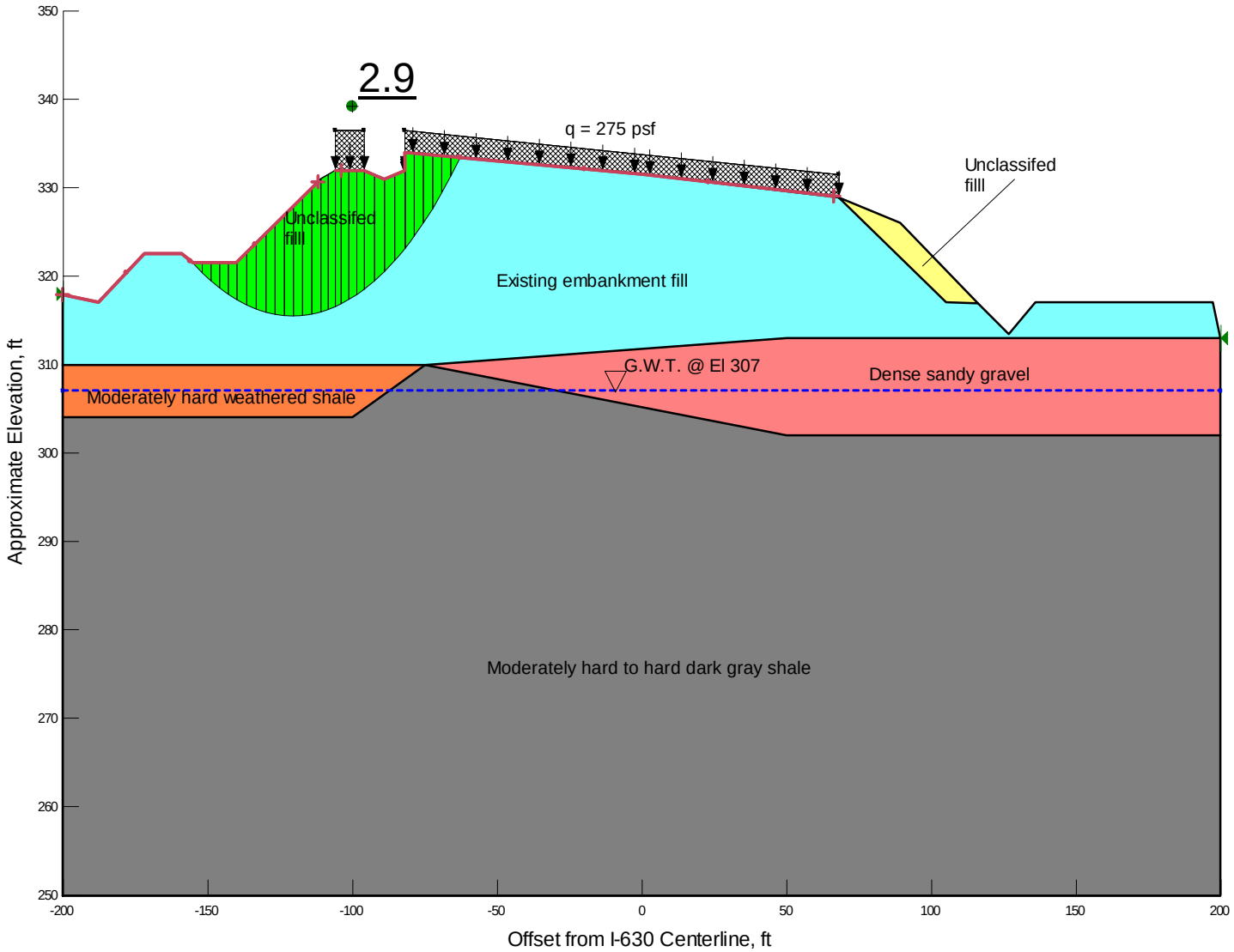
Results of Stability Analyses
Side Slope at West Bridge Abutment – Pedestrian Bridge over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	6.2
Long Term	Groundwater @ El 307±	2.9
	Design flood @ El 321.5	2.6
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.8
Rapid Drawdown	Drawdown from design flood to embankment toe	2.6



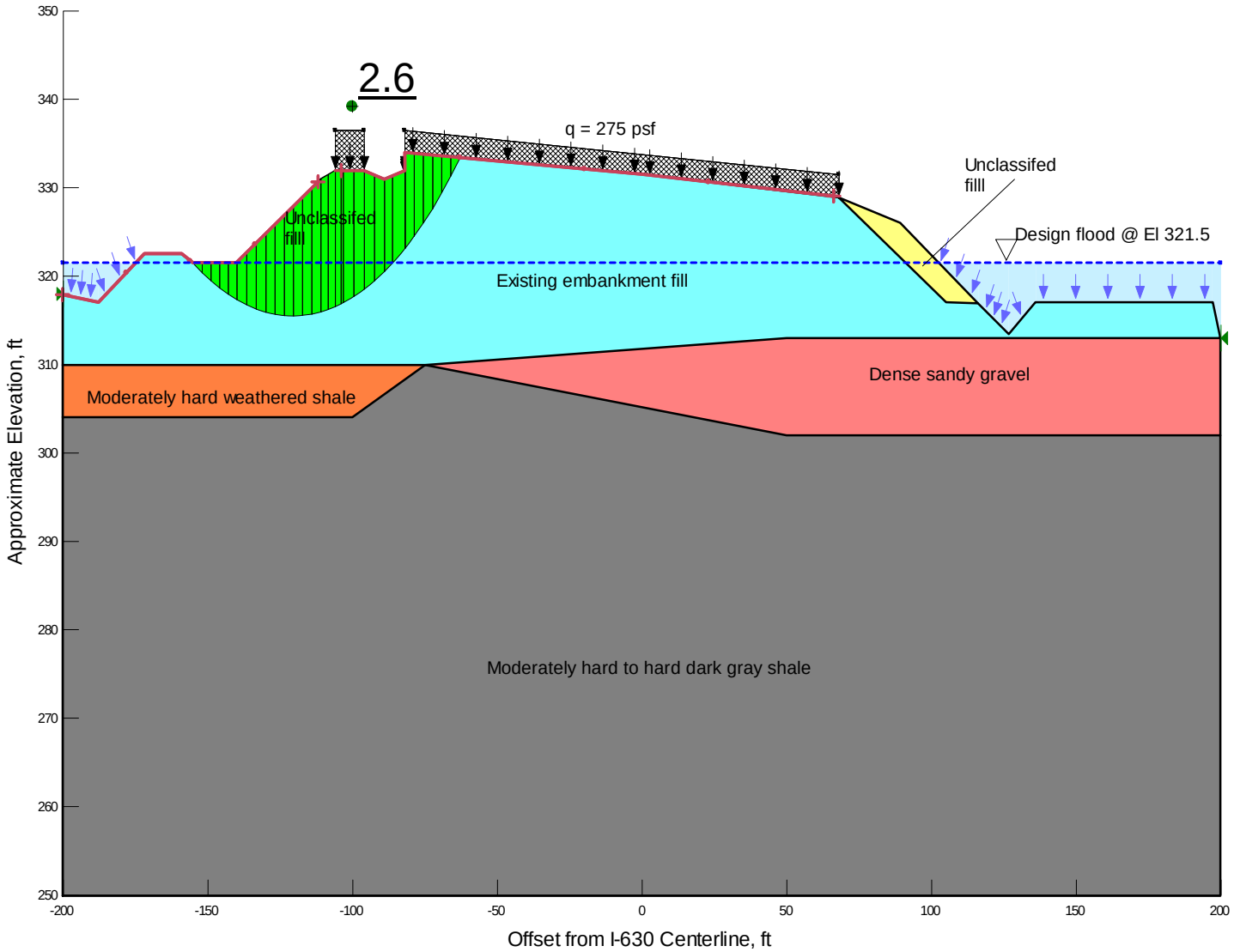
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
Side Slope @ West Bridge Abutment –Pedestrian Bridge over Rock Creek

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



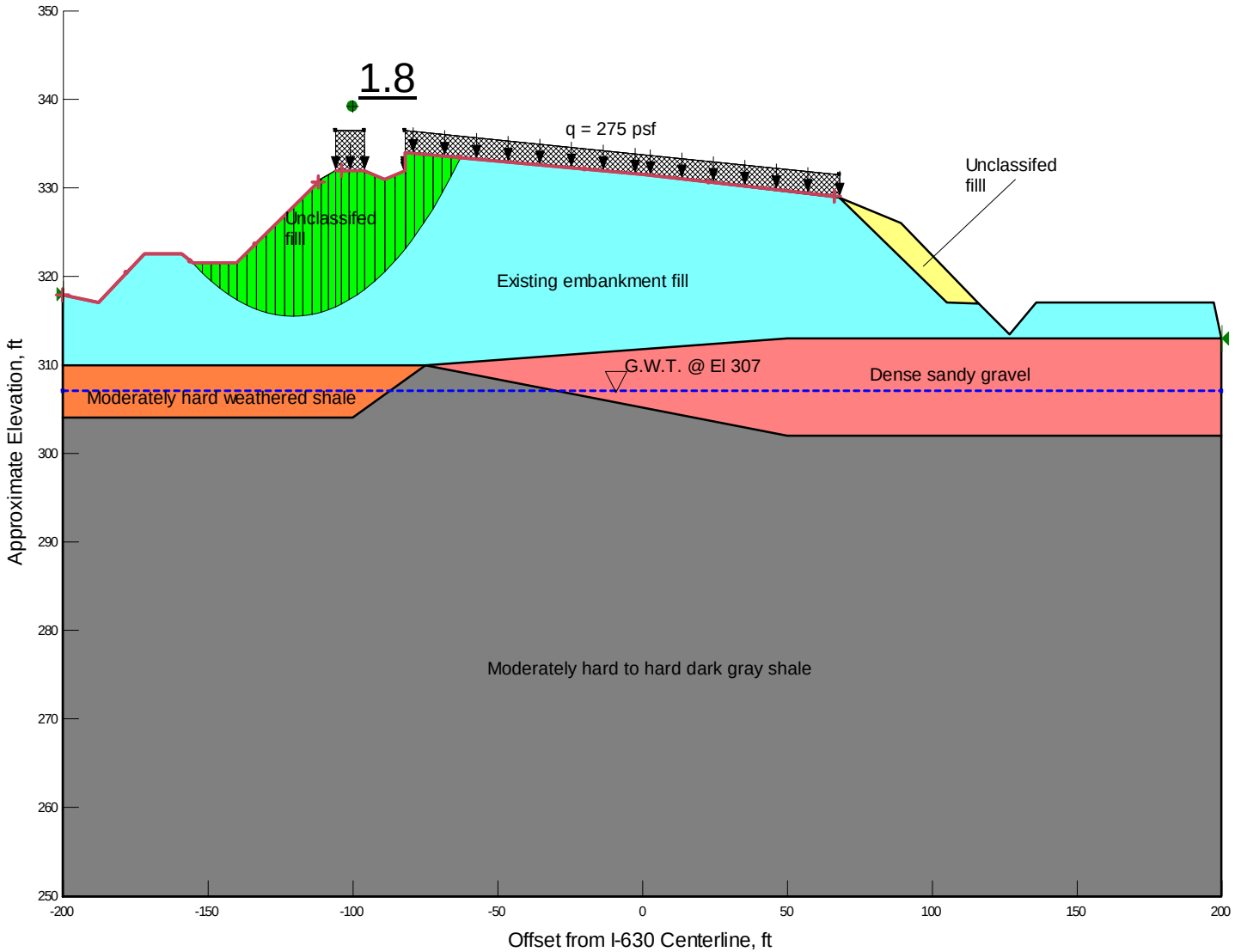
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
Side Slope @ West Bridge Abutment – Pedestrian Bridge over Rock Creek

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



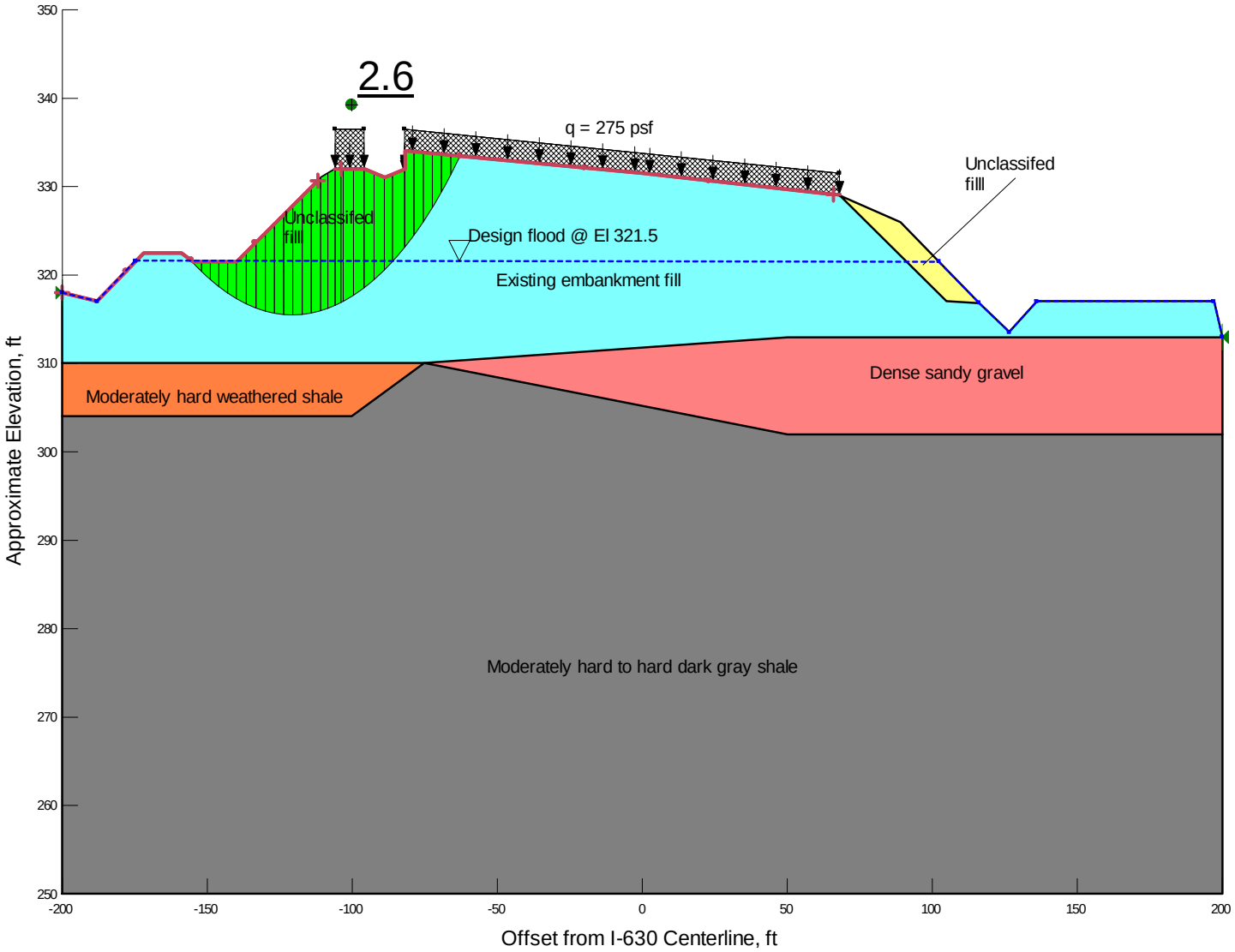
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
Side Slope @ West Bridge Abutment – Pedestrian Bridge over Rock Creek

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



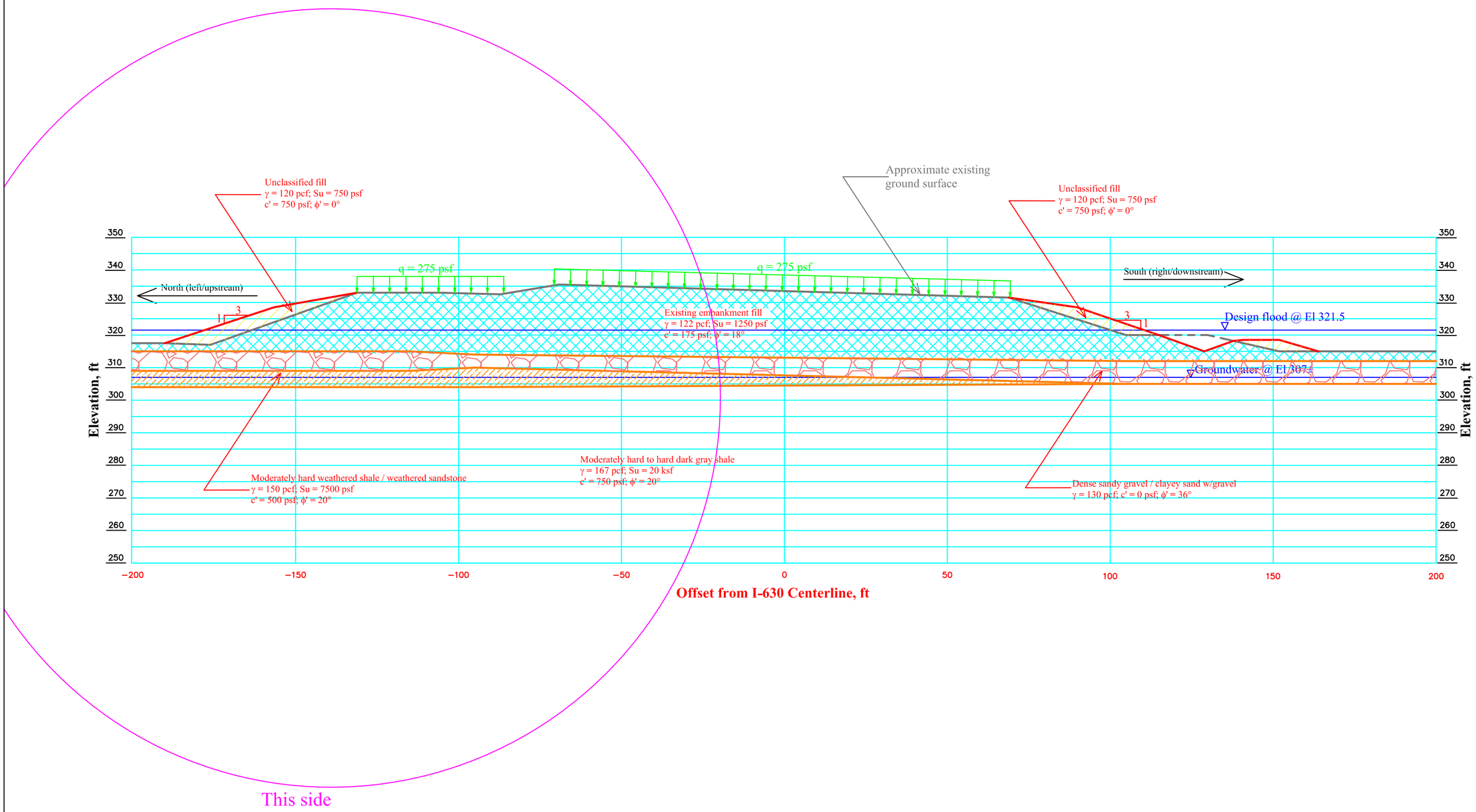
Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
Side Slope @ West Bridge Abutment – Pedestrian Bridge over Rock Creek

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



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
Side Slope @ West Bridge Abutment – Pedestrian Bridge over Rock Creek

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



15 ft 0 15 30 ft

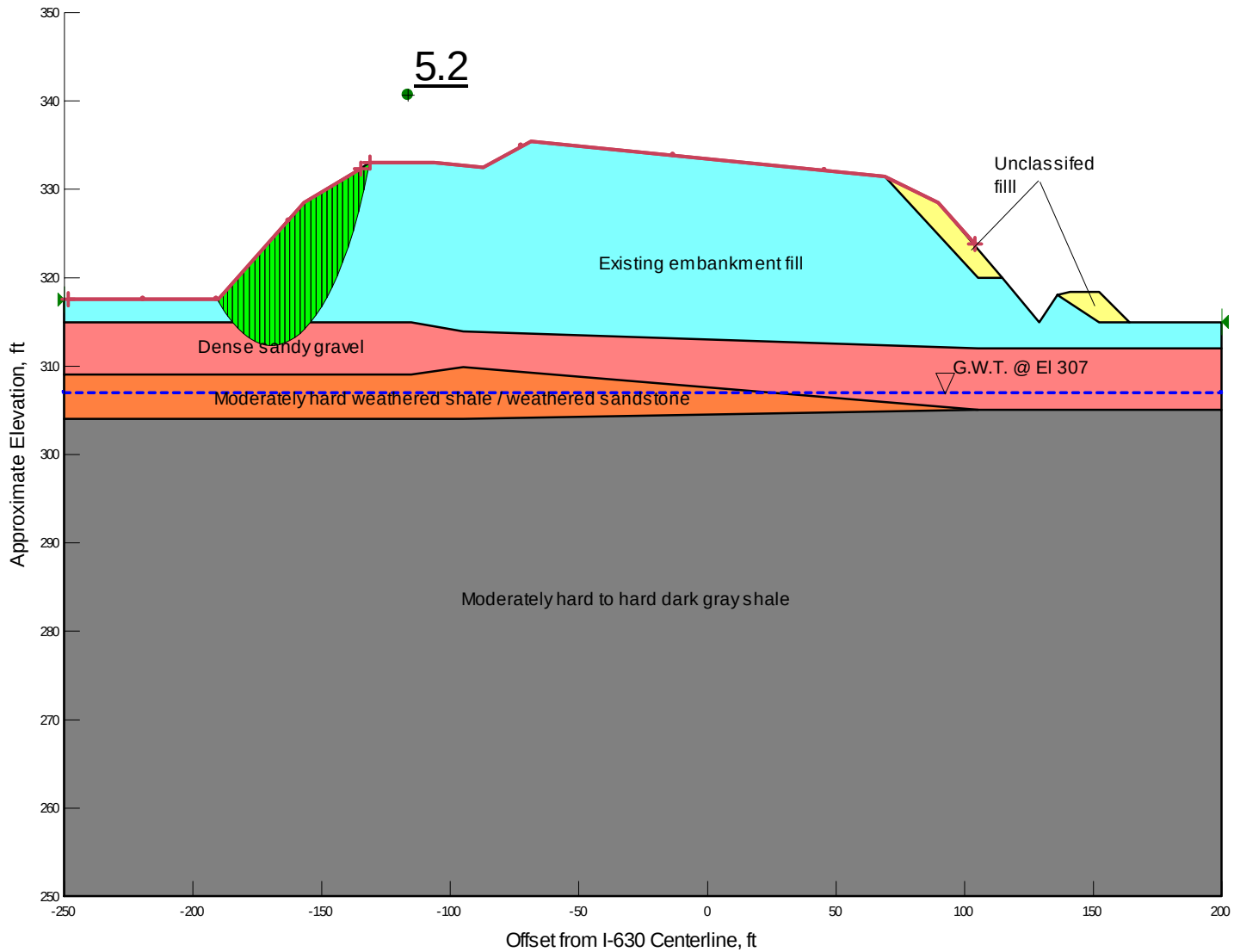
Grubbs, Hoskyn, Barton & Wyatt, Inc.
Consulting Engineers

Section and Material Parameters for Stability Analysis
Side Slope at East Bridge Abutment - Pedestrian Bridge over Rock Creek
AHTD Job No. CA0608: Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

GHBW Job No.: 14-030	Scale: As Shown
March 2, 2015	Plate

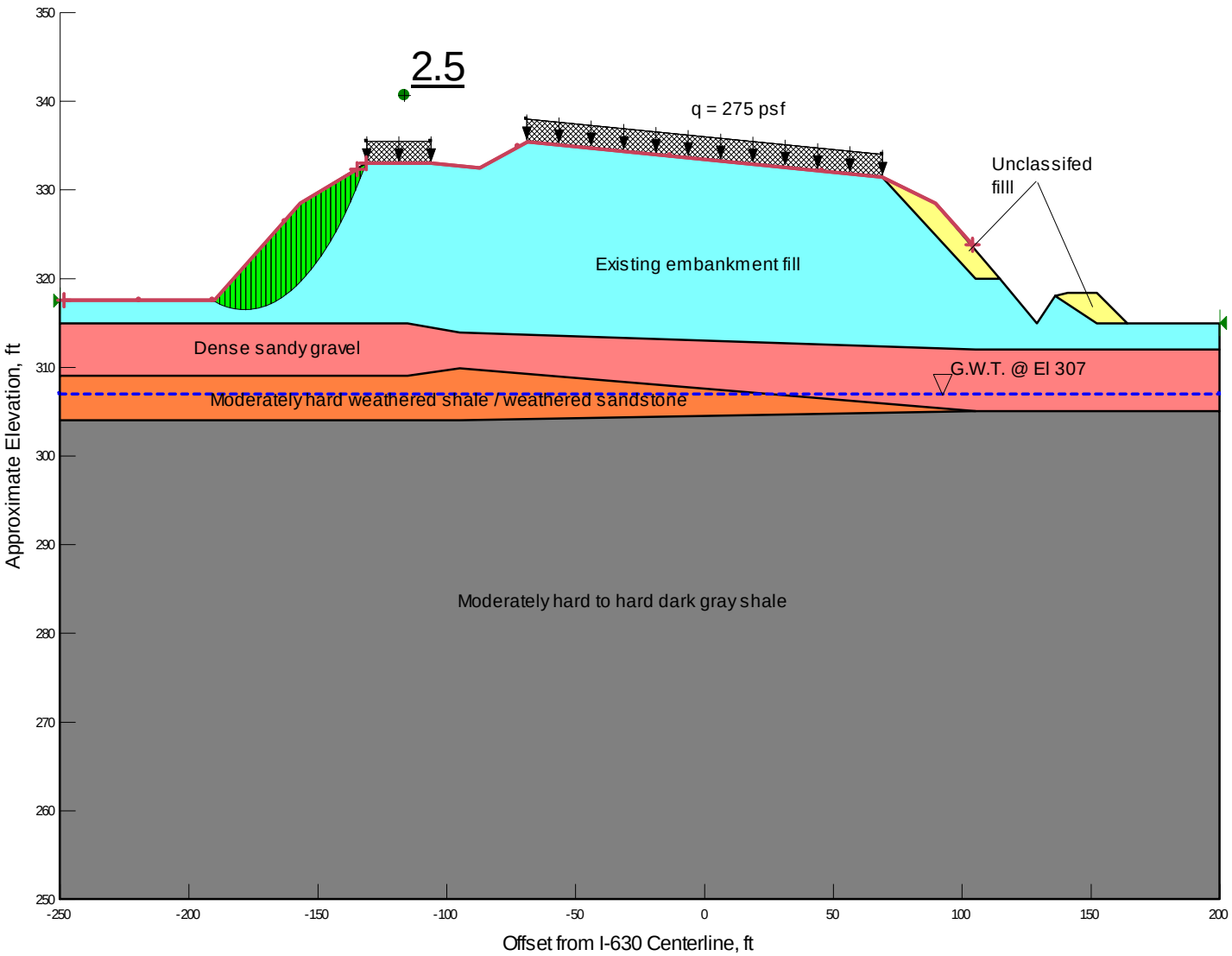
Results of Stability Analyses
Side Slopes at East Bridge Abutment – Pedestrian Bridge over Rock Creek
AHTD JOB CA0608:Baptist Hospital-University Avenue (Widening)(S)
Little Rock, Pulaski County, Arkansas

Design Loading Condition	Design Water Condition	Calculated Minimum Factor of Safety
End of Construction	Groundwater @ El 307±	5.2
Long Term	Groundwater @ El 307±	2.5
	Design flood @ El 321.5	2.4
Seismic ($k_h = 1.0A_S = 0.13$)	Groundwater @ El 307±	1.6
Rapid Drawdown	Drawdown from design flood to embankment toe	2.2



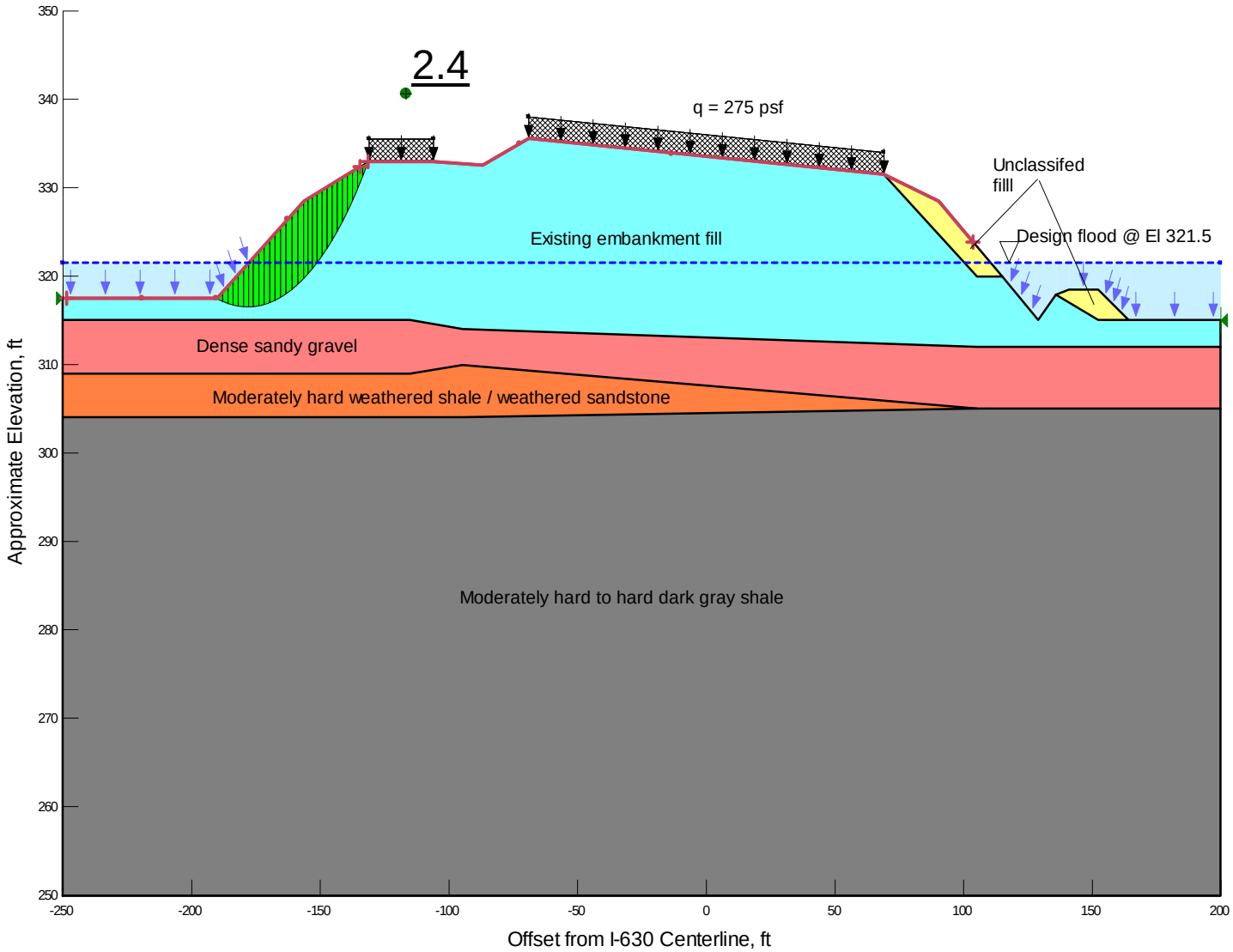
Results of Stability Analyses – End of Construction Condition
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – Pedestrian Bridge over Rock Creek

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



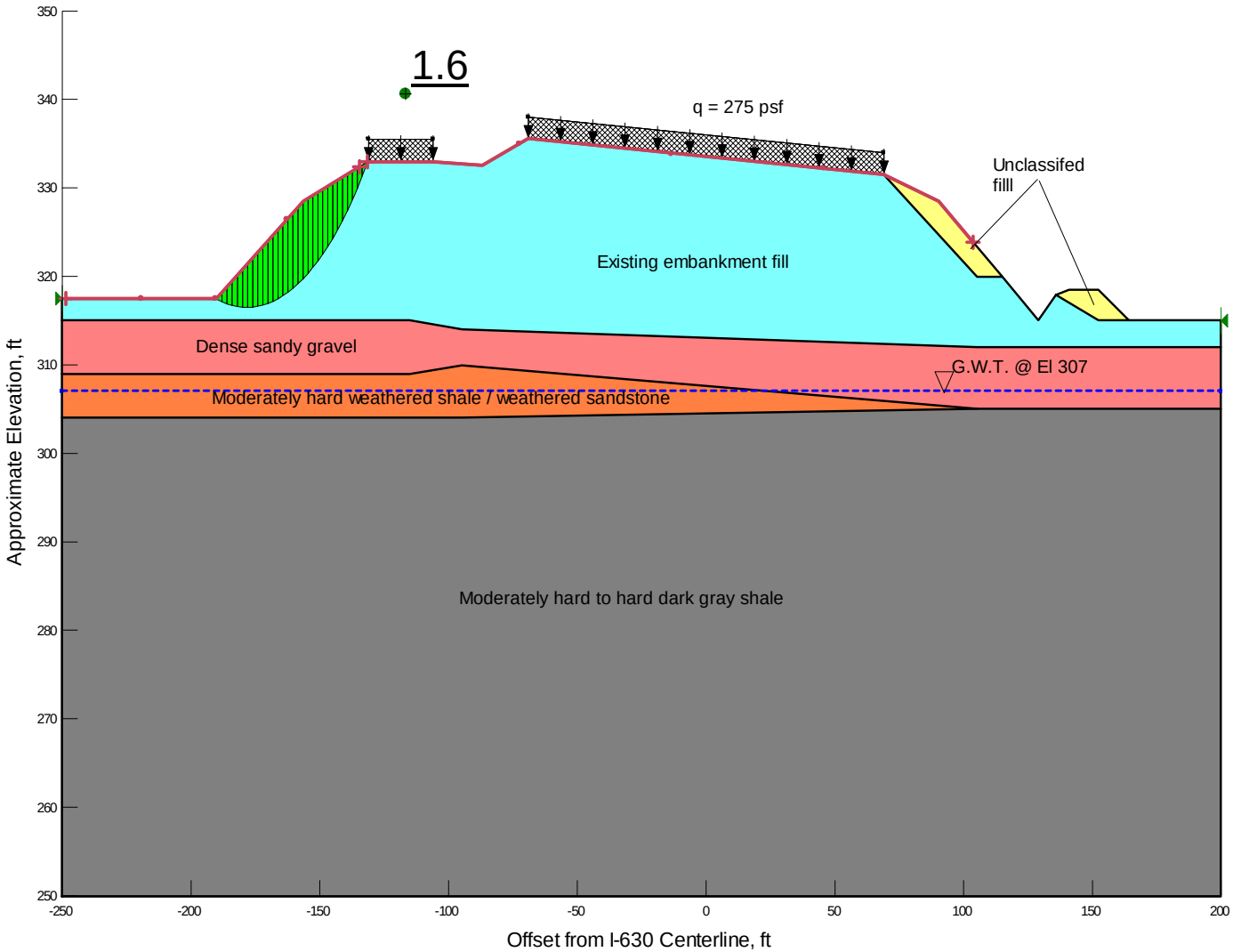
Results of Stability Analyses – Long Term Condition
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – Pedestrian Bridge over Rock Creek

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



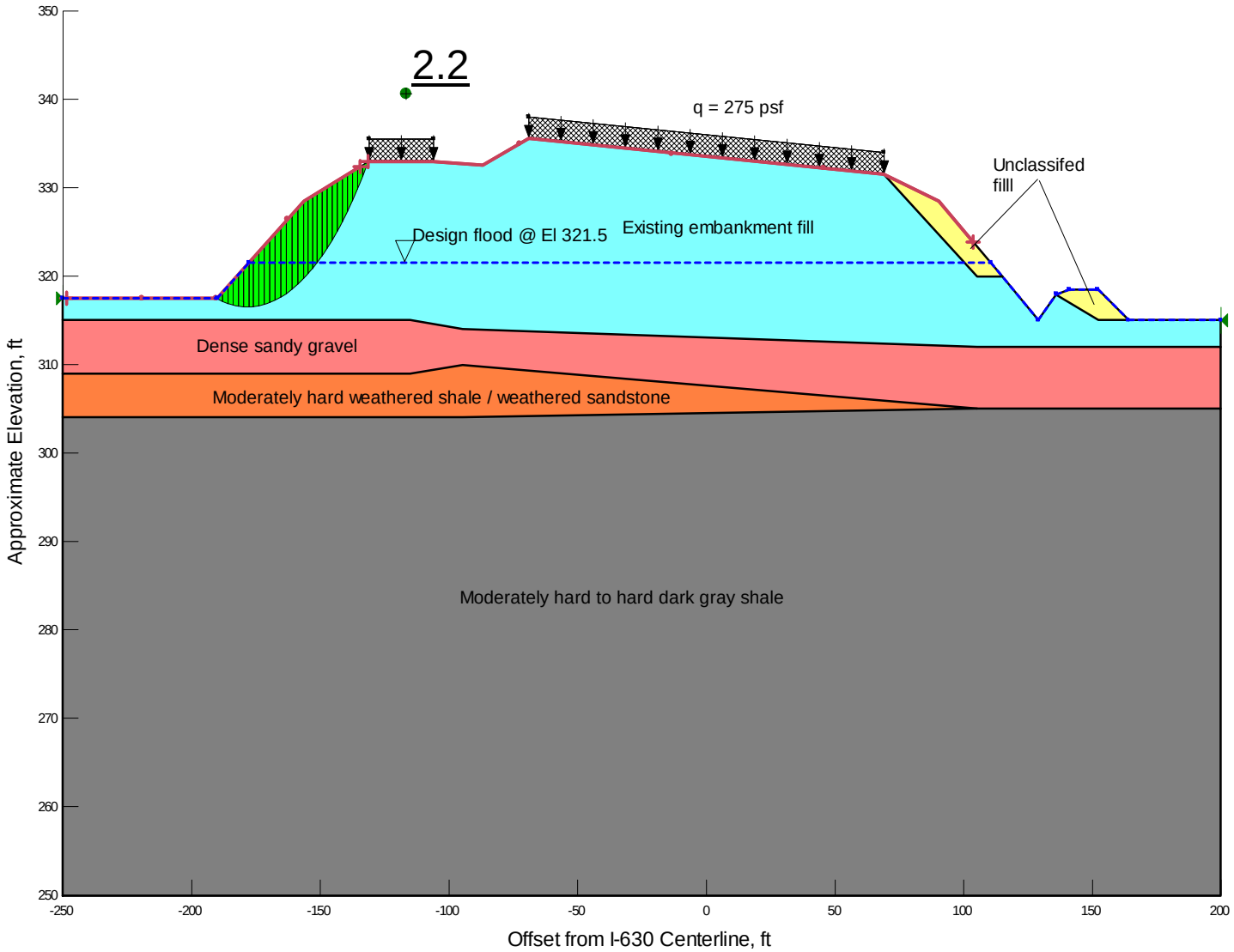
Results of Stability Analyses – Long Term Condition
Design Flood @ El 321.5
Side Slope @ East Bridge Abutment – Pedestrian Bridge over Rock Creek

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



Results of Stability Analyses – Seismic Condition ($k_h = 1.0A_S = 0.13$)
Groundwater @ El 307±
Side Slope @ East Bridge Abutment – Pedestrian Bridge over Rock Creek

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



Results of Stability Analyses – Rapid Drawdown Condition
Drawdown from Design Flood to Embankment Toe
Side Slope @ East Bridge Abutment – Pedestrian Bridge over Rock Creek

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