APPENDIX M

Preliminary Geotechnical Evaluation Report Yeh, May 2014 and Appendix

Draft

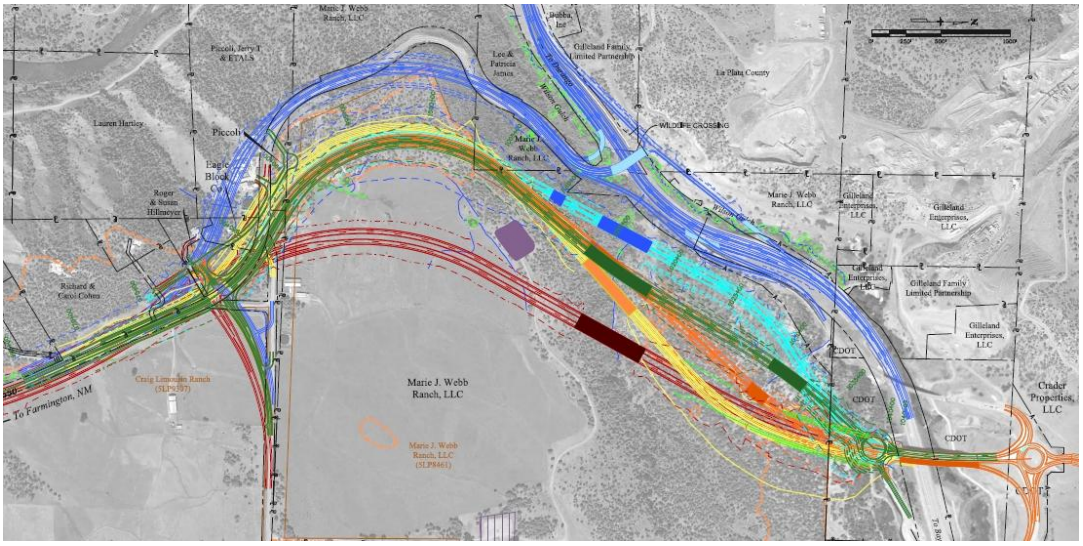
Preliminary Geotechnical Evaluation Report

US 550 South Connection to US 160:

Independent Alternatives Analysis

FSA 5501-021 (19378)

La Plata County, Colorado



Yeh Project No.: 213-174

May 16, 2014

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Consulting Engineers & Scientists

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1. Purpose and Scope of Study

The Colorado Department of Transportation (CDOT) is evaluating alternatives for the relocation of US Highway 550 south of Durango, Colorado. The highway will be re-aligned between approximate mile marker 15.0 and mile marker 16.6, in the segment known as Farmington Hill. Yeh and Associates, Inc, as a sub-consultant to AMEC of Denver, Colorado, has evaluated geotechnical conditions that may influence the design and construction of the alignment alternatives. The work was performed to support the Supplemental Environmental Impact Statement (SEIS) for the US 550 realignment.

A geotechnical investigation was performed to evaluate conditions along a new alternative alignment (R5), the previously identified RG Modified (RGM) alternate and five design variations for the RGM alignment: RGM2 through RGM6. The investigation consisted of a preliminary geologic reconnaissance, including visual surface and limited subsurface investigations; research of the project geologic setting and reports from previous geotechnical investigations, and review of available records from recent construction activity in the area.

This report presents the results of our geotechnical evaluation and provides preliminary design recommendations for the proposed alignments. The scope of study for the evaluation for this section included the following tasks:

1. Research available geologic maps, reports and construction documentation.
2. Perform site geologic mapping based on observation of surface features.
3. Limited subsurface exploration within the right-of-way for US 550 and La Plata County Road 220.
4. Perform geotechnical laboratory testing on selected subsurface samples.
5. Evaluate and summarize the field and laboratory data obtained.
6. Perform global stability analyses to evaluate proposed cut and fill slopes.
7. Provide geotechnical recommendations for the preliminary design of cut and fill slope grading, retaining walls and bridge foundations.
8. Recommend appropriate asphalt pavement thickness consistent with Colorado Department of Transportation (CDOT) design criteria.

The study was performed in general accordance with our proposal to AMEC dated May 13, 2013. A discussion of the proposed construction and geotechnical considerations for each proposed alignment and design variation are included in this report. A geologic map of the project area, logs of exploratory borings, laboratory test results and results of global stability analyses are provided in the appendices. The project location is shown on Figure 1.

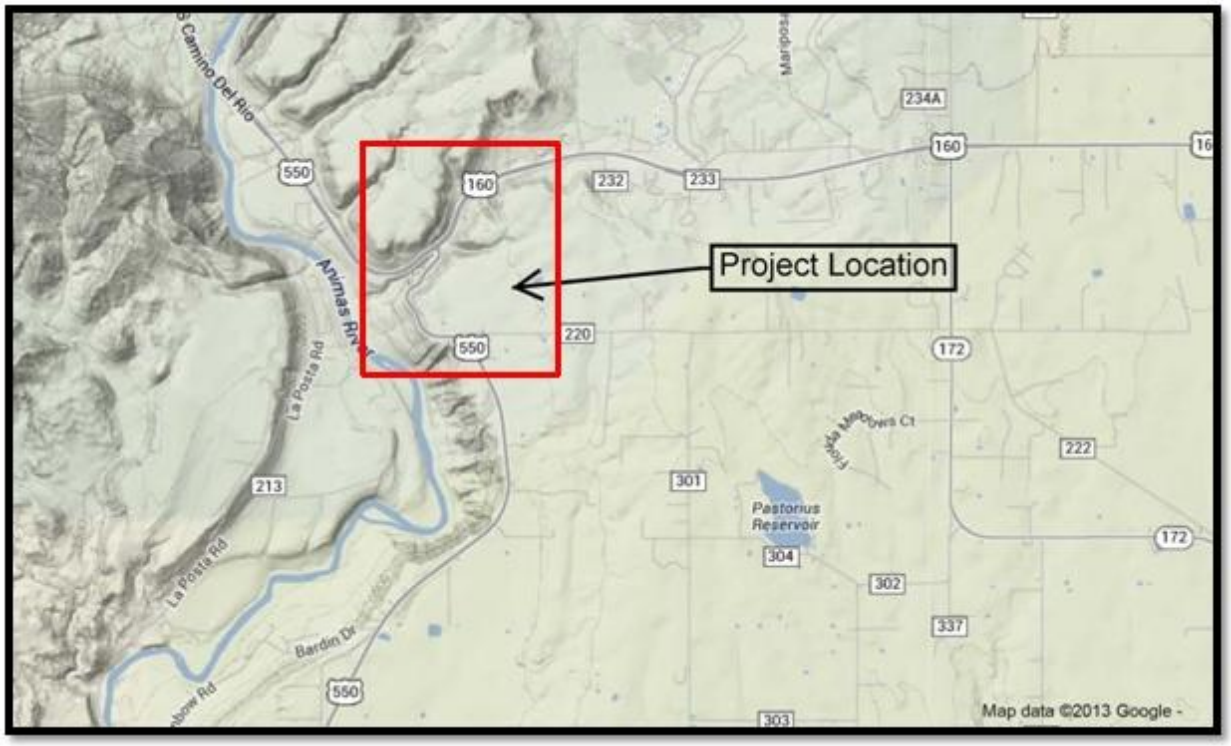

Figure 1: Project Location

2. Proposed Construction

Plans showing two alternative alignments, R5 and RGM, were provided by AMEC. The plans also show five design variations for the RGM alignment. The variations are denoted: RGM2 through RGM6. The new alignments for US 550 will be designed with two northbound and two southbound lanes. The alignments will connect from US 550 at approximate Mile Marker 15.0 to either the existing signalized intersection of US 550/US 160 at the bottom of Farmington Hill or to the recently completed interchange for US 550/US 160 approximately 0.6 miles north of the existing intersection. The preliminary plans include horizontal alignments, roadway profiles and cross-sections. Grading for the proposed alignments will include cuts and fills with heights ranging from a few feet to approximately 135 feet. Retaining walls are proposed to support the deep cuts and fills. Bridge structures are proposed where the alignments cross deep natural drainages and as an option at deep side-slope fill locations. The alignments and design variations are shown on Figures A-1 through A-10 in Appendix A.

Our understanding is that the preliminary geotechnical recommendations will be used to evaluate the limits of earthwork for the alignments. These limits will define the areas required for right of way acquisition and quantify impacts to environmental, cultural and historic resources for each alternate and design variation.

2.1 Alternate R5

This alignment generally follows the current alignment of US 550 from approximately Mile Marker 15.0 (Sta. 1+90) to the existing US 550/US 160 intersection (Sta. 258+02). The R5 alignment profile grade is 5% and will require cuts and fills of as much as about 20 feet between Sta. 1+90 and Sta. 219+00. Preliminary cross-section plans show a fill of as much as 75 feet deep supported by a retaining wall between Sta. 219+50 and Sta. 221+50. After an initial review of the plans, Yeh and Associates commented that this wall would approach or exceed the practical constructible height limitation for retaining structures on slopes. The plans were later modified to show a bridge structure at this location. Excavation for Alignment R5 will include a significant cut on the right side from approximate Sta. 222+00 to Sta. 236+00. Deep cuts are proposed right and left from Sta. 236+00 to Sta. 243+00, and on the right from Sta. 243+00 to Sta. 244+00. The heights of these proposed cuts range from about 35 feet near Sta. 223+00 to approximately 135 feet at Sta. 241+00. Cuts are proposed on the right and fills on the left from Sta. 244+00 to Sta. 253+00. The preliminary cross-sections show retaining walls supporting fills with heights ranging to about 35 feet. Cut heights for this segment of R5 are as much as 50 feet. Embankment fill right and left is shown from Sta. 253+00 to Sta. 258+00. Fill heights range from 20 to 30 feet.

The R5 Alternate includes a new bridge for US 550 over US 160 and Wilson Gulch at the location of the existing intersection. Four additional ramps are proposed to complete the intersection: Ramp J, for southbound US 550 traffic to access westbound US 550/US 160; Ramp K, for eastbound traffic on US 160 to proceed southbound US 550; Ramp L, for traffic travelling northbound on US 550 to access eastbound US 160; and Ramp M, to allow westbound US 160 traffic to access the intersection and southbound US 550. New bridges will be required over Wilson Gulch on Ramp K and on Ramp M. Ramps J, L and M will have retaining walls supporting cuts of as much as 35 feet. Walls to retain embankment fills with heights ranging to 38 feet will be required along Ramps K and L. Profile grades on the ramps are as steep as -7%.

2.2 Alternate RGM

This alternate follows the current alignment of US 550 from approximate Mile Marker 15.0 (Sta. 195+00) for approximately 1,000 feet to Sta. 205+00. The RGM alignment then curves northeast at the top of Farmington Hill and continues in a northerly direction on the slope above US 160 to the recently completed interchange of US 550/160 (Sta. 265+50). The RGM alternate connects to the recently constructed US 550 bridge over US 160 on a shallow curve that closely follows the bridge alignment. Maximum grades for the vertical alignment of US 550 range from -5% to +2% and the alignment has three changes in the roadway profile. The cross-sections show cuts up to

10 feet and fills up to 20 feet from Sta. 195+00 to Sta. 241+00. The profile has a deep unsupported fill of as much as 100 feet from Sta. 241+00 to Sta. 246+00; although a bridge structure is being considered as an option. Cut slopes graded at 3:1 are shown from Sta. 246+00 to Sta. 265+20. The deepest cut is as much as 120 feet near Sta. 262+00.

2.3 Design Variation RGM2

The alignment of the RGM2 design variation follows existing US 550 from the beginning of the project at approximate Mile Marker 15.0 (Sta. 965+00) to the existing US 550/CR 220 intersection (approximate Sta. 986+00). The alignment then continues in a northerly direction on the slope above US 550 before bearing northeast on the slope above US 160 to the recently completed interchange of US 550/160 (Sta. 1043+00). The RGM2 alignment is relatively straight at the connection to the recently constructed bridge over US 160. Grades for the vertical profile range from -3% to +2.22% and the alignment has four changes in the roadway profile. The cross-sections show unsupported cut heights up to 25 feet and fills up to 20 feet from Sta. 965+00 to Sta. 1015+00. Retaining walls are shown supporting the cut on the right from Sta. 1002+00 to Sta. Sta. 1003+50 and supporting the fill on the left from Sta. 1006+00 to 1011+00. Cuts of as much as 35 feet with no retaining structures are shown from Sta. 1011+50 to Sta. 1014+50. A bridge structure is shown from Sta. 1015+00 to Sta. 1020+50. Deep cuts right and left are shown from Sta. 1021+00 to the existing bridge at Sta. 1042+00. Proposed cut heights are as much as 110 feet and retaining walls are shown on both sides of the alignment.

2.4 Design Variation RGM3

The RGM3 design variation alignment closely follows the RGM2 alignment from the beginning of the project to Sta. 1005+00. At this location, the RGM3 alignment diverges from the RGM2 alignment toward the west and follows the edge of the slope above US 160. The alignment then continues in a northerly direction along the face of the slope before bearing northeast to the recently completed interchange of US 550/160 (Sta. 1047+75). Two alternative vertical profiles are being considered for Design Variation RGM3. The first alternate (RGM3-1) has deeper cuts and has grades ranging from -0.5% to +3% with three changes in the roadway profile. The second alternate profile (RGM3-2) is optimized to minimize cut and fill depths with grades ranging from -0.5% to +3.31% with three changes in the roadway profile. A ramp for southbound US 550 traffic to access eastbound US 160 is included at the interchange with this design variation.

The profile for RGM3-1 shows cuts and fills of as much as 15 feet from Sta. 965+00 to Sta. 989+00. Deep cuts of as much as 100 feet are shown from Sta. 989+00 to Sta. 1015+00. A

bridge structure is shown on the cross-sections from approximately Sta. 1015+50 to Sta. 1017+50. Between Sta. 1017+50 and Sta. 1030+50, where the alignment follows the slope above US 160, the preliminary cross-sections show cuts on the right of as much as 110 feet and shallower cuts on the left of as much as 30 feet. Short segments of the alignment between Sta. 1020+50 and Sta. 1025+50 will have fills on the left supported by retaining walls with heights of as much as 30 feet. The RGM3-1 profile from Sta. 1030+50 to 1032+50 shows fills of as much as 30 feet supported by retaining walls. Near the north end of the alignment, between Sta. 1032+50 and Sta. 1041+00, deep cuts of as much as 140 feet are shown.

The RGM3-2 profile begins at Sta. 965+00 with unsupported cuts and fills of as much as 15 feet to Sta. 988+00. Cuts of up to 45 feet and fills of as much as 60 feet are shown on the cross-sections from Sta. 988+00 to Sta. 1014+00. In general, cuts are on the right and are graded at 2H : 1V to Sta. 1011+00. Beyond Sta. 1011+00 the cuts are supported by retaining walls. Fills are on the left and are supported by retaining walls. The preliminary cross-sections for the RGM3-2 profile show a bridge structure between Sta. 1400+00 and 1800+00. Cuts and fills from Sta. 1018+00 to 1027+00 will be similar to the segment before the bridge. The maximum cut height on the right is shown as 90 feet and fills on the left will have heights of as much as 35 feet. The cross-sections show retaining walls supporting the cuts and fills. Deep cuts, of as much as 110 feet, supported by retaining structures are shown on the preliminary cross-sections from Sta. 1027+00 to Sta. 1030+50. Shallow cuts on the right and deep fills of as much as 30 feet on the left are shown from Sta. 1030+50 to Sta. 1032+50. Where the fill heights exceed about 15 feet, retaining walls are shown. Between Sta. 1032+50 and Sta. 1042+00 the RGM3-2 profile has deep cuts of as much as 130 feet supported by retaining structures.

2.5 Design Variation RGM4

The RGM4 alignment follows the RGM3 alignment from Sta. 965+00 to Sta. 1005+00. At this location, the RGM4 alignment diverges from the RGM3 alignment toward the east and passes through hilly terrain southeast of US 160 instead of following the edge of the slope. The RGM4 alignment is relatively straight at the connection to the recently constructed bridge to carry US 550 over US 160. Grades for the vertical profile range from -3% to +2.27% and the alignment has 4 changes in the roadway profile. The cross-sections show unsupported cut heights up to 25 feet and fills up to 20 feet from Sta. 965+00 to Sta. 1014+00. Retaining walls are shown supporting the cut on the right from Sta. 1002+00 to Sta. 1003+50 and supporting the fill on the left from Sta. 1006+00 to Sta. 1011+00. Cuts of as much as 35 feet with no retaining structures are shown from Sta. 1011+50 to Sta. 1014+50. A bridge structure is shown from Sta. 1014+50 to Sta. 1020+00. A second bridge structure is proposed from approximate Sta. 1029+50 to Sta.

1030+50. Deep cuts right and left are shown from Sta. 1021+00 to the existing bridge at Sta. 1042+00. Proposed cut heights are as much as 120 feet and retaining walls are shown on both sides of the alignment.

2.6 Design Variation RGM5

The RGM5 design variation has an alignment that closely follows the RGM4 alignment from Sta. 965+00 to Sta. 1018+00. The RGM5 alignment diverges from RGM4 toward the northwest from Sta. 1018+00 to Sta. 1040+00 where this variation includes a roundabout on the south side of the existing US 550 bridge over US 160. Cut and fill heights are similar to those shown for RGM4 except that the maximum proposed cut is 115 feet. The north bridge structure for RGM5 is longer than the structure at this location on RGM4 and extends from approximate Sta. 1029+50 to Sta. 1033+00. The grades and vertical profile are similar to RGM4.

2.7 Design Variation RGM6

Design variation RGM6 follows the alignment of RGM5 with minor changes to horizontal curves between approximate Sta. 975+00 and Sta. 987+00 and between approximate Sta. 1010+00 and Sta. 1025+00. Grades, cut and fill heights, structures and the roundabout configuration at the US 550 over US 160 bridge are the same for the two alignments.

3. Geological Setting and Site Conditions

3.1 Geologic Setting

The project is located on Florida Mesa in La Plata County, south of Durango Colorado. The mesa elevation is slightly over 6800 feet above sea level at its highest point, approximately 300 feet above the Animas River Valley along the western flank of the mesa. The project area extends north from La Plata County Road 220 to U.S. Highway 160. A geologic map showing the thicknesses and surface elevations of the major geologic strata within the project area was prepared based on information obtained from observation of surface features and from subsurface exploration. The map and accompanying legend are provided in Appendix A, Figure A-11.

The observable geology of the mesa consists of Quaternary aged dense gravel terrace alluvium overlying claystone and sandstone bedrock. The gravel alluvium is composed of gravel to boulder size, rounded to sub-rounded rocks in a sandy clay matrix. The alluvium is identified on the geologic map by the symbol Qg. The visible bedrock outcrops are lower Tertiary aged and composed of the Animas Formation, which consists of interlayered arkosic sandstone, shale, conglomerate and volcaniclastics. Locally this formation has variable interbedded layers that

include claystones and isolated zones of volcanic tuff. The sandstone and tuff layers are relatively hard and erosion resistant while the softer shale and claystone layers are prone to erosion. The bedrock is shown on the geologic map with the symbol Ka. Zones of volcanic tuff are identified as Tk on the geologic map.

Surficial deposits on the Florida Mesa consist of alluvium and terrace gravels. The near surface soils are predominately sandy clay and clayey sand overlying dense sandy gravel containing cobbles and boulders. The sand and clay soils, shown as symbol Clay Qa on the geologic map, generally have low strength and high plasticity due to the large proportion of clay. The dense gravels have historically been exploited as an aggregate resource.

Small landslides and shallow slumping soils were observed along the north and west edges of the mesa and on the sides of the deep ravines. These are unstable deposits composed primarily of alluvial gravel mixed with sandy clay that have eroded from the overlying strata to fill shallow drainages on the bedrock surface. Seeps and springs shown on the map are generally associated with these shallow slide features.

3.2 Site Conditions

The existing US Highway 550 runs north-south along the western edge of the Florida Mesa until it descends in a steep winding path from the northern edge of the mesa to the intersection with US Highway 160. These two highways share a common route for approximately four miles to the northwest where they diverge near the southern end of the City of Durango. Both of these roadways are major transportation routes for commuter traffic as well as shipping, and gas and oil services in the four corners region. La Plata County Road 220 (CR 220) marks the southern edge of the project area. This is a relatively narrow road with travel lanes as narrow as 9 feet and little to no shoulders. The road extends east from US 550 on the west to State Highway 172. It and provides access to rural residential, and ranch properties.

The project area encompasses private residential, commercial and agricultural properties including Eagle Block Company, located west of US 550 at the top of Farmington Hill, and the Webb Ranch, located east of US 550 and north of CR 220. Eagle Block Company is a manufacturer and retail distributor of building block and masonry materials. The Webb Ranch consists of several hundred acres of irrigated livestock pasture, agricultural and residential buildings, wooded acreage, gas production wells and associated infrastructure.

Terrain within the project area ranges from relatively flat pasture land at the Webb Ranch to steep ravines and vertical cliffs along the southern edge of the mesa above US 550 and along the

western edge of the mesa above US 160. A significant east-west trending steep sided ravine is located below the northern limit of the Webb Ranch pasture. The ravine has a maximum depth below the surrounding terrain of approximately 120 feet. Elevation above sea level ranges from about 6500 feet at the existing US 550/US 160 intersection at the bottom of Farmington Hill to about 6840 feet at the northeast limit of the project area.

Vegetation along the edges of the Florida Mesa consists of dense piñon and juniper forest with gambrel oak and shrub understory. The irrigated pasture on the Webb Ranch is primarily grass. Shallow ponds are located above and below the existing US 550 at the top of Farmington Hill. The Webb Ranch is crossed by several irrigation ditches that generally flow from the southwest toward the northeast.

4. SUBSURFACE INVESTIGATIONS

The subsurface investigations that were used to develop the preliminary design recommendations presented in this report include:

1) A 2005 report by URS Corporation titled "Preliminary Geotechnical Investigation, US Highway 160, Sections 1 through 4, La Plata County, Colorado", Project No. FC-NH(CX) 160-2(048).
2) A 2007 investigation report by Yeh and Associates titled "Geotechnical Investigation Report, US Highway 160 4th Lane at Farmington Hill" dated November 27, 2007, Project No. 26-235.
3) A 2008 investigation by CDOT reported in a memo titled "Addendum No. 3, Final Geotechnical Report, Ramp C Bridge (P-05-W) and US 550 Bridge (P-05-AG), US160/US550 Fourth Lane Project" dated June 23, 2008.
4) Four Design Letters, dated 2008, prepared by Shannon & Wilson, Inc. for Wilson & Company, the design-build consultant on the US 160/US 550 Fourth Lane Project, NH 1602-114.
5) Recent borings by Yeh and Associates drilled in 2013 as part of the scope of work for this alternatives analysis.

The reports from the previous investigations 1) through 5) are included in Appendix E. A summary of the subsurface conditions encountered by previous and current investigations of the proposed alternates (URS 2005, Yeh and Associates 2007, CDOT 2008 and Yeh and Associates 2013) is presented in the Subsurface Conditions section of this report. Details of the recent investigation are provided below.

The recent subsurface exploration was performed on October 23 and 24, 2013. Six exploratory borings numbered YA-1 through YA-6 were advanced in the project area, three borings along existing US 550 and three borings along CR 220. Two borings (YA-5 & YA-6) located east of the CR 220 at US 550 intersection were shallow borings drilled to assess pavement thickness and subgrade conditions. These borings were advanced to a depth of 5.5 feet below the paved surface. Boring YA-1 through YA-4 were drilled to depths ranging from 10 to 39.5 feet below the ground surface. Logs of the borings are provided in Appendix B.

The borings were drilled with a truck-mounted CME 55 drilling rig using solid stem auger. Samples were obtained at selected intervals using a 1.5-inch or 2.0-inch I.D. split-spoon sampler. The split-spoon sampler was driven into the subsoil with a 140-pound automatic hammer falling 30 inches. The number of blows needed to drive the sampler 12 inches constitutes the blow count, N, reported on the Boring Logs. The blow count can be used as a relative measure of the material stiffness or density. Bulk samples of auger cuttings were also obtained from the borings at selected intervals. Upon completion, the borings were backfilled with auger cuttings. Where borings were in the pavement, the holes were patched with a minimum of 3 inches of quick-set concrete.

Samples retrieved during the field exploration were returned to our Durango, Colorado laboratory for observation by the project geotechnical engineer. An applicable program of laboratory testing was developed to determine engineering properties of the subsurface materials. The soils were classified in the laboratory in accordance with the American Association of State Highway and Transportation Officials (AASHTO) and Uniform Soil Classification System (USCS) methods. The results of the laboratory classifications were used to confirm or modify the field descriptions, and boring logs were prepared.

Laboratory tests performed included gradation (ASTM D 421, C 136 and AASHTO T 27), Atterberg limits (AASHTO T 89/T 90), and moisture content (AASHTO T 265). Gradation and Atterberg limits test results were used to classify the soils in accordance with the AASHTO classification system and the USCS. A summary of the laboratory test results is presented in Appendix C, and the results are shown on the boring logs in Appendix B.

5. SUBSURFACE CONDITIONS

Soils in the project area generally consist of 5 to 30 feet of alluvial or aeolian sandy clay and clayey sand over 10 to 20 feet of alluvial terrace gravels with cobbles and boulders. The bedrock consists of interlayered sandstone and claystone with scattered conglomerate and volcaniclastic inclusions. The depths of the overburden soils along the proposed alignments were estimated

from field observations and results from limited subsurface exploration. Along the rim of the Florida Mesa and on the side slopes of valleys and drainages crossed by the proposed alignments, colluvial soil deposits have accumulated to cover the bedrock outcrops. These deposits consist of a variable mix of gravel, cobbles and sandy clay transported by gravity and erosion. The colluvial deposits are unconsolidated and may be unstable where the underlying bedrock surface is steeply sloping and where groundwater is perched at the colluvium/bedrock contact. The depth of the unconsolidated deposits over the bedrock is estimated to range from a few feet to about 10 feet.

5.1 URS 2005

URS investigated subsurface conditions for the US 550 Bridge over US 160 and for the associated ramps and walls of the US 550/US 160 Interchange. Borings drilled along the northern edge of the Florida Mesa (2B16, 2B18 and 2B30) encountered about 5 to 35 feet of dense gravel with cobbles and boulders over claystone bedrock. Groundwater was encountered in Boring 2B16 at a depth of 32 feet, near the bedrock surface.

5.2 Yeh and Associates 2007

The subsurface conditions at the existing ramp for eastbound traffic exiting US 160 (Ramp A and Wall P-05-F) were investigated by Yeh and Associates in 2007. A portable drilling rig was used to obtain BX cores from Borings YA-14, YA-15 and YA-17. Boring YA-18 was drilled using a rubber-track mounted drilling rig to obtain HQ cores. The borings were located on the slope above and below the ramp. The subsurface conditions at these boring locations consist of about 2 to 6 feet of medium dense to dense clayey to gravelly sand and sandy clay with gravel and cobbles over soft to hard, interlayered sandstone, shale and sandstone conglomerate bedrock. The bedrock has closely spaced fractures and weathered horizontal joints as indicated by the relatively low RQD for samples with high recovery. Groundwater was not encountered in these borings.

Borings YA-25 and YA-26 were drilled along the Wilson Gulch drainage, near the existing Farmington Hill US 550/US 160 intersection. These borings encountered 22.5 to 38 feet of medium dense to dense sandy, clayey gravel with cobbles and possible boulders overlying sandstone bedrock. The borings were advanced using HQ coring methods with water to flush the cuttings and accurate measurements of groundwater levels were not obtainable.

5.3 CDOT 2008

The drilling for the CDOT investigation was performed on April 21 thru 24, 2008. The borings were drilled with an ATV mounted CME-550 drilling rig using wire-line coring methods. Three holes B-201, B-202, and B-203 were drilled above the south abutment of the US 550 bridge. Selected samples of bedrock obtained from the borings were tested for moisture content, unit weight, unconfined compressive strength, and shear strength. The subsurface conditions encountered in the CDOT borings generally consist of alluvial deposits overlying interbedded shale and sandstone. Boring B-201 encountered stiff to hard clay with sand and gravel from the ground surface at approximate elevation 6,846 feet to elevation 6,821 feet where alluvial gravel was encountered. The alluvial gravel/bedrock contact was encountered in Boring B-201 at approximate elevation 6,798 feet. Boring B-202 encountered sand with gravel and cobbles from the ground surface elevation at approximately 6,812 feet to the gravel/bedrock contact at elevation 6,770 feet. Boring B-203 encountered two feet of gravel road surfacing and native sand and gravel underlain by weathered bedrock from a depth of 2 feet to total drilling depth of 15 feet. Groundwater was measured in Boring B-201 at the bedrock contact three days after drilling.

5.4 Yeh and Associates 2013

Recent borings YA-1 through YA-4 were drilled along the proposed R5 alignment, most of which follows the existing US 550 roadway. Boring YA-1 was located at the near the existing US 160 / 550 intersection. Subsurface conditions encountered in Boring YA-1, below 8 inches of asphalt, consisted of 2.2 feet of granular aggregate base course (ABC) material over seven feet of clayey sand (SC) embankment fill. The boring encountered native soil, consisting of a mixture of clayey Gravel (GC) and sandy clay (CL), below the fill at a depth of 11 feet. The claystone-sandstone bedrock surface was encountered at a depth of 19 feet below the paved surface. Boring YA-2 encountered native soil below a pavement section consisting of 7 inches of asphalt over 12 inches of ABC. The native soils consisted of dense, poorly sorted gravel (GP) with large cobbles and boulders to auger refusal at a depth of 10 feet. Borings YA-3 and YA-4 were drilled at the top of Farmington Hill near the CR 220 intersection. The subsurface material encountered in Boring YA-3, below 6 inches of asphalt and 6 inches of ABC, consisted of 31.5 feet of hard to stiff sandy clay over 7 feet of dense clayey gravel. The pavement section at the location of Boring YA-4 consisted of 8 inches of asphalt over 10 inches of ABC. Below the pavement section Boring YA-4 encountered 26.5 feet of very stiff to stiff sandy clay over 9.5 feet of dense to very dense sandy gravel with cobbles. Auger refusal was encountered in Boring YA-4 at a depth of 37.5 feet. Subsurface conditions encountered in Borings YA-5 and YA-6 consisted of 7 inches of asphalt

over 8 to 13 inches of ABC, on native sandy clay (CL) subgrade soil. Detailed logs of the borings and a site map showing boring locations are provided in Appendix B.

5.5 Laboratory Test Results

Laboratory tests indicate the clayey sand embankment fill encountered in Boring YA-1 has medium plasticity and an AASHTO soil classification of A-2-6. Samples of the sandy clay soils from Borings YA-3, YA-5 and YA-6 have little to no gravel, high liquid limits and medium to high plasticity. These soil samples have AASHTO classifications of A-6 with group indices ranging from 13 to 20. A sample of sandy clay from Boring YA-4 had a trace of gravel, very high liquid limit and high plasticity. This sample classifies as A-7-6 with a group index of 18 in accordance with AASHTO. Based on these test results, the sandy clay soils are expected to have poor pavement support characteristics and may swell when wet if used as compacted embankment or backfill.

5.6 Groundwater

The geologic mapping and results of subsurface exploration indicate groundwater within the project area is perched on the claystone/sandstone bedrock, below the alluvial gravel. The exploratory borings did not encounter groundwater within the gravel layer. The observed seeps and springs appear to be located at the gravel/bedrock contact or below pockets of gravelly soil that have been deposited in low areas and ravines along the edge of the mesa. The gravelly pockets provide a permeable zone for surface runoff to infiltrate or for groundwater to flow from gravel/bedrock contact and emerge as springs or seeps farther down the slope. Groundwater levels in the project area are influenced by infiltration from irrigation of the agricultural land on Florida Mesa. Seasonal fluctuations in groundwater levels are likely.

6. GLOBAL STABILITY ANALYSIS

The computer software program SLIDE was used as an aid to evaluate global stability of proposed cut and fill slope grading for the alternate alignments and design variations. The software uses limit-equilibrium analysis to compare forces that drive slope movements with forces that resist movements. The result is a numerical Factor of Safety (FoS) against failure calculated for a given slope from variables that include slope geometry, engineering properties of the materials, internal and external loads, and groundwater levels. A FoS of at least 1.30 is considered acceptable for the design of supported or unsupported slope grading on transportation projects.

Slope geometries for the analyses were based on the dimensions of selected roadway cross-sections along each preliminary design profile provided by AMEC. The elevations of the upper and lower contacts of the clay soil, alluvial gravel and claystone/sandstone geologic materials, as determined by the geologic mapping and subsurface exploration, were used to assign materials types to vertical intervals on the cross-sections.

The engineering properties for the soil, gravel and bedrock were assumed for the analyses based on the results of the subsurface investigations and Yeh and Associates' experience monitoring retaining wall and foundation construction for the US 160/US 550 Interchange project. The engineering properties assumed for the geologic strata considered in the global stability analyses are shown in Table 1.

Traffic loading of 250 psf, applied per AASHTO guidelines, was considered in the stability analyses of cross-sections that include fill slopes or retaining walls supporting the proposed highway. Loads from the weight of embankment fills and retaining structures were considered where they are shown on the preliminary plans. The preliminary engineering properties assumed for embankment fill and retaining wall backfill materials are included in Table 1.

Table 1: Preliminary Engineering Properties of Materials

Material	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Angle of Internal Friction (deg)	Cohesion (psf)	Anchor Bond Stress (psi)
Sandy Clay	120	130	25	200	--
Alluvial Gravel	130	140	33	50	--
Claystone/Sandstone	140	145	33	500	55
New Fill and Backfill	135	140	34	0	--

The analyses included loads from design features that improve the stability of cut and fill slopes by resisting movement. These include reinforcements in Mechanically Stabilized Earth (MSE) retaining walls, soil nails and tie-back ground anchor systems.

The perched groundwater table was modeled as a single piezometric surface within the alluvial gravel layer. The analyses assume the sandy clay above the gravel and the bedrock below the gravel are not saturated.

The output from the global stability analysis is presented as idealized cross sections with critical failure surfaces and factors of safety shown. Results for selected cross-sections are provided in Appendix D. A summary of the results with required reinforcement types and lengths for adequate FoS is provided in Table D-1 in Appendix D. Reinforcement types include geosynthetics for MSE fill walls, soil nails and post-tensioned tie-back anchors for walls that support cut slopes.

7. GRADING RECOMMENDATIONS

The preliminary recommendations presented below for cut and fill grading along the proposed alignments are intended to provide grading that is stable for the long term. Retaining walls to support deep cuts and fills are recommended to provide stability and minimize the extent of the right-of-way required for construction. Full height pre-cast or cast-in-place panel facing, similar to that constructed in other segments of the corridor, is assumed for cut and fill retaining walls. Wall facing should have a batter of 12V:1H or flatter. Moment slabs, designed in accordance with CDOT standards, should be provided at the tops of walls where guardrail or bridge rail is required.

7.1 Cut and Fill Slopes

Un-retained cut and fill slopes should be graded at 3H:1V or flatter to create stable slopes, reduce erosion and promote re-vegetation. Retaining structures such as MSE walls may be more efficient than slope grading to limit right-of-way impacts for fill heights in excess of about 20 feet. Cuts through the sandy clay overburden soil and the underlying alluvial gravel will have heights ranging from less than 5 feet to about 60 feet. The results from the global stability analyses indicate that cuts in the alluvial gravel steeper than 3:1 may have adequate long-term factors of safety. However, steeper cuts in the gravel layer will be difficult to re-vegetate and will increase the potential for erosion related rockfall hazard. Cast-in-place retaining walls or MSE walls can be used to support cuts in the gravel alluvium at selected locations, thereby steepening the overall cut slope and reducing the right-of-way required for construction.

7.2 Retaining Structures for Cuts

The preliminary cross-section plans show the heights of cuts in the claystone/sandstone bedrock will range from less than 5 feet to about 100 feet. Tiered retaining walls with a maximum tier height of 30 feet and 15-foot wide benches between tiers were used to evaluate the retained cut slopes in the bedrock. The 30-foot tier height limit will aid in constructability and is similar to other walls in the area along US 160. Bench widths of 15 feet will facilitate construction of the support system and provide room for future access between tiers for maintenance.

The results of the global stability analyses indicate tiered cuts in bedrock with total heights, H, of up to 60 feet can be supported by soil nail wall systems. Preliminary estimates of required soil nail lengths range from 70 percent to 100 percent of the total wall height, H, for soil nails installed in a 5 foot by 5 foot grid pattern. Based on the preliminary evaluation, use of post-tensioned tie-back anchors may be more efficient than soil nails for walls in rock cuts with heights greater than 50 feet.

The required length of the tie-back anchors is estimated to range from 100 percent of the total wall height to 120 percent of the total wall height, H. Estimated tie-back anchor lengths are based on a preliminary analysis of two horizontal rows of tiebacks in each 30-foot tier with 10 foot horizontal spacing between anchors.

The extent of soil nails and tie-back anchors behind the cut face should be considered when estimating the right-of-way requirements for retaining wall construction. Recommended grading for cuts along each alignment is provided in Tables 2 through 6. The tables show the approximate elevations of each of the primary geologic strata for a range of stations and give the recommended cut slope grades for each layer.

Table 2: R5 Alignment Cut Slopes

Station		Approximate Elevation (ft)		Recommended Slope (H : V)	Material
Begin	End	Bottom	Top		
1+90	220+00	6695	6730	3 : 1	Silty clay
222+00	223+50	6685	6710	3 : 1	Silty clay
		6675	6685	3 : 1	Sandy Gravel
223+50	236+00	6695	6715	3 : 1	Silty clay
		6670	6695	3 : 1	Sandy Gravel
		6615	6670	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
236+00	244+00	6705	6730	3 : 1	Silty clay
		6675	6705	3 : 1	Sandy Gravel
		6575	6675	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
247+00	253+50	6590	6605	3 : 1	Sandy Gravel/Clayey Gravel
		6555	6590	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock

Table 3: RGM Alignment Cut Slopes

Station		Approximate Elevation (ft)		Recommended Slope (H : V)	Material
Begin	End	Bottom	Top		
185+00	247+00	6710	6750	3 : 1	Silty clay
247+00	255+00	6730	6760	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6760	6800	3 : 1	Sandy Gravel
		6800	6820	3 : 1	Silty Clay
255+00	264+00	6695	6795	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6795	6820	3 : 1	Sandy Gravel
		6820	6845	3 : 1	Silty Clay

Table 4: RGM2, RGM4, RGM5 and RGM6 Alignment Cut Slopes

Station		Approximate Elevation (ft)		Recommended Slope (H : V)	Material
Begin	End	Bottom	Top		
994+50	1014+50	6710	6750	3 : 1	Silty clay
1021+00	1022+50	6730	6745	3 : 1	Sandy Gravel
		6745	6765	3 : 1	Silty Clay
1022+50	1029+50	6720	6745	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6745	6780	3 : 1	Sandy Gravel
		6780	6800	3 : 1	Silty Clay
1029+50	1032+00	6720	6760	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6760	6800	3 : 1	Sandy Gravel
		6800	6820	3 : 1	Silty Clay
1032+00	1041+00	6695	6795	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6795	6820	3 : 1	Sandy Gravel
		6820	6845	3 : 1	Silty Clay

Table 5: RGM3 Alignment Cut Slopes

Station		Approximate Elevation (ft)		Recommended Slope (H : V)	Material
Begin	End	Bottom	Top		
975+00	991+00	6710	6730	3 : 1	Silty clay
991+00	996+00	6685	6705	3 : 1	Sandy Gravel
		6705	6740	3 : 1	Silty Clay
996+00	1009+50	6620	6680	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6680	6700	3 : 1	Sandy Gravel
		6700	6745	3 : 1	Silty clay
1009+50	1015+00	6620	6680	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6680	6710	3 : 1	Sandy Gravel
		6710	6740	3 : 1	Silty Clay
1017+50	1026+50	6620	6735	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
1026+50	1028+50	6650	6740	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6740	6765	3 : 1	Sandy Gravel
		6765	6775	3 : 1	Silty Clay
1028+50	1031+00	6650	6760	15'H x 30'V Tiered soil nail or tie-back walls	Sandstone/Claystone Bedrock
		6760	6770	3 : 1	Sandy Gravel

7.2 Retaining Structures for Fills

Retaining walls to support embankment fills can be cast-in-place or MSE walls. Fill wall heights should not exceed about 60 feet and walls with total heights greater than 30 feet should be tiered in a manner similar to cut walls. Bridge structures should be considered where embankment fill heights would be in excess of 40 feet. Reinforcement lengths for MSE walls that retain fill are estimated to range from 0.7 x T to 1.0 x T, where T is the height of an individual tier. The longer reinforcement lengths will be required for walls constructed on sloping ground.

8. Preliminary Foundation Recommendations

Spread footing foundations may be used to support full-height panel facing elements for the retaining walls. Cast-in-place wall footings and MSE walls should bear on undisturbed claystone/sandstone bedrock or dense alluvial terrace gravel. The gravelly and clayey soils on the steep slopes at the edges of the mesa are presumed to be unconsolidated colluvial deposits and are unsuitable for support of structure foundations. Where wall foundations are proposed in

areas identified as slumps or shallow landslides, or where unconsolidated colluvium has accumulated, the unsuitable materials should be removed to expose bedrock or dense alluvial terrace gravel. Deep foundation systems such as drilled shafts or micropiles that penetrate the bedrock or terrace gravel can be used to support retaining walls where site specific subsurface exploration identifies significant depths of unsuitable materials.

Bridge structure foundations should consist of drilled shafts or spread footings bearing in the claystone/sandstone bedrock or dense alluvial terrace gravel. It is anticipated that geotechnical parameters for bridge foundation design will be similar to those for the bridges of the recently constructed US 550/US 160 interchange. A minimum of 10 feet of unconsolidated colluvial deposits should be assumed at the locations of proposed bridge foundation elements for preliminary design purposes. The unconsolidated deposits are not suitable for foundation support and resistance from these deposits should be neglected in preliminary foundation design calculations.

Construction of drilled shafts that penetrate through colluvial gravel deposits to bedrock will be difficult. Casing will likely be required above the bedrock. Drilling will require large equipment capable of using augers of sufficient diameter to remove cobbles and boulders from the shafts. Penetrating hard bedrock may require rock augers or core barrels.

An unfactored bearing resistance of 10 ksf can be assumed for preliminary design of spread footings placed on bedrock or dense alluvial terrace gravel. Unfactored tip resistance of 75 ksf and unfactored skin resistance of 7 ksf can be assumed for preliminary design of drilled shafts that penetrate the bedrock.

9. Preliminary Pavement Design Recommendations

Traffic volume projections were not available at the time of this draft preliminary report. Average Daily Traffic (ADT) and corresponding 20-year design 18-kip equivalent single axle loads (ESALs) were assumed similar to the recently constructed Ramp A of the US 160/US 550 interchange.

Subgrade soils for the alignments are expected to consist of in-place weathered claystone/sandstone bedrock or pulverized and recompacted claystone/sandstone used as embankment fill. The in-place bedrock and compacted fill will have poor pavement support characteristics, similar to medium to high plasticity clay soil. The Ramp A pavement section was designed for these poor subgrade soils.

The construction plans for the US 160/US 550 interchange (CDOT Project No. NH 1602-114) show the design pavement section for Ramp A consists of 8 inches of Hot Mix Asphalt (HMA) over 6 inches of Aggregate Base Course (ABC) (Class 6) over 18 inches of ABC (Class 2). The required pavement structural section for the proposed alignments should be assumed the same as for Ramp A for preliminary design purposes.

La Plata County Road 220 was evaluated for use as a temporary detour for several months during critical phases of construction of the R5 Alternative. Pavement structural section improvements consisting of a minimum of 2 inches of new HMA overlay will be adequate to carry US 550 traffic for this short term. If the detour will require reconstruction to meet safety requirements, then full depth reclamation of the existing asphalt and base course for re-use as base course and 5 inches of new HMA is recommended.

10. GEOTECHNICAL CONSIDERATIONS

Areas of unstable slopes were identified along the north edge of the mesa, above US 160. These areas are shown as landslides and slumps on the geologic map in Appendix A, Figure A-11. The unstable slopes appear to consist of a mixture of sandy clay and gravel colluvium that has been transported by erosion from the alluvial clay and terrace gravel deposits to fill shallow depressions on the face of the slope. The unstable slopes are unsuitable for support of embankment fills or structures. The unstable soils should be removed from areas where unsupported embankment fills are proposed. Deep foundations, such as drilled shafts that penetrate through the unstable soils, can be used to support bridge structures and retaining walls. Drilled shaft foundations may also be designed to improve stability by directly supporting unstable slopes.

Groundwater was observed seeping from the unstable soil deposits and from isolated locations along the gravel bedrock contact. Subsurface drains may be required under embankments and structures to intercept shallow groundwater and divert it to a suitable outlet.

The sandy clay soils and claystone/sandstone bedrock may expand when wetted. The expansion potential of these materials will be increased if they are compacted. The sandy clay and bedrock materials should not be used as fill under concrete slabs-on-grade or as backfill for structures.

Excavations in the alluvial gravel layer and in the bedrock could create hazardous rockfall conditions during and after construction. The benches on tiered retaining walls provide catchment area that will help contain rockfall. However, rockfall some mitigation will be required at wall locations and where slopes in the gravel soils are unsupported. Evaluation of the potential

rockfall hazard and recommendations for rockfall mitigation are outside the scope of this preliminary investigation.

Results from previous geotechnical studies indicate a Corrosion Resistance Level of 2 (CR 2) should be assumed for design of buried concrete. Reinforcing steel and hardware for soil nails and tie-back anchors should be epoxy coated or otherwise protected from corrosion.

11. LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for use by the client for preliminary design purposes. The conclusions submitted in this report are based upon the data obtained from widely spaced exploratory borings, a field reconnaissance and the proposed type of construction. Yeh and Associates should review the adequacy of the field investigation and our recommendations as the design proceeds. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. If during construction, fill, soil, or water conditions appear to be different from those described herein, this office should be advised at once so reevaluation of the recommendations may be made. We recommend on-site observation of excavations and foundation bearing strata by a representative of the geotechnical engineer.

The scope of services for this project did not include, specifically or by implication, any environmental or biological (e.g., mold, fungi, and bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions or biological conditions. If the owner is concerned about the potential for such contamination, conditions or pollution, other studies should be undertaken.

The report was prepared in substantial accordance with the generally accepted standards of practice for geotechnical engineering as exist in the site area at the time of our investigation. No warranties, express or implied, are intended or made. The recommendations in this report are based on the assumption that Yeh and Associates will conduct an adequate program of construction testing and observation to evaluate compliance with our recommendations.

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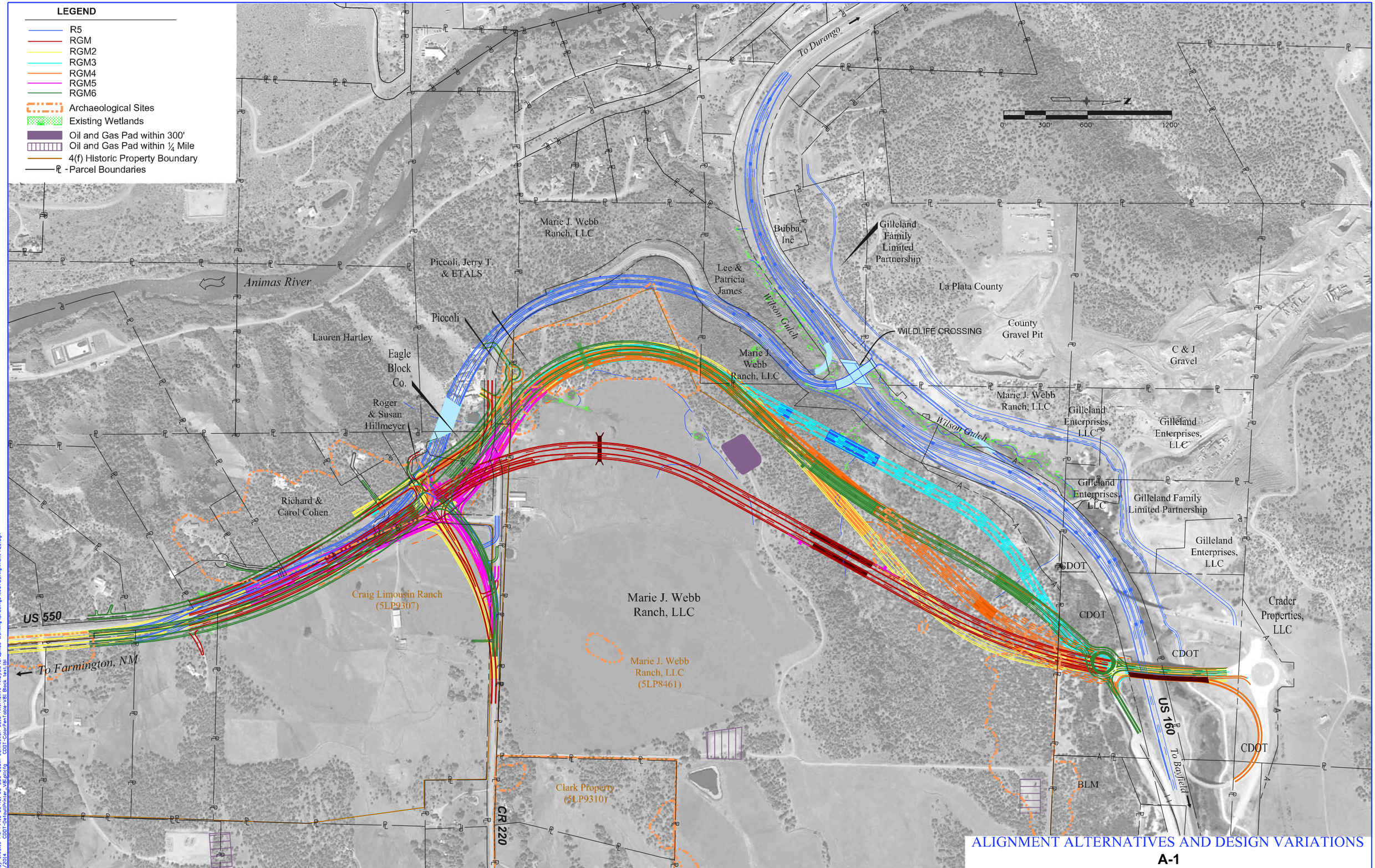
Appendix A

MAPS



ALIGNMENT ALTERNATIVES AND DESIGN VARIATIONS

A-1

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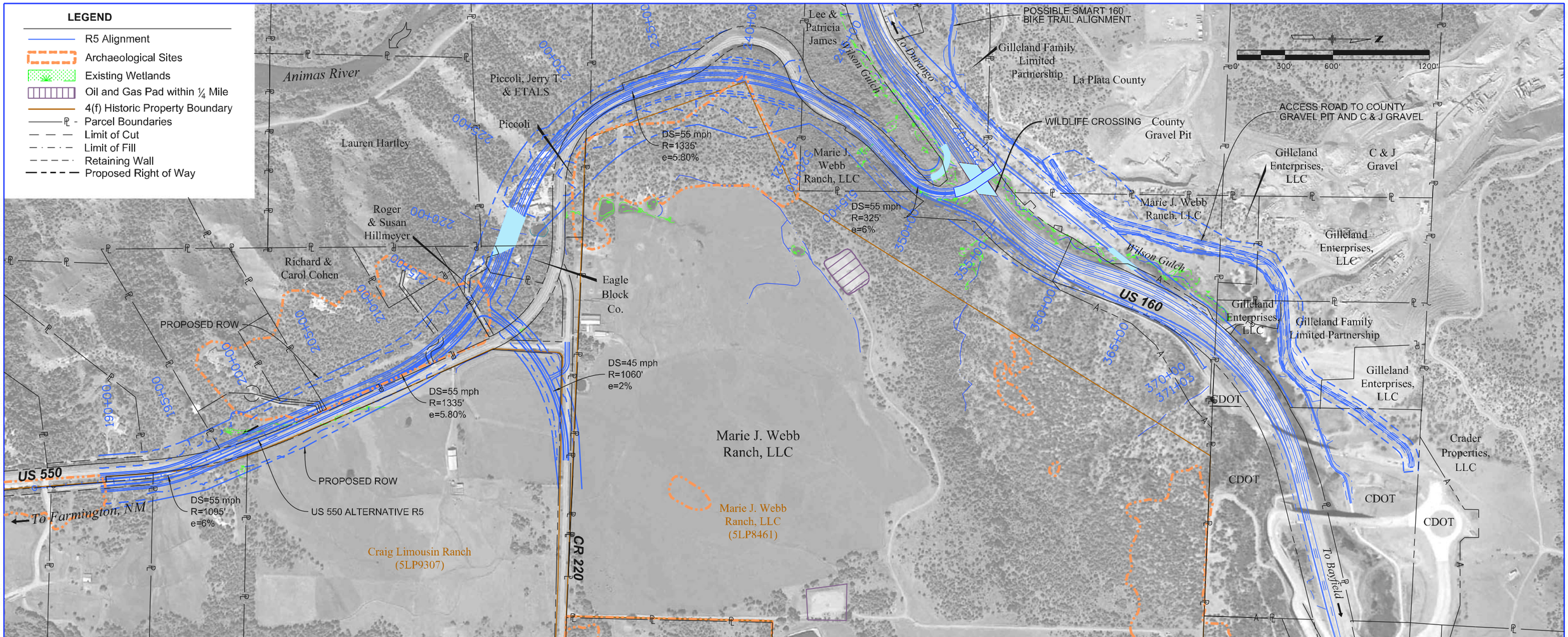

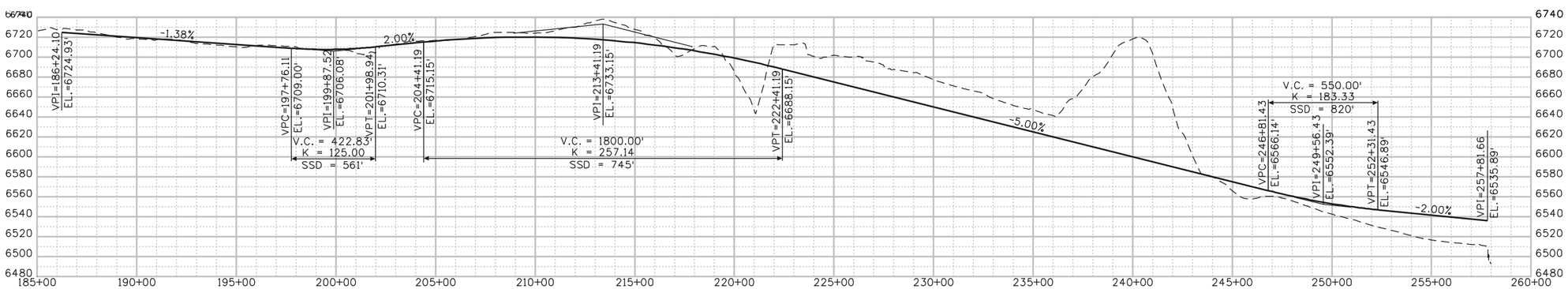

ALTERNATIVE R5 US 550 PLAN AND PROFILE

A-2

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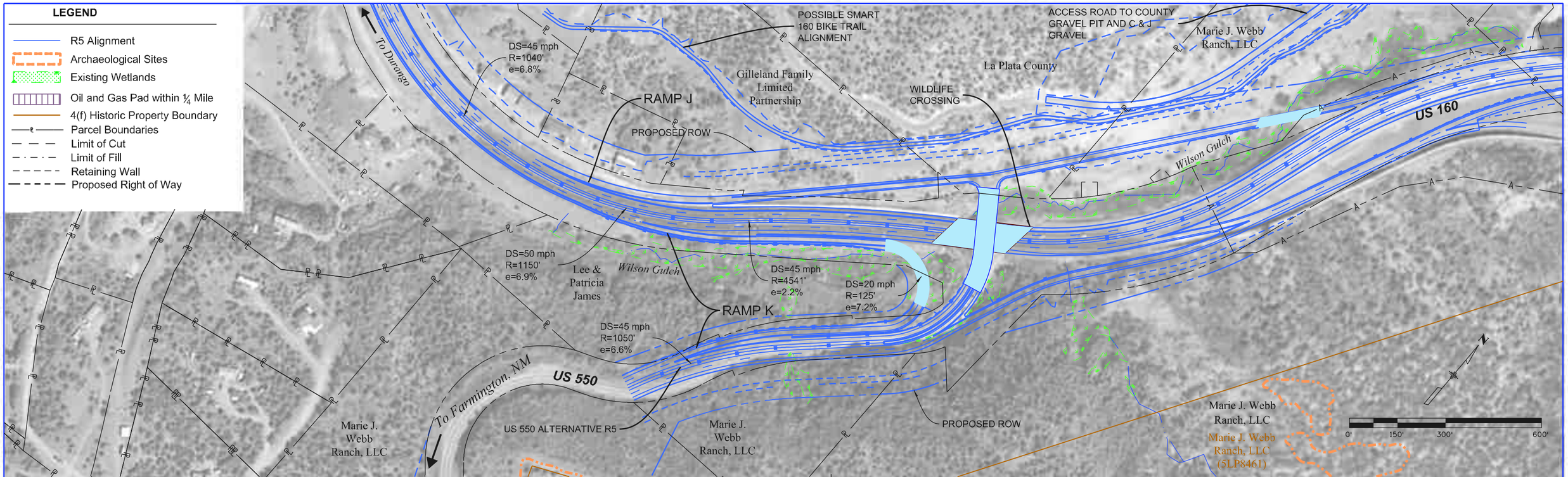

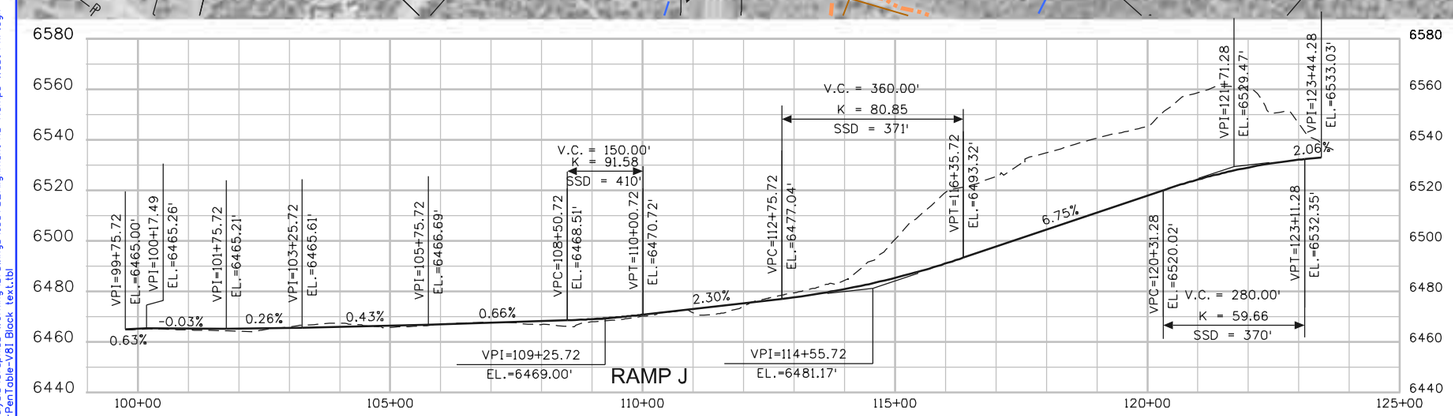

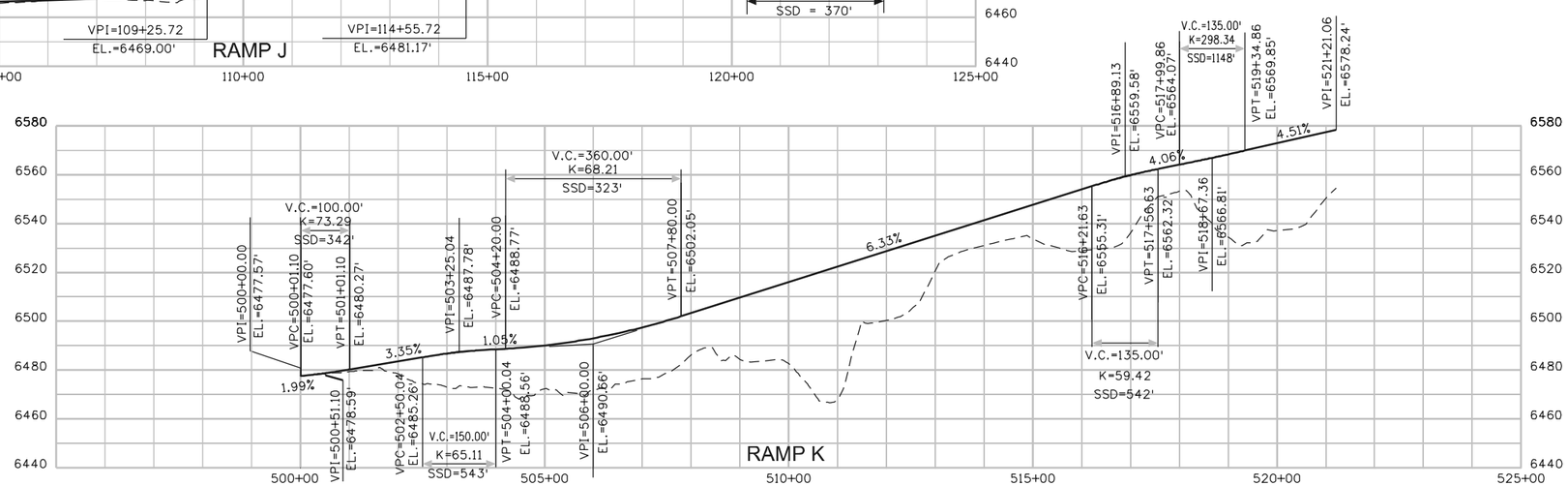

ALTERNATIVE R5
INTERCHANGE WEST RAMPS
J & K PLAN AND PROFILE

A-3

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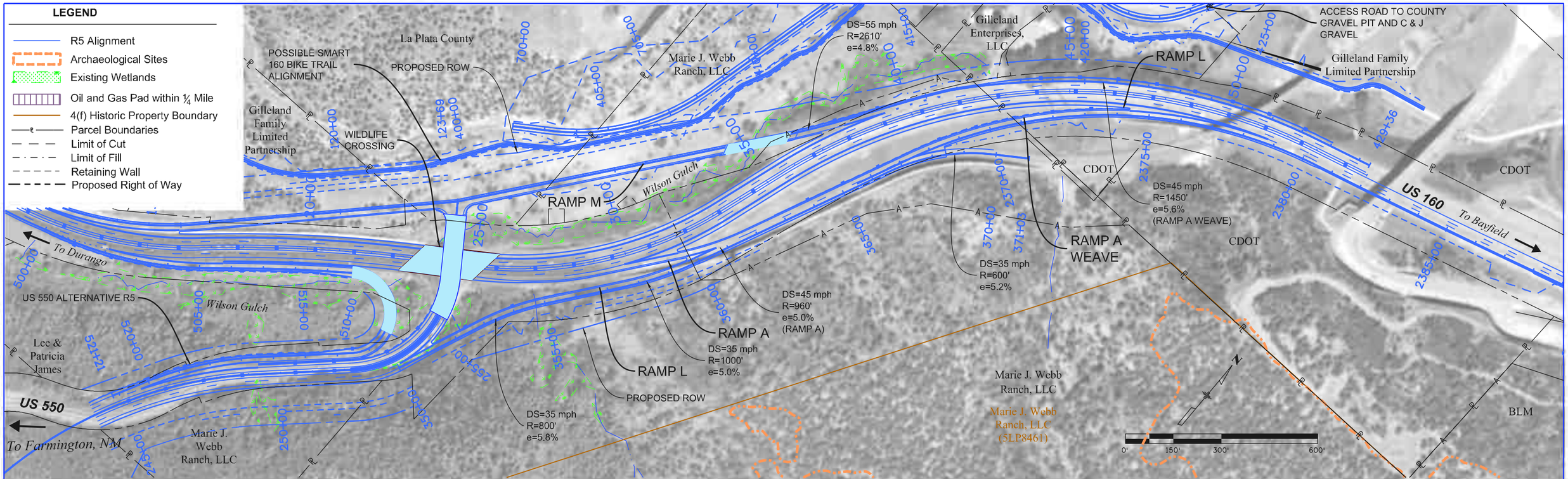

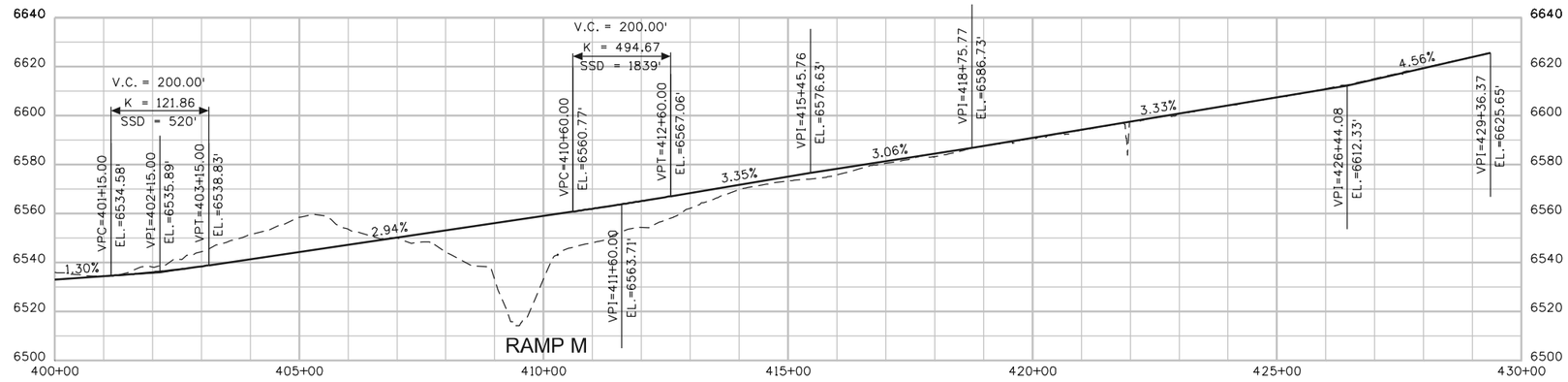

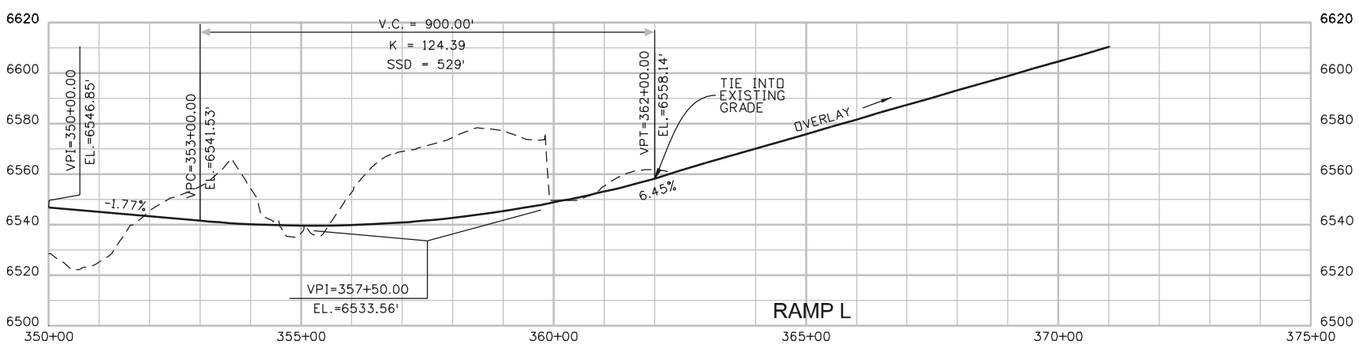

ALTERNATIVE R5
INTERCHANGE EAST RAMPS
L & M PLAN AND PROFILE
A-4

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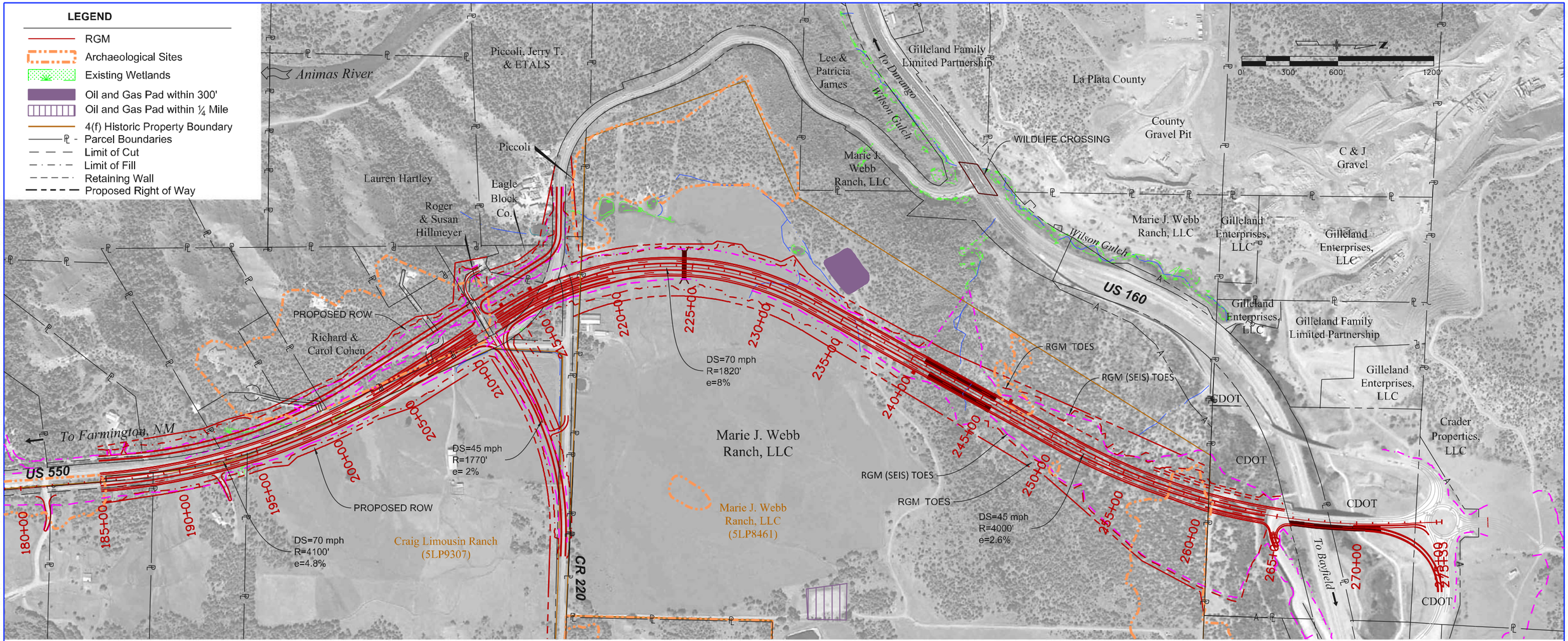

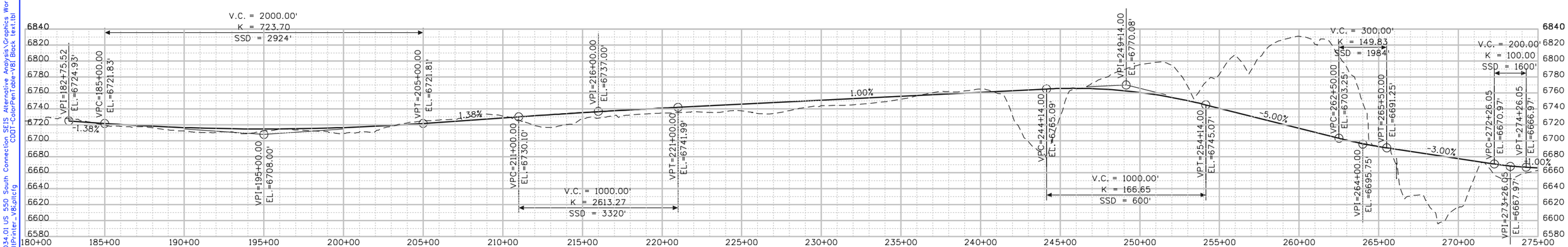

ALTERNATIVE RGM PLAN AND PROFILE

A-5

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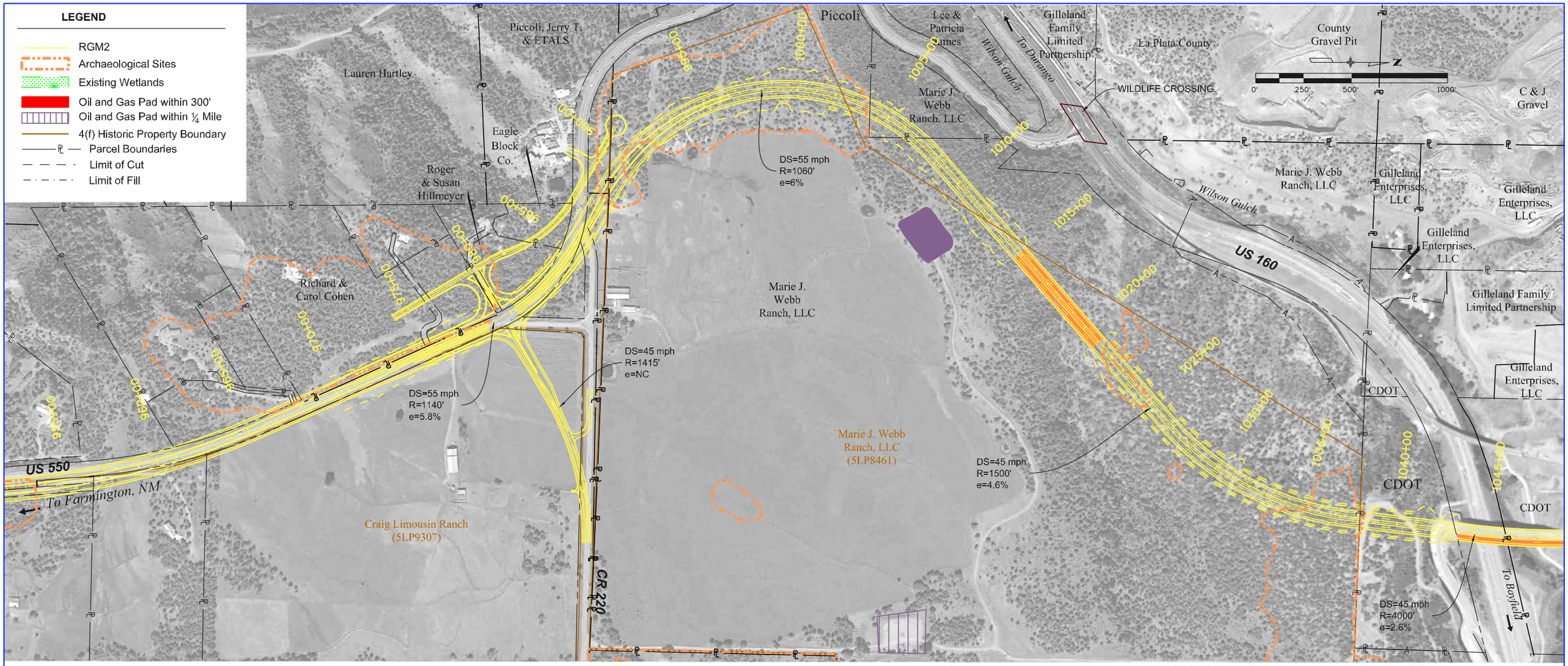

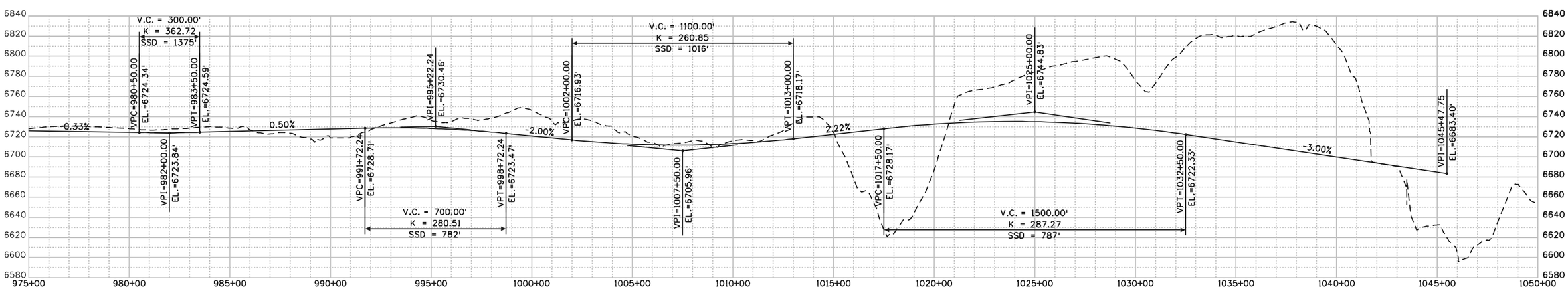

DESIGN VARIATION RGM2 PLAN AND PROFILE

A-6

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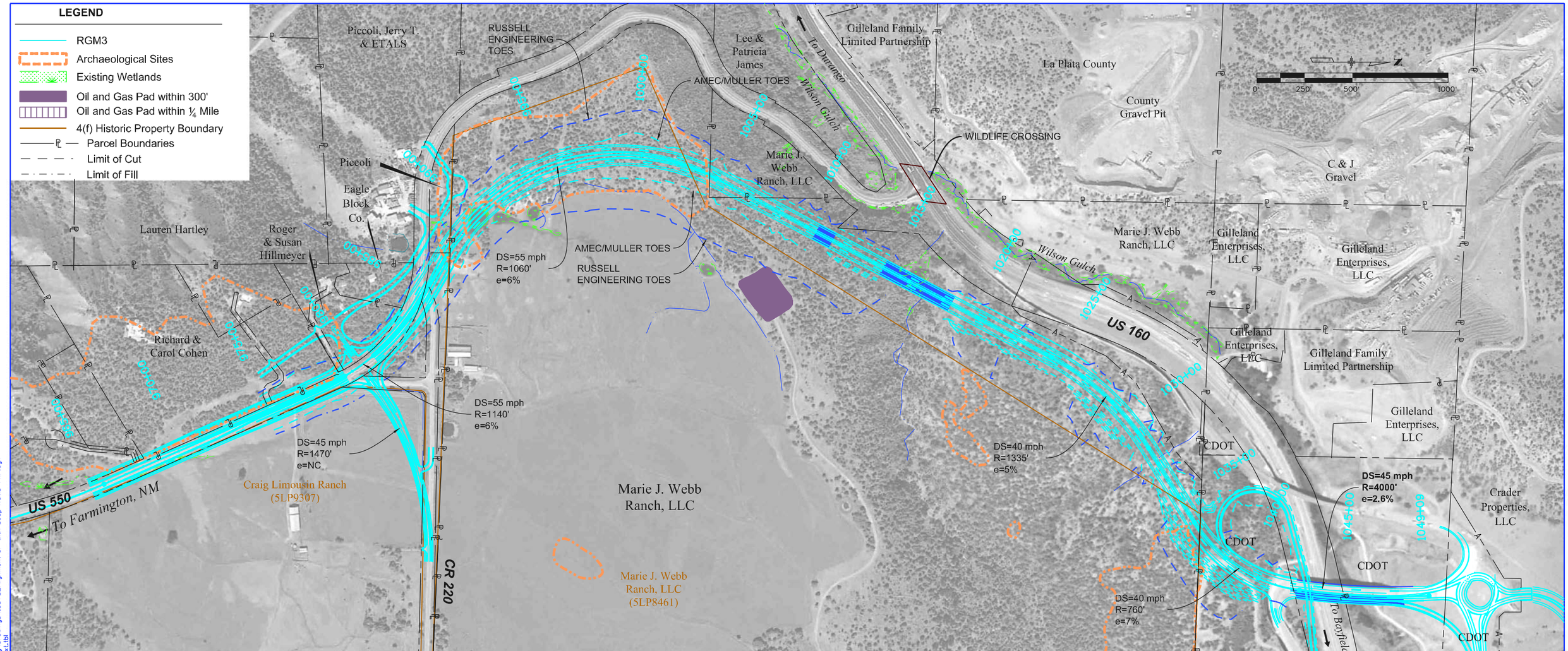

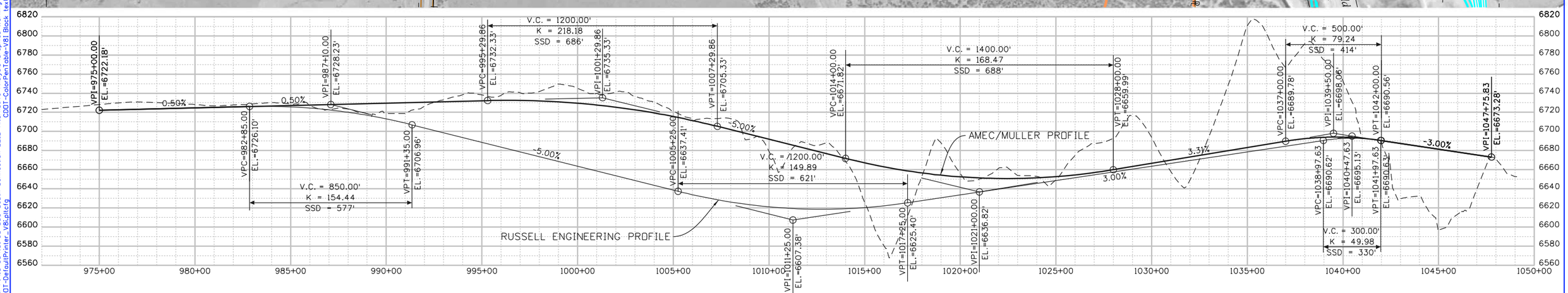

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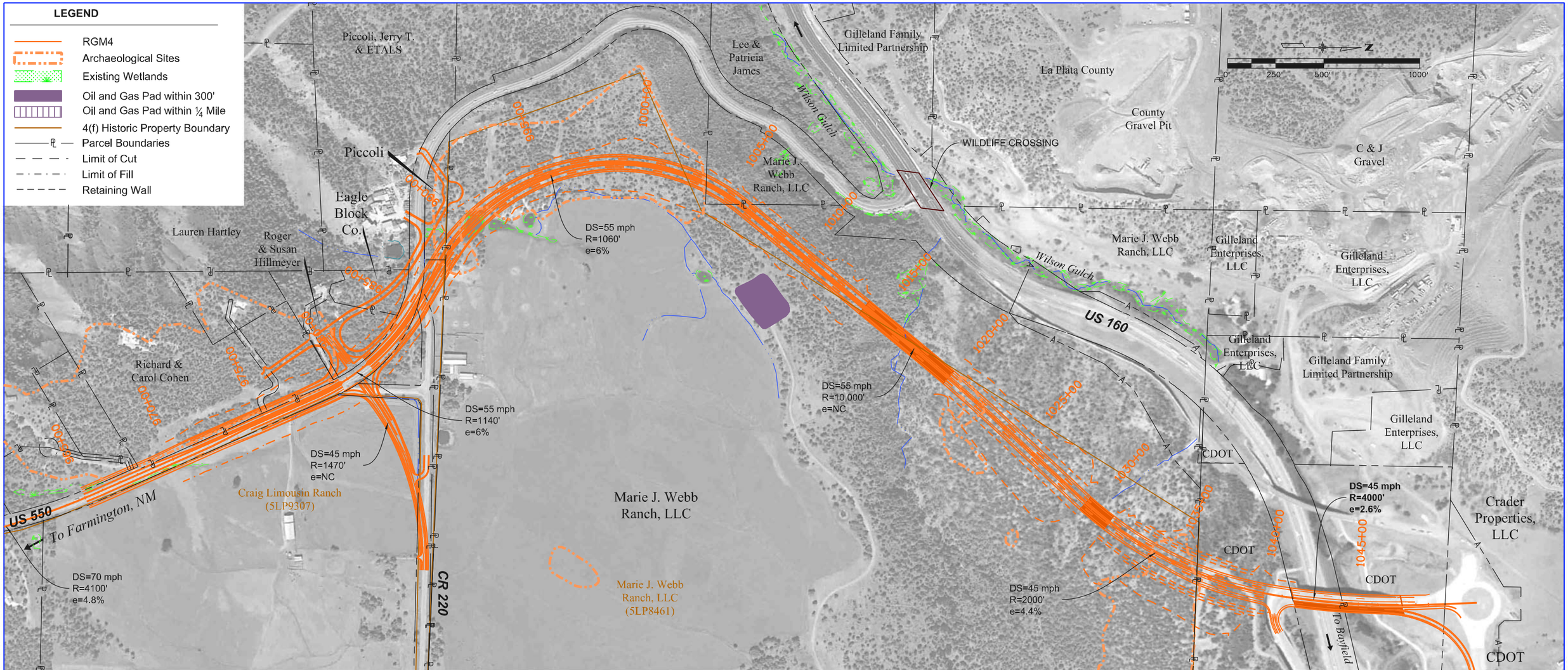

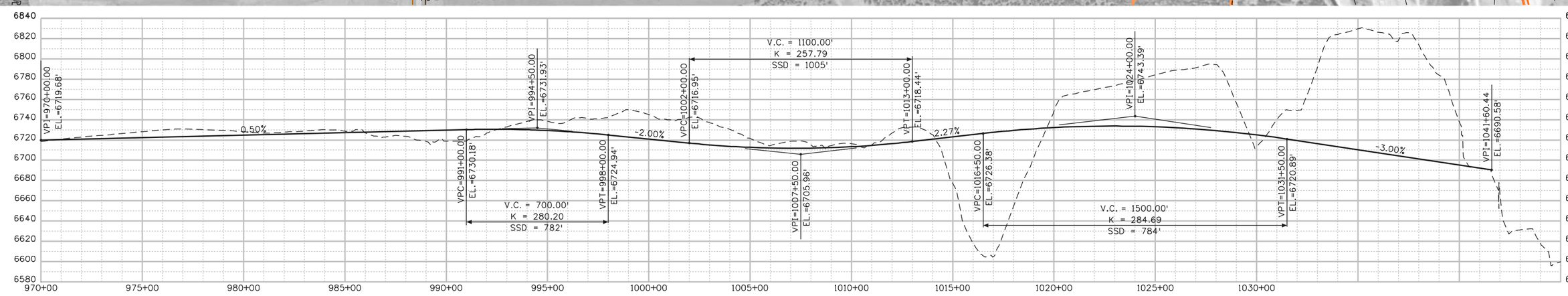

DESIGN VARIATION RGM4 PLAN AND PROFILE

A-8

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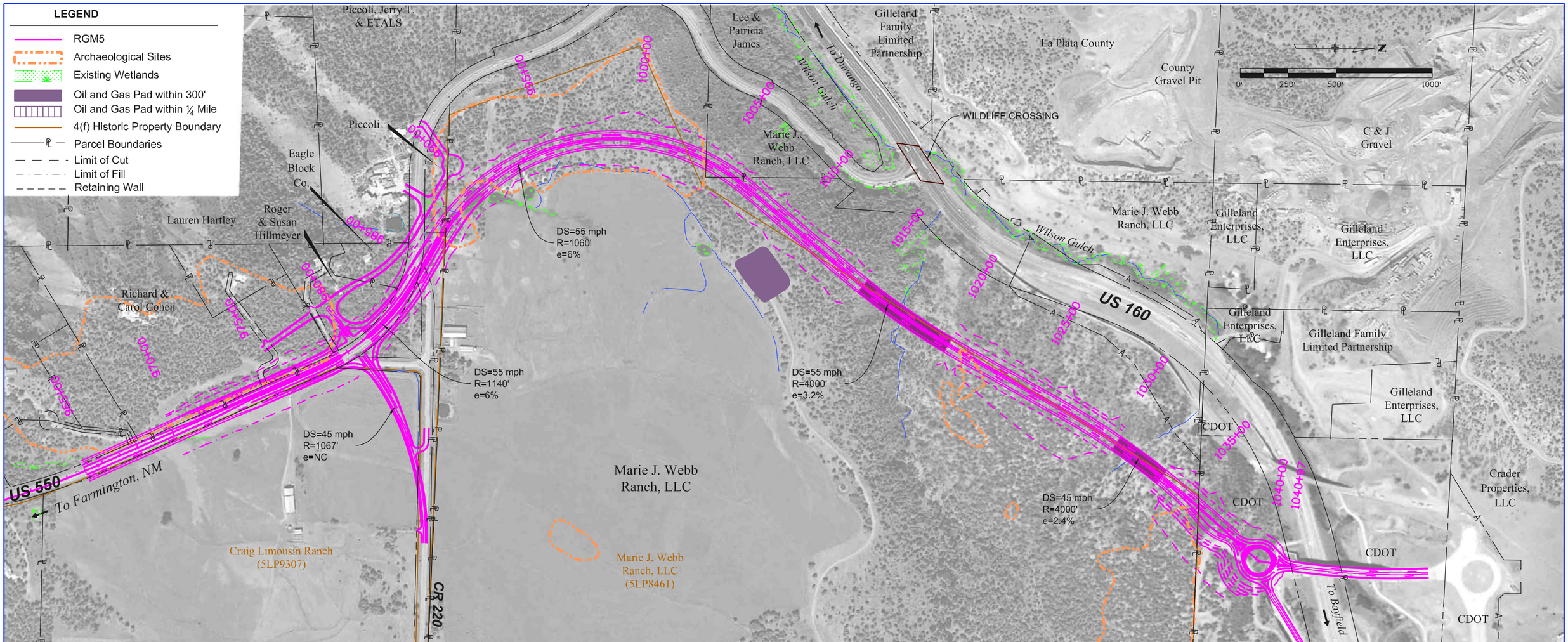

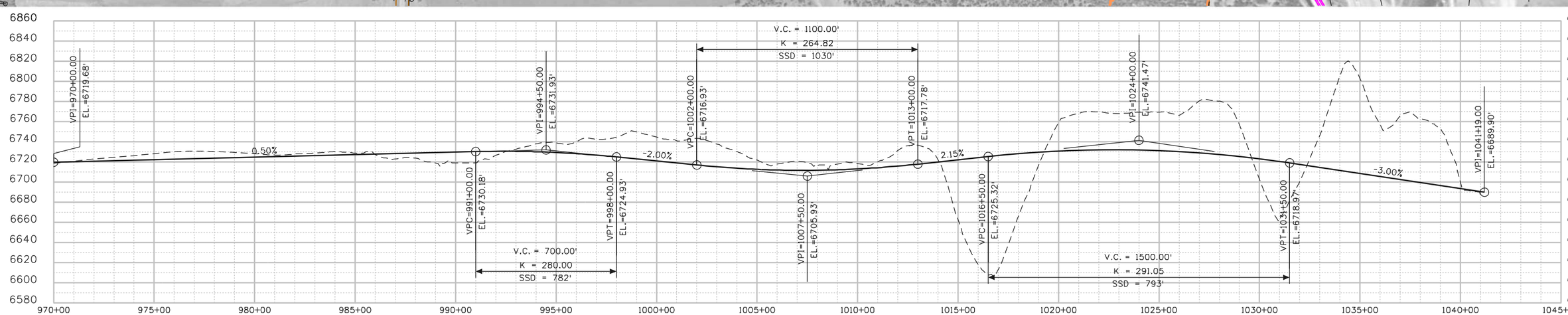

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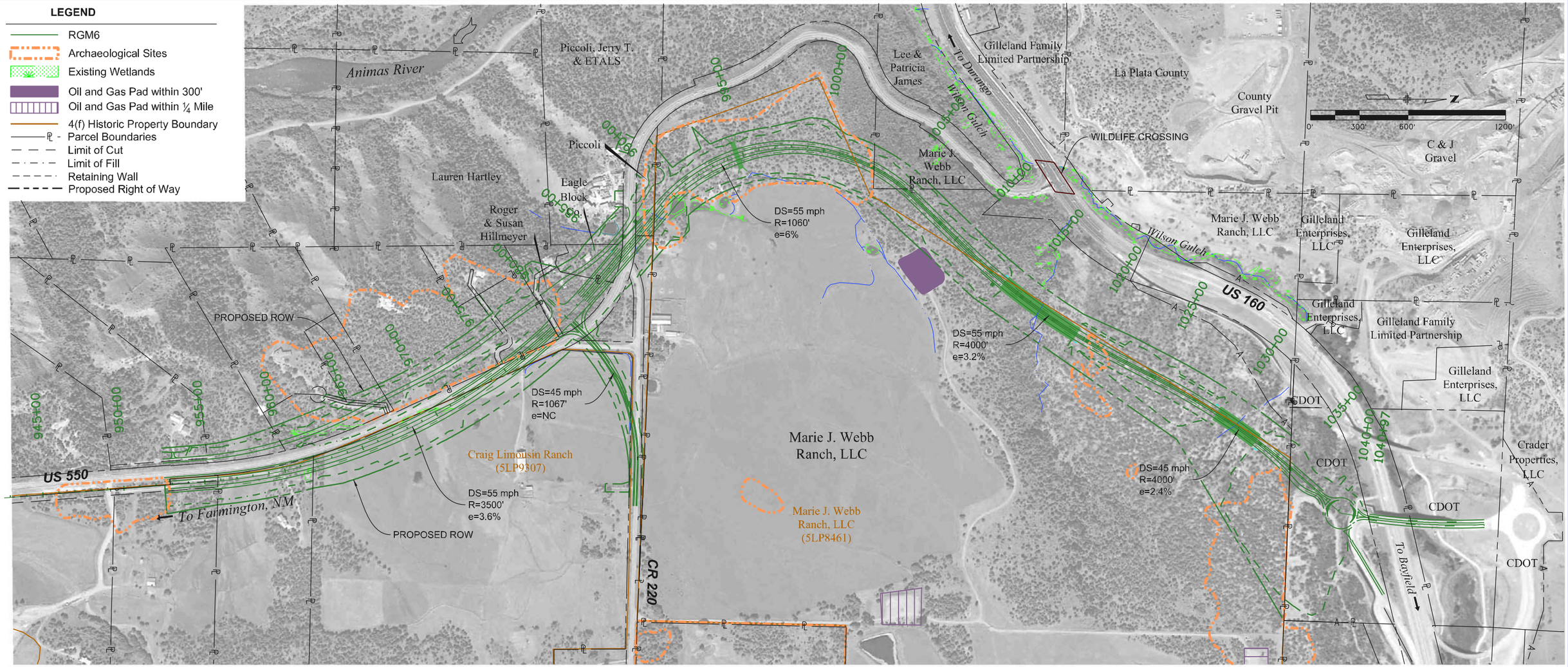

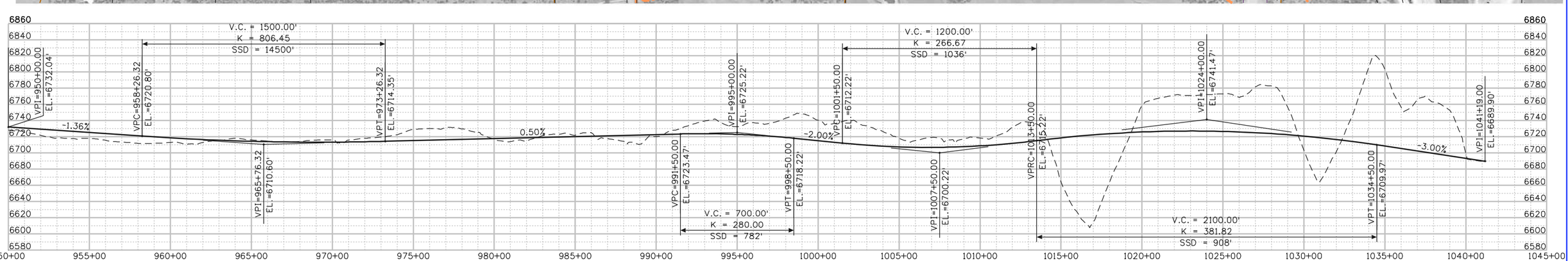

DESIGN VARIATION RGM6 PLAN AND PROFILE

A-10

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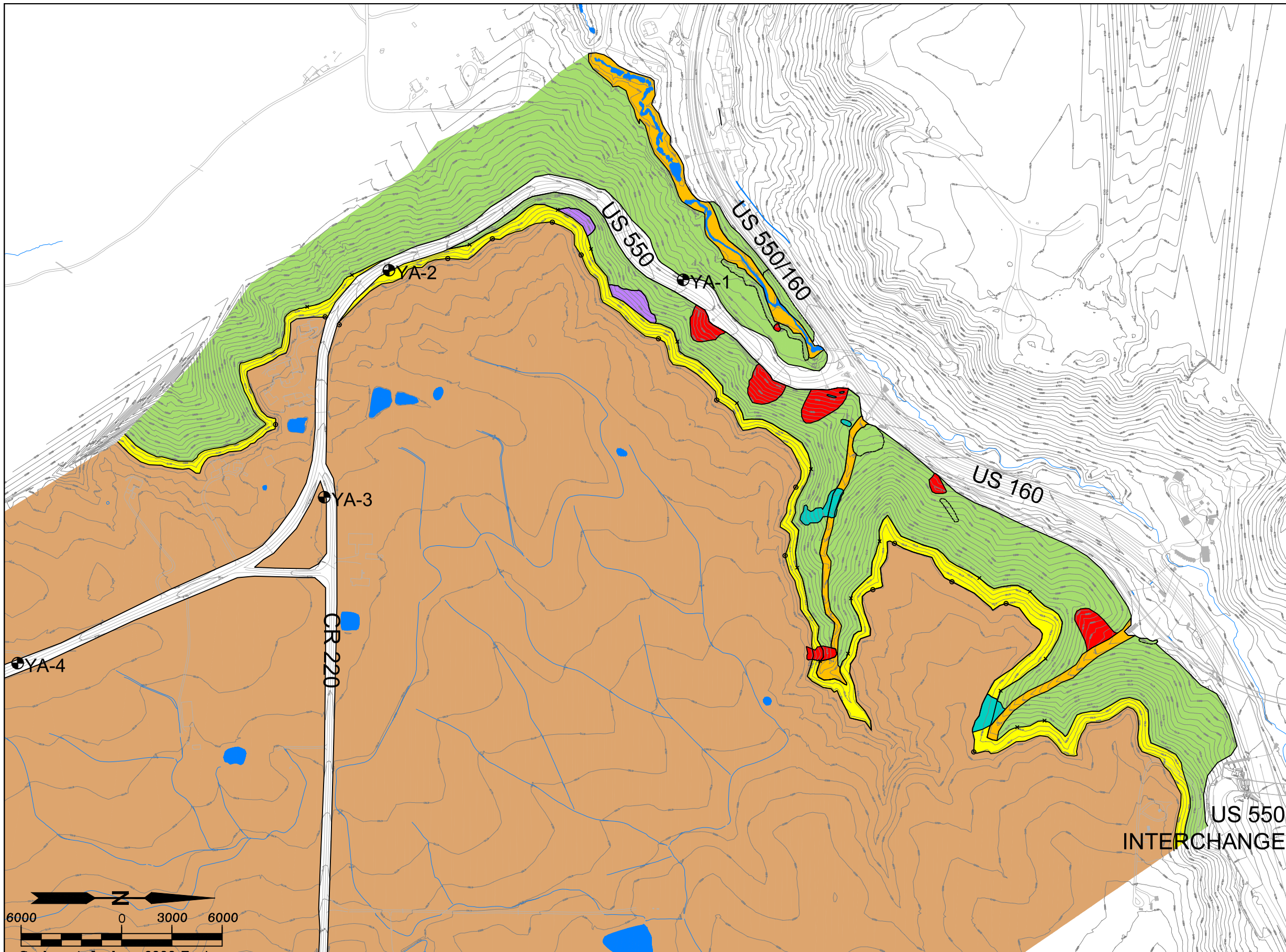

Geologic Map Legend

- Qa - Quaternary Alluvium: Unconsolidated silt, sand, and gravel deposited mainly in stream beds and on floodplains.
- Qls - Quaternary Landslide Deposits: Unconsolidated clay, silt, sand, and gravel which has been displaced by slumping, usually water-induced
- Seep
- Ka Outcrop - Cretaceous Animas Formation
- Ka - Cretaceous Animas Formation: Soft to medium hard, reddish brown and brown shale and claystone, moderately weathered, OR hard, resistant, gray sandstone, fresh to slightly weathered OR hard, resistant, gray, black, and pink conglomeratic sandstone, fresh to slightly weathered.
- Qg - Quaternary Terrace Gravels: Pebbles, cobbles, and gravel deposited on benches bordering stream valleys.
- Clay Qa - Modern Alluvium and Aeolian deposits
- Fill - Artificial Fill: Unconsolidated silt and sand placed during previous construction activities.
- Water - Includes streams active at time of mapping, rivers, and reservoirs.
- Kat - Welded Volcanic Tuff: isolated, interbedded in the Animas Formation
- × Observed Qg/Ka contact
- ⊙ Observed Qg/Clay Qa contact
- YA-1 Boring location with boring number

Yeh and Associates, Inc.
Consulting Engineers & Scientists

NO.	SHEET REVISION	BY	DATE

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DRAWN BY: GE DATE: 02-05-2014
CHECKED BY: TLA DATE:
DESIGNED FOR:
PROJECT NUMBER: 213-174
SCALE
HORIZ: 1:5000 VERT: 1:5000

PROJECT: CDOT US 550 SEIS

A-11

SHEET 1 OF 1

C:\Users\Gwen\Documents\Projects\213-174 US 550 Farmington Hill\geologic map rev 1.dwg, 2/12/2014 2:48:09 PM

Appendix B

LOGS OF EXPLORATORY BORINGS

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-1**
Sheet 1 of 1

Boring Began: 10/23/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/23/2013
Drill Bit:
Casing:
Weather:

Total Depth: 29.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.8 ft. 10 inches asphalt.	
								0.8 - 1.5 ft. sandy GRAVEL 8 inches ABC.	
								1.5 - 2.0 ft. sandy GRAVEL few cobbles, tanish brown, dry to moist, dense.	
								2.0 - 3.0 ft. clayey GRAVEL sandy, brown, moist, medium dense.	
					5/6/8	11		**3.0 - 11.0 ft. sandy CLAY** some small garvel, brown, moist, medium dense.	
	5								
					5/5/7	10			
	10							**11.0 - 15.0 ft. clayey GRAVEL** sandy, some cobbles, brown, moist, dense to very dense.	
					20/23/20	43			
	15							**15.0 - 19.0 ft. sandy CLAY** some gravel, tan to brown, moist, stiff.	
					8/15/17	23		**19.0 - 25.0 ft. CLAYSTONE INTERBEDDED WITH SANDSTONE**, tan-brown, predominantly decomposed, soft, (ANIMAS FORMATION).	
	20								
					30/50				
	25							**25.0 - 29.0 ft. CLAYSTONE INTERBEDDED WITH SANDSTONE**, tan-brown, slightly weathered to fresh, medium hard to hard, (ANIMAS FORMATION).	
					50/5"				
								Bottom of Hole at 29.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-2**
Sheet 1 of 1

Boring Began: 10/23/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/23/2013
Drill Bit:
Casing:
Weather:

Total Depth: 10.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.6 ft. 7 inches of asphalt.	
								0.6 - 1.6 ft. sandy GRAVEL 12 inches ABC.	
								1.6 - 10.0 ft. gravelly COBBLES and BOULDERS, tan and brown, dry to moist, very dense.	Auger refusal at 21 inches on first attempt. Offsett 5 feet south.
					30/35/30/3"	65			
	5								
					50/6"				
	10							Bottom of Hole at 10.0 ft.	Auger refusal at 10 feet in boulders

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-3**
Sheet 1 of 1

Boring Began: 10/23/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/23/2013
Drill Bit:
Casing:
Weather:

Total Depth: 39.5 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.5 ft. 6 inches of asphalt.	
								0.5 - 1.0 ft. sandy GRAVEL 6 inches ABC.	
								1.0 - 5.0 ft. sandy CLAY, dark brown, moist, very stiff.	
					7/8/16	15			
								5.0 - 19.0 ft. sandy CLAY, reddish brown, moist, hard, strong cementation, visible bands of calcium.	
	10				11/20/30	31			
					10/18/28	28			
	20				6/13/15	19		**19.0 - 23.0 ft. sandy CLAY**, tan-brown, moist, very stiff.	
					6/7/7	13		**23.0 - 32.5 ft. sandy CLAY**, brown, damp, medium stiff to stiff.	
	30							**32.5 - 39.5 ft. clayey GRAVEL** sandy, brown, damp, very dense.	
					50/6"				
	40							Bottom of Hole at 39.5 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-4**
Sheet 1 of 1

Boring Began: 10/24/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/24/2013
Drill Bit:
Casing:
Weather:

Total Depth: 37.5 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.7 ft. 8 inches of asphalt.	
								0.7 - 1.5 ft. sandy GRAVEL 10 inches ABC.	
								1.5 - 5.0 ft. sandy CLAY, reddish brown, moist, stiff.	
					5/6/9	11			
								5.0 - 16.0 ft. sandy CLAY, brown, moist, very stiff.	
	10				6/8/13	14			
					6/8/12	14			
								16.0 - 28.0 ft. sandy CLAY, reddish brown, moist, stiff.	
	20				5/7/9	12			
								28.0 - 37.5 ft. clayey GRAVEL and Cobbles, brown, moist, dense to very dense.	
	30				9/15/17	24			
								Bottom of Hole at 37.5 ft.	Auger refusal in cobbles at 37.5 feet

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-5**
Sheet 1 of 1

Boring Began: 10/24/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/24/2013
Drill Bit:
Casing:
Weather:

Total Depth: 5.5 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.7 ft. 7 inches of asphalt.	
								0.7 - 1.3 ft. 8 inches ABC.	
								1.3 - 5.5 ft. sandy CLAY, reddish brown, moist, stiff.	
					3/6/7	9			
					6/6/8	12			
	5								
								Bottom of Hole at 5.5 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: US 550 SEIS
Project Number: 213-174 Date:

Boring: **YA-6**
Sheet 1 of 1

Boring Began: 10/24/2013
Drilling Method: Solid-Stem Auger
Drill: CME 55
Driller: Trautner Geotech
Logged By: B. Bunker
Final By:
Inclination: Vertical

Completed: 10/24/2013
Drill Bit:
Casing:
Weather:

Total Depth: 5.5 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.7 ft. 7 inches of asphalt.	
								0.7 - 1.7 ft. 13 inches of ABC.	
								1.7 - 5.5 ft. sandy CLAY, reddish brown, moist, very stiff.	
					8/10/15	18			
					9/11/13	20			
	5								
								Bottom of Hole at 5.5 ft.	

Appendix C

LABORATORY TEST RESULTS

YEH & ASSOCIATES, INC

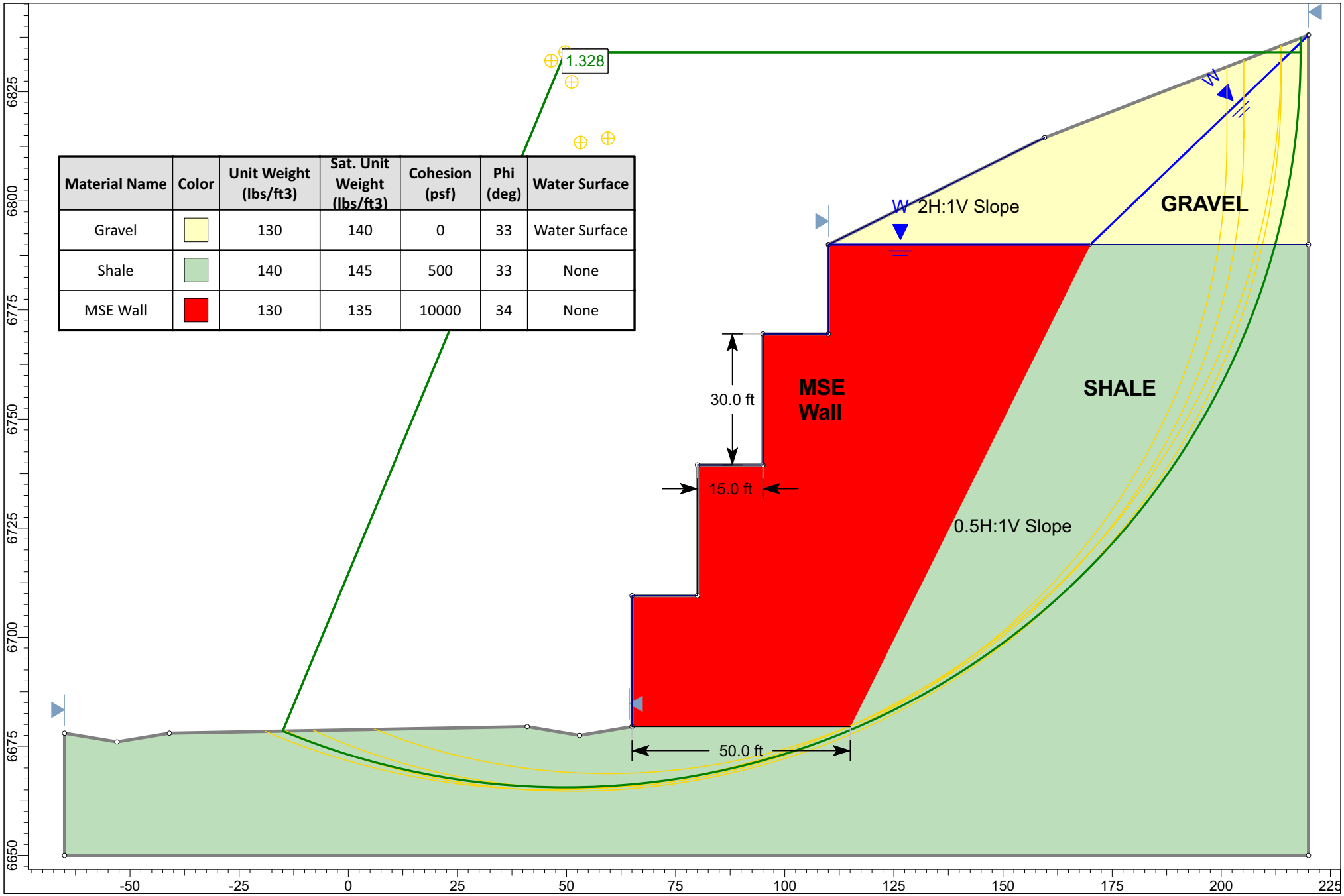
Summary of Laboratory Test Results

Project No: 213-174 Project Name: U.S Highway 550 SEIS Date: 12/23/2013

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate %	Chloride %	% Swell (+) / Consoli-dation (-)	Max Dry Density AASHTO T180-A	CLASSIFICATION	
Test Sample #	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
YA-1	3.5-8.5	Auger	10.6		13	59	28	35	25	15						A-2-6 (0)	SC
YA-1	8.5-13.5	Auger	8.8		18	50	32	39	21	18						A-2-6 (1)	SC
YA-3	4.0-9.0	Auger	12.8		0	7	93	40	19	21						A-6 (20)	CL
YA-4	1.5-4.0	Auger	18.4		1	24	75	47	22	25						A-7-6 (18)	CL
YA-5	1.5-3.0	SS	18.4		0	23	77	37	19	18						A-6 (13)	CL
YA-6	2.0-3.5	SS	11.9		3	18	79	37	20	17						A-6 (13)	CL

Appendix D

GLOBAL STABILITY ANALYSES



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	0	33	Water Surface
Shale		140	145	500	33	None
MSE Wall		130	135	10000	34	None

Scale
1:350

213-174, SH 550
RGM3 Alignment, STA 1035+00
STA 1035+00, 15 ft bench, 30 ft wall, Min 1.3 FOS

Global Stability Analysis Summary

Alignment RGM3

Station	Alignment Option	Analysis Section	FOS	Reinforcement/Nail/ Tieback Length	Remarks
989+00	25 ft fill, no cuts	-	1.42	10 ft Reinforcement length (=0.7H)	14 ft fill wall
	20 ft cut in clay	Upper Slope	2.62	-	No wall, 3:1 upper slope in clay (cut)
		Lower Slope	3.35	-	No wall, 4:1 lower slope in backfill (fill)
1003+50	36.5 ft cut in bedrock, 30 ft cut in gravel, 30 ft cut in clay	-	1.33	22 ft Soil Nails (=0.6H) Grade 60, #6 Bars	30 & 6.5 ft Soil Nail walls w/ 15 ft bench 2H:1V cut in gravel, 3H:1V cut in clay
1008+50	60 ft cut into Bedrock (cross section drawn in blue)	-	1.31	39 ft Soil Nails (=0.7H) Grade 60, #7 Bar	30 & 25.5 ft Soil Nail Walls w/ 15 ft bench 2:1 cut in gravel, 3:1 cut in clay
	20 ft cut into Gravel (cross section drawn in red)	Upper Slope	1.37	13 ft Soil Nails (=0.8H) Grade 60, #6 Bars	16.5 ft Soil Nail wall in Gravel, 2:1 cut in gravel, 3:1 cut in clay
		Lower Slope	1.93		
	20 ft cut into Clay (cross section drawn in orange)	Upper Slope	2.43	8.5 ft Reinforcement length (=0.7H)	12.5 ft fill wall, 2:1 cut in gravel, 3:1 cut in clay
		Lower Slope	1.68		
1009+50	60 ft cut into Bedrock (cross section drawn in blue)	-	1.30	55 ft long Tiebacks (=0.93H) Req'd 160 kip tendon strength	30 & 29 ft Tieback walls w/ a 15 ft bench Req'd 2 rows of tiebacks per 30 ft tier
	30 ft cut into gravel/bedrock (cross section drawn in red)	Upper Slope	2.01	Unretained upper cut slope	2H:1V cut in gravel; 3H:1V cut in clay
		Middle Slope	1.46	12.5 ft Soil Nails (=0.64H) Grade 60, #6 bars	19.5 ft Soil Nail Wall
		Lower Slope	1.30	21 ft Reinforcement length (=1.0H)	21.5 ft lower fill wall
	25 ft cut into clay/gravel (cross section drawn in orange)	Upper Slope	1.98	No upper wall	2H:1V cut in gravel, 3H:1V cut in clay
		Lower Slope	1.30	39.5 ft Reinforcement length (=1.13H)	35 ft lower fill wall
1010+50	60 ft cut into Bedrock (cross section drawn in blue)	-	1.30	55 ft long Tiebacks (=0.91H) Req'd 170 kip tendon strength	2 x 30 ft Tieback walls w/ a 15 ft bench Req'd 2 rows of tiebacks per 30 ft tier
		Lower Slope	1.40	No Lower Wall	1H:1V slope in shale bedrock
	60 ft fill wall (cross section drawn in orange)	Upper Slope	1.73	No Upper Wall	2H:1V cut slope in gravel
		Lower Slope	1.31	43 ft Reinforcement length (=0.7H)	61.5 ft fill wall
1013+00	60 ft cut into Bedrock (cross section drawn in blue)	Upper Slope	1.30	55 ft Tiebacks (=0.9H) Req'd 170 kip tendon strength	30 & 32.5 ft Tieback walls w/ a 15 ft bench Req'd 3 rows of tiebacks per 30 ft tier
		Lower Slope	2.02	No Lower Wall	Existing lower slope
	30 ft cut into gravel (cross section drawn in orange)	Upper Slope	1.55	No Upper Wall	2:1 cut slope in gravel
		Lower Slope	1.36	18.5 ft Reinforcement length (=0.7H)	27 ft lower fill wall

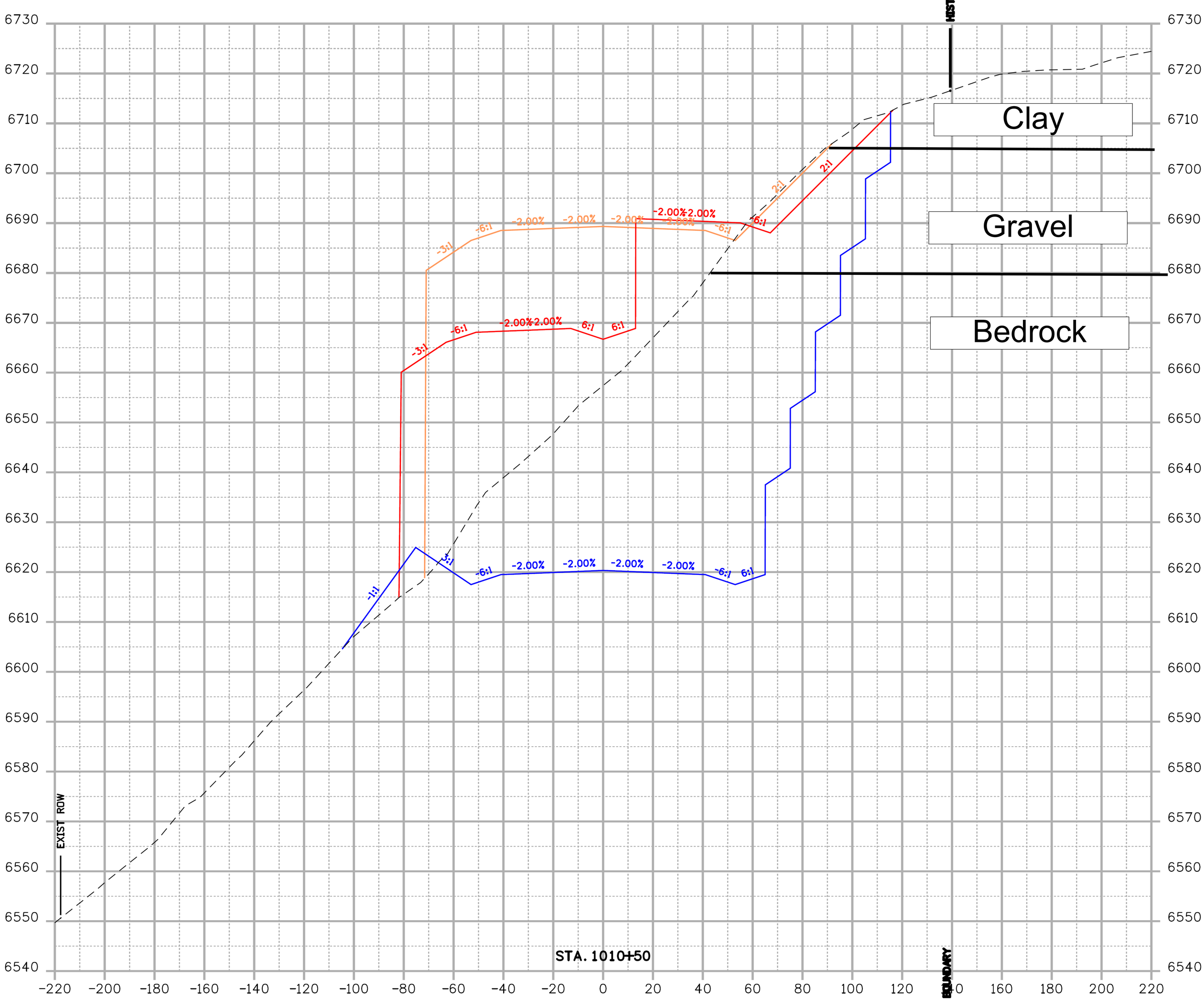
Global Stability Analysis Summary

Alignment RGM3 Cont.

Station	Alignment Option	Analysis Section	FOS	Reinforcement/Nail/ Tieback Length	Wall Remarks
1019+00	90 ft cut in bedrock (cross section drawn in blue)	-	1.30	65 ft Tiebacks (=0.72H) Req'd 170 kip tendon strength	3 x 30 ft Tieback walls w/ 15 ft benches Req'd 3 rows of tiebacks per 30 ft tier
	70 ft cut into shale, stepped alignment (cross section drawn in red)	Upper Slope	1.39	42 ft Tiebacks (=0.76H) Req'd 150 kip tendon strength	30 & 25 ft Tieback walls w/ a 15 ft bench Req'd 2 rows of tiebacks per 30 ft tier
		Middle Slope	2.41	12 ft Soil Nails (=0.69H)	17.5 high middle Soil Nail wall
		Lower Slope	2.06	No Lower Wall	Approx 1.8H:1V existing slope
1020+50	70 ft cut into shale (cross section drawn in blue)	Upper Slope	1.30	34 ft Nails (=0.6H), Gr 60, #6 bars	30&26.5 ft Soil Nail walls w/ 15 ft benches
		Lower Slope	1.89	9 ft Reinforcement length (=0.7H)	12.5 ft high lower wall (fill)
	40 ft cut into shale, 30 ft fill, tiered walls (cross section drawn in orange)	Upper Slope	1.50	17.5 ft Nails (=0.6H), Gr 60, #6 bars	29 ft upper Soil Nail wall, 13.5 ft middle wall (cut/fill), 30.5 ft lower wall (fill)
		Middle Slope	2.57	9.5 ft Reinforcement length (=0.7H)	
		Lower Slope	1.55	21 ft Reinfrocement length (=0.7H)	
1023+00	60 ft cut into shale (cross section drawn in blue)	Upper Slope	1.30	22.5 ft Nails (=0.6H), Gr 60, #6 bars	30&17.5 ft Soil Nail walls w/ 15 ft bench
		Lower Slope	1.99	7.5 ft Reinforcement length (=0.7H)	10.5 ft lower fill wall
1024+50	20 ft cut into bedrock, 35 ft fill (cross section drawn in red)	Upper Slope	1.55	14 ft Nails (=0.6H), Gr 60, #6 bars	23.5 ft upper Soil Nail wall
		Lower Slope	1.59	23 ft Reinforcement length (=0.7H)	33 ft lower fill wall
1027+00	95 ft cut into shale (cross section drawn in blue)	-	1.31	65 ft Tiebacks (=0.77H) Req'd 160 kip tendon strength	2x30 & 1x24 ft Tieback walls w/ 15 ft benches; Req'd 3 rows of tiebacks per 30 ft tier
1029+00	105 ft cut into shale (cross section drawn in blue)	-	1.30	67 ft Tiebacks (=0.65H) Req'd 175 kip tendon strength	30,30,&12.5 ft Tieback walls w/ 15 ft bench; Req'd 3 rows of tiebacks per 30 ft tier
1031+50	35 ft fill, no cuts (cross section drawn in blue)	-	1.37	17 ft Reinforcement length (=0.7H)	24.5 ft fill wall
1035+00	110 ft cut in shale, 30 ft cut in gravel	-	1.31	80 ft Tiebacks (=0.72H) Req'd 200 kip tendon strength	3x30 ft and 1x20.5 ft tieback walls w/ 15 ft benches; Req'd 3 rows of tiebacks per 30 ft tier

RGM3 Alignment

Station 1010+50


Clay
Gravel
Bedrock
EXIST ROW
STA. 1010+50
-3:1
-6:1
-2.00%
-2.00%
-2.00%
-2.00%
-2.00%
-6:1
2:1
2:1
-6:1
-3:1
-6:1
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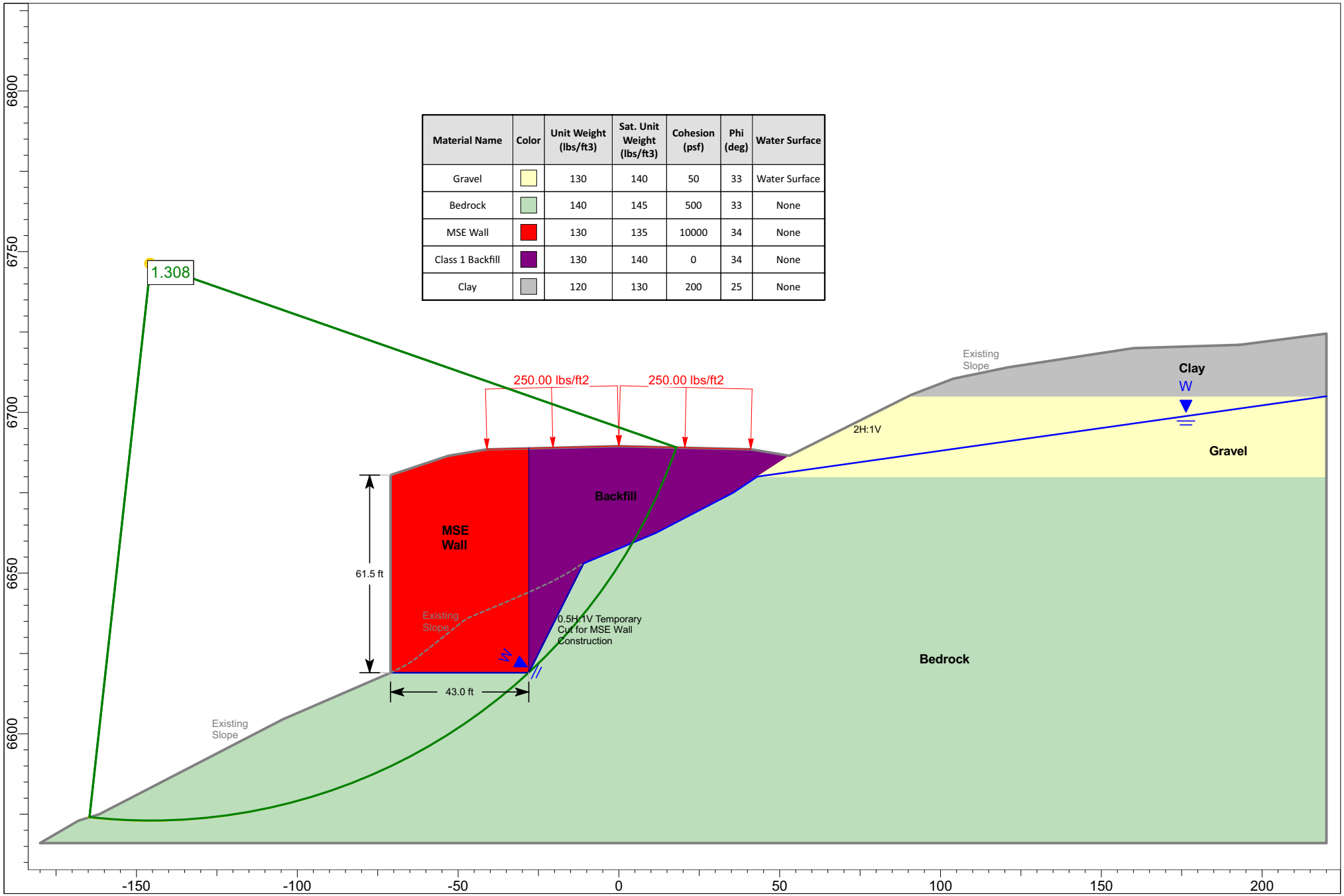
20
40
60
80
100
120
140
160
180
200
220

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	50	33	Water Surface
Bedrock		140	145	500	33	None
MSE Wall		130	135	10000	34	None
Class 1 Backfill		130	140	0	34	None
Clay		120	130	200	25	None

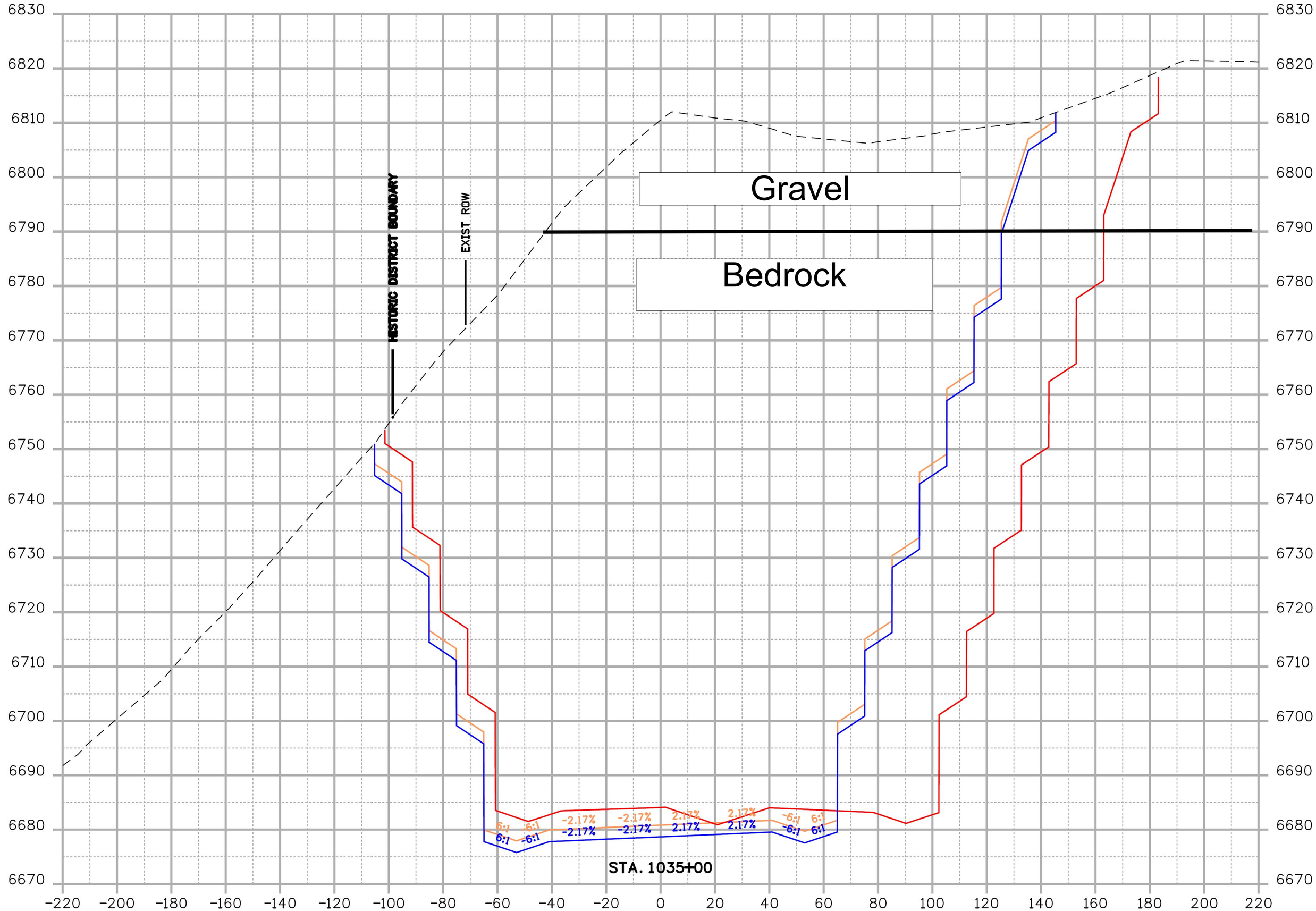


Scale
1:475

213-174, SH 550
RGM3 Alignment, STA 1010+50
STA 1010+50, Gravel Cut, Lower 61.5 ft Fill Wall

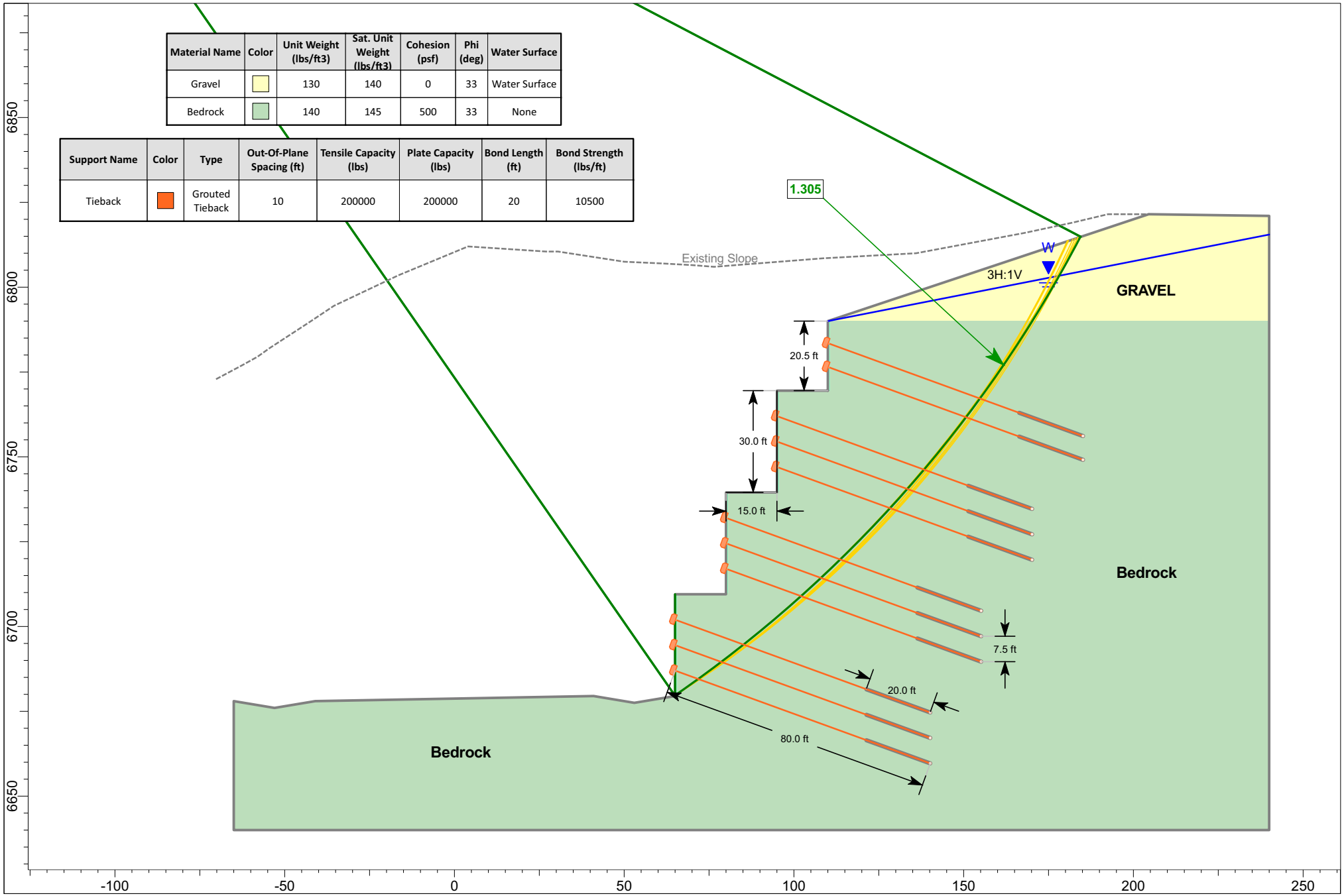
RGM3 Alignment

Station 1035+00


Gravel
Bedrock
HISTORIC DISTRICT BOUNDARY
EXIST ROW
STA. 1035+00
6:1
-6:1
-2.17%
2.17%
6830
6820
6810
6800
6790
6780
6770
6760
6750
6740
6730
6720
6710
6700
6690
6680
6670
-220
-200
-180
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60
80
100
120
140
160
180
200
220

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	0	33	Water Surface
Bedrock		140	145	500	33	None

Support Name	Color	Type	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Length (ft)	Bond Strength (lbs/ft)
Tieback		Grouted Tieback	10	200000	200000	20	10500



Scale
1:450

213-174, SH 550
RGM3 Alignment, STA 1035+00
STA 1035+00, 110 ft Tieback Wall, 15 ft benches

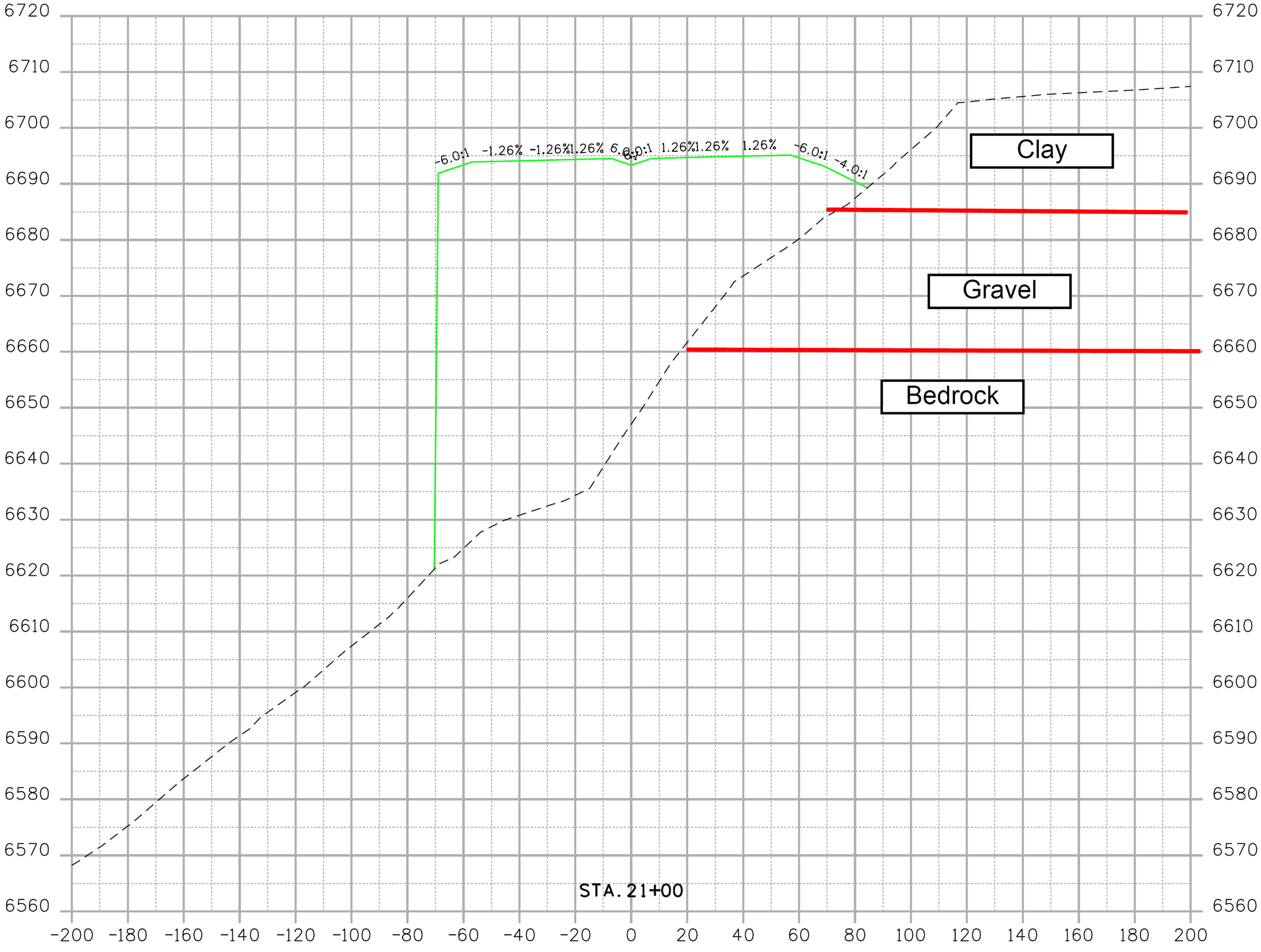
Global Stability Analysis Summary

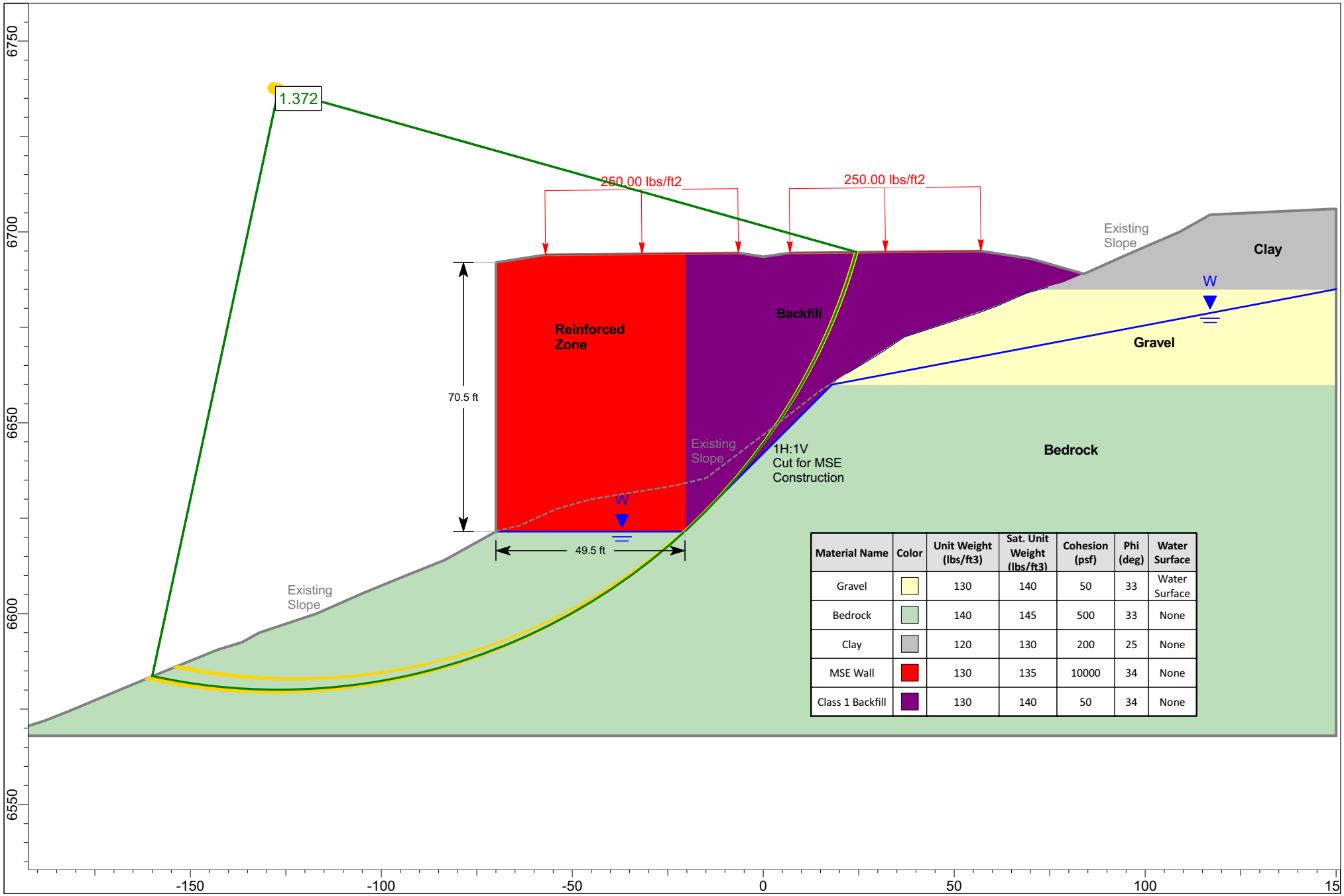
Alignment R5

Station	Alignment Option	Analysis Section	FOS	Reinforcement/Nail/Tieback Length	Remarks
21+00	70.5 ft of fill on bedrock	-	1.37	49.5 ft Reinforcement length (=0.7H)	70.5 ft fill wall 1H:1V cut in rock during construction
25+00	25 ft Cut into Gravel, 3H:1V Open cut in clay	-	1.33	18 ft nails (=0.8H) Grade 60, #7 bars	22.5 ft Soil Nail Wall 3H:1V open cut slope in clay
	25 ft cut into gravel, 20 ft cut into clay	Upper Slope	1.31	24 ft nails (=0.8H), Grade 60, #7 bars	41 ft Soil Nail Wall, No open cut slopes
		Lower Slope	1.83	No Lower Wall	Existing Slope, Approx. 1.5H:1V
30+00	15 ft cut into Bedrock, 3H:1V open cuts in gravel & clay	-	1.51	13.5 ft nails (=0.6H) Grade 60, #6 bar	22.5 ft Soil Nail Wall 3H:1V cuts in Gravel & Clay
	15 ft cut into bedrock, 30 ft cut into gravel	-	1.36	32 ft nails (=0.64H) Grade 60, #6 bar	50 ft Soil Nail Wall No open cut slopes
34+50	45 ft cut into bedrock, 3H:1V open cuts in gravel & clay	Upper Slope	1.36	29 ft Nails (=0.64H), Grade 60, #6 bar	30 ft upper Soil Nail wall, 10.5 ft lower fill wall 3H:1V Cut in Gravel & Clay
		Lower Slope	1.96	7.5 ft Reinforcement length (=0.7H)	
	65 ft cut into bedrock and gravel, 3H:1V cut in gravel	Upper Slope	1.35	37 ft Nails (=0.65H), Grade 60, #7 bar	57 ft upper Soil Nail wall, 10.5 ft lower fill wall 3H:1V cut slope in Gravel
		Lower Slope	1.96	7.5 ft Reinforcement length (=0.7H)	
39+50	75 ft cut into bedrock, 3H:1V open cuts in gravel & clay	-	1.30	55 ft long Tiebacks (=0.73H) Req'd 150 kip tendon strength	2x30 ft, 1x15 ft Tieback walls w/ 15 ft benches Req'd 2 rows of tiebacks per tier
	105 ft cut into bedrock and gravel, 3H:1V open cut in clay	-	1.30	70 ft long Tiebacks (=0.67H) Req'd 170 kip tendon strength	3x30 ft, 1x15 ft Tieback walls w/ 15 ft benches Req'd 3 rows of tiebacks per 30 ft tier
40+50	85 ft cut into bedrock, 3H:1V open cuts in gravel & clay	-	1.30	60 ft long Tiebacks (=0.71H) Req'd 150 kip tendon strength	2x30 ft, 1x25 ft Tieback walls w/ 15 ft benches Req'd 3 rows of tiebacks per 30 ft tier
	110 ft cut into bedrock & gravel, 3H:1V open cut in clay	-	1.31	75 ft long Tiebacks (=0.68H) Req'd 170 kip tendon strength	3x30 ft, 1x20 ft Tieback walls w/ 15 ft benches Req'd 3 rows of tiebacks per 30 ft tier
	135 ft cut into bedrock, gravel, & clay, no open cut slopes	-	1.30	80 ft long Tiebacks (=0.6H) Req'd 210 kip tendon strength	4x30 ft, 1x13 ft Tieback walls w/ 15 ft benches Req'd 3 rows of tiebacks per 30 ft tier
46+00	40 ft of fill on bedrock/embankment fill	-	1.66	28 ft Reinforcment length (=0.7H)	40 ft fill wall 1H:1V cut in rock during construction

R5 Alignment

Station 21+00


6720
6710
6700
6690
6680
6670
6660
6650
6640
6630
6620
6610
6600
6590
6580
6570
6560
-6.0:1
-1.26%
-1.26%
1.26%
6.0:1
1.26%
1.26%
1.26%
-6.0:1
-4.0:1
Clay
Gravel
Bedrock
STA. 21+00
-200
-180
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60
80
100
120
140
160
180
200



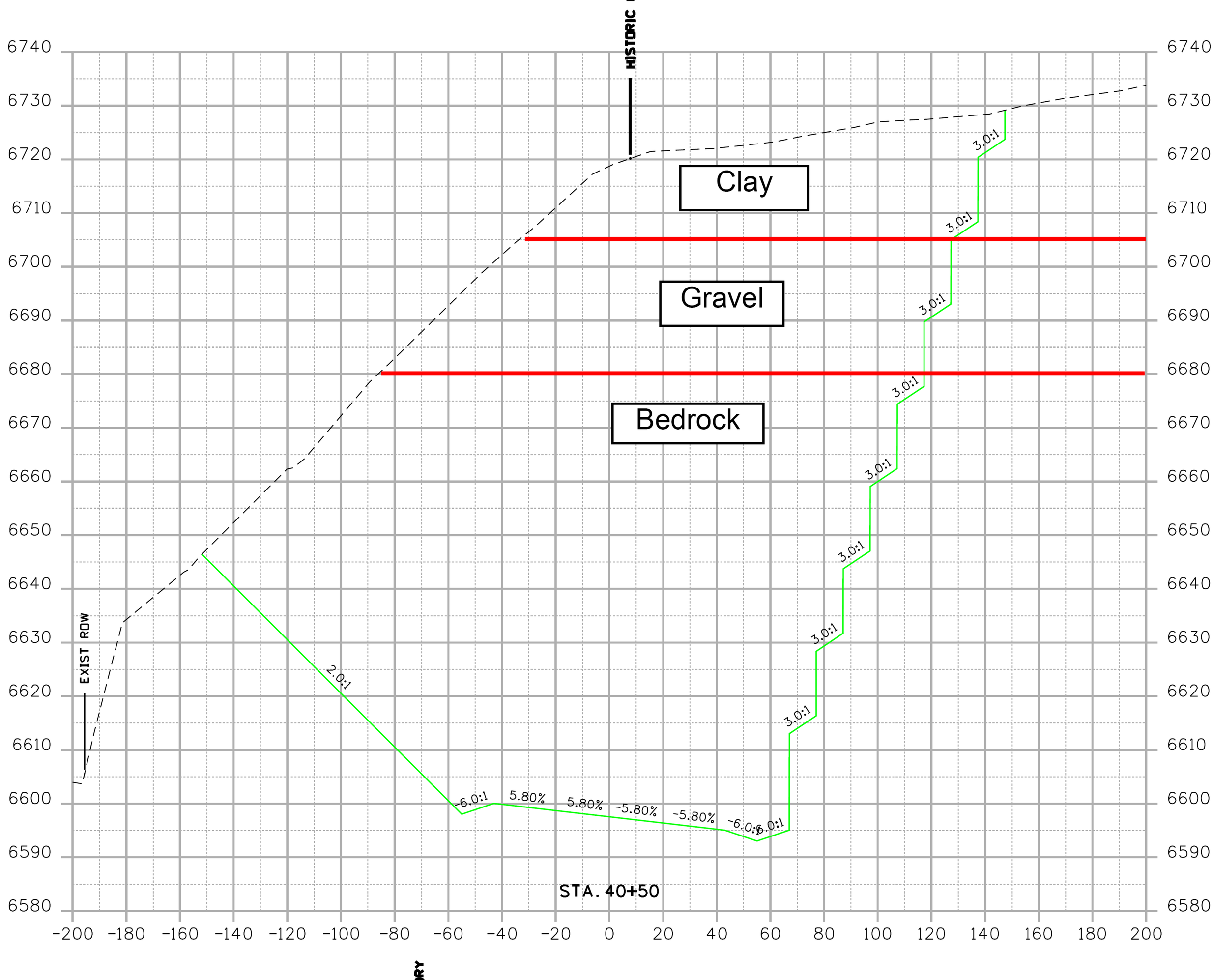
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	50	33	Water Surface
Bedrock		140	145	500	33	None
Clay		120	130	200	25	None
MSE Wall		130	135	10000	34	None
Class 1 Backfill		130	140	50	34	None

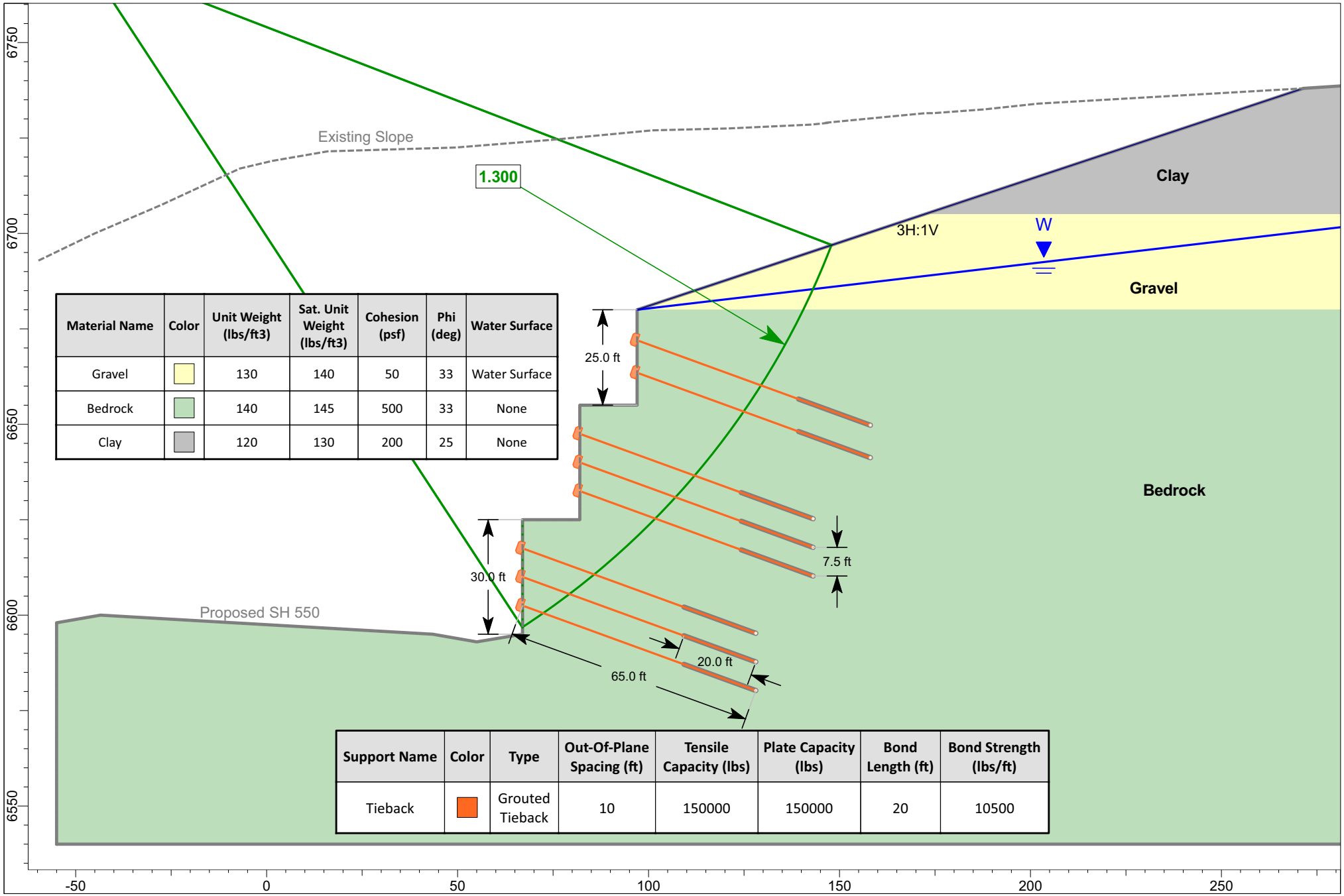
Scale
1:400

213-174, SH 550
R5 Alignment, STA 21+00
STA 21+00, 70 ft Fill Wall

R5 Alignment

Station 40+50


HISTORIC
Clay
Gravel
Bedrock
EXIST ROW
2.0:1
-6.0:1
5.80%
5.80%
-5.80%
-5.80%
-6.0:1
6.0:1
3.0:1
STA. 40+50
6740
6730
6720
6710
6700
6690
6680
6670
6660
6650
6640
6630
6620
6610
6600
6590
6580
-200
-180
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60
80
100
120
140
160
180
200

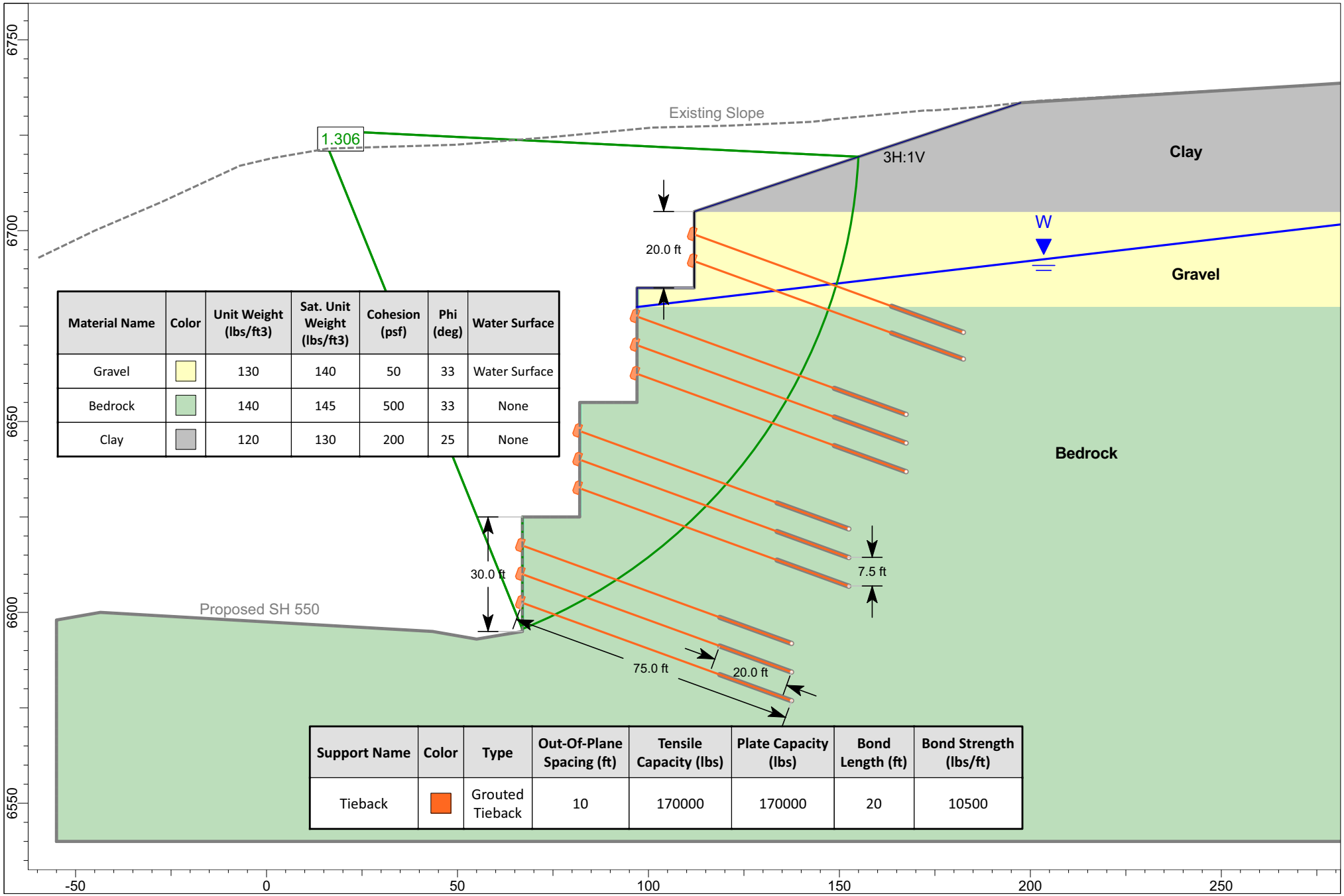


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	50	33	Water Surface
Bedrock		140	145	500	33	None
Clay		120	130	200	25	None

Support Name	Color	Type	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Length (ft)	Bond Strength (lbs/ft)
Tieback		Grouted Tieback	10	150000	150000	20	10500

Scale
1:400

213-174, SH 550
R5 Alignment, STA 40+50
STA 40+50, 85 ft Tieback Wall, 3H:1V Cut in Clay & Gravel

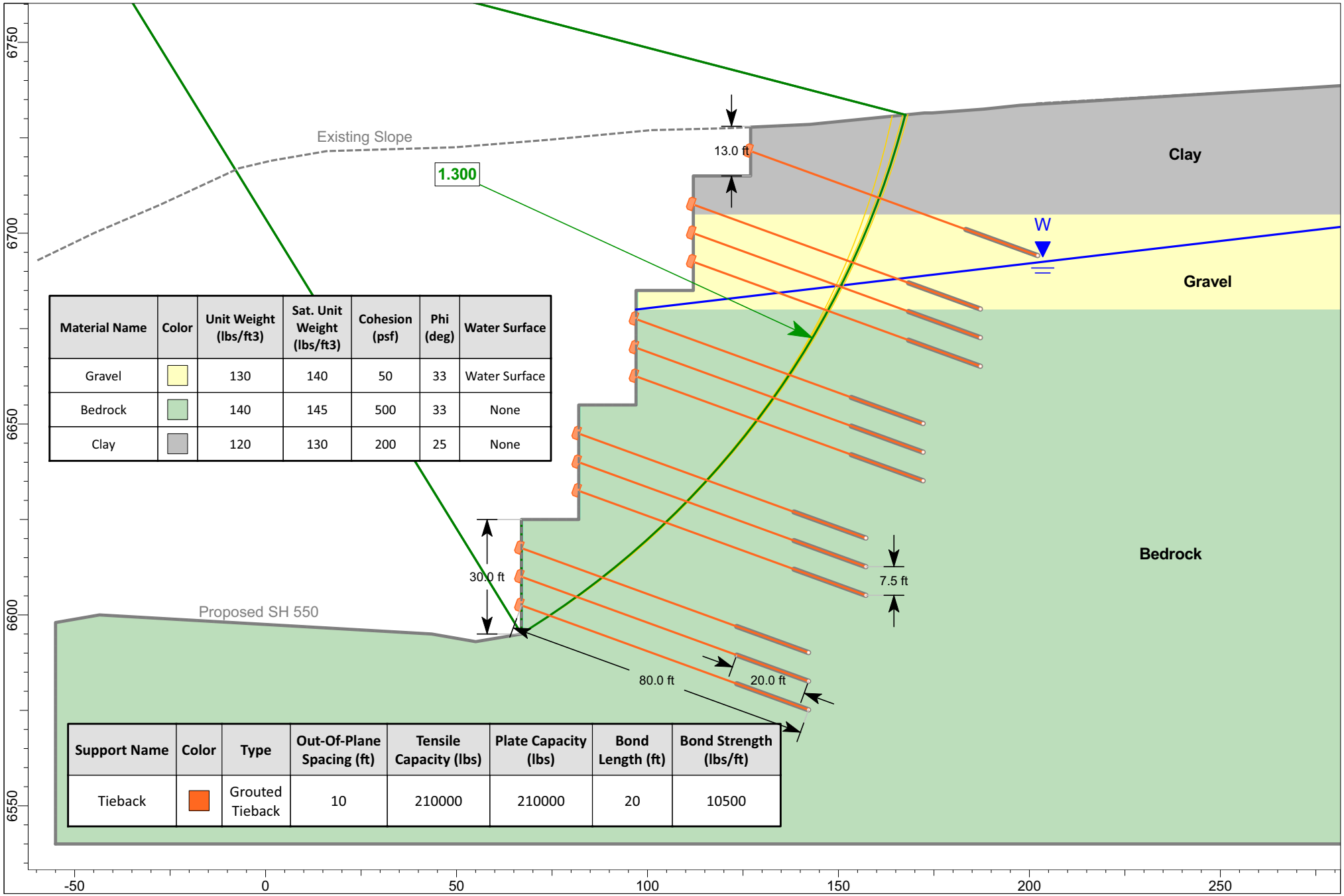


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	50	33	Water Surface
Bedrock		140	145	500	33	None
Clay		120	130	200	25	None

Support Name	Color	Type	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Length (ft)	Bond Strength (lbs/ft)
Tieback		Grouted Tieback	10	170000	170000	20	10500

Scale
1:400

213-174, SH 550
R5 Alignment, STA 40+50
STA 40+50, 110 ft Tieback Wall, 3H:1V Cut in Clay



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)	Water Surface
Gravel		130	140	50	33	Water Surface
Bedrock		140	145	500	33	None
Clay		120	130	200	25	None

Support Name	Color	Type	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Bond Length (ft)	Bond Strength (lbs/ft)
Tieback		Grouted Tieback	10	210000	210000	20	10500

Scale
1:400

213-174, SH 550
R5 Alignment, STA 40+50
STA 40+50, 133 ft Tieback Wall, No Cut Slopes

Appendix E

REFERENCED REPORTS

FINAL REPORT

PRELIMINARY GEOTECHNICAL INVESTIGATION

US HIGHWAY 160, SECTIONS 1 THROUGH 4
LA PLATA COUNTY, COLORADO
PROJECT NO. FC-NH(CX)160-2(048)

Prepared for
Colorado Department of Transportation
Region 5
3803 North Main Avenue, Suite 100
Durango, CO 81301

May 2005

URS

URS Corporation
8181 East Tufts Avenue
Denver, Colorado 80237

Project No. 21711361

FINAL REPORT

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Appendix A Laboratory Test Results

URS Corporation (URS) was retained by the Colorado Department of Transportation (CDOT) to perform engineering for an Environmental Impact Statement (EIS) for the re-alignment of US Highway 160 (US 160) and US Highway 550 (US 550) east of Durango, Colorado. This work is being completed under CDOT Project Number FC-NH(CX) 160-2(048). This report presents the results of a geotechnical investigation conducted to provide an in-depth analysis for the EIS, specifically for proposed structures in Sections 1 through 4.

The scope of the geotechnical investigation consisted of a field investigation and laboratory testing to characterize subsurface conditions, geotechnical engineering analyses for planned construction, development of geotechnical foundation design and construction recommendations, and preparation of structural and roadway reports containing the results of the investigation.

This report contains the results of the field investigation, laboratory testing, foundation recommendations for proposed structures, and geotechnical recommendations for associated structures and improvements within Sections 1 through 4 of the project. Geotechnical construction recommendations are also provided.

2.1 SITE DESCRIPTION

The project area consists of approximately 3.5, 1.8, 0.2, and 0.3 miles of highway on US 160, US 550, CR 233, and CR 234/SH172, respectively (Figure 2-1). US Highway 550 (US 550) extends south from US 160 at about Station 49+00. County Road 233 (CR 233) intersects US 160 at about Station 137+80 and extends to the north and south. The intersection of County Road 234 (CR 234) and CO State Highway 172 (SH 172) at US 160, locally known as Elmore's Corner, is at approximate Station 210+00.

The US 160 project limits for Sections 1 through 4 extend from Station 42+00 at the western project terminus to Station 227+00 at the eastern end just past Elmore's Corner. US 160 begins at the western end as a 4-lane, divided, asphaltic concrete highway and transitions into a 3-lane (2 eastbound, 1 westbound), asphaltic concrete highway just past the US 550 T-intersection. The highway continues with three lanes until expanding to a 4-lane, divided highway at the small bridge just west of Elmore's Corner. The US 160 alignment at Elmore's Corner uses turn and merge lanes for a total of six lanes before returning to two lanes east of the intersection through the project limits.

US 550 is a two-lane, undivided, asphaltic concrete highway extending south from the T-intersection with US 160. US 550 ascends a steep hill, locally known as Farmington Hill, with a steep dropoff from the southbound lane into Wilson Gulch.

CR 233 is a two-lane, undivided, asphaltic concrete highway extending north and south from US 160 at Station 137+80. North bound CR 233 travels north into residential areas. The southern portion of CR 233 turns east-west and travels parallel to US 160 to service businesses along the highway. CR 233 again crosses US 160 at Station 181+00 and is known as CR 233 East.

CR 234 extends north from Elmore's Corner and SH 172 extends to the south. Both are 2-lane, undivided highways.

2.2 PROJECT DESCRIPTION

CDOT plans to reconfigure the US 550 and US 160 interchange including realignment of the north end of US 550 and conversion of US 160 into a controlled access highway in the vicinity of the new interchange. Plans include eight new bridges, including on and off ramps at US 550 and controlled access interchanges at CR 233, CR 233 East, and Elmore's Corner. This report addresses the proposed bridges and retaining walls. Widening of roads in the project limits is addressed in the roadway report.

Eight bridges are included in the proposed plans. Table 2.1 gives a general description of the bridges and each bridge's associated retaining walls. Additional information about the proposed bridges follows Table 2.1 (station numbers refer to the proposed US 160 alignment). Figures 2-1 and 2-2 show proposed bridge and wall locations.

Table 2.1
Proposed Bridges

Bridge Designation	Structure No.	Engineering Geology Sheet	Spans	Approx. Length (ft)	Approx. Width (ft)	Associated Retaining Wall Structures
US 550 Over US 160	P-05-U P-05-AG	2-3	3	520	60	Wall-P-05-R Wall-P-05-T
Ramp A	P-06-V	2-4	4	535	28	Wall-P-05-Q Wall-P-05-F
Ramp C	P-05-W	2-5	2	315	34	Wall-P-05-U Wall-P-05-Z
Ramp D	P-05-Y	2-6	2	290	28	Wall-P-05-V Wall-P-05-W
Ramp E	P-05-X	2-7	2	255	28	Wall-P-05-Q Wall-P-05-L
US 160 Over CR 233	P-05-AD	2-8	1	230	95	Wall-P-05-AA Wall-P-05-AB
US 160 Over CR 233 (East)	P-05-AE	2-9	1	95	61	Wall-P-05-AI Wall-P-05-AC
US 160 Over SH 172	P-05-AF	2-10	1	230	95	Wall-P-05-AG Wall-P-05-AH

The US 550 alignment will be moved approximately ½ mile east to intersect US 160 at about Station 80+80. The new alignment will begin at the current intersection with CR 220 and continue northward down a large cut to US 160. The new intersection at US 160 will be designed as a controlled access interchange with access to and from US 160 on five ramps, Ramps A through E. Four of the ramps require bridges over US 160 and/or Wilson Gulch, located just north of US 160.

US 550 will cross US 160 on a proposed 3-span bridge approximately 520 feet in length. The proposed bridge will also cross Wilson Gulch and is shown on Figure 2-3. The southern bridge approach will be through a cut in the slope located south of US 160. The south abutment will require about 20 feet of fill at the abutment. This fill will be retained behind a planned 22-foot high retaining wall. Beneath the bridge, cuts of up to about 12 feet are planned between the south abutment and Pier 2 to create a drainage ditch. Fill up to about 10 feet thick is planned underlying the realigned US 160. This fill will require a second retaining wall, about 15 feet in height, north of US 160. On the north side of US 160, existing grades will be maintained across Wilson Gulch. A 49-foot high embankment is planned at the north abutment. From the top of the embankment, grades will be transitioned to the existing grade beneath the bridge by a 20-foot high retaining wall and a 2H:1V (horizontal:vertical) slope below the wall to the south.

Ramp A will serve as the exit ramp from eastbound US 160 onto northbound and southbound US 550, crossing US 160 at about Station 75+40 (Figure 2-4). About 6 feet of fill will be required at the south abutment. Beneath the bridge, planned cuts up to about 14 feet will require a retaining wall about 14 feet high. The westbound lanes of US 160 will be built on up to 23 feet of fill

retained by an approximately 30 foot retaining wall just north of US 160. Realignment of Wilson Gulch will require cuts of 10 to 15 feet. These cuts will be made with slopes of 3H:1V or flatter. A 26-foot high embankment is planned at the north abutment. This embankment will be separated from the regraded Wilson Gulch by an approximately 25-foot high retaining wall.

Ramp B will be the entrance ramp to US 160 from northbound US 550. A retaining wall is proposed on the south side of the ramp. Proposed cuts and the retaining wall to construct the ramp are addressed in the roadway report.

Ramp C is a loop ramp to westbound US 160 from northbound US 550 and will bridge Wilson Gulch (Figure 2-5). The proposed northern abutment design requires 33 feet of embankment fill and a 14 -foot high retaining wall. Below the retaining wall south of the abutment, the 2H:1V fill slopes away from the retaining wall, until reaching existing grade. The proposed southern abutment will also have a 14-foot retaining wall to build a 10-foot deep embankment at the abutment. Wall-P-05-Z extends from the abutment along westbound US 160 about 310 feet and is discussed further in the roadway report.

Ramp D will be the exit ramp from westbound US 160 to US 550 and will bridge Wilson Gulch (Figure 2-6). The proposed west abutment (Abutment 1) will be constructed on a 19.5-foot thick embankment with a 13.5-foot high retaining wall. A 2H:1V fill below the wall will slope down to meet existing grade near a proposed multi-use path beneath the bridge. A 22.5-foot retaining wall will raise grades from existing to a proposed 27-foot embankment at the east abutment.

Ramp E will serve as the entrance ramp to westbound US 160 from northbound US 550 with a bridge over Wilson Gulch (Figure 2-7). An approximate 25-foot thick embankment is planned at the south bridge abutment requiring an approximately 20-foot high retaining wall. This wall will be about 10 feet of fill south of the wall for the southerly approach and 10 feet of cut north of the wall. The north abutment will be built on an approximate 16-foot embankment requiring an 14-foot high retaining wall.

Outside of the US 550 interchange, US 160 will cross CR 233 in two locations – the first at about Station 138+00 (CR 233) and the other at about Station 181+00 (CR 233 East). There will be no access to and from CR 233 East from US 160. The CR 233 bridge will be single-span (Figure 2-8). The bridge approaches will be built up with about 36 feet of fill at each abutment. Retaining walls, 17 to 19 feet tall, are planned for each abutment At CR 233 East, the county road will be cut below the existing grade of US 160 (Figure 2-9). The proposed cuts are approximately 30 feet from the existing grade. Soldier pile walls will be used to retain the proposed cuts.

The US 160 intersection with SH 172 and CR 234 will be a single-point interchange. US 160 will cross over the SH 172/CR 234 on a single-span bridge (Figure 2-10). The overcrossing will be constructed using a 20-foot embankment at the west abutment and a 30-foot embankment at the east abutment. These embankments will require approximately 15-foot high retaining walls at each abutment.

3.1 FIELD INVESTIGATION

URS conducted a field investigation between September 2004 and March 2005 to evaluate subsurface conditions at the proposed bridge locations. Thirty-four structure borings were drilled to develop geotechnical recommendations for the proposed bridges and retaining walls.

Spectrum Exploration of Colorado Springs, Colorado, drilled the borings with either truck-mounted rotary or tracked all-terrain vehicular rotary rigs under the direction of a URS field engineer. Tubex equipment was used to advance borings through dense gravel and cobble layers.

Samples were generally collected at 5-foot intervals. Samples were collected using a 2-inch inner diameter California sampler or a 2-inch outer diameter split-spoon sampler. California samples were retained in brass liners, sealed at both ends with plastic end caps. The split-spoon samples were retained in sealed plastic bags. Standard Penetration Tests (SPTs) were performed during the sampling operations. SPT testing consists of counting the number of blows from a 140-pound hammer falling 30 inches required to advance a split-spoon sampler 12 inches. A URS field engineer recorded the blow counts and logged the samples.

The borings were backfilled with auger cuttings and/or native soil. Borings drilled in existing roadways were patched with asphalt concrete.

3.2 SUBSURFACE CONDITIONS

RockSol Consulting Group of Boulder, Colorado performed the laboratory testing. The results of laboratory tests performed on selected samples from the borings are described below and summarized in Table 3-1. Laboratory data sheets are provided in Appendix A. Subsurface conditions and groundwater are discussed in further detail in Section 4.1.

The fine-grained soils tested had fines contents (percent passing the No. 200 sieve) ranging from 54 to 87 percent. The liquid limits of the samples tested ranged from 31 to 50 percent. The plasticity indices of the samples tested ranged from 7 to 26 percent. The moisture contents of samples tested ranged from 10.9 to 33.3 percent, with an average moisture content of 24.5 percent. The measured dry densities of samples tested ranged from 89.5 to 96.0 pcf, with an average dry density of 93.8 pcf.

The coarse-grained soils tested had fines contents (percent passing the No. 200 sieve) ranging from 16 to 47 percent. The liquid limits of the samples tested ranged from 30 to 42 percent. The plasticity indices of the samples tested ranged from 8 to 22 percent. The moisture contents of samples tested ranged from 9.1 to 26.6 percent, with an average moisture content of 19.0 percent.

Based on index test results, the soils were classified in accordance with the Unified Soil Classification System (USCS) as lean clay and silt, and silty to clayey sand.

A swell test was performed on a sample of a higher plasticity clay to evaluate swell potential. The sample was saturated after loading to 1,000 psf. The soil sample tested, taken at 4 feet below ground surface (bgs), had dry unit weight of 105.3 pcf, a moisture content of 18.8%, and a negligible swell potential (0 percent).

A consolidation test was performed on a sample of potentially compressible clay soil. The test found the soil, taken at 19 feet bgs, had a dry unit weight of 94.4 pcf, a moisture content of 29.1%, and was moderately compressible.

In general, the clay samples tested exhibit low to moderate strength. Blow counts and field observations indicate the natural clay will have similar characteristics at their natural moisture contents. Although testing was not performed on bedrock samples, blow counts and field observations indicate the bedrock has high strength and low compressibility.

Table 3.1
Summary of Laboratory Test Results

Boring No.	Sample Depth (ft)	USCS Symbol(1)	Atterberg Limits		Fines Content (- No. 200 Sieve)	Moisture / Density	
			Liquid Limit (%)	Plasticity Index (%)		Moisture Content (%)	Dry Density (pcf)
2B15	10.0	SC	30	8	22		
2B17	14.0	CL	37	13			
2B17	4.0	CL				18.8	105.3
2B17	19.0	ML	31	7	54	25.8	
2B17	24.0	SM	35	10	35	26.6	
2B31	4.0	SC	42	22	37	19.2	
3B8	15.0	ML	36	10	73	33.3	89.5
3B8	19.0	CL	41	21	60	29.1	94.4
3B8	30.0	ML	46	17	72	28.9	96.0
3B9	15.0	CL	47	24	67	27.2	95.1
4B4	14.5	ML(2)	38	11	65		
4B5	4.0	CL	43	20	61	23.2	
4B5	9.0	CL	40	20	68	20.8	
4B5	14.0	CL	45	21	79	23.5	
4B5	19.0	SM	37	11	47	21.2	
4B6	4.5	ML(2)	33	7	63	13.4	121.4
4B7	4.0	CL	44	19	87	25.9	
4B7	14.0	CL	38	14	70	22.4	
4B7	19.0	MH	50	20	83	24.2	
4B18	2.0	SC	30	14	16	9.1	
4B26	2.0	CL	49	26	80	23.0	
4B26	19.0	ML	49	18	68	10.9	

(1) Estimated by visual observation and available data.

(2) Siltstone bedrock.

4.1 FOUNDATION RECOMMENDATIONS

Bridge recommendations are summarized in Table 4.1. A description of subsurface conditions and specific foundation recommendations for each proposed structure follows the Table 4.1. General recommendations for foundations and retaining walls are presented in Section 4.2 and 4.3, respectively.

Table 4.1

Bridge Foundation Recommendations

Bridge	Structure No.	Foundation Bearing Layer	Recommended Foundation Types	Retaining Wall Location (Abutment No.)	Approximate Wall Height (ft)	Planned Wall Type
US 550 Over US 160	P-05-U	Bedrock	Drilled Piers or Driven Piles	North (4)	21	MSE
				South (1)	24	
Ramp A	P-06-V	--See Ramp A Description Below--	--See Ramp A Description Below--	North (5)	30	MSE
				South (1)	15.5	
Ramp C	P-05-W	Bedrock	Driven Piles	North (1)	14	MSE
				South (3)	14	
Ramp D	P-05-Y	Bedrock	Drilled Piers or Driven Piles	Northwest (1)	19.5	MSE
				Southeast (3)	22.5	
Ramp E	P-05-X	Bedrock	Drilled Piers	North (3)	14	MSE
				South (1)	21	
US 160 Over CR 233	P-05-AD	Bedrock	Drilled Piers	West (1)	17	MSE
				East (2)	19	
US 160 Over CR 233 (East)	P-05-AE	Bedrock	Drilled Piers	West (1)	24	Soldier Pile
				East (2)	25.5	
US 160 Over SH 172	P-05-AF	Gravel, Cobbles and Boulders	Driven Piles	West (1)	15	MSE
				East (2)	13	

US 550 Over US 160 (Figure 2-3)

Subsurface conditions encountered in the borings at the proposed bridge location generally consisted of alluvium of variable depth overlying very hard claystone, siltstone, and sandstone bedrock. The alluvium typically consisted of medium stiff to very stiff sandy clay and loose to medium dense clayey sand. A four-foot thick layer of gravel was encountered in boring 2B34, just east of proposed Pier 3. Cobbles and boulders were observed near the ground surface in the vicinity of the proposed south abutment and Pier 2. An approximate six-foot thick layer of cobbles and boulders was also observed in the area north of proposed Pier 3 (boring 2B35). The depth to bedrock in the borings near the proposed bridge ranged from 3 feet bgs at the southern abutment to 32 feet bgs at the northern abutment.

We recommend deep foundations bearing in bedrock to found the proposed bridge abutments and piers. Drilled piers or driven piles may be used. Dense to very dense gravel layers within the alluvium may make pier drilling and pile driving difficult. Piers or piles should penetrate into bedrock as the gravel layers are too thin to provide adequate foundation support. Pile driving shoes or tips should be used to penetrate the gravels. Casing and large pier-drilling rig may be required to complete drilled piers.

The three planned retaining walls can be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and slopes should be constructed in accordance with the recommendations provided in Section 5.

Ramp A (Figure 2-4)

Subsurface conditions encountered in the borings in the vicinity of Ramp A generally consisted of alluvial deposits overlying very hard claystone, siltstone and sandstone bedrock. Subsurface soils ranged from medium stiff to very stiff, sandy clay and silt, to medium dense, clayey, silty sand. A 15-foot thick, medium dense to very dense gravel layer was observed from 15 to 30 feet bgs in boring 2B20, just north of proposed Pier 4. Cobbles and boulders were encountered in all of the borings in the area of the proposed bridge. Some of the cobble and boulder layers observed were up to 35 feet thick. Near-surface cobbles and boulders were encountered in borings near the proposed south abutment and Pier 2. The depth to bedrock ranged from 29 to 49 feet in the borings, and was deepest at the abutments.

Due to the cobbles and boulders, construction of deep bridge foundations such as piers or piles will be difficult. Drilled piers or driven piles will need to be pre-drilled through the cobble and boulder layers. Drilling may require percussive drilling to penetrate the cobbles and boulders. Footings bearing on the dense cobbles and boulder could be used to found the south abutment and Pier 2 but will not provide scour protection. Footings should not be used for Piers 3 and 4 and the north abutment (Abutment 5), as they will settle. Footings will settle more than drilled piers. Therefore, deep foundations are the preferred alternative. We recommend additional investigation(s) and construction of test piers or piles.

The proposed retaining walls can be constructed as cantilever walls in accordance with Section 4.3 or as MSE walls. Walls able to tolerate settlement may be founded on footings. Settlements are estimated to range from 4 to 5 inches. If walls cannot tolerate settlement, deep foundations should be used.

Ramp C (Figure 2-5)

Subsurface conditions in the borings at the proposed bridge location generally consisted of alluvial deposits overlying very hard claystone and sandstone bedrock. The alluvium typically consisted of medium stiff to stiff, sandy clay and loose to medium dense, clayey sand with a gravel layer identified near proposed Pier 2 (boring 2B42). The depth to bedrock ranged from 27 to 47 feet bgs, and generally corresponded to approximate elevation 6594 feet.

Due to shallow groundwater and the sand, gravel, cobble and boulder layers, drilled piers will be difficult to construct. We recommend H-piles driven into bedrock be used for the abutment and pier foundations.

The proposed retaining walls may be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and should be constructed in accordance with the recommendations provided in Section 5.

Ramp D (Figure 2-6)

Subsurface conditions in the borings at the proposed Ramp D bridge generally consisted of alluvial deposits overlying very hard sandstone bedrock. The alluvium was typically medium stiff to very stiff, sandy clay to loose to medium dense, clayey sand. A four-foot thick, medium dense gravel layer was encountered in boring 2B53, near the proposed north abutment, at about 37 feet bgs. Depth to bedrock ranged from 41 to 42 feet bgs, and tends to be 20 feet lower in elevation near the south abutment as compared to the north abutment.

We recommend drilled straight-shaft piers or driven piles founded in bedrock for the abutments and pier.

The proposed retaining walls may be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and should be constructed in accordance with the recommendations provided in Section 5.

Ramp E (Figure 2-7)

Subsurface conditions in the borings at the proposed bridge location generally consist of alluvium overlying hard to very hard claystone and sandstone bedrock. The alluvium typically consists of medium stiff to very stiff, sandy clay and loose to medium dense, clayey sand. Three-foot thick layers of cobbles and boulders were observed in two of the borings (2B8 and 2B12). The depth to bedrock ranged from 3 to 26 feet bgs, and tends to be shallower at the south abutment.

We recommend straight-shaft piers drilled into bedrock be used for the abutments. Driven piles founded in bedrock are not recommended due to the shallow bedrock at Abutment 1 that will limit embedment.

The proposed retaining walls may be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and should be constructed in accordance with the recommendations provided in Section 5.

US 160 Over CR 233 (Figure 2-8)

Subsurface conditions at the proposed bridge location generally consisted of alluvium of variable depth overlying siltstone and sandstone bedrock. The alluvium typically consisted of slightly sandy to sandy clay with very silty layers. The depth to bedrock was approximately 25 feet near the south side of the west abutment, 39 feet near the north side of the west abutment, and 3 feet near the south side of the east abutment. Groundwater was not observed in our borings during drilling in the vicinity of this structure. However, groundwater can fluctuate with seasons and changes in precipitation and may be encountered during construction.

Based on the subsurface conditions and the proposed construction, we recommend deep foundations bearing in bedrock be used to found the proposed bridge abutments and piers. Deep foundation alternatives include steel H-piles and drilled piers. Due to the shallow bedrock present near the east abutment, we recommend drilled piers be used to avoid differential settlement. Driven piles may achieve sufficient length due to the shallow bedrock present on the southern and eastern sides of the proposed structure to develop adequate load capacity. Shallow foundations could be considered for the east abutment. Borings to the east of the proposed structure suggest the depth to bedrock increases on the northern side of the roadway. Shallow foundations may not reach bedrock at reasonable depths on the north side of the abutment. We consider straight-shaft drilled piers an option with less risk for foundation movement and for design changes during construction.

The proposed retaining walls may be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and should be constructed in accordance with the recommendations provided in Section 5.

US 160 Over CR 233 EAST (Figure 2-9)

Subsurface conditions encountered in the borings at the proposed bridge generally consist of sandy clay and silt of variable depth overlying silty sandstone bedrock. Sandstone was encountered near the existing ground surface on the northern side of the proposed structure and at a depth of 23 feet below existing ground surface at the southern side. Groundwater was not observed in the borings during drilling. However, groundwater levels may fluctuate seasonally and water may be encountered during construction. Groundwater may be present at the top of relatively impermeable bedrock layers.

Based on the subsurface conditions and the proposed construction, we recommend straight-shaft drilled piers drilled into the sandstone bedrock be used to found the abutments and to construct permanent soldier pile retaining walls on both sides of CR 233 East. Use of footings would result in large differential settlement between the north and south sides of the proposed structure. The concrete deck can be cast-in-place over the piers and the CR 233 alignment below the deck can then be excavated and constructed from the sides possibly reducing construction closures on US 160.

US 160 Over SH 172 (Figure 2-10)

Subsurface conditions at the proposed bridge generally consist of cobbles and boulders, with infilling soil consisting of sandy clay to clayey, sandy silt, to clayey sand overlying very hard claystone and siltstone bedrock. Depth to bedrock ranged from 30 to greater than 41 feet bgs.

Foundations for the abutments will likely be founded in the cobbles and boulders. We recommend driven piles bearing in the cobbles and boulders be used, designed with a working stress of 9 ksi.

The proposed retaining walls may be designed and constructed as MSE walls. Permanent fill slopes should be constructed at 2H:1V or flatter and should be constructed in accordance with the recommendations provided in Section 5.

4.2 BRIDGE FOUNDATIONS

Drilled Piers

1. Specific conditions at each bridge outlined above should be reviewed prior to foundation design.
2. Straight-shaft drilled piers should be at least 10 feet long and be drilled into bedrock. Piers should be designed for an allowable end-bearing pressure of 30 ksf and an allowable skin friction of 3 ksf for the portion of the pier in unweathered bedrock. Piers should penetrate at least two pier diameters or 5 feet into bedrock. Downdrag has been considered in these allowable capacities.
3. Uplift due to structural loadings on the piers can be resisted by using 75 percent of the skin friction plus the pier weight, provided the sides of the pier hole in bedrock are grooved or otherwise artificially roughened as described below.
4. Drilled piers, properly designed and constructed according to the above criteria, should experience individual pier settlement less than 0.5 to 1 inch.
5. The LPILE parameters given below can be used for lateral load analysis.

Table 4.1

LPILE Parameters for Lateral Load Analysis

Material	Field Density/Consistency	Subgrade Modulus (pci)	ϕ (degrees)	c (psf)	ε_{50}	Total Unit Weight (pcf)
New Embankment Fill	Medium Dense to Very Dense	120	30	0	---	125
Sandy Clay and Silt	Medium Stiff to Very Stiff	80	0	500	0.02	110
Gravel	Medium Dense to Very Dense	80	35	0	--	125
Sandstone Bedrock	Medium Dense to Very Dense	2000	40	0	--	125
Claystone Bedrock	Hard to Very Hard	2000	0	5000	0.004	125

6. The piers should have a minimum spacing of three times the pier diameter. Piers spaced closer than 3 diameters will have reduced capacity.

7. Bedrock penetration in pier holes greater than 30 inches in diameter should be provided with shear rings 3 to 5 inches high and 2 to 3 inches deep, and spaced at 2-foot centers to assist in the development of peripheral shear stress between the pier and the bedrock. Bedrock penetration in pier holes less than 30 inches in diameter can be provided with roughening the hole sides with a sidetooth on the auger.
8. Pier holes should be properly cleaned prior to the placement of concrete.
9. Concrete used in the piers should be fluid mix with sufficient slump to fill the void between the reinforcing steel and the pier hole. We recommend a slump in the range of 5 to 8 inches or use of a plasticizing admixture in the concrete mix.
10. Concrete should be placed in drilled holes immediately after the holes are cleaned and reinforcement placed. Pier holes should not be left open more than 12 hours. If pier holes are left open more than 12 hours the pier should be deepened and reamed.
11. Groundwater may be encountered in drilled pier holes, and sands and gravels may slough. Casing may be required to complete drilled piers.
12. Concrete should not be placed in more than 3 inches of water. If water cannot be removed with the use of temporary casing and dewatering equipment prior to placement of concrete, the tremie method should be used after the hole has been cleaned.
13. The drilled pier subcontractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration. Localized, well-cemented zones of bedrock could be encountered, requiring the use of a core barrel or pilot hole.
14. Boulders and cobbles requiring excavation by equipment other than augers may also be encountered. Contractor should submit a plan and unit costs for excavation of cobbles and boulders requiring special equipment.
15. The geotechnical engineer should observe pier drilling operations on a full-time basis to assist in identification of bedrock strata and to monitor pier construction procedures.

Driven Piles

1. Specific conditions at each bridge outlined above should be reviewed prior to foundation design.
2. Steel H-piles driven to refusal, as determined in accordance with CDOT Standard Specifications for Road and Bridge Construction, Section 502, driven in bedrock may be designed using a working stress of 12.5 ksi. Piles driven to refusal in gravel may be designed using a working stress of 9 ksi. Steel pipe piles, 10.75-inch diameter filled with concrete, may be designed using a working stress of 40 percent of the compressive strength of the concrete used to fill the pile and 25 percent of the strength of steel. Steel for pipe piles should be a minimum of 50 ksi and concrete should have a minimum compressive strength of 3.5 ksi.
3. Piles should be provided with driving shoes to protect the pile tips from damage when penetrating dense granular soils or cobbles. Individual pile settlement should be 0.5 inch or less.

4. Groups of piles, spaced closer than three diameters center to center, will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pile group. These reductions should be evaluated on an individual basis based on actual pile loadings and spacings.
5. Lateral resistance can be provided by the bending resistance of piles or by battered piles. A battered pile can resist the same axial load as a vertical pile of the same type and size driven to the same elevation. The vertical and horizontal components of the load will depend on the batter inclination. Batters should not exceed 1H:4V. The LPILE parameters provided under Drilled Piers may be used for lateral capacity evaluations.
6. The contractor should select a driving hammer and cushion combination capable of driving the selected piling without overstressing the pile material. The selection and approval of the pile driving equipment should be in accordance with Section 502 of the CDOT Standard Specifications for Road and Bridge Construction manual. The contractor should submit the pile driving plan and the pile hammer-cushion combination to the engineer for evaluation of driving stresses well in advance of pile installation. Section 502.05 of the CDOT manual indicated that wave equation analyses program (WEAP) will be used to determine approval when the equipment is of questionable adequacy. This can be avoided if the contractor has already performed an appropriate wave equation analysis.
7. The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. Pile driving operations should be observed under the direction of a qualified geotechnical engineer. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. No specific minimum loading is required and downdrag need not be considered.
8. We recommend driving and testing of test piles to further evaluate driveability and capacity prior to final design.

The following table gives estimated pile tip elevations for the recommended structures from above:

Table 4.2
Estimated Pile Tip Elevations

Bridge	Structure	Bearing Material	Anticipated Top-of-Pile Elevation (ft)	Estimated Tip Elevation[1] (ft)
US 550 Over US 160	Abutment 1	Bedrock	6692	6649
	Pier 2	Bedrock	6680.5	6615
	Pier 3	Bedrock	6674	6586
	Abutment 4	Bedrock	6676	6585
Ramp A	Abutment 1	Bedrock	6640	6573
	Pier 2	Bedrock	6635	6577
	Pier 3	Bedrock	6636	6541
	Pier 4	Bedrock	6636.5	6552
	Abutment 5	Bedrock	6642	6555
Ramp C	Abutment 1	Bedrock	6675	6651
	Pier 2	Bedrock	6665	6649
	Abutment 3	Bedrock	6665	6650
Ramp D	Abutment 1	Bedrock	6679	6607
	Pier 2	Bedrock	6677	6610
	Abutment 3	Bedrock	6688	6622
US 160 Over SH 172	Abutment 1	Gravel, Cobbles and Boulders	6995	6969
	Abutment 2	Gravel, Cobbles and Boulders	6995	6961

[1] Based on 5-foot penetration into bearing material.

Footings

1. Specific conditions at each bridge outlined above should be reviewed prior to foundation design.
2. Natural granular soil supporting shallow foundations should be densified with a vibratory compactor prior to foundation construction.
3. Loose and soft soils, and unsuitable material not indicated by the subsurface explorations, encountered within the bearing zone of the foundations should be removed and replaced with granular structural fill. The soil removal should extend laterally a minimum 3 feet beyond the foundation edges.
4. Structural fill placed beneath foundations, and against the sides of foundations to resist lateral loads, should consist of granular material such as CDOT Class 1 backfill. In general, soil excavated during site grading and foundation construction should be

evaluated by the geotechnical engineer, including performing appropriate laboratory tests, if the soil is planned for use as structural fill.

5. Structural fill should be placed in uniform lifts with a maximum loose lift thickness of 8 inches. Fill placed beneath foundations should be compacted to at least 95 percent of the Proctor maximum dry density (AASHTO T-99) at a moisture content within 2 percent of optimum. Fill placed against the sides of foundations should be compacted to at least 90 percent of the Proctor maximum dry density (AASHTO T-99).
6. The lateral resistance of a footing placed on natural granular soil or structural fill will be a combination of the sliding resistance of the foundation on the bearing material and the passive earth pressure against the side of the foundation. Resistance to sliding at the bottom of the foundation can be calculated based on a coefficient of friction of 0.35. Passive pressure against the side of the foundation and foundation wall can be calculated using an ultimate equivalent fluid unit weight of 300 pcf.
7. Shallow foundations should be founded at least 2.5 feet below final grades for frost protection.
8. Estimated settlements are discussed in Section 5.5.
9. A representative of the geotechnical engineer should observe and test foundation excavations, structural fill placement and compaction during construction.

4.3 RETAINING WALLS

Mechanically Stabilized Earth Walls

The following recommendations should be incorporated into design and construction of MSE walls up to 30 feet high. Walls taller than 30 feet should be evaluated individually for design and construction.

1. Fill material within the reinforced zone should consist of granular soil with less than 25% fines (material passing the No. 200 sieve), a liquid limit less than 30, a plasticity index less than 10. A friction angle of 32 degrees may be used for MSE wall design.
2. Fill within the reinforced zone should be compacted in accordance with the recommendations in Section 5 (see Figure 5-1).
3. Water should not be allowed to run down the face of the MSE walls. Surface water should be collected and directed into properly designed structures.
4. MSE wall design should be analyzed for global stability, including potential deep failures as well as internal stability. Tall walls may need to be tiered or have grid lengths increased to satisfy global stability.
5. Wall settlement will vary depending on wall height and subsurface conditions. Settlement is discussed in Section 5.5.

Cantilever Walls

Geotechnical design parameters will vary depending on the restrained/unrestrained condition and the wall height.

1. Concrete gravity walls should be founded on spread footings with a minimum width of 18 inches. Footings should bear at least 30 inches below the lowest adjacent grade for frost protection. Footings may be designed using an allowable bearing pressure of 3,000 psf. Toe pressures should not exceed 6,000 psf.
2. Unrestrained walls are walls allowed to rotate more than 0.001 times the height of the retaining portion of the wall in feet. For walls up to 12 feet tall, unrestrained walls having a level backfill surface extending horizontally behind the wall a distance equal to or greater than the height of the wall should be designed for an active soil pressure equivalent to a fluid weight of 35 pcf. Where backfill will be inclined, no steeper than 2:1, an active soil pressure of 45 pcf is recommended. These pressures are allowable loads and assume backfill consists of compacted granular backfill. If clay backfill is used, an equivalent fluid weight of 50 pcf should be used for level backfill and 70 pcf for inclined backfill.
3. For walls up to 12 feet tall, walls restrained from movement at the top require an additional uniform pressure of 7 psf times the height of the retaining portion of the wall in feet. Where a traffic surcharge occurs within a horizontal distance less than two-thirds the height of the retaining portion of the wall, an additional uniform horizontal pressure of 70 psf should be applied to the retaining wall. The resultant force should be applied at the midheight of the retaining portion of the wall.
4. For walls taller than 12 feet, resultant uniform lateral pressure in psf of 25 times the height of the retaining portion of the wall in feet should be used for design. The resultant force should be applied at the midheight of the retaining portion of the wall.
5. For resistance to lateral loads, an allowable passive earth pressure equivalent to 300 pcf is recommended for footing poured neat against properly compacted granular fill soils or undisturbed natural soils. This allowable lateral pressure assumes either the fill extends a horizontal distance of at least 5 feet behind the wall or three times the height of the surface generating the resultant passive pressure.
6. An allowable coefficient of friction between soil and concrete is 0.35. This friction coefficient may be combined with the allowable passive pressure when calculating lateral resistance.
7. Wall settlement is discussed in Section 5.5.
8. Retaining walls should be provided with a drainage system that prevents buildup of hydrostatic forces. The use of drainage openings through the base of the wall (weep holes, etc.) is not recommended where seepage could impact adjacent structures and/or property.

Soldier Pile Wall

A soldier pile wall consists of closely-spaced drilled piers. The following criteria should be used for design:.

1. Straight-shaft drilled piers should be at least 10 feet long and be drilled into bedrock. Piers should be designed for an allowable end-bearing pressure of 20 ksf and an allowable skin friction of 2 ksf for the portion of the pier in unweathered bedrock. Piers should penetrate at least two pier diameters or 5 feet into bedrock.
2. The piers should be spaced no greater than 12 inches apart (edge to edge). Piers should be a minimum of 24 inches in diameter.
3. Uplift due to structural loadings on the piers can be resisted by using 75 percent of the skin friction plus the pier weight, provided the sides of the pier hole in bedrock are grooved or otherwise artificially roughened as described below.
4. Drilled piers, properly designed and constructed according to the above criteria, should experience individual pier settlement less than 0.5 to 1 inch.
5. The LPILE parameters given in Drilled Piers can be used for lateral load analysis, if applicable.
6. Bedrock penetration in pier holes greater than 30 inches in diameter should be provided with shear rings 3 to 5 inches high and 2 to 3 inches deep, and spaced at 2-foot centers to assist in the development of peripheral shear stress between the pier and the bedrock. Bedrock penetration in pier holes less than 30 inches in diameter can be provided with roughening the hole sides with a sidetooth on the auger.
7. Pier holes should be properly cleaned prior to the placement of concrete.
8. Concrete used in the piers should be fluid mix with sufficient slump to fill the void between the reinforcing steel and the pier hole. We recommend a slump in the range of 5 to 8 inches or use of a plasticizing admixture in the concrete mix.
9. Concrete should be placed in drilled holes immediately after the holes are cleaned and reinforcement placed. Pier holes should not be left open more than 12 hours. If pier holes are left open more than 12 hours the pier should be deepened and reamed.
10. Groundwater may be encountered in drilled pier holes, and sands and gravels may slough. Casing may be required to complete drilled piers.
11. Concrete should not be placed in more than 3 inches of water unless the tremie method is used. If water cannot be removed with the use of temporary casing and dewatering equipment prior to placement of concrete the tremie method should be used after the hole has been cleaned.
12. The drilled pier subcontractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration. Localized, well-cemented zones of bedrock could be encountered, requiring the use of a core barrel or pilot hole.
13. Boulders and cobbles requiring excavation by equipment other than augers may also be encountered. Contractor should submit a plan and unit costs for excavation of cobbles and boulders requiring special equipment.
14. The geotechnical engineer should observe pier drilling operations on a full-time basis to assist in identification of bedrock strata and to monitor pier construction procedures.

5.1 GENERAL

Vegetation, debris, rubble, and deleterious material should be removed from the areas to be graded. Removal of all debris within the work area should be completed before fill placement begins.

5.2 EXCAVATIONS

OSHA regulations should be followed for temporary excavation slopes. Soils observed in the borings classify as OSHA Type C soils. Construction excavations in the fill and natural soil should be inclined no steeper than 2H:1V or as recommended by OSHA standards. Weathered bedrock classifies as Type A soil and may be sloped at 1H:1V or steeper if deemed appropriate by the contractor's designated safety personnel. Undercutting of excavation faces should not be permitted. The contractor should designate a responsible person to evaluate excavation safety, including soil types encountered.

Groundwater may be encountered in shallow excavations in some areas of the site. The presence of groundwater may decrease excavation stability. The contractor should designate a representative to monitor excavations and determine appropriate soil classifications and excavation slopes.

Groundwater may interfere with the earthwork procedures. Dewatering may be accomplished by pumping from sumps. Seepage in deep cut areas may be controlled by pumping from sumps or gravity drainage.

Excavations into the bedrock may require a dozer with ripper teeth, hoerams and extra effort. Boulders and cobbles may also be encountered. Deeper excavations may require blasting.

5.3 SLOPES

Temporary cut and fill slopes should not be steeper than 2H:1V. Permanent cut slopes steeper than 3H:1V should be evaluated by a geotechnical engineer. Fill slopes may be inclined at 2H:1V or flatter if the fill is compacted as described below and the slope is vegetated or armored. Cut slopes in bedrock can be sloped at 1H:1V. Slopes steeper than 1H:1V in bedrock may be possible if evaluated by a geotechnical engineer. Surface water should be directed away from slopes faces. Permanent slopes should be revegetated and/or armored as soon as practical to reduce erosion.

5.4 FILL

All fill should be free of lumps, wood, topsoil, frozen or other deleterious material. Excavated bedrock should be broken down into particles less than 6 inches (in maximum dimension) prior to use as fill.

Fill should be placed in uniform lifts with a maximum loose lift thickness of 8 inches. Fill should be compacted to a minimum of 95 percent compaction (in accordance with AASHTO T-99). The moisture content of the fill material should be distributed uniformly throughout each layer of the material prior to and during compaction. The fill should be placed at a moisture content within 2 percent of optimum.

Fill should be benched into existing hillsides having slopes steeper than 4H:1V.

5.5 SETTLEMENT

Compressible clay deposits were encountered in our borings throughout the project area. These deposits will consolidate if loaded by footings or new fill. The magnitude of settlement depends on the size and shape of the loaded area and the thickness of the compressible soils. Larger loads and larger sized loaded areas will settle more than smaller loads and smaller loaded areas. The amount of settlement will be directly related to the thickness of compressible deposits.

Settlement of Embankments

The fill within large embankments will settle independently of the underlying compressible deposits. Fill settlement will generally occur as elastic deformation and consolidation of the fill during and after placement. Settlement of a properly compacted fill constructed of granular soil is estimated to be on the order of 0.25 to 0.5 percent of the fill height.

Planned construction of embankments for most of the proposed bridges requires construction of large embankments over potentially compressible clay deposits. Based on thickness of compressible deposits in the borings near each proposed structure and the planned embankments, we calculated estimated settlement at each bridge abutment where a fill embankment is planned. However, subsurface conditions are highly variable throughout the site and actual settlement will be dependent on the depth of compressible deposits at any specific location. Table 5-1 gives total estimated settlement of fill embankments for each of the bridge abutments. Total estimated settlement includes embankment settlement and settlement of compressible deposits underlying the embankment.

Table 5.1
Total Estimated Embankment Settlement

Bridge	Abutment Number	Approximate Planned Embankment Height (feet)	Total Estimated Post-Construction Settlement (inches)
US 550 Over US 160	1	20	6 – 8
	4	49	22 – 29
Ramp A	1	6	2 – 4
	5	26	20 – 27
Ramp C	1	33	19 – 25
	3	10	7 – 9
Ramp D	1	19.5	11 – 14
	3	27	15 – 19
Ramp E	1	25	2 – 4
	3	16	5 – 7
US 160 Over CR 233	1	36	21 – 28
	2	36	3 –5
US 160 Over SH 172	1	20	12 – 15
	2	30	14 – 19

These estimated settlements will cause differential settlement between the abutment supported on deep foundations in the relatively incompressible bedrock and the embankment. At the edge of the abutment, this differential settlement will be nearly equal to the total estimated settlement. Settlement of the embankment will largely occur during construction. Settlement of underlying compressible deposits may take 3 months or longer. The impact of the settlement may be decreased by allowing time between embankment construction and bridge and roadway construction.

The time required for settlement of the compressible deposits may be decreased by use of surcharge and vertical drains. The surcharge is fill placed above the height of the planned embankment. Once settlement has occurred, the extra surcharge fill is removed. Design of surcharge should be performed when bridge plans are finalized.

Bridge Settlement

Settlement of bridge abutments and piers founded on deep foundations founded in the bedrock is estimated as 0.5 to 1 inch. Settlement at the abutments of the SH 172 bridge, founded in the gravels and cobbles, is estimated as 0.5 to 1 inch.

Retaining Wall Settlement

The settlement of the MSE retaining walls will be similar to the embankment settlement estimates given in Table 5.1.

Settlement at the proposed 10-foot high cantilever wall (the bottom portion of the proposed retaining wall) at Abutment 1 of Ramp E, with footings founded on bedrock, is estimated as 0.5 to 1 inch.

Site drainage and moisture protection is an important design consideration to reduce the potential for moisture-related volume change of the subgrade soil. Drainage should be designed to prevent infiltration of water into the soil and below pavements during construction and for the life of the roadways.

Soil within subgrade and foundation excavations should be maintained at in-situ moisture content during each phase of construction. Water infiltration should be prevented to reduce potential for softening and weakening exposed subgrade soil. Drying of exposed soils should be avoided by timely backfilling of excavations. Soils should not be left exposed for more than one week without moisture treatment to avoid drying. Handling of stormwater runoff will be required during and after construction.

Site grading should be designed to direct water away from structures and should be maintained as needed.

Our investigation found conditions at the project site are highly variable, especially near Wilson Gulch. The observed cobble and boulder layers are dispersed throughout the site and may exist at proposed structure abutment and pier locations. Compressible deposits were observed near Wilson Gulch and may also be present underlying planned piers and abutments.

We recommend additional investigation at final pier and abutment locations before finalizing bridge plans. Borings should be drilled at locations that were not accessible for this investigation. Due to highly variable conditions revealed by this investigation, additional borings, test pits, and laboratory testing is recommended to further characterize geologic conditions at each pier and abutment location. We also recommend constructing test piers and piles to develop final designs and plans.

The recommendations presented in this report are based on subsurface conditions indicated in our exploratory borings. Conditions between borings will vary and may differ substantially from those indicated by the borings. These recommendations are subject to revision based upon field engineering, monitoring, and testing during construction. Additionally, if loadings or proposed roadway and/or structure locations are modified from those provided to us for these analyses, the recommendations contained herein should not be considered valid until reevaluated by URS.

URS represents that our services were performed within the limits prescribed by our Client, with the usual thoroughness and competence of the engineering profession. No warranty, guarantee, or other representation, expressed or implied, is included or intended in our proposals, contracts, or reports.

This report was developed by URS for the sole use by its Client. The use of any part of this report by other parties or for other projects without the expressed written permission of URS is prohibited.

The following URS personnel were involved in the geotechnical investigation work for this project.

Geotechnical Investigation and Field Activities

Juan Sorensen managed the geotechnical investigation. Field explorations were performed under the direction of Scott Walker, Kelly Young, and Christina Winckler. The URS Durango office supplied field support under the direction of Steven Cross and Ron Pemberton.

Reporting

Juan Sorensen managed preparation of this report. Kelly Young and Juan Sorensen wrote the geotechnical report. Chris Snyder and Randy Pike drafted and detailed the figures. Jason Boomer performed the detail check. Jim Scott was the independent technical reviewer.

Ron Gibson served as overall manager for the US 160 project.

Appendix A
Laboratory Test Results

RockSol
Consulting Group,Inc.

SUMMARY OF LABORATORY TEST RESULTS

CLIENT URS

PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)	Unconfined Compression (psi)	PH	R-Value at Exudation Pressure 300 psi
1B6	2.0	35	18	17	94	10.7	118.5			
1B6	15.0	33	20	13	91	15.9	110.2			
1B6	30.0	NP	NP	NP	23					
1P1	2.0	39	25	14						
1P18	2.0	37	28	9	25					
1P2	2.0	30	18	12						
1P20	4.0				21					
1P20	14.0							15.3		
1P20	19.0	36	20	16	73					
1P22	2.0	32	20	12	22	12.2				
1P3	2.0							23.2		
1P4	4.0	40	24	16	97					
1W23	5.0	28	22	6	21					
1W7	2.0	23	19	4	27					
1W7	14.0	39	19	20	48					
1W7	24.0	49	28	21	55					
2B15	10.0	30	22	8	22					
2B17	14.0	37	24	13						
2B17	19.0	31	24	7	54	25.8				
2B17	24.0	35	25	10	35	26.6				
2B31	4.0	42	20	22	37	19.2				
2P28	4.0	44	23	21	34					
2P54	2.0	32	24	8	9					
2P55	2.0	46	28	18	36					
3B8	15.0	36	26	10	73	33.3	89.5			
3B8	19.0	41	20	21	60	30.2	93.7	10.5		
3B8	30.0	46	29	17	72	28.9	96.0			
3B9	15.0	47	23	24	67	27.2	95.1			
3P1	4.0	45	31	14	45					
3P5	4.0							22.9		
4B18	2.0	30	16	14	16	9.1				
4B26	2.0	49	23	26	80	23.0				
4B26	19.0	49	31	18	68	10.9				
4B4	14.5	38	27	11	65					
4B5	4.0	43	23	20	61	23.2				
4B5	9.0	40	20	20	68	20.8				
4B5	14.0	45	24	21	79	23.5				
4B5	19.0	37	26	11	47	21.2				
4B6	4.5	33	26	7	63	13.4	121.4			
4B7	4.0	44	25	19	87	25.9				
4B7	14.0	38	24	14	70	22.4				
4B7	19.0	50	30	20	83	24.2				
4P1	2.0	48	20	28	89	25.6				

RockSol
Consulting Group,Inc.

SUMMARY OF LABORATORY TEST RESULTS

CLIENT URS

PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	%<#200 Sieve	Water Content (%)	Dry Density (pcf)	Unconfined Compression (psi)	PH	R-Value at Exudation Pressure 300 psi
4P11	4.0					21.3	104.5			
4P12	2.0	25	19	6						
4P2	2.0	52	22	30	67	24.5				
4P20	2.0	38	17	21	95	18.8				
4P27	2.0	44	19	25						
4P27	5.0	35	25	10	56	13.4	110.2			
4P28	2.0				97	20.7				
4P29	2.0					15.7	112.4			
4P29	4.0	35	17	18	49	14.5				
4P3	2.0					15.1	109.4			
4P30	2.0					16.6	113.5	90.8		
4P31	2.0	27	16	11	67	22.1				
4P9	2.0					23.3	103.3	75.7		
4P9	4.0	45	20	25	79	21.3				
4W13	4.5					16.7	109.3			
4W13	9.5	36	17	19	84	20.2	106.9			
6B7	9.0	40	18	22	75					
6B9	5.0				17					
6P2	14.0	18	14	4	27	0.4				
6P4	2.0	33	19	14						
6P6	9.5								8.7	
6W4	2.0	28	21	7	61	14.7	113.0			
6W4	5.5				11	12.3	122.2			
S-48	2.0					16.3	115.0			
S-48	4.0	47	20	27	15	15.2				
S-50	2.0	42	21	21	23	16.2				
S-50	5.0									8.0
S-51	2.0	41	23	18						
S-51	4.0					11.9	113.1			
S-52	2.5					24.5	100.0	40.0		
S-52	4.0	41	31	10	75	24.3				
S-53	2.5	53	20	33	83	26.1				
S-53	4.5								8.4	
S-54	2.0	41	19	22	18	6.3				
S-54	3.5	38	22	16	61	19.4				
S-54	5.0									26.0

RockSol
Consulting Group,Inc.

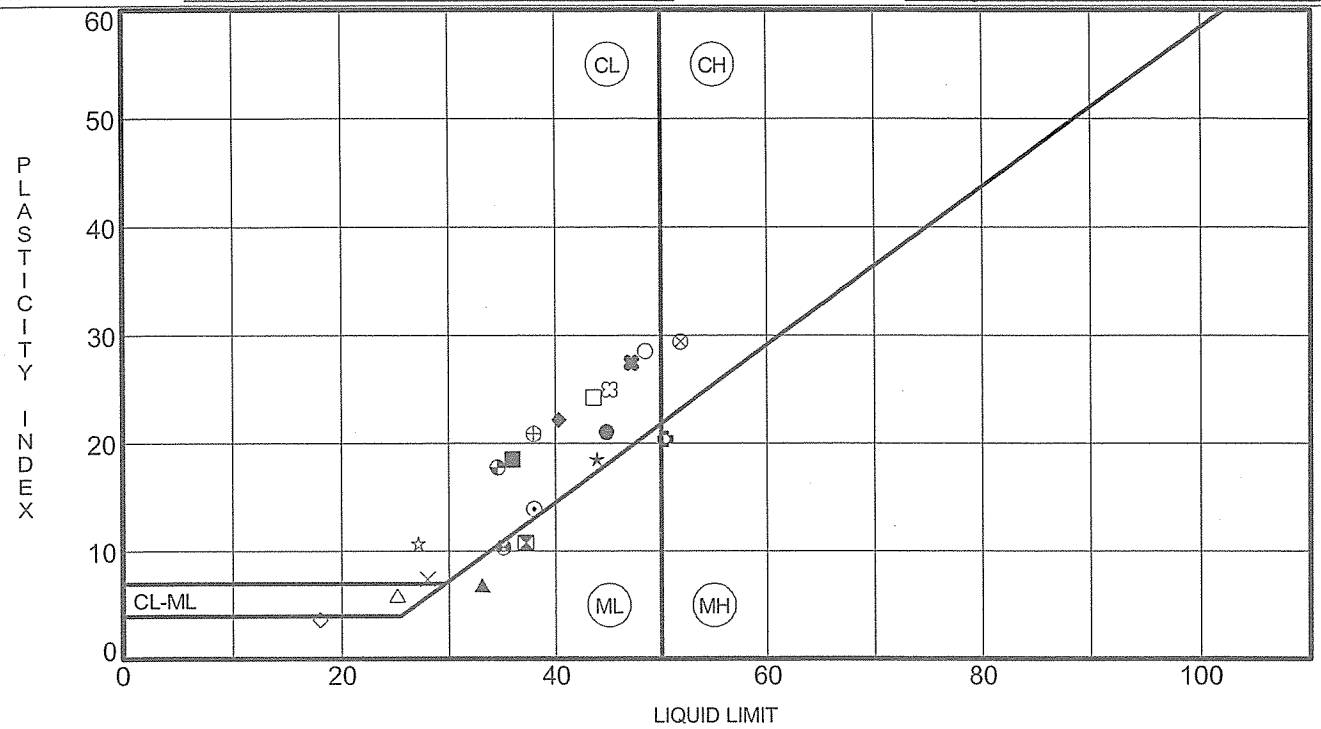
ATTERBERG LIMITS' RESULTS

CLIENT URS

PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado



	Specimen Identification		LL	PL	PI	Fines	Classification
●	4B5	14.0	45	24	21	79	LEAN CLAY with SAND(CL)
⊠	4B5	19.0	37	26	11	47	SILTY SAND(SM)
▲	4B6	4.5	33	26	7	63	SANDY SILT(ML)
★	4B7	4.0	44	25	19	87	LEAN CLAY(CL)
⊙	4B7	14.0	38	24	14	70	SANDY LEAN CLAY(CL)
✿	4B7	19.0	50	30	20	83	ELASTIC SILT with SAND(MH)
○	4P1	2.0	48	20	28	89	LEAN CLAY(CL)
△	4P12	2.0	25	19	6		
⊗	4P2	2.0	52	22	30	67	SANDY FAT CLAY(CH)
⊕	4P20	2.0	38	17	21	95	LEAN CLAY(CL)
□	4P27	2.0	44	19	25		
⊖	4P27	5.0	35	25	10	56	SANDY SILT(ML)
◐	4P29	4.0	35	17	18	49	CLAYEY SAND(SC)
☆	4P31	2.0	27	16	11	67	SANDY LEAN CLAY(CL)
⊠	4P9	4.0	45	20	25	79	LEAN CLAY with SAND(CL)
■	4W13	9.5	36	17	19	84	LEAN CLAY with SAND(CL)
◆	6B7	9.0	40	18	22	75	LEAN CLAY with SAND(CL)
◇	6P2	14.0	18	14	4	27	SILTY, CLAYEY SAND(SC-SM)
×	6W4	2.0	28	21	7	61	SANDY SILTY CLAY(CL-ML)
✠	S-48	4.0	47	20	27	15	CLAYEY SAND(SC)

RockSol
Consulting Group,Inc.

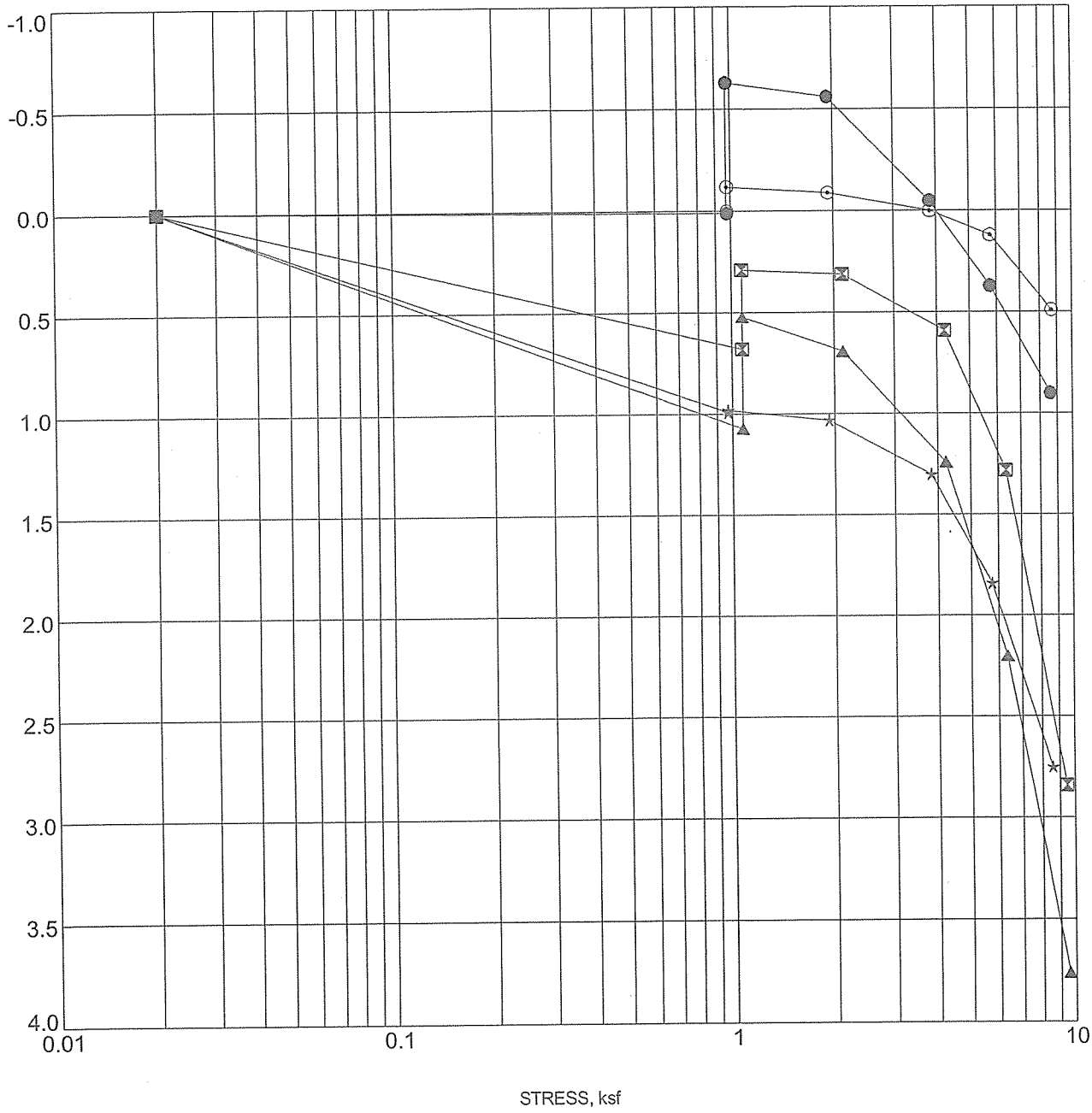
SWELLING - CONSOLIDATION TEST

CLIENT URS

PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado



Specimen Identification			Classification	γ_d	MC%
●	1P18	2.0	SILTY SAND(SM)		
⊠	1P20	2.0			
▲	1P4	2.0			
✶	2B17	4.0			
⊙	2P57	2.0			

RockSol
Consulting Group,Inc.

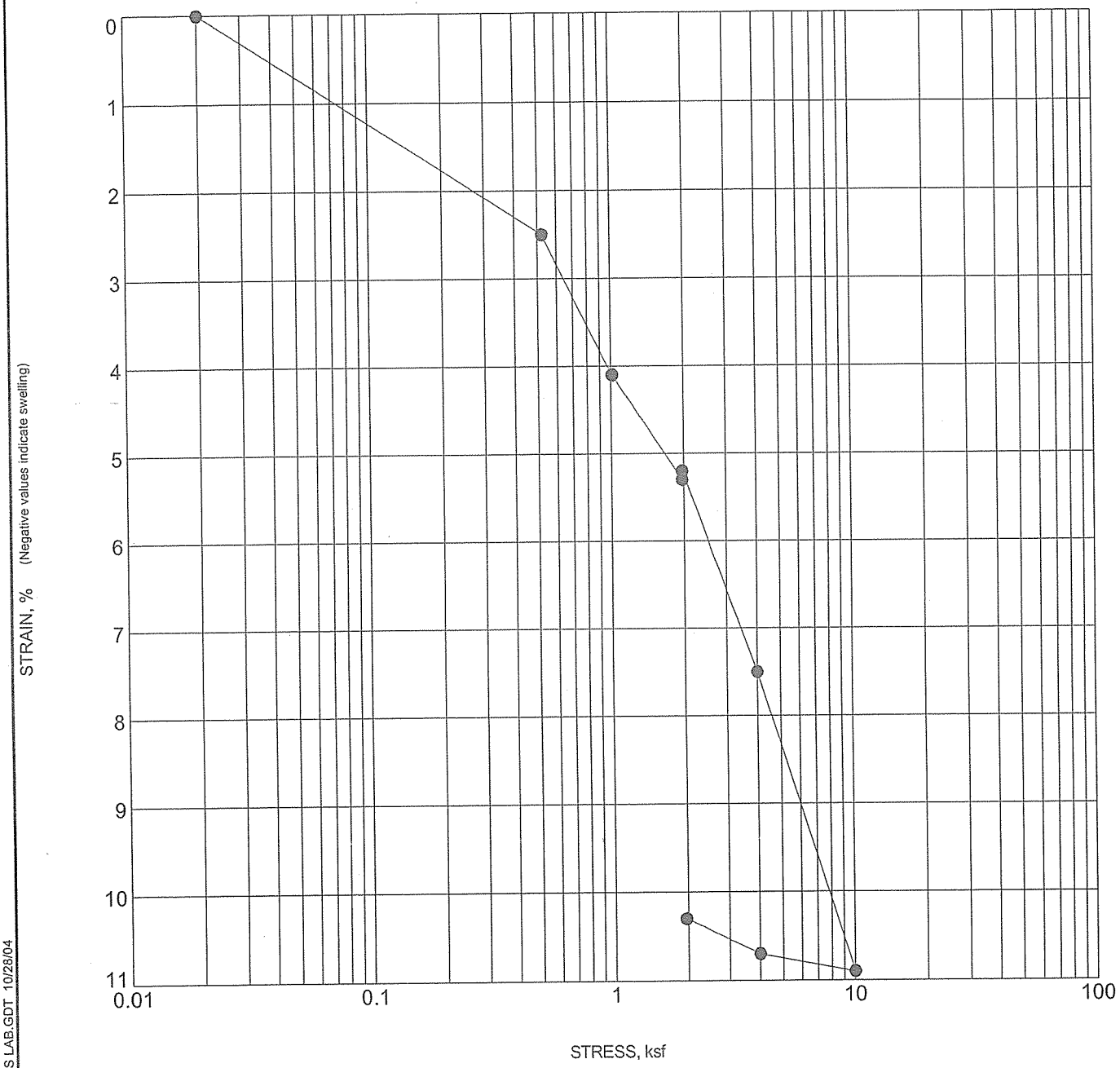
SWELLING - CONSOLIDATION TEST

CLIENT URS

PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado



Specimen Identification		Classification	γ_d	MC%
● 3B8	19.0	SANDY LEAN CLAY(CL)	95	28

SWELL DURANGO DURANGO BRIDGE.GPJ GINT US LAB.GDT 10/28/04

ATTERBERG LIMITS' RESULTS

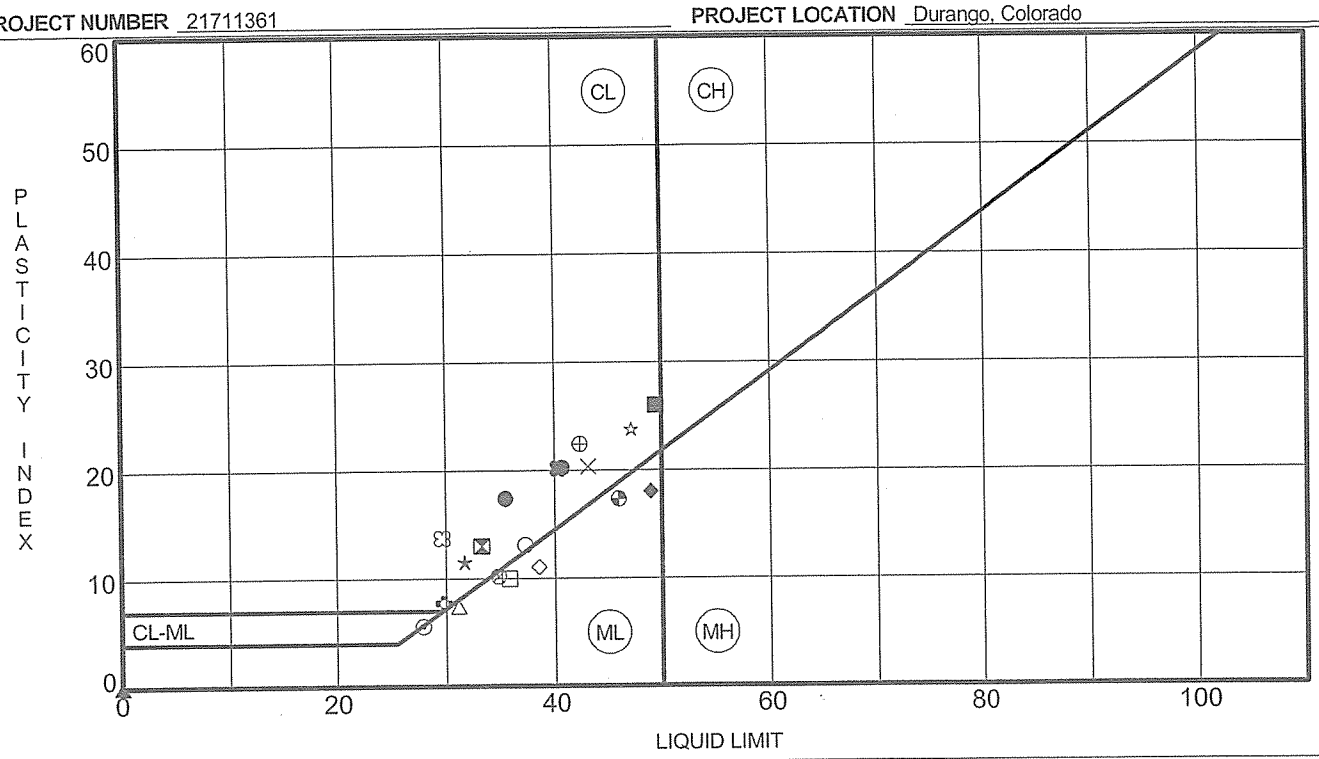
RockSol
Consulting Group,Inc.

CLIENT URS

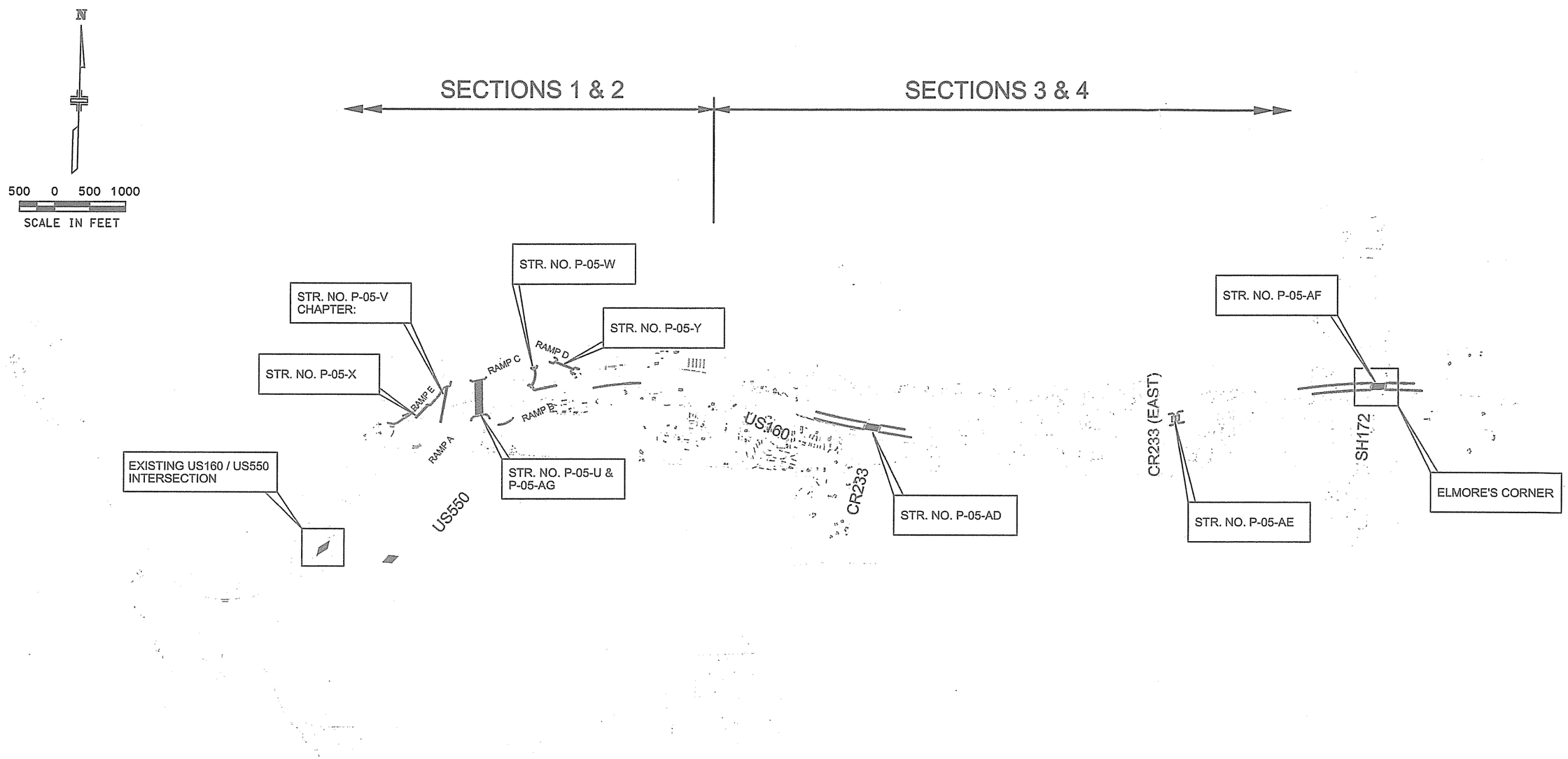
PROJECT NAME US160

PROJECT NUMBER 21711361

PROJECT LOCATION Durango, Colorado



	Specimen Identification		LL	PL	PI	Fines	Classification
●	1B6	2.0	35	18	17	94	LEAN CLAY(CL)
⊠	1B6	15.0	33	20	13	91	LEAN CLAY(CL)
▲	1B6	30.0	NP	NP	NP	23	SILTY SAND(SM)
✶	1P22	2.0	32	20	12	22	CLAYEY SAND(SC)
⊙	1W23	5.0	28	22	6	21	SILTY, CLAYEY SAND(SC-SM)
✿	2B15	10.0	30	22	8	22	CLAYEY SAND(SC)
○	2B17	14.0	37	24	13		
△	2B17	19.0	31	24	7	54	SANDY SILT(ML)
⊗	2B17	24.0	35	25	10	35	SILTY SAND(SM)
⊕	2B31	4.0	42	20	22	37	CLAYEY SAND(SC)
□	3B8	15.0	36	26	10	73	SILT with SAND(ML)
⊖	3B8	19.0	41	20	21	60	SANDY LEAN CLAY(CL)
⊕	3B8	30.0	46	29	17	72	SILT with SAND(ML)
☆	3B9	15.0	47	23	24	67	SANDY LEAN CLAY(CL)
✿	4B18	2.0	30	16	14	16	CLAYEY SAND(SC)
■	4B26	2.0	49	23	26	80	LEAN CLAY with SAND(CL)
◆	4B26	19.0	49	31	18	68	SANDY SILT(ML)
◇	4B4	14.5	38	27	11	65	SANDY SILT(ML)
×	4B5	4.0	43	23	20	61	SANDY LEAN CLAY(CL)
✿	4B5	9.0	40	20	20	68	SANDY LEAN CLAY(CL)



US160 SECTION 1 - 4 BRIDGES

FIG. 2-1

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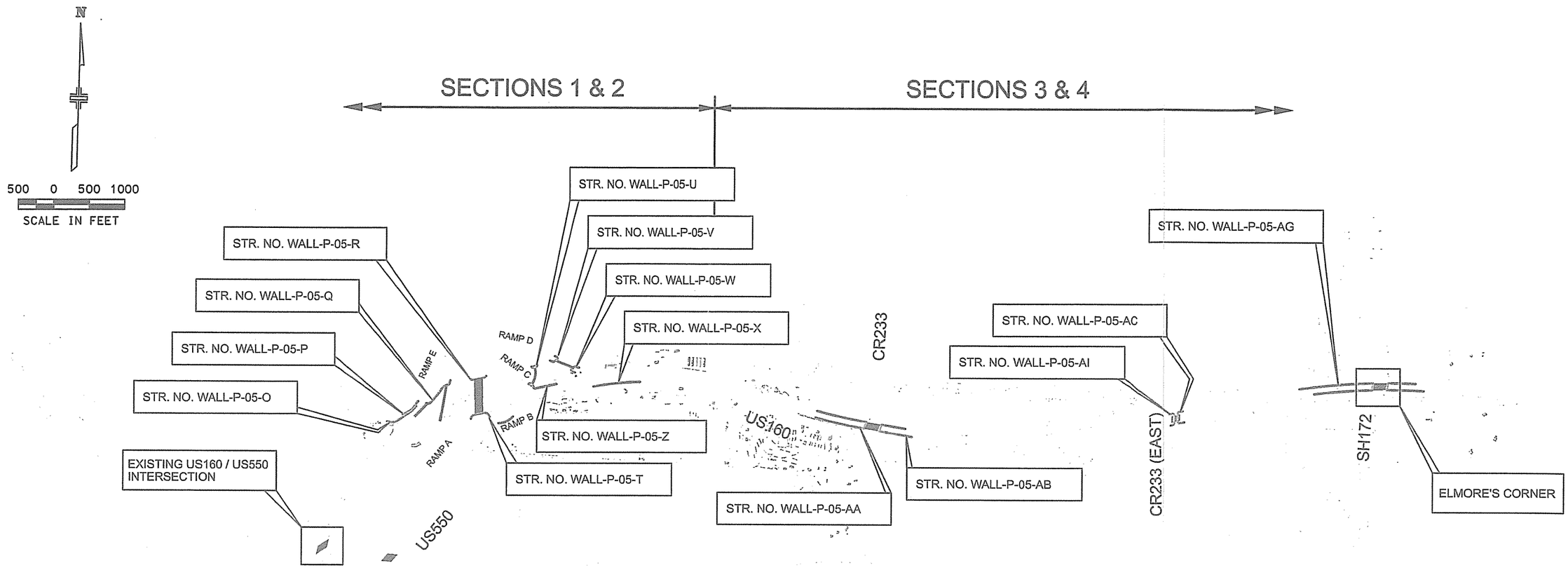

US160 SECTION 1 - 4 WALLS

FIG. 2-2

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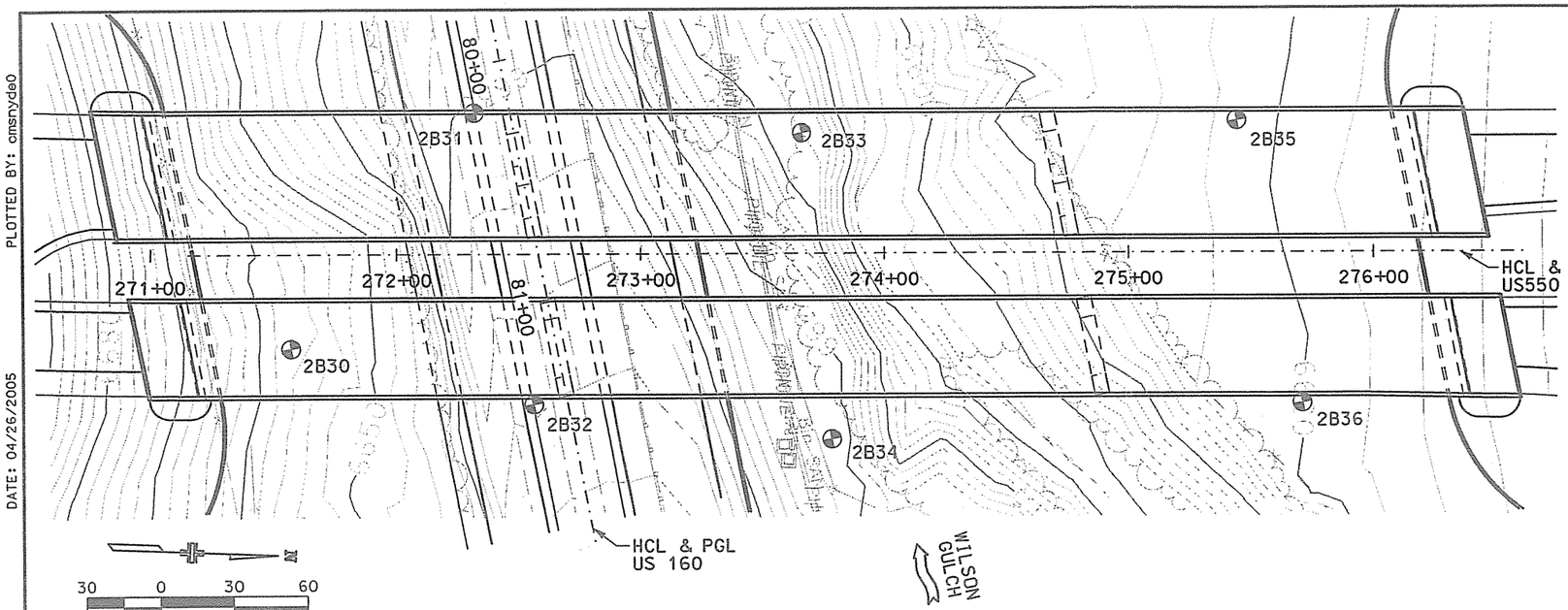

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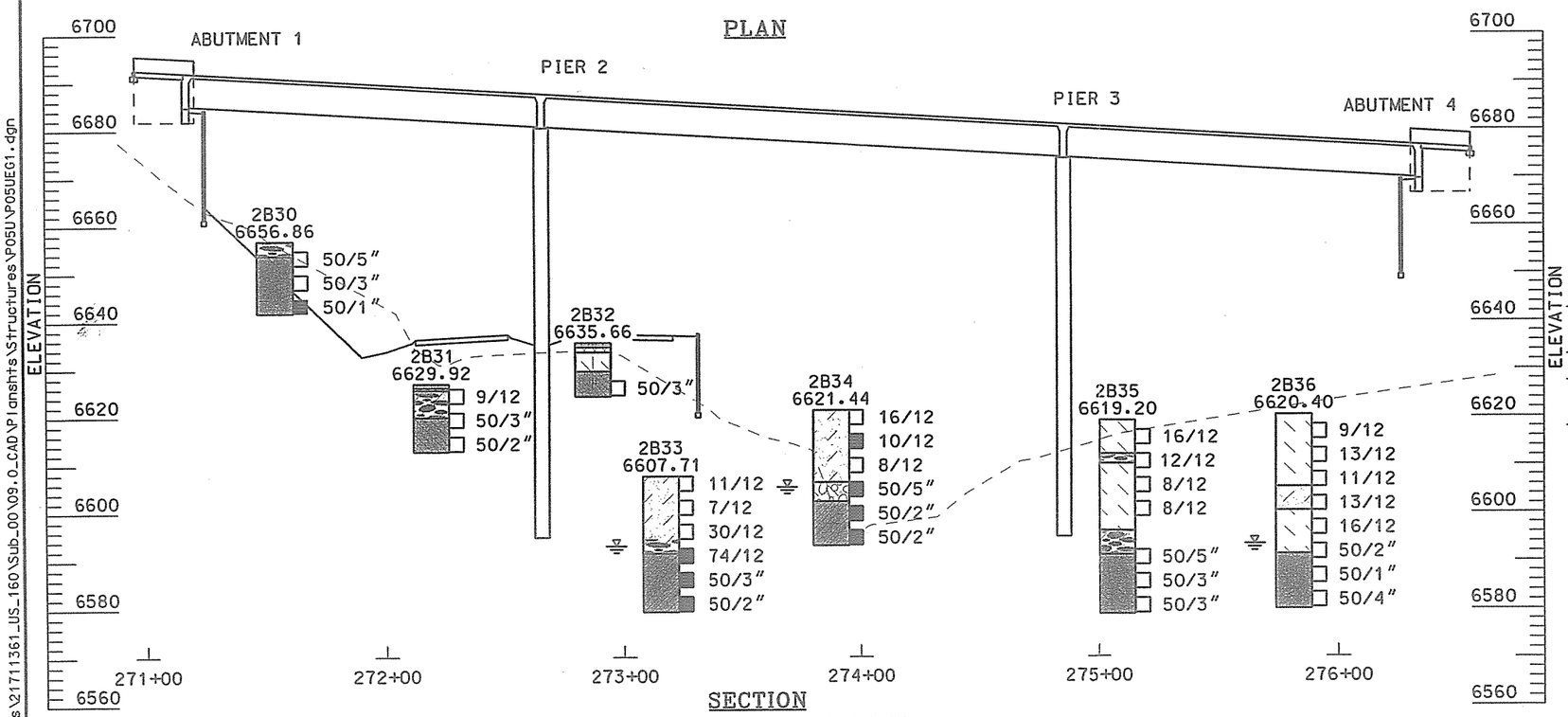

SECTION
(TAKEN AT N.B. LAYOUT LINE)

SOIL LEGEND

- ASPHALT CONCRETE PAVEMENT
- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL TO SLIGHTLY GRAVELLY, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST, GREY TO BROWN (CL)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY SILTY TO VERY SILTY, TRACE GRAVEL TO GRAVELLY, LOOSE TO MEDIUM DENSE, LOW TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, DENSE TO VERY DENSE, MOIST TO WET, BROWN WITH VARIABLE ACCESSORY COLORS (GP, GM, GC, GP-GM, GP-GC)
- COBBLES AND BOULDERS
- CLAYSTONE, SANDY TO VERY SANDY, TRACE SILT TO SILTY, VERY HARD, DRY TO WET, GREY TO BROWN (BEDROCK)
- SILTSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING
- APPROXIMATE BORING LOCATION

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

PLOTTED BY: omsnyde0
DATE: 04/26/2005
FILE NAME: n:\projects\21711361_US_160\Sub_00\09.0_CAD\Plansht's\Structures\P05U\P05UEG1.dgn

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Full Path: SEE PLOT STAMP		
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Microstation Ver.	Scale: AS SHOWN	Units:

Index of Revisions

DOT DEPARTMENT OF TRANSPORTATION Region 5

URS
10 Burnett Court Suite 113
Durango, Colorado 81301
(970) 259-9212
Fax (970) 259-9241

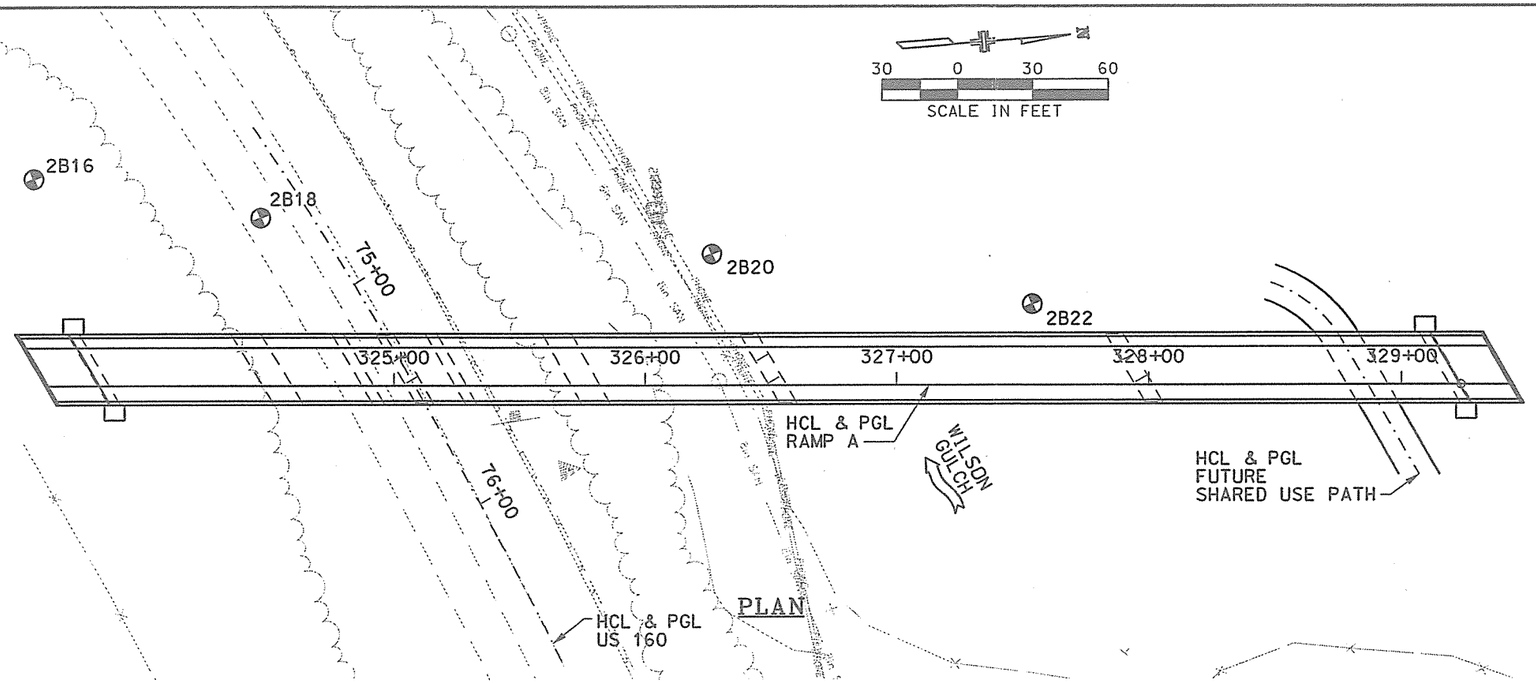
As Constructed
No Revisions:
Revised:
Void:

US550 OVER US160
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: C. SNYDER
Structure Numbers: P-05-U
SheetSubset:
Subset Sheets:

Project No./Code

Sheet Number FIG. 2-3



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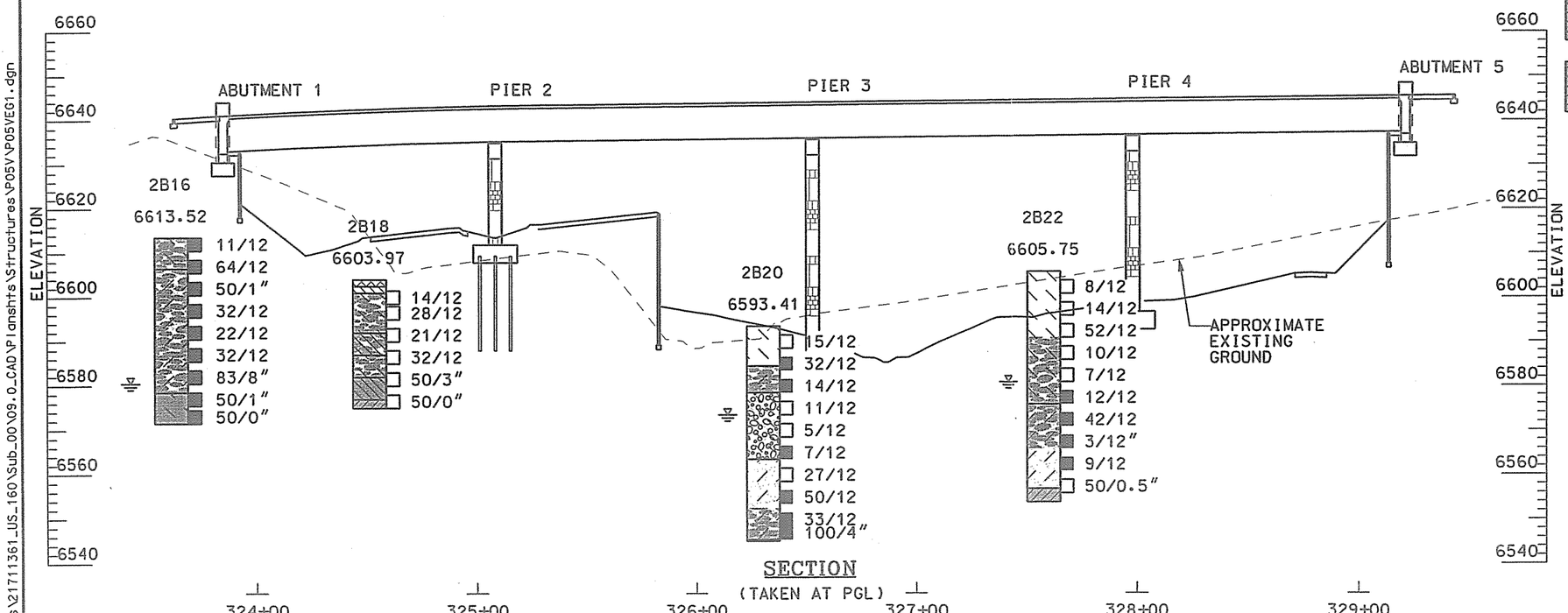

SECTION
(TAKEN AT PGL)

LEGEND

- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN.
- CLAY, SLIGHTLY SANDY TO SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL TO SLIGHTLY GRAVELLY, STIFF TO VERY STIFF, LOW PLASTICITY, MOIST TO WET, MEDIUM BROWN (CL)
- CLAY, SLIGHTLY SANDY TO SANDY, SILTY TO VERY SILTY, SOFT TO STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY (CH)
- SILT, SLIGHTLY CLAYEY TO CLAYEY, SANDY TO VERY SANDY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, BROWN (ML)
- SAND, CLAYEY TO VERY CLAYEY, SILTY, TRACE GRAVEL TO GRAVELLY, MEDIUM DENSE TO VERY DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, LOOSE TO MEDIUM DENSE, MOIST TO WET, BROWN (GP, GM, GC)
- COBBLES AND BOULDERS
- CLAYSTONE, SANDY TO VERY SANDY, TRACE SILT TO SILTY, VERY HARD, MOIST TO WET, GREY (BEDROCK)
- SILTSTONE, SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, MOIST TO WET, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, MOIST TO WET, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

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Full Path: SEE PLOT STAMP		R–
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Microstation Ver. V8	Scale: AS SHOWN Units:	R–

Index of Revisions

DOT DEPARTMENT OF TRANSPORTATION Region 5

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As Constructed
No Revisions:
Revised:
Void:

RAMP A
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: R. PIKE
SheetSubset:
Structure Numbers: P-05-V
Subset Sheets:

Project No./Code
FC-NH(CX)160-2(048)
91050
Sheet Number FIG. 2-4

PLOTTED BY: rgplkex0
DATE: 04/25/2005
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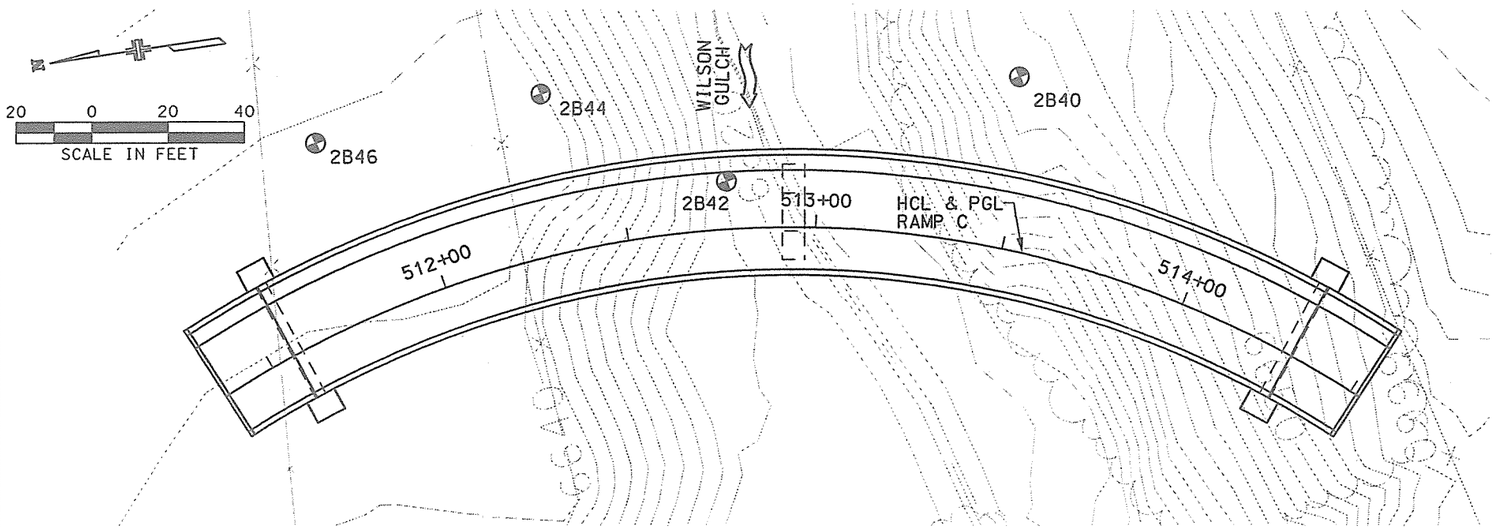

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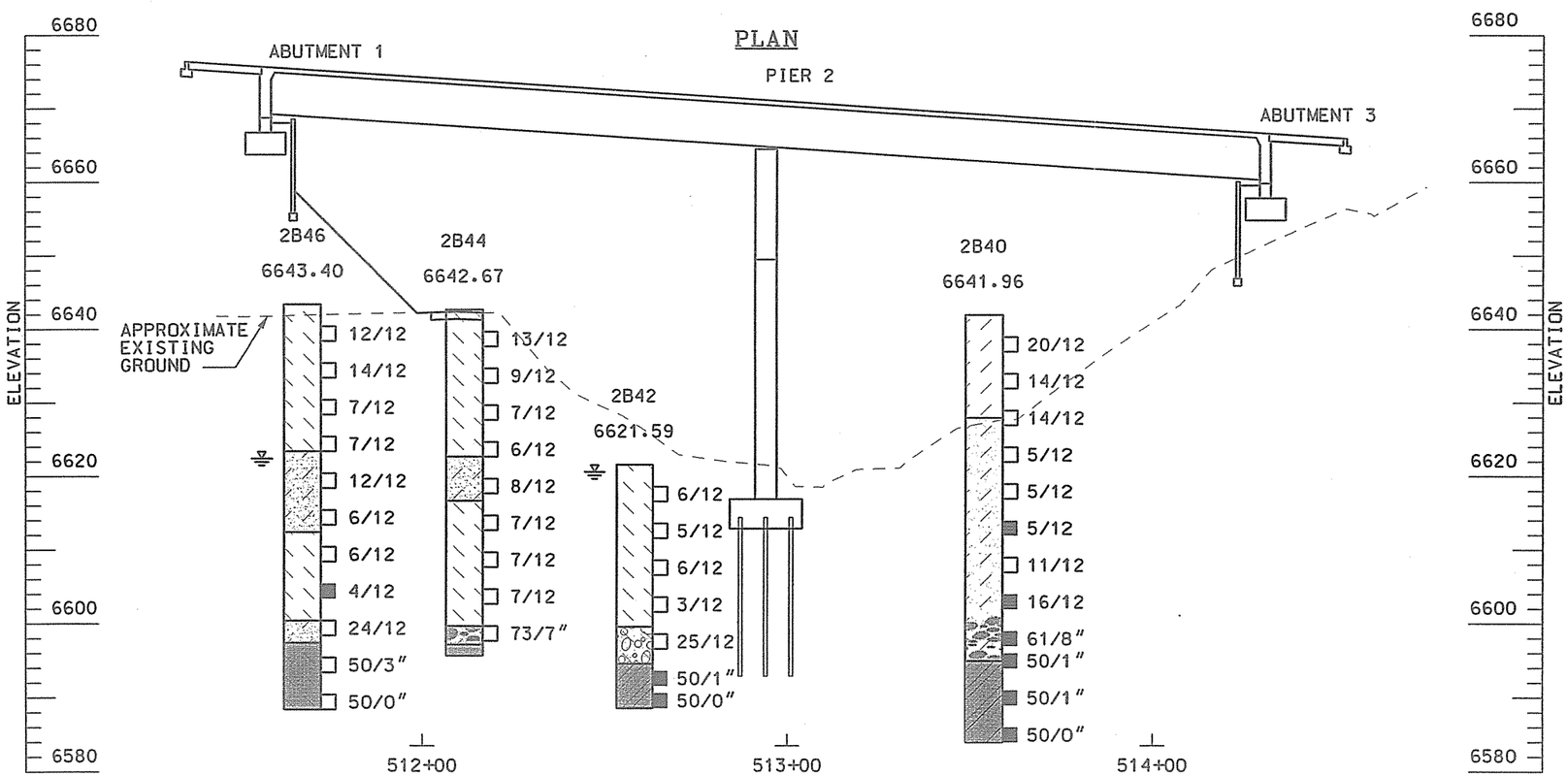

SECTION
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LEGEND

- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, DARK GREY TO DARK BROWN (CL)
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SILTY TO VERY SILTY, STIFF TO VERY STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY TO MEDIUM BROWN (CH)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY SILTY, SLIGHTLY GRAVELLY, LOOSE TO MEDIUM DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, MEDIUM DENSE TO DENSE, MOIST, BROWN (GP, GM, GC)
- COBBLES AND BOULDERS
- CLAYSTONE, TRACE SILT TO SILTY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

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03/31/2005

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RAMP C
ENGINEERING GEOLOGY

Designer:	K. YOUNG	Structure Numbers	P-05-W
Detailer:	R. PIKE		
SheetSubset:		Subset Sheets:	

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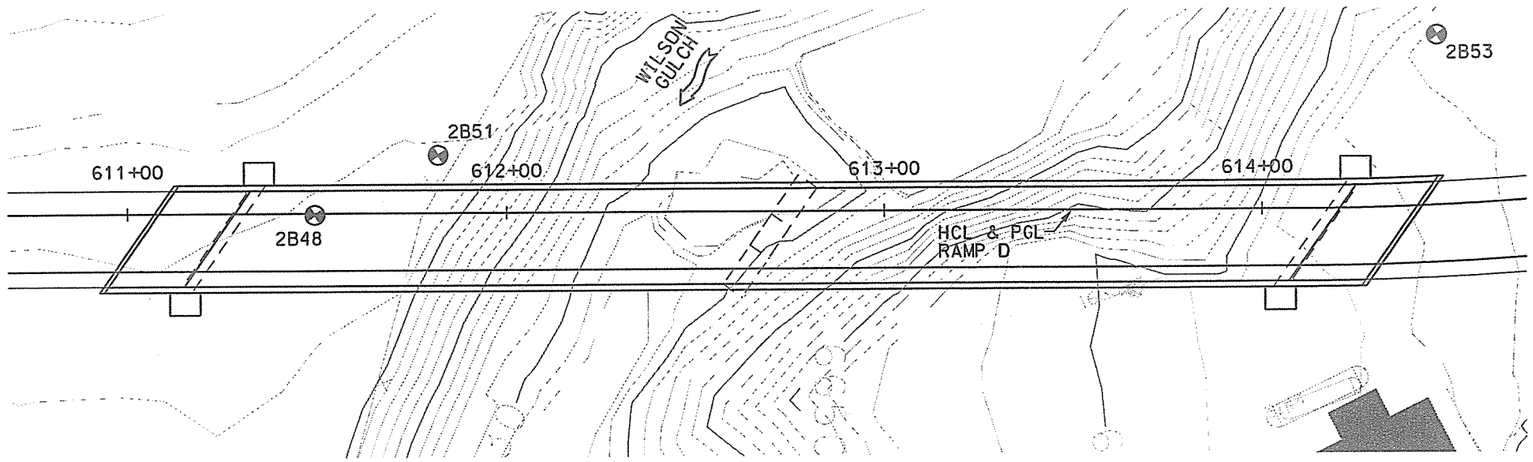

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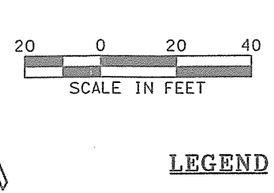
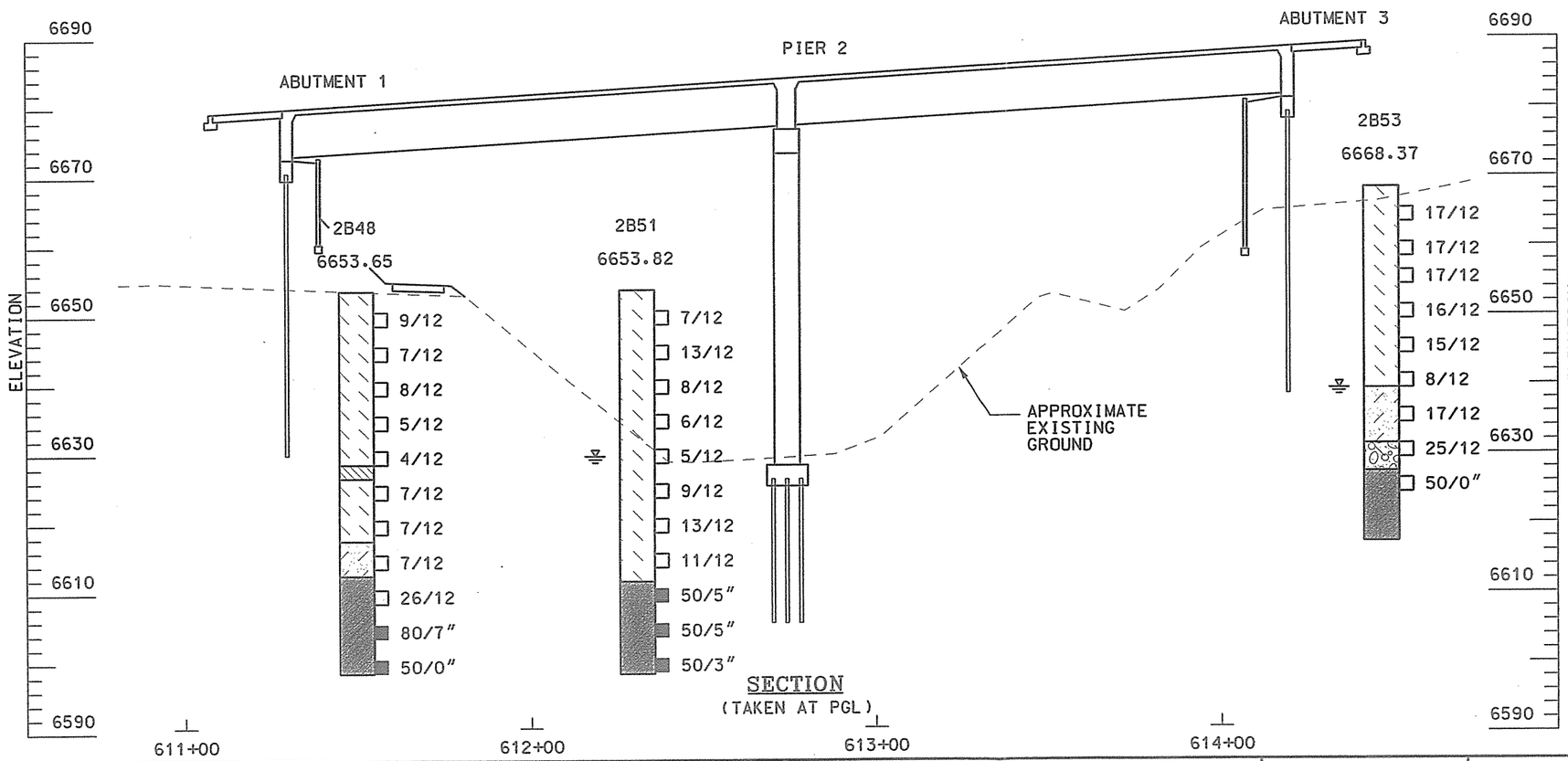


LEGEND

- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST TO WET, DARK GREY TO BROWN (CL)
- SILT, SLIGHTLY CLAYEY TO CLAYEY, SANDY TO VERY SANDY, STIFF TO VERY STIFF, LOW PLASTICITY, MOIST, DARK GREY TO MEDIUM BROWN (ML)
- SAND, CLAYEY TO VERY CLAYEY, TRACE GRAVEL, LOOSE TO MEDIUM DENSE, LOW TO MODERATE PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, MEDIUM DENSE, WET, BROWN (GP, GM, GC)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)



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SYMBOL LEGEND

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Revised:	Designer: K. YOUNG	Structure Numbers	P-05-Y	91050
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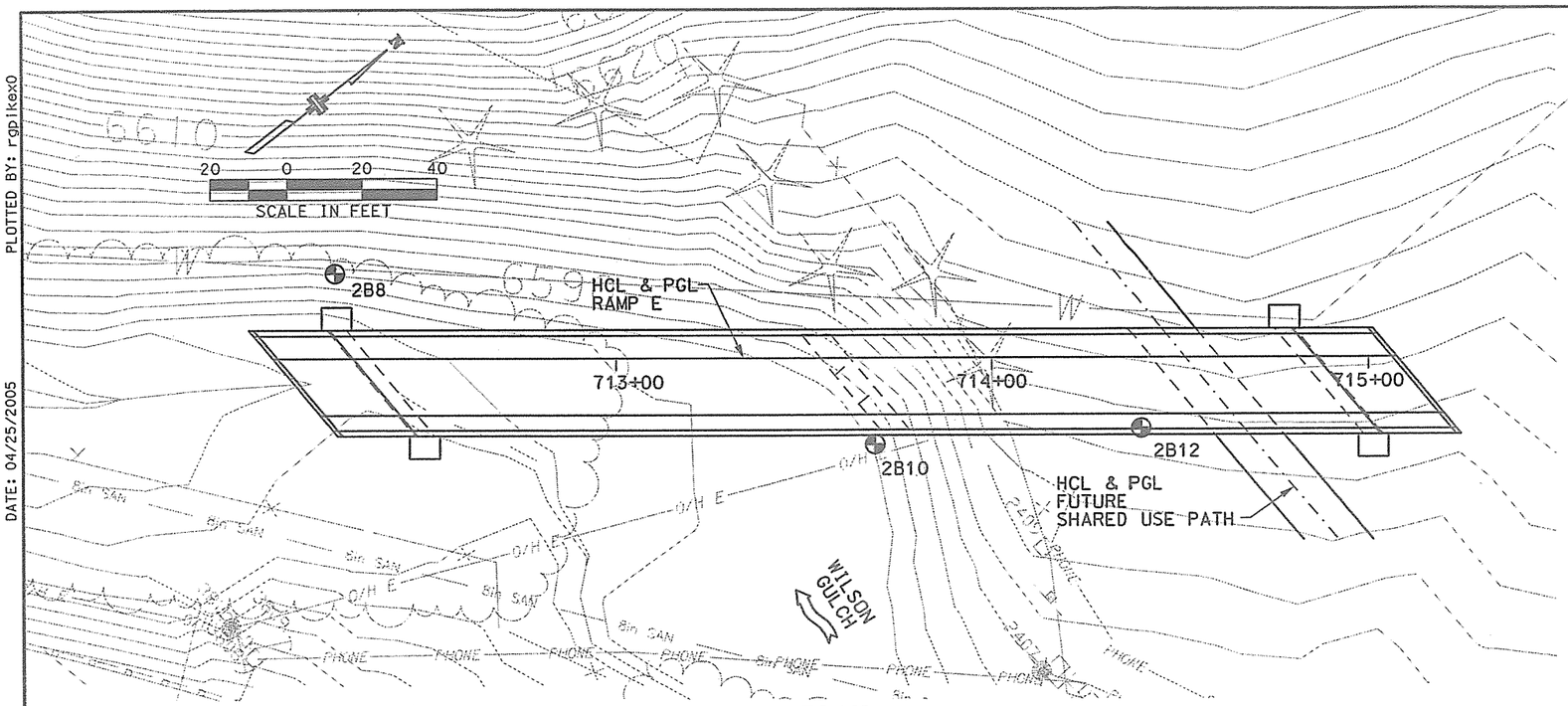

PLAN

LEGEND

- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST TO WET, BROWN (CL)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY GRAVELLY, LOOSE TO MEDIUM DENSE, LOW TO MODERATE PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- COBBLES AND BOULDERS
- CLAYSTONE, SANDY TO VERY SANDY, SILTY, HARD TO VERY HARD, DRY, GREY TO BLUISH-GREY (BEDROCK)
- SANDSTONE, SLIGHTLY CLAYEY TO CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

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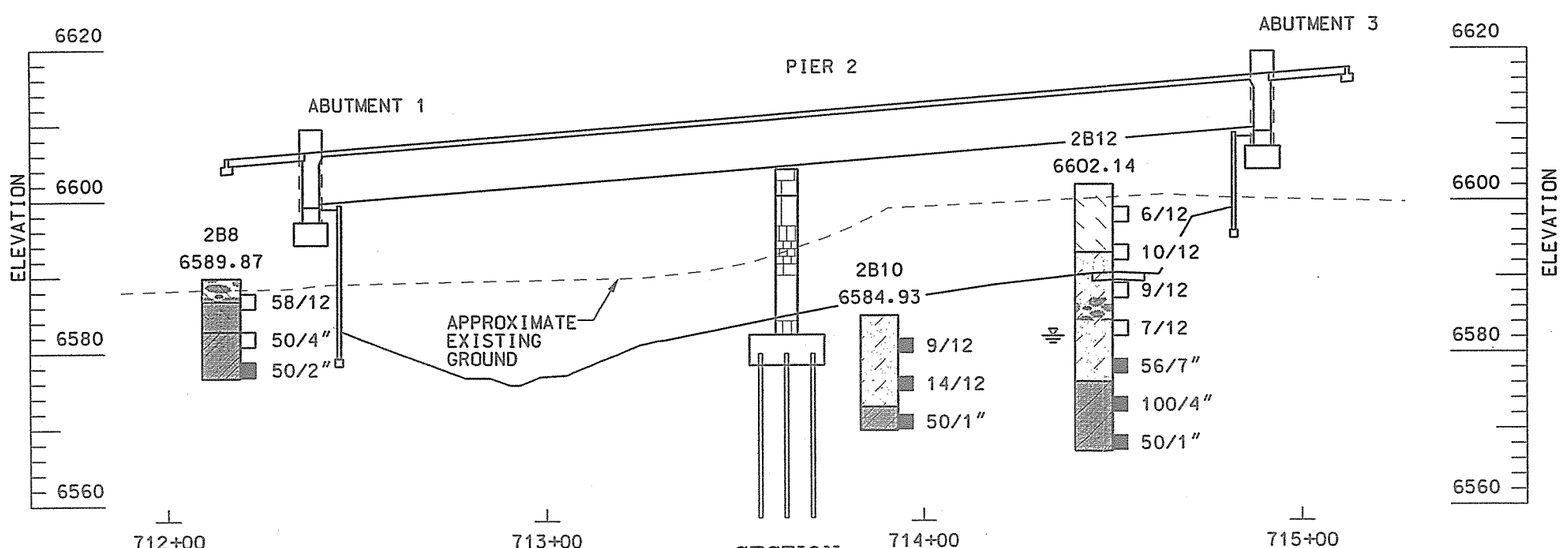
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING



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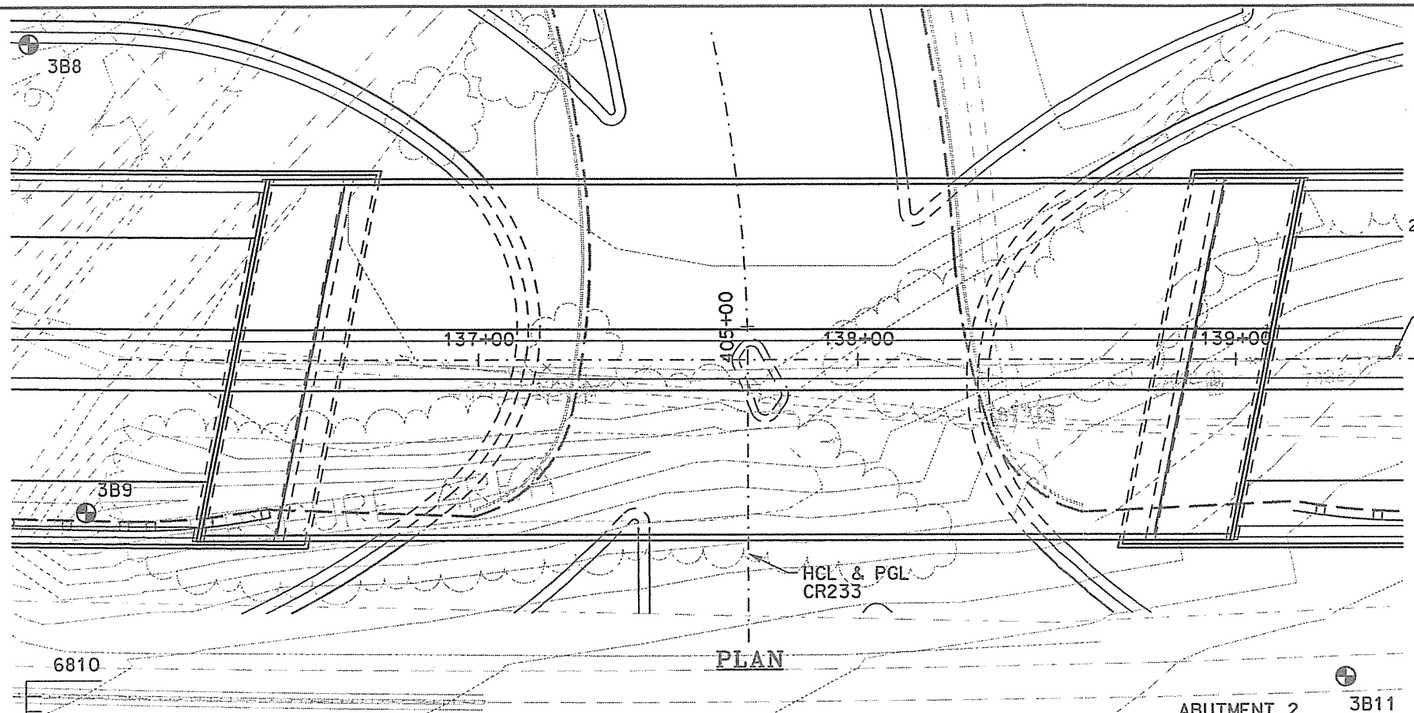

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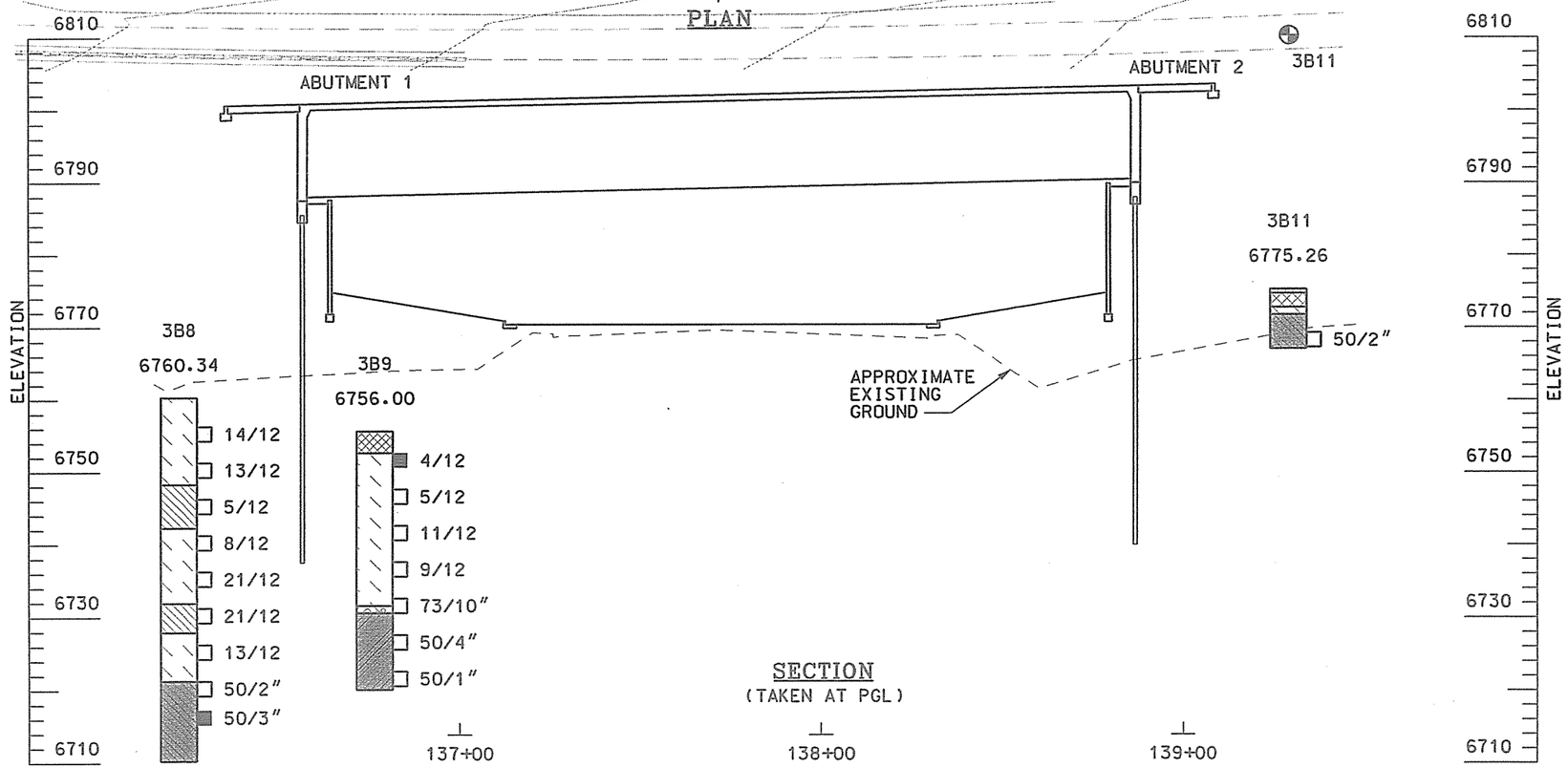

SECTION
(TAKEN AT PGL)

LEGEND

- ASPHALT CONCRETE PAVEMENT
- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN.
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST TO WET, GREY TO BROWN (CL)
- SILT, SLIGHTLY CLAYEY TO CLAYEY, SANDY TO VERY SANDY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, MEDIUM BROWN (ML)
- GRAVEL, SANDY, SLIGHTLY SILTY , CLAYEY TO VERY CLAYEY, VERY DENSE, MOIST, BROWN WITH VARIABLE ACCESSORY COLORS (GP, GC)
- SILTSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)

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- APPROXIAMTE BORING LOCATION

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US160 OVER CR233
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: R. PIKE
SheetSubset:
Structure Numbers: P-05-AD
Subset Sheets:

Project No./Code
FC-NH(CX)160-2(048)
91050
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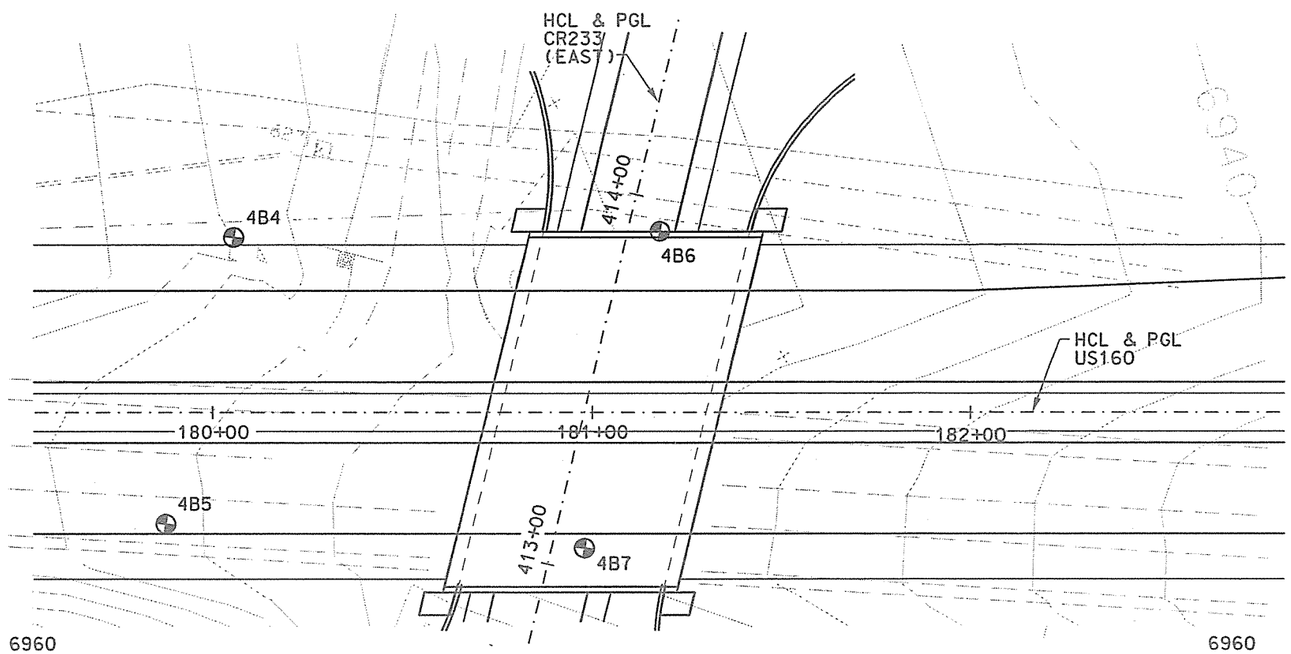

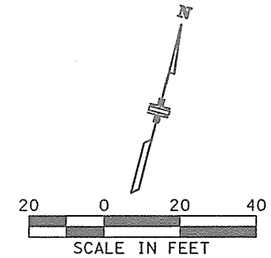

LEGEND

- TOPSOIL
- ASPHALT CONCRETE PAVEMENT
- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN.
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO VERY SILTY, STIFF TO VERY STIFF, LOW PLASTICITY , MOIST, DARK GREY TO BROWN (CL)
- SILT, CLAYEY TO VERY CLAYEY, SANDY TO VERY SANDY, ELASTIC, STIFF TO VERY STIFF, MOIST, MEDIUM BROWN (MH)
- SAND, SLIGHTLY CLAYEY TO CLAYEY, SILTY TO VERY SILTY, LOW PLASTICITY (FINES CONTENT), SLIGHTLY MOIST TO MOIST, LIGHT TO MEDIUM BROWN (SM)
- SILTSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)

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- APPROXIMATE BORING LOCATION

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

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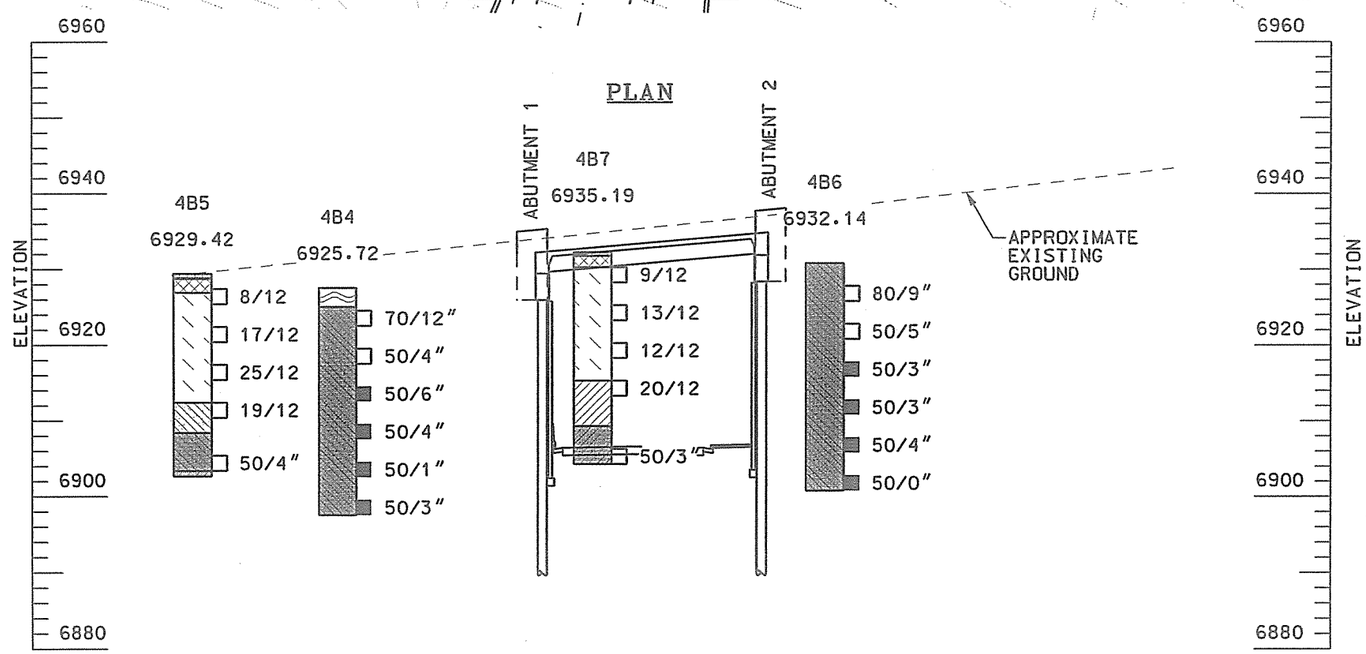

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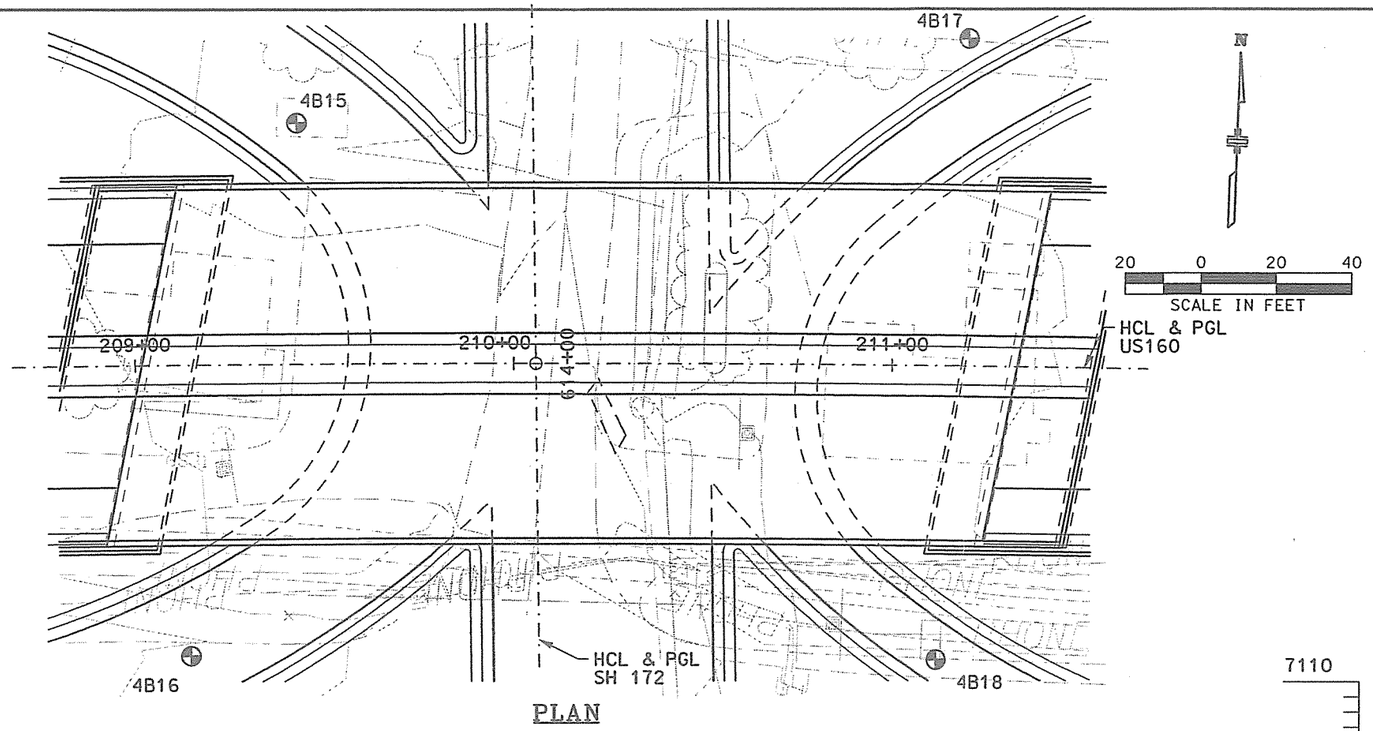

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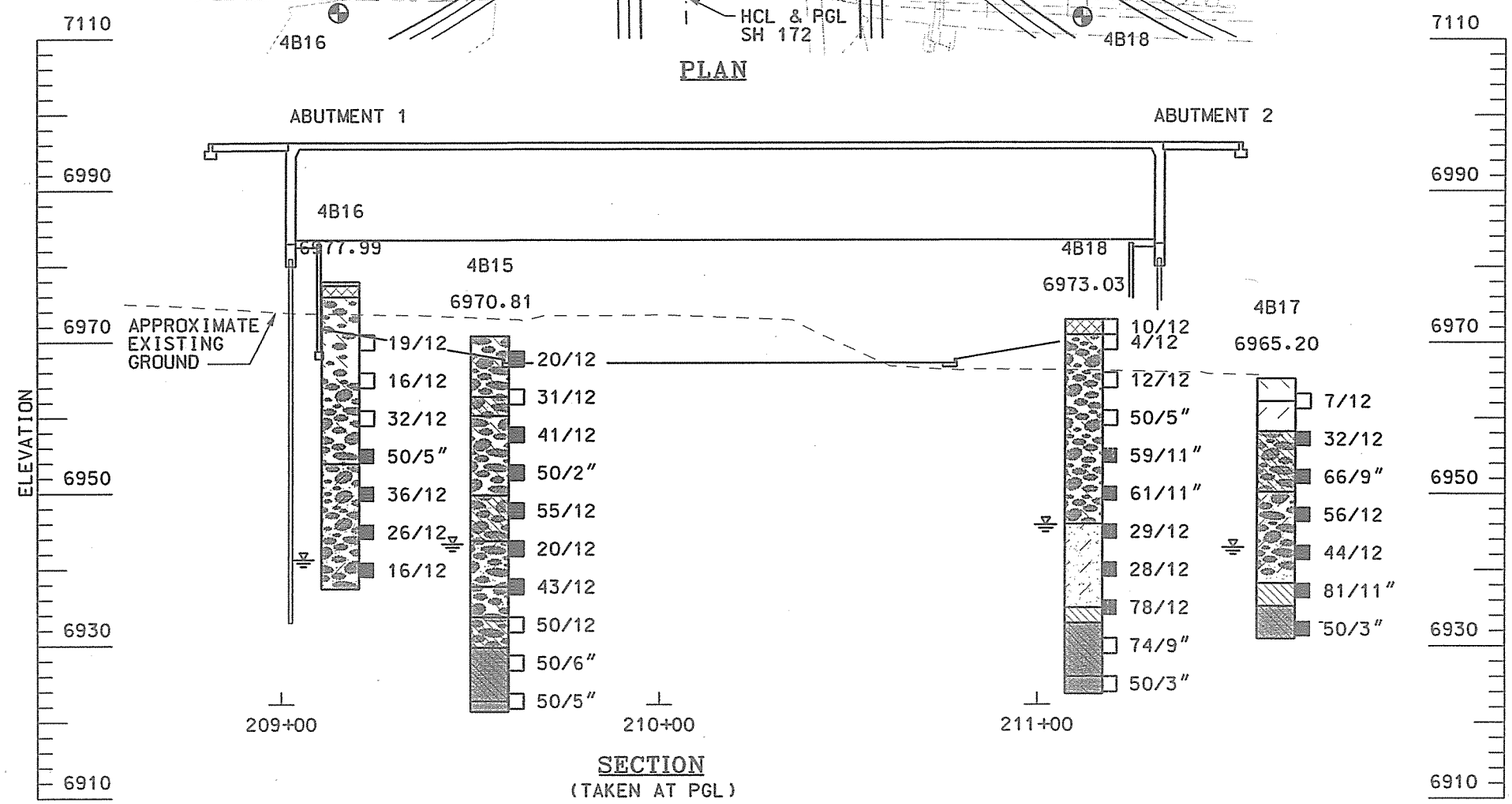

SECTION
(TAKEN AT PGL)

LEGEND

- ASPHALT CONCRETE PAVEMENT
- FILL: SAND, GRAVELLY, MEDIUM DENSE, MOIST, LIGHT BROWN TO BROWN.
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST, DARK GREY TO DARK BROWN (CL)
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SILTY TO VERY SILTY, MEDIUM STIFF TO STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY TO MEDIUM BROWN (CH)
- SILT, SLIGHTLY CLAYEY TO CLAYEY, SANDY TO VERY SANDY, STIFF TO VERY STIFF, LOW PLASTICITY, MOIST, YELLOW TO LIGHT BROWN (ML)
- SAND, CLAYEY TO VERY CLAYEY, SILTY, TRACE GRAVEL, MEDIUM DENSE TO VERY DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- SAND, SLIGHTLY CLAYEY TO CLAYEY, SILTY TO VERY SILTY, MEDIUM DENSE TO DENSE, LOW TO MODERATE PLASTICITY (FINES CONTENT), SLIGHTLY MOIST TO WET, LIGHT TO MEDIUM BROWN (SM)
- COBBLES AND BOULDERS
- CLAYSTONE, SILTY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT GREY (BEDROCK)
- SILTSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT BROWN (BEDROCK)

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- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

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US160 OVER SH172
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: R. PIKE
Structure Numbers: P-05-AF
SheetSubset:
Subset Sheets:

Project No./Code
FC-NH(CX)160-2(048)
91050
Sheet Number FIG. 2-10

GEOTECHNICAL INVESTIGATION REPORT
US HIGHWAY 160 4TH LANE
AT FARMINGTON HILL
Colorado Department of Transportation
Project No. NH 1602-114
Project Code 16042

November 27, 2007
Project No. 26-235

Prepared for:

Muller Engineering Company, Inc.
777 S. Wadsworth Boulevard Ste. 4-100
Lakewood, CO 80226
Attn: Mr. Rob Carlson, PE

Colorado Department of Transportation
Region 5, Durango Residency
3803 North Main Avenue, Suite 200
Durango, CO 81301
Attn: Mr. Ed Archuleta

Prepared by:

Yeh and Associates, Inc.
570 Turner Dr., Suite D
Durango, Colorado 81303
Phone: 970-382-9590
Fax: 970-382-9583

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1 INTRODUCTION

The purpose of this report is to provide geotechnical information for the design and construction of bridge structures, retaining walls, pavements and earthwork for the proposed US 160 4th Lane at Farmington Hill. The project is to be constructed using a "modified design-build" process administered by the Colorado Department of Transportation (CDOT). This report is intended to be a part of the project documents and references other project documents including plans and previous reports by others to identify and locate existing and proposed features including structures, roadways, embankments, topography and exploratory boring locations.

Significant features of the project include two off-ramps for the future US 160/US 550 Interchange and an additional westbound lane on US 160 at Farmington Hill. These proposed improvements have bridge structures, retaining walls and embankments associated with them. This report presents the results of a geotechnical investigation to evaluate subsurface conditions for the proposed construction as shown on the project plans. The investigation included review of previous Preliminary Geotechnical Reports by URS Corporation dated May 2005 and as-constructed drawings from CDOT Project C 1601-037 dated July 3, 1997 and evaluation of conditions encountered in 23 exploratory borings drilled by Yeh and Associates between December 2006 and April 2007. Discussions of the site geology and geotechnical considerations for design and construction of the major structures are included in this report. Logs of the exploratory borings and the results of laboratory testing by Yeh and Associates are included in the appendices.

2 PROPOSED CONSTRUCTION

2.1 Ramp A

This proposed ramp will be the eastbound off-ramp from US 160 to the future interchange with US 550. The ramp will be a single lane located roughly parallel to and south of the existing US 160 alignment beginning at the Farmington Hill intersection and extending about 2900 feet toward the northeast. Structure P-05-F will be a retaining wall supporting the cut slope along the south side of the ramp. The ramp will cross US 160 and the Wilson Gulch drainage on Bridge Structure P-05-V. Retaining walls P-05-AK and P-05-Q will be located at the south and north abutments, respectively. Embankments for the bridge approaches will be located north of Wilson Gulch and south of US 160. The location of Ramp A and associated structures is shown on the project plans.

2.1.1 Wall P-05-F

This is a retaining wall for support of the cut slope along the south side of Ramp A. The height of the wall ranges from less than 4 feet to a maximum of about 44 feet. The conceptual design for the wall shows a single tier 1365 feet long and 2, two-tiered sections with lengths of about 140 and 300 foot. The wall will have full-height pre-cast concrete facing meeting the aesthetic requirements of the project.

2.1.2 Bridge P-05-V

This is a 440-foot long multi-span bridge that will carry Ramp A traffic over US 160, Wilson Gulch and a future shared-use path at the north abutment. The project plans show a single pier in the Wilson Gulch channel. The approach embankments will have a maximum height of about 25 feet. Retaining Wall P-05-AK will wrap the face of the south abutment and extend to the southwest about 300 feet to support the fill slope downhill from Ramp A. The height of this wall ranges from about 8 feet at the west end to about 20 feet where it passes below the bridge. Retaining Wall P-05-Q will wrap the north abutment. Conceptual plans show the wall to be approximately 100 feet long and with a maximum height of about 25 feet. The future shared-use path will be located at the toe of the wall.

2.2 Ramp D

Ramp D is the westbound off-ramp from US 160 to the future US 160/US 550 Interchange. The ramp consists of Bridge Structure P-05-Y with approach embankments at the north and south ends and Retaining Walls P-05-V and P-05-W at the south and north abutments, respectively.

2.2.1 Bridge P-05-Y

The conceptual design for the bridge is a 216-foot long single span structure that carries Ramp D traffic over Wilson Gulch and a future shared-use path. The approach embankments will have maximum fill heights of about 30 feet. Design and construction of the bridge should avoid long term impacts to wetland areas in the Wilson Gulch channel. The location of Ramp D is shown on the project plans. The heights of the abutment walls are about 15 feet as shown on the conceptual plans.

2.3 US 160 4th Lane

Westbound US 160 will be widened from the existing single lane to two lanes. Construction of the 4th Lane will require placing fill along the Wilson Gulch channel northeast of the Farmington Hill intersection and excavating the toe of the slope northwest of the intersection. Retaining walls P-05-L and P-05-AJ will support the new fill on the north side of the highway. Wall P-05-K will support the cut slope northwest of the intersection. The project plans show the location of the 4th Lane widening and the retaining walls.

2.3.1 Wall P-05-L

The embankment for the additional westbound lane will be supported by retaining wall P-05-L. The wall will be located between the existing westbound lane of US 160 and the Wilson Gulch channel. The proposed wall is approximately 910 feet long and has a maximum height of about 30 feet. The design of this wall should consider potential scour in Wilson Gulch and should avoid permanent impacts to designated wetlands.

2.3.2 Wall P-05-AJ

This wall will be located north of the Farmington Hill intersection where Wilson Gulch drainage enters a concrete box culvert and flows beneath US 160. The height of this wall ranges from about 8 to 13 feet and the west portion of the wall is located above the box culvert. The wall is approximately 134 feet long.

2.3.3 Wall P-05-K

Widening of US 160/550 west of the Farmington Hill intersection will be accomplished by excavating a cut in the hillside along the north side of the highway. The cut will be supported by Wall P-05-K, a full-height pre-cast panel faced wall. The proposed wall will be approximately 860 feet long and will have a maximum height of about 15 feet. The wall design should avoid conflicts with the existing utilities near the toe and top of the wall.

Table of Proposed Structures

Structure Number	Route	Begin/End Stations	Structure Type	Structure Length (feet)
Wall P-05-F	US Highway 160, Ramp A	298+00/310+60	Ground Nail Wall	1260
Bridge P-05-V	US Highway 160, Ramp A	316+96/321+39	Multi-Span Bridge	443
Wall P-05-AK	US Highway 160	71+08/74+33	MSE Abutment Wall	325
Wall P-05-Q	US Highway 160, Ramp A	321+39	Abutment Wall	Contractor Designed
Bridge P-05-Y	US Highway 160, Ramp D	611+02/612+82	Single-Span Bridge	180
Wall P-05-V	US Highway 160, Ramp D	611+02	Abutment Wall	Contractor Designed
Wall P-05-W	US Highway 160, Ramp D	612+82	Abutment Wall	Contractor Designed
Wall P-05-L	US Highway 160	50+79/60+50	MSE Retaining Wall	971
Wall P-05-AJ	US Highway 160	48+33/49+67	MSE Retaining Wall	134
Wall P-05-K	US Highway 160	40+30/48+06	Ground Nail Wall	776

3 GEOLOGIC SETTING

The site is located in a tributary valley to the Animas River valley that was eroded through alluvium and bedrock by Wilson Gulch, a perennial stream that drains the broad upland valleys to the northeast. The valley is steep sided at the west end and broadens toward the east. The Wilson Gulch channel is deeply incised through most of the project area.

Surficial deposits consist of clay, silt and sand soils derived from the sedimentary bedrock and ancient gravel terrace alluvium deposited by the Animas River. At some locations the finer-grained soils have mixed with gravel alluvium that has eroded from the overlying terraces.

The exposed bedrock is primarily interlayered shale and sandstone of the Animas Formation. These dark, varicolored sandstones, shales and conglomerates contain abundant volcanic and arkosic detritus. The McDermott member, composed of interlayered reddish brown to purple

sandstone, breccia and conglomerate; is visible along the north side of the highway, just west of the stoplight at Farmington Hill.

A map showing the surficial geology of the site is provided in Appendix A.

4 GEOTECHNICAL INVESTIGATION

4.1 Subsurface Exploration

The subsurface exploration program consisted of drilling 22 borings at the locations shown on the plans. Yeh and Associates contracted Salisbury and Associates, Inc. to drill borings YA-01 through YA-09, YA-11, YA-12 and YA-14 through YA-17 using a portable GH-5 Viper drilling rig. The specialty drilling rig was required to access boring locations on the steep slopes above Ramp A and along the Wilson Gulch channel. The borings were advanced through the overburden soils and bedrock using BX (2 5/16-inch O.D.) wireline coring. DA Smith Drilling, Inc. was contracted to drill Borings YA-18, YA19 and YA 21 through YA 26 using a rubber track mounted Dietrich D-50 drilling rig. These borings were advanced with hollow stem auger to practical refusal and then into bedrock using HQ (2 ½ -inch) wireline coring.

Soil samples were obtained using Standard Penetration Tests (SPT) at pre-determined depths from the BX core and hollow stem auger borings. To perform the SPT, a standard 1.5-inch inside diameter split spoon sampler was seated at the bottom of the bore hole, then driven in three 6-inch intervals (or fraction thereof) with blows from an automatic standard hammer weighing 140 pounds and falling a distance of 30 inches. The number of blows required to drive the sampler the 2nd and 3rd 6-inch intervals or a fraction thereof, are summed to give the blow count (N-value) in inches per foot. The N-value, as described in ASTM Designation D1586, when properly evaluated, is an index to the consistency or relative density of the material tested. A modified form of the SPT test was also conducted in some of the borings to provide samples for additional testing in the laboratory. The modified procedure substitutes a 2-inch inside diameter drive barrel with brass liners, generally termed a California drive barrel, for the standard (SPT) sampler. The test procedure is the same as the SPT test and the resulting penetration values are used to estimate the relative density or consistency of the soil, similar to the SPT test.

The borings were logged by a representative of Yeh and Associates in general accordance with Federal Highway Administration (FHWA) guidelines. Rock core percent recovery, RQD, general drilling resistance and visual soil/rock classification were recorded in the field. The logs of the borings are shown in Appendix B. Recovered core and soil samples were returned to the laboratory for testing.

4.2 Laboratory testing

Samples of soil and rock were tested in the laboratory to determine the classification and engineering characteristics of the on-site soil and bedrock. Laboratory tests including soil classification, natural moisture content, natural dry density, pH, water-soluble sulfates, water-soluble chlorides, resistivity and R-value were performed to determine soil properties and support

characteristics for design of foundations, retaining structures and pavements. Swell/consolidation tests were performed on relatively undisturbed soil samples obtained from borings drilled near the proposed bridge approach embankments. The results of the laboratory testing are shown on the boring logs and in Appendix C.

Based on the laboratory test results, the finer grained clayey sand, sandy silt and sandy clay soils have AASHTO classifications of A-2-6, A-7-6 and A-6 with group indices ranging from 0 to 8. The shale bedrock, when crushed to a soil consistency is classified as A-6 with a group index of 5.

Five soil samples and one bedrock sample were tested for pH and water soluble sulfates. Three of these soil samples and the bedrock sample were also tested to determine laboratory resistivity. The test results show the pH of the samples ranges from 6.6 to 8.8. Water soluble sulfates content ranges from less than 0.001 to 0.021 percent. Resistivity values range from 914 to 2016 ohms-cm.

Hveem (R-value) tests were performed on bulk samples from Boring YA-18, located near the east end of Ramp A, and Boring YA-22, located at the north abutment of the Ramp D bridge. The test results indicate the sample from Ramp A has an R-value of 14 and the sample from Ramp D has an R-value less than 5.

Swell/consolidation tests were performed on a sample from Boring YA-21 at a depth of 5 feet and two samples from YA-22 at 5 and 10 feet. Consolidation of these samples ranged from about 1 percent to about 3 percent when wetted under a surcharge load of 1000 psf and as much as 7.5 percent under a surcharge load of 2000 psf.

5 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the locations of each of the major structures on the project are summarized in the following sections. Exploratory boring locations and graphical depictions of the subsurface materials are shown on the Wall Profile sheets and Bridge Engineering Geology sheets included in the plans.

5.1 Wall P-05-F

The subsurface conditions at Wall P-05-F were investigated by using a portable drilling rig to obtain BX cores from Borings YA-14, YA-15 and YA-17. Boring YA-18 was drilled using a rubber-track mounted drilling rig to obtain HQ cores. The subsurface conditions at these boring locations consist of about 2 to 6 feet of medium dense to dense clayey to gravelly sand and sandy clay with gravel and cobbles over soft to hard interlayered sandstone, shale and sandstone conglomerate bedrock. The bedrock has closely spaced fractures and weathered horizontal joints as indicated by the relatively low RQD for samples with high recovery. Groundwater was not encountered in these borings.

5.2 Wall P-05-AK

Boring YA-19 was drilled to determine the depth to bedrock on the alignment of Wall P-05-AK west of the south abutment of the Ramp A bridge. The subsoils encountered consist of 6 feet of very stiff sandy clay. Weathered to hard sandstone bedrock was encountered at 6 feet. Outcrops of sandstone and shale bedrock are visible in the cut slopes east and south of the wall alignment.

5.3 Structure P-05-V, Ramp A Bridge

Yeh and Associates Borings YA-02, YA-23 and YA-24 were drilled in the vicinity of the Ramp A bridge. Borings in this area drilled by URS include 2B16, 2B18, 2B20, and 2B22.

The soil encountered in Boring YA-02, drilled near the north abutment, consists of about 22 feet of medium dense clayey sand with gravel (and possibly cobbles and boulders) in the lower 7 feet. Sandstone bedrock was encountered at 22 feet to the maximum depth of YA-02, 42.2 feet. The sandstone is medium hard to hard and grades to hard conglomerate sandstone at a depth of 32 feet.

Boring YA-23 was drilled north of the north abutment of P-05-V in an area of proposed deep embankment fill. The soil at this location consists of about 11 feet of loose to medium dense silty sand over 9 feet of medium dense gravelly sand with silt. The soil contains cobbles (and possibly boulders) that increase in frequency with depth. YA-23 was drilled to a depth of 20 feet where practical auger refusal was encountered, most likely on a cobble or boulder.
Boring YA-24 is located about 25 feet north of the proposed location of the pier for P-05-V. Stiff clay with varying amounts of sand was encountered to a depth of 11 feet. Very hard sandstone bedrock was encountered below the clay soils to the bottom of the hole at 15 feet.

The subsurface conditions at P-05-V are described in the URS report. These conditions are similar to those encountered in the borings by Yeh and Associates.

5.4 Structure P-05-Y, Ramp D Bridge

Yeh and Associates drilled borings at each proposed abutment of the Ramp D Bridge. Boring YA-21 was located at the south abutment and encountered 28 feet of loose to medium dense, clayey sand interlayered with sandy clay lenses. Below 28 feet the soil consists of very dense, gravelly cobbles with sand. The gravelly soils are described on the boring log as wet, however, the depth to groundwater could not be accurately measured because water was used as drilling fluid for coring below 32 feet and the hole caved soon after completion. Sandstone bedrock with very soft and hard layers was encountered at 41 feet to 45 feet. The sandstone became medium hard at 45 feet and is interlayered with conglomerate below 65 feet to the bottom of YA-21 at 70 feet.

Subsurface conditions at the north abutment were investigated by drilling Boring YA-22. The soil encountered consists of 46 feet of very soft to stiff, sandy clay over about 2 feet of cobbles and gravel. Medium hard to hard sandstone bedrock was encountered below the cobble layer to the full depth of exploration, 79.5 feet. Groundwater was encountered at a depth of 20 feet at the time

of drilling. A standpipe piezometer was installed in the boring to allow long term monitoring of the groundwater level. Laboratory tests indicate that the clay overburden soil is moderately compressible under loads representative of the proposed embankment loads at this location.

5.5 Wall P-05-L

Subsurface conditions at the location of Wall P-05-L were evaluated by drilling borings YA-08, YA-09 and YA-26 and a review of as-constructed plans for the existing cast-in-place retaining wall between Stations 357+20 and 359+00. URS borings in the area are 1W12, 1P20 and 1P21 drilled in the existing roadway. Generally, the subsurface conditions in the borings by Yeh and Associates consist of about 16 to 40 feet of soft to medium stiff sandy clay and medium dense clayey sand with layers or zones of gravel and cobbles in a sandy clay matrix. Interlayered sandstone, shale and conglomerate sandstone bedrock was encountered in the borings below the sand and clay soil. Groundwater was encountered in Boring YA-09 at a depth of 24 feet.

5.6 Wall P-05-AJ

Boring YA-25 was drilled near the east end of Wall P-05-AJ. The soil encountered in the boring consists of about 5 feet of gravelly cobble fill over 33 feet of medium dense to dense gravel and cobbles in a sandy clay matrix. Medium hard conglomerate sandstone was encountered at a depth of 38 feet and extended to the maximum depth of the boring, 55 feet.

5.7 Wall P-05-K

Borings YA-11 and YA-12 were drilled at the edge of the La Plata county access road above the location of proposed Wall P-05-K. Soil in BoringYA-11 consists of about 6 feet of man-placed clayey, sandy gravel fill over medium dense to very dense clayey gravel with boulders and cobbles. Interlayered soft to hard sandstone and shale bedrock was encountered below the boulders and cobbles at a depth of about 15 feet. The bedrock transitioned to hard conglomerate sandstone at 33 feet to the maximum depth explored, 44.2 feet. About 2 feet of sandy gravel fill was encountered in Boring YA-12 at the surface. Below the fill, the bedrock consists of highly fractured shale to about 14 feet below the ground surface at which point, hard to very hard sandstone was encountered which extended to the bottom of the boring at 23.2 feet.

5.8 Miscellaneous Borings

Borings YA-01, YA-03, YA-04, YA-05, YA-06 and YA-07 were drilled at locations of structures or portions of structures that were deleted from the project early in the design process. A discussion of the conditions encountered in these borings is included for completeness. The soil in these borings consist of a variable mixture of sandy silt, clayey sand, silty gravel and sandy clay with zones of gravel, cobbles and boulders. These soil are typical for the area of the Wilson Gulch drainage. The soils have slightly basic pH, low water soluble sulfates content and resistivity less than 2000 ohms-cm.

With the exception of Boring YA-05, bedrock was encountered at depths ranging from about 7 feet to 30 feet. The bedrock consists of interlayered sandstone and shale, shale and conglomerate sandstone. Bedrock was not encountered in Boring YA-05 to the maximum depth of drilling, 60 feet. Groundwater levels in these borings could not be recorded at the time of drilling because water was used as a drilling fluid. The borings caved soon after drilling.

6 GEOTECHNICAL CONSIDERATIONS

6.1 Structure P-05-V

The foundations for support of the Ramp A bridge may consist of a combination of shallow spread footings and deep foundations such as drilled caissons or driven piles. Considering the subsurface conditions encountered in Boring YA-19 and reported for URS Borings 2B16 and 2B18, a spread footing foundation bearing on the dense alluvial gravels may be appropriate for the south abutment. Deep foundations are recommended for the pier and the north abutment due to the depth to bedrock and the poor foundation support characteristics of the overburden soils. The potential for flooding and scour should be considered when designing foundations near the Wilson Gulch channel. A mechanically stabilized earth (MSE) or cast-in-place cantilever retaining wall may be used for both abutments, however, at the north abutment ground improvement measures, as discussed below, should be used to reduce potential settlement.

The overburden soils at the north abutment are moderately compressible and will settle under the anticipated loading from the proposed embankment. The potential long term settlement of the embankment could be as much as 4 inches with no ground improvement. Settlement can be mitigated by several methods including:

1. Complete removal of the compressible soils and replacement with properly compacted embankment. This would entail removal of the soft, silty sand soils down to the gravelly sand soils at a depth of about 15 feet.

2. Excavate large diameter drilled shafts through the compressible soils to the deeper gravelly sand soils and backfill the shafts with compacted granular material. This is a proprietary ground improvement method and should be designed by the supplier.

3. Apply a surcharge load of deep fill to the approach embankment area for several months prior to roadway construction. The time required for the surcharge to remain in place will depend on the load imposed by the fill material and subsurface drainage conditions. The ground improvement should extend to the toes of the proposed embankment and to 5 feet beyond the abutment wall foundations. Settlement should be monitored from the time of initial construction of the preloading surcharge until most of the potential settlement has occurred. Refer to Book 2 Section 13 for allowable settlement criteria.

Slope stability should be considered when designing embankments at Ramp A. Refer to Book 2 Section 15 for Design and Performance Criteria.

6.2 Structure P-05-Y

Foundations for the Ramp D Bridge should consist of driven piles or drilled caissons bearing in the bedrock. The depth to bedrock is approximately 41 feet at the south abutment and 48 feet at the north abutment. Caisson drilling may be difficult in the dense alluvial gravels encountered approximately between 28 and 41 feet below the ground surface at the south abutment. Cobbles, boulders, groundwater and caving soils may be encountered in excavations at any depth. Casing may be required for drilled caisson construction. Design of these foundations should consider the potential for flooding and scour in the Wilson Gulch channel.

The clayey sand and sandy clay overburden soils encountered at both abutments are moderately to highly compressible. Long term settlements in excess of 2 feet could occur at the proposed approach embankments. Ground improvement methods such as described above for Ramp A should also be used at the Ramp D approach embankments and retaining walls. Under existing subsurface drainage conditions several years may be required for the majority of the settlement to occur under the proposed embankment loads. The time for settlement under pre-loading can be reduced to several months if subsurface drainage is improved by the installation of wick drains. Settlement should be monitored from the time of initial construction of the preloading surcharge until most of the potential settlement has occurred. Refer to Book 2 Section 13 for allowable settlement criteria.

An analysis was performed to evaluate the stability of the approach embankment at the north abutment of Ramp D. The analysis included the proposed embankment fill but did not include the influence of deep foundations or ground improvement. The results indicate the slope at the abutment will have a factor of safety less than 1.2 under the load imposed by the embankment. When the embankment, bridge foundations and ground improvements have been designed, a stability analysis should be performed to show that the design meets the slope stability requirements in Book 2 Section 15.

6.3 Wall P-05-AK

This wall will wrap the south abutment of Bridge P-05-V and support an embankment near the east end of Ramp A. The structure is an MSE wall bearing on very stiff sandy clay and/or sandstone/shale bedrock. The depth to bedrock is about 6 feet at the west end and is exposed at the surface on the east end. Excavation of the bedrock to construct the wall will require heavy duty equipment with rippers and rock teeth. Cobbles, boulders, groundwater, hard bedrock layers and caving soils may be encountered in foundation excavations at any depth.

6.4 Walls P-05-F and P-05-K

These walls will support deep cuts through shallow alluvial gravel overburden and shale/sandstone bedrock. Cast-in-place cantilever walls or shotcrete-faced ground nail walls are appropriate for both locations. MSE walls are not recommended because they would require additional excavation of the relatively hard bedrock. Ground nail walls have the advantage over cast-in-place

cantilever walls because they are constructed from the top down and temporary shoring to support gravel soils and fractured bedrock during construction would not be required. Excavation of the bedrock to construct the walls will require heavy duty equipment with rippers and rock teeth. Cobbles, boulders, groundwater, hard bedrock layers and caving soils may be encountered in excavations at any depth.

6.5 Walls P-05-L and P-05-AJ

Cast-in-place cantilever walls or MSE walls are appropriate options at these locations. Cantilever walls typically have relatively high foundation loads and deep foundations may be required to transfer the wall loads to the dense gravel alluvium or bedrock encountered at depths ranging from 8 to 40 feet below the ground surface. MSE wall designs effectively distribute foundation loads over a large area. The presence of clay soils in a portion of Wall P-05-L may result in long term settlement due to wall and backfill loads. In this area, ground improvement methods could include pre-loading of the soils as discussed above for Structure P-05-V and/or installation of vertical drains to increase the rate of consolidation. Construction of MSE walls will require excavation into the existing roadway to provide the necessary reinforcement embedment lengths. Cobbles, boulders, groundwater, hard bedrock layers and caving soils may be encountered in excavations at any depth.

The design of these walls should consider the potential for flooding and scour in the Wilson gulch channel. Where scour can undermine the toes of the walls, deep foundations such as drilled caissons or driven piles may be needed. Replacing the erodable foundation soils with riprap or rock fill to below the anticipated scour depth can reduce the potential for loss of foundation support in the event of flooding. The potential for scour damage may also be reduced by placing riprap at the toes of the walls or by re-directing flows away from the walls.

6.6 Pavements

R-value test results indicate the on-site clayey soils have relatively poor pavement support characteristics. Designing flexible or rigid pavement structures to be placed on a few feet of imported granular fill will mitigate the effects of the poor quality soils and allow for a consistent pavement section design throughout the project.

6.7 Corrosivity

Results of laboratory tests to determine corrosive potential of the soil and bedrock indicate these materials have near neutral to slightly basic pH, low water soluble sulfate content and relatively low resistivity. When evaluated using the criteria provided in the CDOT Field Materials Manual, which considers pH and sulfates, the corrosion potential of these materials is classified as low. However, because the samples have laboratory resistivity values less than 2000 ohms-cm, the corrosion potential for buried metal should be considered moderate and structures should be designed for Level 2 Corrosion Resistance in accordance with the CDOT Standard Specifications for Road and Bridge Construction.

6.8 Excavation

The soils and soft to medium hard bedrock at the site can probably be excavated with conventional heavy duty equipment. Hard to very hard bedrock, such as the sandstone conglomerate may require blasting at some locations.

Excavations may encroach on or endanger nearby facilities such as overhead utility poles and underground utilities. Temporary shoring or trench boxes may be needed to limit the effects of excavation on these facilities.

Excavations and drilling for caisson or ground nails may encounter cobbles, boulders or hard layers of bedrock at any depth. Temporary shoring or casing may be required to prevent caving of the excavations, borings or shafts in cohesionless soils, soft cohesive soils or where groundwater is present.

6.9 Site Grading

Site grading will be necessary to complete the earthwork around the abutments and for the approach embankments. We did not observe signs of major slope instability during our field investigation; however, small slump features have been mapped in the area. Stability problems are not anticipated if site grading is carefully designed and constructed. Permanent unretained cut and fill slopes in the project area should not be steeper than 2:1 (horizontal: vertical) and may need to be flattened in areas where the shale bedrock is present at the base of the slope. If steeper slopes are considered, a stability analysis should be performed to show that the factor of safety against slope failure meets the criteria in Book 2 Section 15. The risk of slope instability will be increased if seepage is encountered in cuts. If seepage is encountered in permanent excavations, an investigation should be conducted to determine if the seepage would adversely affect the cut stability. Additional drainage elements such as strip drains, piped outlets and/or horizontal drains may be necessary to contain the seepage

The ground surface underlying all fills should be carefully prepared by removing all organic material or other deleterious materials, scarifying to a depth of 12 inches and compacting to 95 percent of standard Proctor density at a moisture content within 2 percent of optimum.
Good surface drainage should be provided around all permanent cuts and fills to direct surface runoff away from the slope faces. Fill slopes, cut slopes and other stripped areas should be protected from erosion by revegetation or other methods.

All site grading should conform to the CDOT Standard Specifications for Road and Bridge Construction. Fill materials should not contain organic matter or other deleterious material. The granular native materials encountered are suitable for use in compacted fill. Sandstone bedrock material may be used for embankment fill provided it is well broken and placed in accordance with the Standard Specifications. The use of native clay soils and processed shale bedrock should be limited to non-critical areas of construction, such as outside of reinforced or active earth pressure zones for retaining walls and well below the pavement subgrade zone.

6.10 Seismic Considerations

United States Geological Survey Seismic Hazard Maps (2003) were used to estimate the peak horizontal acceleration due to earthquakes at the site. The maps indicate the earthquake peak ground acceleration (PGA) with a 10% probability of exceedance in 50 years is approximately 0.04 g.

7 LIMITATIONS

This report was prepared for the exclusive use of the Colorado Department of Transportation and their representatives for specific use on the US Highway 160, 4th Lane at Farmington Hill transportation project. The work was prepared in accordance with generally accepted geotechnical engineering practices in this area for use by the client for design purposes. No warranty expressed or implied, is made. The conclusions and recommendations made are based upon the data obtained and described in this report, including subsurface investigations. These investigations indicate the subsurface conditions only at the specific locations and to the depths specified when the data was obtained. This data may not necessarily reflect variations in the subsurface conditions and water levels occurring at other locations. Also variations in the data may occur with the passage of time. If variations in the subsurface conditions from those described in this report are discovered, then the recommendations contained in this report must be re-evaluated. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, this office should be advised at once so reevaluation of the recommendations may be made. We recommend on-site observation of excavations and foundation bearing strata by a representative of the geotechnical engineer.

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8 REFERENCES

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Colorado Department of Transportation, *Standard Specifications for Road and Bridge Construction*, 2005

American Society for Testing and Materials, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils, ASTM D1586-99*

APPENDIX A – SITE GEOLOGIC MAP

Qa	**Quaternary Alluvium: Unconsolidated silt, sand, and gravel deposited mainly in stream beds and on floodplains.**
Slump	**Indicates the location of a small, recently active slope failure feature**
Qls	**Quaternary Landslide Deposits: Unconsolidated clay, silt, sand, and gravel which has been displaced by slumping, usually water-induced**
Qg	**Quaternary Terrace Gravels: Pebbles, cobbles, and gravel deposited on benches bordering stream valleys.**
	"Qg OVERLYING Ka": Quaternary terrace gravels, 1-10 ft. thick, overlying Cretaceous Animas Formation rock.
Ka	**Cretaceous Animas Formation: Soft to medium hard, reddish brown and brown shale and claystone, moderately weathered, OR hard, resistant, gray sandstone, fresh to slightly weathered OR hard, resistant, gray, black, and pink conglomeratic sandstone, fresh to slightly weathered.**
Fill	**Artificial Fill: Unconsolidated silt and sand placed during previous construction activities**
	Water - Includes streams active at time of mapping, rivers, and reservoirs.
	Seeps or Springs - Only seeps or springs active during mapping are shown.
YA–12	**Test Hole Location**

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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: **CDOT Region 5**
Muller Engineering

Project Number: **27-031**

Drawn by: **MDN**

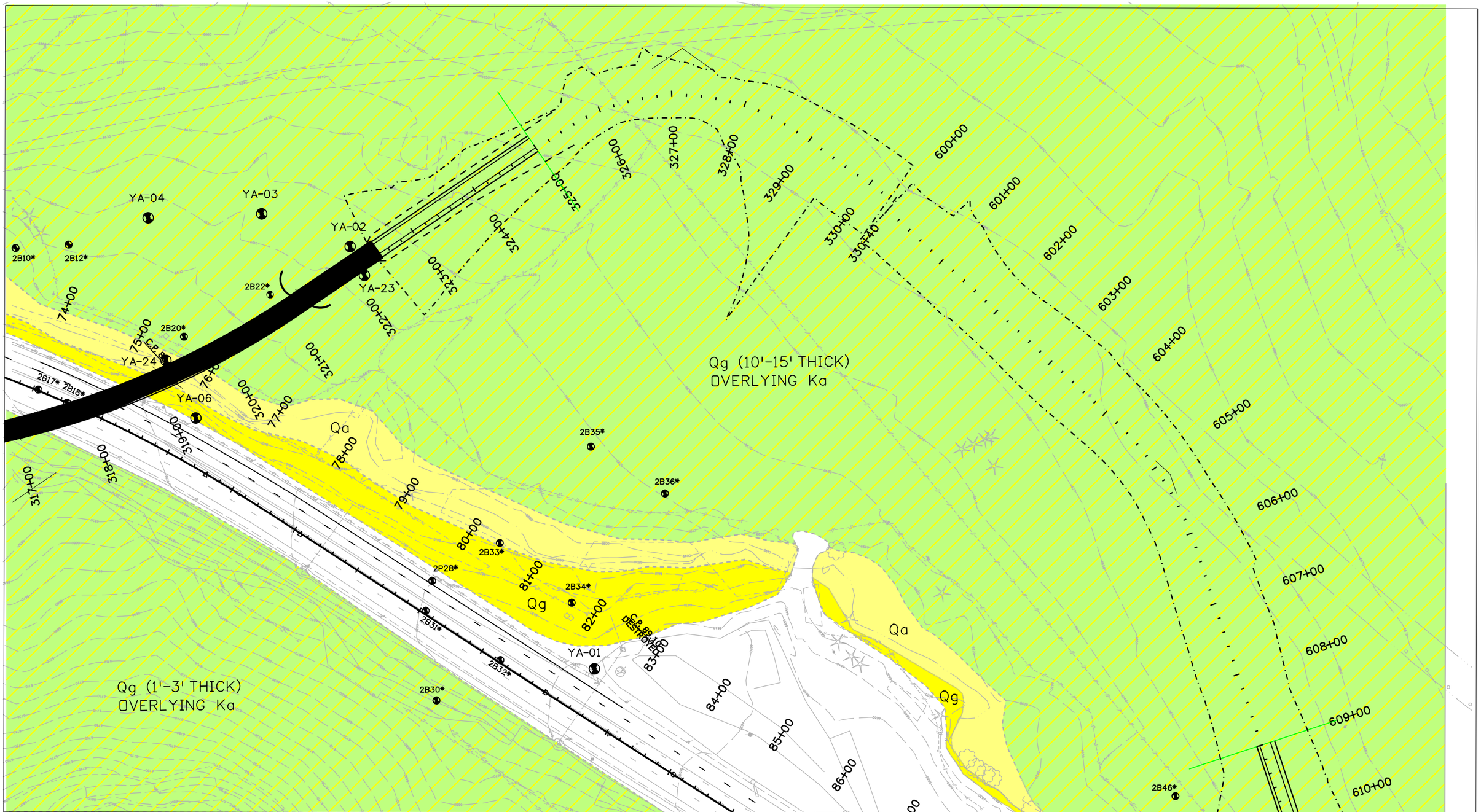
Checked by: **TLA**

Date: **July 2, 2007**

Geologic Map Legend

Project: **US 160 4th Lane**

Sheet Number: **1 of 5**



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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: **CDOT Region 5**
Muller Engineering

Project Number: **27-031**

Drawn by: **MDN**

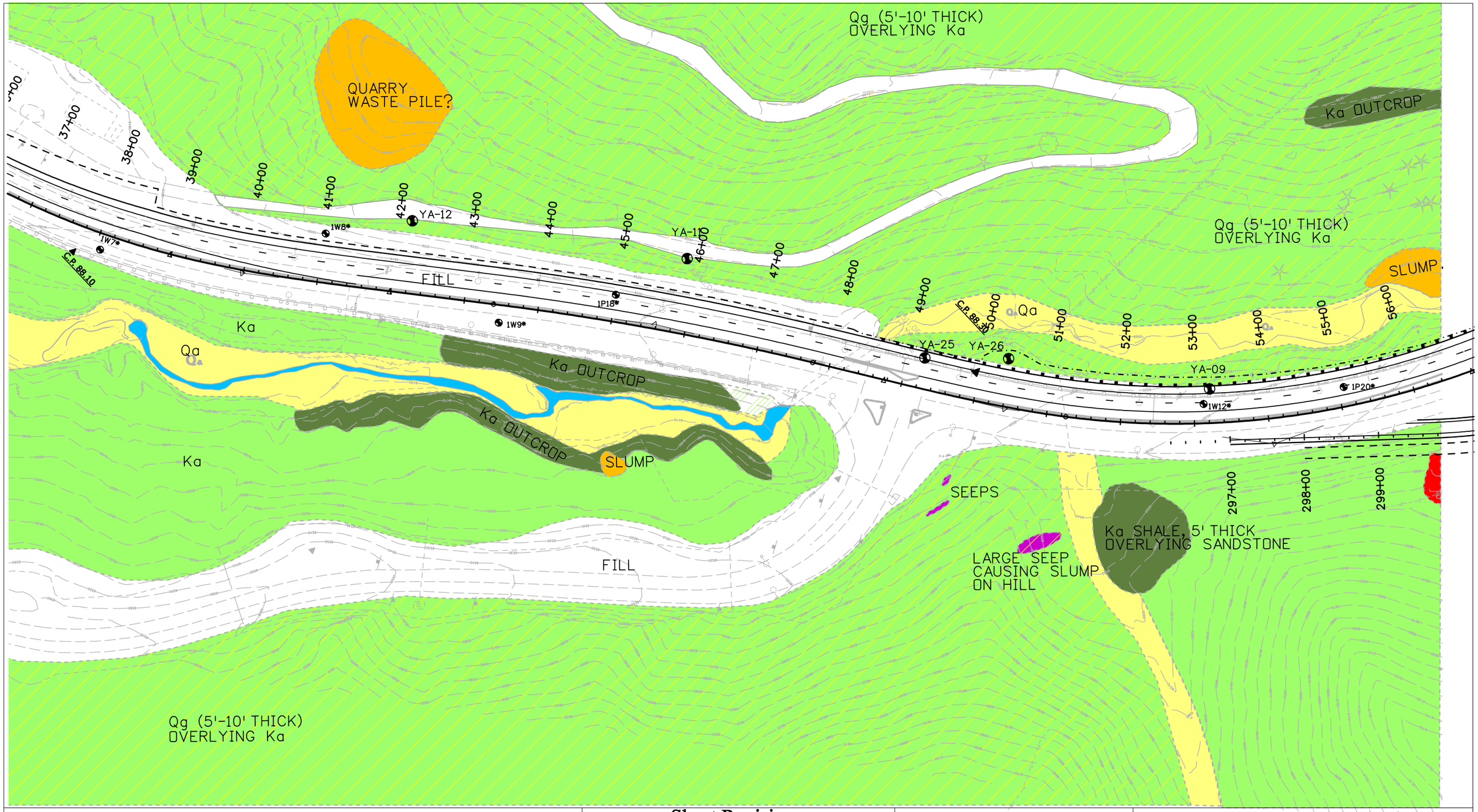
Checked by: **TLA**

Date: **July 2, 2007**

Geologic Map

Project: **US 160 4th Lane**

Sheet Number: **4 of 5**



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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: **CDOT Region 5**
Muller Engineering

Project Number: **27-031**

Drawn by: **MDN**

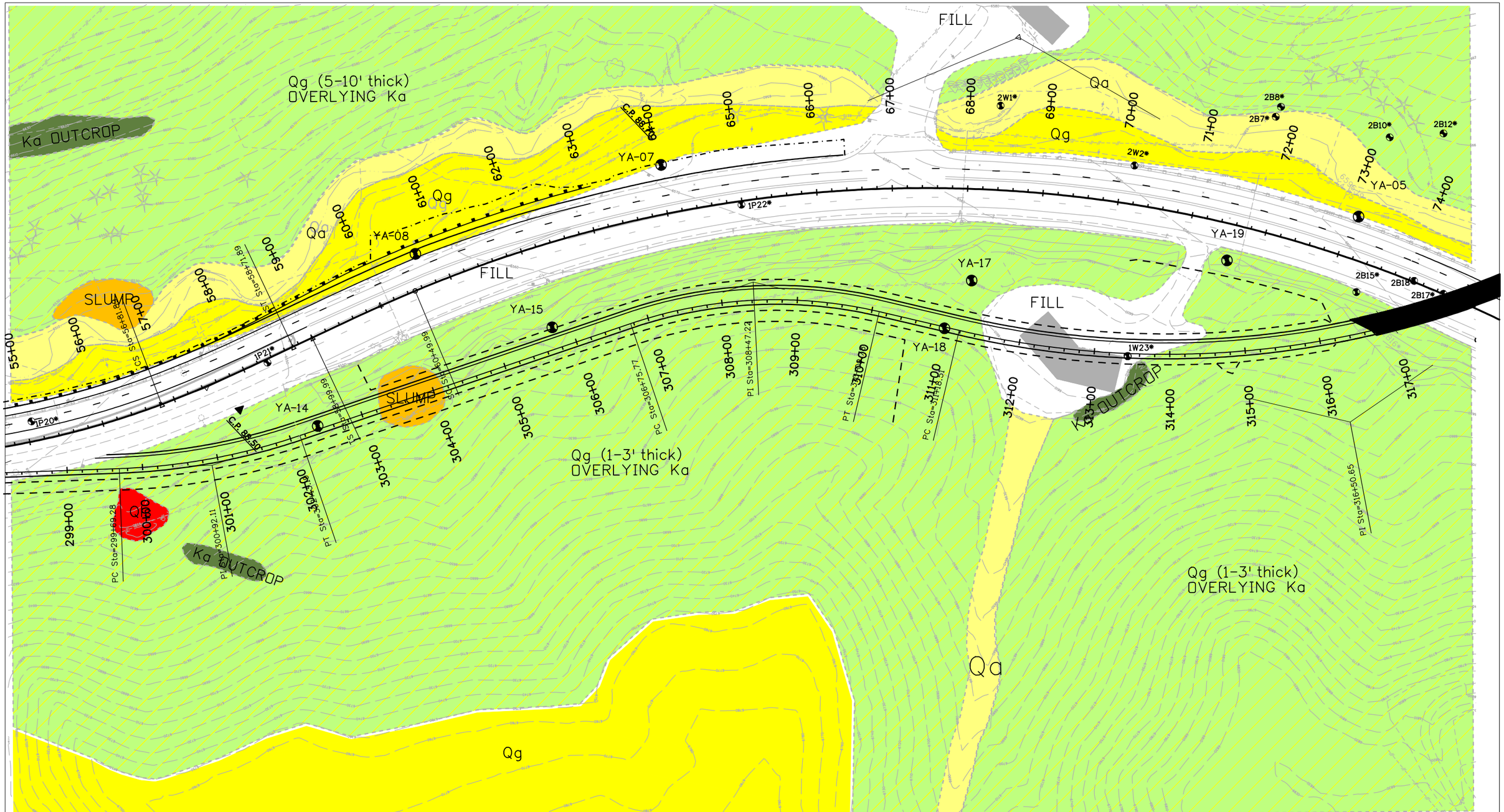
Checked by: **TLA**

Date: **July 2, 2007**

Geologic Map

Project: **US 160 4th Lane**

Sheet Number: **2 of 5**



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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: **CDOT Region 5**
Muller Engineering

Project Number: **27-031**

Drawn by: **MDN**

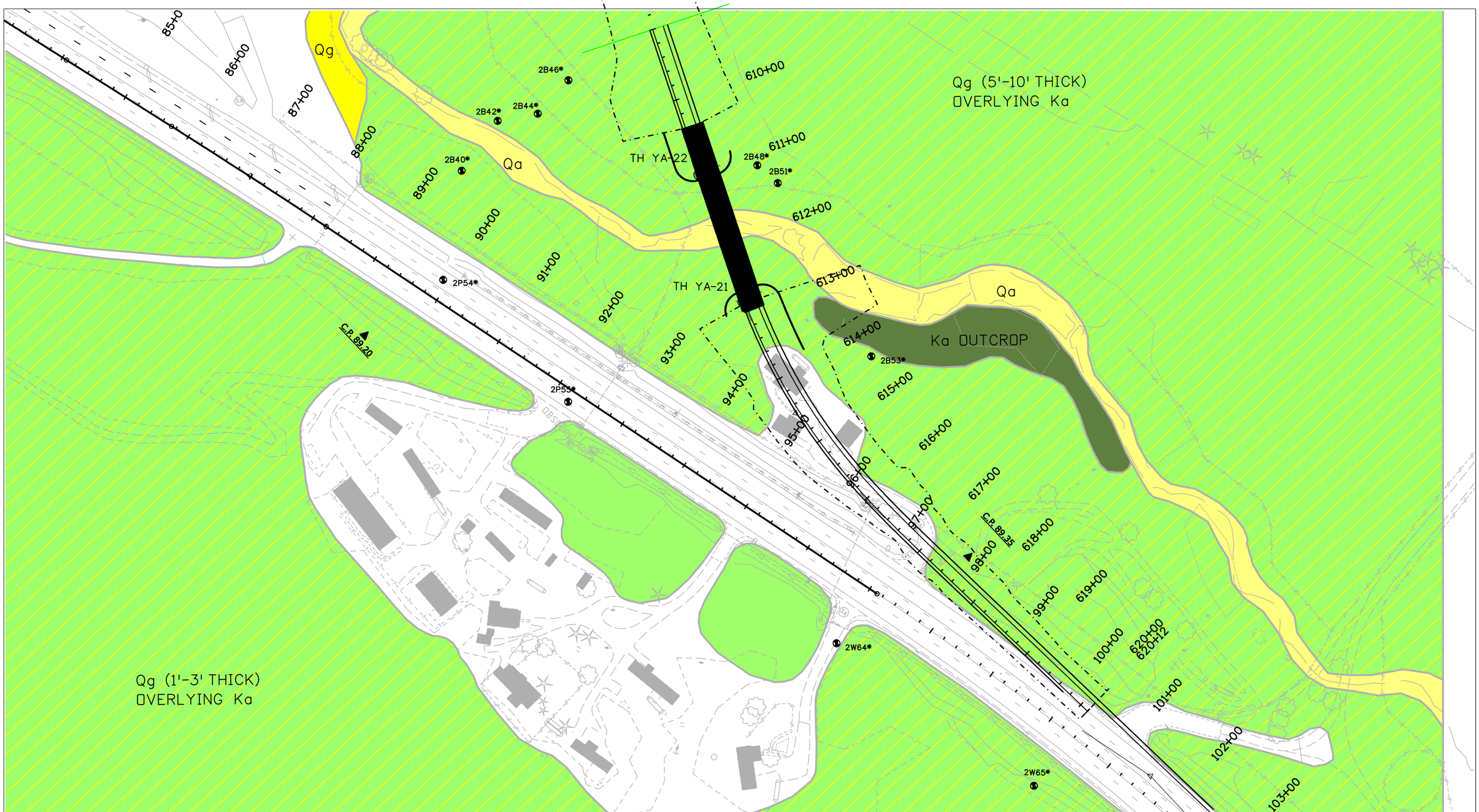
Checked by: **TLA**

Date: **July 2, 2007**

Geologic Map

Project: **US 160 4th Lane**

Sheet Number: **3 of 5**



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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: **CDOT Region 5**
Muller Engineering

Project Number: **27-031**

Drawn by: **MDN**

Checked by: **TLA**

Date: **July 2, 2007**

Geologic Map

Project: **US 160 4th Lane**

Sheet Number: **5 of 5**

APPENDIX B – LOGS OF EXPLORATORY BORINGS

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-01** Sheet 1 of 2

Boring Began: 12/5/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/5/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 35.0 ft
Ground Elevation: 6641.4 ft
Location: US160 Sta. 80+60, 80 ft. left
Coordinates: N: 2,319,901.9 E: 1,213,252.8

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6640								**0.0 - 2.3 ft.** Topsoil.	Casing in; blew out material
			45					**2.3 - 3.4 ft. gravelly COBBLES** (fines washed out), no plasticity, rounded, varing lithology.	
	5				3/7/6	13		**3.4 - 12.5 ft. clayey SAND** with gravel and some cobbles, brown, medium plasticity, medium dense, subrounded to subangular, cobbles are rounded to subrounded.	
6635									
	10				10/9/5	14			MC= 23.3 % #200= 17 % LL= 34 PL= 20 PI= 14 pH= 8.8 S= 0.009 % AASHTO: A-2-6 (0) USCS: SC
6630			22						
			100					**12.5 - 23.5 ft. sandy CLAY** with gravel and some cobbles, brown, medium plasticity, medium stiff, gravel is subrounded to subangular.	
	15				2/2/4	6			MC= 30.8 % #200= 56 % LL= 41 PL= 22 PI= 19 AASHTO: A-7-6 (8) USCS: CL
6625			9						
	20				3/5/50:0"	50:0"			Refusal due to boulder
6620									
	25		100	58				**23.5 - 35.0 ft. SANDSTONE W/SHALE LAYERS**, blue-gray and brown, slightly weathered, hard, parting close, open fractures, iron oxide stains, slightly rough surfaces.	
6615			88	0					

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-01**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6610			80	15					
	35							Bottom of Hole at 35.0 ft.	
6605									
	40								
6600									
	45								
6595									
	50								
6590									
	55								
6585									
	60								
6580									
	65								

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-02** Sheet 1 of 2

Boring Began: 11/30/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/1/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 42.2 ft
Ground Elevation: 6615.0 ft
Location: approx. Sta. 5+25, Wall Q
Coordinates: N: 2,319,270.0 E: 1,213,409.4

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 14.7 ft. clayey SAND, brown, medium plasticity, medium dense, subangular to subrounded.	
			81						
6610	5				6/8/11	19			MC= 20.3 % #200= 47 % LL= 33 PL= 21 PI= 12 AASHTO: A-6 (3) USCS: SC
			41						
6605	10				3/8/10	18			
			10						
			12						
6600	15				6/8/10	18		**14.7 - 22.0 ft. clayey SAND** with gravel, brown, medium plasticity, medium dense, subangular to subrounded.	MC= 15.5 % #200= 29 % LL= 30 PL= 19 PI= 11 AASHTO: A-2-6 (0) USCS: SC
			50						
			94						
6595	20				9/12/50:0"	50:0"			Refusal due to boulder
								22.0 - 42.2 ft. SANDSTONE W/SHALE LAYERS, gray - brown, slightly weathered, medium hard to hard, foliation , grades to fresh hard conglomeratic sandstone at 32'.	
6590	25		100	61					Lost circulation
			94	27					

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-02**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6580	35		100	88					
6575	40		90	33					
								Bottom of Hole at 42.2 ft.	
6570	45								
6565	50								
6560	55								
6555	60								
6550	65								

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-03** Sheet 1 of 2

Boring Began: 12/1/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/1/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 45.0 ft
Ground Elevation: 6615.4 ft
Location: approx. Sta. 3+50, Wall Q
Coordinates: N: 2,319,149.0 E: 1,213,341.6

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6615								**0.0 - 16.8 ft. clayey SAND** with gravel and few cobbles, brown, low plasticity, loose to medium dense, subrounded to subangular.	Casing in - no material
			33						
6610	5				3/5/6	11			
6605	10				11/7/9	16			
			71						
			56						
6600	15				7/11/8	19			MC= 17.6 % #200= 29 % LL= 30 PL= 19 PI= 11 pH= 7.2 S= 0.001 % Re= 1664 ohms-cm AASHTO: A-2-6 (0) USCS: SC
			91						
			43					**16.8 - 29.5 ft. GRAVEL** with silt, brown, medium dense, subrounded to subangular, fines mostly washed away.	
6595	20				11/50:5"	50:5"			
			73						
			46						
6590	25		100						
			39						
			86						
								29.5 - 45.0 ft. SANDSTONE W/SHALE LAYERS,	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-03**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6585			100					blue-gray and brown, medium hard to hard.	
			84	22					
6580	35								
			96	10					
6575	40								
			82	41					
6570	45							Bottom of Hole at 45.0 ft.	
6565	50								
6560	55								
6555	60								
6550	65								

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-04** Sheet 1 of 2

Boring Began: 12/2/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/3/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 44.2 ft
Ground Elevation: 6605.2 ft
Location: approx. Sta. 2+00, Wall Q
Coordinates: N: 2,319,049.9 E: 1,213,225.2

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	N	Lithology	Material Description	Field Notes and Lab Tests
6605			15					**0.0 - 16.7 ft. silty GRAVEL** with few cobbles up to 8" diam., brown, no plasticity, medium dense to dense, subangular to subrounded.	
			73						
6600	5				12/12/31	43			
			100						
			55						
6595	10				8/9/15	24			
			22						
			38						
6590	15		28						
			18					**16.7 - 28.8 ft. clayey SAND** with gravel and a few cobbles, brown, medium plasticity, medium dense, subangular to subrounded.	
			88						
6585	20				6/11/9	20			MC= 24.5 % #200= 31 % LL= 40 PL= 21 PI= 19 AASHTO: A-2-6 (1) USCS: SC
			50						
			35						
			56						No drive at 24.2'; driller believed hole would collapse
6580	25		81						
			29						
			89						
			100					**28.8 - 44.2 ft. CONGLOMERATIC SANDSTONE**, blue-gray, fresh, hard, clasts up to 5".	
6575	30		100	63					

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-04**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			100	76					
6570	35		100	79					
			95	68					
6565	40		100	80					
								Bottom of Hole at 44.2 ft.	
6560	45								
6555	50								
6550	55								
6545	60								
6540	65								

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-05** Sheet 1 of 2

Boring Began: 12/7/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Deuto
Final By: S. McKay
Inclination: Vertical

Completed: 12/7/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 60.0 ft
Ground Elevation: 6596.2 ft
Location: US160 Sta. 73+00, 80 ft. left
Coordinates: N: 2,319,006.1 E: 1,212,965.4

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6595			0					**0.0 - 8.6 ft. clayey SAND** with gravel and a few cobbles, brown, medium to high plasticity, loose to medium dense, subrounded to subangular, cobbles are subrounded to rounded.	MC= 22.4 % #200= 43 % LL= 40 PL= 19 PI= 21 AASHTO: A-6 (5) USCS: SC
			50						
			19						
	5								
6590					5/6/10	16			
			50						
			95					**8.6 - 13.6 ft. clayey SAND** with silt and gravel, few cobbles, brown, low to medium plasticity, medium dense, subrounded to subangular, lensed, cobbles subrounded. Also some organic material & rust staining.	
	10								
6585					10/13/18	31			
			69						
			21					**13.6 - 17.0 ft. CLAY** with gravel, gray mottled with brown, medium to high plasticity, stiff, gravel consists of pieces of shale.	
	15								
6580					3/8/7	15			
			94					**17.0 - 35.5 ft. clayey SAND** with gravel and few cobbles, brown, low to medium plasticity, very loose, subrounded to subangular.	
			63						
	20								
6575			28						
			14						
	25								
6570					1/1/1	2			
			0						

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-05** Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6565					2/2/2	4			
			0						
	35								
6560					2/2/3	5		**35.5 - 40.1 ft. clayey SAND**, gray to black, medium plasticity, very loose.	MC= 29.3 % #200= 42 % LL= 37 PL= 24 PI= 13 pH= 8.2 S= 0.021 % AASHTO: A-6 (2) USCS: SC
			26						
	40							**40.1 - 60.0 ft. clayey SAND** with gravel and cobbles, gray to black, low plasticity, medium dense, subrounded to subangular, cobbles are subrounded to rounded. Fines (sand+clay) washed out in some runs.	MC= 20 % #200= 14 % LL= 29 PL= 19 PI= 10 AASHTO: A-2-4 (0) USCS: SC
6555					7/13/21	34			
			29						
			33						
			100						
	45								
6550			20						
			71						
			67						
			100						
	50								
6545			18						
	55		14						
6540									
			50						
	60							Bottom of Hole at 60.0 ft.	
6535									
	65								
6530									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-06** Sheet 1 of 1

Boring Began: 12/6/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/6/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 24.2 ft
Ground Elevation: 6611.4 ft
Location: US160 Sta. 76+10, 40 ft. left
Coordinates: N: 2,319,290.3 E: 1,213,089.4

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6610			29					**0.0 - 6.7 ft. sandy SILT** with gravel and cobbles, brown, medium plasticity, hard, gravel is subangular, cobbles are subrounded to rounded.	
			88						
	5				19/22/20	42			MC= 23.6 % #200= 56 % LL= 44 PL= 28 PI= 16 AASHTO: A-7-6 (7) USCS: ML
6605								**6.7 - 14.2 ft. WEATHERED CLAYSTONE**, brown to black, moderately weathered, soft to medium hard.	
	10				43/50:3"	50:3"			
6600			100	11					
	15							**14.2 - 24.2 ft. CONGLOMERATIC SANDSTONE**, blue-gray, fresh, medium hard to hard.	
6595			90	45					
	20								
6590			98	46					
	25							Bottom of Hole at 24.2 ft.	
6585									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-07** Sheet 1 of 1

Boring Began: 12/8/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/8/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 31.1 ft
Ground Elevation: 6568.7 ft
Location: US160 Sta. 73+20, 70 ft. left
Coordinates: N: 2,318,321.5 E: 1,212,341.5

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			35					**0.0 - 14.7 ft. silty SAND** with layers of CLAY, with gravel and a few cobbles, brown, low plasticity, dense, subangular to subrounded, cobbles are subrounded to rounded.	
			40						
6565			62						
	5		100						
					10/20/20	40			MC= 16.5 % #200= 18 % LL= 32 PL= 20 PI= 12 AASHTO: A-6 (4) USCS: CL
			100						
			88						
			77						
6560									
	10		92						
					29/29/29	58			
			43						
6555			100						
	15		100					**14.7 - 16.7 ft. SANDSTONE W/SHALE LAYERS**, brown, slightly weathered, medium hard, parting no infilling, smooth surfaces.	
6550			100	31				**16.7 - 31.1 ft. CONGLOMERATIC SANDSTONE**, blue-gray, fresh, hard, parting no infilling, smooth surfaces, large clasts. Thin lens of sand/mud at ~28'.	
	20								
6545			94	63					
	25								
			100						
6540			100	64					
	30								

Bottom of Hole at 31.1 ft.

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-08** Sheet 1 of 1

Boring Began: 12/11/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/11/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 26.7 ft
Ground Elevation: 6556.3 ft
Location: US160 Sta. 60+30, 30 ft. left
Coordinates: N: 2,318,184.1 E: 1,212,023.2

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6555			0					**0.0 - 16.0 ft. sandy CLAY** with gravel and cobbles, brown, medium plasticity, stiff, sand and gravel are subangular to subrounded.	
			33						
	5				9/11/6	17			
6550			92						
	10				7/9/8	17			MC= 26.8 % #200= 58 % LL= 37 PL= 20 PI= 17 AASHTO: A-6 (7) USCS: CL
6545			80						
			100						
	15				18/22/50:3"	50:3"			
6540			100					**16.0 - 21.6 ft. SANDSTONE W/SHALE LAYERS**, brown-gray, moderately weathered, medium hard, parting iron oxide stains, slightly rough surfaces.	
			100	15					
	20		100	12					
6535								**21.6 - 26.7 ft. CONGLOMERATIC SANDSTONE**, blue-gray, fresh, hard, parting no infilling, smooth surfaces, medium-coarse clasts.	
			100	10					
	25								
6530								Bottom of Hole at 26.7 ft.	

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-09** Sheet 1 of 2

Boring Began: 12/15/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/15/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 45.3 ft
Ground Elevation: 6527.0 ft
Location: approx. Sta. 353+00, Wall L
Coordinates: N: 2,317,873.9 E: 1,211,367.9

Ground Water Notes:

Depth	∇ 24.0 ft	-	-	-
Date	2/6/07	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6525								**0.0 - 4.2 ft. sandy GRAVEL** with some organic material.	1" Diameter PVC slotted pipe installed to 20'.
	5				4/5/5	10		**4.2 - 36.5 ft. CLAY** with gravel and cobbles, brown-gray, medium plasticity, soft to medium stiff, cobbles are rounded to subrounded.	MC= 31.6 % #200= 51 % LL= 42 PL= 22 PI= 20 AASHTO: A-7-6 (7) USCS: CL
6520			71						
	10				2/3/4	7			
6515			43						lost circulation
	15				3/4/5	9			MC= 32.9 % #200= 53 % LL= 42 PL= 23 PI= 19 AASHTO: A-7-6 (7) USCS: CL
6510									
	20				3/2/3	5			
6505			32						
			20						
	25				3/2/3	5			
6500			40						
			32						

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-09**

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6495			24						
			43						
	35		100						
			25						
6490								**36.5 - 39.6 ft. sandy GRAVEL** with cobbles, boulder from 38.6-39.6'.	
			92						
			100						blocked circulation
	40		100	0				**39.6 - 45.3 ft. SANDSTONE**, hard, very fractured, jointed.	poor circulation
6485			100	20					
	45		100	0					
								Bottom of Hole at 45.3 ft.	
6480									
	50								
6475									
	55								
6470									
	60								
6465									
	65								

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-11**
Sheet 1 of 2

Boring Began: 12/16/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: T. Allen
Final By: S. McKay
Inclination: Vertical

Completed: 12/16/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 44.2 ft
Ground Elevation: 6546.1 ft
Location: approx. Sta. 145+90, Wall K
Coordinates: N: 2,317,263.5 E: 1,210,974.8

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6545								**0.0 - 6.0 ft.** FILL: clayey, sandy GRAVEL with scattered cobbles, brown, loose.	
			63						
			38						
	5		43						
6540								**6.0 - 14.7 ft. clayey GRAVEL** sandy, with cobbles, boulders, brown to reddish brown, medium dense to very dense, boulders/cobbles consist of granite, sandstone pieces.	
			43						
			58						
			58						
	10		33						
6535			79						
			31						
			80						
	15		63	0				**14.7 - 33.0 ft. SANDSTONE W/SHALE LAYERS**, gray to green, predominantly decomposed, soft to hard.	
6530			125	0					
			100	0					
	20								
6525			90	11					
	25								
6520			68	18					

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-11** Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6515			94	11					
								33.0 - 44.2 ft. CONGLOMERATIC SANDSTONE, blue-gray, fresh, hard.	
	35								
6510			98	80					
	40								
6505			100	90					
								Bottom of Hole at 44.2 ft.	
	45								
6500									
	50								
6495									
	55								
6490									
	60								
6485									
	65								
6480									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-12** Sheet 1 of 1

Boring Began: 12/17/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: T. Allen
Final By: S. McKay
Inclination: Vertical

Completed: 12/17/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 23.4 ft
Ground Elevation: 6527.4 ft
Location: approx. Sta. 142+10, Wall K
Coordinates: N: 2,316,973.4 E: 1,210,739.8

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 2.1 ft. FILL.	
6525			0					**2.1 - 14.4 ft. CLAYSTONE**, brown, predominantly decomposed, highly fractured.	
			100	0					
	5		79	0					MC= 8.7 % #200= 53 % LL= 39 PL= 24 PI= 15 pH= 7.2 S= 0.001 % Re= 914 ohms-cm AASHTO: A-6 (5) USCS: CL
6520			74	0					
	10		100	0					
6515									
	15		100	30				**14.4 - 23.4 ft. SANDSTONE**, blue-gray, hard to very hard, vertical joints.	
6510									
	20		100	70					
6505									
								Bottom of Hole at 23.4 ft.	
	25								
6500									

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-14**
Sheet 1 of 1

Boring Began: 12/13/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/13/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 21.7 ft
Ground Elevation: 6568.6 ft
Location: approx. Sta. 402+50, Wall F
Coordinates: N: 2,318,250.8 E: 1,211,785.6

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			28					**0.0 - 2.5 ft. clayey SAND**, brown, low plasticity, medium dense, subangular to subrounded.	
			100	0				**2.5 - 21.7 ft. SANDSTONE**, brown to reddish brown, slightly weathered, medium hard to hard, parting slightly rough surfaces.	
			100	0					
6565	5		100	15					
6560	10		96	0					
6555	15		100	0					
6550	20		76	0					
			100	0					
								Bottom of Hole at 21.7 ft.	
6545	25								
6540									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-15** Sheet 1 of 1

Boring Began: 12/12/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/12/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 19.2 ft
Ground Elevation: 6580.3 ft
Location: approx. Sta. 405+90, Wall F
Coordinates: N: 2,318,379.0 E: 1,212,088.9

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6580			0					**0.0 - 6.2 ft. clayey SAND** with gravel and cobbles, brown, low plasticity, medium dense, cobbles are rounded to subrounded.	
6575	5				7/11/20	31			MC= 29.6 % #200= 29 % LL= 38 PL= 24 PI= 14 AASHTO: A-2-6 (0) USCS: SC
			100					**6.2 - 19.2 ft. SANDSTONE W/SHALE LAYERS**, brown, moderately weathered, medium hard, open fractures, slightly rough surfaces.	
			100	0					
6570	10		100	0					
			80	0					
6565	15		100	62					
								Bottom of Hole at 19.2 ft.	
6560	20								
6555	25								

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GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-17**
Sheet 1 of 1

Boring Began: 12/14/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/14/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 21.7 ft
Ground Elevation: 6599.1 ft
Location: Ramp A approx. Sta. 311+00, 80 ft. left
Coordinates: N: 2,318,699.1 E: 1,212,537.7

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			67					**0.0 - 1.8 ft. gravelly SAND**, brown, low plasticity, gravel consists of weathered shale pieces.	
			83						
			100	81				**1.8 - 21.7 ft. SHALE**, brown to gray, slightly weathered to moderately weathered, medium hard, open fractures, slightly rough surfaces.	
6595	5		100	0					
6590	10		100	0					
6585	15		100	0					
			90	0					
6580	20		100	22					
								Bottom of Hole at 21.7 ft.	
6575	25								
6570									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-18** Sheet 1 of 2

Boring Began: 3/27/2007
Completed: 3/27/2007
Total Depth: 35.0 ft
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill Bit: HQ
Ground Elevation: 6620.9 ft
Drill: Dietrich D-50 Rubber Track
Casing:
Location: approx. Sta. 410+20, Wall F
Driller: DA Smith - Roger
Weather: sunny and warm
Coordinates: N: 2,318,734.9 E: 1,212,462.7
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6620								**0.0 - 5.0 ft. sandy CLAY** with gravel, light brown, medium plasticity, dry, medium dense to dense.	MC= 8.2 % #200= 52 % LL= 36 PL= 19 PI= 17 pH= 6.6 S= 0.001 % R-Value= 14 Re= 2016 ohms-cm AASHTO: A-6 (6) USCS: CL
	5				36/50:5"	50:5"		**5.0 - 15.0 ft. SHALE**, light brown and gray, slightly weathered to moderately weathered, soft to medium hard, moderately to highly fractured. Poor to fair induration.	Begin HQ Coring at 6'
6615			65	18					
	10								
6610			72	24					Lost 50% of fluid return
	15							**15.0 - 35.0 ft. SANDSTONE INTERBEDDED WITH CONGLOMERATE**, brown and green-gray, fresh to slightly weathered, soft to hard, parting close, horizontal, open fractures, clay and iron oxide infilling, lens of black shale (coal?) at 25.5'.	
6605			96	12					
	20								Driller notes frequent loss of fluid return
6600			100	14					
	25								
6595			72	30					

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-18**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6590			56	51					Piece of core fell out of barel-recovery and RQD likely would have been higher
	35							Bottom of Hole at 35.0 ft.	
6585									
	40								
6580									
	45								
6575									
	50								
6570									
	55								
6565									
	60								
6560									
	65								
6555									

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-19**
Sheet 1 of 1

Boring Began: 3/27/2007
Drilling Method: Hollow-Stem Auger
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/27/2007
Drill Bit:
Casing:
Weather: sunny and warm

Total Depth: 6.6 ft
Ground Elevation: 6599.1 ft
Location: approx. Sta. 472+00, Wall AK
Coordinates: N: 2,318,699.1 E: 1,212,537.7

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 6.0 ft. sandy CLAY, brown, medium plasticity, dry to moist, very stiff, Soft in first 2'.	
6595									
	5				9/13	22			
					50:1"	50:1"		**6.0 - 6.6 ft. SANDSTONE**, no sample; based on driller's observations.	Driller notes it feels like sandstone or relatively soft rock
								Bottom of Hole at 6.6 ft.	
6590									
	10								
6585									
	15								
6580									
	20								
6575									
	25								
6570									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-21** Sheet 1 of 3

Boring Began: 3/31/2007
Completed: 3/31/2007
Total Depth: 70.0 ft
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill Bit: HQ
Ground Elevation: 6656.4 ft
Drill: Dietrich D-50 Rubber Track
Casing:
Location: Ramp D south abutment
Driller: DA Smith - Roger
Weather: sunny and cool
Coordinates: N: 2,320,935.6 E: 1,213,673.9
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6655								**0.0 - 28.0 ft. clayey SAND** with silt, lenses sandy clay, brown, low to medium plasticity, dry to wet, loose to medium dense, subangular.	
	5				9/8	17			MC= 14.5 % DD= 86.0 pcf S/C= -2.9 %
6650									
	10				7/6	13			
6645									
	15				7/8	15			
6640									
	20				5/6	11			
6635									
	25				2/4	6			
6630								**28.0 - 41.0 ft. gravelly COBBLES** with sand, brown and reddish brown, no plasticity, wet, very dense, angular, cobbles are pieces of sandstone.	

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-21** Sheet 2 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
					37/50:3"	50:3"			
6625									Begin HQ coring at 32'
			17	0					
	35								
6620			40	0					
	40								Hole is deflected near 25 or 30', coring steel sheared off. 12:00-1:30pm fishing. Recovered.
6615			34	8				**41.0 - 45.0 ft. SANDSTONE**, gray, fresh to predominantly decomposed, medium hard to very soft, parting close to very close, horizontal, iron oxide stains.	
	45							**45.0 - 65.0 ft. SANDSTONE**, gray, fresh, medium hard, parting close, horizontal, clay and iron oxide infilling.	
6610			90	64					
	50								
6605			84	71					
	55								
6600			100	100					
	60								
6595			100	60					
	65							**65.0 - 70.0 ft. SANDSTONE INTERBEDDED WITH CONGLOMERATE**, gray, fresh, medium	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-21**

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			90	54				hard, parting close, horizontal, clay and iron oxide infilling.	
	70							Bottom of Hole at 70.0 ft.	Coring steel & coring barrel sheared off again. Caved in above top of sheared section, not recovered.
6585									
	75								
6580									
	80								
6575									
	85								
6570									
	90								
6565									
	95								
6560									
	100								
6555									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-22** Sheet 1 of 3

Boring Began: 3/27/2007
Completed: 3/28/2007
Total Depth: 79.5 ft
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill Bit: HQ
Ground Elevation: 6650.9 ft
Casing:
Location: Ramp D north abutment
Drill: Dietrich D-50 Rubber Track
Weather: snowy and cold
Coordinates: N: 2,320,771.3 E: 1,213,747.3
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Ground Water Notes:

Depth	∇ 20.0 ft	-	-	-
Date	3/27/07	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6650								**0.0 - 46.0 ft. sandy CLAY** interbedded with CLAY, brown with green-gray, medium to high plasticity, moist to wet, very soft to stiff.	MC= 10.7 % #200= 67 % LL= 52 PL= 23 PI= 29 pH= 7.8 S= 0.001 % R-Value= 5 Re= 1189 ohms-cm AASHTO: A-7-6 (18) USCS: CH
	5				8/6	14			MC= 19.8 % DD= 81.5 pcf S/C= -0.8 %
6645									
	10				7/7	14			MC= 17.6 % DD= 85.4 pcf S/C= -1.2 %
6640									
	15				4/5	9			
6635									
	∇ 20				2/3	5			
6630									
	25				2/3	5			
6625									Top of screened interval at 28'

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-22** Sheet 2 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6620			0		1/0	1			
	35				0/0	0			Sampler advanced without requiring blows-drill pressure only.
6615									
	40		0		2/3	5			
6610									
	45		0		4/27	31			
6605								**46.0 - 48.0 ft. COBBLES**, based on driller's observations only. No sample.	Driller notes it feels like large cobbles or similar
								48.0 - 79.5 ft. SANDSTONE, gray, fresh, medium hard to hard, parting moderately close to close, horizontal, iron oxide stains.	Bottom of screened interval at 48' Begin HQ Coring at 50'
	50								
6600			60	0					
	55								
6595			90	48					
	60								
6590			100	90					
	65								1.5' of core fell out of barrel-recovered in next run
6585									

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-22**
Sheet 3 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			100	66					
	70		100	90					This core run came out almost a 5' stick of core
6580			60	60					
	75		100	90					
6575									
	80							Bottom of Hole at 79.5 ft.	Fractured at bottom because it was drilled twice to make sure we got it.
6570									
	85								
6565									
	90								
6560									
	95								
6555									
	100								
6550									

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-23**
Sheet 1 of 1

Boring Began: 3/28/2007
Drilling Method: Hollow-Stem Auger
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/28/2007
Drill Bit:
Casing:
Weather: snowy and cold

Total Depth: 20.0 ft
Ground Elevation: 6612.5 ft
Location: Ramp A approx. Sta. 324+00
Coordinates: N: 2,319,301.7 E: 1,213,383.8

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6610								**0.0 - 11.0 ft. silty SAND** with many small layers of gravel, brown, no to low plasticity, dry to moist, loose to medium dense, Cobbles especially from 3-4'.	
	5				5/5	10			MC= 12.3 %
6605									
	10				8/11	19			
6600								**11.0 - 20.0 ft. gravelly SAND** with silt, many cobble layers (3-5" diameter?), brown, no plasticity, dry, medium dense.	
	15				11/12	23			
6595									
	20				50:0"	50:0"		**20.0 - 20.0 ft.** Bedrock (?). Driller thinks we're on a boulder, however could easily be bedrock at this depth. Auger refusal at 20'. Sample had no recovery.	
6590								Bottom of Hole at 20.0 ft.	
	25								
6585									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-24** Sheet 1 of 1

Boring Began: 3/29/2007
Drilling Method: Hollow-Stem Auger
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/29/2007
Drill Bit:
Casing:
Weather: sunny and cold

Total Depth: 15.0 ft
Ground Elevation: 6589.1 ft
Location: Ramp A, 25' N of Pier 2
Coordinates: N: 2,319,204.4 E: 1,213,110.7

Ground Water Notes:

Depth	∇ 1.0 ft	-	-	-
Date	3/29/07	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 11.0 ft. CLAY grades to sandy clay near 10', dark brown, medium to high plasticity, wet, stiff.	
6585									
	5				1/9	10			
6580									
	10				4/50:2"	50:2"			California sampler had pieces of gray sandstone in the shoe-reason for high blows at the end.
								11.0 - 15.0 ft. SANDSTONE, gray, based on observation of auger cuttings; no sample or core was taken.	
6575									
	15				50:0"	50:0"		Bottom of Hole at 15.0 ft.	Auger cuttings 11-15 are gravel-sized pieces of gray sandstone. Drill broke down before coring begun. Unable to core due to repairs, and loss of coring barrel on hole #YA-21.
6570									
	20								
6565									
	25								
6560									

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-25** Sheet 1 of 2

Boring Began: 3/30/2007
Completed: 3/30/2007
Total Depth: 55.0 ft
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill Bit: HQ
Ground Elevation: 6511.1 ft
Drill: Dietrich D-50 Rubber Track
Casing:
Location: US160 approx. Sta. 50+00, 40 ft. left
Driller: DA Smith - Roger
Weather: sunny and cold
Coordinates: N: 2,317,579.0 E: 1,211,114.2
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6510								**0.0 - 5.0 ft. gravelly COBBLES** FILL.	
	5							**5.0 - 35.0 ft. clayey GRAVEL** and COBBLES, brown with gray, Much of clay matrix washed out in core cuttings. Where no recovery, description is based on driller's observations.	No California sample here because on top of a large boulder. Begin HQ coring at 5'.
6505			30						
	10								
6500			56						
	15								
6495			22						
	20								Driller notes it feels like gravelly clay
6490			0						
	25								
6485			46						

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-25**

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6480			0						Driller notes it feels like gravelly clay
	35								
6475			50					**35.0 - 38.0 ft. COBBLES** with gravel, rounded, composition includes granite, diorite, volcanics, sandstone.	
			100						
			100	85				**38.0 - 55.0 ft. SANDSTONE INTERBEDDED WITH CONGLOMERATE**, gray and pink, fresh to slightly weathered, medium hard, parting close, iron oxide stains, Fair- to well-indurated..	
	40								
6470			98	95					
	45								
6465			84	71					
	50								
6460			78	34					
	55							Bottom of Hole at 55.0 ft.	Driller notes bit is worn through to carbide--would have to re-drill entire hole to get any further due to caving materials. Stop at 17' into bedrock.
6455									
	60								
6450									
	65								
6445									

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-26**
Sheet 1 of 1

Boring Began: 3/30/2007
Completed: 3/30/2007
Total Depth: 30.0 ft
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill Bit: HQ
Ground Elevation: 6514.1 ft
Casing:
Location: Wall L, approx. Sta. 351+00
Drill: Dietrich D-50 Rubber Track
Weather: sunny and cold
Coordinates: N: 2,317,657.2 E: 1,211,195.9
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6510	5				7/7	14		**0.0 - 12.0 ft. clayey SAND** with gravel. Cobble layers from 2-3', 8-10', 11-12', dark brown, medium plasticity, moist, medium dense to dense.	MC= 13.8 % DD= 91.3 pcf
6505	10				17/21	38			Cuttings are rounded rocks MC= 12.5 %
6500	15		20					**12.0 - 22.5 ft. clayey GRAVEL** with many COBBLES and some BOULDERS, brown and gray, most of clay matrix washed away in cuttings. Cobbles/boulders are sandstone/conglomerate pieces, hard, fresh, well-indurated.	Auger refusal, begin HQ coring at 12'.
6495	20		76						Driller notes this run felt consistent, but fluid return varies widely
6490	25		96	18				**22.5 - 30.0 ft. SANDSTONE**, gray, fresh, medium hard to hard, parting close, horizontal, clay and iron oxide infilling, Well-indurated. Some visible horizontal lamination. Interbedded with some sandy conglomerate near 28'.	
6485	30		98	85				Bottom of Hole at 30.0 ft.	

APPENDIX C – LABORATORY TEST RESULTS

YEH & ASSOCIATES, INC

Summary of Laboratory Test Results

Project No: 26 - 235 Project Name: US 160 4th Lane Date: 1/29/2007

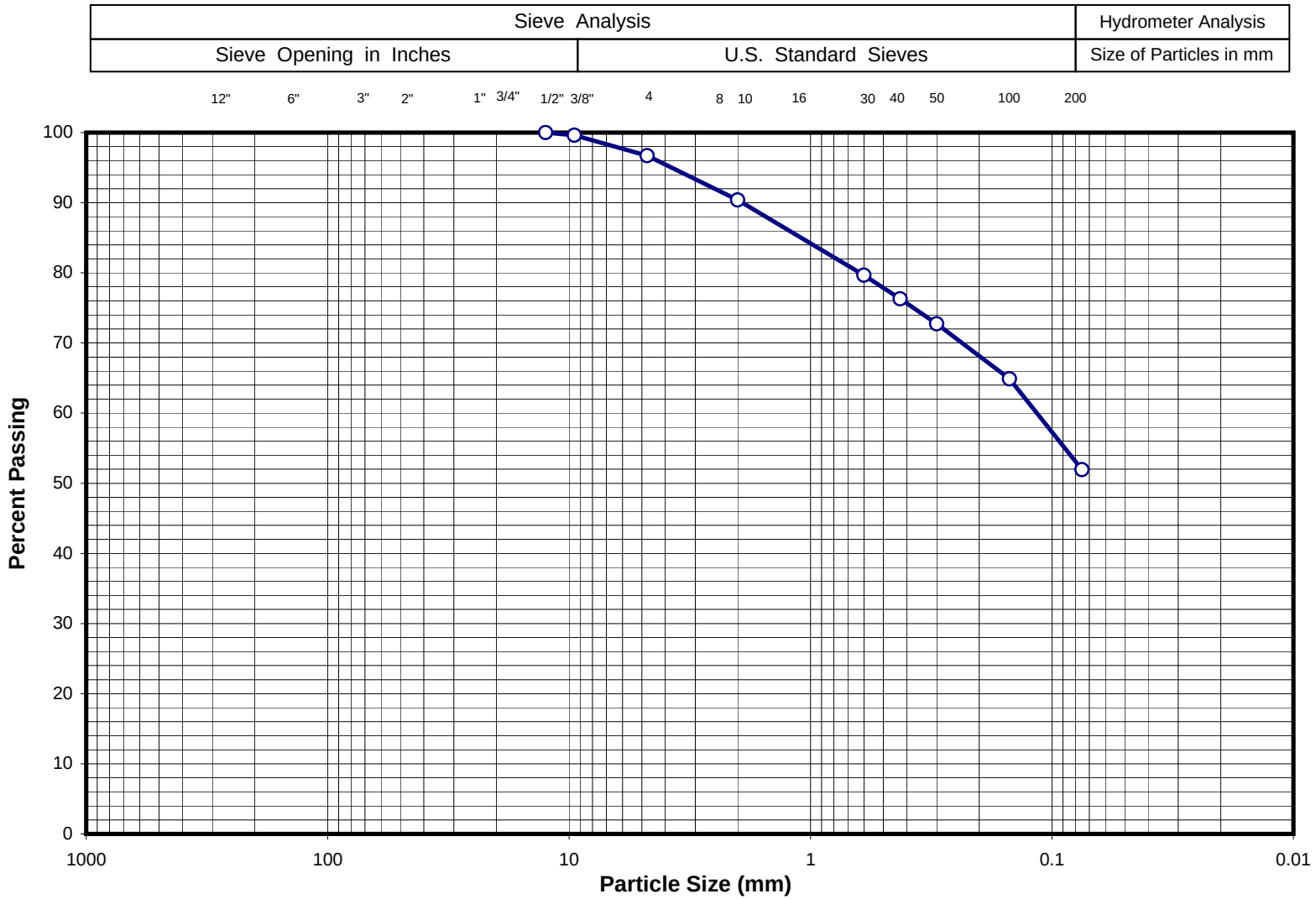
Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate (%)	% Swell (+) / Consoli-dation (-)	R-Value	Resistivity (ohms-cm)	CLASSIFICATION	
Boring NO.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
YA-01	9.2-10.5	SS	23.3	_	34	49	17	34	20	14	8.8	0.009	–	–	–	A-2-6 (0)	SC
YA-01	14.2-15.7	SS	30.8	–	–	–	56	41	22	19	–	–	–	–	–	A-7-6 (8)	CL
YA-02	5-6.5	SS	20.3	–	1	52	47	33	21	12	–	–	–	–	–	A - 6 (3)	SC
YA-02	14.7-16.2	SS	15.5	–	21	50	29	30	19	11	–	–	–	–	–	A-2-6 (0)	SC
YA-03	14.2-15.7	SS	17.6	–	9	62	29	30	19	11	7.2	0.001	–	–	1664	A-2-6 (0)	SC
YA-04	19.2-20.7	SS	24.5	–	29	40	31	40	21	19	–	–	–	–	–	A-2-6 (1)	SC
YA-05	5.5-7	SS	22.4	–	15	42	43	40	19	21	–	–	–	–	–	A - 6 (5)	SC
YA-05	35.5-37	SS	29.3	–	9	49	42	37	24	13	8.2	0.021	–	–	–	A - 6 (2)	SC
YA-05	40.5-42	SS	20.0	–	38	48	14	29	19	10	–	–	–	–	–	A-2-4 (0)	SC
YA-06	5.2-6.7	SS	23.6	–	–	–	56	44	28	16	–	–	–	–	–	A-7-6 (7)	ML
YA-07	5.2-6.7	SS	16.5	–	38	44	18	32	20	12	–	–	–	–	–	A - 6 (4)	CL
YA-08	9.2-10.7	SS	26.8	–	–	–	58	37	20	17	–	–	–	–	–	A - 6 (7)	CL
YA-09	4.2-5.7	SS	31.6	–	–	–	51	42	22	20	–	–	–	–	–	A-7-6 (7)	CL
YA-09	14.2-15.7	SS	32.9	–	–	–	53	42	23	19	–	–	–	–	–	A-7-6 (7)	CL
YA-12	6.2-10	Core	8.7	–	–	–	53	39	24	15	7.2	0.001	–	–	914	A - 6 (5)	CL
YA-15	4.2-5.7	SS	29.6	–	2	69	29	38	24	14	–	–	–	–	–	A-2-6 (0)	SC
18	0 - 3	Bulk	8.2	–	3	45	52	36	19	17	6.6	<0.001	–	14.0	2016.0	A - 6 (6)	CL
21	5 - 6	CA	14.5	86.0	–	–	–	–	–	–	–	–	-2.9	–	–	–	–

YEH & ASSOCIATES, INC

Summary of Laboratory Test Results

Project No: 26 - 235 Project Name: US 160 4th Lane Date: 1/29/2007

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate (%)	% Swell (+) / Consoli-dation (-)	R-Value	Resistivity (ohms-cm)	CLASSIFICATION	
Boring NO.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
22	0 - 5	Bulk	10.7	–	2	31	67	52	23	29	7.8	< 0.001	–	< 5	1189	A-7-6 (18)	CH
22	5 - 6	CA	19.8	81.5	–	–	–	–	–	–	–	–	-0.8	–	–	–	–
22	10 - 11	CA	17.6	85.4	–	–	–	–	–	–	–	–	-1.2	–	–	–	–
23	5 - 6	CA	12.3	–	–	–	–	–	–	–	–	–	–	–	–	–	–
26	5 - 6	CA	13.8	91.3	–	–	–	–	–	–	–	–	–	–	–	–	–
26	10 - 11	CA	12.5	–	–	–	–	–	–	–	–	–	–	–	–	–	–



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	-
½"	100
⅜"	100
#4	97
#10	90
#40	76
#200	52

Gravel (%)	3	LL	36	Project Name:	US 160 4th Lane
Sand (%)	45	PL	19	Sample ID:	YA-18
Fines (%)	52	PI	17	Sample Depth (ft.):	0 - 5
Sample Description:	CL / A - 6 (6)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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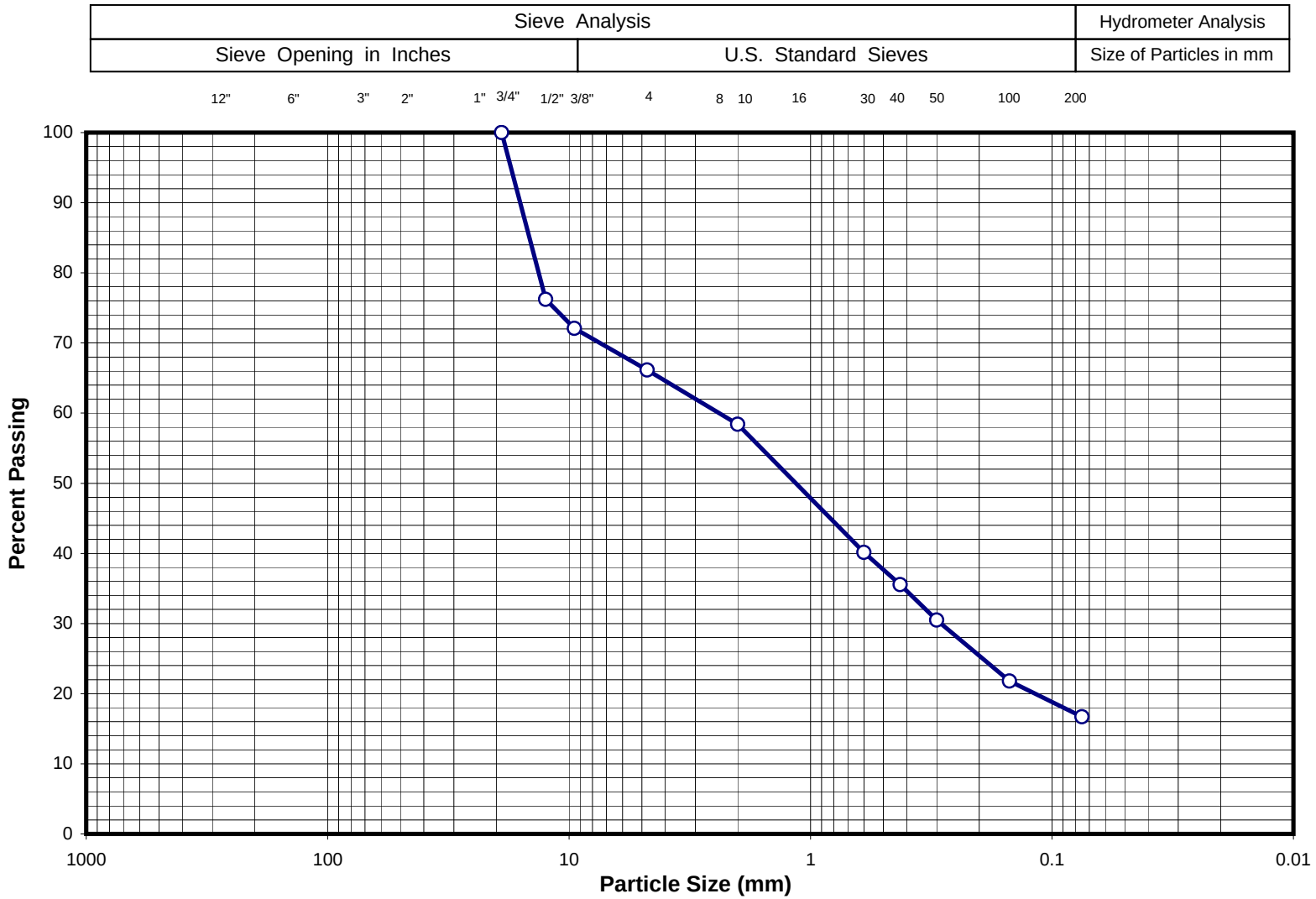

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	76
⅜"	72
#4	66
#10	58
#40	36
#200	17

Gravel (%)	34	LL	34	Project Name:	US 160 4th Lane
Sand (%)	49	PL	20	Sample ID:	YA-01
Fines (%)	17	PI	14	Sample Depth (ft.):	9.2 - 10.7
Sample Description:	SC / A - 2 - 6 (0)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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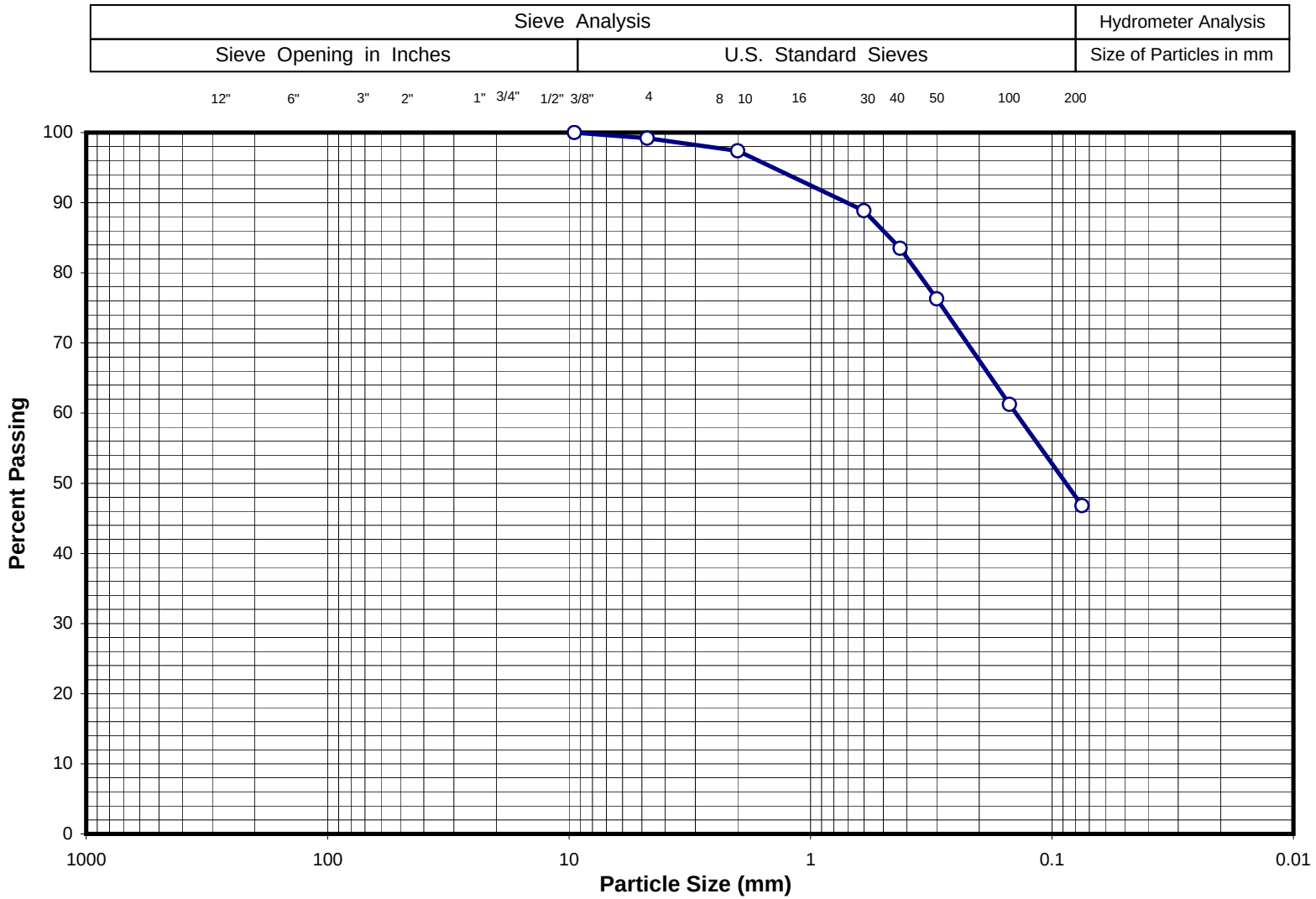

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	-
½"	-
⅜"	100
#4	99
#10	97
#40	84
#200	47

Gravel (%)	1	LL	33	Project Name:	US 160 4th lane
Sand (%)	52	PL	21	Sample ID:	YA-02
Fines (%)	47	PI	12	Sample Depth (ft.):	5.0 - 6.5
Sample Description:	SC / A - 6 (3)				

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Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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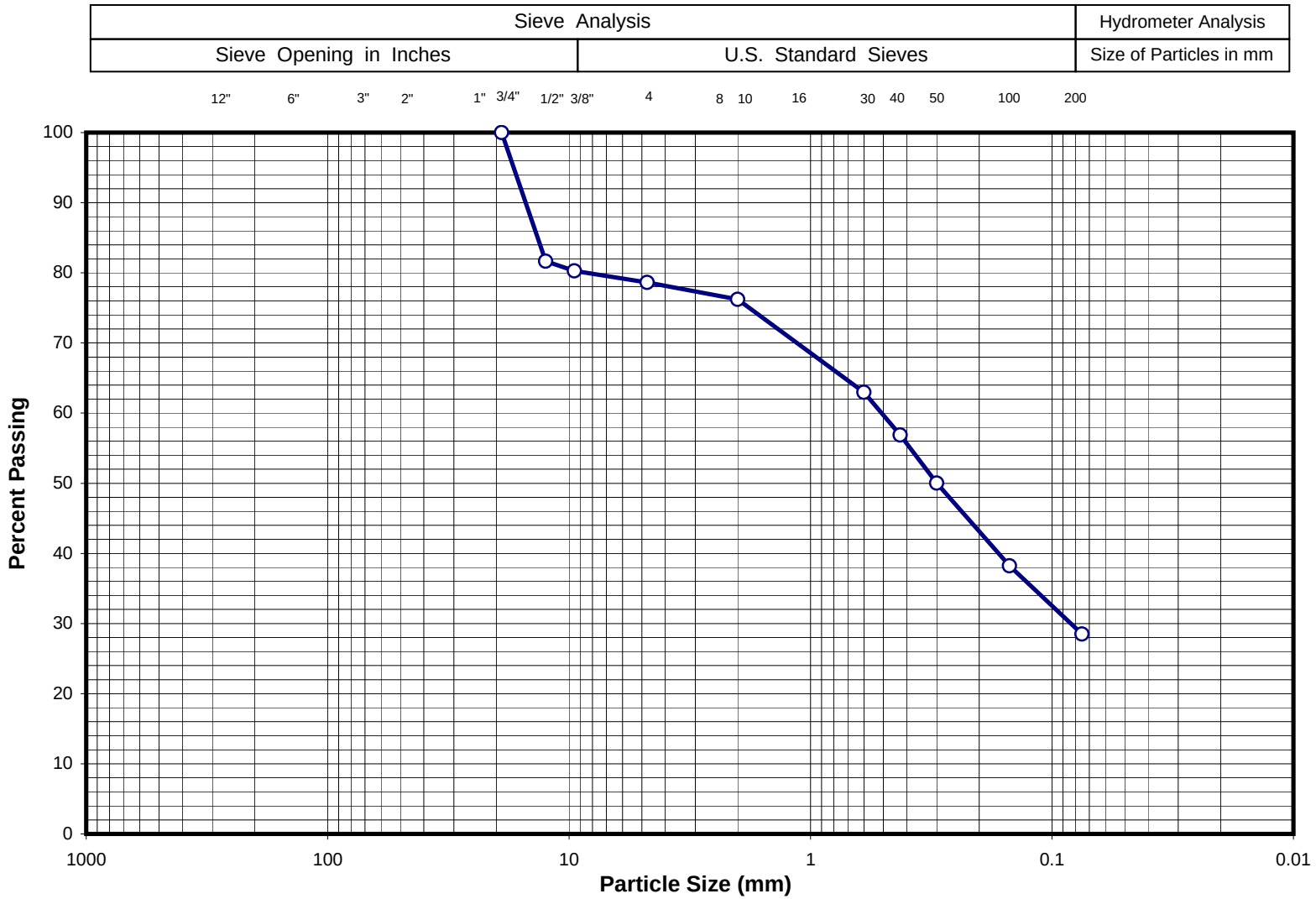

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	82
⅜"	80
#4	79
#10	76
#40	57
#200	29

Gravel (%)	21	LL	30	Project Name:	US 160 4th Lane
Sand (%)	50	PL	19	Sample ID:	YA-02
Fines (%)	29	PI	11	Sample Depth (ft.):	14.7 - 16.2
Sample Description:	SC / A - 2 - 6 (0)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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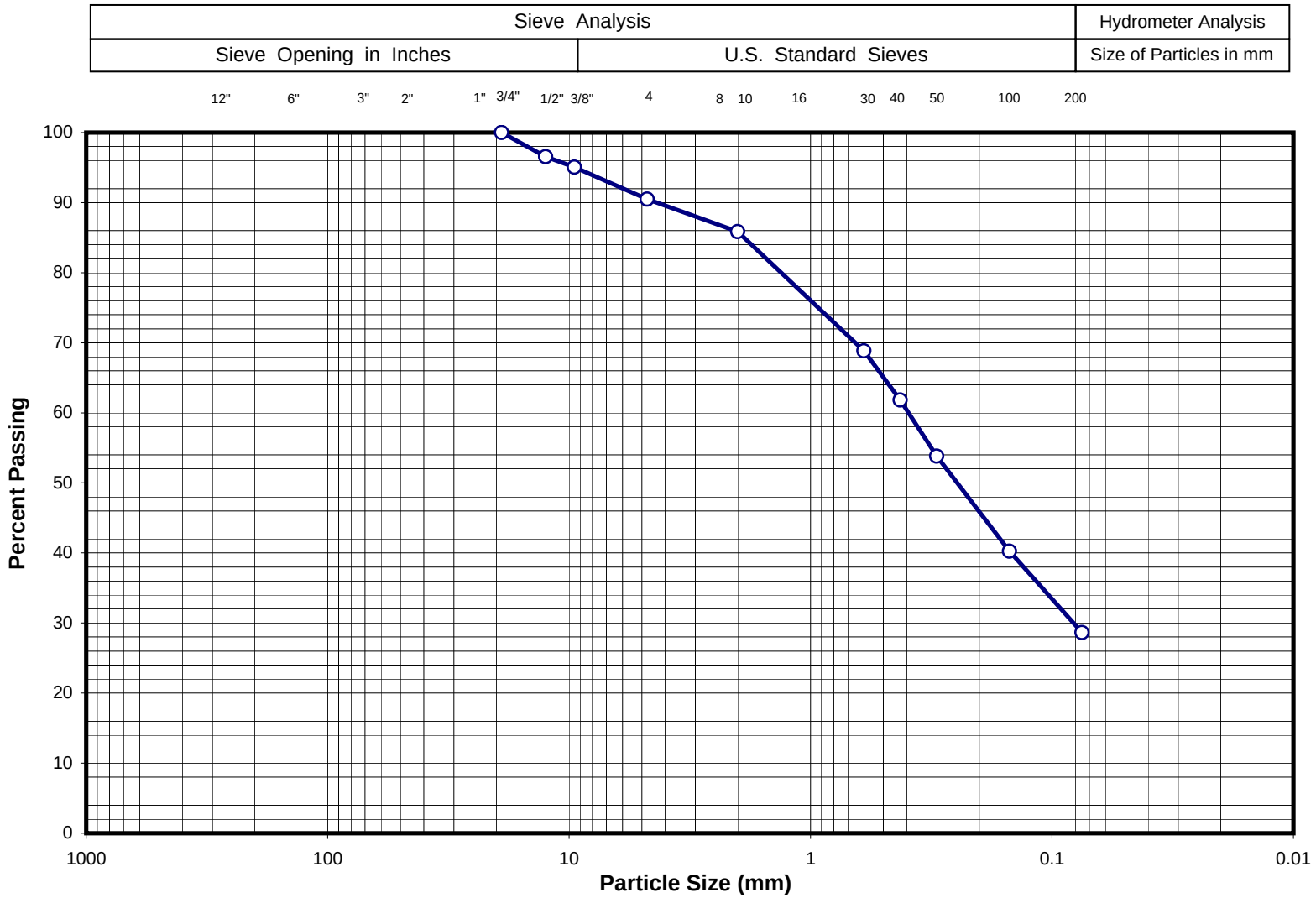

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	97
⅜"	95
#4	91
#10	86
#40	62
#200	29

Gravel (%)	9	LL	30	Project Name:	US 160 4th Lane
Sand (%)	62	PL	19	Sample ID:	YA-03
Fines (%)	29	PI	11	Sample Depth (ft.):	14.2 - 15.7
Sample Description:	SC / A - 2 - 6 (0)				

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Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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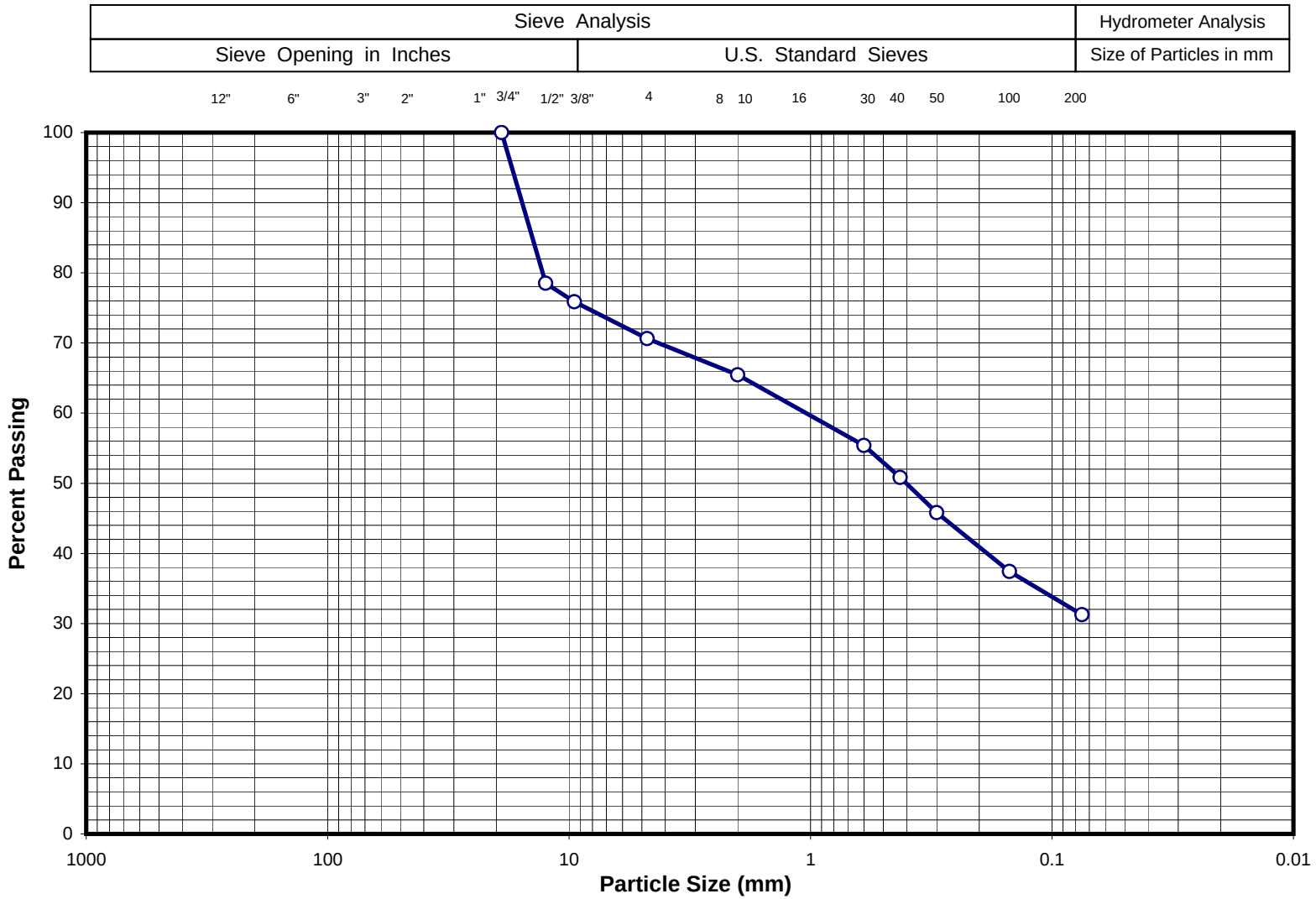

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	79
⅜"	76
#4	71
#10	65
#40	51
#200	31

Gravel (%)	29	LL	40	Project Name:	US 160 4th Lane
Sand (%)	40	PL	21	Sample ID:	YA-04
Fines (%)	31	PI	19	Sample Depth (ft.):	19.2 - 20.7
Sample Description:	SC / A - 2 - 6 (1)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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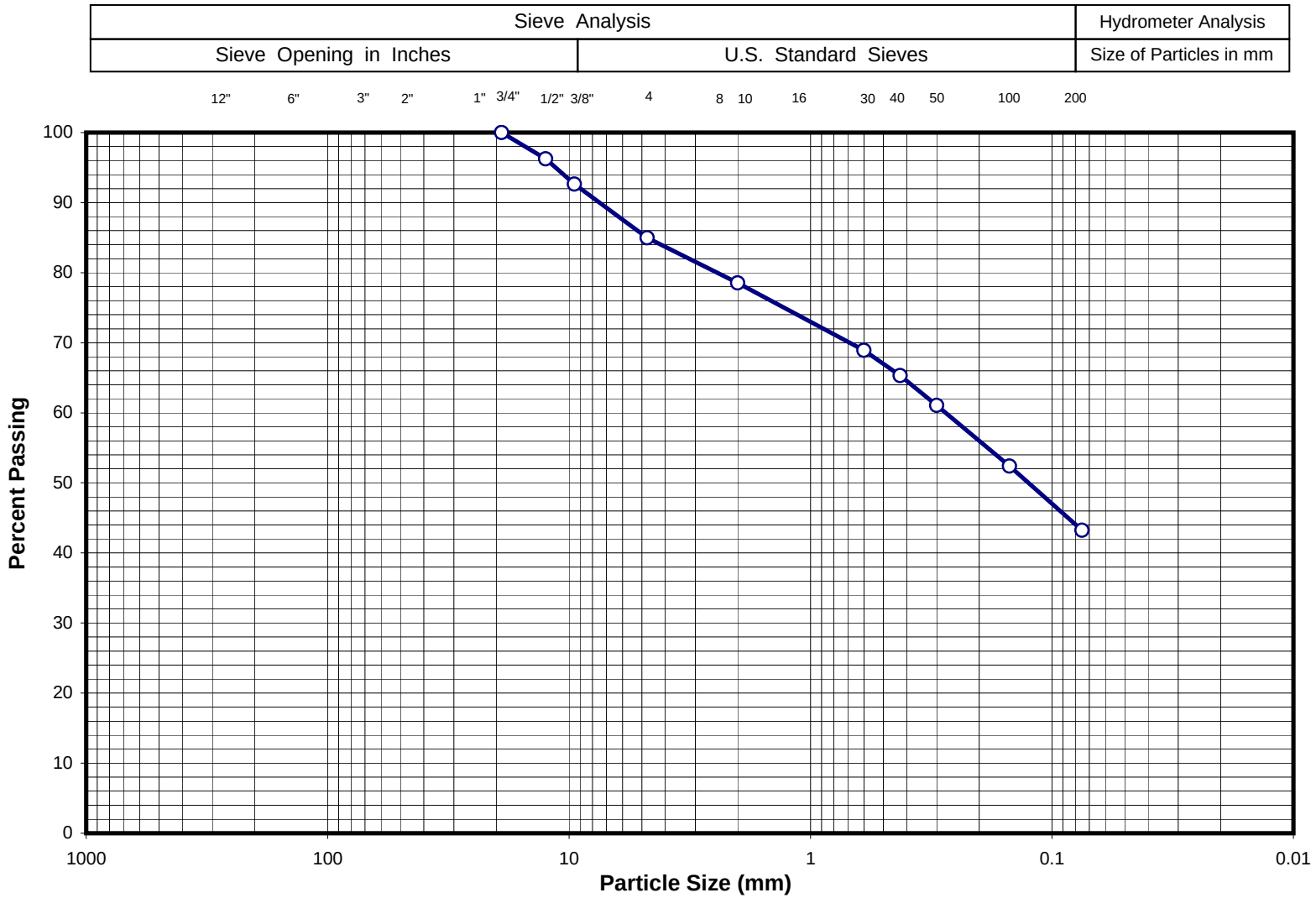

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	96
⅜"	93
#4	85
#10	79
#40	65
#200	43

Gravel (%)	15	LL	40	Project Name:	US 160 4th Lane
Sand (%)	42	PL	19	Sample ID:	YA-05
Fines (%)	43	PI	21	Sample Depth (ft.):	5.5 - 7
Sample Description:	SC / A - 6 (5)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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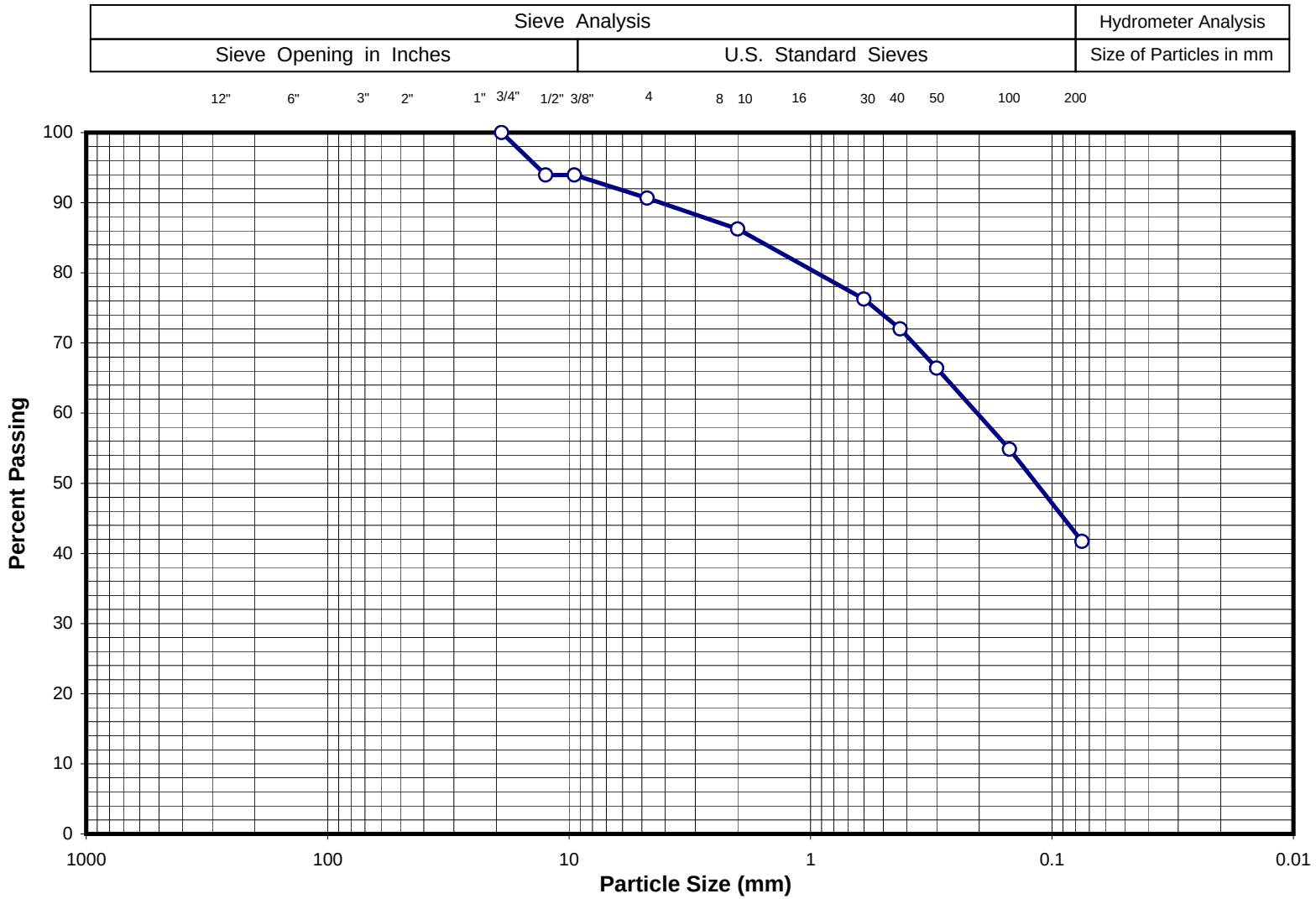

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	94
⅜"	94
#4	91
#10	86
#40	72
#200	42

Gravel (%)	9	LL	37	Project Name:	US 160 4th Lane
Sand (%)	49	PL	24	Sample ID:	YA-05
Fines (%)	42	PI	13	Sample Depth (ft.):	35.5 - 37
Sample Description:	SC / A - 6 (2)				

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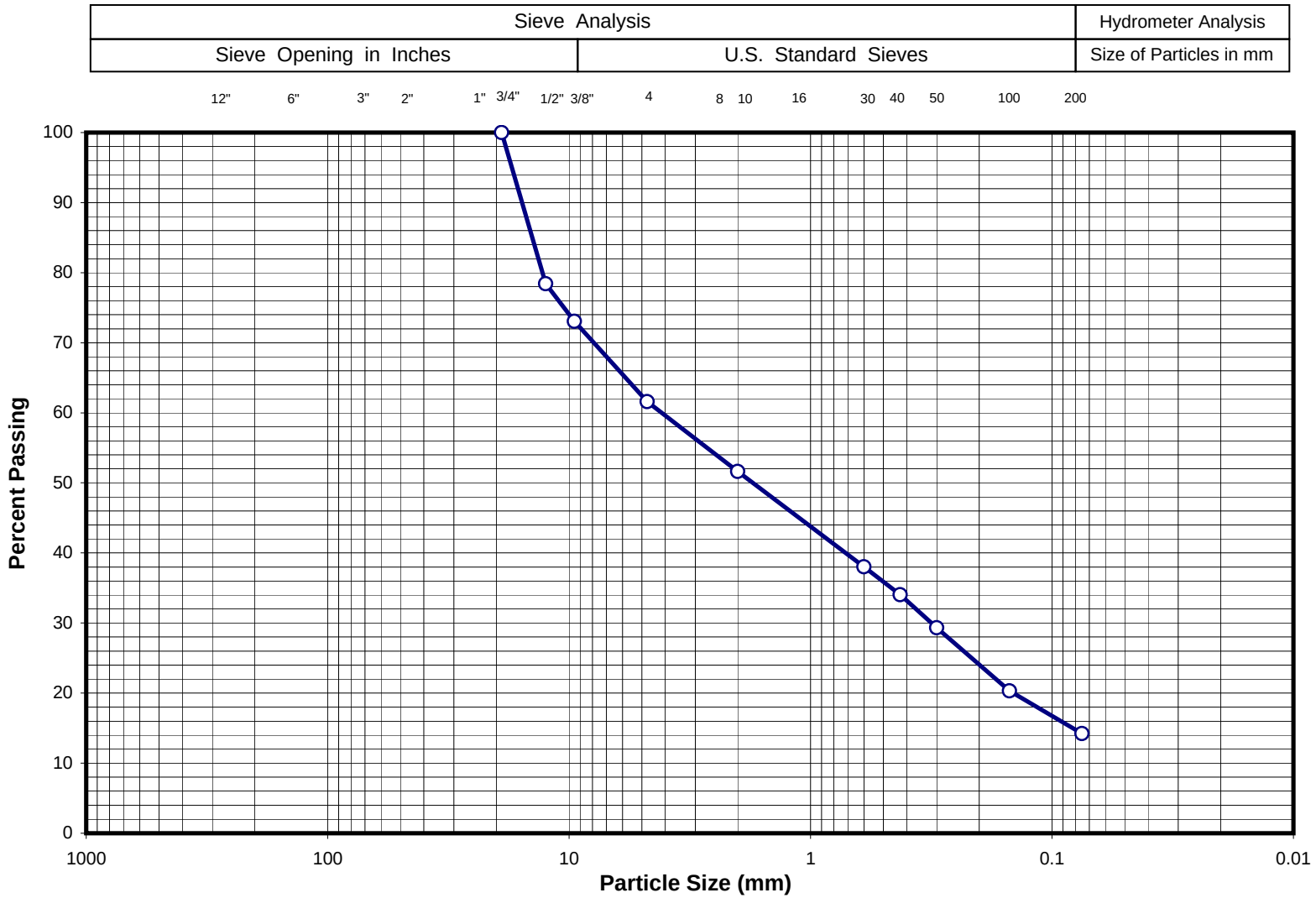
SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	78
⅜"	73
#4	62
#10	52
#40	34
#200	14

Gravel (%)	38	LL	29	Project Name:	US 160 4th Lane
Sand (%)	48	PL	19	Sample ID:	YA-05
Fines (%)	14	PI	10	Sample Depth (ft.):	40.5 - 42
Sample Description:	SC / A - 2 - 4 (0)				

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SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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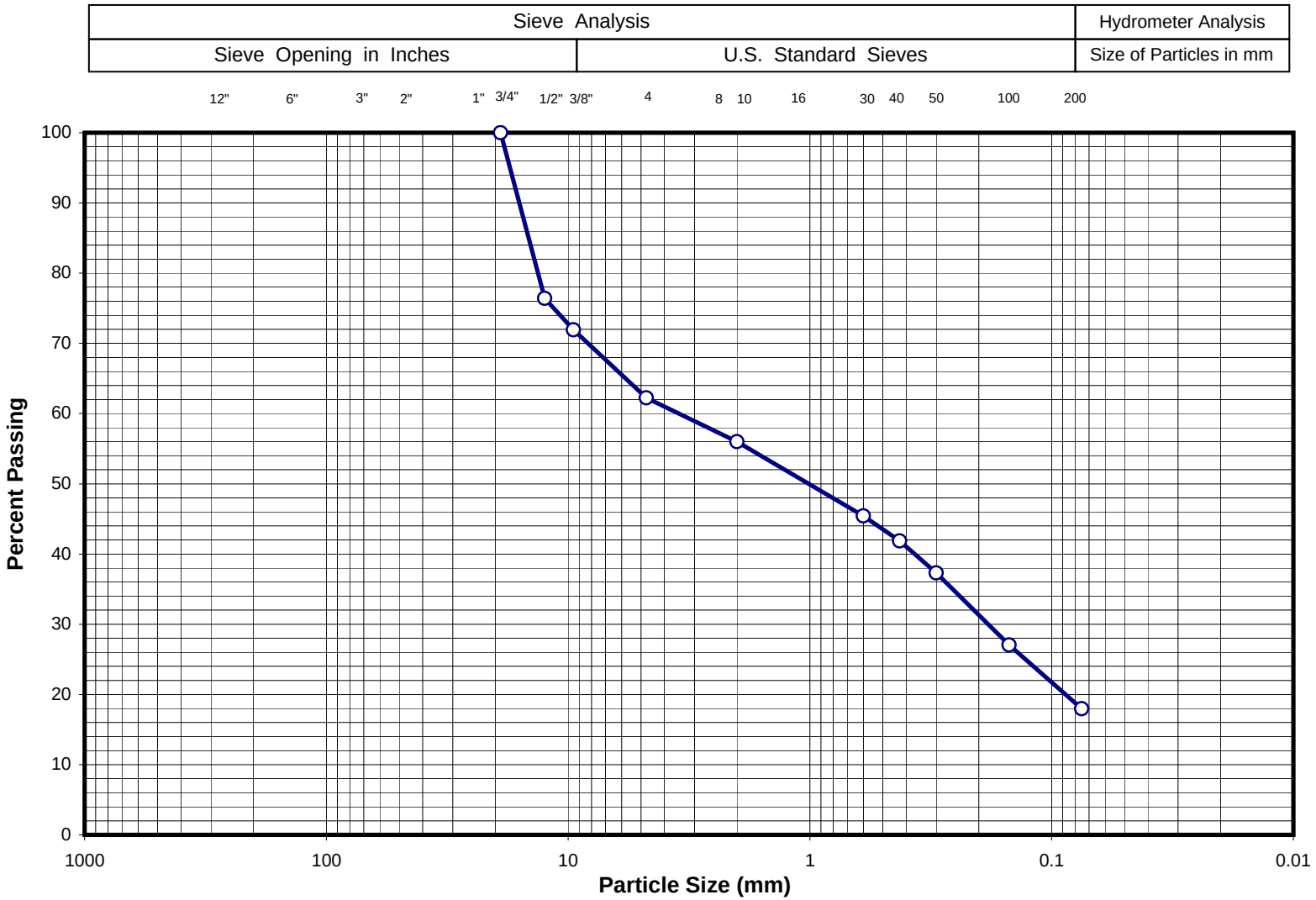

Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	76
⅜"	72
#4	62
#10	56
#40	42
#200	18

Gravel (%)	38	LL	32	Project Name:	US 160 4th Lane
Sand (%)	44	PL	20	Sample ID:	YA-06
Fines (%)	18	PI	12	Sample Depth (ft.):	5.2 - 6.7
Sample Description:	CL / A - 6 (4)				

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Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

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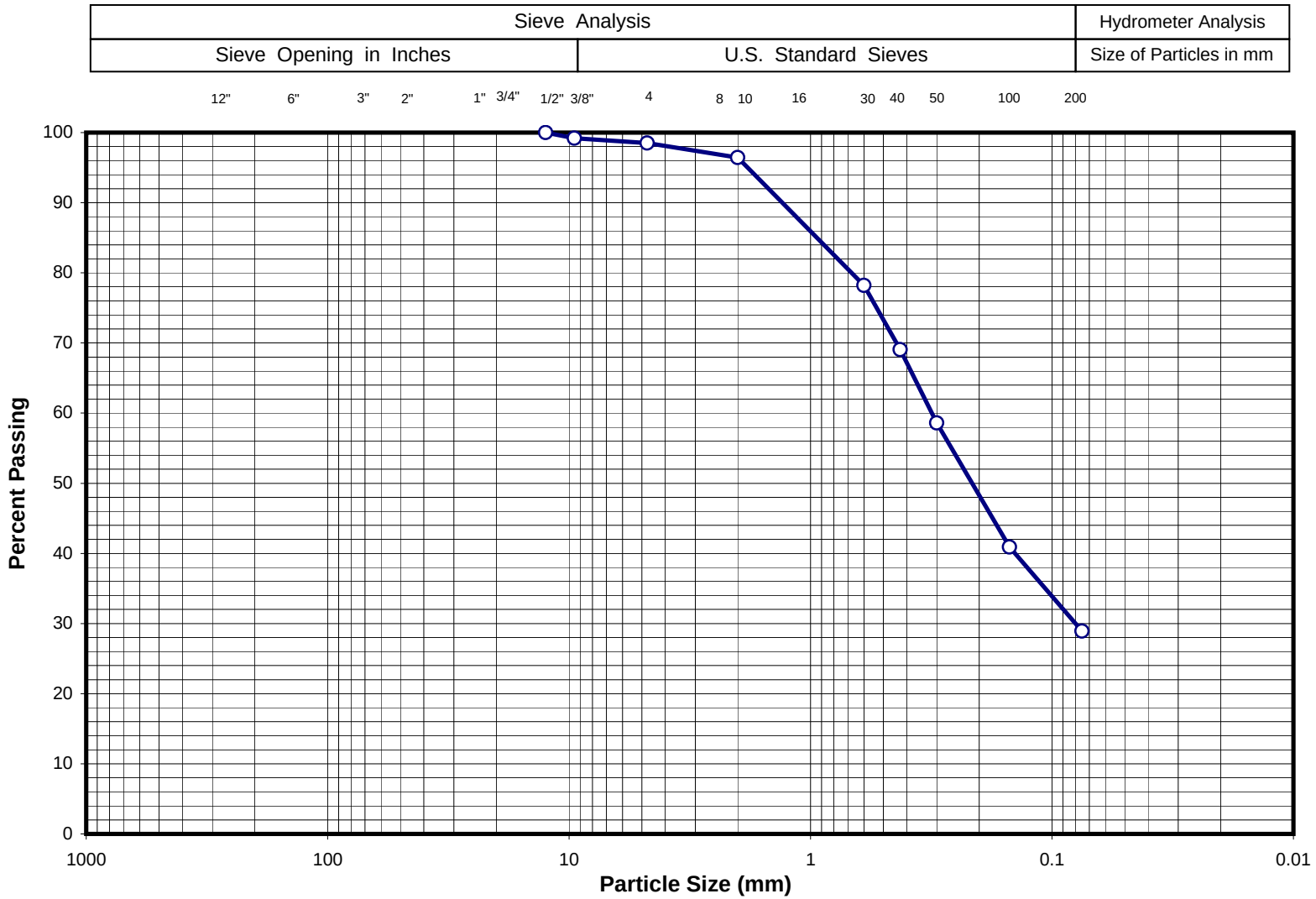


Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	-
½"	100
⅜"	99
#4	98
#10	96
#40	69
#200	29

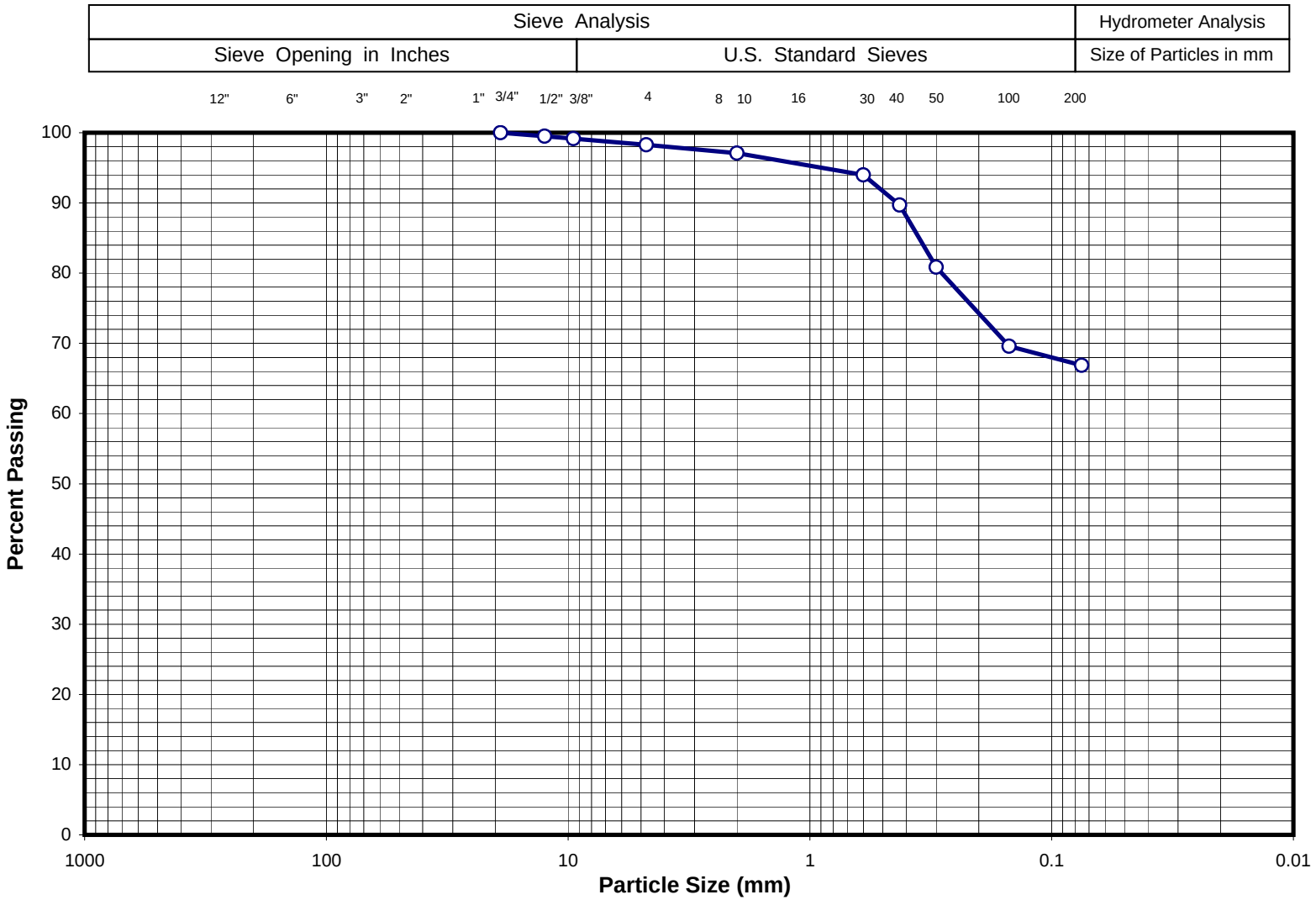
Gravel (%)	2	LL	38	Project Name:	US 160 4th Lane
Sand (%)	69	PL	24	Sample ID:	YA-15
Fines (%)	29	PI	14	Sample Depth (ft.):	4.2 - 5.7
Sample Description:	SC / A - 2 - 6 (0)				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	01/25/07	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	-
½"	100
⅜"	99
#4	98
#10	97
#40	90
#200	67

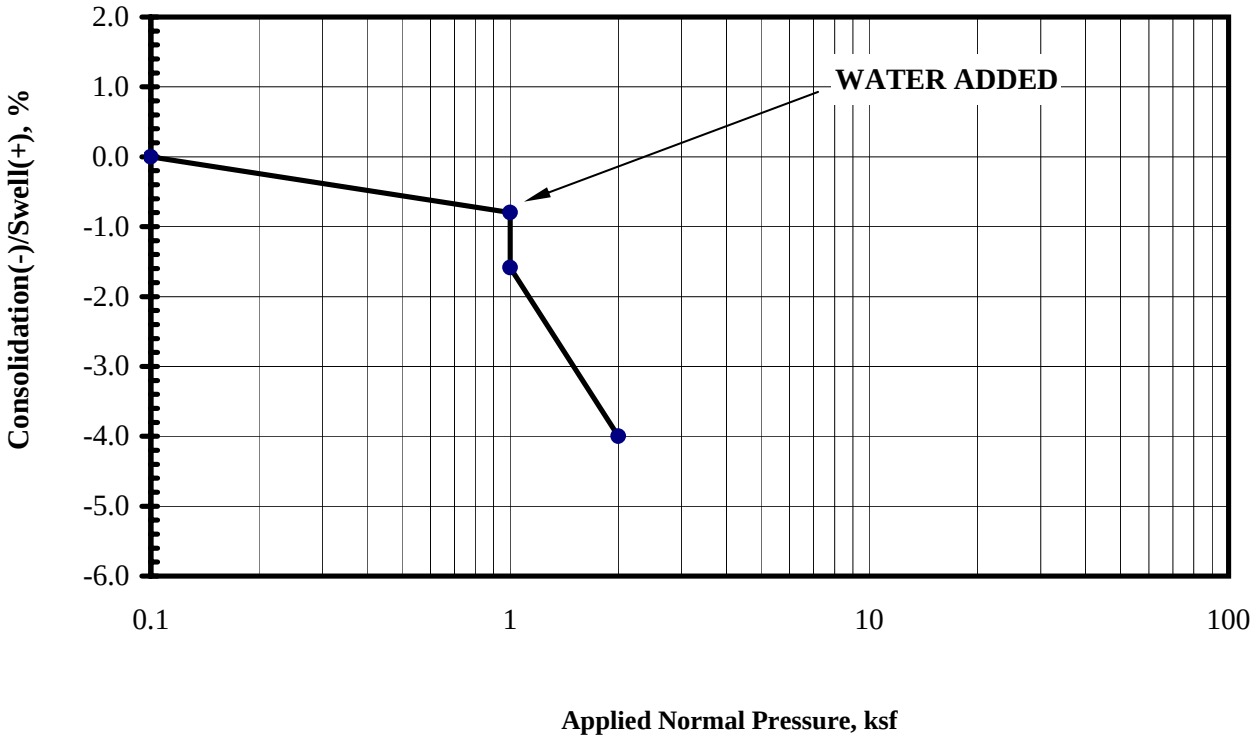
Gravel (%)	2	LL	52	Project Name:	US 160 4th Lane
Sand (%)	31	PL	23	Sample ID:	YA-22
Fines (%)	67	PI	29	Sample Depth (ft.):	0 - 5
Sample Description:	CH / A - 7 - 6 (18)				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

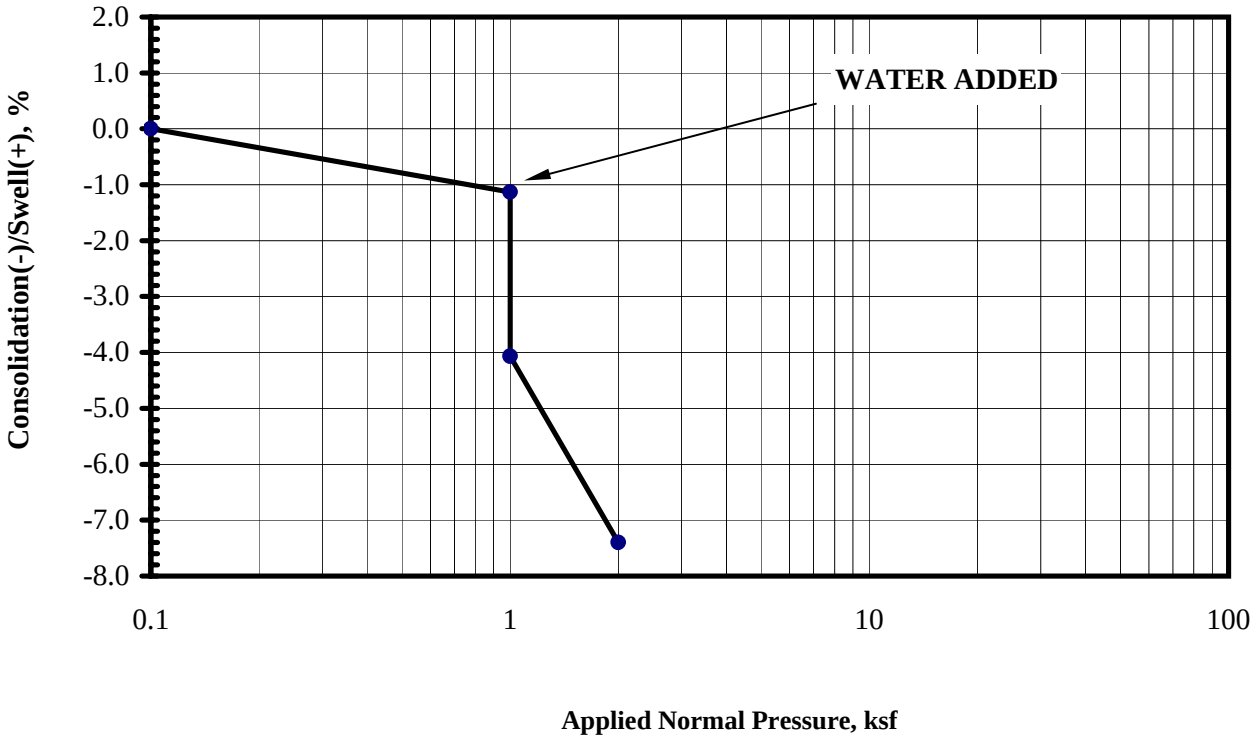
Drawn By:	MA	Project No.:	26 - 235
Checked By:	SY		
Date:	04/11/07	Figure No.:	-

Revised 04/27/2004



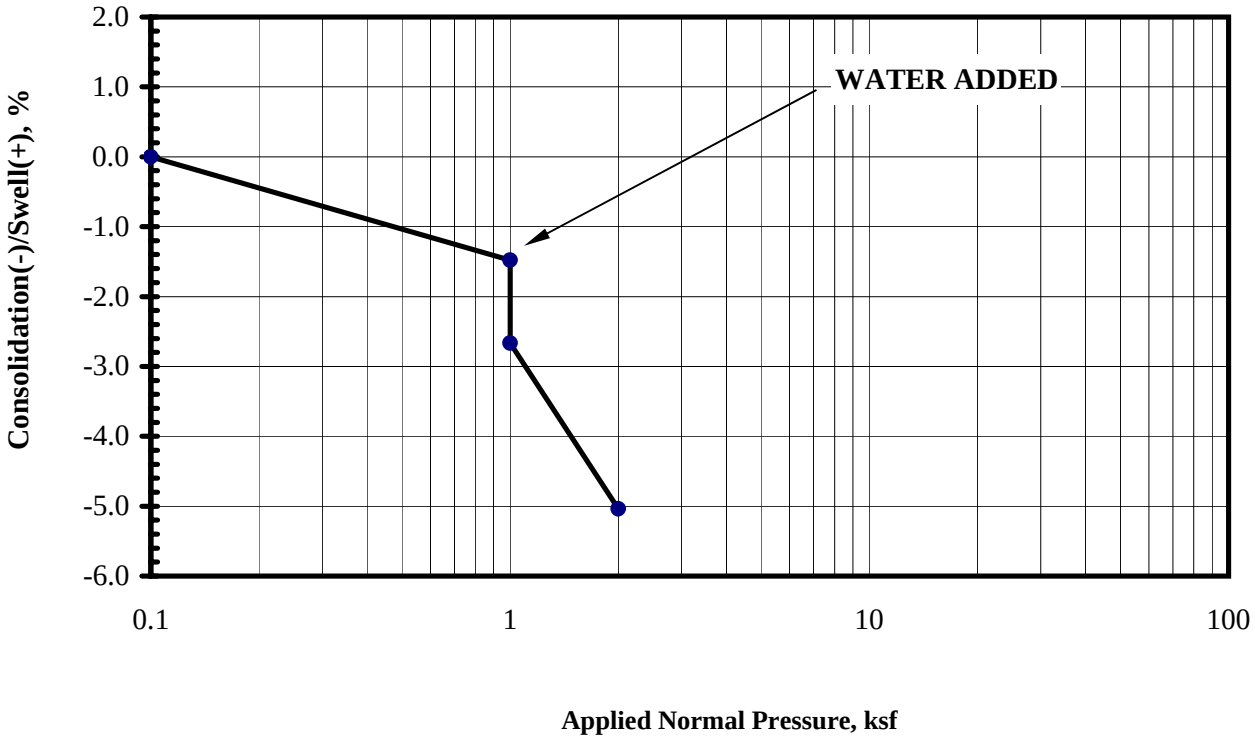
Boring Number	Depth, ft	Natural Dry Density, pcf	Moisture Content, %	Consolidation(-) /Swell(+), %	Soil Description	SWELL / CONSOLIDATION GRAPH
YA-22	5 - 6	81.5	19.8	-0.8		Drawn By: MA
Job No: 26 - 235	**Project Name:**			US 160 4th Lane		Checked By: SY

YEH & ASSOCIATES, INC



Boring Number	Depth, ft	Natural Dry Density, pcf	Moisture Content, %	Consolidation(-) /Swell(+), %	Soil Description	SWELL / CONSOLIDATION GRAPH
YA-21	5 - 6	86	14.5	-2.9		Drawn By: MA
Job No: 26 - 235	**Project Name:**			US 160 4th Lane		Checked By: SY

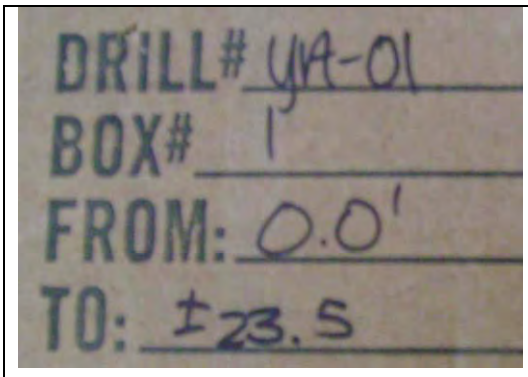
YEH & ASSOCIATES, INC



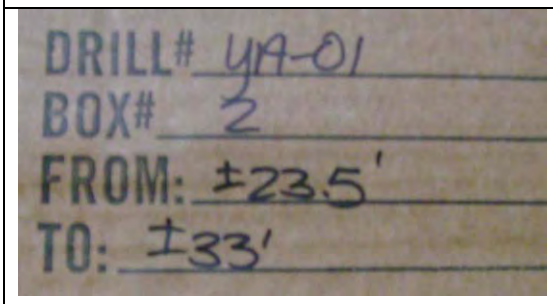
Boring Number	Depth, ft	Natural Dry Density, pcf	Moisture Content, %	Consolidation(-) /Swell(+), %	Soil Description	SWELL / CONSOLIDATION GRAPH
YA-22	10 - 11	85.4	17.6	-1.2		Drawn By: MA
Job No: 26 - 235	**Project Name:**			US 160 4th Lane		Checked By: SY

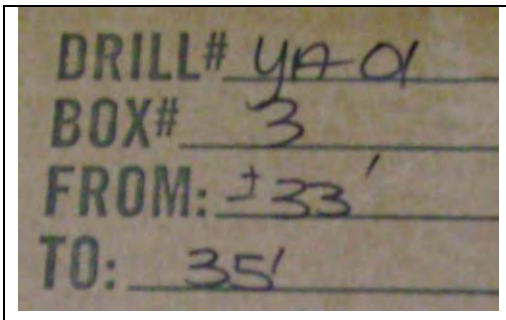
YEH & ASSOCIATES, INC

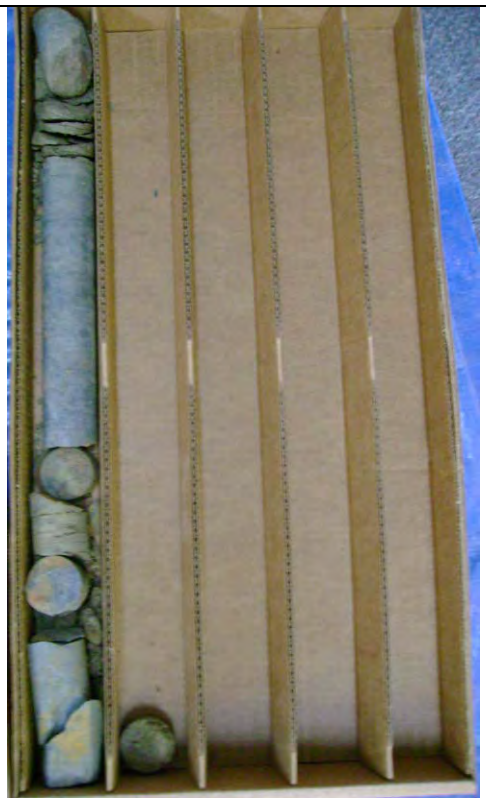
APPENDIX D – PHOTOS OF RECOVERED CORES

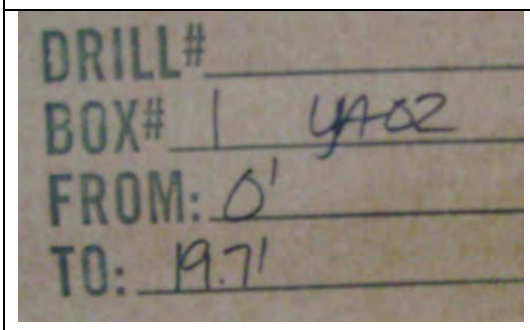

DRILL# YA-01
BOX# 1
FROM: 0.0'
TO: ±23.5

Hefty
Hefty

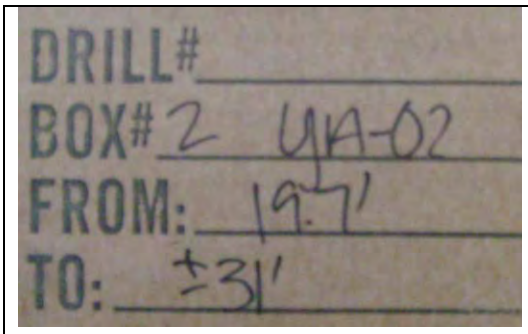

DRILL# YA-01
BOX# 2
FROM: ±23.5'
TO: ±33'


DRILL# YA-01
BOX# 3
FROM: ±33'
TO: 35'

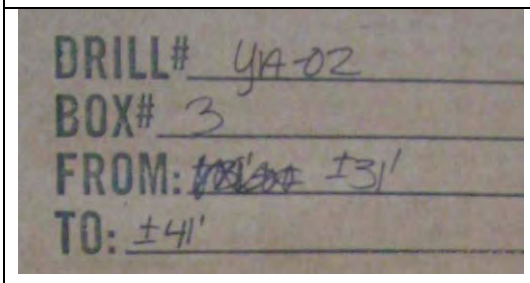



DRILL#
BOX# 1 YA02
FROM: 0'
TO: 19.7'

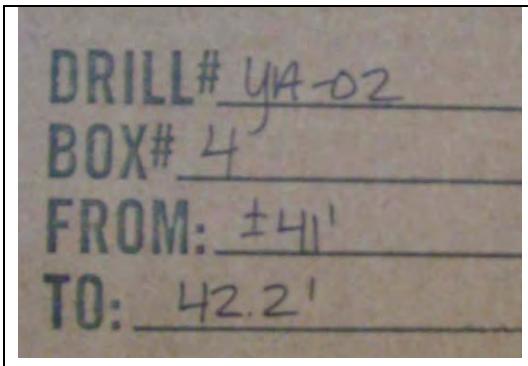
Hefty
Hefty

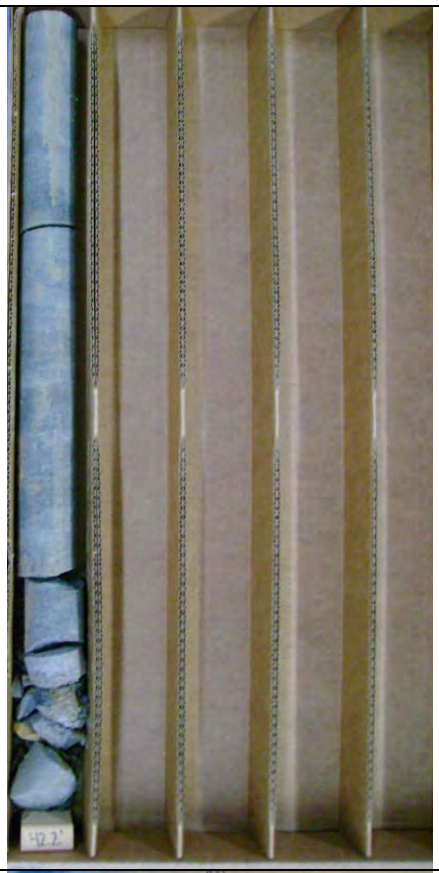

DRILL#
BOX# 2 YA-02
FROM: 19.7'
TO: ±31'

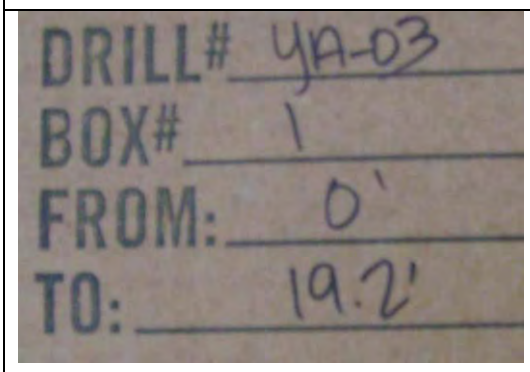
27.3
24.2'

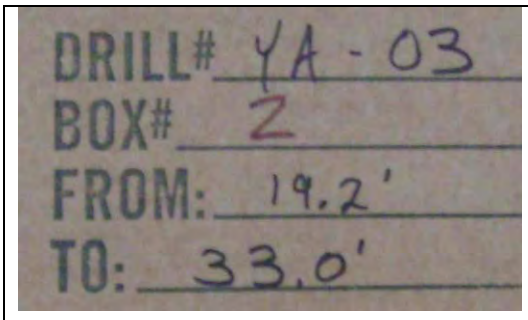

DRILL# YA-02
BOX# 3
FROM: ±31'
TO: ±41'

37.1

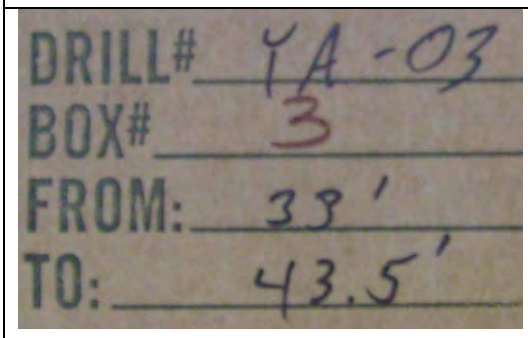

DRILL# YA-02
BOX# 4
FROM: ±41'
TO: 42.2'



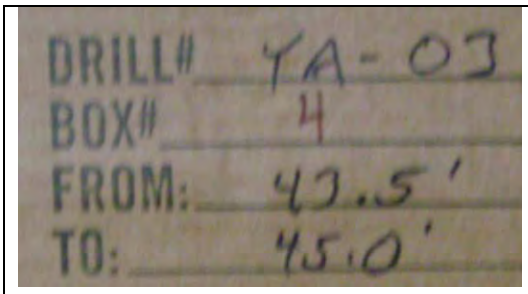

DRILL# YA-03
BOX# 1
FROM: 0'
TO: 19.2'

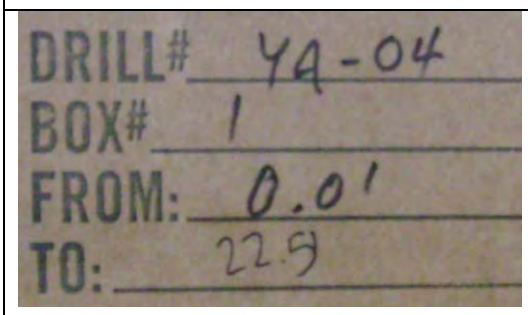

DRILL# YA-03
BOX# 2
FROM: 19.2'
TO: 33.0'

24.2'
28.8
31.5
21.8'
25.3
29.5

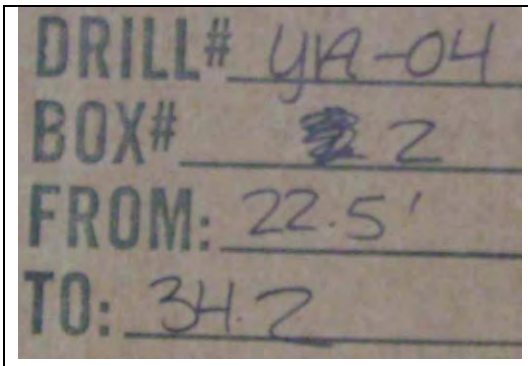

DRILL# YA-03
BOX# 3
FROM: 33'
TO: 43.5'

36.6
41.6

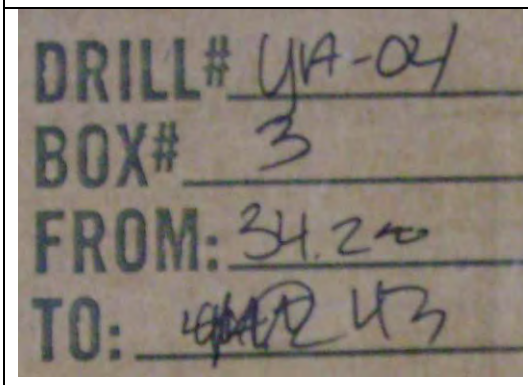

DRILL# YA-03
BOX# 4
FROM: 43.5'
TO: 45.0'

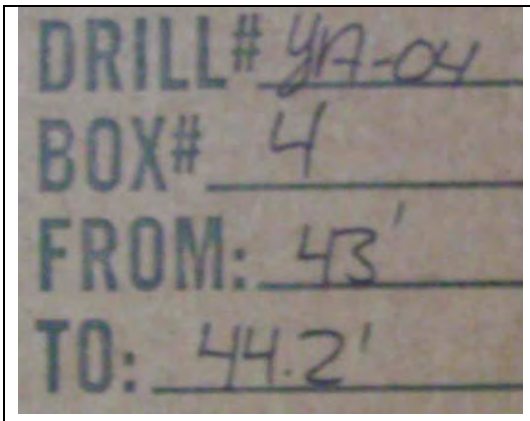

DRILL# YA-04
BOX# 1
FROM: 0.0'
TO: 22.9

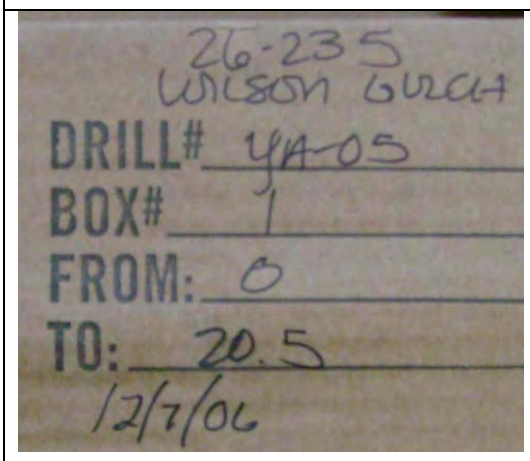
0.0
7.0
11.6
3.9
16.7
5.0


DRILL# YA-04
BOX# 2
FROM: 22.5'
TO: 34.2

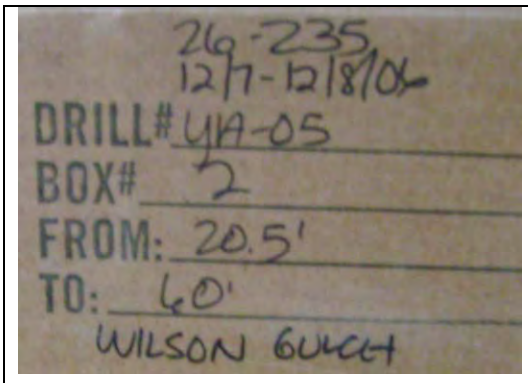
24.2
27.2
27.9
30.8


DRILL# YA-04
BOX# 3
FROM: 34.2
TO: 43

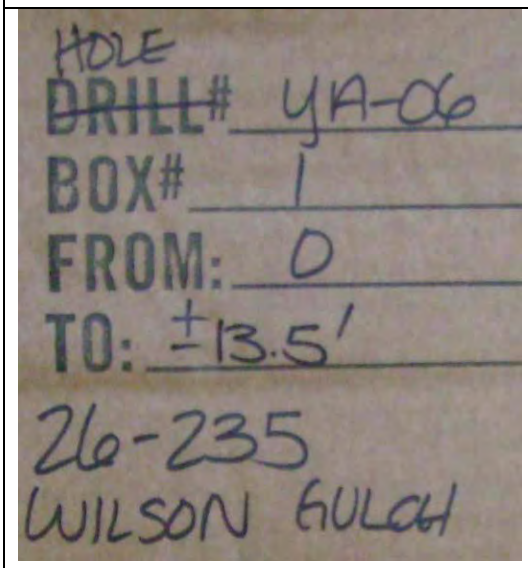

DRILL# YA-04
BOX# 4
FROM: 43'
TO: 44.2'


26-235
WILSON GULCH
DRILL# YA-05
BOX# 1
FROM: 0
TO: 20.5
12/7/06

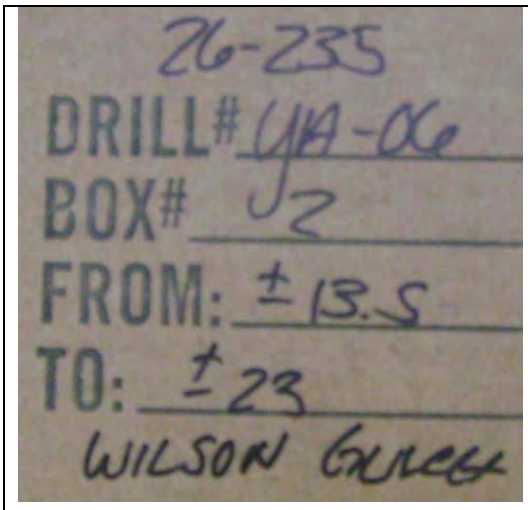
1.8
8.6
15.5
18.6
10.5
13.6'


26-235
12/7-12/8/06
DRILL# YA-05
BOX# 2
FROM: 20.5'
TO: 60'
WILSON GULCH

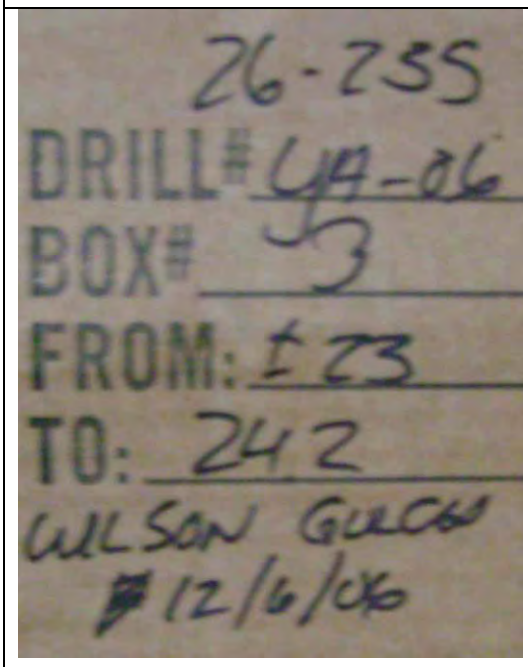
45.1
41.3
53
44
48.9
58
110.5

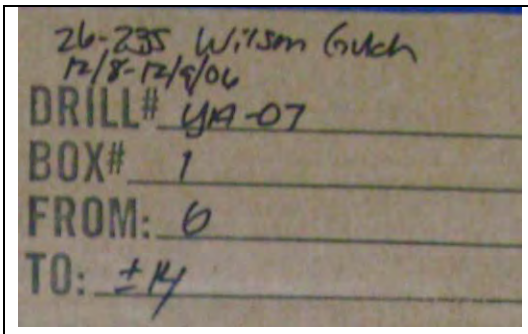

HOLE
~~DRILL~~# YA-06
BOX# 1
FROM: 0
TO: ±13.5'
26-235
WILSON GULCH

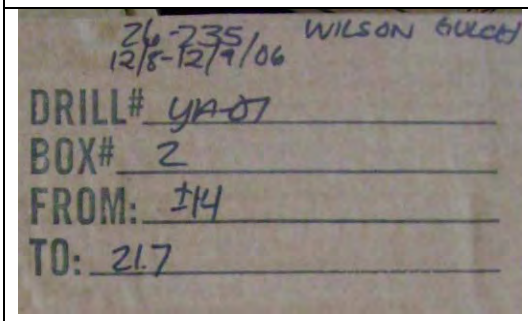
3.5'
5.2

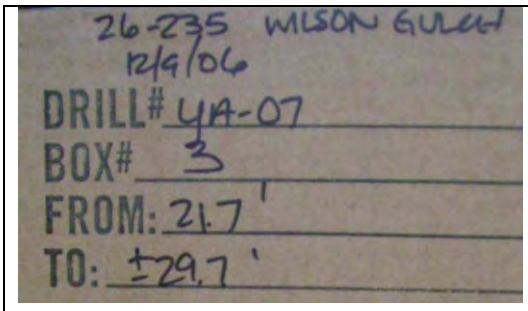

26-235
DRILL# YA-06
BOX# 2
FROM: ±13.5
TO: ±23
WILSON GULCH

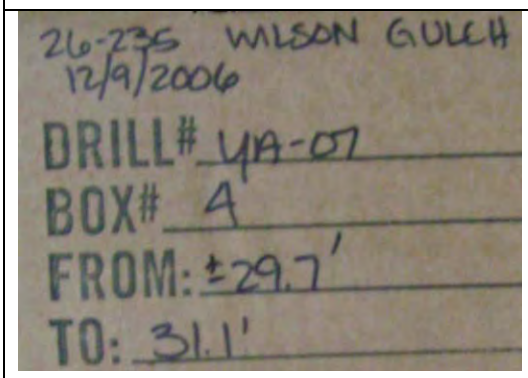
142

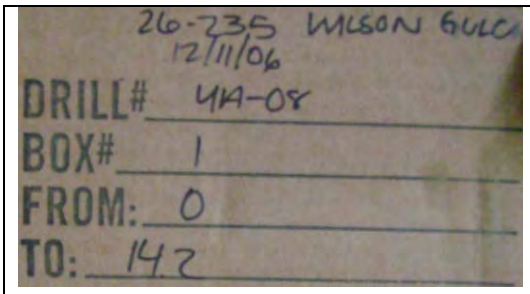

26-255
DRILL# YA-06
BOX# 3
FROM: ±23
TO: 242
WILSON GULCH
12/6/06

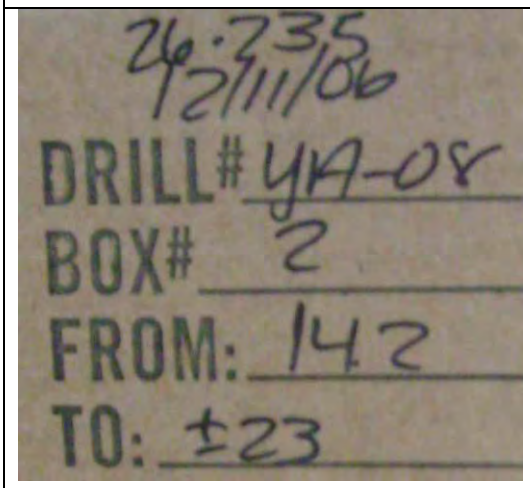

26-235 Wilson Gulch
12/8-12/9/06
DRILL# YA-07
BOX# 1
FROM: 6
TO: ±14

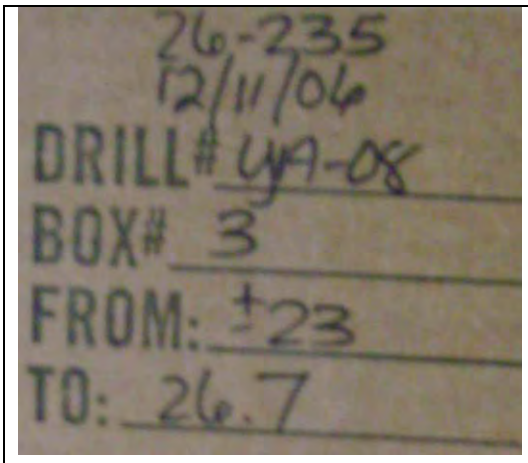
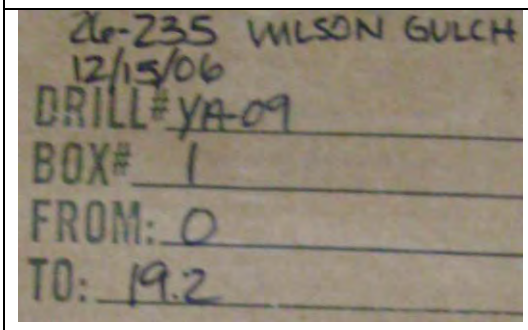

26-235 WILSON GULCH
12/8-12/9/06
DRILL# YA-07
BOX# 2
FROM: ±14
TO: 21.7

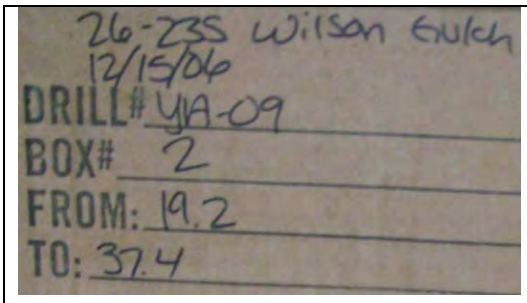

26-235 WILSON GULCH
12/9/06
DRILL# YA-07
BOX# 3
FROM: 21.7'
TO: ±29.7'


26-235 WILSON GULCH
12/9/2006
DRILL# YA-07
BOX# 4
FROM: ±29.7'
TO: 31.1'

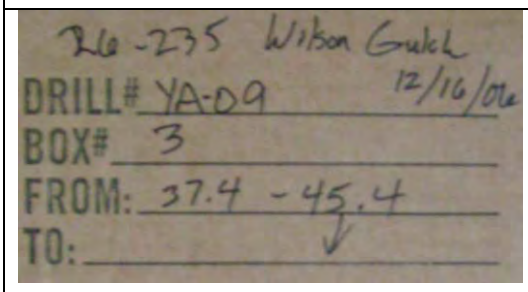

26-235 WILSON GULC
12/11/06
DRILL# YA-08
BOX# 1
FROM: 0
TO: 142


26-235
12/11/06
DRILL# YA-08
BOX# 2
FROM: 142
TO: ±23

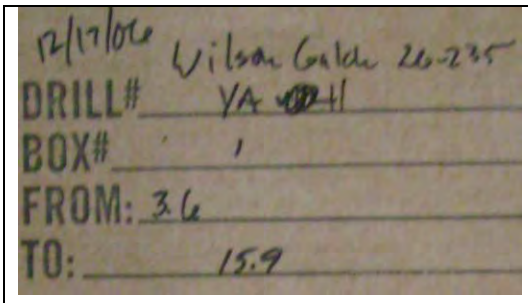
 26-235 12/11/06 DRILL# YA-08 BOX# 3 FROM: ±23 TO: 26.7	
 26-235 WILSON GULCH 12/15/06 DRILL# YA-09 BOX# 1 FROM: 0 TO: 19.2	

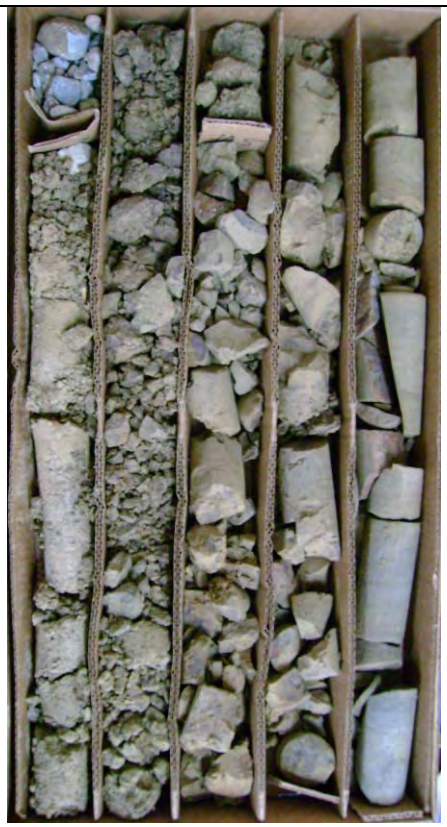

26-235 Wilson Gulch
12/15/06
DRILL# YA-09
BOX# 2
FROM: 19.2
TO: 37.4

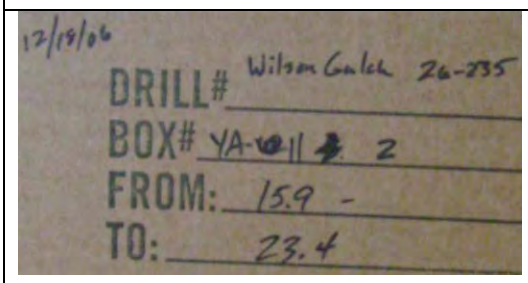
34.2
27
73.5

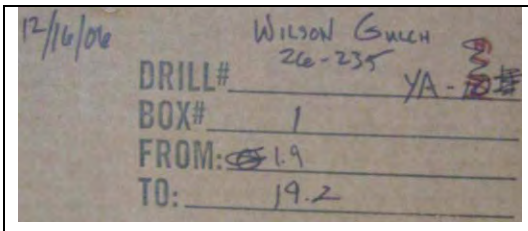

26-235 Wilson Gulch
DRILL# YA-09
12/16/06
BOX# 3
FROM: 37.4 - 45.4
TO:

44.2

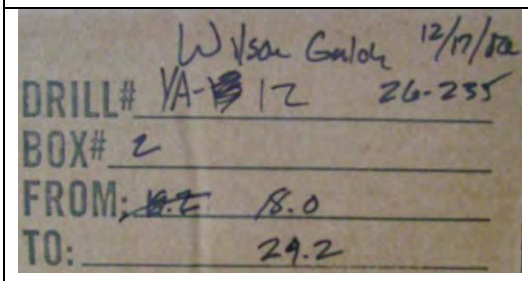

12/17/06 Wilson Gulch 26-235
DRILL# YA-011
BOX# 1
FROM: 3.6
TO: 15.9

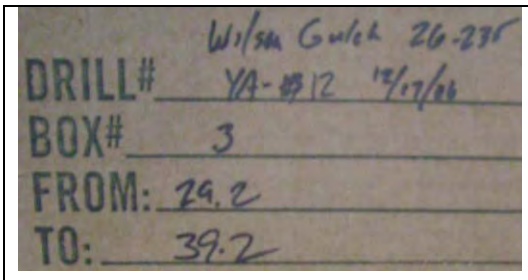



12/18/06
DRILL# Wilson Gulch 26-235
BOX# YA-011 2
FROM: 15.9 -
TO: 23.4

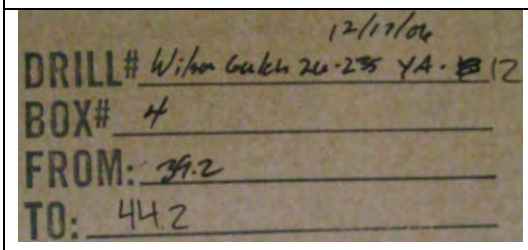

12/16/06
WILSON GULCH
26-235
DRILL# YA-
BOX# 1
FROM: 1.9
TO: 19.2

7.4
16.7

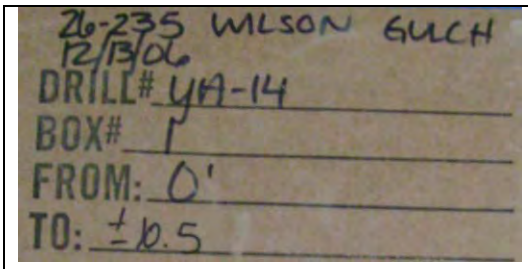

Wilson Gulch 12/17/06
DRILL# YA-12 26-235
BOX# 2
FROM: 8.0
TO: 29.2

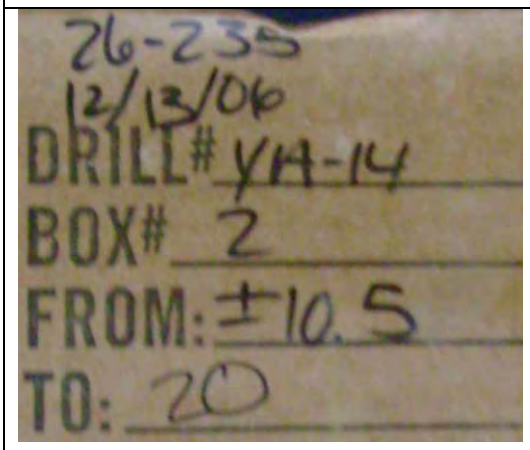

Wilson Gulch 26-235
DRILL# YA-12 12/17/06
BOX# 3
FROM: 29.2
TO: 39.2

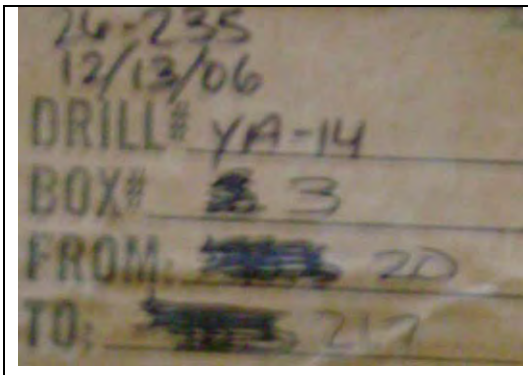
39.2

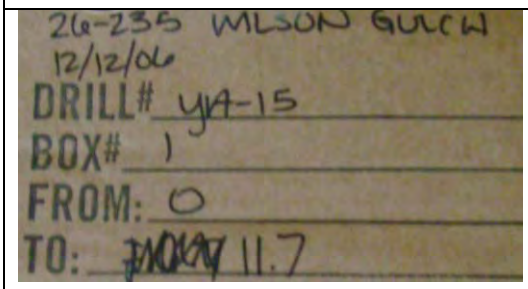

12/17/06
DRILL# Wilson Gulch 26-235 YA-12
BOX# 4
FROM: 39.2
TO: 44.2

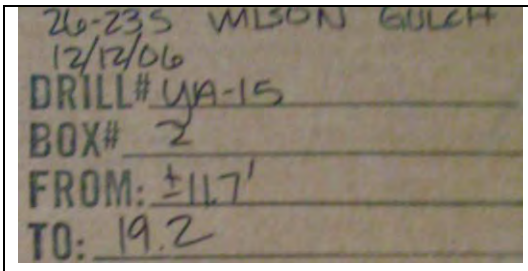
44.2


26-235 WILSON GULCH
12/13/06
DRILL# YA-14
BOX# 1
FROM: 0'
TO: ±10.5


26-235
12/13/06
DRILL# YA-14
BOX# 2
FROM: ±10.5
TO: 20


26-235
12/13/06
DRILL# YA-14
BOX# 3
FROM: 20
TO: 21.7


26-235 WILSON GULCH
12/12/06
DRILL# YA-15
BOX# 1
FROM: 0
TO: 11.7


26-235 WILSON GULCH
12/12/06
DRILL# YA-15
BOX# 2
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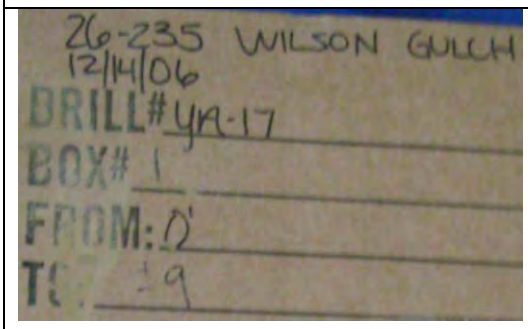
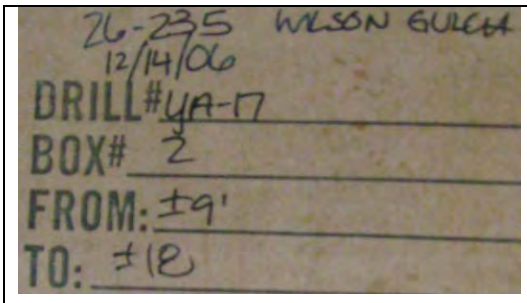
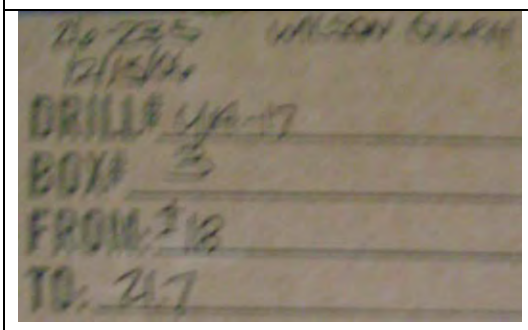
TO: 19.2

14.2
14.6


26-235 WILSON GULCH
12/14/06
DRILL# YA-17
BOX# 1
FROM: 0'
TO: ±9





26-235 WILSON GULCH
26-236 12/14/06
Drill: YA-17
Box: 3
From:18
To: 21.7

Company: 26-235 Property: Grandview
No. Hole: 26 Box No.: 1 of 2
Internal: 17' To: 24

NO. 79738 HQ TOP
COMPANY 26-235
PROPERTY Grandview
HQ. HOLE 25
BOX NO. 1 of 3
INTERVAL 5
TO 37'
10'
15'
20'
25'
30'
37'
5'

NO. 79738 HQ TOP
COMPANY 26-235
PROPERTY Grandview
HQ. HOLE 25
BOX NO. 2 of 3
INTERVAL 37
TO 46'
40'
45'
37'

COMPANY 26-235
PROPERTY Grandview
NO. HOLE 23
BOX NO. 3 of 3
INTERVAL 46'
TO 55'
50'
55'

COMPANY 26-235
PROPERTY Grandview
NO. HOLE 21
BOX NO. 1 of 3
INTERVAL 32'
TO 56'
50'
45'
35'
40'
32'

COMPANY 26-235 PROPERTY Grandview
HQ. HOLE 21 NO. BOX NO. 2 of 3
INTERVAL 50' TO 60'
50'
55'

COMPANY 26-235 PROPERTY Grandview
HQ. HOLE 21 NO. BOX NO. 3 of 3
INTERVAL 60' TO
60'
65'

COMPANY 26-235
PROPERTY Grandview
HQ. HOLE 22
BOX NO. 1 of 4
INTERVAL 51
TO 61'
55'
69'
50'

COMPANY 26-235
PROPERTY Grandview
HQ. HOLE 22
BOX NO. 2 of 4
INTERVAL 61'
TO 68.5'
65'
68.5
61'

COMPANY 26-235
PROPERTY Grandview
NO. HOLE 22
NO.
BOX NO. 3 of 4
INTERVAL 69.5'
TO 79.0'
79.0'
74.5
73.5'
69.5'

COMPANY 26-235
PROPERTY Grandview
NO. HOLE 22
NO.
BOX NO. 4 of 4
INTERVAL 79.0'
TO 79.5'
79.5'
79.0'

COMPANY 26-235 PROPERTY Grandview
HQ. HOLE 18 NO. BOX NO. 1 of 3
INTERVAL 5 TO 11'
15'
10'

COMPANY 26-235 PROPERTY Grandview
HQ. HOLE 18 NO. BOX NO. 2 of 3
INTERVAL 6' TO 26'
25'
20'
16'

COMPANY
PROPERTY
HQ HOLE
NO.
BOX NO.
INTERVAL
TO
35'
30'
25'

……..

Memorandum

MATERIALS AND GEOTECHNICAL BRANCH
GEOTECHNICAL PROGRAM
4670 HOLLY STREET, UNIT A, DENVER, COLORADO 80216 303-398-6512 FAX 303-398-6504

US 160/US 550 Fourth Lane Project
NH-1602-114
PC 16042

US 550 Realignment
NH-5501-014
PC 16575

TO: Ed Archuleta, Region 5 Engineering

FROM: Steve Laudeman, Geotechnical Program

DATE: June 23, 2008

SUBJECT: Addendum No. 3
Final Geotechnical Report
Ramp C Bridge (P-05-W) and US 550 Bridge (P-05-AG)
US 160/US 550 Fourth Lane Project

INTRODUCTION

The Geotechnical Program issued a final report on the geotechnical investigation and recommendations for the Ramp C bridge and US 550 bridge structure foundations for the 4th Lane project on January 9, 2008. Subsequent to the preparation of that report, Region 5 Engineering requested additional drilling at the site of the proposed deep cut for the realignment of US 550. Because the realignment of US 550 will occur at the intersection of two projects, both projects are referenced in the heading above. The results of this additional drilling are described in this Addendum No. 3.

PROJECT DESCRIPTION

The Fourth Lane project includes the addition of an additional through lane on westbound US 160 as well as construction of several bridges for the future reconfiguration of the US 160/US 550 interchange. This project will be executed using a modified design-build process. In addition to the Fourth Lane project, a future project will include the realignment of US 550 south of US 160. This project will require a deep cut adjacent to the south side of US 160. Maximum depth of cut will be approximately 110 feet. The contractor for the Fourth Lane project is planning to use material from the initial US 550 cut as a borrow source. Yeh and Associates has been retained by CDOT Region 5 to perform design and stability analyses on the side slopes of the proposed cut. The investigation described in this memo was intended to provide data on the nature of the material in the cut and design parameters for the stability analysis.

INVESTIGATION

The drilling for this investigation was performed on April 21 thru 24, 2008. Borings were drilled with an ATV mounted CME-550 drill rig using wireline coring drilling methods. Three holes were drilled: B-201, B-202, and B-203. Standard penetration testing (ASTM D 1586) was performed at select intervals in boring B-201. Locations of borings were staked in the field by Region 5 staff prior to drilling. The actual drilled locations of the borings were as close as possible to the staked locations, but some adjustments were made due to drill rig accessibility and underground utility line locations. The as-drilled locations are shown on the attached geology sheets. After drilling, boring B-201 was completed as a temporary piezometer. B-202 was left as an open hole with a temporary cover for safety. At the time of the investigation, it was planned to complete that B-202 as a piezometer. However, the drill crew did not have sufficient materials to complete the piezometer, so the hole was left open. Since it now appears that a piezometer is not necessary at B-202, this hole will be backfilled. B-203 was backfilled with cuttings.

Selected samples of bedrock were tested for moisture content, unit weight, unconfined compressive strength, and shear strength. No samples of the overburden material were submitted for lab analysis. Samples from the core drilling were placed in labeled core boxes and transported to the Durango office of Yeh and Associates for storage.

GEOLOGIC PROFILE

Surface conditions at the project site consist of the steep southern slope of the Wilson Gulch valley. Vegetation includes native grasses, pinon trees, and scrub oak. Land use is primarily natural, with some residential buildings on the property.

The geology of the site generally consists of alluvial deposits overlying interbedded shale and sandstone. At boring B-201, drilling encountered approximately 24.5 feet of stiff to hard clay with sand and gravel, overlying sand with gravel and cobbles from 24.5 to 47 feet, and bedrock from a depth of 47 feet to the total drilling depth of 74.5 feet. At B-202, drilling encountered sand with gravel and cobbles from ground surface to a depth of 42 feet, and bedrock from 42 feet to total drilling depth of 120 feet. At B-203, drilling encountered two feet of gravel road surfacing and native sand and gravel, underlain by weathered bedrock from a depth of 2 feet to total drilling depth of 15 feet. Depth to groundwater at the B-201 piezometer was 47.3 feet below ground surface three days after drilling. Geologic boring logs are attached to this report. The geologic condition are also represented on the attached geology sheets.

Samples of the bedrock were selected for unconfined compressive strength testing and direct shear testing based on the rock type (shale or sandstone) and the degree of weathering. Moisture content and unit weight results are presented in Table 1. Unconfined compressive strength test results are presented in Table 2. Direct shear results are presented in Table 3.

Table 1 - Sample Information and Moisture Content and Unit Weight Results

Boring Number	Sample Number	Depth (ft)	Rock Type	Moisture Content (%)	Dry Unit Weight (pcf)
B201	201A	47 - 48	Shale, slightly weathered	4.8	142.1
B201	201B	64.5 - 66.5	Fine Grained Sandstone, unweathered	6.2	143.5
B202	202A	49 - 49.5	Shale, slightly weathered	11.5	131.7
B202	202B	55 - 56	Shale, unweathered	7.2	137.0
B202	202C	80 - 81	Fine Grained Sandstone, unweathered	7.3	138.1
B202	202D	111 - 112	Fine Grained Sandstone, unweathered	7.3	142.1
B203	203A	10 - 11	Fine Grained Sandstone, weathered	8.3	140.4

Table 2 - Unconfined Compressive Strength Results

Sample Number	Unconfined Compressive Strength (psi)
201B	850
202B	710
202C	1020
202D	1580
203A	1010

Table 3 - Direct Shear Results

Sample Number	Cohesion (psi)	Residual Friction Angle (degrees)
201B	53.5	44.3
202A	14.5	27.5
203A	0.0	33.3

Please contact this office at 303-398-6512 for further geotechnical assistance on this project.

REVIEW: Hsing-Cheng Liu

COPY: Reynolds, Region 5 RTD
Coggins, Region 5 Materials
Powers, Program Engineer
Leonard, Staff Bridge
Archuleta, Resident Engineer
Cross, Region 5 Design
Chomsrimake, Staff Bridge
Allen, Yeh and Associates
Macklin, Yeh and Associates
Zufall, Materials and Geotechnical Branch
Kotzer, Materials and Geotechnical Branch
Liu, Geotechnical Program

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-201**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/21/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,845.7ft	74.5ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6845	0.3	0 - 4" Gravel road surface		0.0		50%
		4" - 19 Clay with sand, trace gravel and cobbles, stiff to hard, brown, dry to moist		4.5 4.5	201A	34 40%
6840				9.5 9.5	201B	11 40%
6835				14.5 14.5	201C	14 40%
6830	19.0	19 - 24.5 Increasing cobble content, cobbles are well rounded, up to three inches diameter, sandstone, granite, metamorphic		19.5 19.5	201D	66 20%
6825	24.5	24.5 - 39 Gravel and cobbles, well rounded, up to 6 inches, occasional clay layers		24.5 24.5	201E	50/3" 30%
6820	26.5	26.5 Bit worn, removed drill steel and replace		26.0		30%
6815				29.5		40%
6810				34.5 34.5		50/4" 20%
				39.5		

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Piezometer installed

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 46.3	▼ 47.3			
DATE	4/23/2008	4/24/2008			
TIME	16:00	12:00			

NOTES: CME 550 wireline, 1" piezometer installed

GEOLOGIC BORING LOG 550 GRAND DIG.GPJ CO_DOT.GDT 6/23/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-201**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/21/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,845.7ft	74.5ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6805		24.5 - 39 Gravel and cobbles, well rounded, up to 6 inches, occasional clay layers *(continued)*				30%
				44.5		30% / 15%
6800						
	47.0	47 - 56 BEDROCK, Shale, weathered to lt brown to greenish brown				
				49.5	201F	69/11"
				49.5		80% / 25%
6795						
				54.5		100% / 50%
6790	56.0	56 - 59.5 Shale, weathered along joints, joints are tight with some FeO fill < 1 mm, horiz to ~60 degrees				
	59.5	59.5 - 64.5 Shale, as above, less weathered, few joints		59.5		100% / 85%
6785						
	64.5	64.5 - 66.5 Sandstone, light grey, v. fine grained		64.5		80% / 70%
6780						
	66.5	66.5 - 74.5 Shale, finely bedded, light grey, intact, no weathering or alteration				
	68.0	68 Soft material due to drilling				
	69.5	69.5 - 74.5 recovered 4 inches of core, lost remainder down hole, shale as above		69.5		7% / 0%
6775						
	74.5	Total Boring Depth 74.5ft				
6770						

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Piezometer installed

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 46.3	▼ 47.3			
DATE	4/23/2008	4/24/2008			
TIME	16:00	12:00			

NOTES: CME 550 wireline, 1" piezometer installed

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-202**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/22/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,812.1ft	120.0ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
6810			0 - 42 Sand with gravel and cobbles, cobbles up to 2 inches in diameter, well rounded, primarliy granitic with some other igneous and metamorphic rock types, small amount of sandstone		0.0		10%		
					5.0		20%		
6805									
					10.0		20%		
6800									
					15.0		30%		
6795									
	18.0		18 - 19 Some weathered shale fragments						
					20.0		30%		
6790									
	25.0		25 - 30 Cobbles up to 3 inches diameter		25.0		50%		
6785									
					30.0		30%		
6780									
					35.0		30%		
6775									

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 wireline

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-202**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/22/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,812.1ft	120.0ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
				40.0		40%
6770	42.0	42 BEDROCK, Shale, weathered down to 43 feet, lt. brown to greenish brown, w/ FeO staining				
	43.0	43 - 50 Shale, intact, light grey, sl weathered 46 - 48, vertical joint, uneven, ~ 1/4 inch filling, FeO staining		45.0		75% / 25%
6765						
	49.0	49 feet, drill steel broke at 2.5 feet bgs, retrieval is successful				
	50.0	50 - 54 V. poor recovery, weathered sandstone, fissile, FeO stained, v. fine grained		50.0		20% / 10%
6760						
	54.0	54 - 55 Sandstone w/conglomerate, clasts up to 1/2"				
	55.0	55 - 57.5 Shale, grey, unweathered		55.0		100% / 45%
6755						
	57.5	57.5 - 60 Fine grained sandstone with small (~1/2 inch) shale clasts, grey, some random joints w/ thin (~1 mm) FeO coating, low RQD due to drilling difficulties				
	60.0	60 - 65 Sandstone, grey, v. fine grained, trace coal		60.0		100% / 100%
6750						
	65.0	65 - 70 V. fine grained sandstone, as above		65.0		100% / 100%
6745						
	70.0	70 - 75 Low RQD due to drilling problems, interbedded sandstone and shale, grey, no evidence of weathering		70.0		50% / 0%
6740						
	75.0	75 - 77.4 Shale, grey, intact		75.0		100% / 100%
6735						
	77.4	77-4 - 84 V. fine grained sandstone, grey, intact				

SPT DATA: 5 10 20 40 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 wireline

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-202**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/22/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,812.1ft	120.0ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
			77-4 - 84 V. fine grained sandstone, grey, intact *(continued)*		80.0		100% / 45%		
6730									
	84.0		84 - 85 Med. grained sandstone, vertical joint w/ FeO filling < 1 mm		85.0		70% / 45%		
	85.0								
	85.1		85 feet, core barrel stuck, pull drill steel						
6725	86.8		85 - 86.8 Fine grained sandstone, grey, intact						
	87.8		86.8 - 87.8 Shale, grey, intact						
			87.8 - 90 Fine grained sandstone, grey, intact						
	90.0		90 - 95 Fine grained sandstone, as above		90.0		100% / 100%		
6720									
	95.0		95 - 100 Fine grained sandstone, as above, steeply inclined joint 97 - 98.5, no alteration, tight		95.0		100% / 85%		
6715									
	100.0		100 - 105 Fine grained sandstone, as above		100.0		100% / 100%		
6710									
	105.0		105 - 107 Fine grained sandstone, as above		105.0		100% / 65%		
6705	107.0		107 - 110 Medium grained sandstone, some small ~1/4 inch clasts, grey						
	108.0		108 - 109 Joint inclined approx 60 degrees, with oxide staining, uneven, intact, sandstone weathered to light brown						
	109.0								
	110.0		109 Horizontal joint, slickensides, oxide coating		110.0		100% / 75%		
6700			110 - 113 Fine grained sandstone, grey, intact						
	113.0								
	113.7		113 Six inch bed of conglomerate, clasts are well rounded, green with some brown, up to 1/2 inch diameter						
	115.0				115.0		100% / 70%		
	115.5		113.7 Planar joint, horizontal, FeO stained, intact, no filling						
6695			115 - 115.5 Shale, intact, grey						
			115.5 - 120 Sandstone, grey, fine to medium grained, some conglomerate with ~1/4 inch diameter clasts at 119						
	119.7								

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 wireline

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-202**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/22/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,812.1ft	120.0ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
	120.0		119.7 Joint, inclined 60 degrees, intact, no stain or weathering Total Boring Depth 120.0ft						
6690									
6685									
6680									
6675									
6670									
6665									
6660									
6655									

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 wireline

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-203**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	Grand Dig	4/24/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 550	La Plata	/	

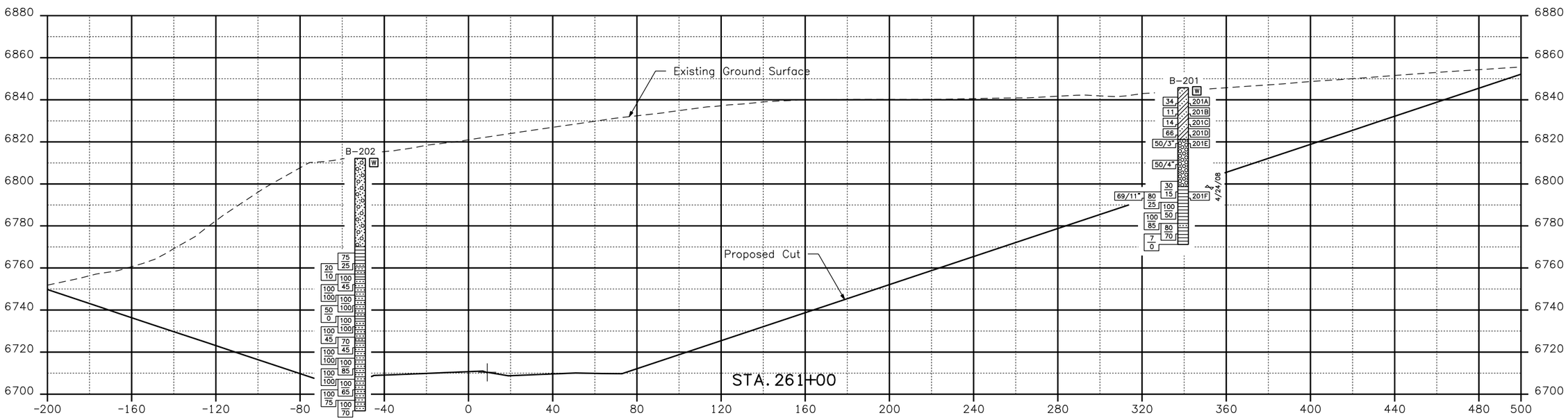
TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,784.6ft	15.0ft		Laudeman/Novak/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
			0 - 2, Gravel surfacing and native (?) gravel and cobbles		0.0		10%		
	2.0		2 - 5, Weathered sandstone, lt. brown with FeO staining						
6780									
	5.0		5 - 10, Interbedded sandstone and shale, highly weathered, lt. brown to greenish brown, joints are uneven, random, FeO stained		5.0		100% / 20%		
6775									
	10.0		10 - 15, Fine grained sandstone, lt. brown, weathered, random joint orientations, joints uneven, oxide stained, some slickensides		10.0		100% / 35%		
6770									
	15.0		Total Boring Depth 15.0ft						
6765									
6760									
6755									
6750									
6745									

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 wireline



The boring logs of the above test holes and geotechnical report are on file in the Geotechnical Program Office, Staff Materials and Geotechnical Branch, (303)398-6601

SUMMARY OF TEST RESULTS

Sample Number	Depth (feet)	Classification: Corps of Engrs. or Visual	Classification: USCS	Classification: AASHTO	Grading Analysis (AASHTO) Percent: Gravel	Coarse Sand	Fine Sand	Silt and Clay	Atterberg Limits: L.L. L_W	P.L. P_W	P.I. I_W	Water Content W %

TYPE OF MATERIAL

- Clay with Sand
- Sand with Gravel & Cobbles
- Gravel & Cobbles
- Shale Bedrock
- Sandstone Bedrock

Note: Boulders may be encountered at any depth

LEGEND

TEST BORING

- 3" Hole Size
- 1A Sample Number
- Blows per foot* 30
  R = Refusal on SPT
  C = California Sample
- 50/0.1
  50 Blows in 0.1 ft
- Water Level
- Core Recovery 50
  R.Q.D. 25
- *Standard Penetration Test (AASHTO T 206-87(2000))

CONTINUOUS PENETRATION TEST

2 Inch Diameter Drive Point
30 Inch Free Fall
140 Pound Hammer

0 50 100
Blows Per Foot

- Location of Test Boring
- Location of Continuous Penetration Test
- W 3 Inch Wireline Boring
- Rotary Boring
- Auger Boring

Print Date: 6/4/2008
Drawing File Name: 12979Geot_Detail0001.dgn
Horiz. Scale: 1:50 Vert. Scale: As Noted
Staff Geotechnical Program HCL

R-X

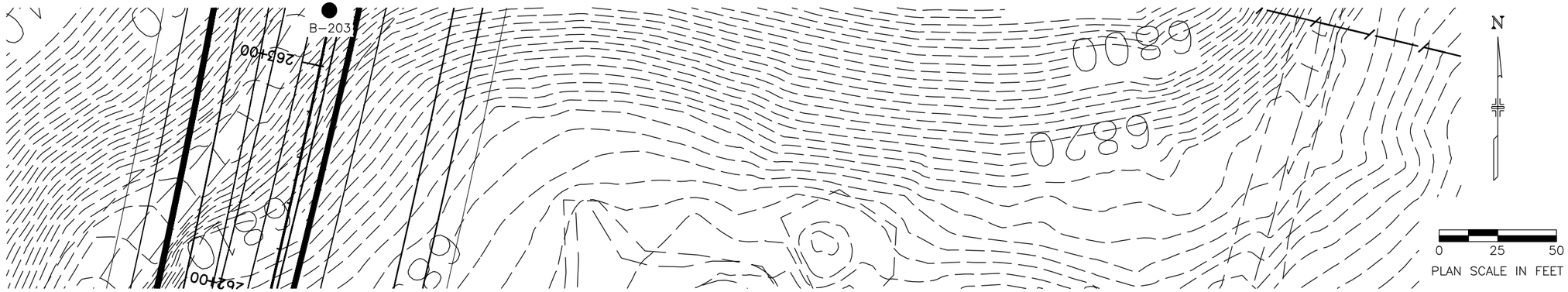
Sheet Revisions		
Date:	Comments	Init.

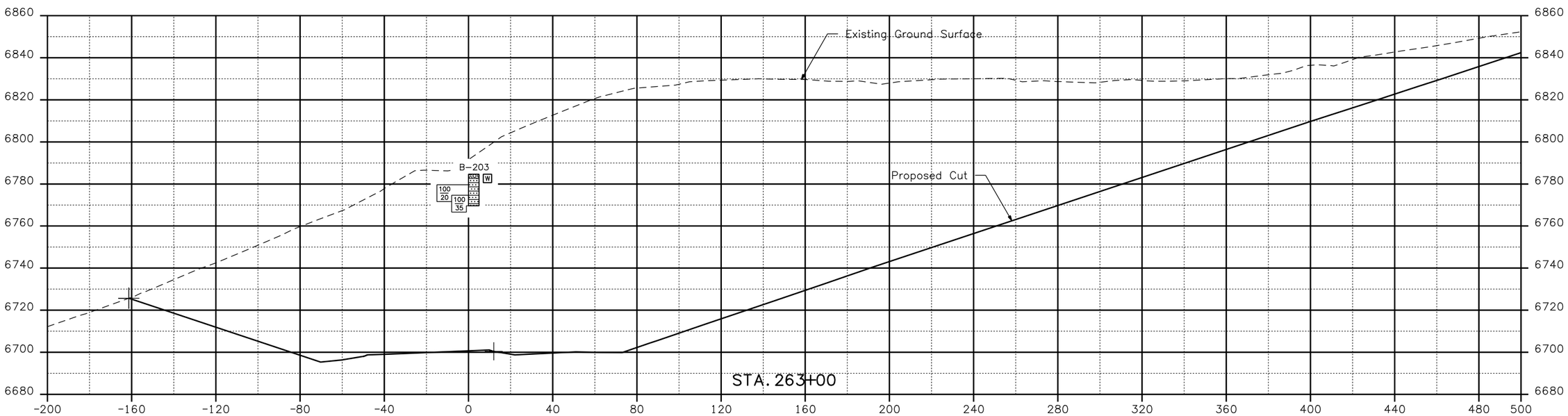
Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
4670 Holly Street, Unit A
Denver, CO 80216
Phone: 303-398-6601 FAX: 303-398-6504
Staff Geotechnical Program HCL

As Constructed
No Revisions:
Revised:
Void:

ENGINEERING GEOLOGY
Designer: S. Laudeman
Detailer: T. McNulty
Sheet Subset: Geology
Structure Numbers
Subset Sheets: XXX of XXX

Project No./Code
NH 1602-114
12979
Sheet Number XXX





The boring logs of the above test holes and geotechnical report are on file in the Geotechnical Program Office, Staff Materials and Geotechnical Branch, (303)398-6601

SUMMARY OF TEST RESULTS

Sample Number	Depth (feet)	Classification: Corps of Engrs. or Visual	Classification: USCS	Classification: AASHTO	Grading Analysis (AASHTO) Percent: Gravel	Coarse Sand	Fine Sand	Silt and Clay	Atterberg Limits: L.L. L_W	P.L. P_W	P.I. I_W	Water Content W %

TYPE OF MATERIAL

Gravel & Cobbles

Weathered Sandstone

Note: Boulders may be encountered at any depth

LEGEND

TEST BORING

Blows per foot* 30
R = Refusal on SPT
C = California Sample
50/0.1
50 Blows in 0.1 ft

3" Hole Size
1A Sample Number
Water Level

Core Recovery 50
R.Q.D. 25

*Standard Penetration Test (AASHTO T 206-87(2000))

CONTINUOUS PENETRATION TEST

2 Inch Diameter Drive Point
30 Inch Free Fall
140 Pound Hammer

0 50 100
Blows Per Foot

Location of Test Boring
Location of Continuous Penetration Test
W 3 Inch Wireline Boring
Rotary Boring
Auger Boring

Print Date: 6/4/2008
Drawing File Name: 12979Geot_Detail0002.dgn
Horiz. Scale: 1:50 Vert. Scale: As Noted
Staff Geotechnical Program HCL

R-X

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
4670 Holly Street, Unit A
Denver, CO 80216
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As Constructed
No Revisions:
Revised:
Void:

ENGINEERING GEOLOGY

Designer: S. Laudeman
Detailer: T. McNulty
Sheet Subset: Geology
Structure Numbers
Subset Sheets: XXX of XXX

Project No./Code
NH 1602-114
12979
Sheet Number XXX

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

SEATTLE
RICHLAND
PORTLAND
FAIRBANKS
ANCHORAGE
DENVER
SAINT LOUIS
SACRAMENTO
JACKSONVILLE

September 11, 2008

Wilson & Company
999 18th Street, Suite 2600
Denver, CO 80202

Attn: Mr. Andy Leifheit

RE: 100% SUBMITTAL, GEOTECHNICAL DESIGN LETTER 004, RAMP C BRIDGE, CDOT PROJECT NO. NH1602-114, DURANGO, COLORADO

This letter provides geotechnical design recommendations for the proposed Ramp C Bridge and supercedes our August 7, 2008, 90 percent submittal.

STRUCTURE DESCRIPTION

The proposed 2-span bridge over Wilson Gulch is designated as Ramp C Bridge, CDOT Structure No. P-05-W (Figure 1). The bridge will trend approximately north-south and be about 29 feet wide. The north abutment is designated as Abutment 1 (Sta. 511+72.5) and the south abutment is designated as Abutment 3 (Sta. 514+71.5). One interior pier, designated Pier 2 (Sta. 513+35), is planned to support the bridge. The abutments and pier will be supported on drilled shaft foundations.

To achieve the proposed roadway grade elevations, fill material will be placed at both abutments. At Abutments 1 and 3, fill heights will approach 24 and 4 feet, respectively. At Abutment 1, the fill embankment will be sloped at 2 Horizontal to 1 Vertical (2H:1V) beneath the bridge and will gradually fan out to 3H:1V on the side of the embankment. We understand that riprap will be utilized on the 2H:1V slopes beneath the bridge.

23-1-01165-003
LR4

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SUBSURFACE CONDITIONS

Our interpretation of subsurface conditions at the bridge site is based on borings drilled by CDOT and others, as well as cone penetrometer tests (CPTs) completed by ConeTec under subcontract to Shannon & Wilson. The approximate location of the borings and the logs for the borings and CPTs are provided in Appendix A. A summary of laboratory test results are presented in Table A-1.

The subsurface profile at the bridge was interpreted from the following borings: 2B40, 2B42, 2B46, B-101, B-102, B-126, CPT-1, and CPT-3. At Abutment 1, subsurface conditions generally consisted of soft to stiff, sandy clay and medium dense to dense sandy gravel with cobbles overlying shale and sandstone bedrock. At Pier 2, the subsurface conditions consisted of soft to medium stiff, sandy clay and dense sandy gravel with cobbles to the top of shale and sandstone bedrock. At Abutment 3, the subsurface conditions generally consisted of medium stiff to stiff, sandy clay from the existing ground surface to the top of shale and sandstone bedrock. The boring logs describe the sandstone as a hard material with foliations and the shale as a hard fractured material with joints. The top of bedrock in the above-referenced borings, ranged from 6,595 to 6,615 feet, with the upper end of this elevation range encountered at Abutment 3.

Groundwater was noted in some of the borings and interpreted from CPT data at elevations ranging from about 6,626 at Abutment 1 to 6,622 feet at Abutment 3. Groundwater levels were recorded in each boring as encountered at the time of drilling. The groundwater elevations used in our analyses are shown in Table 1.

SEISMICITY

We understand that seismic design of the bridge will be in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). Based on data collected from borings at the bridge location, we calculate an average N-value of 16. Using the N-value

method, the soil conditions at the bridge site are consistent with Site Class D, as defined in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim).

The USGS 2007 Seismic Parameters CD, included with the AASHTO LRFD Bridge Design Specifications, was used to calculate the required seismic parameters. For the project site, a latitude of 37.23 and longitude of -107.84 were utilized. For Site Class B, the PGA, S_s, and S_1 are 0.061g, 0.0135g, and 0.041g, respectively for a 7 percent probability of exceedance in 75 years (1,000-year return period).

For Site Class D, the site factors are as follows: F_{pga}= 1.6, F_a= 1.6, and F_v= 2.4. Therefore, for Site Class D, A_s = 0.098g, S_{DS}= 0.216g, and S_{D1}= 0.098g.

Liquefaction, lateral spreading, and dynamic settlement are factors that can negatively affect the stability and load carrying capacity of foundations and structures. In our opinion, because the peak horizontal ground acceleration (PGA) for this area of Colorado is low, there is a low risk potential associated with these seismic hazards, and such seismic hazards were not considered as part of design of the structure.

GEOTECHNICAL RECOMMENDATIONS

Drilled Shafts

We understand that drilled shafts are the preferred foundation support for the abutments and piers. The design criteria presented herein for drilled shafts uses load and resistance factor design (LRFD). The LRFD criteria were developed based on AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). In addition, Federal Highway Administration (FHWA) design methods (O'Neil and Reese, 1999) and a CDOT Research Report 2003-6 (Abu-Hejleh et al., 2003) were used to supplement drilled shaft design procedures.

Axial Capacity Parameters

Axial design parameters for use in sizing bridge foundations, as well as Resistance Factors (ϕ) for both tip and side resistance, are presented in Table 1. The nominal axial capacities in the bedrock are based on Standard Penetration Test (SPT) blow counts, using the methodology presented in Section 6.4.2 in the CDOT Research Report 2003-6. For this method, the shale and sandstone bedrock is defined as a "soil-like claystone". Average blow counts for the shale and sandstone bedrock, as indicated from the borings, were on the order of 50/3 inches. For our analysis, we used 50/5 inches, which would be equivalent to a blow count of 120.

The CDOT Research Report 2003-6 also presents design methodologies for estimating ultimate (nominal) skin and end bearing values for a "very hard sandy claystone" (Section 6.4.4) and "very hard clayey sandstone" (Section 6.4.5). In our opinion, these rock descriptions would be more consistent with the bedrock encountered at the site.

A parametric study between methodologies for "soil-like claystone" (Section 6.4.2), "very hard sandy claystone" (Section 6.4.4), and "very hard clayey sandstone" (Section 6.4.5) was conducted to compare the nominal skin friction and end bearing values determined from each method. An unconfined compressive strength of 1,120 psi (taken from Boring B-126) was used in the study for bedrock, which was the lowest strength value among rock samples tested at this bridge location (see Table A-1). Even though the rock exhibits strength consistent with "very hard sandy claystone" and "very hard clayey sandstone", we used nominal skin friction and end bearing values for the "soil-like claystone" as it yielded the lowest values in comparison to the other methods.

We recommend a minimum drilled shaft tip elevation of Elev. 6589 feet at Abutment 1 and Elev. 6596 feet at Abutment 3, due to low RQD values indicated on the boring logs above these elevations. For drilled shafts that obtain the majority of their axial capacity in rock, no reduction in nominal axial capacity is needed for center-to-center spacings greater than or equal to two shaft diameters. Total settlement at the service design state of individual drilled shafts

that are constructed using good installation techniques and equipment, as described in the 1999 FHWA manual, "Drilled Shafts: Construction Procedures and Design Methods," is anticipated to be about ½ inch. Differential settlement between adjacent piers would be expected to be about 50 to 75 percent of this value. A majority of the settlement is anticipated to occur as the load is applied with the remaining settlement occurring within the first year following construction.

Downdrag Loads

As stated previously, fill material will be placed at the abutment locations. The weight of the embankment fill material will induce consolidation of the underlying cohesive soils and elastic settlement of the granular soils. The elastic settlement will occur relatively rapidly, while the consolidation settlement will occur slowly over time, as discussed in the embankment section below. Depending on the timing of abutment construction and drilled shaft installation, settlement of the existing soils subjected to the abutment loading may add downdrag loads to the drilled shafts.

We understand that embankment at Abutment 1 and the small quantity of fill to be placed at Abutment 3 will be completed prior to drilled shaft installation. For our downdrag analysis, we assumed that the embankment fill materials would be placed a minimum of 6 months prior to drilled shaft installation. At Abutment 1, the sandy clay layer may still be undergoing primary and secondary consolidation at the 6-month timeframe. Therefore, we assumed a downdrag loading at Abutment 1, due to settlement of the embankment fill material from the underlying sandy clay undergoing consolidation. The downdrag load is presented in Table 1. At Abutment 3, our analysis indicate downdrag loads would not need to be considered, as the 4 feet of fill to be placed at the abutment would produce nominal settlement of the underlying sandy clay.

Lateral Load Capacity

We understand that lateral load capacity will be evaluated using the computer program LPILE^{PLUS} by Ensoft, Inc. LRFD design methods for lateral loading of drilled shafts utilize a

Resistance Factor of 1.0 and are controlled by allowable movement criteria. Input parameters for an individual drilled shaft are given in Table 1. Group action can be analyzed using P-multipliers within LPILE^{PLUS}. P-multipliers for loading perpendicular to a line of shafts are presented in Table 1, Note 11. P-multipliers for loading parallel to a line of shafts are presented in Table 2. As an alternative, the computer software Group 6.0 (Reese and Wang, 2003) can also analyze group action.

Abutment Embankments

Based on discussions with Wilson & Co., the embankments will be constructed with material excavated from the construction of Walls P-05-F and P-05-K. We understand that the predominant material that will be excavated from these areas will be a shale-derived soil. Based on our experience with this type of material, we used a unit weight of 130 pounds per cubic foot (pcf), cohesion of 300 psf, and friction angle of 25 degrees in our analysis.

We analyzed the global stability of the native soil/rock under the imposed embankment loadings using PCSTABL slope stability software developed by Purdue University and the Indiana Department of Highways, along with the STEDwin2.90 pre- and post-processor developed by Annapolis Engineering Software. Soil shear strengths used in our analysis were determined from blow counts indicated on the boring logs and data from laboratory and CPT testing.

Three cases were analyzed for the native materials: short-term (undrained), long-term (drained), and seismic. For the seismic case, a horizontal acceleration coefficient of 0.05g was used, based on $A_s = 0.098g$. The embankment stability calculations are provided in Appendix B.

At Abutment 1, the native materials have a factor of safety of 1.2 for short-term loading conditions if the entire embankment is constructed rapidly (without porewater pressure dissipation). Because this lower factor of safety, we recommend installing inclinometers to monitor movement within and below the embankment. We further recommend that the embankment initially be constructed only up to the base of the abutment pilecap prior to the bridge construction, which will reduce the embankment height by roughly 6 feet. If the

instrumentation indicates negligible movement during the construction of the bridge, then fill may be placed to the full embankment height. The native materials at Abutment 1 have a factor of safety of 1.5 for long-term loading and a factor of safety of 1.3 for seismic loading.

At Abutment 3, the small amount of fill being placed, in our opinion, would not induce a short-term (undrained) loading condition. Based on our analysis at Abutment 3, the native materials would have a factor of safety greater than 1.5 for long-term and seismic conditions.

The settlement potential of the fill embankment at Abutment 1 was evaluated. The subsurface conditions generally consisted of about 36 feet of soft to medium dense, sandy clay; overlying 3 feet of medium dense to dense, sandy gravel; overlying shale and sandstone bedrock. We estimate that settlement of these native materials at the center of the abutment due to the weight of the fill will be approximately 11 inches. We anticipate a majority of this settlement will be time-dependent due to the dissipation of excess pore pressures generated during loading. We estimate that (primary) consolidation will take on the order of 11 months. In addition, we anticipate long-term settlement will be approximately 2 inches.

At Abutment 3, the native materials consisted of about 45 feet of medium stiff to stiff, sandy clay overlying bedrock. We estimate nominal settlement should occur at this location due to the relatively small amount of fill being placed.

We estimate that the Abutment 1 embankment itself will also undergo some settlement during or shortly after construction. For properly placed and compacted fill, settlement of embankment fill material should be less than 1/2 percent of the total embankment height. This settlement is anticipated to occur within the first year following construction.

Because of the time-dependent settlement anticipated at Abutment 1, we recommend that instrumentation be installed to monitor settlement with time at Abutment 1.

Sulfates and Corrosion

We reviewed corrosion test results conducted by others to estimate the corrosion potential of the soil/rock. Two samples of sandy clay were tested for water soluble sulfates. The sandy clay was taken from Boring B-101 (Sample 101D) at a depth of 19.6 to 21.1 feet and from Boring B-102 (Sample 102A) at a depth of 5 to 6.5 feet below the ground surface. The test results are summarized in Table A-1.

Corrosion testing was not conducted on rock samples taken at Ramp C Bridge. However, corrosion testing has been performed on rock samples taken at US 550 Bridge and Ramp D Bridge. The boring logs suggest that bedrock encountered at Ramp C Bridge is consistent with that encountered at those two bridges and, in our opinion, the corrosion test results would be applicable for bedrock at Ramp C Bridge.

Based on classifications as defined by the American Concrete Institute (ACI) Manual ACI 318-05 (ACI, 2005), the test results for soil and rock samples suggest a negligible sulfate exposure to concrete in contact with these materials.

CONSTRUCTION CONSIDERATIONS

General

The applicability of the design recommendations provided in this letter is contingent on good construction practice. Poor construction techniques may alter conditions from those on which our recommendations are based, therefore, resulting in reduced foundation capacity and increased settlement. The following sections present additional construction and material considerations for this project.

Drilled Shaft Installation

Based on conditions observed while installing drilled shafts at US 550 Bridge, groundwater inflow should be anticipated above and below top of bedrock. If groundwater inflow is encountered above or within several feet below the top of bedrock, we recommend that casing

be used to seal off local groundwater inflow. Casing should be pushed, rotated, vibrated or driven into an impermeable, firm stratum below the seepage zone. If casing is extended into bedrock, then the length of casing in the rock socket should be discounted from the minimum bedrock penetration length specified on the plans. The shaft should be extended deeper by a length equivalent to the casing protrusion into the bedrock.

If groundwater flow is encountered deeper within the bedrock, such that the use of casing is not possible, we recommend that drilling slurry not be allowed and bedrock exposure to water be kept to the minimum time necessary to complete drilling and place concrete. If the minimum penetration into bedrock and concreting process cannot be completed within the same day that drilling into bedrock initiated, we recommend that the shaft be extended deeper by 1/3 of the minimum bedrock penetration length specified on the plans. If the water depth at the bottom of the shaft is in excess of 2 inches at the time of concrete placement, then concrete should be placed by tremie pipe and pumping in accordance with CDOT Standard Specifications, Subsection 601.12(f), Depositing Concrete Under Water.

Upon completion of the shaft excavation, the hole should be cleaned of loose material and observed by the engineer prior to pouring. The drilling and concreting process should be relatively continuous with minimal stoppage of work between the completion of drilling and cleaning the hole and placement of concrete after setting the rebar cage.

Frictional and end-bearing design parameters used in this analysis assume that the bottom of the shaft is properly cleaned and proper concrete placement techniques are used. To provide sufficient frictional resistance in rock, the specifications should require that the driller make a final pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7 inches from the sides of the auger to roughen the rock socket. The primary purpose of the roughening is to reduce the polished skin of the remolded material that can sometimes form in softer bedrock. This will be particularly important for drilled shafts with bedrock exposed to water.

For the construction of the drilled shafts, we recommend that the field installation reports provide the actual elevation at which bedrock is encountered and tip elevation for each drilled shaft. The blowcount values provided in the boring logs indicate that the shale and sandstone bedrock is a hard material which may create hard drilling conditions. In addition, cobbles and boulders, as inferred from the boring logs, may be present above bedrock at Abutment 1 and Pier 2. Therefore, we recommend that the strength properties of the bedrock and the possibility of encountering cobbles and boulders be disclosed to drilling subcontractors.

Embankment and Subgrade Recommendations

Prior to placing fill, the subgrade soils should be proof-rolled with a large, heavy, vibratory roller or equivalent. Any loose, soft, or yielding areas should be overexcavated and replaced to the design base of wall elevation with Structure Backfill (Class 1) that is compacted to a minimum of 95 percent of its Modified Proctor dry density (AASHTO T-180).

Excavation for embankment, embankment foundation preparation, and embankment construction behind the MSE wall should conform to the requirements of the latest edition of the CDOT Standard Specifications for Road and Bridge Construction. Embankment material should be compacted to a minimum of 95 percent of its Modified Proctor (AASHTO T-180) dry density for cohesionless soils or Standard Proctor (AASHTO T-99) dry density for cohesive soils.

CLOSURE

The analyses, conclusions, and recommendations presented in this letter are based on a limited number of subsurface explorations and laboratory test results. We assume that these explorations are representative of the subsurface conditions beneath the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

This letter was prepared for the exclusive use of Wilson & Company and SEMA for use in design of Ramp C Bridge. It should be not be construed as a warranty of subsurface conditions.

Within the limitations of scope, schedule and budget, the analyses, conclusions and recommendations presented in this letter were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this letter was prepared. We make no other warranty, either express or implied.

Sincerely,

SHANNON & WILSON, INC.

Michael C. Hallman, P.E.
Senior Engineer

COLORADO REGISTERED
GREGORY ROBERT FISCHER
34461
PROFESSIONAL ENGINEER
9/11/08

Gregory R. Fischer, Ph.D., P.E.
Senior Vice President

MCH:GRF/mch

Enclosures: Table 1– Geotechnical Parameters for Design of Bridge Foundations at Ramp C Bridge
Table 2 – LPILE^{PLUS} Efficiency Factors for Loading Acting Parallel to the Line of Shafts
Figure 1 – Ramp C over Wilson Gulch General Layout Sheet
Appendix A – Subsurface Explorations
Appendix B – Embankment Stability Calculations

REFERENCES

American Concrete Institute (ACI), 2005, Building Code Requirements for Structural Concrete and Commentary, Farmington Hills, Michigan, ACI 318-05.

Abu-Hejleh, Naser, O'Neill, M.W., Hanneman, Dennis, Atwooll, W.J., 2003, Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks, Final Report: Colorado Department of Transportation, Research Branch, Report no. CDOT-DTD-R-2003-6, July.

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O'Neil, M.W., and Reese, L. C., 1999, Drilled shafts: Construction Procedures and Design Methods: Washington, D.C., FHWA-IF-99-025.

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TABLE 1
GEOTECHNICAL PARAMETERS FOR DESIGN OF BRIDGE FOUNDATIONS AT RAMP C BRIDGE

Structure	Location	Applicable Borings	Layer Elevation		Soil / Rock Type	Effective Unit Weight	Friction Angle[1]	Undrained Shear Strength	Drilled Shafts[3,4]							LPILE[PLUS] Ver. 4.0 Parameters for Lateral Analyses[1,7,8,9,10,11]			
									Skin Friction			Resistance Factors[2]		Downdrag[5,6]					
			Top	Bottom					k*tanδ	Ultimate Unit Adhesion	Ultimate Unit End Bearing	Skin Friction	End Bearing	Load per Shaft	Maximum Depth of Downdrag	Soil Model Type	Strain at 50% Yielding	Initial Static Modulus of Subgrade Reaction	Initial Cyclic Modulus of Subgrade Reaction
			(feet)	(feet)		γ' (pcf)	ϕ' (deg)	s_u (psf)		(ksf)	(ksf)			(kips)	(ft)		ε_{50}	k_s (pci)	k_c (pci)
Ramp C Bridge	Abutment 1 (North)	B-102, B-126, CPT-1, CPT-3, 2B46	6,666	6,642	New Fill	130	34	--	Ignore resistance because of downdrag					350	6,606	Sand	--	90	90
			6,642	6,626	Medium stiff to stiff, sandy CLAY	115	--	1,200								Stiff Clay w/o Free Water	0.009	300	120
			6,626	6,618	Medium stiff to stiff, sandy CLAY	53	--	1,200								Stiff Clay w/o Free Water	0.009	300	120
			6,618	6,606	Soft to medium stiff, sandy CLAY	53	--	750								Soft Clay	0.01	--	--
			6,606	6,603	Medium dense to dense, sandy GRAVEL w/ cobbles	58	36	--	0.31	--	--	0.55	--			Sand	--	60	60
			6,603		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Pier 2	2B40, 2B42	6,622	6,601	Soft to medium stiff, sandy, silty CLAY	53	--	750	--	--	--	--	--	No Downdrag		Soft Clay	0.01	--	--
			6,601	6,595	Dense, sandy GRAVEL w/ cobbles	63	38	--	0.35	--	--	0.55	--			Sand	--	90	90
			6,595		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Abutment 3 (South)	B-101, 2B40	6,664	6,626	Medium stiff to stiff, sandy, silty CLAY	115	--	1,000	--	0.8	--	0.45	--	No Downdrag		Stiff Clay w/o Free Water	0.009	300	120
			6,626	6,615	Medium stiff to stiff, sandy, silty CLAY	53	--	1,000	--	0.8	--	0.45	--			Stiff Clay w/o Free Water	0.009	300	120
			6,615		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800

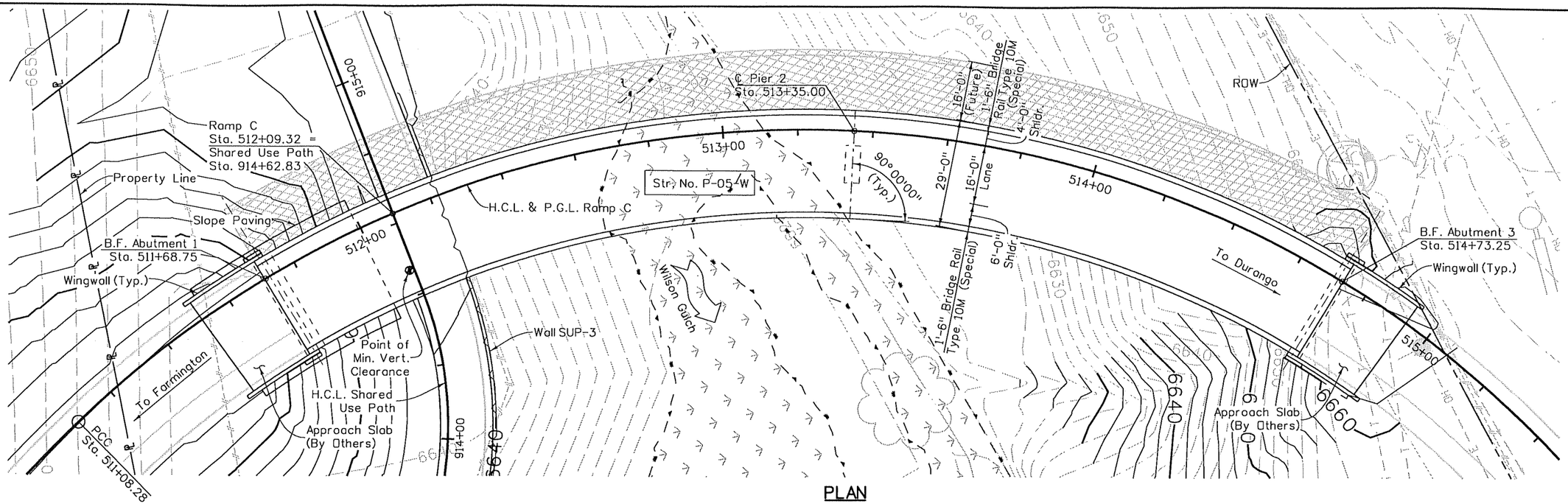
Notes:

1) Friction angle of 34 degrees used for new fill material is an equivalent friction angle for using the "Sand" model in LPILE[PLUS].
2) If a non-redundant, single shaft is used to support a bridge pier, the resistance factors should be reduced by 20 percent. For two closely spaced shafts, redundancy should be evaluated by the structural engineer. A resistance factor of 1.0 should be used for the service and extreme limit states. For uplift resistance, a resistance factor of 0.8 should be used for the extreme limit state.
3) Minimum bedrock penetration for all drilled shafts should be 3 diameters. We recommend a minimum tip elevation of Elev. 6589 feet for Abutment 1 and Elev. 6596 feet at Abutment 3.
4) For drilled shafts that obtain the majority of the their axial capacity in rock, no reduction in axial capacity is needed for center-to-center spacings greater than or equal to 2 diameters.
5) Calculation of downdrag load assumes a 36-inch-diameter drilled shaft.
6) The downdrag should be treated as a load with the appropriate load factors applied.
7) The lateral loading analysis should consider the potential for flooding and scour.
8) The LPILE[PLUS] parameters shown are for horizontal ground surface. Sloping ground surface modifications should be included as per Ensoft, Inc.'s recommendations for the LPILE[PLUS] program.
9) The lateral resistance values provided assume that the lateral loads are short-term (non-sustained) loads. If sustained lateral loads are anticipated, we should be notified to provide additional parameters.
10) A resistance factor of 1.0 should be used for lateral analyses.
11) For tangent shafts where lateral loading is perpendicular to line of shafts, a P-multiplier of 0.5 should be used; for shaft spacing greater than or equal to 3 diameters, center-to-center, no P-multiplier is required. For spacings between 1 and 3 diameters, the P-multiplier can be interpolated between 0.5 and 1.0, respectively. For lateral loading parallel to the line of shafts refer to Table 2.

TABLE 2 - LPILE[PLUS] EFFICIENCY FACTORS FOR LOADING ACTING PARALLEL TO THE LINE OF SHAFTS

Spacing	P-multiplier
2.0 D	0.28
2.5 D	0.50
3.0 D	0.72
3.5 D	0.85
4.0 D	0.91
4.5 D	0.94
5.0 D	0.98
5.5 D	1.00

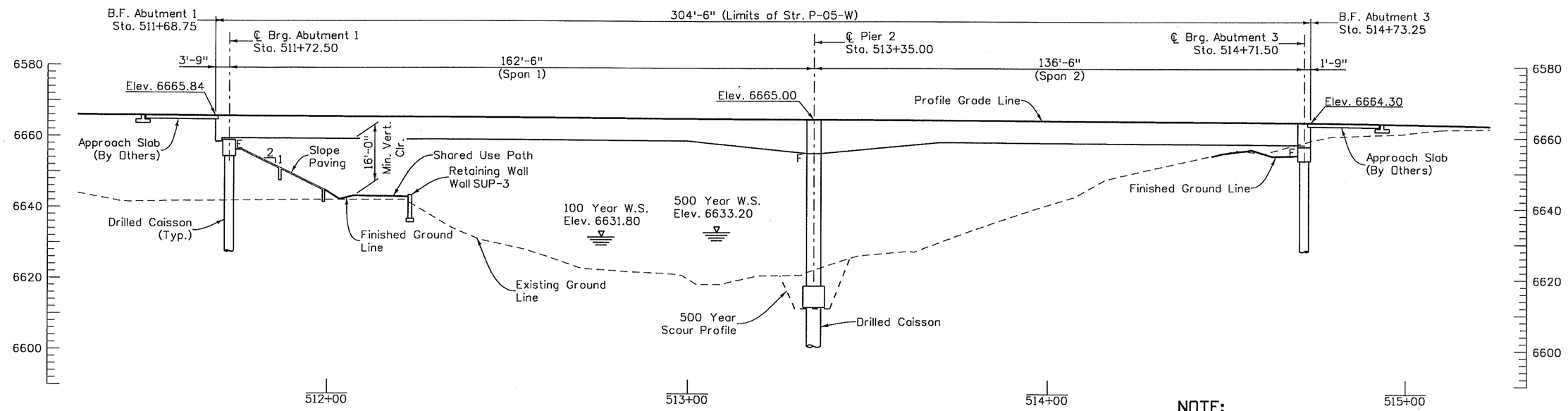
NOTE: The p-multipliers are based on recommendations presented in a 1998 Ensoft Seminar (Ensoft, 1998)



PLAN

RAMP C HORIZONTAL CURVE DATA

Δc = 106° 48'35"
Lc = 512.65'
Rc = 275.00'



SECTION

(Taken Along PGL)

NOTE:
Elevations are taken at top of 3" HMA wearing surface.

NOT FOR CONSTRUCTION - 30% PLANS - 07/28/08

Design	INITIAL	DATE
Designed By	XXX	MM/YY
Checked By	XXX	MM/YY

Detail	INITIAL	DATE
Detailed By	XXX	MM/YY
Checked By	XXX	MM/YY

Quantities	INITIAL	DATE
Quantities By	XXX	MM/YY
Checked By	XXX	MM/YY

Print Date: 7/28/2008
File Name: 16042W_GenLayout_01.dgn
Horiz. Scale: 1:30 Vert. Scale: As Noted
Unit Information 0221 Unit Leader STW

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
CDOT DEPARTMENT OF TRANSPORTATION
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

RAMP C OVER WILSON GULCH
GENERAL LAYOUT

Designer: A. Leifheit
Detailer: D. Anderson
Sheet Subset: Bridge
Structure Numbers: P-05-W
Subset Sheets: B3 of XX

Project No./Code
NH 1602-114
16042
Sheet Number

FIG. 1

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

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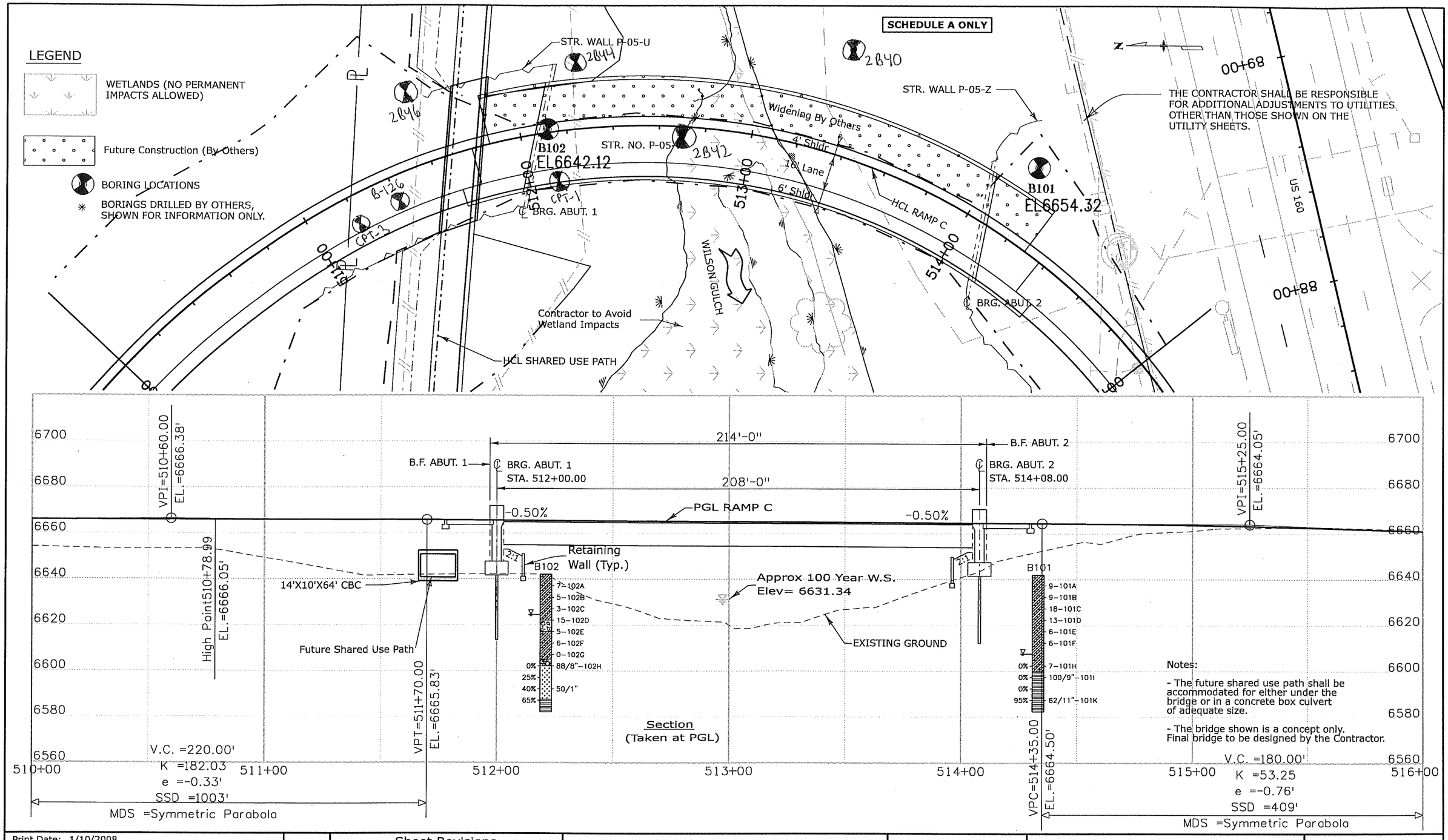
A-6 Log of CPT-1
A-7 Log of CPT-3

TABLE A-1
SUMMARY OF LABORATORY TEST RESULTS BY BORING

SAMPLE DATA		USCS Symbol	AASHTO Symbol	Natural Water Content	Wet Density	Dry Density	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION			STRENGTH TESTING		CORROSION TESTING
Boring	Depth						Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	% Fines	Unconfined Compressive Strength	Strain at UCS	Water Soluble Sulfates
				(%)	(pcf)	(pcf)	(%)	(%)	(%)				(psi)	(%)	(%)
B-101	19.6 - 21.1	CL	A-6 (8)	20.7			34	22	12	4.8	20.4	74.8			0.0
	57.5 - 58.3					153.7							1,732	0.8	
B-102	5 - 6	CL	A-6 (12)	25.0			38	21	17	0.6	22.5	76.9			0.01
	35 - 36.5	CL	A-7-6 (19)	33.1			44	22	22	0.3	14.5	85.2			
	52.5 - 53.3					160.2							8,843	1.0	
B-126	48.9 - 49.5				153.3								1,120	---	



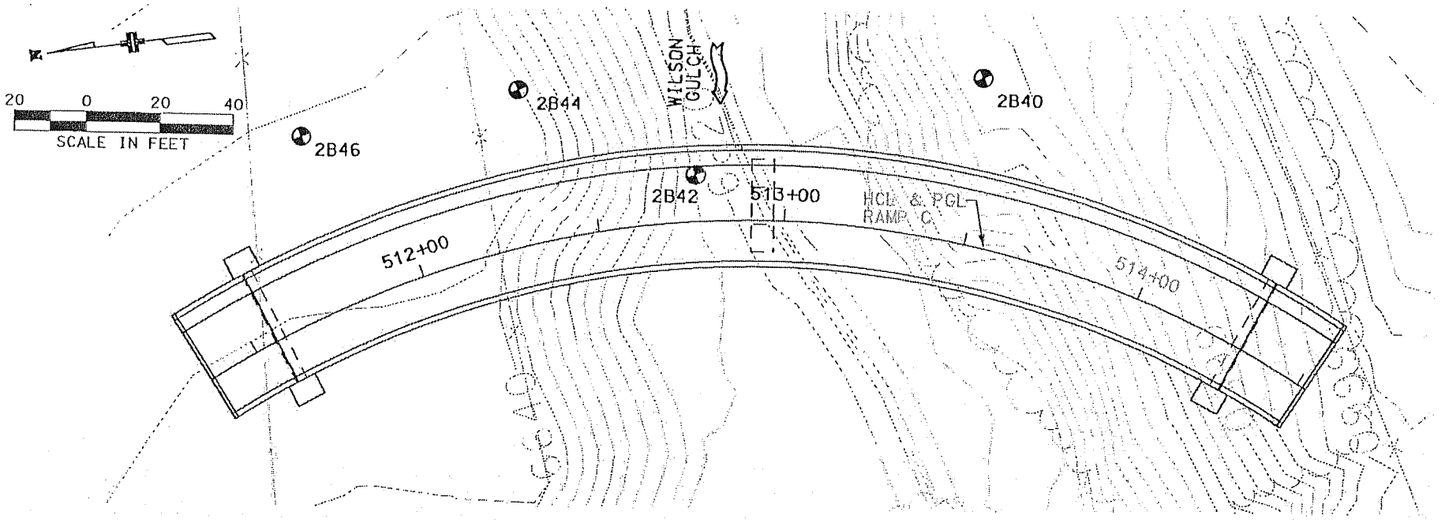
Notes:

- The future shared use path shall be accommodated for either under the bridge or in a concrete box culvert of adequate size.
- The bridge shown is a concept only. Final bridge to be designed by the Contractor.

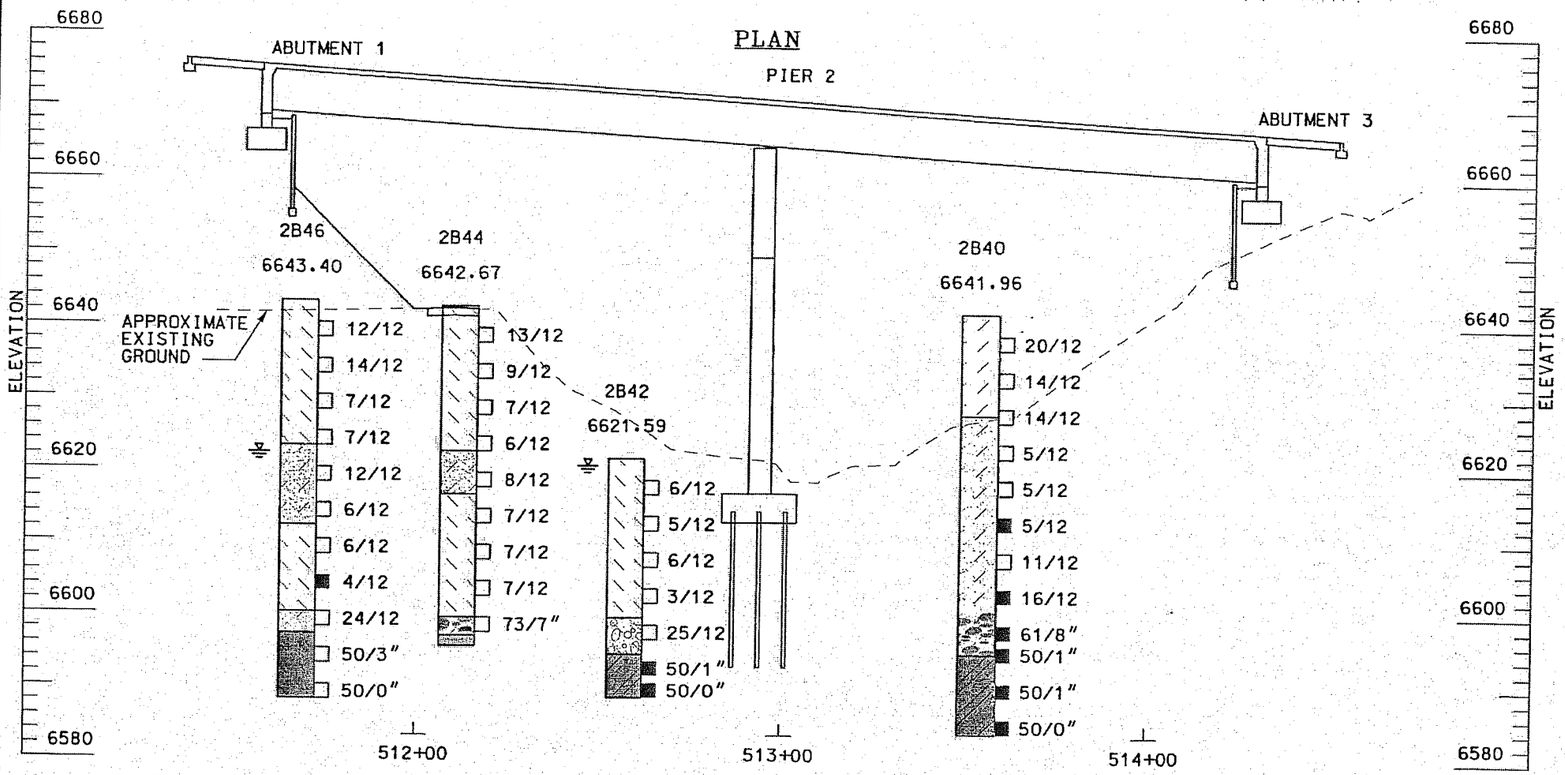
Print Date: 1/10/2008	Sheet Revisions	Colorado Department of Transportation	As Constructed
File Name: Ramp C Gen Layout.dgn	Date: / Comments / Init.	3803 North Main Avenue, Suite 200, Durango, CO 81301	No Revisions:
Horiz. Scale: 1:40 Vert. Scale: As Noted	R-X	Phone: 970-385-1440 FAX: 970-385-8365	Revised:
Unit Information Unit Leader Initials		Region 5 EJA	Void:

RAMP C (P-05-W) GENERAL LAYOUT			Project No./Code
Designer: spc	Structure Numbers	P-05-W	NH-1602-114
Detailer: spc			16042
Sheet Subset: Bridge	Subset Sheets: 3 of 4		Sheet Number 22

FIG. A-1



PLAN



SECTION
(TAKEN AT PGL)

LEGEND

- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, DARK GREY TO DARK BROWN (CL)
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SILTY TO VERY SILTY, STIFF TO VERY STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY TO MEDIUM BROWN (CH)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY SILTY, SLIGHTLY GRAVELLY, LOOSE TO MEDIUM DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, MEDIUM DENSE TO DENSE, MOIST, BROWN (GP, GM, GC)
- COBBLES AND BOULDERS
- CLAYSTONE, TRACE SILT TO SILTY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

Computer File Information		Index of Revisions				
Creation Date: Initials:	R–					
Last Modification Date: SEE PLOT STAMP Initials:	R–					
Full Path: SEE PLOT STAMP	R–					
Drawing File Name: SEE PLOT STAMP	R–					
Microstation Ver. V8 Scale: AS SHOWN Units:	R–					

DOT DEPARTMENT OF TRANSPORTATION Region 5

URS
10 Burnett Court Suite 113
Durango, Colorado 81301
(970) 259-9212
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As Constructed	RAMP C ENGINEERING GEOLOGY			Project No./Code
No Revisions:				FC-NH(CX)160-2(048)
Revised:	Designer: K. YOUNG	Structure Numbers	P-05-W	91050
Void:	Detailer: R. PIKE			
	SheetSubset:	Subset Sheets:		Sheet Number

FIG. A-2

PLOTTED BY: rpplikex0
DATE: 04/25/2005
FILE NAME: n:\projects\21711361_US_160\Sub_00\09.0_CAD\Plansht s\Structures\P05W\P05WEG1.dgn

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-101	NH 1602-114	12979	160 Fourth Lane	11/6/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	Ramp C Bridge/South Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,654.3ft	59.6ft	N1213458.47 E2320509.14	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
			0.0		
6650	4.6	4.6 - 6.1 sandy clay, brown, some roots, stiff	4.6 4.6	101A 3-4-5	9
6645	9.6	9.6 - 11.1 same as above, w/ some coal fragments and roots	9.6 9.6	101B 3-4-5	9
6640	14.6	14.6 - 16.1 sandy clay, lt. brown, dry, v. stiff	14.6 14.6	101C 5-8-10	18
6635	19.6	19.6 - 21.1 same as above	19.6 19.6	101D 3-5-8	13
6630	24.6	24.6 - 26.1 same as above, loose	24.6 24.6	101E 4-2-4	6
6625	29.6 29.7	29.6 - 31.1 same as above 29.6 - 34.6 no recovery from core barrel	29.6 29.6	101F 2-2-4	6
6620			34.6		
6615	39.6	39.6 - 41.1 low N due to sample loss from core barrel, fine sand, same as above	39.6 39.6	101H 3-4-3	7 50% 0%
	42.0	42 bedrock, weathered shale, v. slow drilling			
6610	44.6		44.6		

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Install piezometer

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▼ 35.4	▽ 34.4			
DATE	11/6/2007	11/8/2007			
TIME	13:30	10:00			

NOTES: CME 550 Wireline

FIG. A-3
(Page 1 of 2)

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-101**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	160 Fourth Lane	11/6/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	Ramp C Bridge/South Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,654.3ft	59.6ft	N1213458.47 E2320509.14	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
	44.7	44.6 - 46.1 weathered bedrock, shale, dark grey, very fractured, dry, v. hard 44.6 - 49.6 weathered shale bedrock as above, very fractured 42 bedrock, weathered shale, v. slow drilling *(continued)*		44.6	101I	100/9" 27-50-50/3" 50% 0%
6605	49.6	49.6 - 54.6 weathered claystone as above		49.6		50% 0%
6600	54.6 54.7	54.6 - 56.1 bedrock as above, v. hard 54.6 - 59.6 shale bedrock, unweathered, dry, grey		54.6 54.6	101K	62/11" 12-50/5" 100% 95%
6595	59.6	Total Boring Depth 59.6ft				
6590						
6585						
6580						
6575						
6570						
6565						

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Install piezometer

SPT	CON'T	GRAB	SHELBY	CORE	
H_2O DEPTH	▼ 35.4	▽ 34.4			
DATE	11/6/2007	11/8/2007			
TIME	13:30	10:00			

NOTES: CME 550 Wireline

FIG. A-3
(Page 2 of 2)

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-102	NH 1602-114	12979	160 Fourth Lane	11/6/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	Ramp C Bridge/North Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,642.1ft	60.0ft	N1213670.54 E2320525.46	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
				0.0		
6640						
	5.0	5 - 6.5 clay, sandy, brown, dry, med. stiff		5.0 5.0	102A 4-3-4	7
6635						
	10.0	10 - 11.5 same as above		10.0 10.0	102B 3-2-3	5
6630						
	15.0	15 - 16.5 same as above		15.0 15.0	102C 2-1-2	3
6625						
	20.0	20 - 21.5 sand and gravel, med. dense, large particles are shale		20.0 20.0	102D 5-7-8	15
6620						
	25.0	25 - 26.5 clay, dk. grey to brown, med. stiff		25.0 25.0	102E 1-2-3	5
6615						
	30.0	30 - 31.5 clay, grey, moist, med. stiff		30.0 30.0	102F 0-3-3	6
6610						
	35.0	35 - 36.5 same as above, v. soft		35.0 35.0	102G 0-0-0	0
6605	37.0	37 gravel and cobbles up to 3"				
	40.0	40 bedrock, sandstone, dk. grey, v. hard, dry, fissile, v. fine grained		40.0 40.0	102H 38-50/2"	88/8" 100% 0%
6600						
	44.0	highly fractured, joints primarily horizontal w/				

SPT DATA (N-VALUE): 5 10 20 40 70

WELL DIAGRAM: Backfilled with cuttings

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 17.5
DATE	11/7/2007
TIME	10:10

NOTES: CME 550 Wireline

FIG. A-4
(Page 1 of 2)

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-102	NH 1602-114	12979	160 Fourth Lane	11/6/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	Ramp C Bridge/North Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,642.1ft	60.0ft	N1213670.54 E2320525.46	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
			some ~80 degrees from horizontal, joint surfaces intact to highly weathered		45.1		100% / 25%		
6595	47.0		40 bedrock, sandstone, dk. grey, v. hard, dry, fissile, v. fine grained *(continued)*						
	48.0		thin coal layers						
			conglomerate, clasts up to 1", pink to grey		50.0 / 50.0	50/1"	50/1" / 100% / 40%		Backfilled with cuttings
6590									
	54.5		54.5 shale, grey, fissile, some joints ~80 degrees from horizontal, slightly altered		55.0		100% / 65%		
6585									
	60.0		Total Boring Depth 60.0ft						
6580									
6575									
6570									
6565									
6560									
6555									

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 17.5				
DATE	11/7/2007				
TIME	10:10				

NOTES: CME 550 Wireline

FIG. A-4
(Page 2 of 2)

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-126	NH 1602-114	16042	Fourth Lane	7/9/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-W Ramp C Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,641.3ft	59.5ft	N1213707.1 E23220506.48	Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6640			0 - 35 Sandy clay, brown, moist, medium stiff to stiff		0.0		
6635					4.5 4.5	126A CAL	6
6630					9.5 9.6	126B CAL	7
6625	15.0		15 Trace root fragments		14.5		
6620	19.5		19.5 No recovery on drive sample		19.5 19.5	NR CAL	5
6615					24.5		
6610	29.5		29.5 No recovery on drive sample		29.5 29.5	NR CAL	12
6605	35.0		35 - 38 Gravel, particles up to 1.5 inches, angular to subrounded		34.5		10% 0%
	38.0				39.5		

SPT DATA: 5 10 20 40 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-5 (Page 1 of 2)

GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/29/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-126**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	Fourth Lane	7/9/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-W Ramp C Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,641.3ft	59.5ft	N1213707.1 E23220506.48	Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
6600			38 - 59.5 Sandstone, medium grained, grey to brown, moderately weathered, severely jointed, oxide staining on joint surfaces, joints primarily horizontal *(continued)*				75% / 0%		
					44.5		75% / 10%		
6595	47.0		47 Sandstone as above, less weathered						
	49.5		49.5 Sandstone, as above, intact, some minor joints, no evidence of weathering		49.5		90% / 80%		
6590									
					54.5		100% / 90%		
6585									
	59.5		Total Boring Depth 59.5ft						
6580									
6575									
6570									
6565									

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-5 (Page 2 of 2)

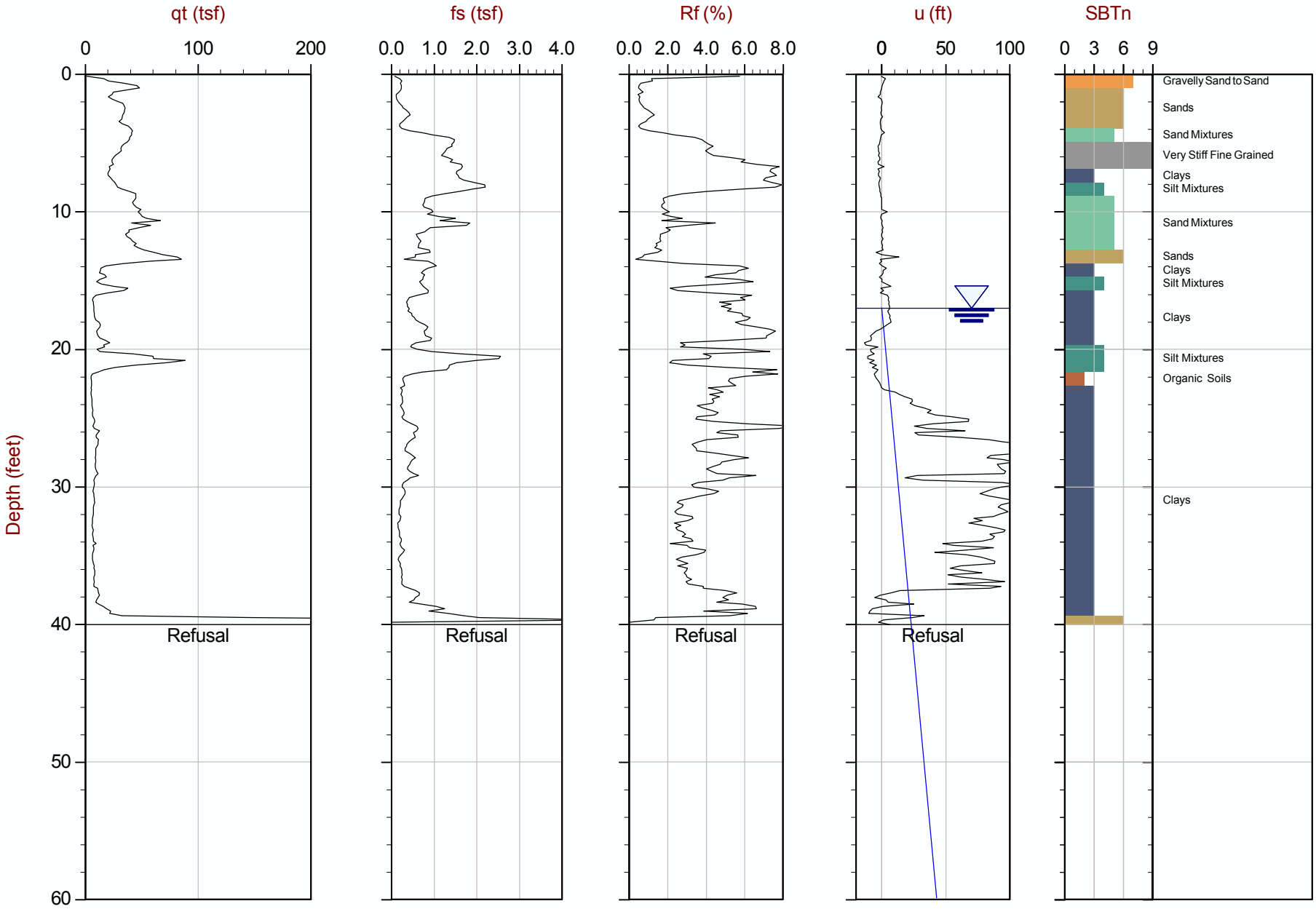
GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/29/08

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 01:33
Site: US160 / US550

Sounding: CPT-1
Cone: STD 20T AD214



Max Depth: 12.200 m / 40.03 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

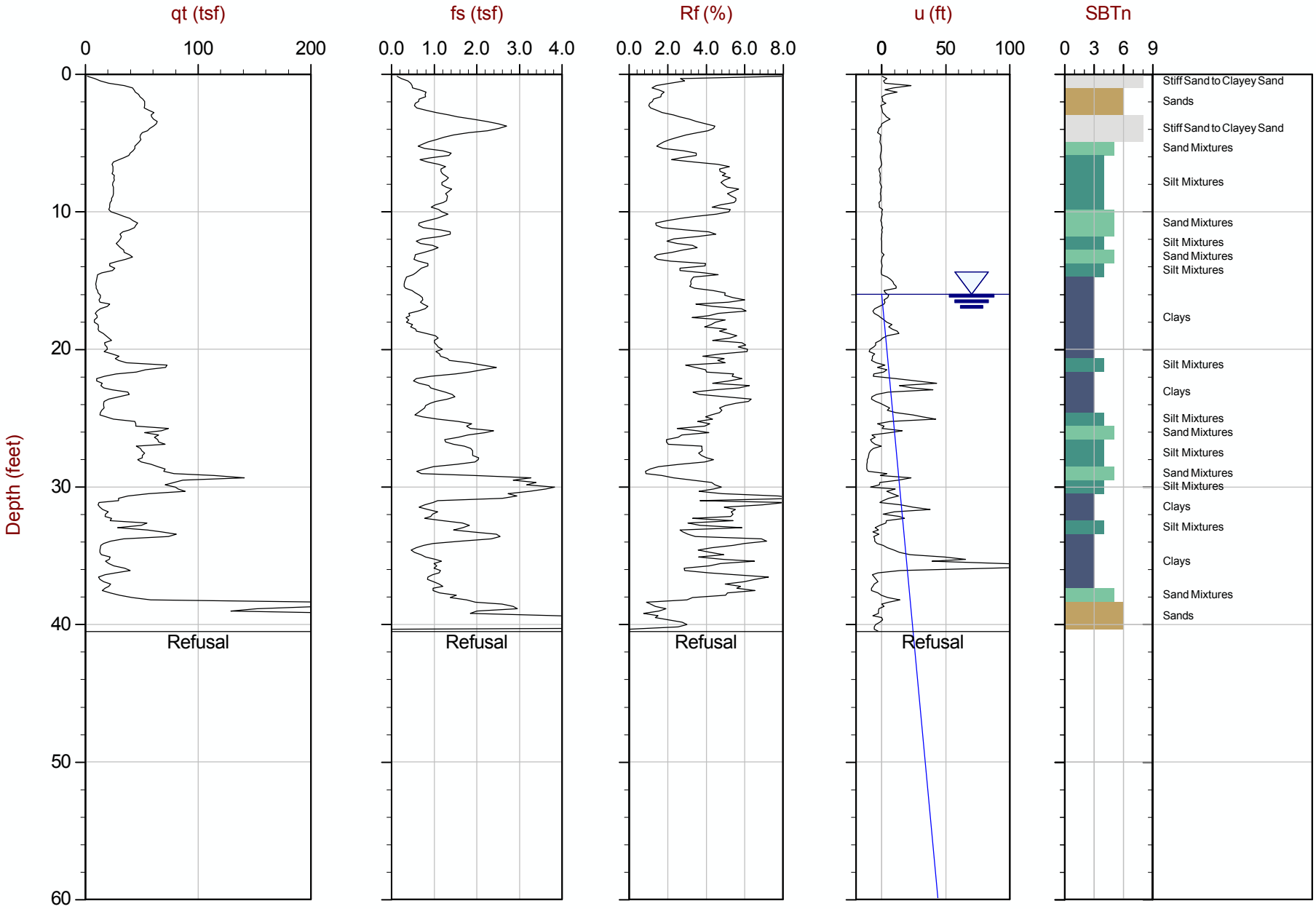
FIG. A-6

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 02:19
Site: US160 / US550

Sounding: CPT-3
Cone: STD 20T AD214



Max Depth: 12.350 m / 40.52 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

FIG. A-7

APPENDIX B

EMBANKMENT STABILITY CALCULATIONS

CALCULATION SUMMARY

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

CALC. NO: 11
JOB NO: 23-1-01165-003
PROJECT: 550 / 160 Design-Build, Durango, CO
SHEETS: 22
CLIENT: Wilson/SEMA/CDOT
SUBJECT: Ramp C Bridge Embankment Slope Stability

Problem:

Determine factor of safety against slope stability failure for the embankments necessary at Abutments 1 and 3.

General Approach / Assumptions:

Use the computer program PCSTABL to calculate the factor of safety for short-term (undrained), long-term (drained) and seismic loading conditions at Abutment 1. Due to the small amount of fill being placed at Abutment 3, in our opinon, a short-term (undrained) condition would not be mobilized. As such, long-term (drained) and seismic conditions were analyzed at Abutment 3.

Sources of Data and Equations:

Used Borings 2B42, 2B46, B-102, B-126, and CPT-1, CPT 3 to generate the subsurface profile for Abutment 1. At Abutment 3, Borings 2B40 and B-101 were used for the subsurface profile. Use blow count correlations, CPT data, and laboratory testing to determine material properties of the substrata. ~~From the USGS website, the project site has a peak ground acceleration of 0.08g for a 1000-year event.~~ A horizontal acceleration of ~~0.04g~~ 0.05g was used for the seismic case BASED ON AN $A_s = 0.09g$.

Summary and Conclusions:

The factor of safety for short-term, long-term, and seismic conditions at Abutment 1 were 1.2, 1.5, and 1.35, respectively. At Abutment 3, results indicated a factor of safety greater than 1.5 for long-term and seismic conditions.

Preliminary Calc.: ☐
Final Calc.: ☒
Supercedes Calc. No:

REV. NO.	CALCULATION BY	DATE	CHECKED BY	DATE
	MCH	8/6/08	GEF	8/6/08
	MCH	9/10/08	GEF	9/13/08

RAMP C SLOPE STABILITY

ABUTMENT 1

USED CPT-1, CPT-3, BORINGS B-102, B-126, 2B42 + 2B46 TO FORMULATE SUBSURFACE PROFILE (SEE PAGE 6)

UNDRAINED STRENGTHS WERE OBTAINED FROM THE CPT DATA
DRAINED STRENGTHS DETERMINED FROM CPT DATA, LAB TESTING, BORINGS

FOR CLAY DRAINED STRENGTH USE FIG. 19.7 TERZAGHI, PECK, + MESRI (PAGE 11) TO DETERMINE ϕ BASED ON PLASTICITY INDEX

FOR UPPER CLAY: PI = 17 FROM BORING B-102, DEPTH 5-6 FEET (PAGE 10)
USE $\phi = 30°$ WITH $c = 50$ psf

FOR LOWER CLAY: PI = 22 FROM BORING B-102, DEPTH 35-36.5 FEET (PAGE 10)
USE $\phi = 28°$

ANALYZED SHORT-TERM (UNDRAINED) (PAGE 3)
F.S. = 1.20

ANALYZED LONG-TERM (DRAINED) (PAGE 4)
F.S. = 1.50

ANALYZED LONG-TERM SEISMIC (PAGE 5) w/ HORZ. ACCEL OF ~~0.04g~~ 0.05g
F.S. = ~~1.25~~ 1.31 → SEE PAGE 5a

ABUTMENT 3

BECAUSE OF SMALL AMOUNT OF FILL PLACEMENT (~4') UNDRAINED CONDITIONS WOULD NOT OCCUR → ANALYZE LONG-TERM + SEISMIC ONLY

FOR DRAINED, USE FIG. 19.7 TERZ., PECK, + MESRI (PAGE 11)
FOR CLAY: PI = 12 BORING B-101, DEPTH 19.6-21.1 FEET (PAGE 10)
→ USE $\phi = 30°$ + $c = 100$ psf

USED BORINGS B-101 + 2B40 TO GENERATE SUBSURFACE PROFILE (PAGE 9)

ANALYZED LONG-TERM (DRAINED) (PAGE 7): FS = 2.21

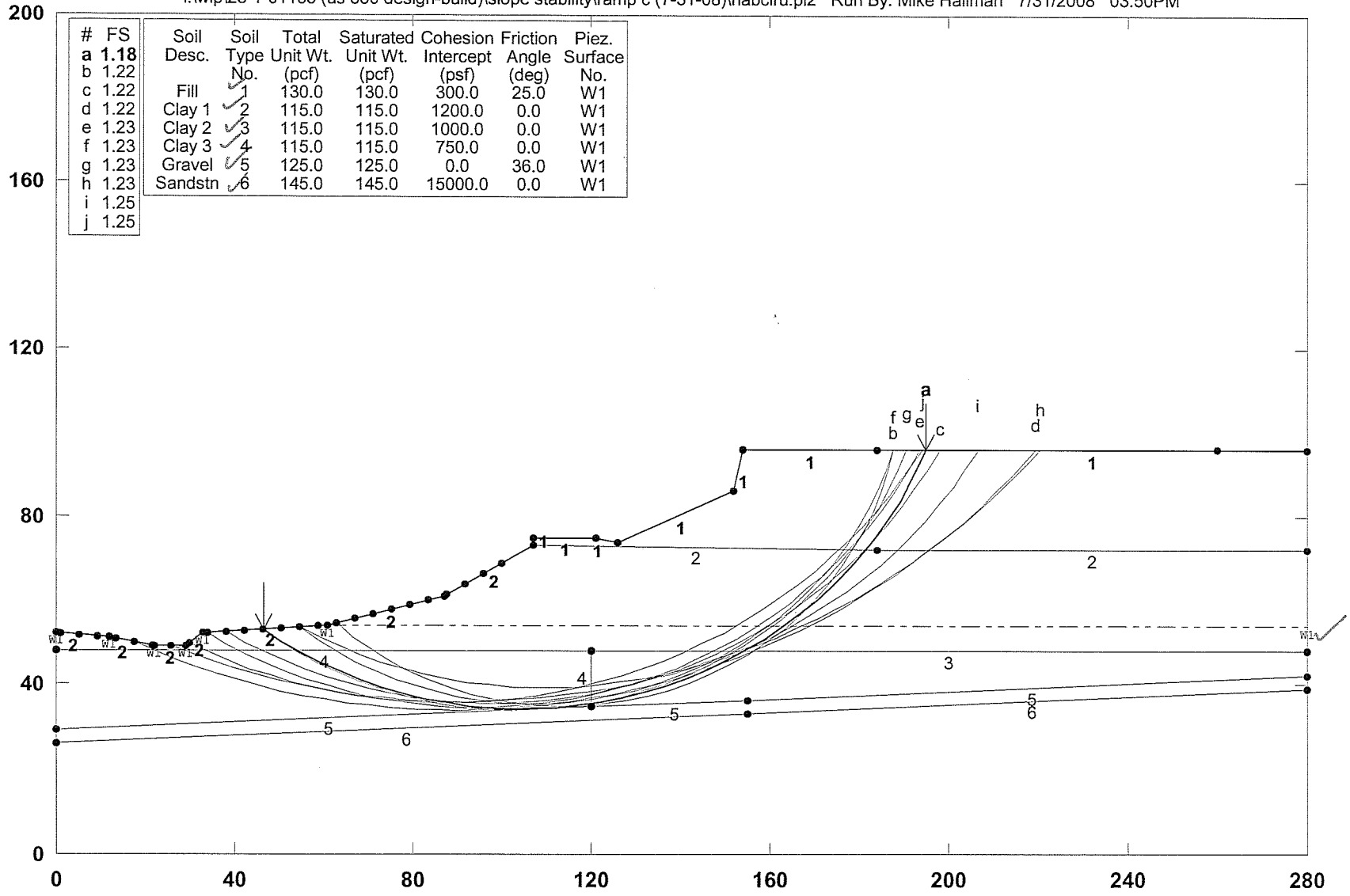
ANALYZED LONG-TERM SEISMIC (PAGE 8): FS = ~~1.95~~ 1.88 (SEE PAGE 8a)

Ramp C Abutment 1 Undrained

i:\wip\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\nabciru.pl2 Run By: Mike Hallman 7/31/2008 03:50PM

#	FS
a	**1.18**
b	1.22
c	1.22
d	1.22
e	1.23
f	1.23
g	1.23
h	1.23
i	1.25
j	1.25

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	300.0	25.0	W1
Clay 1	2	115.0	115.0	1200.0	0.0	W1
Clay 2	3	115.0	115.0	1000.0	0.0	W1
Clay 3	4	115.0	115.0	750.0	0.0	W1
Gravel	5	125.0	125.0	0.0	36.0	W1
Sandstn	6	145.0	145.0	15000.0	0.0	W1



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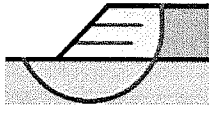
Safety Factors Are Calculated By The Modified Bishop Method

STED



3/22

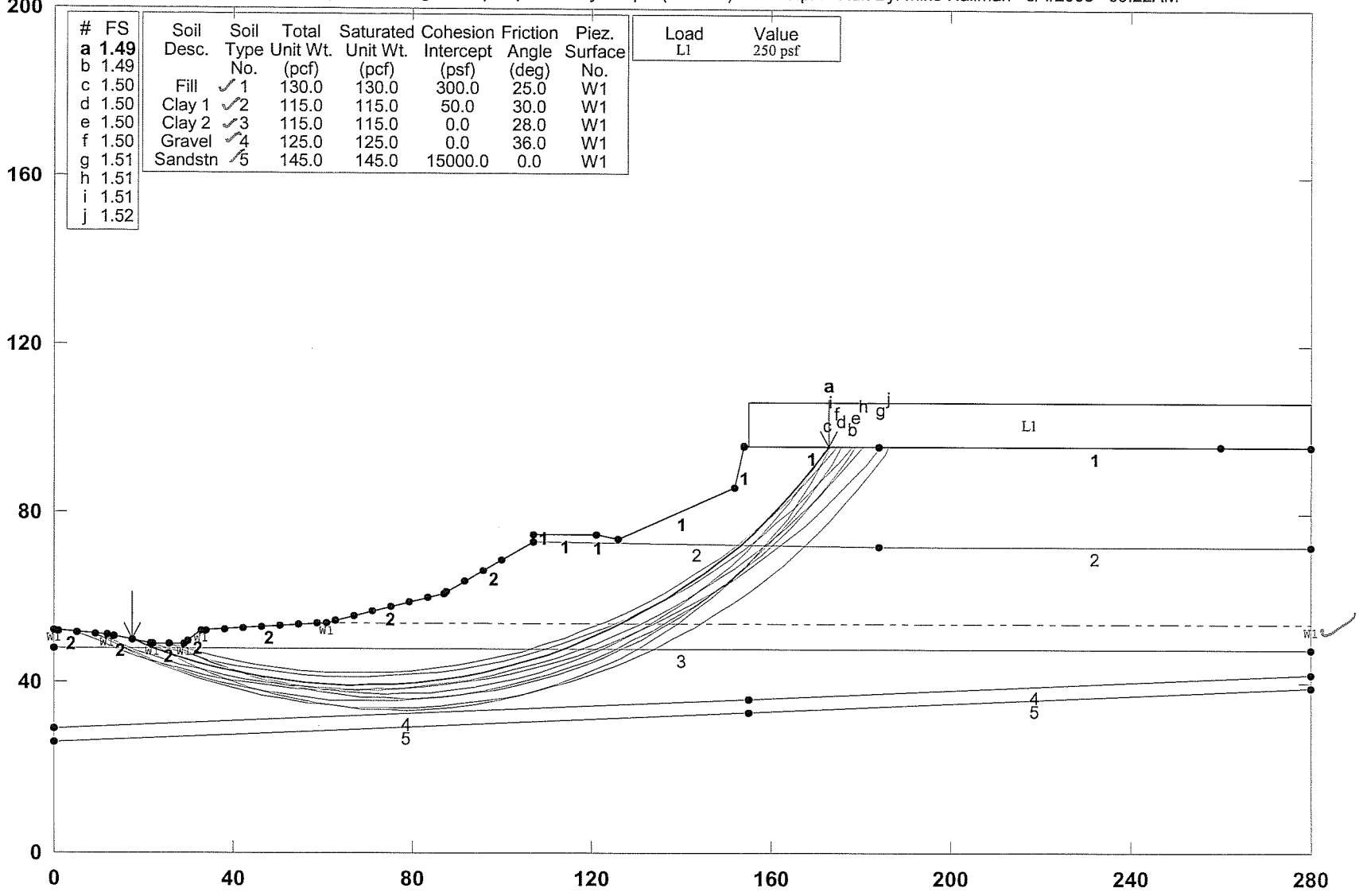
Ramp C Abutment 1 Drained

i:\wip\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\nabcird.pl2 Run By: Mike Hallman 8/4/2008 09:22AM

#	FS
a	**1.49**
b	1.49
c	1.50
d	1.50
e	1.50
f	1.50
g	1.51
h	1.51
i	1.51
j	1.52

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	300.0	25.0	W1
Clay 1	2	115.0	115.0	50.0	30.0	W1
Clay 2	3	115.0	115.0	0.0	28.0	W1
Gravel	4	125.0	125.0	0.0	36.0	W1
Sandstn	5	145.0	145.0	15000.0	0.0	W1

Load	Value
L1	250 psf



PCSTABL5M/si FSmin=1.49

Safety Factors Are Calculated By The Modified Bishop Method

STED

4/22

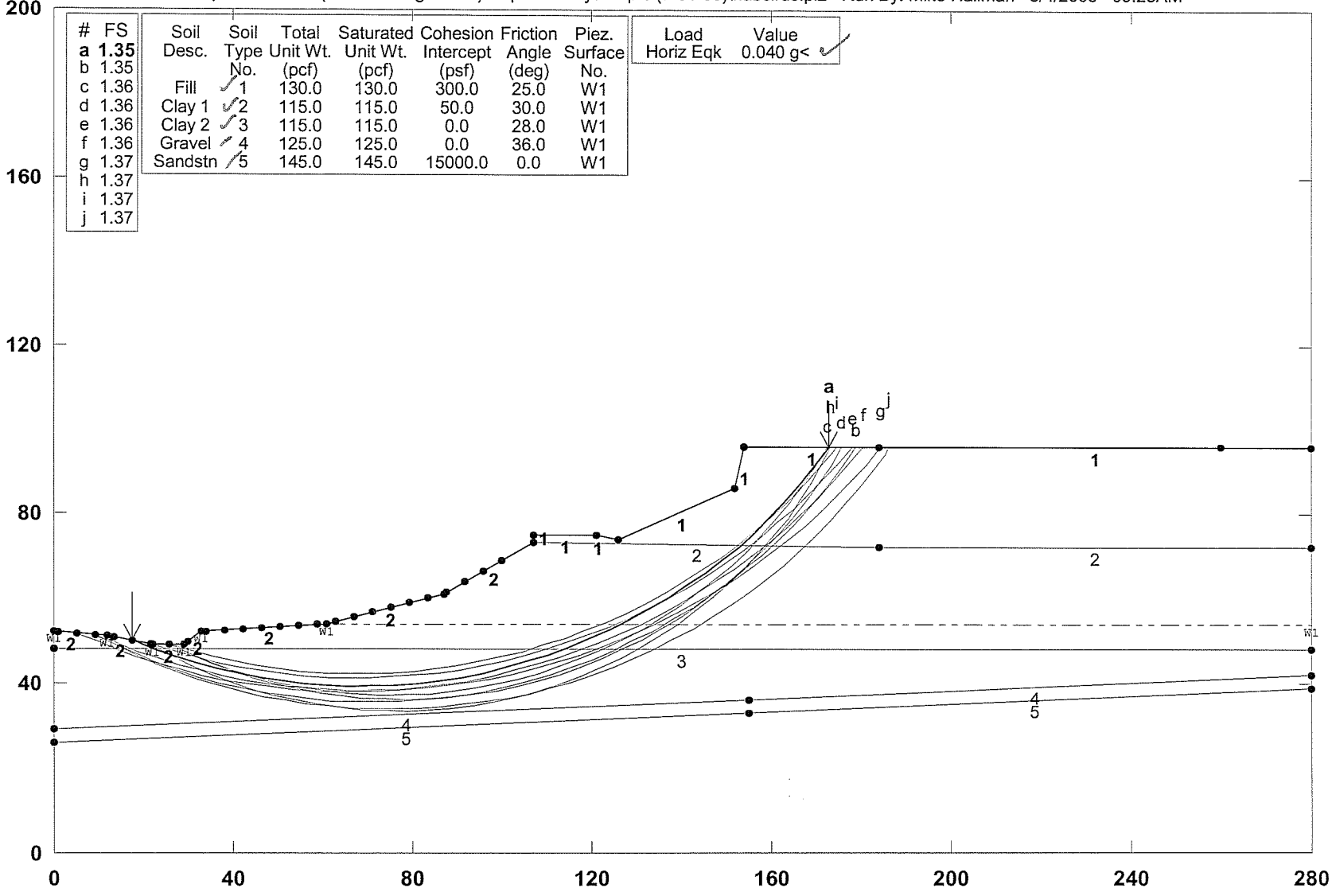
Ramp C Abutment 1 Drained Seismic

i:\wip\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\nabcirds.pl2 Run By: Mike Hallman 8/4/2008 09:23AM

#	FS
a	**1.35**
b	1.35
c	1.36
d	1.36
e	1.36
f	1.36
g	1.37
h	1.37
i	1.37
j	1.37

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	300.0	25.0	W1
Clay 1	2	115.0	115.0	50.0	30.0	W1
Clay 2	3	115.0	115.0	0.0	28.0	W1
Gravel	4	125.0	125.0	0.0	36.0	W1
Sandstn	5	145.0	145.0	15000.0	0.0	W1

Load	Value
Horiz Eqk	0.040 g<



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Safety Factors Are Calculated By The Modified Bishop Method

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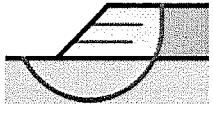


5/22

Ramp C Abutment 1 Drained Seismic

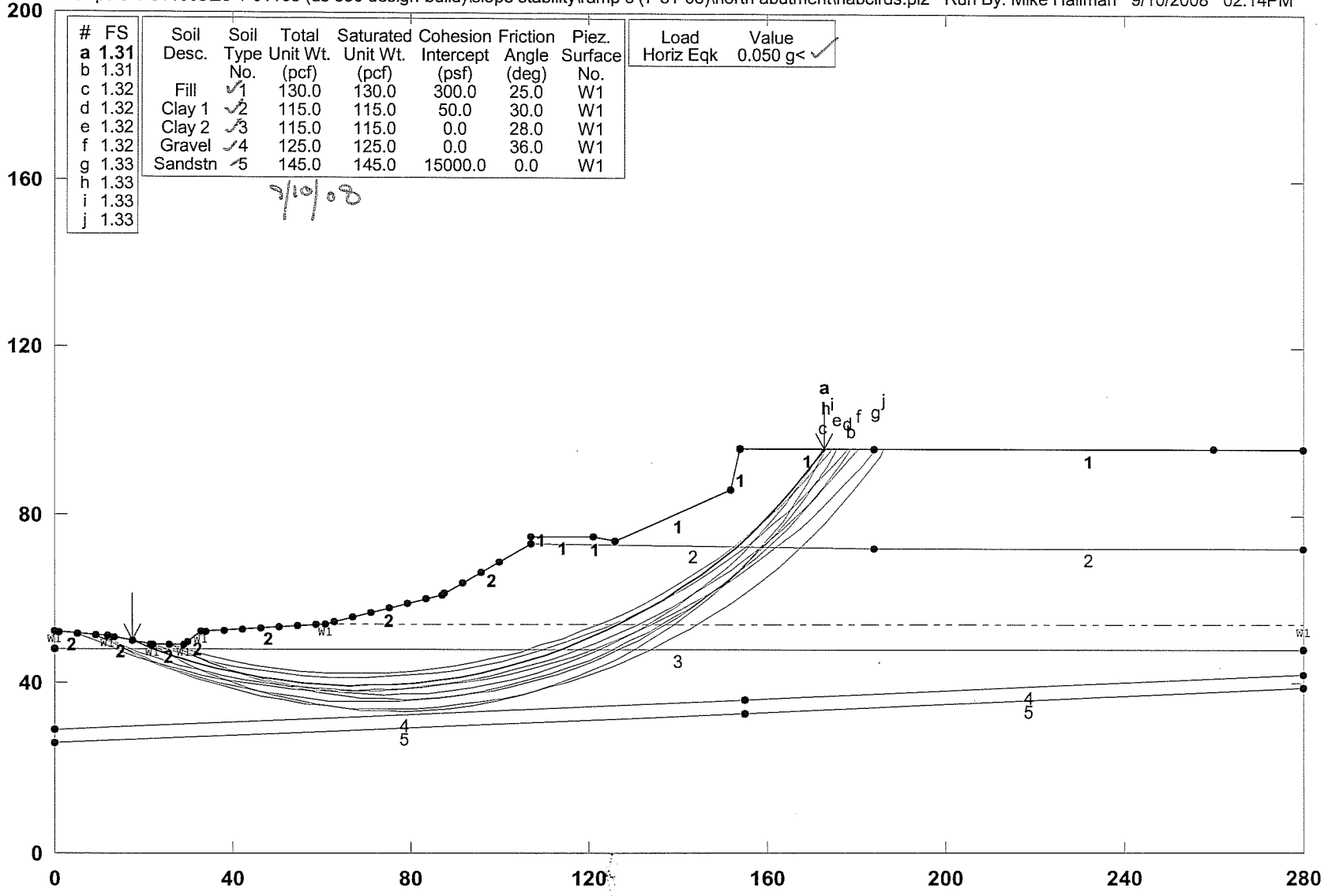
i:\wip\23-1-01100s\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\north abutment\nabcirds.pl2 Run By: Mike Hallman 9/10/2008 02:14PM

#	FS
a	**1.31**
b	1.31
c	1.32
d	1.32
e	1.32
f	1.32
g	1.33
h	1.33
i	1.33
j	1.33

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	300.0	25.0	W1
Clay 1	2	115.0	115.0	50.0	30.0	W1
Clay 2	3	115.0	115.0	0.0	28.0	W1
Gravel	4	125.0	125.0	0.0	36.0	W1
Sandstn	5	145.0	145.0	15000.0	0.0	W1

9/10/08

Load	Value
Horiz Eqk	0.050 g<



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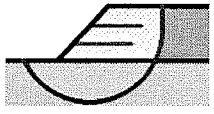
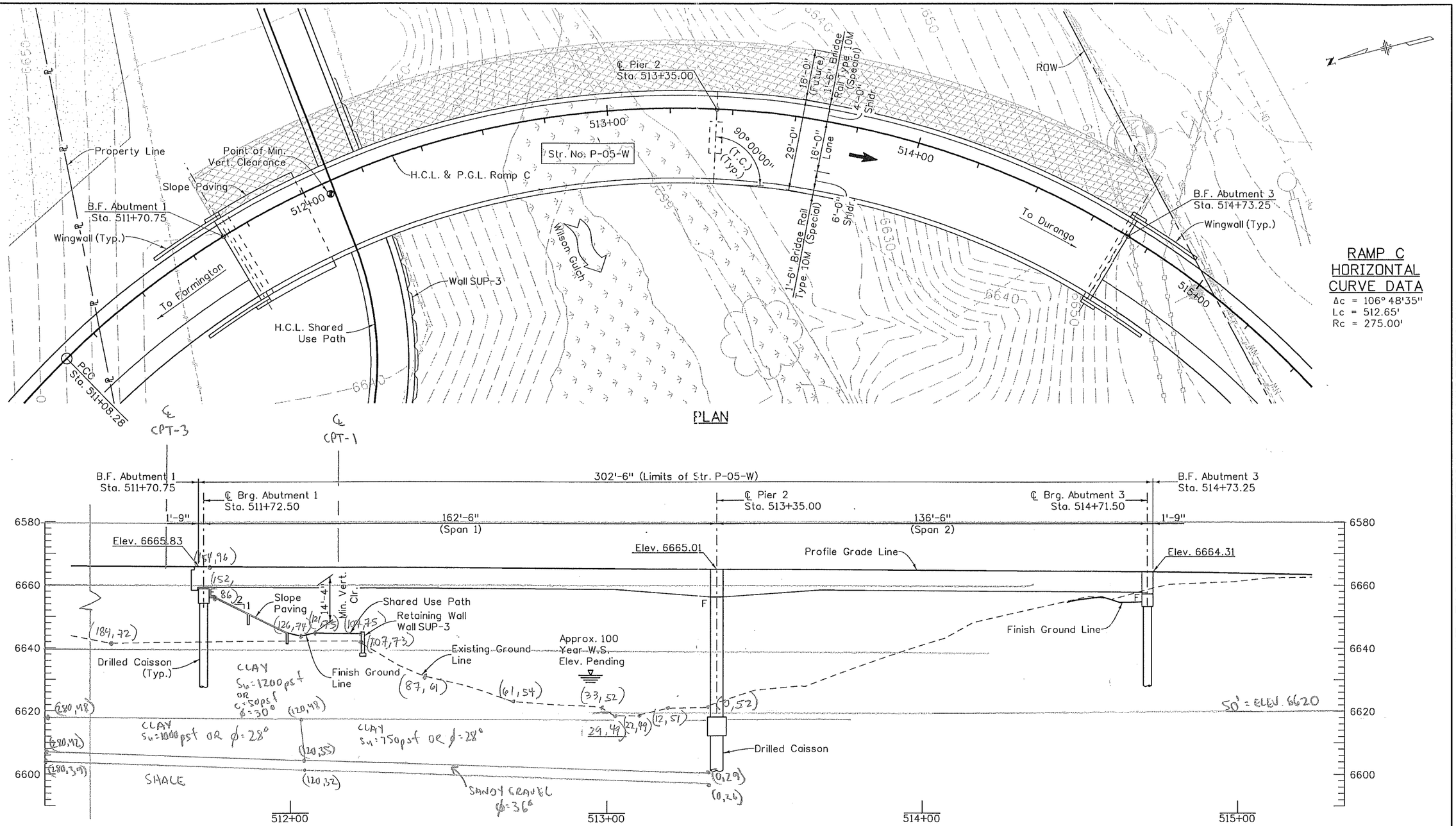
Safety Factors Are Calculated By The Modified Bishop Method

STED



5a/22

6/22



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Quantities By	XXX	MM/YY
Checked By	XXX	MM/YY

Detail	INITIAL	DATE
Detailed By	XXX	MM/YY
Checked By	XXX	MM/YY

Design	INITIAL	DATE
Designed By	XXX	MM/YY
Checked By	XXX	MM/YY

Print Date: 5/6/2008
File Name: 16042W_GenLayout_01.dgn
Horiz. Scale: 1:30 Vert. Scale: As Noted
Unit Information Unit Leader Initials

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

RAMP C OVER WILSON GULCH
GENERAL LAYOUT

Designer:	A. Leifheit	Structure Numbers	P-05-W
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B1 of B2

Project No./Code
NH 1602-114
16042
Sheet Number

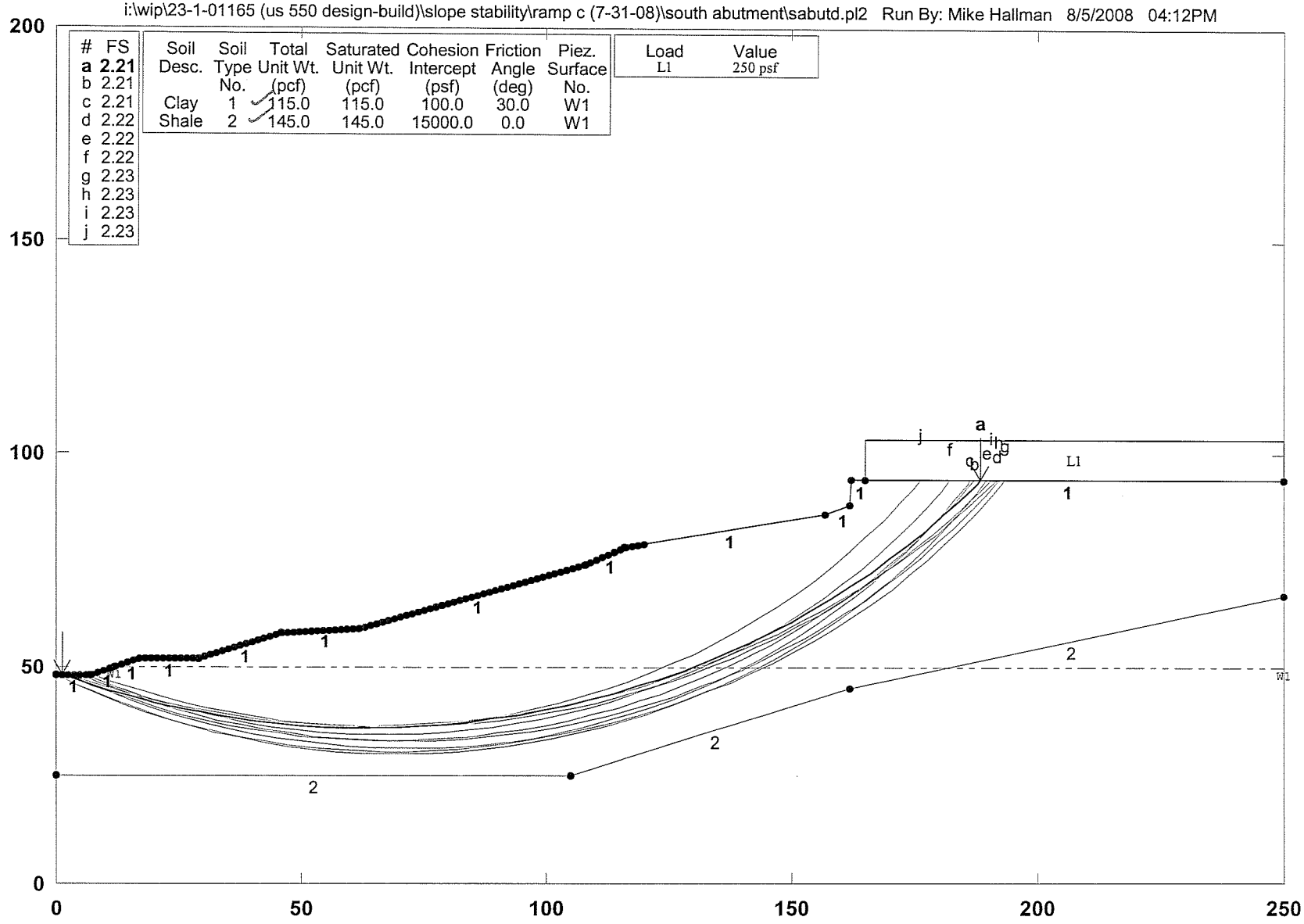
Ramp C Abutment 3 Drained

i:\wip\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\south abutment\sabutd.pl2 Run By: Mike Hallman 8/5/2008 04:12PM

#	FS
a	**2.21**
b	2.21
c	2.21
d	2.22
e	2.22
f	2.22
g	2.23
h	2.23
i	2.23
j	2.23

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Clay	1	115.0	115.0	100.0	30.0	W1
Shale	2	145.0	145.0	15000.0	0.0	W1

Load	Value
L1	250 psf



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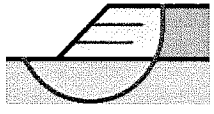
Safety Factors Are Calculated By The Modified Bishop Method

STED



7/22

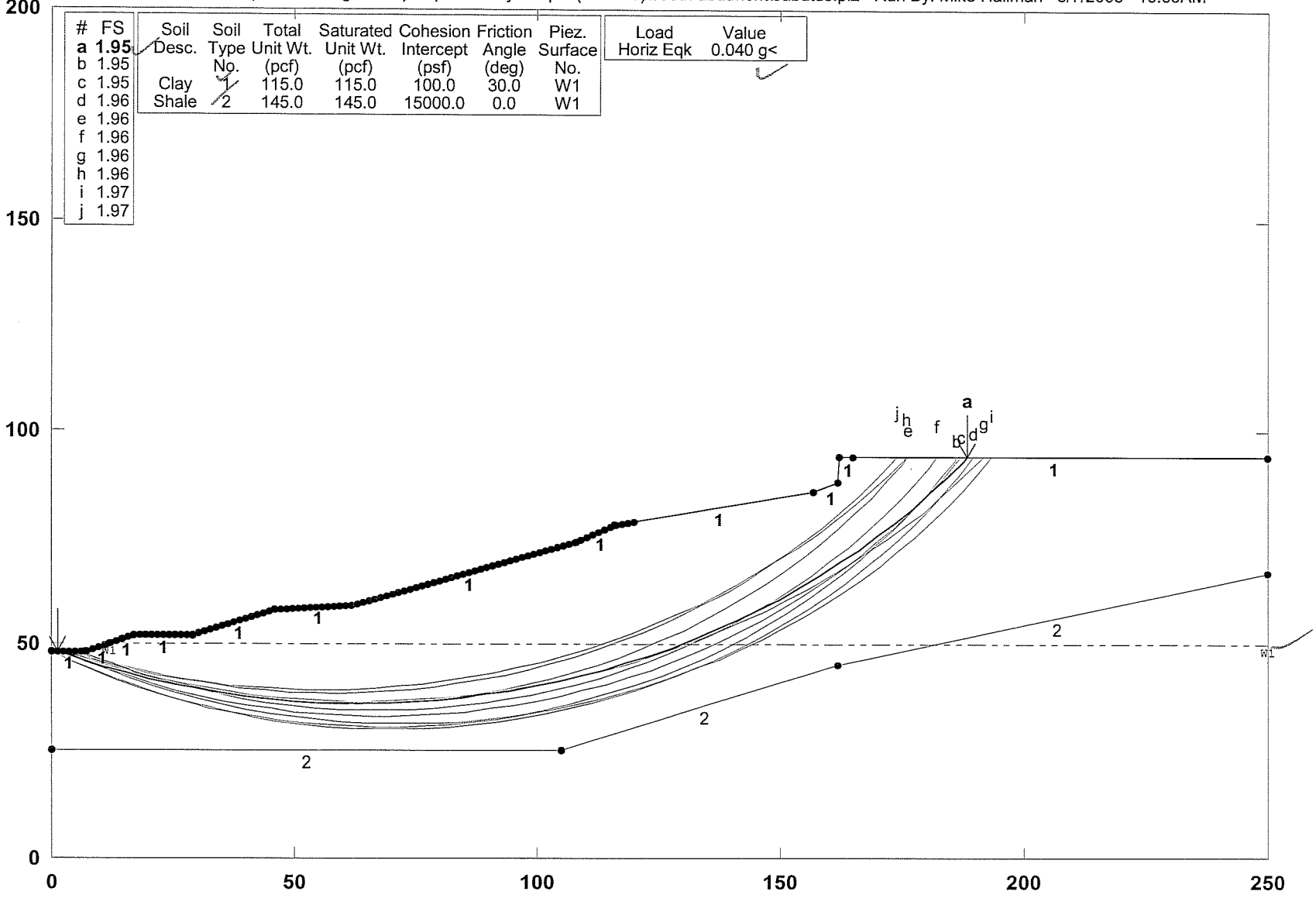
Ramp C Abutment 3 Drained Seismic

i:\wip\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\south abutment\sabutds.pl2 Run By: Mike Hallman 8/7/2008 10:09AM

#	FS
a	**1.95**
b	1.95
c	1.95
d	1.96
e	1.96
f	1.96
g	1.96
h	1.96
i	1.97
j	1.97

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Clay	1	115.0	115.0	100.0	30.0	W1
Shale	2	145.0	145.0	15000.0	0.0	W1

Load	Value
Horiz Eqk	0.040 g<



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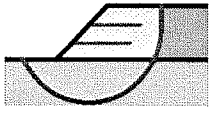
Safety Factors Are Calculated By The Modified Bishop Method

STED



8/22

Ramp C Abutment 3 Drained Seismic

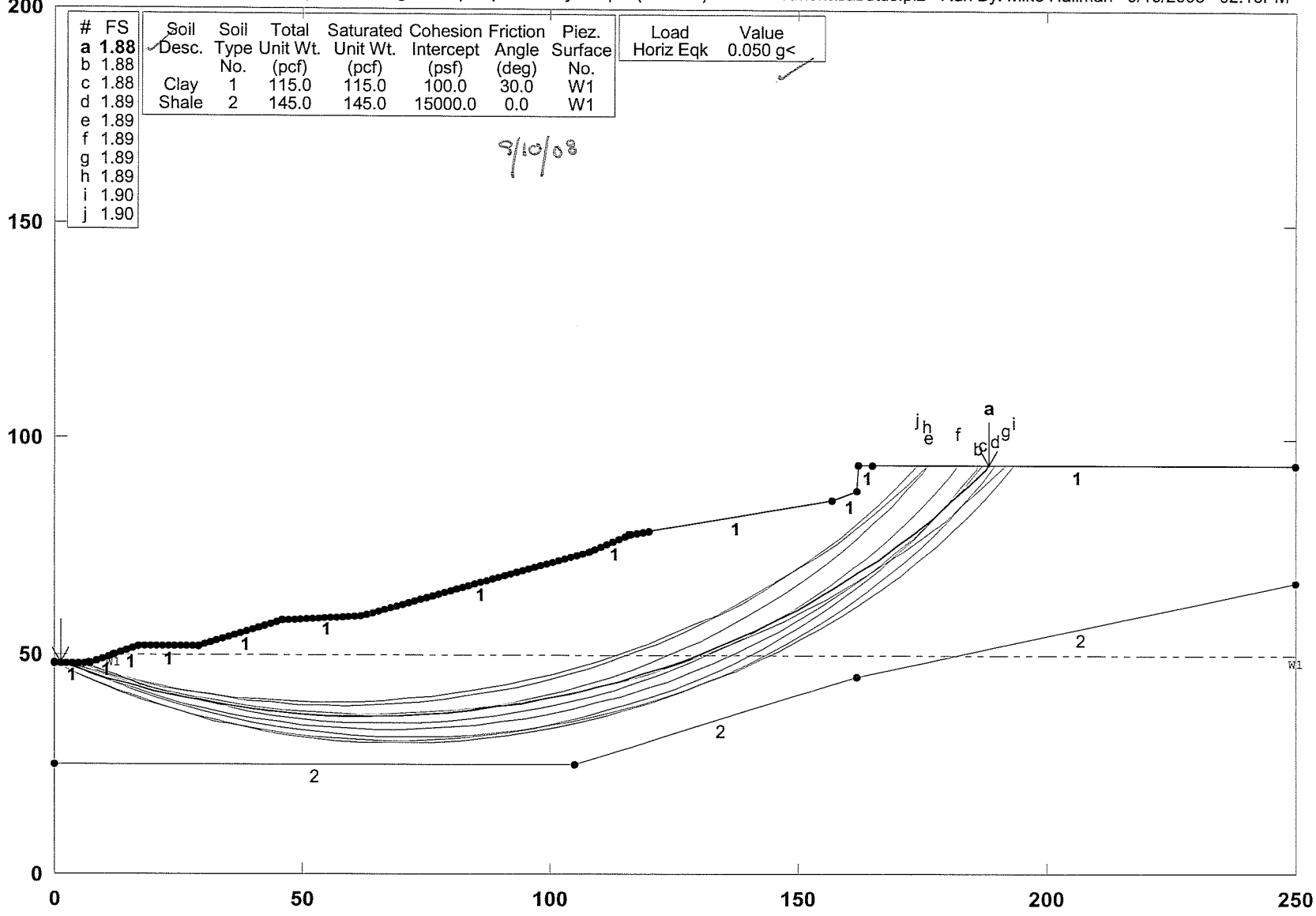
i:\wip\23-1-01100s\23-1-01165 (us 550 design-build)\slope stability\ramp c (7-31-08)\south abutment\sabutds.pl2 Run By: Mike Hallman 9/10/2008 02:16PM

#	FS
a	**1.88**
b	1.88
c	1.88
d	1.89
e	1.89
f	1.89
g	1.89
h	1.89
i	1.90
j	1.90

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Clay	1	115.0	115.0	100.0	30.0	W1
Shale	2	145.0	145.0	15000.0	0.0	W1

Load	Value
Horiz Eqk	0.050 g<

9/10/08



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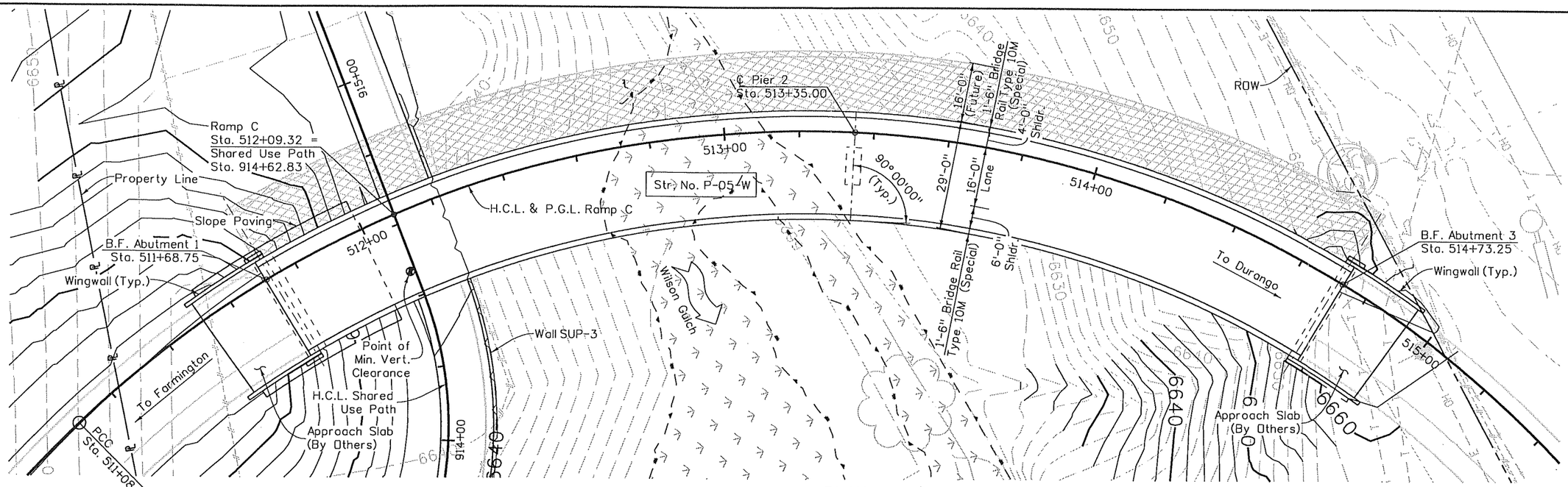
Safety Factors Are Calculated By The Modified Bishop Method

STED

8a/22

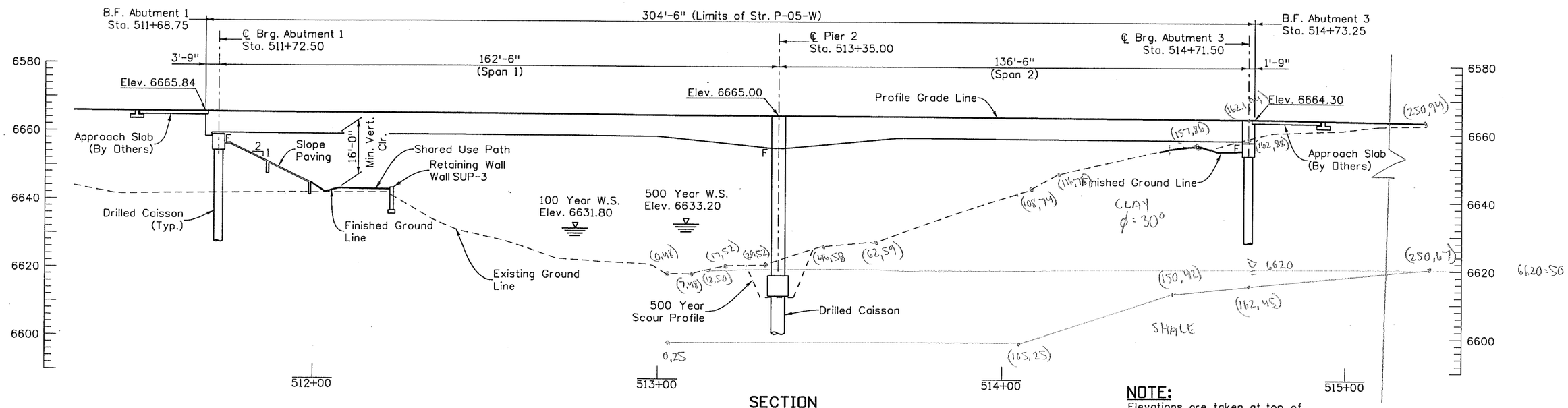
9/22



PLAN

RAMP C HORIZONTAL CURVE DATA

Δc = 106° 48'35"
Lc = 512.65'
Rc = 275.00'



SECTION
(Taken Along PGL)

NOTE:
Elevations are taken at top of 3" HMA wearing surface.

NOT FOR CONSTRUCTION - 30% PLANS - 07/28/08

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Quantities By	XXX	MM/YY
Checked By	XXX	MM/YY

Detail	INITIAL	DATE
Detailed By	XXX	MM/YY
Checked By	XXX	MM/YY

Design	INITIAL	DATE
Designed By	XXX	MM/YY
Checked By	XXX	MM/YY

Print Date: 7/28/2008
File Name: 16042W_GenLayout_01.dgn
Horiz. Scale: 1:30 Vert. Scale: As Noted
Unit Information 0221 Unit Leader STW

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

RAMP C OVER WILSON GULCH
GENERAL LAYOUT

Designer:	A. Leifheit	Structure Numbers	P-05-W
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B3 of XX

Project No./Code
NH 1602-114
16042
Sheet Number

TABLE A-1
SUMMARY OF LABORATORY TEST RESULTS BY BORING

SAMPLE DATA		USCS Symbol	AASHTO Symbol	Natural Water Content	Wet Density	Dry Density	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION			STRENGTH TESTING		CORROSION TESTING
Boring	Depth						Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	% Fines	Unconfined Compressive Strength	Strain at UCS	Water Soluble Sulfates
				(%)	(pcf)	(pcf)	(%)	(%)	(%)				(psi)	(%)	(%)
B-101	19.6 - 21.1	CL	A-6 (8)	20.7			34	22	12	4.8	20.4	74.8			0.0
	57.5 - 58.3					153.7							1,732	0.8	
B-102	5 - 6	CL	A-6 (12)	25.0			38	21	17	0.6	22.5	76.9			0.01
	35 - 36.5	CL	A-7-6 (19)	33.1			44	22	22	0.3	14.5	85.2			
	52.5 - 53.3					160.2							8,843	1.0	
B-126	48.9 - 49.5				153.3								1,120	---	

10/22

TERZAGHI, PECK, MESRI

11/22

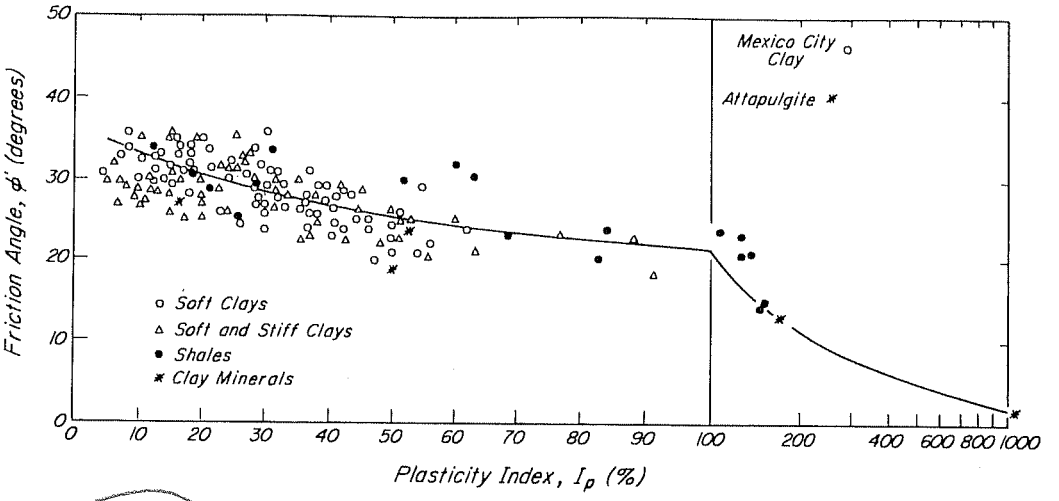


Figure 19.7 Values of friction angle ϕ' for clays of various compositions as reflected in plasticity index.

textured diatom shell fragments that make up the matrix of Mexico City clay are capable of generating unusually large friction angles while retaining an exceptionally large amount of water. At each plasticity index, different values of ϕ' result from the difference in clay size fraction of soils and from the difference in effective normal stress at which ϕ' was measured. The high values of ϕ' correspond to soils with clay size fraction of less than 20% and effective normal stresses less than 50 kPa. The low values are representative of soils with clay size fraction of greater than 50%, subjected to effective normal stresses of higher than 400 kPa. Torsional ring-shear tests by Stark and Eid (1994) show that ϕ' may decrease by 4° as the clay size fraction increases from less than 20% to more than 50% or as effective normal stress increases from less than 50 kPa to more than 400 kPa. Although there is a strong correlation between friction angle and plasticity index, the significant scatter of the data around the empirical relationship shown in Fig. 19.7 indicates that ϕ' should be measured directly on major projects. The drained triaxial compression test on specimens normally consolidated under an equal all-around pressure is most suitable for determining ϕ'.

The drained shear strength of an overconsolidated clay should ordinarily be greater than the drained strength of the same constituents in a normally consolidated state, mainly because the overconsolidated clay has a smaller preshear void ratio. However, the mobilized or available drained shear strength of heavily overconsolidated clays strongly depends on their condition before shear and when they reach failure. An overconsolidated clay mobilizes its intact strength at failure only if it has remained in an intact condition during geological unloading and associated swelling, remains intact in response to a construction operation that eventually leads to a failure, and reaches its peak resistance at the instant of a global instability. On the other hand, an overconsolidated clay has available at the instant of slip only its fully softened shear strength if it has been badly fissured and jointed during the removal of overburden or of lateral support by erosion, has softened further in response to an excavation process that leads to a long-term drained failure, or has been involved in severe progressive failure. An overconsolidated clay mass that has already experienced large displacements along a major plane of shear can mobilize only the residual shear strength if movements are reactivated along the preexisting shear surface. The presence in the fabric of an overconsolidated cohesive soil of discontinuities, such as brecciations, slickensides, fissures, joints, and shears, becomes an increasingly important factor in relation to the available drained shear strength at failure as preconsolidation pressure, overconsolidation ratio, plasticity, and, therefore, stiffness and brittleness increase.

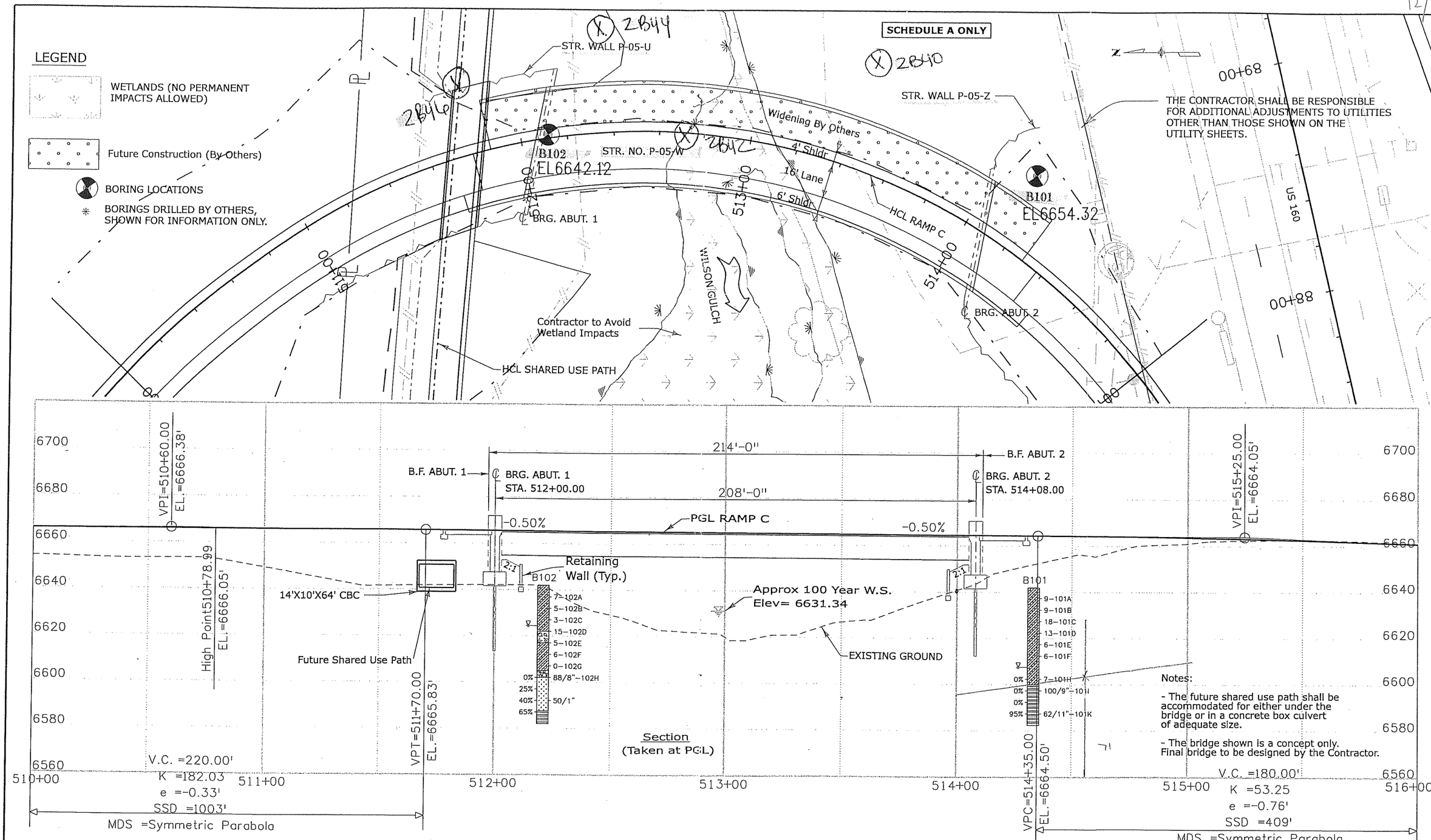
19.2.1 Intact Shear Strength

The drained intact shear strength of a saturated overconsolidated clay is commonly expressed in terms of the intercept c' and the angle ϕ' of a failure envelope in the Mohr diagram, as

$$s = c' + \sigma' \tan \phi' \tag{19.7}$$

In general the angle ϕ' in Eq. 19.7 is not comparable to the friction angle in Eq. 19.6. An example of a failure envelope for an intact sample of an overconsolidated London clay is shown in Fig. 19.8. The sample was taken from a depth of 35 m below ground level, and the best estimates of σ'_{vo} and σ'_p are 390 kPa and 4100 kPa, respectively. London clay was deposited under marine conditions in the Eocene period about 30 million years ago. Subsequently, uplift and erosion in Tertiary and Pleistocene times removed 170 m to 340 m of overlying sediments. The failure envelope of the overconsolidated clay

12/22



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File Name: Ramp C Gen Layout.dgn		Date:	Comments	Init.
Horiz. Scale: 1:40	Vert. Scale: As Noted		
Unit Information	Unit Leader Initials		

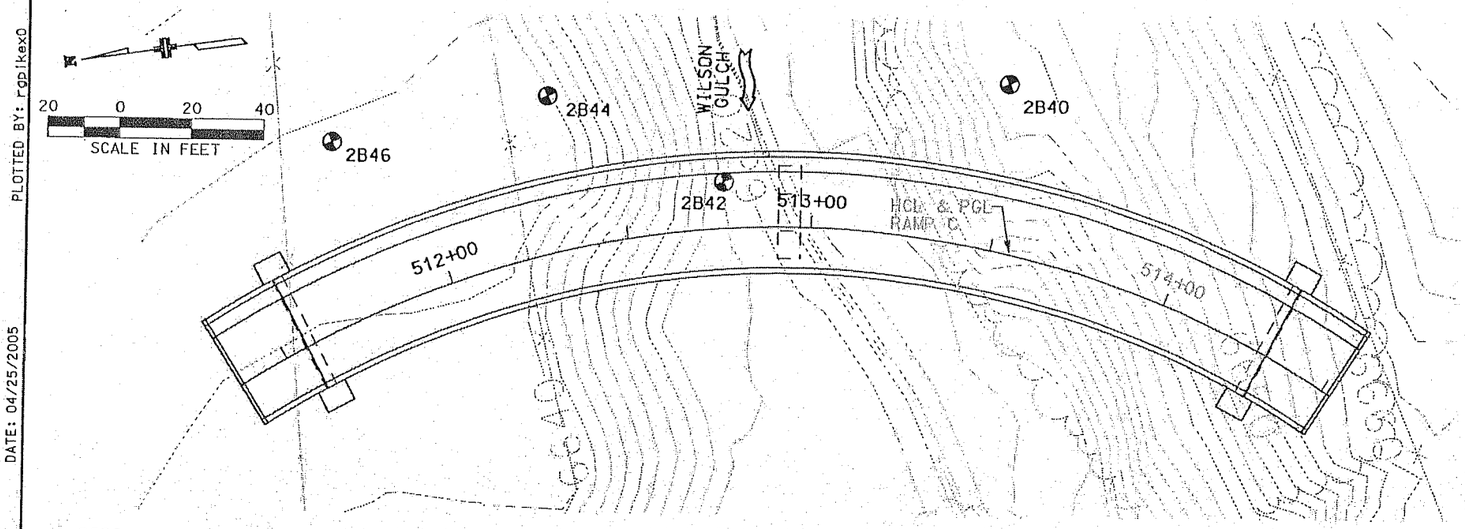
Colorado Department of Transportation
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

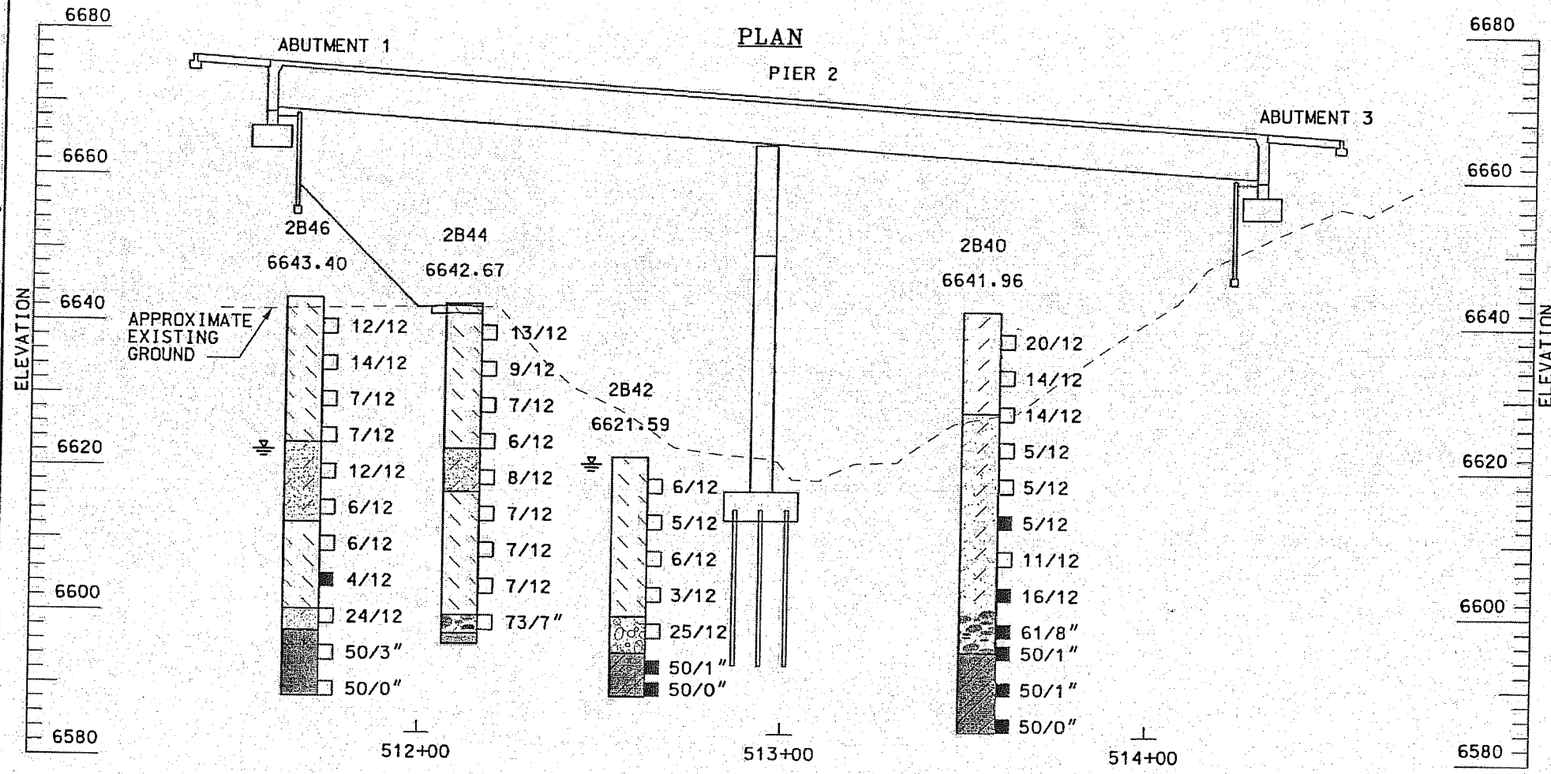
RAMP C (P-05-W)
GENERAL LAYOUT

Designer:	spc	Structure Numbers	P-05-W
Detailer:	spc		
Sheet Subset: Bridge		Subset Sheets: 3 of 4	

Project No./Code
NH-1602-114
16042
Sheet Number 22



PLAN



SECTION
(TAKEN AT PGL)

LEGEND

- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, DARK GREY TO DARK BROWN (CL)
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SILTY TO VERY SILTY, STIFF TO VERY STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY TO MEDIUM BROWN (CH)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY SILTY, SLIGHTLY GRAVELLY, LOOSE TO MEDIUM DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, MEDIUM DENSE TO DENSE, MOIST, BROWN (GP, GM, GC)
- COBBLES AND BOULDERS
- CLAYSTONE, TRACE SILT TO SILTY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

PLOTTED BY: rpike
DATE: 04/25/2005
FILE NAME: n:\projects\21711361_US_160\Sub_00\09.0_CAD\Plans\Structures\P05W\P05WEG1.dgn

Computer File Information		
Creation Date:	Initials:	
Last Modification Date: SEE PLOT STAMP	Initials:	
Full Path: SEE PLOT STAMP		
Drawing File Name: SEE PLOT STAMP		
Microstation Ver. V8	Scale: AS SHOWN	Units:

Index of Revisions

DOT DEPARTMENT OF TRANSPORTATION Region 5

URS
10 Burnett Court Suite 113
Durango, Colorado 81301
(970) 259-9212
Fax (970) 259-9241

As Constructed
No Revisions:
Revised:
Void:

RAMP C
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: R. PIKE
SheetSubset:
Structure Numbers: P-05-W
Subset Sheets:

Project No./Code
FC-NH(CX)160-2(048)
91050
Sheet Number FIG. 2-5

STA. 96+56, (US 160)
REQUIRED: INSTALL 24" FES,
13 CY RIPRAP, D50=14" OUTLET
PROTECTION, SEE DETAIL
SHEET XX, PLACE 24"X80' RCP
INSTALL 4' DIA. STORM
MANHOLE RT PER M-604-20.
Q100= 23.11 CFS
DHW= 6689.00
AHW= 6689.00
EXISTING 24" X160' CMP
W/ DROP INLET &
END SECTION
US 160
EXISTING 18"X145'
CMP W/ DROP INLETS
EXISTING 24"X110'
CMP W/ END
SECTION
95+00
6680
90+00
Str. No. P-05-W
Str. No. P-05-Y
WILSON
GULCH
SHARED USE PATH
920+00
915+00
6660
6650
6670
6690
RAMP D
RAMP C
CP-1
CP-2
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6640
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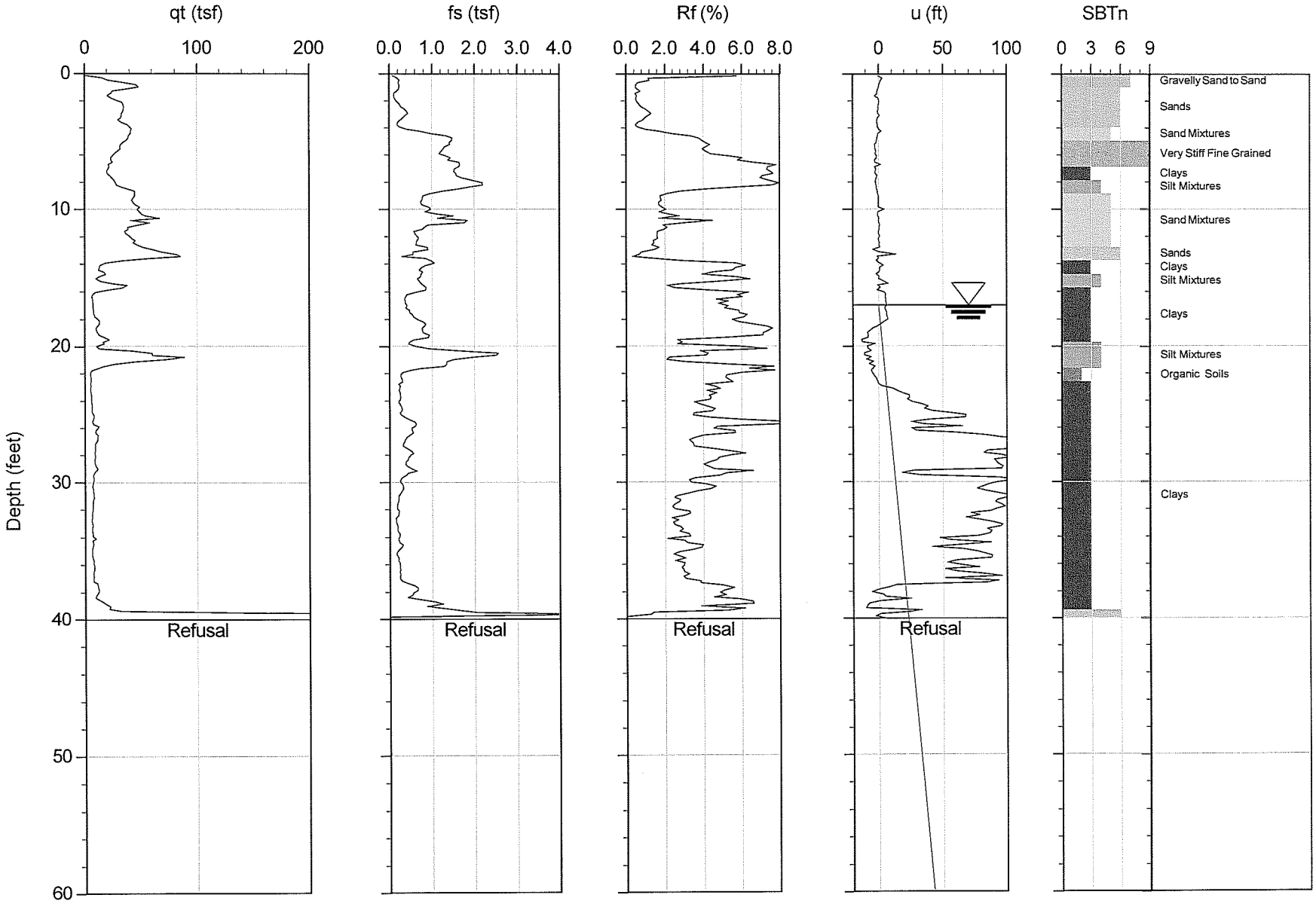
GULCH

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 01:33
Site: US160 / US550

Sounding: CPT-1
Cone: STD 20T AD214



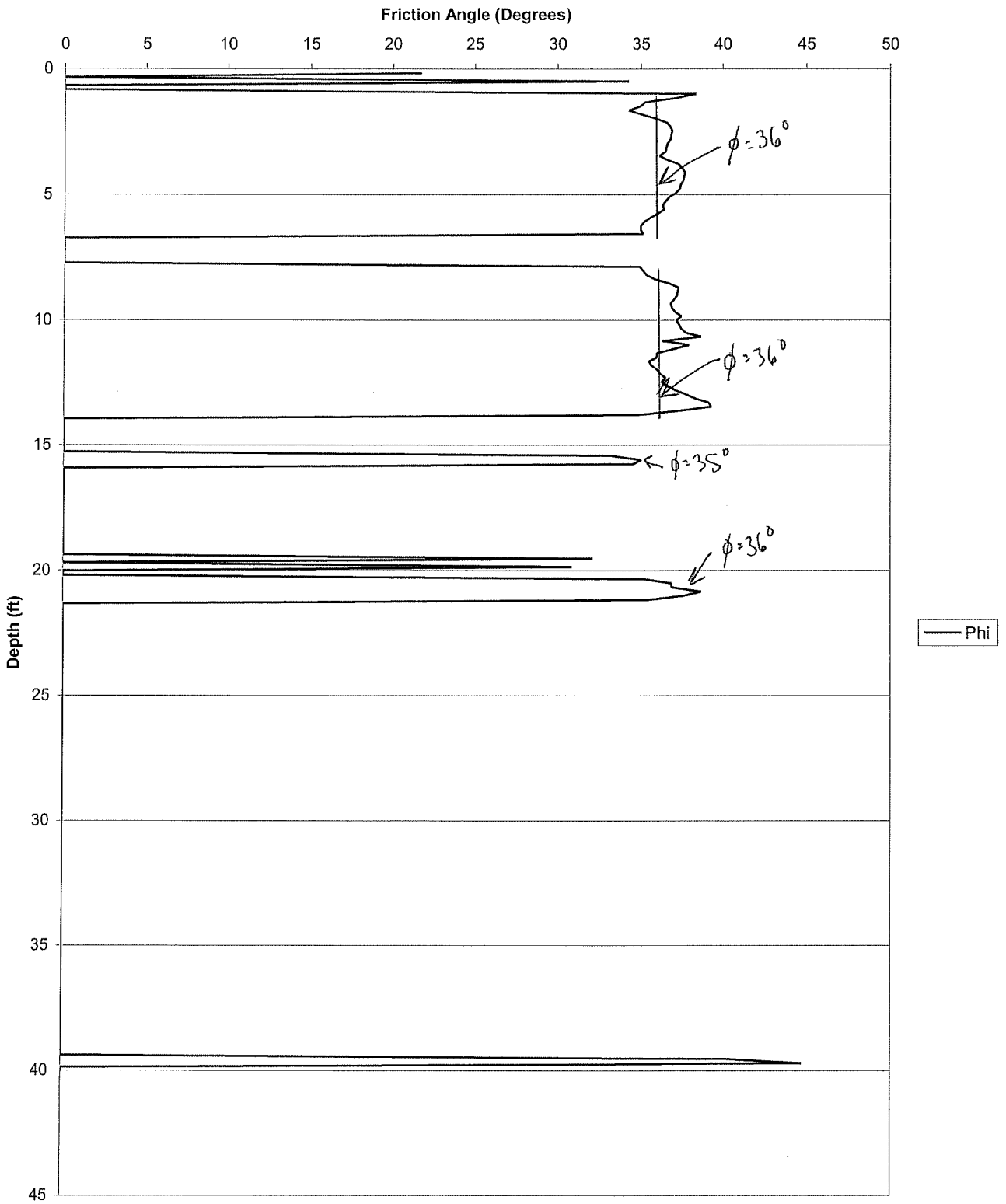
Max Depth: 12.200 m / 40.03 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

15/22

CPT-1 Shear Strength



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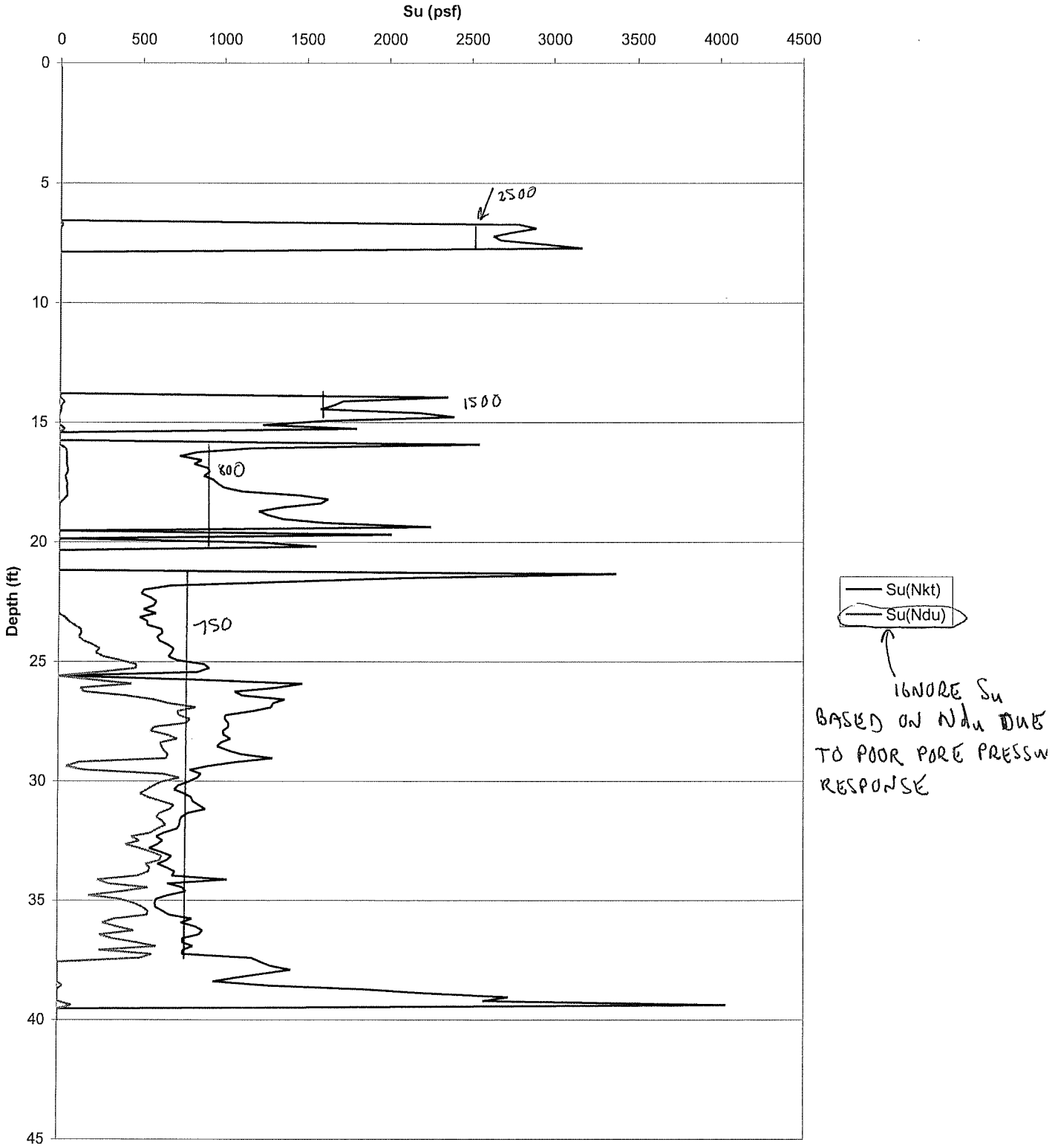
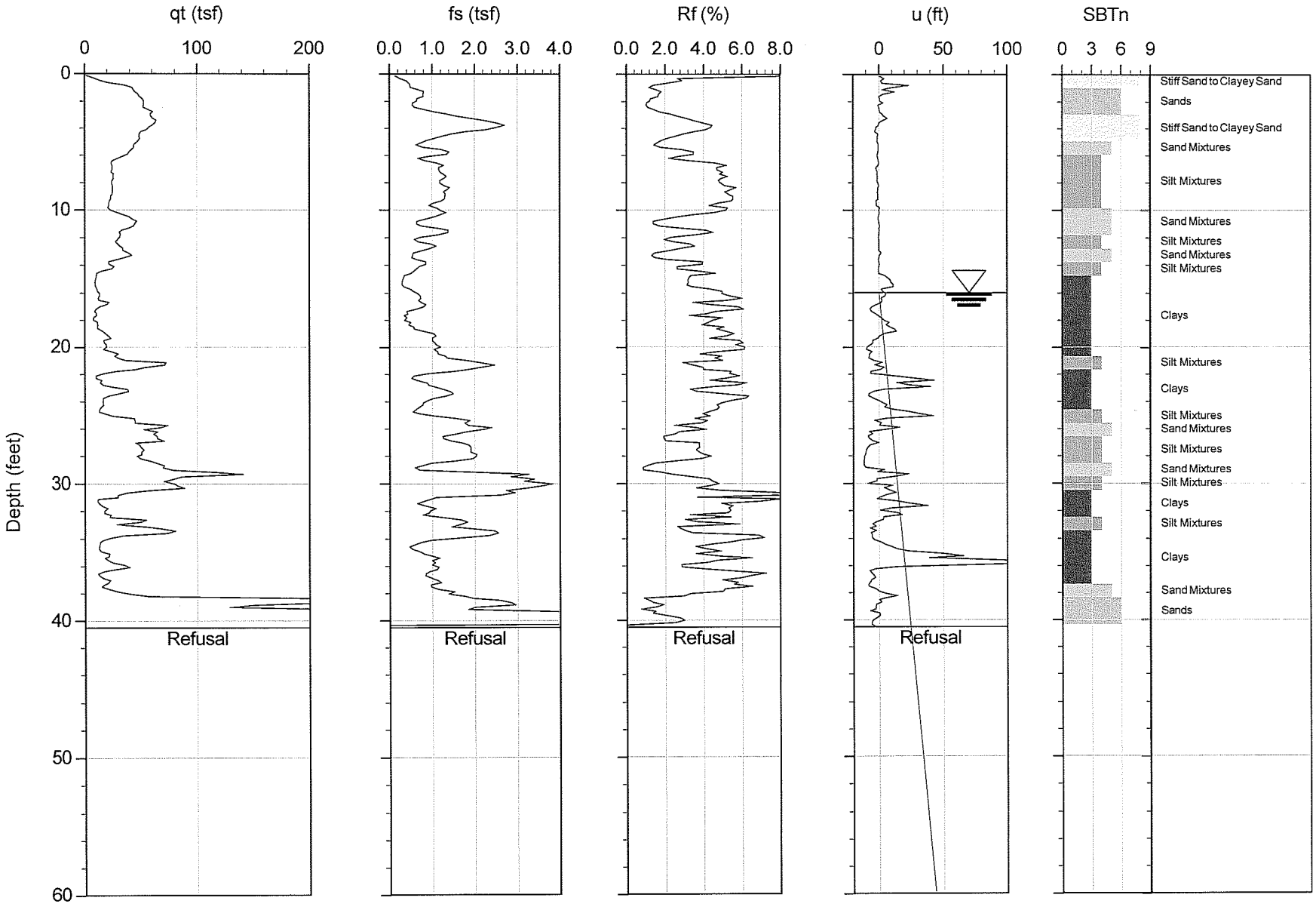


CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 02:19
Site: US160 / US550

Sounding: CPT-3
Cone: STD 20T AD214



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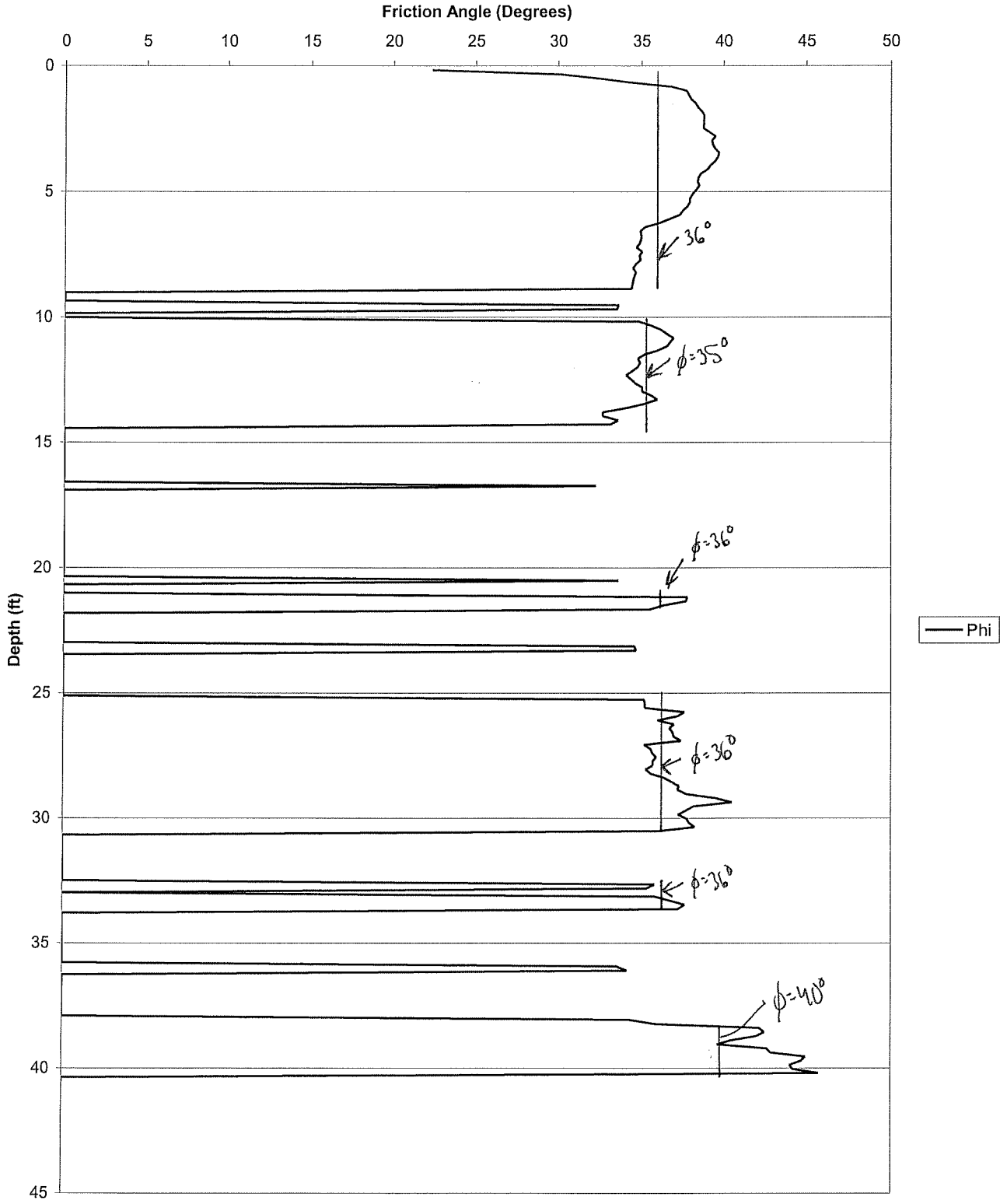
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP02.COR
Unit Wt: SBT Chart Soil Zones

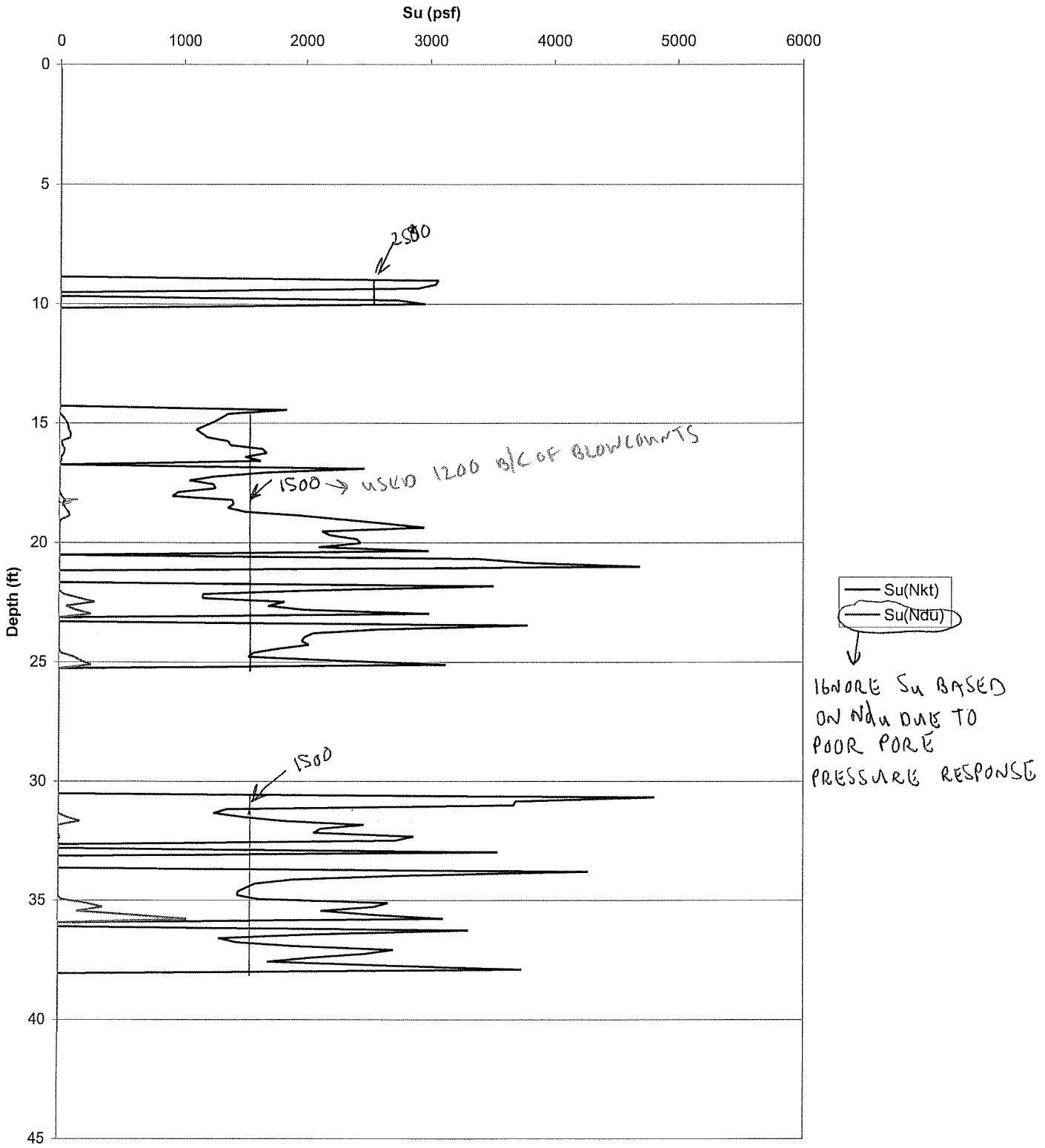
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

18/22

CPT-3 Shear Strength



CPT-3 Shear Strength



DEPARTMENT OF TRANSPORTATION

GEOLOGICAL BORING LOG

BORING # **B-126**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	Fourth Lane	7/9/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-W Ramp C Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,641.3ft	59.5ft	N1213707.1 E23220506.48	Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6640		0 - 35 Sandy clay, brown, moist, medium stiff to stiff		0.0		
				4.5 / 4.5	126A CAL	6
6635						
				9.5 / 9.6	126B CAL	7
6630						
				14.5		
6625	15.0	15 Trace root fragments				
	19.5	19.5 No recovery on drive sample		19.5 / 19.5	NR CAL	5
6620						
				24.5		
6615						
	29.5	29.5 No recovery on drive sample		29.5 / 29.5	NR CAL	12
6610						
	35.0	35 - 38 Gravel, particles up to 1.5 inches, angular to subrounded		34.5		10% / 0%
6605	38.0			39.5		

SPT DATA (N-VALUE): 5, 10, 20, 40, 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

22/22

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-126**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	Fourth Lane	7/9/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-W Ramp C Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,641.3ft	59.5ft	N1213707.1 E23220506.48	Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
6600			38 - 59.5 Sandstone, medium grained, grey to brown, moderately weathered, severely jointed, oxide staining on joint surfaces, joints primarily horizontal *(continued)*				75% / 0%		
					44.5		75% / 10%		
6595	47.0		47 Sandstone as above, less weathered						
	49.5		49.5 Sandstone, as above, intact, some minor joints, no evidence of weathering		49.5		90% / 80%		
6590									
					54.5		100% / 90%		
6585									
	59.5		Total Boring Depth 59.5ft						
6580									
6575									
6570									
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SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

SEATTLE
RICHLAND
PORTLAND
FAIRBANKS
ANCHORAGE
DENVER
SAINT LOUIS
SACRAMENTO
JACKSONVILLE

September 11, 2008

Wilson & Company
999 18th Street, Suite 2600
Denver, CO 80202

Attn: Mr. Andy Leifheit

RE: 100% SUBMITTAL, GEOTECHNICAL DESIGN LETTER 003, RAMP A BRIDGE, CDOT PROJECT NO. NH1602-114, DURANGO, COLORADO

This letter provides geotechnical design recommendations for the proposed Ramp A Bridge, associated retaining wall at Abutment 1, and embankment at Abutment 3. This letter supercedes our August 1, 2008, 90 percent submittal.

STRUCTURE DESCRIPTION

The proposed 2-span bridge over Wilson Gulch is designated as Ramp A Bridge, CDOT Structure No. P-05-V (Figure 1). The bridge will trend approximately north-south and be about 28 feet wide. The north abutment is designated as Abutment 1 (Sta. 316+95.73) and the south abutment is designated as Abutment 3 (Sta. 321+38.73). One interior pier, designated Pier 2 (Sta. 319+17.23), is planned to support the bridge. The abutments and pier will be supported on drilled shaft foundations.

To achieve the proposed roadway grade elevations, fill material will be placed at both abutments. At Abutments 1 and 3, fill heights will approach 28 and 33 feet, respectively. Due to the location of US 160, a retaining wall (CDOT Wall No. P-05-AK) is planned to support the north side of the new fill embankment for Abutment 1. At Abutment 3, the fill embankment will be sloped at 2 Horizontal to 1 Vertical (2H:1V) beneath the bridge and will gradually fan out to 4H:1V on the side of the embankment opposite the bridge abutment. Slope paving will be utilized on the 2H:1V slopes beneath the bridge.

23-1-01165-003
LR3

1060 BANNOCK STREET, SUITE 200
DENVER, COLORADO 80204
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SUBSURFACE CONDITIONS

Our interpretation of subsurface conditions at the bridge site is based on borings drilled by CDOT and others. The approximate location of the borings and the boring logs are provided in the Appendix. A summary of laboratory test results are presented in Table A-1.

The subsurface profile at the bridge was interpreted from the following borings: 2B16, 2B18, 2B20, 2B22, YA-02, YA-23, YA-24, and B-125. At Abutment 1, subsurface conditions generally consisted of medium dense, clayey, silty sand from the existing ground surface to the top of shale and sandstone bedrock. At Pier 2, the subsurface conditions consisted of stiff, sandy clay to the top of shale and sandstone bedrock. In the vicinity of Abutment 3, the subsurface conditions varied from medium stiff to very stiff, sandy clay to medium dense clayey, gravelly sand overlying shale and sandstone bedrock. For our analysis, we assumed that the subsurface conditions at Abutment 3 consisted of only medium stiff to very stiff, sandy clay overlying bedrock. These conditions would be consistent with those indicated from Boring B-125 (completed directly at the abutment location) and would produce the more conservative design approach. The boring logs describe the sandstone as a hard material with foliations and the shale as a hard fractured material, with joints that contain some slickensides. The top of shale and sandstone bedrock appears to be at approximate elevation 6,578 feet at Abutment 1 and Pier 2 and approximate elevation 6,563 feet at Abutment 3. However, the borings indicate a relatively sharp drop in bedrock elevation between Pier 2 and Abutment 3. Boring 2B20 did not encounter bedrock in the boring, which was extended to an elevation of approximately 6,548 feet and Boring 2B22 encountered bedrock at an elevation of approximately 6,559 feet.

Groundwater was noted in some of the borings at elevations ranging from about 6,588 to 6,580 feet. Groundwater levels were recorded in each boring as encountered at the time of drilling. As shown in Table 1, we assumed that the long-term groundwater elevation may be as high as 6,590 feet.

SEISMICITY

We understand that seismic design of the bridge will be in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). Based on data collected from borings at the bridge location, we calculated an average N-value of 37. Using the N-value method, the soil conditions at the bridge site are consistent with Site Class D, as defined in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim).

The USGS 2007 Seismic Parameters CD, included with the AASHTO LRFD Bridge Design Specifications, was used to calculate the required seismic parameters. For the project site, a latitude of 37.23 and longitude of -107.84 were utilized. For Site Class B, the PGA, S_s, and S_1 are 0.061g, 0.0135g, and 0.041g, respectively for a 7 percent probability of exceedance in 75 years (1,000-year return period).

For Site Class D, the site factors are as follows: F_{pga}= 1.6, F_a= 1.6, and F_v= 2.4. Therefore, for Site Class D, A_s = 0.098g, S_{DS}= 0.216g, and S_{D1}= 0.098g.

Liquefaction, lateral spreading, and dynamic settlement are factors that can negatively affect the stability and load carrying capacity of foundations and structures. In our opinion, because the peak horizontal ground acceleration (PGA) for this area of Colorado is low, there is a low risk potential associated with these seismic hazards, and such seismic hazards were not considered as part of design of the structure.

GEOTECHNICAL RECOMMENDATIONS

Drilled Shafts

We understand that drilled shafts are the preferred foundation support for the abutments and piers. The design criteria presented herein for drilled shafts uses load and resistance factor design (LRFD). The LRFD criteria were developed based on AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). In addition, Federal Highway Administration (FHWA) design methods (O'Neil and Reese, 1999) and a CDOT Research

Report 2003-6 (Abu-Hejleh et al., 2003) were used to supplement drilled shaft design procedures.

Axial Capacity Parameters

Axial design parameters for use in sizing bridge foundations, as well as Resistance Factors (ϕ) for both tip and side resistance, are presented in Table 1. The nominal axial capacities in the bedrock are based on Standard Penetration Test (SPT) blow counts, using the methodology presented in Section 6.4.2 in the CDOT Research Report 2003-6. For this method, the shale and sandstone bedrock is defined as a "soil-like claystone". Average blow counts for the shale and sandstone bedrock, as indicated from the borings, were on the order of 50/2 inches, with the lowest value being 50/4 inches. For our analysis, we used 50/5 inches, which would be equivalent to a blow count of 120.

The CDOT Research Report 2003-6 also presents design methodologies for estimating ultimate (nominal) skin and end bearing values for a "very hard sandy claystone" (Section 6.4.4) and "very hard clayey sandstone" (Section 6.4.5). In our opinion, these rock descriptions would be more consistent with the bedrock encountered at the site.

A parametric study between methodologies for "soil-like claystone" (Section 6.4.2), "very hard sandy claystone" (Section 6.4.4), and "very hard clayey sandstone" (Section 6.4.5) was conducted to compare the nominal skin friction and end bearing values determined from each method. An unconfined compressive strength of 4,050 psi (taken from Boring YA-03) was used in the study for bedrock, which was the lowest strength value among rock samples tested at this bridge location (see Table A-1). Even though the rock exhibits strength consistent with "very hard sandy claystone" and "very hard clayey sandstone", we used nominal skin friction and end bearing values for the "soil-like claystone" as it yielded the lowest values in comparison to the other methods.

At the Pier 2 location, we assumed a weakened zone within the top 5 feet of the shale and sandstone bedrock, as shown in Table 1. This weakened zone was used as a conservative

measure for design purposes only, due to the close proximity of the bedrock surface to the top of ground.

For drilled shafts that obtain the majority of their axial capacity in rock, no reduction in nominal axial capacity is needed for center-to-center spacings greater than or equal to two shaft diameters. Total settlement at the service design state of individual drilled shafts that are constructed using good installation techniques and equipment, as described in the 1999 FHWA manual, "Drilled Shafts: Construction Procedures and Design Methods," is anticipated to be about ½ inch. Differential settlement between adjacent piers would be expected to be about 50 to 75 percent of this value. A majority of the settlement is anticipated to occur as the load is applied with the remaining settlement occurring within the first year following construction.

Downdrag Loads

As stated previously, fill material will be placed at the abutment locations. The weight of the embankment fill material will induce consolidation of the underlying cohesive soils and elastic settlement of the granular soils. The elastic settlement will occur relatively rapidly, while the consolidation settlement will occur slowly over time, as discussed in the embankment and retaining wall sections below. Depending on the timing of abutment construction and drilled shaft installation, settlement of the existing soils subjected to the abutment loading may add downdrag loads to the drilled shafts.

We understand that embankments at both abutments will be constructed prior to drilled shaft installation. For our downdrag analysis, we assumed that the embankment fill materials would be placed a minimum of 6 months prior to drilled shaft installation. For this condition, at Abutment 1, our analysis indicated downdrag loads would not need to be considered, as settlement of the underlying granular soils would occur during or shortly after construction of the fill embankment. At Abutment 3, the sandy clay layer may still be undergoing primary and secondary consolidation at the 6-month period. Therefore, we assumed a downdrag loading at Abutment 3 due to settlement of the embankment fill material from the underlying sandy clay undergoing consolidation. The downdrag load is presented in Table 1.

Lateral Load Capacity

We understand that lateral load capacity will be evaluated using the computer program LPILE^{PLUS} by Ensoft, Inc. LRFD design methods for lateral loading of drilled shafts utilize a Resistance Factor of 1.0 and are controlled by allowable movement criteria. Input parameters for an individual drilled shaft are given in Table 1. Group action can be analyzed using P-multipliers within LPILE^{PLUS}. P-multipliers for loading perpendicular to a line of shafts are presented in Table 1, Note 11. P-multipliers for loading parallel to a line of shafts are presented in Table 2. As an alternative, the computer software Group 6.0 (Reese and Wang, 2003) can also analyze group action.

Abutment Embankments

Based on discussions with Wilson & Co., the embankments will be constructed with material excavated from the construction of Walls P-05-F and P-05-K. We understand that the predominant material that will be excavated from these areas will be a shale-derived soil. For our analysis, we assumed that this material will have a unit weight of 130 pounds per cubic foot (pcf), cohesion of 300 psf, and friction angle of 25 degrees.

We analyzed the global stability of the native soil/rock under the imposed embankment loadings using PCSTABL slope stability software developed by Purdue University and the Indiana Department of Highways, along with the STEDwin2.90 pre- and post-processor developed by Annapolis Engineering Software.

Three cases were analyzed for the native materials: short-term (undrained), long-term (drained), and seismic. For the seismic case, a horizontal acceleration coefficient of 0.05g was used, based on $A_s = 0.098g$. Based on our analyses, the native materials under the Abutment 1 retaining wall have a factor of safety greater than 1.5 for short-term, long-term, and seismic loading conditions. At Abutment 3, the native soils have a factor of safety of 1.4 for short-term loading, a factor of safety greater than 1.5 for long-term loading, and a factor of safety of 1.5 for seismic loading conditions.

The settlement potential of the fill embankment and/or retaining wall was evaluated for each abutment. At Abutment 1, the subsurface conditions generally consisted of about 31 feet of medium dense, clayey, silty sand overlying the bedrock. We estimate that settlement of these native materials at the center of the abutment due to the weight of the fill will be approximately 3.5 inches. We anticipate that the settlement will occur during or shortly after construction.

At Abutment 3, the native materials consisted of about 46 feet of medium stiff to very stiff, sandy clay overlying bedrock. We estimate that maximum settlement will be about 8 inches. We anticipate a majority of this settlement will be time-dependent due to the dissipation of excess pore pressures generated during loading. We estimate that primary consolidation will take on the order of 7 months, based on a coefficient of consolidation obtained from the CDOT consolidation test reports and the presence of sand layers in this formation in the project area (we assumed a maximum homogeneous layer thickness of about 25 feet for the time rate calculation). In addition, we anticipate 2 inches of long-term settlement due to secondary compression of the clay.

We estimate that the embankment or retaining wall itself will also undergo some settlement during or shortly after construction. For properly placed and compacted fill, settlement of embankment fill material should be less than 1/2 percent of the total embankment height. This settlement is anticipated to occur within the first year following construction.

Because of the time-dependent settlement anticipated at Abutment 3, we recommend that instrumentation be installed to monitor settlement with time at Abutment 3.

Mechanically Stabilized Earth (MSE) Wall

We understand that a MSE wall is the preferred alternative for Wall P-05-AK. Based on preliminary drawings, we estimate a maximum wall height of approximately 28 feet. The following table summarizes our recommended soil parameters for the MSE wall design. These properties are based on soil descriptions and field and laboratory test results by others, along with local practice and experience.

RECOMMENDED SOIL PARAMETERS FOR MSE WALL DESIGN

DESIGN PARAMETER	RECOMMENDED VALUE	
	Reinforced and Retained Fill [a]	Foundation
Total Unit Weight (pcf)	130	120
Friction Angle (degrees)	34	*-na-*
Nominal Coefficient of Friction for Sliding	*-na-*	0.45 [b]
Nominal Bearing Capacity for Reinforced Zone (psf)	*-na-*	8,000 [c]

NOTES: *pcf = pounds per cubic foot psf = pounds per square foot na = not applicable*

a. Reinforced and retained fill should consist of Colorado Department of Transportation (CDOT) Class 1 backfill. For the retained fill parameters shown above, it is assumed that CDOT Class 1 backfill will be located within a zone extending outward at an approximate 1 Horizontal to 1 Vertical (1H:1V) slope from the vertical from the back of the reinforced zone.
b. The wall should be founded on clayey sand. The coefficient of friction shown is a nominal value. A Resistance Factor (ϕ) of 0.85 should be applied to the nominal value.
c. The nominal bearing capacity assumes a minimum reinforced width of 8 feet. A Resistance Factor (ϕ) of 0.50 should be applied to the nominal value.

MSE wall design should consider the internal and external stability of the wall system and be performed in accordance with the recommendations and procedures presented in the latest edition of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim), including addenda and interim revisions, and Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines (Federal Highway Administration Publication No. FHWA-SA-96-071).

We understand that the MSE wall system manufacturer will analyze the internal and external stability of the MSE wall. Surcharge loads should be included in any wall design, including, but not limited to, concentrated construction equipment and material loads. Wall design should also consider the effects of sloping backfill, where necessary. Passive earth pressures against the face of the wall for that portion which is buried are negligible and should be ignored in wall design.

Based on global, deep-seated, stability analyses, we recommend a minimum reinforcement length of 0.7 times the wall height. However, the minimum reinforcement length should be 8 feet, regardless of wall height, in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim).

We estimate that total settlement of the MSE wall face will be on the order of 2 inches, based on a reinforced length of 20 feet and a service design load of 4,000 pounds per square foot (psf). Differential settlement over a 20-foot spacing along the continuous footing is estimated to be about 1-1/2 inches. Approximately half of the settlement is expected to occur as the load is applied, with the remaining settlement occurring within about one year after construction.

We understand that continuous unreinforced concrete leveling pads are proposed to support the wall face panels. We recommend that the leveling pads have a minimum width of 1 foot and be embedded at least 32 inches below the finished grade elevation of the ground in front of the wall for frost penetration.

We recommend a nominal bearing capacity of 5,000 psf for the leveling pads supported on clayey sand. A Resistance Factor (ϕ) of 0.5 should be applied to this nominal value.

Sulfates and Corrosion

Corrosion testing was not conducted on soil or rock samples taken at the Ramp A Bridge location. However, corrosion testing has been performed on soil and rock samples taken at the other three bridges. These corrosion test results suggest a negligible sulfate exposure to concrete in contact with these materials, based on classifications as defined by the American Concrete Institute (ACI) Manual ACI 318-05 (ACI, 2005).

The boring logs suggest that subsurface materials encountered at Ramp A Bridge are consistent with those at the other bridges. Thus, in our opinion, the subsurface materials at Ramp A Bridge would create a negligible sulfate exposure to concrete.

CONSTRUCTION CONSIDERATIONS

General

The applicability of the design recommendations provided in this letter is contingent on good construction practice. Poor construction techniques may alter conditions from those on which our recommendations are based, therefore, resulting in reduced foundation capacity and increased settlement. The following sections present additional construction and material considerations for this project.

Drilled Shaft Installation

Based on conditions observed while installing drilled shafts at US 550 Bridge, groundwater inflow should be anticipated above and below top of bedrock. If groundwater inflow is encountered above or within several feet below the top of bedrock, we recommend that casing be used to seal off local groundwater inflow. Casing should be pushed, rotated, vibrated or driven into an impermeable, firm stratum below the seepage zone. If casing is extended into bedrock, then the length of casing in the rock socket should be discounted from the minimum bedrock penetration length specified on the plans. The shaft should be extended deeper by a length equivalent to the casing protrusion into the bedrock.

If groundwater flow is encountered deeper within the bedrock, such that the use of casing is not possible, we recommend that drilling slurry not be allowed and bedrock exposure to water be kept to the minimum time necessary to complete drilling and place concrete. If the minimum penetration into bedrock and concreting process cannot be completed within the same day that drilling into bedrock initiated, we recommend that the shaft be extended deeper by 1/3 of the minimum bedrock penetration length specified on the plans. If the water depth at the bottom of the shaft is in excess of 2 inches at the time of concrete placement, then concrete should be placed by tremie pipe and pumping in accordance with CDOT Standard Specifications, Subsection 601.12(f), Depositing Concrete Under Water.

Upon completion of the shaft excavation, the hole should be cleaned of loose material and observed by the engineer prior to pouring. The drilling and concreting process should be

relatively continuous with minimal stoppage of work between the completion of drilling and cleaning the hole and placement of concrete after setting the rebar cage.

Frictional and end-bearing design parameters used in this analysis assume that the bottom of the shaft is properly cleaned and proper concrete placement techniques are used. To provide sufficient frictional resistance in rock, the specifications should require that the driller make a final pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7 inches from the sides of the auger to roughen the rock socket. The primary purpose of the roughening is to reduce the polished skin of the remolded material that can sometimes form in softer bedrock. This will be particularly important for drilled shafts with bedrock exposed to water.

For the construction of the drilled shafts, we recommend that the field installation reports provide the actual elevation at which bedrock is encountered and tip elevation for each drilled shaft. The blowcount values provided in the boring logs indicate that the shale and sandstone bedrock is a hard material which may create hard drilling conditions. In addition, cobbles and boulders, as inferred from the boring logs, may be present above bedrock at Abutments 1 and 3. Therefore, we recommend that the strength properties of the bedrock and the possibility of encountering cobbles and boulders be disclosed to drilling subcontractors.

Mechanically Stabilized Earth (MSE) Wall Foundations and Drainage

Excavations for the MSE wall base should be cleaned of all fill, debris, or loose material. It appears from the preliminary drawings that the base of the MSE wall will be placed within the clayey sand.

Excavations for the MSE retaining wall foundations should be observed by a geotechnical engineer to evaluate the adequacy of the bearing stratum and to confirm the subsurface conditions at or below the bearing elevations are suitable for the design bearing values given. Following excavation, the subgrade should be proof-rolled as discussed in the following Embankment and Subgrade Recommendations section.

The parameters provided for the MSE wall assume a free-draining backfill condition. As such, it will be important to control surface water and reduce its potential to infiltrate into the ground around the retaining walls. Surface water behind the wall should not be allowed to discharge directly into the wall backfill materials. In addition, water should not be allowed to discharge or pond around bridge abutments. A drainage layer should be placed against the back of the reinforced zone to reduce the potential for groundwater pressure to build-up behind the wall. A collection system (such as a perforated pipe surrounded by a layer of pea gravel) will be required to maintain a drained condition.

We recommend that the MSE wall contractor be cautioned that stockpiling of materials or operation of construction equipment on the reinforced zone, during or after wall construction, may cause face panel displacement.

Embankment and Subgrade Recommendations

Prior to placing fill, the subgrade soils should be proof-rolled with a large, heavy, vibratory roller or equivalent. Any loose, soft, or yielding areas should be overexcavated and replaced to the design base of wall elevation with Structure Backfill (Class 1) that is compacted to a minimum of 95 percent of its Modified Proctor dry density (AASHTO T-180).

Excavation for embankment, embankment foundation preparation, and embankment construction behind the MSE wall should conform to the requirements of the latest edition of the CDOT Standard Specifications for Road and Bridge Construction. Embankment material should be compacted to a minimum of 95 percent of its Modified Proctor (AASHTO T-180) dry density or Standard Proctor (AASHTO T-99) dry density for cohesive soils.

CLOSURE

The analyses, conclusions, and recommendations presented in this letter are based on a limited number of subsurface explorations and laboratory test results conducted by others. We assume that these explorations are representative of the subsurface conditions beneath the site; that is,

the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

This letter was prepared for the exclusive use of Wilson & Company and SEMA for use in design of Ramp A Bridge and Wall P-05-AK. It should be not be construed as a warranty of subsurface conditions.

Within the limitations of scope, schedule and budget, the analyses, conclusions and recommendations presented in this letter were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this letter was prepared. We make no other warranty, either express or implied.

Sincerely,

SHANNON & WILSON, INC.

Michael C. Hallman, P.E.
Senior Engineer

COLORADO REGISTERED
GREGORY ROBERT FISCHER
34461
PROFESSIONAL ENGINEER
9/11/08

Gregory R. Fischer, Ph.D., P.E.
Senior Vice President

MCH:KAK:GRF/mch

Enclosures: Table 1– Geotechnical Parameters for Design of Bridge Foundations at Ramp A Bridge
Table 2 – LPILE^{PLUS} Efficiency Factors for Loading Acting Parallel to the Line of Shafts
Figure 1 – Ramp A over US 160 General Layout Sheet
Appendix – Subsurface Explorations by Others

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TABLE 1
GEOTECHNICAL PARAMETERS FOR DESIGN OF BRIDGE FOUNDATIONS AT RAMP A BRIDGE

Structure	Location	Applicable Borings	Layer Elevation		Soil / Rock Type	Effective Unit Weight	Friction Angle[1]	Undrained Shear Strength	Drilled Shafts[3,4]							LPILE[PLUS] Ver. 4.0 Parameters for Lateral Analyses[1,7,8,9,10,11]			
									Skin Friction			Resistance Factors[2]		Downdrag[5,6]					
			Top (feet)	Bottom (feet)		γ' (pcf)	ϕ' (deg)	s_u (psf)	k*tanδ	Nominal Unit Adhesion (ksf)	Nominal Unit End Bearing (ksf)	Skin Friction	End Bearing	Load per Shaft (kips)	Maximum Depth of Downdrag (ft)	Soil Model Type	Strain at 50% Yielding ε_{50}	Initial Static Modulus of Subgrade Reaction k_s (pci)	Initial Cyclic Modulus of Subgrade Reaction k_c (pci)
Ramp A Bridge	Abutment 1 (South)	2B16, 2B18	6,637	6,609	New Fill	130	34	--	0.31	--	--	0.55	--	No Downdrag		Sand	--	90	90
			6,609	6,590	Med dense to dense, clayey, silty SAND	120	36	--	0.31	--	--	0.55	--			Sand	--	90	90
			6,590	6,578	Med dense to dense, clayey, silty SAND	58	36	--	0.31	--	--	0.55	--			Sand	--	60	60
			6,578		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Pier 2	2B20, YA-24	6,588	6,578	Stiff sandy CLAY	53	--	1,500	--	0.8	--	0.45	--	No Downdrag		Stiff Clay w/o Free Water	0.007	500	200
			6,578	6,573	Weakened Shale/Sandstone Bedrock	83	--	8,000	--	5	60	0.75	0.75			Stiff Clay w/o Free Water	0.005	1000	400
			6,573		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Abutment 3 (North)	B-125 2B22, YA-02, YA-23	6,642	6,609	New Fill	130	34	--	Ignore resistance because of downdrag			--	--	340	6,563	Sand	--	90	90
			6,609	6,580	Stiff to very stiff, sandy CLAY	115	--	1,500				--	--			Stiff Clay w/o Free Water	0.007	500	200
			6,580	6,568	Stiff to very stiff, sandy CLAY	53	--	1,500				--	--			Stiff Clay w/o Free Water	0.007	500	200
			6,568	6,563	Medium stiff, sandy CLAY	53	--	800				--	--			Stiff Clay w/o Free Water	0.009	300	120
			6,563		Shale/Sandstone Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800

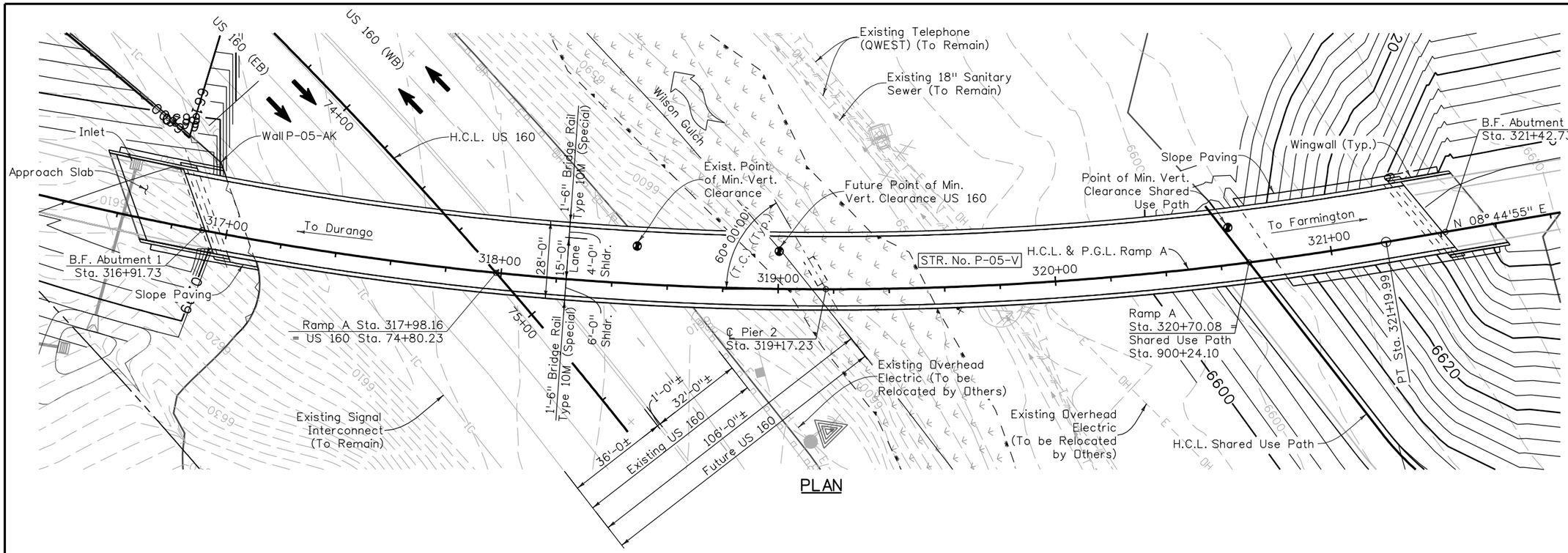
Notes:

1) Friction angle of 34 degrees used for new fill material is an equivalent friction angle for using the "Sand" model in LPILE[PLUS].
2) If a non-redundant, single shaft is used to support a bridge pier, the resistance factors should be reduced by 20 percent. For two closely spaced shafts, redundancy should be evaluated by the structural engineer. A resistance factor of 1.0 should be used for the service and extreme limit states. For uplift resistance, a resistance factor of 0.8 should be used for the extreme limit state.
3) Minimum bedrock penetration for all drilled shafts should be 3 diameters.
4) For drilled shafts that obtain the majority of the their axial capacity in rock, no reduction in axial capacity is needed for center-to-center spacings greater than or equal to 2 diameters.
5) Calculation of downdrag load assumes a 36-inch-diameter drilled shaft.
6) The downdrag should be treated as a load with the appropriate load factors applied.
7) The lateral loading analysis should consider the potential for flooding and scour.
8) The LPILE[PLUS] parameters shown are for horizontal ground surface. Sloping ground surface modifications should be included as per Ensoft, Inc.'s recommendations for the LPILE[PLUS] program.
9) The lateral resistance values provided assume that the lateral loads are short-term (non-sustained) loads. If sustained lateral loads are anticipated, we should be notified to provide additional parameters.
10) A resistance factor of 1.0 should be used for lateral analyses.
11) For tangent shafts where lateral loading is perpendicular to line of shafts, a P-multiplier of 0.5 should be used; for shaft spacing greater than or equal to 3 diameters, center-to-center, no P-multiplier is required. For spacings between 1 and 3 diameters, the P-multiplier can be interpolated between 0.5 and 1.0, respectively. For lateral loading parallel to the line of shafts refer to Table 2.

TABLE 2 - LPILE[PLUS] EFFICIENCY FACTORS FOR LOADING ACTING PARALLEL TO THE LINE OF SHAFTS

Spacing	P-multiplier
2.0 D	0.28
2.5 D	0.50
3.0 D	0.72
3.5 D	0.85
4.0 D	0.91
4.5 D	0.94
5.0 D	0.98
5.5 D	1.00

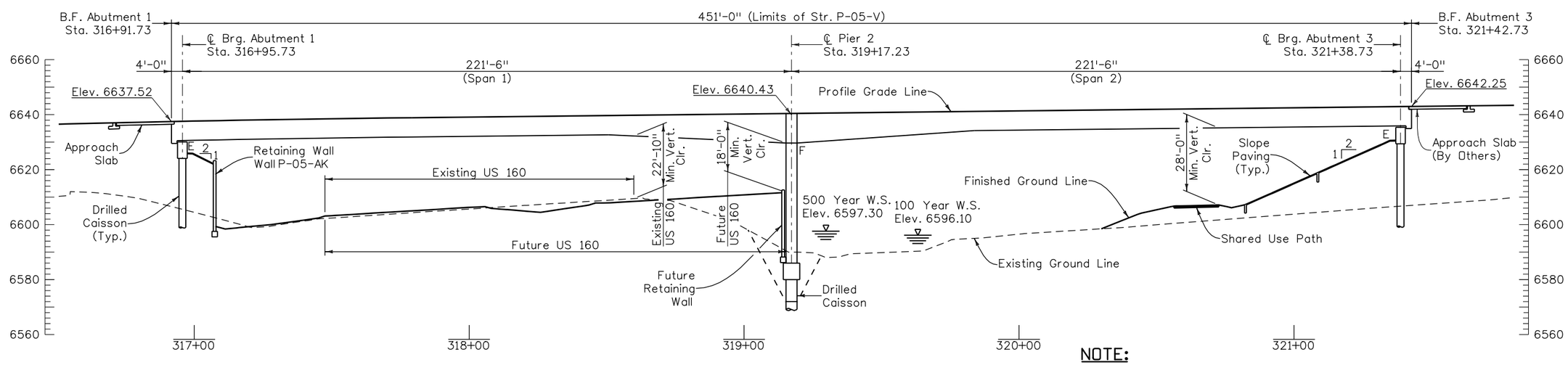
NOTE: The p-multipliers are based on recommendations presented in a 1998 Ensoft Seminar (Ensoft, 1998)



PLAN

RAMP A HORIZONTAL CURVE DATA

Δc=47° 49'49" RT.
Tc=532.15'
Lc =1001.76'
Rc =1200.00'
PI = Sta. 316+50.39



SECTION
(Taken Along P.G.L.)

NOTE:
Elevations are taken at top of 3" HMA Wearing Surface.

NOT FOR CONSTRUCTION - 30% PLANS - 07/28/08

Quantities	INITIAL	DATE
Quantities By		
Checked By		

Detail	INITIAL	DATE
Detailed By		
Checked By		

Design	INITIAL	DATE
Designed By		
Checked By		

Print Date: 7/29/2008
File Name: 16042V_GenLayout_01.dgn
Horiz. Scale: 1:40 Vert. Scale: As Noted
Unit Information Unit Leader STW

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
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Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

RAMP A OVER US 160
GENERAL LAYOUT

Designer:	B. Allen	Structure Numbers	P-05-V
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B3 of XX

Project No./Code
NH 1602-114
16042
Sheet Number

FIG. 1

APPENDIX A

SUBSURFACE EXPLORATIONS BY OTHERS

APPENDIX A

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TABLE

Table No.

A-1 Summary of Laboratory Test Results by Boring

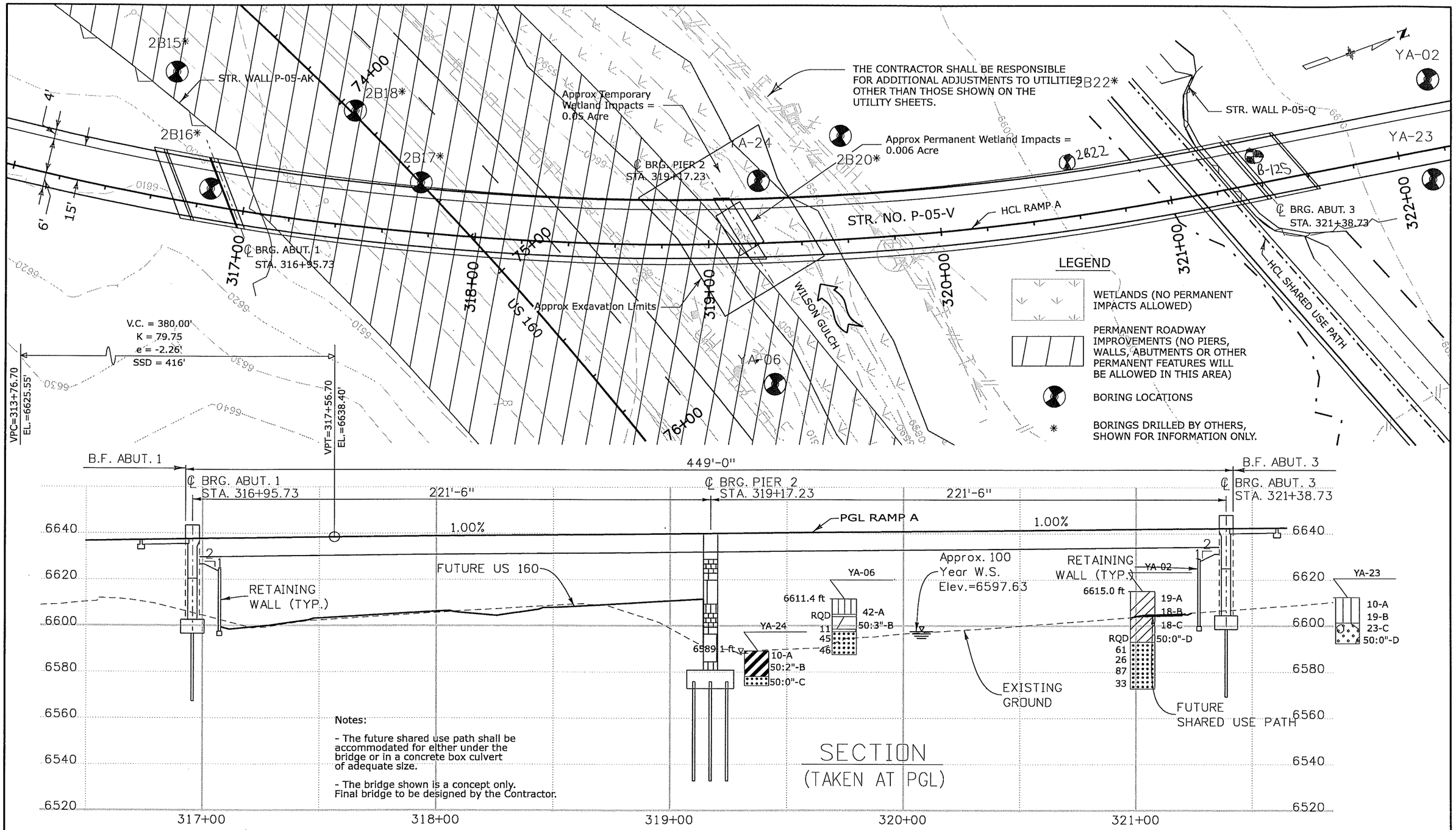
LIST OF FIGURES

Figure No.

A-1 Boring Locations (Ramp A (P-05-V) General Layout Sheet)
A-2 Summary Logs of Borings 2B16, 2B18, 2B20, 2B22
A-3 Log of Boring YA-02 (2 Pages)
A-4 Log of Boring YA-06
A-5 Log of Boring YA-23
A-6 Log of Boring YA-24
A-7 Log of Boring B-125

TABLE A-1
SUMMARY OF LABORATORY TEST RESULTS BY BORING

SAMPLE DATA		USCS Symbol	AASHTO Symbol	Natural Water Content	Wet Density	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION			STRENGTH TESTING
Boring	Depth					Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	% Fines	Unconfined Compressive Strength
				(%)	(pcf)	(%)	(%)	(%)				(psi)
2B15	10.0	SC				30	22	8			22	
2B17	14.0					37	24	13				
	19.0	ML		25.8		31	24	7			54	
	24.0	SM		26.6		35	25	10			35	
YA-02	5 - 6.5	SC	A-6(3)	20.3		33	21	12	1	52	47	
	14.7 - 16.2	SC	A-2-6(0)	15.5		30	19	11	21	50	29	
	26.5 - 27.1				147.6							4,330
YA-03	32 - 32.4				143.3							4,050
YA-04	31.1 - 31.7				153.6							7,670
YA-06	5.2 - 6.7	ML	A-7-6 (7)	23.6		44	28	16			56	
YA-23	5 - 6			12.3								



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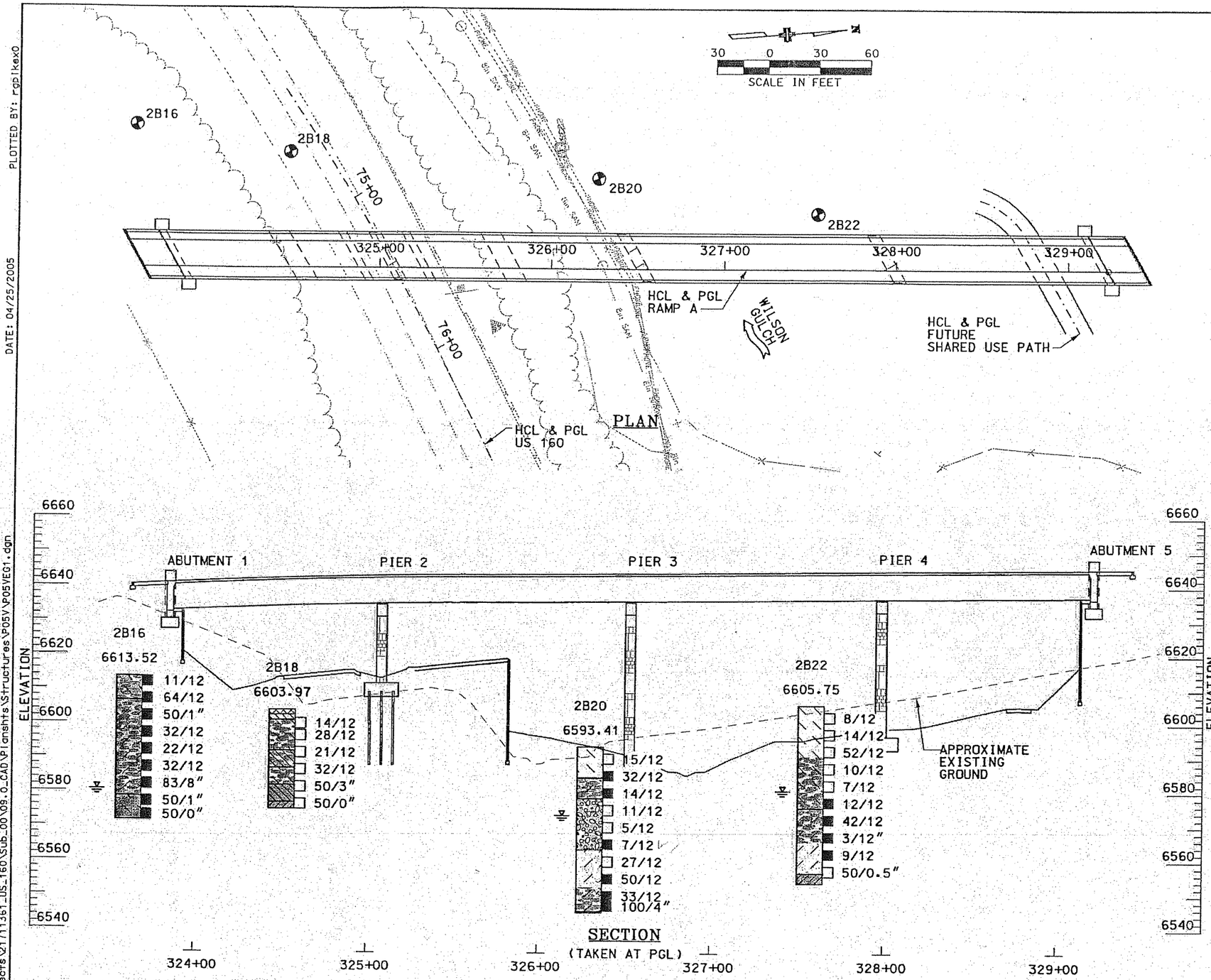
Horiz. Scale: 1:40 Vert. Scale: As Noted		Phone: 970-385-1440 FAX: 970-385-8365	Revised:
Unit Information EJA		Region 5 EJA	Void:

RAMP A (P-05-V) GENERAL LAYOUT			Project No./Code
Designer: spc	Structure Numbers	P-05-V	NH 1602-114
Detailer: spc			16042
Sheet Subset: Bridge	Subset Sheets: 2 of 4		Sheet Number 21

FIG. A-1



LEGEND

- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN.
- CLAY, SLIGHTLY SANDY TO SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL TO SLIGHTLY GRAVELLY, STIFF TO VERY STIFF, LOW PLASTICITY, MOIST TO WET, MEDIUM BROWN (CL)
- CLAY, SLIGHTLY SANDY TO SANDY, SILTY TO VERY SILTY, SOFT TO STIFF, HIGH PLASTICITY, MOIST TO WET, DARK GREY (CH)
- SILT, SLIGHTLY CLAYEY TO CLAYEY, SANDY TO VERY SANDY, MEDIUM STIFF TO STIFF, LOW PLASTICITY, MOIST, BROWN (ML)
- SAND, CLAYEY TO VERY CLAYEY, SILTY, TRACE GRAVEL TO GRAVELLY, MEDIUM DENSE TO VERY DENSE, MODERATE TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, BROWN TO GREY (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, LOOSE TO MEDIUM DENSE, MOIST TO WET, BROWN (GP, GM, GC)
- COBBLES AND BOULDERS
- CLAYSTONE, SANDY TO VERY SANDY, TRACE SILT TO SILTY, VERY HARD, MOIST TO WET, GREY (BEDROCK)
- SILTSTONE, SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, MOIST TO WET, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, MOIST TO WET, LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- APPROXIMATE BORING LOCATION
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

PLOTTED BY: rpikex0
DATE: 04/25/2005
FILE NAME: n:\projects\21711361_US_160\Sub_00\09.0_CAD\Plansht's\Structures\P05V\P05VE01.dgn

Computer File Information		Index of Revisions			
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Last Modification Date: SEE PLOT STAMP	Initials:	R—			
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Drawing File Name: SEE PLOT STAMP		R—			
Microstation Ver. V8	Scale: AS SHOWN Units:	R—			

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As Constructed	RAMP A ENGINEERING GEOLOGY			Project No./Code
No Revisions:				FC-NH(CX)160-2(048)
Revised:	Designer: K. YOUNG	Structure Numbers	P-05-V	91050
Void:	Detailer: R. PIKE			Sheet Number
	SheetSubset:	Subset Sheets:		

FIG. A-2

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-02** Sheet 1 of 2

Boring Began: 11/30/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/1/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 42.2 ft
Ground Elevation: 6615.0 ft
Location: approx. Sta. 5+25, Wall Q
Coordinates: N: 2,319,270.0 E: 1,213,409.4

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 14.7 ft. clayey SAND, brown, medium plasticity, medium dense, subangular to subrounded.	
			81						
6610	5				6/8/11	19			MC= 20.3 % #200= 47 % LL= 33 PL= 21 PI= 12 AASHTO: A-6 (3) USCS: SC
			41						
6605	10				3/8/10	18			
			10						
			12						
6600	15				6/8/10	18		**14.7 - 22.0 ft. clayey SAND** with gravel, brown, medium plasticity, medium dense, subangular to subrounded.	MC= 15.5 % #200= 29 % LL= 30 PL= 19 PI= 11 AASHTO: A-2-6 (0) USCS: SC
			50						
			94						
6595	20				9/12/50:0"	50:0"			Refusal due to boulder
								22.0 - 42.2 ft. SANDSTONE W/SHALE LAYERS, gray - brown, slightly weathered, medium hard to hard, foliation , grades to fresh hard conglomeratic sandstone at 32'.	
6590	25		100	61					Lost circulation
			94	27					

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-3
(Page 1 of 2)

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-02**
Sheet 2 of 2

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6580	35		100	88					
6575	40		90	33					
								Bottom of Hole at 42.2 ft.	
6570	45								
6565	50								
6560	55								
6555	60								
6550	65								

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-3
(Page 2 of 2)

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-06**
Sheet 1 of 1

Boring Began: 12/6/2006
Drilling Method: BX Wireline
Drill: GH-5 Viper
Driller: Salisbury & Associates
Logged By: J. Ebel
Final By: S. McKay
Inclination: Vertical

Completed: 12/6/2006
Drill Bit: BX
Casing: Steel Casing
Weather:

Total Depth: 24.2 ft
Ground Elevation: 6611.4 ft
Location: US160 Sta. 76+10, 40 ft. left
Coordinates: N: 2,319,290.3 E: 1,213,089.4

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6610			29					**0.0 - 6.7 ft. sandy SILT** with gravel and cobbles, brown, medium plasticity, hard, gravel is subangular, cobbles are subrounded to rounded.	
			88						
	5				19/22/20	42			MC= 23.6 %
6605									#200= 56 %
								6.7 - 14.2 ft. WEATHERED CLAYSTONE, brown to black, moderately weathered, soft to medium hard.	LL= 44
									PL= 28
									PI= 16
									AASHTO: A-7-6 (7)
	10				43/50:3"	50:3"			USCS: ML
6600			100	11					
	15							**14.2 - 24.2 ft. CONGLOMERATIC SANDSTONE**, blue-gray, fresh, medium hard to hard.	
6595			90	45					
	20								
6590			98	46					
	25							Bottom of Hole at 24.2 ft.	
6585									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-4

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-23**
Sheet 1 of 1

Boring Began: 3/28/2007
Drilling Method: Hollow-Stem Auger
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/28/2007
Drill Bit:
Casing:
Weather: snowy and cold

Total Depth: 20.0 ft
Ground Elevation: 6612.5 ft
Location: Ramp A approx. Sta. 324+00
Coordinates: N: 2,319,301.7 E: 1,213,383.8

Ground Water Notes: No groundwater encountered

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6610								**0.0 - 11.0 ft. silty SAND** with many small layers of gravel, brown, no to low plasticity, dry to moist, loose to medium dense, Cobbles especially from 3-4'.	
	5				5/5	10			MC= 12.3 %
6605									
	10				8/11	19			
6600								**11.0 - 20.0 ft. gravelly SAND** with silt, many cobble layers (3-5" diameter?), brown, no plasticity, dry, medium dense.	
	15				11/12	23			
6595									
	20				50:0"	50:0"		**20.0 - 20.0 ft.** Bedrock (?). Driller thinks we're on a boulder, however could easily be bedrock at this depth. Auger refusal at 20'. Sample had no recovery.	
6590								Bottom of Hole at 20.0 ft.	
	25								
6585									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-5

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-24** Sheet 1 of 1

Boring Began: 3/29/2007
Drilling Method: Hollow-Stem Auger
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/29/2007
Drill Bit:
Casing:
Weather: sunny and cold

Total Depth: 15.0 ft
Ground Elevation: 6589.1 ft
Location: Ramp A, 25' N of Pier 2
Coordinates: N: 2,319,204.4 E: 1,213,110.7

Ground Water Notes:

Depth	∇ 1.0 ft	-	-	-
Date	3/29/07	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 11.0 ft. CLAY grades to sandy clay near 10', dark brown, medium to high plasticity, wet, stiff.	
6585	5				1/9	10			
6580	10				4/50:2"	50:2"			California sampler had pieces of gray sandstone in the shoe-reason for high blows at the end.
								11.0 - 15.0 ft. SANDSTONE, gray, based on observation of auger cuttings; no sample or core was taken.	
6575	15				50:0"	50:0"		Bottom of Hole at 15.0 ft.	Auger cuttings 11-15 are gravel-sized pieces of gray sandstone. Drill broke down before coring begun. Unable to core due to repairs, and loss of coring barrel on hole #YA-21.
6570	20								
6565	25								
6560									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-6

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B125**

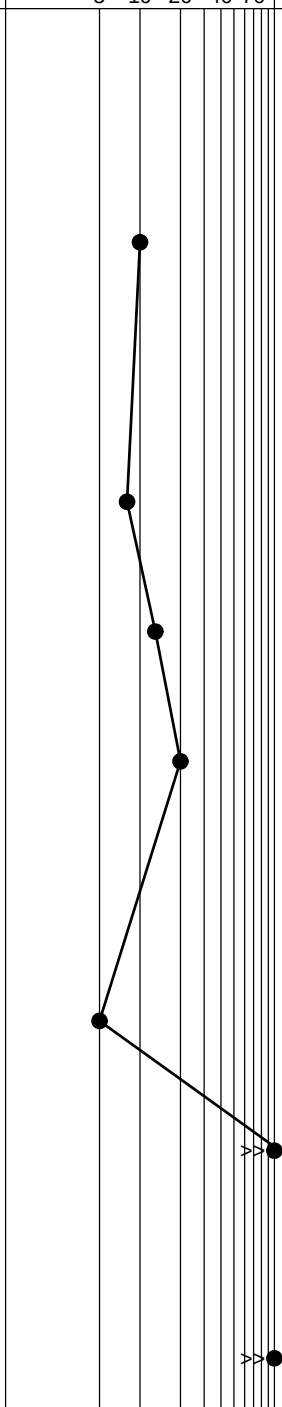
PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	160 Fourth Lane	6/4/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	Ramp A/North Abut	Durango

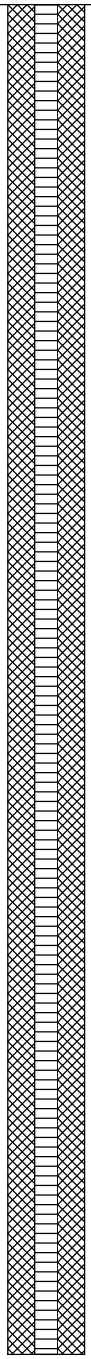
TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,607.0ft	52.0ft		I. Ksouri/D. Novak

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6605, 6600, 6595, 6590, 6585, 6580, 6575, 6570, 6565, 6560, 6555		Silty to sandy clay, medium stiff to stiff, dry to wet, brown to tan, some gravel encounter at 19' and below				
				9.0	A	10
				19.0	B	8
				24.0	C	13
				29.0	D	20
	39.0	Clay, medium stiff, high plasticity, wet, blue gray		39.0	E	5
	44.0	Shale bedrock, very hard, blue gray		44.0	F	50/4"
	52.0	Total Boring Depth 52.0ft		52.0	G	50/1"

SPT DATA



WELL DIAGRAM



SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 28.0				
DATE	6/4/08				
TIME	2:00 PM				

NOTES: Elevation Approximated

DRAFT

FIG. A-7

GEOLOGIC BORING LOG US160-4THLANE.GPJ CO_DOT.GDT 6/13/08

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

SEATTLE
RICHLAND
PORTLAND
FAIRBANKS
ANCHORAGE
DENVER
SAINT LOUIS
SACRAMENTO
JACKSONVILLE

August 1, 2008

Wilson & Company
999 18th Street, Suite 2600
Denver, CO 80202

Attn: Mr. Andy Leifheit

RE: 100% SUBMITTAL, GEOTECHNICAL DESIGN LETTER 001, REVISION 1 US 550 BRIDGE, CDOT PROJECT NO. NH1602-114, DURANGO, COLORADO

This letter provides geotechnical design recommendations for the proposed US 550 Bridge and associated retaining wall at Abutment 1. This letter supersedes previous submittals for this bridge.

STRUCTURE DESCRIPTION

The proposed 3-span bridge over Wilson Gulch is designated as the US 550 Bridge, Colorado Department of Transportation (CDOT) Structure No. P-05-AG (Figure 1). The bridge will trend approximately north-south and be about 59 feet wide. The south abutment is designated as Abutment 1 (Sta. 266+00) and the north abutment is designated as Abutment 4 (Sta. 271+22). Two interior piers, designated Pier 2 (Sta. 268+11.5) and Pier 3 (Sta. 270+23) are planned to support the bridge. The abutments and piers will be supported on drilled shaft foundations.

To achieve the proposed roadway grade elevations at the bridge, fill material will be placed at both abutments. At Abutments 1 and 4, fill heights will approach 32 and 48 feet, respectively. Due to the slope of the existing ground surface north of Abutment 1 and the location of US 160, a retaining wall (CDOT Wall No. P-05-T) is planned to support the north side of the new fill embankment at Abutment 1. At Abutment 4, the fill embankment will be sloped at 2 Horizontal to 1 Vertical (2H:1V) with slope paving utilized on the south side of the abutment.

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SUBSURFACE CONDITIONS

Our interpretation of subsurface conditions at the bridge site is based on borings drilled by CDOT and others. The approximate location of the borings and the boring logs are provided in Appendix A. A summary of laboratory test results are presented in Table A-1.

The subsurface profile at the bridge was interpreted from the following borings: 2B30, 2B31, 2B33, 2B35, B-103, B-104, B-105, B-121, and B-122. In general, subsurface conditions consisted of medium stiff to stiff, sandy to silty clay overlying dense sand and gravel, overlying shale bedrock. At Abutment 1, the conditions were slightly different, with medium dense, silty sand encountered from the existing ground surface to the top of shale bedrock. The boring logs describe the shale as a hard fractured material, with joints that contain some slickensides and iron staining. The top of shale bedrock varied across the site and appears to follow the existing ground surface. The top of shale bedrock in the above-referenced borings, ranged from 6,656 to 6,591 feet.

Groundwater was noted in some of the borings at elevations ranging from about 6,606 to 6,595 feet. Groundwater was not encountered in borings drilled adjacent to Abutment 1. Groundwater levels were recorded in each boring as encountered at the time of drilling. We assumed that the upper range of these measurements is representative of long-term conditions. The groundwater elevations used in our analyses are shown in Table 1.

SEISMICITY

Seismic Site Classification

We understand that seismic design of the bridge will be in accordance AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). Based on data collected from the borings, the soil conditions at the site are consistent with Site Class C.

Seismic Hazards

Liquefaction, lateral spreading, and dynamic settlement are factors that can negatively affect the stability and load carrying capacity of foundations and structures. Based on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim) and USGS (USGS, 2002), the peak ground acceleration for a 1,000-year event and Site Class C is 0.08g. In our opinion, because the peak horizontal ground acceleration (PGA) for this area of Colorado is low, there is a low risk potential associated with these seismic hazards, and such seismic hazards were not considered as part of design of the structure.

GEOTECHNICAL RECOMMENDATIONS

Drilled Shafts

We understand that drilled shafts are the preferred foundation support for the abutments and piers. The design criteria presented herein for drilled shafts uses load and resistance factor design (LRFD). The LRFD criteria were developed based on AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). In addition, Federal Highway Administration (FHWA) design methods (O'Neil and Reese, 1999) and a CDOT research report (Abu-Hejleh et al., 2003) were used to supplement drilled shaft design procedures.

Axial Capacity Parameters

Axial design parameters for use in sizing bridge foundations, as well as, Resistance Factors (ϕ) for both tip and side resistance, are presented in Table 1. The nominal axial capacities in the bedrock are based on Standard Penetration Test (SPT) blow counts, using the methodology presented in Section 6.4.2 in the CDOT Research Report 2003-6.

For this method, the shale bedrock is defined as a "soil-like claystone". Average blow counts for the shale bedrock, as indicated from the borings, were on the order of 50/3". For our analysis, we used 50/5" which would be equivalent to a blow count of 120.

The CDOT Research Report 2003-6 also presents design methodologies for estimating ultimate (nominal) skin and end bearing values for a " very hard sandy claystone" (Section 6.4.4) and "very hard clayey sandstone" (Section 6.4.5). In our opinion, these rock descriptions would be more consistent with the bedrock encountered at the site.

A parametric study between methodologies for "soil-like claystone" (Section 6.4.2), "very hard sandy claystone" (Section 6.4.4), and "very hard clayey sandstone" (Section 6.4.5) was conducted to compare the nominal skin friction and end bearing values determined from each method. An unconfined compressive strength of 598 psi (taken from Sample 105-2 in Boring B-105) was used in the study for bedrock, which was the lowest strength value among rock samples tested at this bridge location (see Table A-1). Our study concluded that our nominal skin friction and end bearing values using the "soil-like claystone" procedure were conservative in comparison to the other methods.

At the Abutment 1 location, we assumed a weakened zone within the top 17 feet of the shale bedrock, as shown in Table 1. This weakened zone was used as a conservative measure for design purposes only, due to the lower blow count and low RQD values shown in Boring B-121.

For drilled shafts that obtain the majority of their axial capacity in rock, no reduction in nominal axial capacity is needed for center-to-center spacings greater than or equal to two shaft diameters. Total settlement at the service design state of individual drilled shafts designed using the parameters discussed above and constructed using good installation techniques and equipment, as described in the 1999 FHWA manual, "Drilled Shafts: Construction Procedures and Design Methods," is anticipated to be about ½ inch. Differential settlement between adjacent piers would be expected to be about 50 to 75 percent of this value. A majority of the settlement is anticipated to occur as the load is applied with the remaining settlement occurring within the first year following construction.

Downdrag Loads

As stated previously, fill material will be placed at the abutment locations. The weight of the embankment fill material will induce consolidation of the underlying cohesive soils and elastic settlement of the granular soils. The elastic settlement will occur relatively rapidly, while the consolidation settlement will occur slowly over time, as discussed in the embankment and retaining wall sections below. Depending on the timing of abutment construction and drilled shaft installation, settlement of the existing soils subjected to the abutment loading may add downdrag loads to the drilled shafts.

At Abutment 1, shallow bedrock was encountered at the existing ground surface. In our opinion, downdrag loads need not be considered at this location. We understand that the embankment at Abutment 4 will be constructed shortly before drilled shaft installation. At Abutment 4, the sandy, silty clay layer will be undergoing primary consolidation at the time of drilled shaft installation. Therefore, we assumed a downdrag loading at Abutment 4, due to settlement of the embankment fill material from the underlying sandy, silty clay layer undergoing consolidation. The downdrag load is presented in Table 1.

Lateral Load Capacity

We understand that lateral load capacity will be evaluated using the computer program LPILE^{PLUS} by Ensoft, Inc. LRFD design methods for lateral loading of drilled shafts utilize a Resistance Factor of 1.0 and are controlled by allowable movement criteria. Input parameters for an individual drilled shaft are given in Table 1. Group action can be analyzed using P-multipliers within LPILE^{PLUS}. P-multipliers for loading perpendicular to a line of shafts are presented in Table 1, Note 11. P-multipliers for loading parallel to a line of shafts are presented in Table 2. As an alternative, the computer software Group 6.0 (Reese and Wang, 2003) can also analyze group action.

Abutment Embankments

Embankment fills are necessary at each abutment. At Abutment 1, a 32-foot-high embankment is planned with a mechanically stabilized earth (MSE) wall supporting the north side. At Abutment 4, a 48-foot-high embankment is planned with side slopes of 2H:1V.

At Abutment 1, the shale bedrock was encountered at the existing ground surface. For our slope stability analysis, we assumed that the strength of the shale bedrock was consistent throughout the unit. In addition, based on observations during out site reconnaissance, it appears that bedding planes within the shale are not oriented downslope.

Based on discussions with Wilson & Co., the embankments will be constructed with material excavated from the construction of Walls P-05-F and P-05-K. We understand that the predominant material that will be excavated from these areas will be a shale-derived soil. For our analysis, we assumed that this material will have a unit weight of 130 pounds per cubic foot (pcf), cohesion of 300 psf, and friction angle of 25 degrees.

We analyzed the stability of the native soil/rock under the imposed embankment loading using PCSTABL slope stability software developed by Purdue University and the Indiana Department of Highways, along with the STEDwin2.90 pre- and post-processor developed by Annapolis Engineering Software.

Three cases were analyzed for the native materials: short-term (undrained), long-term (drained), and seismic. For the seismic case, a horizontal acceleration coefficient of 0.03g was used, based on a peak horizontal ground acceleration of 0.06g for a 1,000-year event. The native materials under the Abutment 1 fill embankment have a factor of safety greater than 1.5 for short-term, long-term, and seismic loading conditions, due to the presence of shallow shale bedrock. At Abutment 4, the native soils have a factor of safety of 1.2 for short-term loading, a factor of safety of 1.5 for long-term and seismic loading.

The settlement potential of the fill embankments at each abutment was evaluated. At Abutment 1, shale bedrock was encountered at the existing ground surface. We estimate that

settlement of the bedrock due to the weight of the fill embankment will be less than 1 inch. We anticipate that the settlement will occur during or shortly after construction.

At Abutment 4, the native materials consisted of about 31 feet of sandy, silty clay overlying shale bedrock. We estimate that settlement of the sandy, silty clay will be on the order of 9 inches. Settlement will be time-dependent due to the dissipation of excess pore pressures in the clay generated during loading. We estimate that primary consolidation (9-inch magnitude of settlement) may be on the order of 10 months. We anticipate an additional 1.5 inches of settlement within the next 10 years following primary consolidation, due to secondary compression of the clay.

In addition to the estimated settlement of the native materials, we estimate that the embankment itself will undergo some settlement during or shortly after construction. For properly placed and compacted fill, settlement of embankment fill material should be less than 1/2 percent of the total embankment height. This settlement is anticipated to occur within the first year following construction.

Because of the time-dependent settlement anticipated at Abutment 4, we recommend that instrumentation be installed to monitor settlement with time. Typically, settlement plates are used to monitor embankment settlement. The base plate should be installed just below the current ground surface before placement of the fill and connection pipes should extend vertically through the fill to the top of the constructed embankment. The elevation of the top of pipe should be surveyed periodically and plotted versus time to evaluate the magnitude and rate of consolidation both during construction and after the embankment is built. A typical settlement plate configuration is shown in Figure 2.

Mechanically Stabilized Earth (MSE) Wall

We understand that a MSE wall is the preferred alternative for Wall P-05-T. Based on preliminary drawings, we estimate a maximum wall height of approximately 30 feet. The following table summarizes our recommended soil parameters for the MSE wall design. These

properties are based on soil descriptions and field and laboratory test results by others, along with local practice and experience.

RECOMMENDED SOIL PARAMETERS FOR MSE WALL DESIGN

DESIGN PARAMETER	RECOMMENDED VALUE	
	Reinforced and Retained Fill [a]	Foundation
Total Unit Weight (pcf)	130	125
Friction Angle (degrees)	34	*-na-*
Nominal Coefficient of Friction for Sliding	*-na-*	0.45 [b]
Nominal Bearing Capacity for Reinforced Zone (psf)	*-na-*	8,000 [c]

NOTES: *pcf = pounds per cubic foot psf = pounds per square foot na = not applicable*

a. Reinforced and retained fill should consist of Colorado Department of Transportation (CDOT) Class 1 backfill. For the retained fill parameters shown above, it is assumed that CDOT Class 1 backfill will be located within a zone extending outward at an approximate 1 Horizontal to 1 Vertical (1H:1V) slope from the vertical from the back of the reinforced zone.
b. The wall should be founded on shale bedrock. The coefficient of friction shown is a nominal value for shale bedrock. A Resistance Factor (ϕ) of 0.85 should be applied to the nominal value.
c. The nominal bearing capacity assumes a minimum reinforced width of 8 feet. A Resistance Factor (ϕ) of 0.50 should be applied to the nominal value.

MSE wall design should consider the internal and external stability of the wall system and be performed in accordance with the recommendations and procedures presented in the latest edition of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim), including addenda and interim revisions, and Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines (Federal Highway Administration Publication No. FHWA-SA-96-071).

We understand that the MSE wall system manufacturer will analyze the internal and external stability of the MSE wall. Surcharge loads should be included in any wall design, including, but not limited to, concentrated construction equipment and material loads. Wall design should

also consider the effects of sloping backfill, where necessary. Passive earth pressures against the face of the wall for that portion which is buried are negligible and should be ignored in wall design.

Based on global, deep-seated, stability analyses, we recommend a minimum reinforcement length of 0.7 times the wall height. However, the minimum reinforcement length should be 8 feet, regardless of wall height, in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). To provide adequate bearing capacity and global stability for the wall, we recommend the base be supported on the shale bedrock.

We estimate that total settlement of the MSE wall base will be less than ½ inch, based on a reinforced length of 21 feet and a service design load of 4,000 pounds per square foot (psf). Differential settlement over a 20-foot spacing along the continuous footing is estimated to be less than ⅜ inch. Approximately half of the settlement is expected to occur as the load is applied, with the remaining settlement occurring within about one year after construction.

We understand that continuous unreinforced concrete leveling pads are proposed to support the wall face panels. We recommend that the leveling pads have a minimum width of 1 foot and be embedded at least 32 inches below the finished grade elevation of the ground in front of the wall for frost penetration. In accordance with AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim), the toe of the leveling pad should be positioned a minimum horizontal distance of 4 feet from the sloping ground surface.

We recommend a nominal bearing capacity of 7,000 psf for the leveling pads supported on shale bedrock. A Resistance Factor (ϕ) of 0.5 should be applied to this nominal value.

Sulfates and Corrosion

We reviewed corrosion test results conducted by others to estimate the corrosion potential of the soil/rock. Two samples of sandy, silty clay at a depth of 4.5 to 6 feet below the existing ground surface in Borings B-103 and B-104 were tested for water soluble sulfates. Two samples were taken. In addition, we completed one additional corrosion test on a sample taken

in the shale bedrock at a depth of approximately 35 to 40 feet below the existing ground surface in Boring B-122. The test results are summarized in Table A-1.

Based on classifications as defined by the American Concrete Institute (ACI) Manual ACI 318-05 (ACI, 2005), the test results suggest that the sandy, silty clay and the shale bedrock create a negligible sulfate exposure to concrete in contact with these materials.

CONSTRUCTION CONSIDERATIONS

General

The applicability of the design recommendations provided in this letter is contingent on good construction practice. Poor construction techniques may alter conditions from those on which our recommendations are based, therefore, resulting in reduced foundation capacity and increased settlement. The following sections present additional construction and material considerations for this project.

Drilled Shaft Installation

Based on conditions observed at the project site, groundwater inflow may be anticipated above the shale bedrock. Depending on the type of drilling techniques employed and the speed of installation, casing likely may be required to seal off local groundwater inflow. Casing should be pushed, rotated, vibrated or driven into an impermeable, firm stratum below the seepage zone. Groundwater inflow must be sealed off prior to penetration into the shale bedrock and the use of slurry within the rock socket zone should not be allowed. Installation techniques should be compliant with the 1999 FHWA manual, "Drilled Shafts: Construction Procedures and Design Methods."

Upon completion of the shaft excavation, the hole should be cleaned of loose material and observed by the engineer prior to pouring. The drilling and concreting process should be relatively continuous with minimal stoppage of work between the completion of drilling and

cleaning the hole and placement of concrete after setting the rebar cage, and under no circumstances after the day of drilling.

Frictional and end-bearing design parameters used in this analysis assume that the bottom of the shaft is properly cleaned and proper concrete placement techniques are used. To provide sufficient frictional resistance in rock, drillers should not be allowed to use drilling slurry or add water during drilling or to leave any casing in the rock portion of the drilled shaft after drilling. The specifications should require that the driller make a final pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7 inches from the sides of the auger to roughen the rock socket. The primary purpose of the roughening is to reduce the polished skin of the remolded material that can sometimes form in softer shale bedrock.

The blowcount values provided in the boring logs indicate that the shale bedrock is a hard material which may create hard drilling conditions. In addition, cobbles and boulders, as inferred from the boring logs, may be present in the overburden soils. Therefore, we recommend that the strength properties of the bedrock and the possibility of encountering cobbles and boulders be disclosed to drilling subcontractors.

Mechanically Stabilized Earth (MSE) Wall Foundations and Drainage

Excavations for the MSE wall base should be cleaned of all fill, debris, or loose material. It appears from the preliminary drawings that the base of the MSE wall will be placed within the shale bedrock. We recommend that the shale not be exposed to air for prolonged time periods due to an increased potential to weaken. Therefore, we recommend that excavation areas be kept small, exposure to water be avoided, and exposure to air kept to the minimum time necessary to complete the foundation.

Excavations for the MSE retaining wall foundations should be observed by a geotechnical engineer to evaluate the adequacy of the bearing stratum and to confirm the subsurface conditions at or below the bearing elevations are suitable for the design bearing values given. Prior to beginning wall or leveling pad construction, the subgrade should be proof-rolled as described in the Embankment and Subgrade Recommendations section of this report.

The parameters provided for the MSE wall assume a free-draining backfill condition. As such, it will be important to control surface water and reduce its potential to infiltrate into the ground around the retaining walls. Surface water behind the wall should not be allowed to discharge directly into the wall backfill materials. In addition, water should not be allowed to discharge or pond around bridge abutments. A drainage layer should be placed against the back of the reinforced zone to reduce the potential for groundwater pressure to build-up behind the wall. A collection system (such as a perforated pipe surrounded by a layer of pea gravel) will be required to maintain a drained condition.

We recommend that the MSE wall contractor be cautioned that stockpiling of materials or operation of construction equipment on the reinforced zone, during or after wall construction, may cause face panel displacement.

Embankment and Subgrade Recommendations

Prior to placing fill, the subgrade soils should be proof-rolled with a large, heavy, vibratory roller or equivalent. Any loose, soft, or yielding areas should be overexcavated and replaced to the design base of wall elevation with Structure Backfill (Class 1) that is compacted to a minimum of 95 percent of its Modified Proctor dry density (AASHTO T-180).

Excavation for embankment, embankment foundation preparation, and embankment construction behind the MSE wall should conform to the requirements of the latest edition of the CDOT Standard Specifications for Road and Bridge Construction. Embankment material should be compacted to a minimum of 95 percent of its Modified Proctor (AASHTO T-180) dry density or Standard Proctor (AASHTO T-99) dry density for cohesive soils.

CLOSURE

The analyses, conclusions, and recommendations presented in this letter are based on a limited number of subsurface explorations and laboratory test results conducted by others. We assume that these explorations are representative of the subsurface conditions beneath the site; that is,

the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

This letter was prepared for the exclusive use of Wilson & Company and SEMA for use in design of the US 550 Bridge and Wall P-05-T. It should not be construed as a warranty of subsurface conditions.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this letter were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this letter was prepared. We make no other warranty, either express or implied.

Sincerely,

SHANNON & WILSON, INC.

Michael C. Hallman, P.E.
Senior Engineer

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Enclosures: Table 1– Geotechnical Parameters for Design of Bridge Foundations at US 550 Bridge
Table 2 – LPILE^{PLUS} Efficiency Factors for Loading Acting Parallel to the Line of Shafts
Figure 1 – US 550 over US 160 General Layout Sheet
Figure 2 – Typical Settlement Plate Configuration
Appendix A – Subsurface Explorations by Others

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Reese, L.C., and Wang, S. T., 2003, Documentation of Computer Program Group, Version 6.0: Austin, Tex., Ensoft, Inc.

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TABLE 1
GEOTECHNICAL PARAMETERS FOR DESIGN OF BRIDGE FOUNDATIONS AT US 550 BRIDGE

Structure	Location	Applicable Borings	Layer Elevation		Soil / Rock Type	Effective Unit Weight	Friction Angle[1]	Undrained Shear Strength	Drilled Shafts[3,4]							LPILE[PLUS] Ver. 4.0 Parameters for Lateral Analyses [1,7,8,9,10,11]			
									Skin Friction			Resistance Factors[2]		Downdrag[5,6]					
			Top	Bottom					k*tanδ	Nominal Unit Adhesion	Nominal Unit End Bearing	Skin Friction	End Bearing	Load per Shaft	Maximum Depth of Downdrag	Soil Model Type	Strain at 50% Yielding	Initial Static Modulus of Subgrade Reaction	Initial Cyclic Modulus of Subgrade Reaction
			(feet)	(feet)		γ' (pcf)	ϕ' (deg)	s_u (psf)		(ksf)	(ksf)			(kips)	(ft)		ε_{50}	k_s (pci)	k_c (pci)
550 Bridge	Abutment 1 (South)	2B30, 2B31, B-105, B-121	6,690	6,658	New Fill	130	34	--	0.31	--	--	0.55	--	No Downdrag		Sand	--	90	90
			6,658	6,641	Weakened Shale Bedrock	145	--	8,000	--	5	60	0.75	0.75			Stiff Clay w/o Free Water	0.005	1000	400
			6,641		Blue-Gray Shale Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Pier 2	2B31, 2B33, B-104	6,616	6,592	Medium stiff to stiff, sandy CLAY	58	--	1,000	--	0.8	--	0.45	--	No Downdrag		Stiff Clay w/o Free Water	0.009	300	120
			6,592		Shale Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Pier 3	2B35, B-103	6,613	6,591	Medium stiff to stiff, sandy CLAY	58	--	1,000	--	0.8	--	0.45	--	No Downdrag		Stiff Clay w/o Free Water	0.009	300	120
			6,591		Shale Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800
	Abutment 4 (North)	2B35, B-103, B-122	6,674	6,626	New Fill	130	34	--	Ignore resistance because of downdrag			--	--	560	6,605	Sand	--	90	90
			6,626	6,605	Stiff, sandy, silty CLAY	120	--	1,500				--	--			Stiff Clay w/o Free Water	0.007	500	200
			6,605	6,591	Dense, gravelly SAND w/ cobbles	58	36	--	0.31			--	--			Sand	--	60	60
			6,591		Shale Bedrock	83	--	15,000	--	9	110	0.75	0.75			Stiff Clay w/o Free Water	0.004	2000	800

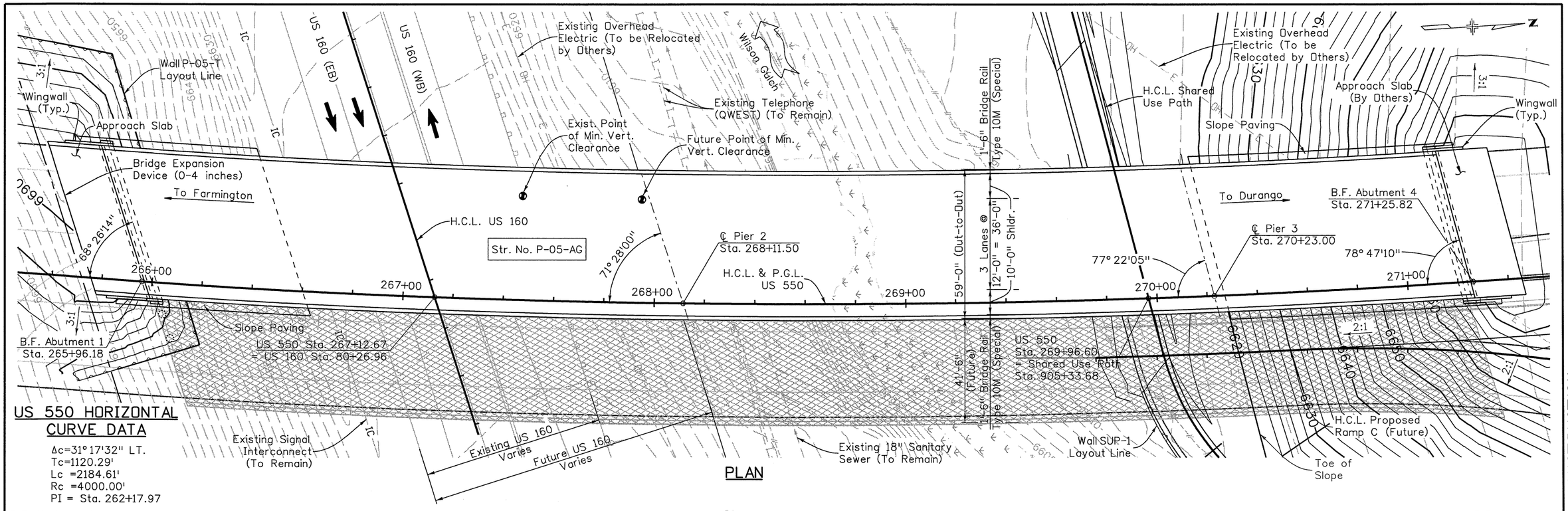
Notes:

1) Friction angle of 34 degrees used for new fill material is an equivalent friction angle for using the "Sand" model in LPILE[PLUS].
2) If a non-redundant, single shaft is used to support a bridge pier, the resistance factors should be reduced by 20 percent. For two closely spaced shafts, redundancy should be evaluated by the structural engineer. A resistance factor of 1.0 should be used for the service and extreme limit states. For uplift resistance, a resistance factor of 0.8 should be used for the extreme limit state.
3) Minimum bedrock penetration for drilled shafts at Abutment 1 should be 1 diameter into the blue-gray shale bedrock. At Pier 2, Pier 3, and Abutment 4, minimum bedrock penetration for drilled shafts should be 3 diameters.
4) For drilled shafts that obtain the majority of the their axial capacity in rock, no reduction in axial capacity is needed for center-to-center spacings of greater than or equal to 2 diameters.
5) Calculation of downdrag load assumes a 36-inch-diameter drilled shaft.
6) The downdrag should be treated as a load with the appropriate load factors applied.
7) The lateral loading analysis should consider the potential for flooding and scour.
8) The LPILE[PLUS] parameters shown are for horizontal ground surface. Sloping ground surface modifications should be included as per Ensoft, Inc.'s recommendations for the LPILE[PLUS] program.
9) The lateral resistance values provided assume that the lateral loads are short-term (non-sustained) loads. If sustained lateral loads are anticipated, we should be notified to provide additional parameters.
10) A resistance factor of 1.0 should be used for lateral analyses.
11) For tangent shafts where lateral loading is perpendicular to line of shafts, a P-multiplier of 0.5 should be used; for shaft spacing greater than or equal to 3 diameters, center-to-center, no P-multiplier is required. For spacings between 1 and 3 diameters, the P-multiplier can be interpolated between 0.5 and 1.0, respectively. For lateral loading parallel to the line of shafts refer to Table 2.

TABLE 2 - LPILE[PLUS] EFFICIENCY FACTORS FOR LOADING ACTING PARALLEL TO THE LINE OF SHAFTS

Spacing	P-multiplier
2.0 D	0.28
2.5 D	0.50
3.0 D	0.72
3.5 D	0.85
4.0 D	0.91
4.5 D	0.94
5.0 D	0.98
5.5 D	1.00

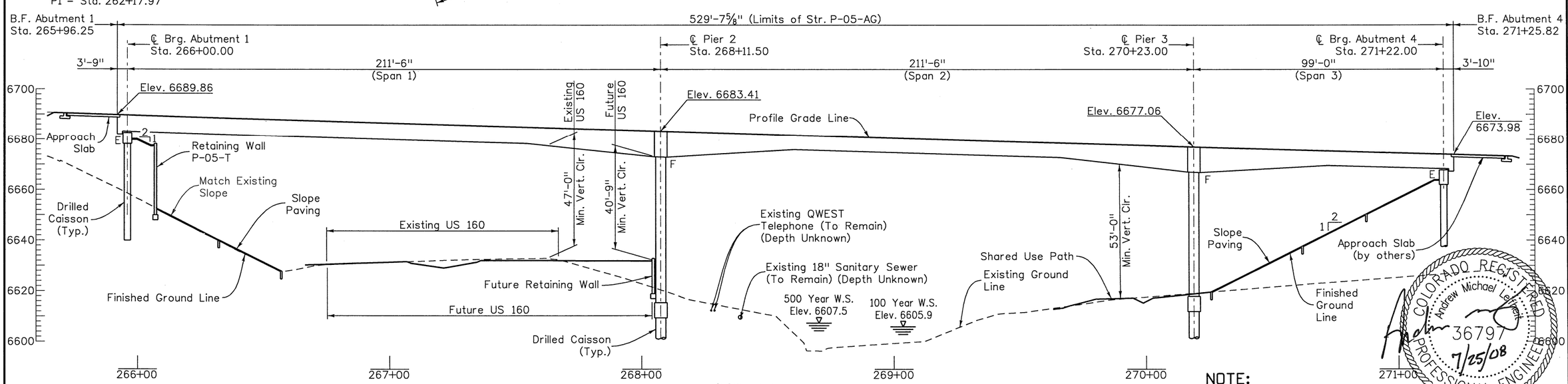
NOTE: The p-multipliers are based on recommendations presented in a 1998 Ensoft Seminar (Ensoft, 1998)



US 550 HORIZONTAL CURVE DATA

$\Delta c = 31° 17' 32''$ LT.
Tc=1120.29'
Lc =2184.61'
Rc =4000.00'
PI = Sta. 262+17.97

PLAN



SECTION

(Taken Along P.G.L.)

NOTE:
Elevations are taken at top of 3" HMA wearing surface.

COLORADO REGISTERED PROFESSIONAL ENGINEER
Andrew Michael Leifheit
36797
7/25/08

RELEASED FOR CONSTRUCTION

Quantities	INITIAL	DATE
Quantities By		
Checked By		

Detail	INITIAL	DATE
Detailed By		
Checked By		

Design	INITIAL	DATE
Designed By		
Checked By		

Print Date: 7/25/2008
File Name: 16042AG_GenLayout_01.dgn
Horiz. Scale: 1:40 Vert. Scale: As Noted
Unit Information 0221 Unit Leader STW

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

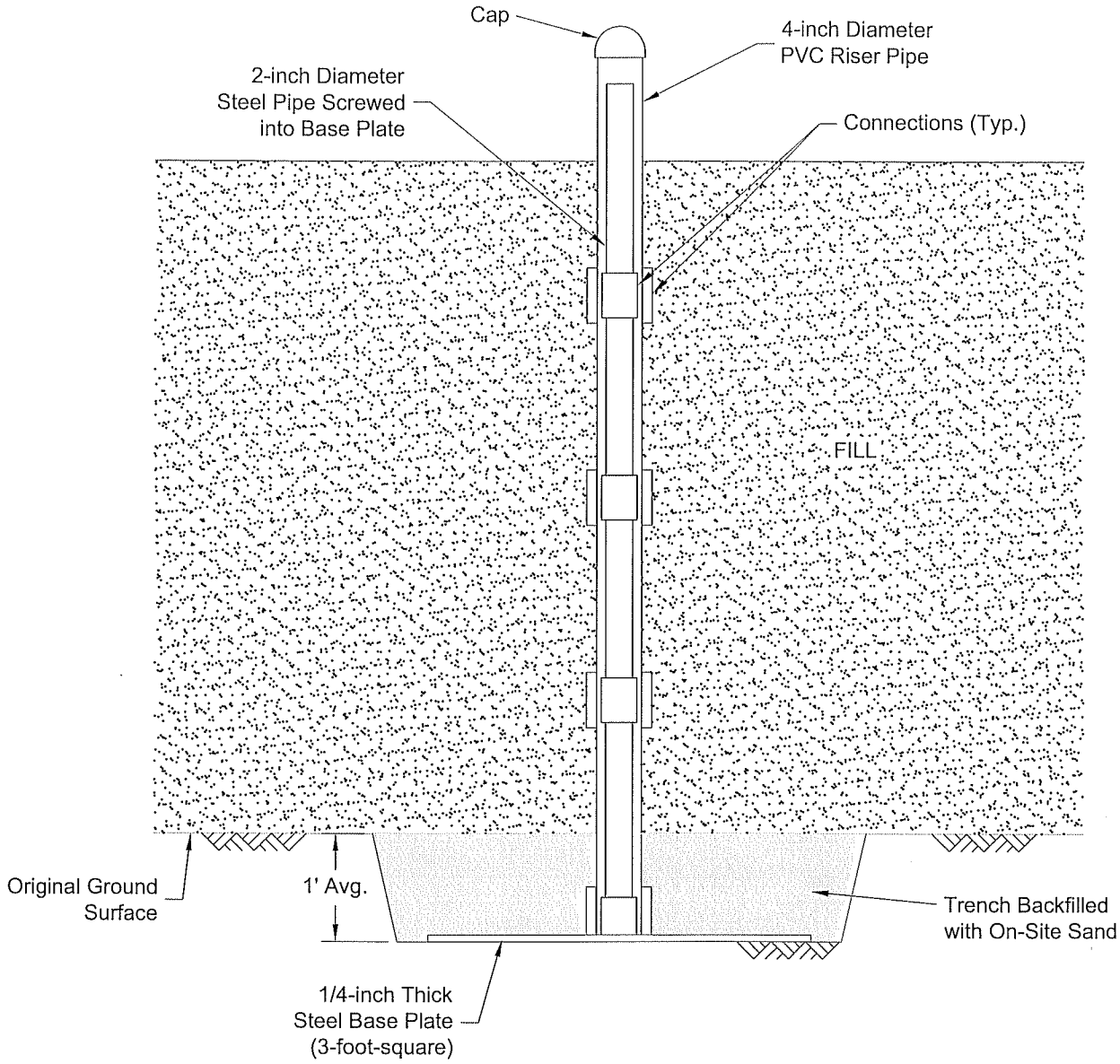
As Constructed
No Revisions:
Revised:
Void:

US 550 OVER US 160
GENERAL LAYOUT

Designer:	A. Leifheit	Structure Numbers	P-05-AG
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B3 of B37

Project No./Code
NH 1602-114
16042
Sheet Number

FIG. 1



US 550 Bridge
CDOT Project No. NH1602-114
Durango, Colorado

TYPICAL SETTLEMENT PLATE CONFIGURATION

August 2008 23-1-01165-003

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2

APPENDIX A

SUBSURFACE EXPLORATIONS BY OTHERS

APPENDIX A

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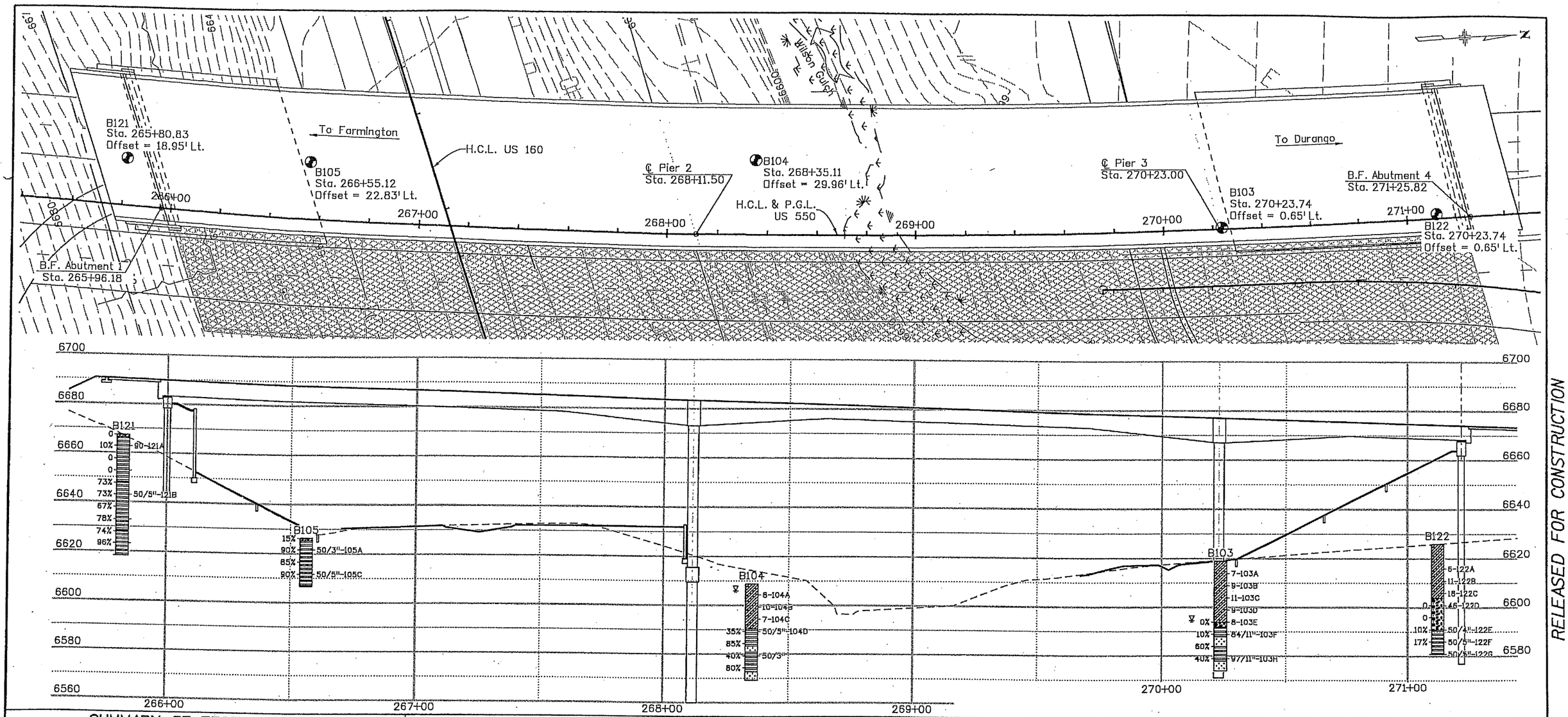
A-2 Summary Logs of Borings 2B30, 2B31, 2B33, 2B35
A-3 Log of Boring B-103
A-4 Log of Boring B-104
A-5 Log of Boring B-105
A-6 Log of Boring B-121
A-7 Log of Boring B-122

TABLE A-1
SUMMARY OF LABORATORY TEST RESULTS BY BORING

SAMPLE DATA		USCS Symbol	AASHTO Symbol	Natural Water Content	Wet Density	Dry Density	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION			STRENGTH TESTING		CORROSION TESTING		
Boring	Depth						Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	% Fines	Unconfined Compressive Strength	Strain at UCS	Water Soluble Sulfates	pH	Resistivity
				(%)	(pcf)	(pcf)	(%)	(%)	(%)				psi	(%)	%		(OHM-cm)
2B31	4.0	SC		19.2			42	20	22			37					
2P28	4.0	SC	A-2-7(3)				44	23	21			34					
B-103	4.5 - 6														0		
	15 - 16.5	CL	A-6(4)	18.3			31	20	11	2	38	60					
	44.2 - 45					156.8							11,662	1.6			
B-104	4.5 - 6					142.8									0.01		
	26.5 - 27.5	CL	A-6(10)	17.8			35	21	14	2	21	78	1,392	0.7			
B-105	10.8 - 11.8					133.4							598	1.0			
B-121	30.2 - 30.9				162.2								4,570	----			
B-122	35 - 40														0	7.2	3,500



SUMMARY OF TEST RESULTS

Sample No.	Depth	Classification: Corps of Engrs. or Visual	Classification: AASHTO	Grading Analysis (AASHTO) Percent: Gravel	Coarse Sand	Fine Sand	Silt and Clay	Atterberg Limits: Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Water Content %	Dry Unit Weight γ P.C.F.
103C	15-16.5	Sandy Clay	A-6(4)	2.1	13.9	23.7	60.3	31	20	11	18.3	
103-1	44.2-45											158.5
104A	4.5-6	Sandy Clay	A-6(10)	1.9	3.3	17.2	77.6	35	21	14	17.8	
104-2	26.5-27.5											142.8
105-2	10.8-11.6											133.4

TYPE OF MATERIAL

- Sandy Clay
- Gravel and Cobbles
- Bedrock - Shale
- Bedrock - Sandstone

Note: Boulders may be encountered at any depth

COLORADO LICENSED
KYLE ALLEN KERSHAW
38896
PROFESSIONAL ENGINEER
7/25/08

LEGEND

- B105 — Boring Designation
- 50/5" - 105C — Partial Blowcount - 50 blows per 5 inches; Sample Designation
- Groundwater Depth
- 90% — R.Q.D.
- 7 - 105C — Standard Penetration Test Blows per Foot
- Location of Boring

Quantities	INITIAL	DATE
Quantities By	KAK	08/08
Checked By	GWK	08/08

Detail	INITIAL	DATE
Detailed By	DRA	08/08
Checked By	GWK	08/08

Design	INITIAL	DATE
Designed By	KAK	08/08
Checked By	GWK	08/08

RELEASED FOR CONSTRUCTION

Print Date: 7/24/2008
File Name: 16042AG_EngGeology_01.dgn
Horiz. Scale: 1:1 Vert. Scale: As Noted
Unit Information 0221 Unit Leader STW

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions

Date:	Comments	Init.

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Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

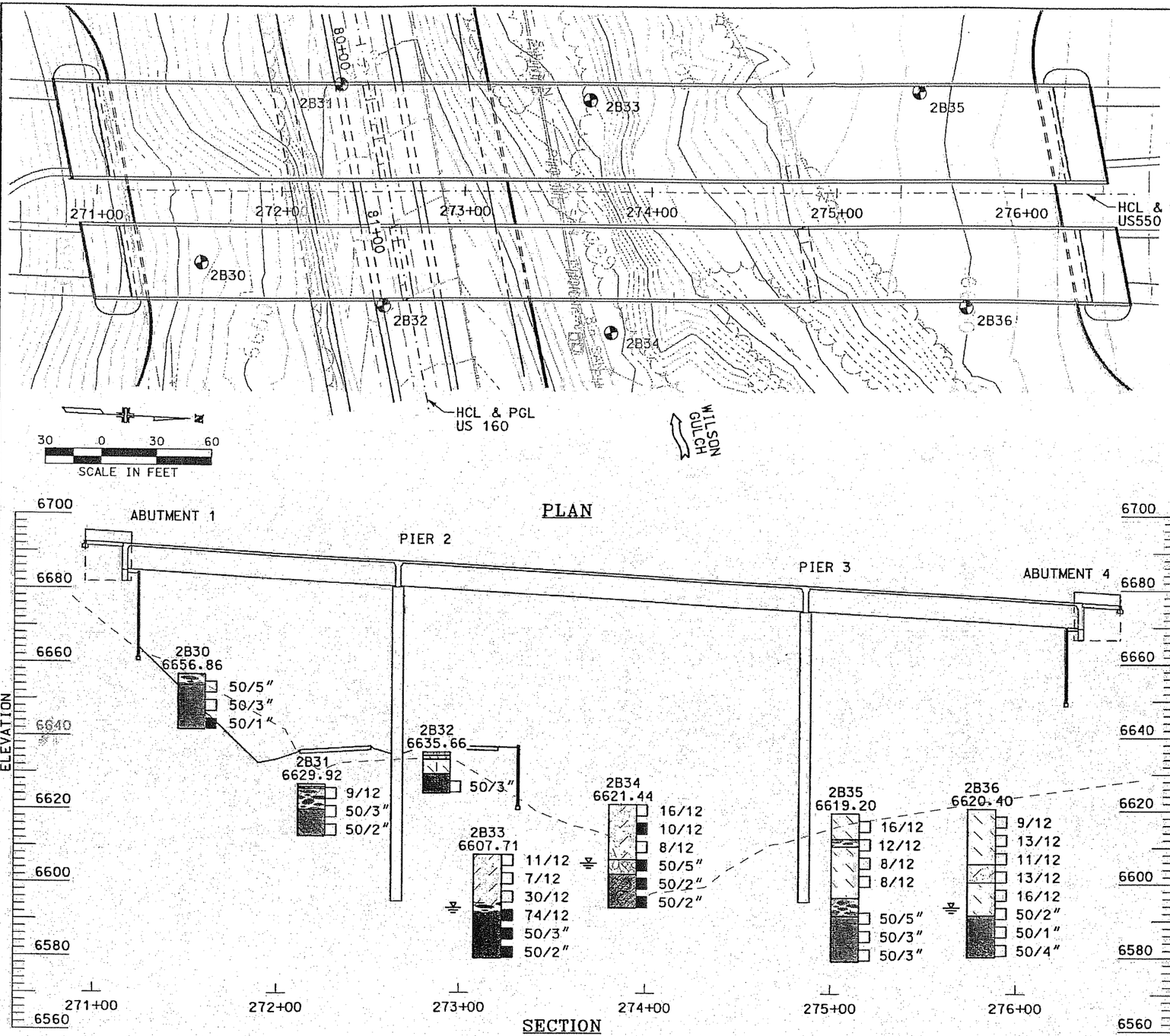
As Constructed
No Revisions:
Revised:
Void:

US 550 OVER US 160
ENGINEERING GEOLOGY

Designer:	K. Kershaw	Structure Numbers	P-05-AG
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B6 of B37

Project No./Code
NH 1602-114
16042
Sheet Number

FIG. A-1



SOIL LEGEND

- ASPHALT CONCRETE PAVEMENT
- FILL: SAND, GRAVELLY, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN TO BROWN
- CLAY, SLIGHTLY SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, TRACE GRAVEL TO SLIGHTLY GRAVELLY, MEDIUM STIFF TO VERY STIFF, LOW PLASTICITY, MOIST, GREY TO BROWN (CL)
- SAND, CLAYEY TO VERY CLAYEY, SLIGHTLY SILTY TO VERY SILTY, TRACE GRAVEL TO GRAVELLY, LOOSE TO MEDIUM DENSE, LOW TO HIGH PLASTICITY (FINES CONTENT), MOIST TO WET, LIGHT TO DARK BROWN (SC)
- GRAVEL, SANDY TO VERY SANDY, SLIGHTLY SILTY TO SILTY, CLAYEY TO VERY CLAYEY, DENSE TO VERY DENSE, MOIST TO WET, BROWN WITH VARIABLE ACCESSORY COLORS (GP, GM, GC, GP-GM, GP-GC)
- COBBLES AND BOULDERS
- CLAYSTONE, SANDY TO VERY SANDY, TRACE SILT TO SILTY, VERY HARD, DRY TO WET, GREY TO BROWN (BEDROCK)
- SILTSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SANDY TO VERY SANDY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)
- SANDSTONE, TRACE CLAY TO SLIGHTLY CLAYEY, SILTY TO VERY SILTY, VERY HARD, DRY, LIGHT BROWN TO LIGHT GREY (BEDROCK)

SYMBOL LEGEND

- 15/12 — INDICATES 15 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH O.D. SPLIT SPOON SAMPLER 12 INCHES
- 21/12 — INDICATES 21 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2-INCH I.D. CALIFORNIA SAMPLER 12 INCHES
- INDICATES GROUNDWATER OBSERVED AT THE TIME OF DRILLING
- APPROXIMATE BORING LOCATION

CONCEPTUAL PLAN
NOT FOR CONSTRUCTION
03/31/2005

Computer File Information		Index of Revisions
Creation Date:	Initials:	
Last Modification Date: SEE PLOT STAMP	Initials:	
Full Path: SEE PLOT STAMP		
Drawing File Name: SEE PLOT STAMP		
Microstation Ver.	Scale: AS SHOWN Units:	

DOT DEPARTMENT OF TRANSPORTATION Region 5

URS
10 Burnett Court Suite 113
Durango, Colorado 81301
(970) 259-9212
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As Constructed
No Revisions:
Revised:
Void:

US550 OVER US160
ENGINEERING GEOLOGY

Designer: K. YOUNG
Detailer: C. SNYDER
Structure Numbers: P-05-U
SheetSubset:
Subset Sheets:

Project No./Code
Sheet Number

FIG. A-2

FILE NAME: n:\projects\21711361_US_160\Sub_00\09.0_CAD\Planshts\Structures\P05U\P05UEG1.dgn
DATE: 04/26/2005
PLOTTED BY: cmsnyde0

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-103**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	12979	160 Fourth Lane	11/7/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	550 Bridge/North Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,618.9ft	45.0ft	N1213450.34 E2319730.26	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
				0.0		
6615						
	5.0	5 - 6.5 clay, with sand and silt, brown, dry, med. stiff		5.0 5.0	103A 4-3-4	7
6610						
	10.0	10 - 11.5 same as above		10.0 10.0	103B 2-5-4	9
6605						
	15.0	15 - 16.5 sandy clay, lt. brown, stiff		15.0 15.0	103C 5-5-6	11
	17.0	trace gravel				
6600						
	20.0	20 - 21.5 same as above, trace gravel up to 1/2", loose		20.0 20.0	103D 5-5-4	9
6595						
	24.7	3" cobble at 24.7		25.0		
	25.0	25 - 26.5 No recovery on SPT		25.0	103E 4-3-5	8 50% 0%
	25.0	25 - 27 gravel and cobbles up to 3"				
	27.0	27 bedrock, weathered shale, highly fractured, dk. grey , dry, v. hard				
6590						
	31.0	31 find grained sandstone, lt. grey to dark grey, highly fractured, joints ~40 - 60 degrees from horizontal, joint surfaces FeO stained, intact, not altered		30.0 30.0	103F 34-50/5"	84/11" 80% 10%
6585						
	35.0	35 shale, lt. grey, fissile at 35 - 35.3		35.0		100% 60%
	36.5	36.5 - 37.5 severely fractured, some joints inclined ~60 degrees w/slickensides and FeO staining				
	37.5	37.5 - 40 intact shale				
6580						
	41.0	41 sandstone, lt. grey, fine grained, w/ conglomerate at 43.5 - 44.5, joints horizontal and ~60 degrees, joint surfaces intact, no staining		40.0 40.0	103H 47-50/5"	97/11" 100% 40%
6575						

SPT DATA (5 10 20 40 70)

WELL DIAGRAM: Set piezometer

SPT | Total Boring Depth 45.0ft | CONT | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 24.4				
DATE	11/8/2007				
TIME	09:25				

NOTES: CME 550 Wireline

FIG. A-3

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-104	NH 1602-114	12979	160 Fourth Lane	11/7/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	550 Bridge/Center Pier	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,608.8ft	39.5ft	N1213264.07 E2319730.95	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
				0.0		
6605						
	4.5	4.5 - 6 sandy clay, lt. brown, trace roots, dry, med. stiff		4.5 4.5	104A 3-4-4	8
6600						
	9.5	9.5 - 11 same as above, trace gravel, stiff		9.5 9.5	104B 4-5-5	10
6595						
	14.5	14.5 - 16 same as above, shale fragments up to 1/4"		14.5 14.5	104C 5-4-3	7
6590	18.5	18.5 shale bedrock, v. slow drilling, fractured, dk. grey				
	19.5	19.5 - 21 shale bedrock, highly fractured, dry, dk. grey to brown, v. hard		19.5 19.5	104D 50/5"	50/5" 100% 35%
6585	23.0	joints horizontal to ~45 degrees, w/ FeO staining, joint surfaces hard		24.5		100% 85%
	25.5	25.5 sandstone, grey, v. fine grained, joints horizontal w/ some ~60 degrees from horizontal, joint surfaces fresh				
6580	28.0	28 shale, intact, joint surfaces fresh with no weathering				
	29.5	29.5 - 31 shale, grey, dry, v. hard		29.5 29.5	50/3"	50/3" 100% 40%
	32.0	32 - 33 very fractured, joint at 32 ~60 degrees, slickensides, no gouge, joint surfaces unaltered				
6575	34.7	34.7 joint w/ slickensides, joint surfaces unaltered		34.5		100% 80%
	36.0 36.2	36 joint w/ slickensides, joint surfaces unaltered				
6570		36.2 sandstone, lt. grey, v. fine grained				
	39.5	Total Boring Depth 39.5ft				
6565						

SPT DATA (N-VALUE): 5 10 20 40 70

WELL DIAGRAM: Piezometer installed

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH	▽ 2.9				
DATE	11/8/2007				
TIME	09:15				

NOTES: CME 550 Wireline

FIG. A-4

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-105	NH 1602-114	12979	160 Fourth Lane	11/8/07

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	550 Bridge/South Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,626.3ft	19.6ft	N1213085.42 E2319682.15	Laudeman/Blailes

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6625	1.0	0 - 1 sandy clay, grey, dry		0.0		50% / 15%
		1 bedrock, shale, severely fractured, dk. grey, w/ FeO staining				
	3.7	3.7 lt. grey, joints ~40 - 60 degrees from vertical w/ FeO staining		4.7		
	4.7	4.7 - 6.2 shale as above		4.7	105A 50/3"	50/3" / 100% / 90%
6620						
	9.0	9 sandstone, lt. grey, v. fined grained, vertical joint w/ FeO staining, ~1/8" soft filling		9.7		100% / 85%
	10.0	10 shale, lt. grey, competent				
6615						
	14.2	14.2 sandstone, lt. grey, fine grained, higly jointed, joint surfaces stained w/ weak filling ~1/2" thick		14.7	105C 50/5"	50/5" / 100% / 90%
	15.0	15 shale, lt. grey, competent, v. slow drilling		14.7		
6610						
	19.2	19.2 - 19.4 joint, inclined ~60 degrees from vertical, w/ slickensides, no weathering or alteration				
	19.7	Total Boring Depth 19.6ft				
6605						
6600						
6595						
6590						
6585						

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Backfilled with cuttings

SPT	CON'T	GRAB	SHELBY	CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550 Wireline

FIG. A-5

GEOLOGIC BORING LOG 160 DURANGO.GPJ CO_DOT.GDT 1/9/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B121**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	160 Fourth Lane	6/5/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	550 Bridge/South Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,656.0ft	49.0ft		I. Ksouri/D. Novak

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6655		Shale bedrock, highly fractured, very hard, tan with iron stains. At 20 feet and below, high Recovery and RQD, blue-gray color.		0.0		27% / 0%
				2.0	A	90
				5.0		100% / 10%
6650						
				10.0		33% / 0%
6645						
				15.0		75% / 0%
6640						
				20.0	B	93% / 73%
6635				20.1		50/5"
				25.0		100% / 73%
6630						
				30.0		90% / 67%
6625						
				35.0		100% / 78%
6620						
				40.0		100% / 74%
6615						
				44.5		100% / 96%
6610						
	49.0	Total Boring Depth 49.0ft				
6605						

SPT DATA: 5 10 20 40 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: Elevation Approximated

DRAFT

FIG. A-6

GEOLOGIC BORING LOG US160-4THLANE.GPJ CO_DOT.GDT 6/13/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B122**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	160 Fourth Lane	6/4/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	550 Bridge/North Abut	Durango

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
6,626.0ft	45.0ft		I. Ksouri/D. Novak

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
6625		Silty to sandy clay, medium stiff to very stiff, low plasticity, brown	0.0		28% / 0%
6620					
6615			10.0 / 10.1	A	25% / 0% / 6
6610			15.0 / 15.1	B	50% / 0% / 11
6605			20.0 / 20.1	C	33% / 0% / 18
	22.0	Sand and gravel with boulders and cobbles encounter, some silt and clay, dense to very dense, brown			
6600			25.0 / 25.1	D	33% / 0% / 46
6595			30.0		33% / 0%
	35.0	Shale bedrock, highly fractured, very hard, tan with iron stains	35.0 / 35.1	E	55% / 10% / 50/4"
6590					
6585			40.0 / 40.1	F	92% / 17% / 50/5"
	45.0	Total Boring Depth 45.0ft	45.1	G	50/5"
6580					

SPT DATA: 5 10 20 40 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES:

DRAFT

FIG. A-7

GEOLOGIC BORING LOG US160-4THLANE.GPJ CO_DOT.GDT 6/13/08

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

SEATTLE
RICHLAND
PORTLAND
FAIRBANKS
ANCHORAGE
DENVER
SAINT LOUIS
SACRAMENTO
JACKSONVILLE

July 25, 2008

Wilson & Company
999 18th Street, Suite 2600
Denver, CO 80202

Attn: Mr. Andy Leifheit

RE: 100% SUBMITTAL, GEOTECHNICAL DESIGN LETTER 002, RAMP D BRIDGE, CDOT PROJECT NO. NH1602-114, DURANGO, COLORADO

This letter provides geotechnical design recommendations for the proposed Ramp D Bridge. This letter supersedes previous submittals for this bridge.

STRUCTURE DESCRIPTION

The proposed 2-span bridge over Wilson Gulch is designated as Ramp D Bridge, CDOT Structure No. P-05-Y (Figure 1). The bridge will trend approximately northwest-southeast and be about 28 feet wide. The northwest abutment is designated as Abutment 1 (Sta. 610+85.25) and the southeast abutment is designated as Abutment 3 (Sta. 613+11.75). One interior pier, designated Pier 2 (Sta. 612+08) is planned to support the bridge. The abutments will be supported on driven piles and the pier on drilled shafts.

To achieve the proposed roadway grade elevations at the bridge, fill material will be placed at both abutments. At Abutments 1 and 3, fill heights will approach 17 feet. Fill embankments at both abutments will be sloped at 2 Horizontal to 1 Vertical (2H:1V) beneath the bridge and will gradually fan out to 4H:1V on the side of the embankment opposite the bridge abutment. Slope paving will be utilized on the 2H:1V slopes beneath the bridge.

23-1-01165-003
LR2

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SUBSURFACE CONDITIONS

Our interpretation of subsurface conditions at the bridge site is based on borings drilled by CDOT and others, as well as cone penetrometer tests (CPTs) completed by ConeTec under subcontract to Shannon & Wilson. The approximate location of the borings and the logs for the borings and CPTs are provided in Appendix A. A summary of laboratory test results are presented in Table A-1.

The subsurface profile beneath Ramp D Bridge was interpreted based on the following borings: 2B48, 2B51, YA-21, YA-22, B-123, B-124, B-128. In addition, we utilized information from CPT-4, CPT-5, CPT-6, CPT-7, CPT-8, and CPT-9 to further refine the subsurface profile and assist in determining soil strength parameters. At Abutment 1 and Pier 2, the subsurface conditions generally consisted of interbedded medium dense, clayey sand and medium stiff to very stiff, sandy clay overlying shale and sandstone bedrock. At Abutment 3, the overburden soils were slightly different, with medium dense, clayey sand and very dense sandy gravel and cobbles encountered from the existing ground surface to the top of shale and sandstone bedrock. The boring logs describe the bedrock as very weathered to fresh, medium hard material, with generally horizontal bedding. The top of bedrock in the above-referenced borings, ranged from 6,598 to 6,633 feet, with the upper end of this elevation range encountered near Pier 2 and Abutment 3. Groundwater was only encountered in borings and CPTs drilled adjacent to Abutment 1, at an elevation of roughly 6,630 feet.

Groundwater levels were recorded in each boring at the time of drilling. We assumed that the upper range of these measurements is representative of long-term conditions. The groundwater elevations used in our analyses are shown in Table 1.

SEISMICITY

Seismic Hazards

Liquefaction, lateral spreading, and dynamic settlement are factors that can negatively affect the stability and load carrying capacity of foundations and structures. The peak ground

acceleration for a 475-year event is 0.035g, per the United States Geological Survey (USGS) (USGS, 2002). Per the American Association of State Highway and Transportation Officals (AASHTO) LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim) and USGS (USGS, 2002), the peak ground acceleration for a 1000-year event is 0.06g. In our opinion, because the peak horizontal ground acceleration (PGA) for this area of Colorado is low, there is a low risk potential associated with these seismic hazards, and such seismic hazards were not considered as part of design of the structure.

Seismic Site Classification

We understand that seismic design of the bridge will be in accordance AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). Based on data collected from the CPTs, the soil conditions at the site are consistent with Site Class C.

GEOTECHNICAL RECOMMENDATIONS

General

We understand that driven piles and drilled shafts are the preferred foundation support for the abutments and piers, respectively. The design criteria presented herein for driven piles and drilled shafts uses load and resistance factor design (LRFD). The LRFD criteria were developed based on AASHTO LRFD Bridge Design Specifications (AASHTO, 2007 with 2008 Interim). In addition, Federal Highway Administration (FHWA) design methods (O'Neil and Reese, 1999) and a CDOT research report (Abu-Hejleh et al., 2003) were used to supplement drilled shaft design procedures.

Bridge Foundations

Driven Piles

Driven piles for Abutments 1 and 3 will extend through overburden soils and develop capacity in the shale and sandstone bedrock. In our opinion, piles that are driven to practical refusal in the bedrock can be designed for the structural capacity of the steel member (i.e., the

geotechnical capacity will not control), provided that a wave equation analysis is performed to ensure that the hammer-cushion-pile system is capable of achieving the necessary capacity without damaging the pile. A Resistance Factor (ϕ) of 0.5 is recommended for the structural limit state of the pile.

A minimum spacing of 30 inches or 2.5 diameters (center-to-center) for axial loads is recommended. This will reduce the stress overlap between the adjacent piles.

For properly designed and installed piles, we estimate settlement to be 1/2 inch. Differential settlement between adjacent piles would be expected to be about 50 to 75 percent of this value. A majority of the settlement is anticipated to occur as the load is applied with the remaining settlement occurring within the first year following construction.

A wave equation analysis must be conducted to identify a hammer that will not overstress the piles. Use of higher strength steel (50 kips per square inch [ksi] instead of 36 ksi) may be desirable to allow for higher driving stresses. The maximum stress during pile driving shall not exceed 90% of the pile steel yield strength. A Resistance Factor of 1.0 is recommended for driving stress.

Drilled Shafts

Axial design parameters for use in sizing the drilled shafts at Pier 2, as well as, Resistance Factors (ϕ) for both tip and side resistance, are presented in Table 1. The nominal axial capacities in the bedrock are based on Standard Penetration Test (SPT) blow counts, typical to local practice. For drilled shafts that obtain the majority of their axial capacity in rock, no reduction in nominal axial capacity is needed for center-to-center spacings greater than two shaft diameters.

Total settlement at the service design state of individual drilled shafts designed using the parameters discussed above and constructed using good installation techniques and equipment, as described in the 1999 FHWA manual, "Drilled Shafts: Construction Procedures

and Design Methods," is anticipated to be about ½ inch. Differential settlement between adjacent piers would be expected to be about 50 to 75 percent of this value. A majority of the settlement is anticipated to occur as the load is applied with the remaining settlement occurring within the first year following construction.

Downdrag Loads

As stated previously, fill material will be placed at the abutment locations. The weight of the embankment fill material will induce settlement of the underlying soils, and subsequently, add downdrag loads to the driven piles at the abutments. At Abutments 1 and 3, we estimate downdrag loads of 115 and 120 kips per pile, respectively (based on a 14-inch H-pile). However, if the piles at Abutment 3 are installed after the new fill is placed, the downdrag load would not need to be considered (settlement of the underlying granular soils would occur during or shortly after construction of the fill embankment). We recommend waiting a period of approximately 2 weeks between placement of the fill and pile installation. Downdrag loads should be considered a structural load in accordance with the AASHTO LRFD Manual.

The estimated downdrag loads assume that a casing sleeve will be used to isolate the piles within the embankment fill such that the downward movement of the embankment is not transferred to the shaft as a downdrag load. We recommend using thin-walled steel or polyvinyl chloride (PVC) pipe, sized appropriately to resist lateral earth pressures from the embankment material. Further, we recommend a 24-inch inside diameter (I.D) smooth pipe for the HP14x89 piles.

Pea gravel should be placed in the annulus such that the downdrag loading from the embankment soil is not transferred to the pile. The following table summarizes our recommended gradation specifications for the pea gravel. If the piles at Abutment 3 are installed after the embankment is constructed, it is our opinion that the pipe is not needed within the embankment fill.

GRADATION SPECIFICATIONS FOR PEA GRAVEL

U.S. STANDARD SIEVE SIZE	MASS PERCENT PASSING BY WEIGHT
9.5 mm (3/8-inch)	100
4.75 mm (No. 4)	0 – 15
75 μm (No. 200)	0 – 1.0

NOTES:
μm = micrometer
mm = millimeter

Lateral Load Capacity

We understand that lateral load capacity will be evaluated using the computer program LPILE^{PLUS} by Ensoft, Inc. LRFD design methods for lateral loading of driven piles or drilled shafts should utilize a Resistance Factor of 1.0 and are controlled by allowable movement criteria. Input parameters for an individual pile and shaft are given in Table 1. Group action can be analyzed using P-multipliers within LPILE^{PLUS}. P-multipliers for loading perpendicular to a line of piles/shafts are presented in Table 1, Note 12. P-multipliers for loading parallel to a line of piles/shafts are presented in Table 2. As an alternative, the computer software Group 6.0 (Reese and Wang, 2003) can also be used to analyze group action. Because it may be difficult to get large penetration into the bedrock with the driven piles, we recommend that lateral analyses be completed assuming a minimal embedment depth.

Abutment Embankments

Based on discussions with Wilson & Co., the embankments will be constructed with material excavated from the construction of Walls P-05-F and P-05-K. We understand that the predominant material that will be excavated from these areas will be a shale-derived soil. For our analysis, we assumed that this material will have a unit weight of 130 pounds per cubic foot (pcf), cohesion of 300 psf, and friction angle of 25 degrees. We analyzed the stability of the native soil/rock under the imposed embankment loading using PCSTABL slope stability

software developed by Purdue University and the Indiana Department of Highways, along with the STEDwin2.90 pre- and post-processor developed by Annapolis Engineering Software.

Three cases were analyzed for the native materials: short-term (undrained), long-term (drained), and seismic. For the seismic case, a horizontal acceleration coefficient of 0.03 g was used, based on a peak horizontal ground acceleration of 0.06g for a 1000-year event.

At Abutment 1, the native materials have a factor of safety of 1.1 for short-term loading conditions if the entire embankment is constructed at one time. Because this factor of safety is low, we recommend installing inclinometers to monitor movement and piezometers to monitor pore pressures within and below the embankment. We further recommend that the embankment initially be constructed only up to the base of the abutment pilecap prior to the bridge construction, which will reduce the embankment height by roughly 6 feet. If the instrumentation indicates negligible movement and pore pressure dissipation during the construction of the bridge, then fill may be placed to the full embankment height. The native materials at Abutment 1 have a factor of safety greater than 1.3 for long-term loading and a factor of safety greater than 1.1 for seismic loading.

At Abutment 3, the native soils have a factor of safety of 1.2 for short-term loading, a factor of safety of 1.5 for a long-term loading, and a factor of safety greater than 1.1 for seismic loading.

The settlement potential of the fill embankments at each abutment was evaluated. At Abutment 1, the native materials consisted of about 8 feet of clayey sand; overlying 47 feet of sandy clay; overlying shale and sandstone bedrock. We estimate that total settlement of these native materials will be on the order of 18 inches. Based on our interpretation of Borings 2B48 and YA-22, as well as data from CPT-4, CPT-5, and CPT-6, groundwater is located at a depth of 23 feet from the existing ground surface. We estimate that the settlement of the clayey sand will be on the order of 1-inch, and we anticipate this settlement will occur during or shortly after construction. The majority of settlement for the sandy clay located above the water table (15-foot zone) should occur within 1 year after the embankment is constructed. Settlement of this 15-foot zone is on the order of 4 inches. Settlement of the sandy clay below the water table

(32-foot zone), in our opinion, will be time-dependent due to the dissipation of excess pore pressures in the clay generated during loading. Settlement of this 32-foot zone is on the order of 13 inches and we estimate that primary consolidation may take up to about 2.5 years. We anticipate an additional 1-1/2 inches of settlement within the next 10 years following primary consolidation, due to secondary compression of the clay.

At Abutment 3, the native materials consisted of about 20 feet of clayey sand; overlying 5 feet of sandy gravel with cobbles; overlying shale and sandstone bedrock. We estimate that settlement of these native materials will be one the order of 3.5 inches. We anticipate that the settlement will occur during or shortly after construction and will likely be complete within about 2 weeks following construction.

In addition to the estimated settlement of the native materials, we estimate that the embankment itself will undergo some settlement during or shortly after construction. For properly placed and compacted fill, settlement of embankment fill material should be less than 1/2 percent of the total embankment height. This settlement is anticipated to occur within the first year following construction.

Because of the relatively large, time-dependent settlement anticipated at Abutment 1, we recommend that instrumentation be installed to monitor settlement with time. Typically, settlement plates are used to monitor embankment settlement. The base plate should be installed at the current ground surface before placement of the fill and the connection pipe should extend vertically through the fill to the top of the constructed embankment. The elevation of the top of pipe should be surveyed periodically and plotted versus time to evaluate the rate of consolidation. As the rate of settlement decreases, the degree of consolidation increases. A typical settlement plate configuration is shown in Figure 3.

Sulfates and Corrosion

We reviewed corrosion test results conducted by others to estimate the corrosion potential of the soil/rock. One bulk sample of sandy clay at a depth of 0 to 5 feet below the existing ground surface in Boring YA-22 was tested for water soluble sulfates. In addition, we completed one

additional corrosion test on a sample taken in the sandstone bedrock at a depth of approximately 50 to 55 feet below the existing ground surface in Boring YA-22. The test results are summarized in Table A-1.

Based on classifications as defined by the American Concrete Institute (ACI) Manual ACI 318-05 (ACI, 2005), these test results suggest a negligible sulfate exposure to concrete in contact with these materials.

CONSTRUCTION CONSIDERATIONS

General

The applicability of the design recommendations provided in this letter is contingent on good construction practice. Poor construction techniques may alter conditions from those on which our recommendations are based, therefore, resulting in reduced foundation capacity and increased settlement. The following sections present additional construction and material considerations for this project.

Driven Pile Installation

A wave equation analysis must be completed using the selected pile type and the contractor's pile driving equipment. Information required for such an analysis is presented in Figure 2. The wave equation analysis must be completed prior to the start of driving production piles at the site. We can assist with such an analysis, if desired.

In accordance with the project contract documents, dynamic monitoring of driven piles must be performed in accordance with ASTM International [ASTM] D-4945 on a minimum of one pile at each abutment and a minimum of 2 percent of the total bridge piles. The dynamic monitoring shall include collecting stress wave measurements at the end-of-initial-driving (EOID) and beginning of re-strike (BOR) for each monitored pile. The waiting period between EOID and BOR shall be a minimum of 12 hours.

Fixed-lead pile driving equipment is recommended for driving the piles. The use of hanging or swinging leads is not recommended unless they are constructed to be held in a fixed position during driving operations.

Pile installation may encounter hard driving within the shale and sandstone bedrock and in the gravel and cobbles anticipated overlying bedrock at Abutment 3. We recommend that a protective rock tip be welded to the end of each pile to reduce the possibility of pile damage during driving. Even with point reinforcements, there is a possibility that the piles could encounter refusal before reaching the required depth. The top of rock elevation as shown in Table 1 should be considered the minimum tip elevation at each abutment.

Pile driving should be monitored and a continuous driving record obtained for each pile. For this purpose, the pile driving contractor should paint a line across the pile in 1-foot increments for the full pile length prior to driving. A system should be established so that the measurements can be determined to the nearest inch during final driving. It has often been difficult in the past to estimate the energy delivered by diesel hammers. The Saximeter, developed by Pile Dynamics, Inc., can be used to record hammer strokes and provide an estimate of the driving energy of diesel hammers.

Drilled Shaft Installation

Based on conditions observed at the project site, groundwater inflow may be anticipated above the shale and sandstone bedrock. Depending on the type of drilling techniques employed and the speed of installation, casing likely may be required to seal off local groundwater inflow. Casing should be pushed, rotated, vibrated or driven into an impermeable, firm stratum below the seepage zone. Groundwater inflow must be sealed off prior to penetration into the shale and sandstone bedrock and the use of slurry within the rock socket zone should not be allowed. Installation techniques should be compliant with the 1999 FHWA manual, "Drilled Shafts: Construction Procedures and Design Methods."

Upon completion of the shaft excavation, the hole should be cleaned of loose material and observed by the engineer prior to pouring concrete. The drilling and concreting process should

be relatively continuous with minimal stoppage of work between the completion of drilling and cleaning the hole and placement of concrete after setting the rebar cage, and under no circumstances after the day of drilling.

Frictional and end-bearing design parameters used in this analysis assume that the bottom of the shaft is properly cleaned and proper concrete placement techniques are used. To provide sufficient frictional resistance in rock, drillers should not be allowed to use drilling slurry or add water during drilling or to leave any casing in the rock portion of the drilled shaft after drilling. The specifications should require that the driller make a final pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7 inches from the sides of the auger to roughen the rock socket. The primary purpose of the roughening is to reduce the polished skin of the remolded material that can sometimes form in softer bedrock.

The blowcount values provided in the boring logs indicate that the shale and sandstone bedrock is a hard material which may create hard drilling conditions. Therefore, we recommend that the strength properties of the bedrock be disclosed to drilling subcontractors.

Embankment and Subgrade Recommendations

Prior to placing fill, the subgrade soils should be proof-rolled with a large, heavy, vibratory roller or equivalent. Any loose, soft, or yielding areas should be overexcavated and replaced to the design base of wall elevation with Structure Backfill (Class 1) that is compacted to a minimum of 95 percent of its Modified Proctor dry density (AASHTO T-180).

Excavation for embankment, embankment foundation preparation, and embankment construction should conform to the requirements of the latest edition of the CDOT Standard Specifications for Road and Bridge Construction. Embankment material should be compacted to a minimum of 95 percent of its Modified Proctor (AASHTO T-180) dry density.

CLOSURE

The analyses, conclusions, and recommendations presented in this letter are based on a limited number of subsurface explorations and laboratory test results conducted by others. We assume that these explorations are representative of the subsurface conditions beneath the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

This letter was prepared for the exclusive use of Wilson & Company and SEMA for use in design of the Ramp D Bridge. It should not be construed as a warranty of subsurface conditions.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this letter were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this letter was prepared. We make no other warranty, either express or implied.

Sincerely,

SHANNON & WILSON, INC.

Michael C. Hallman, P.E.
Geotechnical Engineer

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PROFESSIONAL ENGINEER

7/25/08

Kyle A. Kershaw, P.E.
Principal Engineer

MCH:KAK:GRF/mch

Enclosures: Table 1– Geotechnical Parameters for Design of Bridge Foundations at Ramp D Bridge
Table 2 – LPILE^{PLUS} Efficiency Factors for Loading Acting Parallel to the Line of Shafts
Figure 1 – Ramp D over Wilson Gulch General Layout Sheet
Figure 2 – Pile and Driving Equipment Data
Figure 3 – Typical Settlement Plate Configuration
Appendix A – Subsurface Explorations

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TABLE 1
GEOTECHNICAL PARAMETERS FOR DESIGN OF BRIDGE FOUNDATIONS AT RAMP D BRIDGE

Structure	Location	Applicable Borings	Layer Elevation		Soil / Rock Type	Effective Unit Weight	Friction Angle[1]	Undrained Shear Strength	Drilled Shafts[3,4]						Driven Piles[7]		LPILE[PLUS] Ver. 4.0 Parameters for Lateral Analyses[1,8,9,10,11,12]			
											Resistance Factors[2]		Downdrag[5,6]		Downdrag[5,6]					
			Top	Bottom					Nominal Unit Skin Friction	Nominal Unit End Bearing	Skin Friction	End Bearing	Load per Shaft	Maximum Depth of Downdrag	Load per Pile	Maximum Depth of Downdrag	Soil Model Type	Strain at 50% Yielding	Initial Static Modulus of Subgrade Reaction	Initial Cyclic Modulus of Subgrade Reaction
			(feet)	(feet)		γ' (pcf)	ϕ' (deg)	s_u (psf)	(ksf)	(ksf)			(kips)	(ft)	(kips)	(ft)		ε_{50}	k_s (pci)	k_c (pci)
Ramp D Bridge	Abutment 1 (Northwest)	B-123, CPT-4, CPT-5, CPT-6, YA-22	6,673	6,653	New Fill	130	34	--	Not Applicable						115	6,598	Sand	--	90	90
			6,653	6,645	Med dense, clayey SAND	120	36	--									Sand	--	90	90
			6,645	6,630	Medium stiff to stiff, sandy CLAY	115		1,500									Stiff Clay w/o Free Water	0.007	500	200
			6,630	6,610	Medium stiff, sandy CLAY	53		800									Soft Clay	0.009	300	120
			6,610	6,598	Stiff to very stiff, sandy CLAY	53	--	1,500									Stiff Clay w/o Free Water	0.007	500	200
			6,598	6,593	Weakened Shale/Sandstone Bedrock	83		8,000									Stiff Clay w/o Free Water	0.005	1000	400
			6,593		Shale/Sandstone Bedrock	83	--	15,000									Stiff Clay w/o Free Water	0.004	2000	800
	Pier 2	B-128	6,629	6,627	Medium stiff to stiff, sandy, silty CLAY	58	--	1,000	0.8	--	0.45	--	No Downdrag		Not Applicable		Stiff Clay w/o Free Water	0.009	300	120
			6,627	6,622	Weakened Shale/Sandstone Bedrock	83		8,000	5	60	0.75	0.75					Stiff Clay w/o Free Water	0.005	1000	400
			6,622		Shale/Sandstone Bedrock	83	--	15,000	9	110	0.75	0.75					Stiff Clay w/o Free Water	0.004	2000	800
	Abutment 3 (Southeast)	B-124, CPT-7, CPT-8, CPT-9, YA-21	6,676	6,658	New Fill	130	34	--	Not Applicable						120	6,633	Sand	--	90	90
			6,658	6,638	Med dense, clayey SAND	120	36	--									Sand	--	90	90
			6,638	6,633	Med dense to very dense sandy GRAVEL/COBBLES	125	38	--									Sand	--	175	175
			6,633	6,629	Weakened Shale/Sandstone Bedrock	83		8,000									Stiff Clay w/o Free Water	0.005	1000	400
			6,629		Shale/Sandstone Bedrock	83	--	15,000									Stiff Clay w/o Free Water	0.004	2000	800

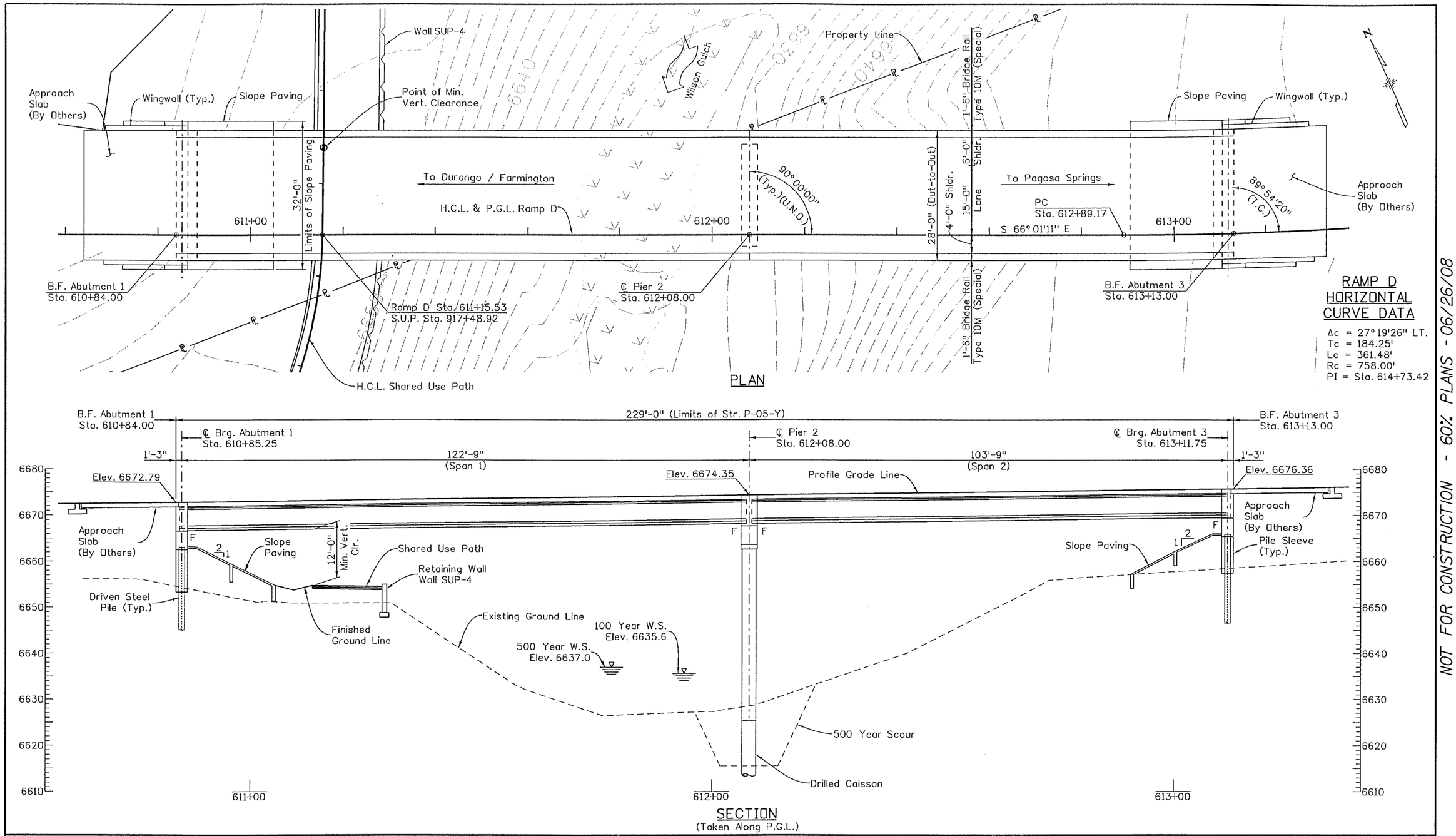
Notes:

1) Friction angle of 34 degrees used for new fill material is an equivalent friction angle for using the "Sand" model in LPILE[PLUS]. Actual assumed fill material properties are cohesion of 300 psf and friction angle of 25 degrees.
2) If a non-redundant, single shaft is used to support a bridge pier, the resistance factors should be reduced by 20 percent. For two closely spaced shafts, redundancy should be evaluated by the structural engineer. A resistance factor of 1.0 should be used for the service and extreme limit states. For uplift resistance, a resistance factor of 0.8 should be used for the extreme limit state
3) Minimum bedrock penetration for all drilled shafts should be 3 diameters.
4) For drilled shafts that obtain the majority of the their axial capacity in rock, no reduction in axial capacity is needed for center-to-center spacings of greater than 2 diameters.
5) Calculation of downdrag load assumes a 14-inch H-pile with no downdrag contribution from the new fill material.
6) The downdrag should be treated as a load with the appropriate load factors applied. At Abutment 3, if the piles are driven at least 2 weeks after the new fill material is placed, the downdrag load would be negligible.
7) Piles should be driven to refusal in rock and designed for the structural capacity of the pile. We anticipate refusal will occur at 1 to 5 feet of bedrock penetration. A resistance factor of 1.0 should be used for driving stresses.
8) The lateral loading analysis should consider the potential for flooding and scour.
9) The LPILE[PLUS] parameters shown are for horizontal ground surface. Sloping ground surface modifications should be included as per Ensoft, Inc.'s recommendations for the LPILE[PLUS] program.
10) The lateral resistance values provided assume that the lateral loads are short-term (non-sustained) loads. If sustained lateral loads are anticipated, we should be notified to provide additional parameters.
11) A resistance factor of 1.0 should be used for lateral analyses.
12) For tangent shafts/piles where lateral loading is perpendicular to line of shafts/piles, a P-multiplier of 0.5 should be used; for shaft/pile spacing greater than or equal to 3 diameters, center-to-center, no P-multiplier is required. For spacings between 1 and 3 diameters, the P-multiplier can be interpolated between 0.5 and 1.0, respectively. For lateral loading parallel to the line of piles/shafts refer to Table 2.

TABLE 2 - LPILE[PLUS] EFFICIENCY FACTORS FOR LOADING ACTING PARALLEL TO THE LINE OF PILES/SHAFTS

Spacing	P-multiplier
2.0 D	0.28
2.5 D	0.50
3.0 D	0.72
3.5 D	0.85
4.0 D	0.91
4.5 D	0.94
5.0 D	0.98
5.5 D	1.00

NOTE: The p-multipliers are based on recommendations presented in a 1998 Ensoft Seminar (Ensoft, 1998)



NOT FOR CONSTRUCTION - 60% PLANS - 06/26/08

Design			Detail			Quantities		
	INITIAL	DATE		INITIAL	DATE		INITIAL	DATE
Designed By	XXX	MM/YY	Detailed By	XXX	MM/YY	Quantities By	XXX	MM/YY
Checked By	XXX	MM/YY	Checked By	XXX	MM/YY	Checked By	XXX	MM/YY

Print Date: 6/25/2008
File Name: 16042Y_GenLayout_01.dgn
Horiz. Scale: 1:20 Vert. Scale: As Noted
Unit Information Unit Leader Initials

SEMA CONSTRUCTION
WILSON & COMPANY

Sheet Revisions		
Date:	Comments	Init.

Colorado Department of Transportation
DOT DEPARTMENT OF TRANSPORTATION
3803 North Main Avenue
Suite 200
Durango, CO 81301
Phone: 970-385-1440 FAX: 970-385-8365
Region 5 EJA

As Constructed
No Revisions:
Revised:
Void:

RAMP D OVER WILSON GULCH
GENERAL LAYOUT

Designer:	B. Allen	Structure Numbers	P-05-Y
Detailer:	D. Anderson		
Sheet Subset:	Bridge	Subset Sheets:	B3 of XX

Project No./Code
NH 1602-114
16042
Sheet Number

FIG. 1

Contract No.: _____ Structure Name and/or No.: _____

Project: _____

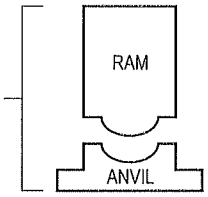
_____ Pile Driving Contractor or Subcontractor: _____

County: _____ _____
(piles driven by)

HAMMER

Manufacturer: _____ Model: _____
Type: _____ Serial No.: _____
Rated Energy: _____ @ _____ Length of Stroke
Explosive Force: _____
(for diesel hammers)

HAMMER COMPONENTS

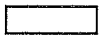


RAM

Ram Weight: _____ Ram Length: _____
Ram Cross Sectional Area: _____ (for diesel hammers)

ANVIL

(with diesel hammers) Anvil weight: _____

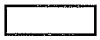


CAPBLOCK

Material: _____ Area: _____
Thickness: _____
Modulus of Elasticity - E: _____ (psi)
Coefficient of Restitution - *e*: _____

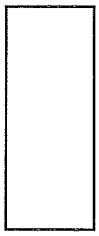
PILE CAP

Helmet
Bonnet
Anvil Block
Drivehead
Weight: _____



CUSHION

Cushion Material: _____ Area: _____
Thickness: _____
Modulus of Elasticity - E: _____ (psi)
Coefficient of Restitution - *e*: _____



PILE

Type: _____
Pile Size: Length (in leads): _____
Diameter: _____
Wall Thickness: _____ Taper: _____
Material: _____ Weight/Ft.: _____
Design Pile Capacity: _____ (tons)
Description of Splice: _____
_____
Tip Treatment Description: _____
_____

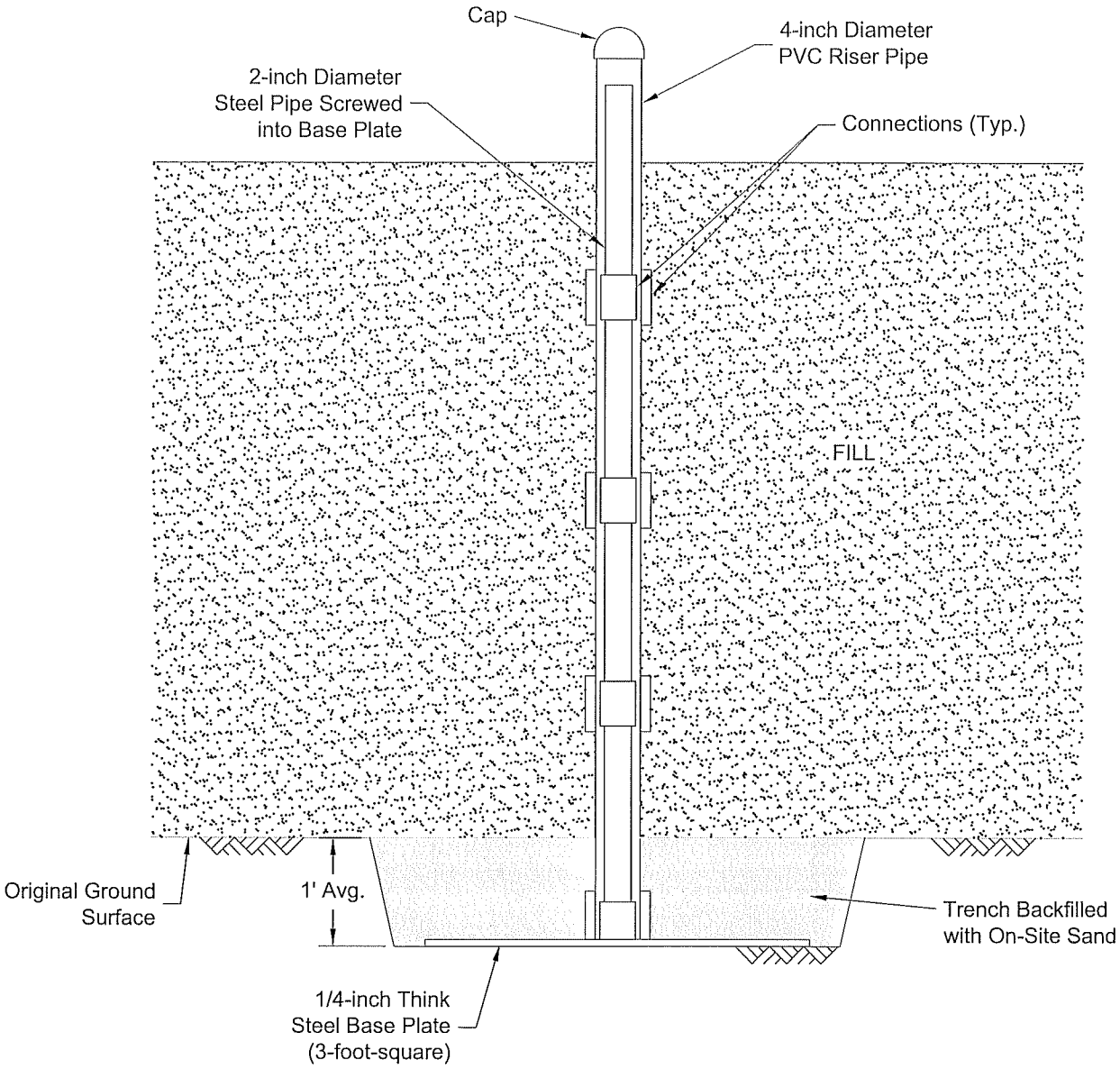
Ramp D Bridge
CDOT Project No. NH1602-114
Durango, Colorado

PILE AND DRIVING EQUIPMENT DATA

July 2008 23-1-01165-003

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2



Ramp D Bridge
CDOT Project No. NH1602-114
Durango, Colorado

SETTLEMENT PLATE SCHEMATIC

July 2008 | 23-1-01165-003

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Geotechnical and Environmental Consultants

FIG. 3

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

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TABLE

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A-1 Summary of Laboratory Test Results by Boring

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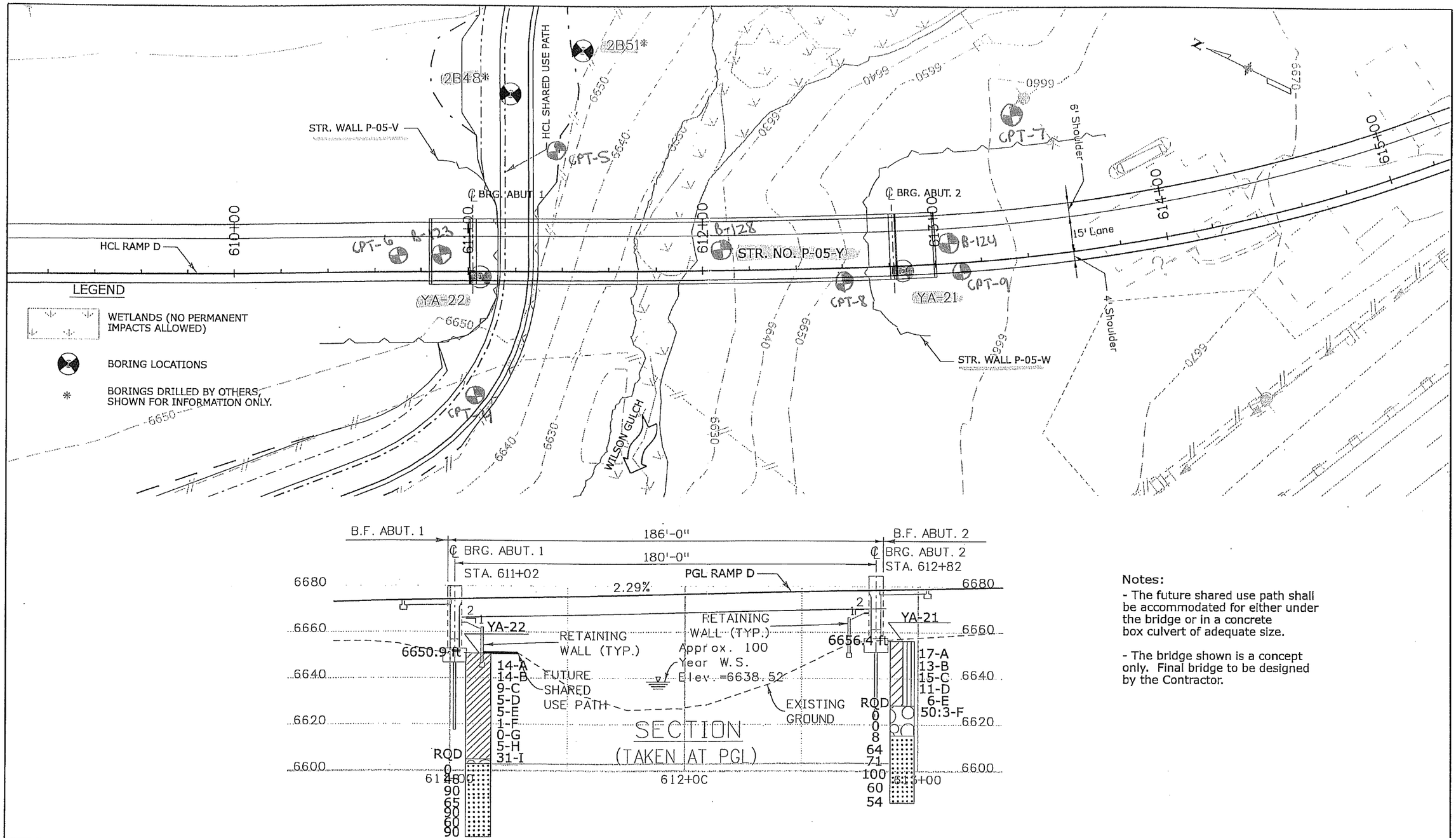
A-3 Log of Boring YA-21 (3 Pages)
A-4 Log of Boring YA-22 (3 Pages)
A-5 Log of Boring B-123 (2 Pages)
A-6 Log of Boring B-124 (2 Pages)
A-7 Log of Boring B-128
A-8 Log of CPT-4
A-9 Log of CPT-5
A-10 Log of CPT-6
A-11 Log of CPT-7
A-12 Log of CPT-8
A-13 Log of CPT-9

TABLE A-1
SUMMARY OF LABORATORY TEST RESULTS BY BORING

SAMPLE DATA		USCS Symbol	AASHTO Symbol	Natural Water Content	Dry Density	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION			SWELL / CONSOLIDATION		R-value	CORROSION TESTING	
Boring	Depth					Liquid Limit	Plastic Limit	Plasticity Index	Gravel	Sand	% Fines	Swell (+) Consolidation (-)	Inundation Pressure		Resistivity	Water Soluble Sulfates
				(%)	(pcf)	(%)	(%)	(%)				(%)	(psf)		ohms-cm	%
YA-21	5 - 6			14.5	86.0							-2.9	Consol			
YA - 22	0 - 5	CH	A-7-6(18)	10.7		52	23	29	2	31	67			<5	1189	<0.001
	5 - 6			19.8	81.5							-0.8	Consol			
	10 - 11			17.6	85.4							-1.2	Consol			
	50 - 55														2400	0.017



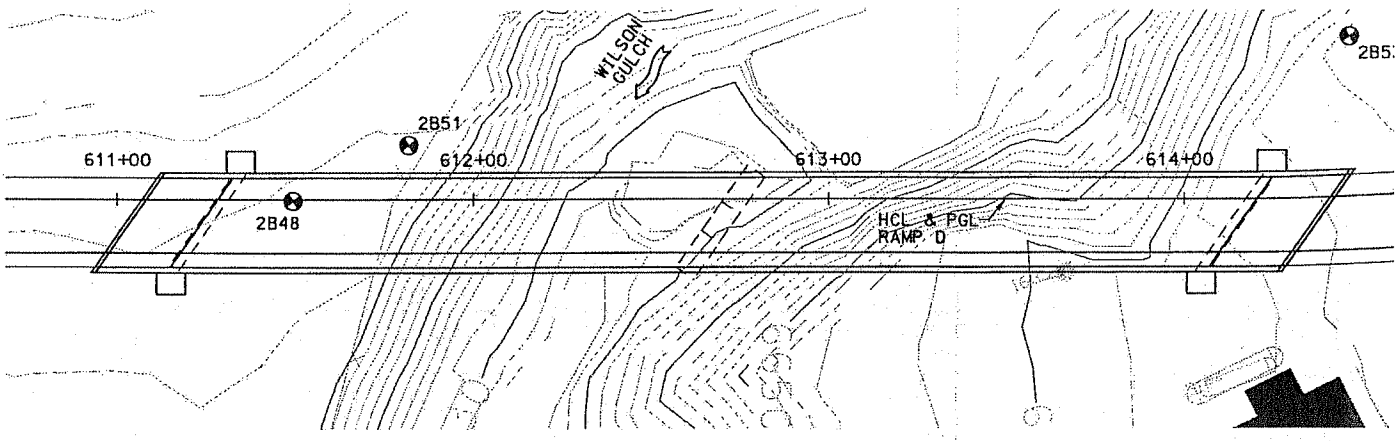
Notes:

- The future shared use path shall be accommodated for either under the bridge or in a concrete box culvert of adequate size.

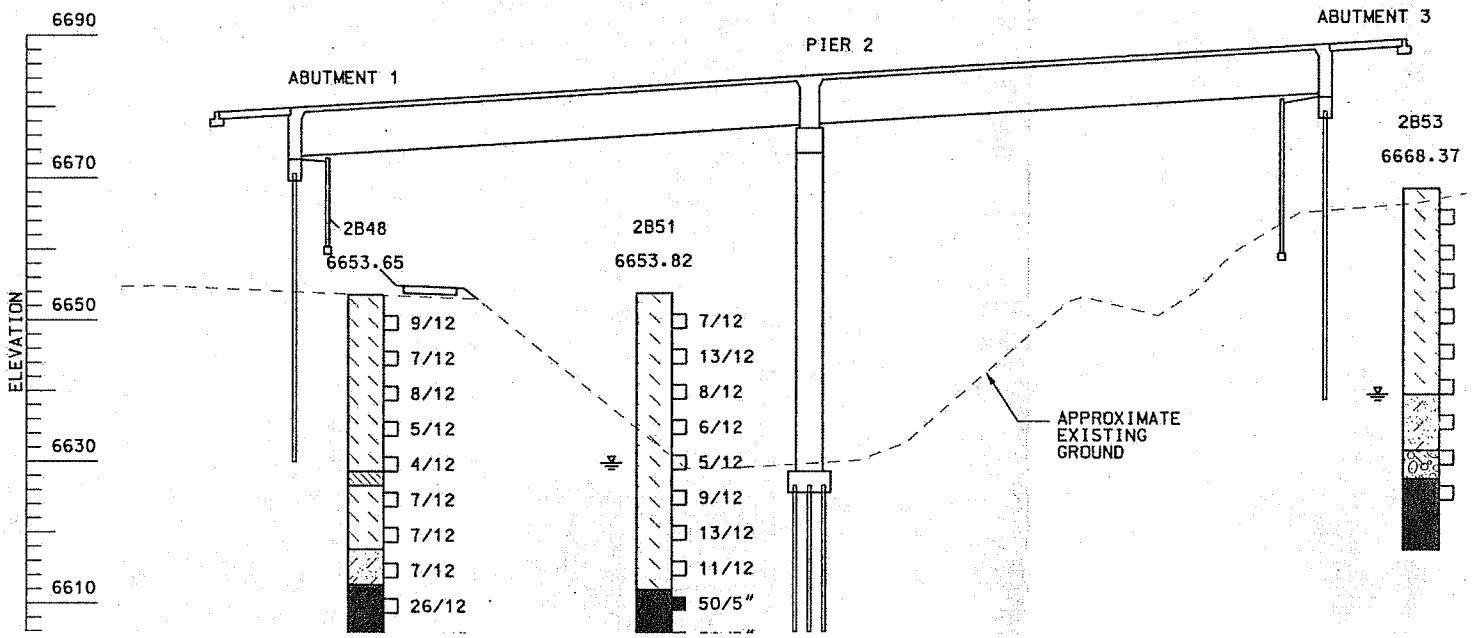
- The bridge shown is a concept only. Final bridge to be designed by the Contractor.

Print Date: 1/3/2008	Sheet Revisions	Colorado Department of Transportation	As Constructed	RAMP D (P-05-Y) GENERAL LAYOUT	Project No./Code
File Name: Ramp D Gen Layout.dgn	Date: / Comments / Init.	3803 North Main Avenue, Suite 200, Durango, CO 81301	No Revisions:		NH 1602-114
Horiz. Scale: 1:40 Vert. Scale: As Noted		Phone: 970-385-1440 FAX: 970-385-8365	Revised:	Designer: spc / Structure Numbers P-05-Y	16042
Unit Information EJA		Region 5 EJA	Void:	Detailer: spc / Sheet Subset: Bridge / Subset Sheets: 4 of 4	Sheet Number 23

FIG. A-1



PLAN



PLOTTED BY: amsnyder

DATE: 04/26/2005

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FIG. A-2

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-21** Sheet 1 of 3

Boring Began: 3/31/2007
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/31/2007
Drill Bit: HQ
Casing:
Weather: sunny and cool

Total Depth: 70.0 ft
Ground Elevation: 6656.4 ft
Location: Ramp D south abutment
Coordinates: N: 2,320,935.6 E: 1,213,673.9

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6655								**0.0 - 28.0 ft. clayey SAND** with silt, lenses sandy clay, brown, low to medium plasticity, dry to wet, loose to medium dense, subangular.	
	5				9/8	17			MC= 14.5 % DD= 86.0 pcf S/C= -2.9 %
6650									
	10				7/6	13			
6645									
	15				7/8	15			
6640									
	20				5/6	11			
6635									
	25				2/4	6			
6630								**28.0 - 41.0 ft. gravelly COBBLES** with sand, brown and reddish brown, no plasticity, wet, very dense, angular, cobbles are pieces of sandstone.	

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-3
(Page 1 of 3)

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-21** Sheet 2 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
					37/50:3"	50:3"			
6625									Begin HQ coring at 32'
			17	0					
	35								
6620			40	0					
	40								Hole is deflected near 25 or 30', coring steel sheared off. 12:00-1:30pm fishing. Recovered.
6615			34	8				**41.0 - 45.0 ft. SANDSTONE**, gray, fresh to predominantly decomposed, medium hard to very soft, parting close to very close, horizontal, iron oxide stains.	
	45							**45.0 - 65.0 ft. SANDSTONE**, gray, fresh, medium hard, parting close, horizontal, clay and iron oxide infilling.	
6610			90	64					
	50								
6605			84	71					
	55								
6600			100	100					
	60								
6595			100	60					
	65							**65.0 - 70.0 ft. SANDSTONE INTERBEDDED WITH CONGLOMERATE**, gray, fresh, medium	

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-3
(Page 2 of 3)

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-21** Sheet 3 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			90	54				hard, parting close, horizontal, clay and iron oxide infilling.	
	70							Bottom of Hole at 70.0 ft.	Coring steel & coring barrel sheared off again. Caved in above top of sheared section, not recovered.
6585									
	75								
6580									
	80								
6575									
	85								
6570									
	90								
6565									
	95								
6560									
	100								
6555									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-3
(Page 3 of 3)

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-22** Sheet 1 of 3

Boring Began: 3/27/2007
Drilling Method: Hollow-Stem Auger/Wireline Coring
Drill: Dietrich D-50 Rubber Track
Driller: DA Smith - Roger
Logged By: S. McKay
Final By: S. McKay
Inclination: Vertical

Completed: 3/28/2007
Drill Bit: HQ
Casing:
Weather: snowy and cold

Total Depth: 79.5 ft
Ground Elevation: 6650.9 ft
Location: Ramp D north abutment
Coordinates: N: 2,320,771.3 E: 1,213,747.3

Ground Water Notes:

Depth	∇ 20.0 ft	-	-	-
Date	3/27/07	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6650								**0.0 - 46.0 ft. sandy CLAY** interbedded with CLAY, brown with green-gray, medium to high plasticity, moist to wet, very soft to stiff.	MC= 10.7 % #200= 67 % LL= 52 PL= 23 PI= 29 pH= 7.8 S= 0.001 % R-Value= 5 Re= 1189 ohms-cm AASHTO: A-7-6 (18) USCS: CH
	5				8/6	14			MC= 19.8 % DD= 81.5 pcf S/C= -0.8 %
6645									
	10				7/7	14			MC= 17.6 % DD= 85.4 pcf S/C= -1.2 %
6640									
	15				4/5	9			
6635									
	∇ 20				2/3	5			
6630									
	25				2/3	5			
6625									Top of screened interval at 28'

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-4
(Page 1 of 3)

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: CDOT US160 US550 Grandview
Project Number: 26-235 Date: 4/2/07

Boring: **YA-22**
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
6620			0		1/0	1			
	35								
6615					0/0	0			Sampler advanced without requiring blows-drill pressure only.
	40		0		2/3	5			
6610									
	45		0		4/27	31			
6605								**46.0 - 48.0 ft. COBBLES**, based on driller's observations only. No sample.	Driller notes it feels like large cobbles or similar
								48.0 - 79.5 ft. SANDSTONE, gray, fresh, medium hard to hard, parting moderately close to close, horizontal, iron oxide stains.	Bottom of screened interval at 48'
	50								Begin HQ Coring at 50'
6600			60	0					
	55								
6595			90	48					
	60								
6590			100	90					
	65								1.5' of core fell out of barrel-recovered in next run
6585									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-4
(Page 2 of 3)

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: CDOT US160 US550 Grandview Project Number: 26-235 Date: 4/2/07	Boring: **YA-22** Sheet 3 of 3

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
			100	66					
	70		100	90					This core run came out almost a 5' stick of core
6580			60	60					
	75		100	90					
6575									
	80							Bottom of Hole at 79.5 ft.	Fractured at bottom because it was drilled twice to make sure we got it.
6570									
	85								
6565									
	90								
6560									
	95								
6555									
	100								
6550									

BORING LOG YEH BORINGS.GPJ YEH ASSOCIATES.GDT 5/10/07

FIG. A-4
(Page 3 of 3)

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-123	NH 1602-114	16042	Fourth Lane	7/8/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-Y Ramp D Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
	74.0ft		Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
			0 - 8 Fill, clayey sand with cobbles		0.0		0%
	4.0		4 - 5 No recovery on drive sample		4.0 4.0	123A CAL	11
	8.0		8 - 28 Sandy clay, trace root fragments 8-10 ft, brown, moist, soft to very stiff		9.0 9.0	123B CAL	15
					14.0 14.0	123C CAL	16
					19.0 19.0	123D CAL	7
					24.0 24.0	123E CAL	4
	28.0		28 - 42 Sandy clay, very soft to medium stiff, brown, moist		29.0 29.0	123F CAL	2
					34.0		
					39.0 39.0	123G	7

SPT DATA: 5 10 20 40 70

WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-5 (Page 1 of 2)

GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/17/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

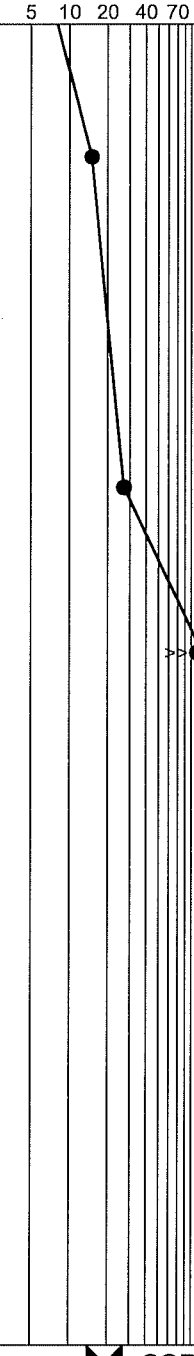
BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-123	NH 1602-114	16042	Fourth Lane	7/8/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-Y Ramp D Bridge/North Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
	74.0ft		Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
		28 - 42 Sandy clay, very soft to medium stiff, brown, moist *(continued)*			CAL	
	42.0	42 - 55 Sandy clay with trace gravel, brown, moist, gravel is angular, particles up to 1.5 inches				
				44.0	123H	15
				44.0	CAL	
				49.0		
				54.0	123I	27
				54.0	CAL	100%
	55.0	55 - 59 Sandstone, grey to light brown, severely weathered, FeO staining on joint surfaces, joint orientation varies widely, medium grained with thin layers of conglomerate				0%
	59.0	59 - 62 Sandstone, as above, less weathered		59.0	123J	50/3"
				59.0	CAL	100%
						35%
	62.0	62 - 74 Shale, intact, no joints, grey, unweathered				
	64.0	64 - 69 Shale, as above, some purple staining (oxide?)		64.0		100%
						75%
	69.0	69 - 74 Shale, as above, grey, unweathered		69.0		100%
						80%
	74.0	Total Boring Depth 74.0ft				

SPT DATA



WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-5 (Page 2 of 2)

GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/17/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

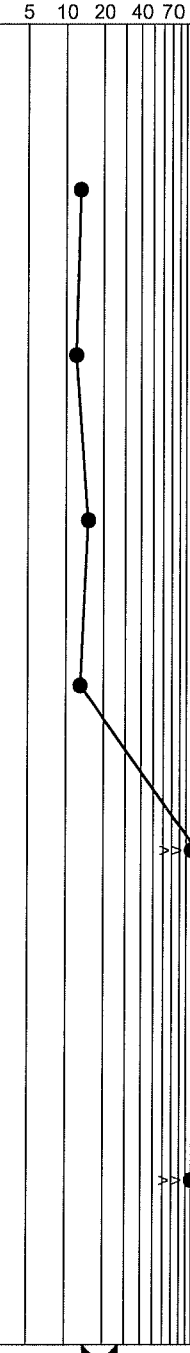
BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-124	NH 1602-114	16042	Fourth Lane	7/8/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-Y Ramp D Bridge/South Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
	45.0ft		Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
			0 - 20 Sandy clay, brown, stiff, moist		0.0		
					5.0 / 5.0	124A CAL	13
					10.0 / 10.0	124B CAL	12
					15.0	124C CAL	15
	20.0		20 - 24.9 Coarse sand with clay, brown, moist, med. dense, trace gravel, sandstone cobble approx. 3 inches diam.		20.0 / 20.0	124D CAL	13
	24.9		24.9 - 34 Shale bedrock, highly fractured, light brown to dark brown, FeO staining on fractures, v. weathered		25.0 / 25.0	124E CAL	58/11" 50% 0%
					30.0		75% 7%
	34.0		34 - 40 Shale, slightly weathered, fractured, grey where intact, lt. brown where weathered along fractures		35.0 / 35.0	124F CAL	50/3" 50% 15%

SPT DATA



WELL DIAGRAM

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-6 (Page 1 of 2)

GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/17/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING #	PROJECT ID	SA	PROJECT NAME	DATE DRILLED
B-124	NH 1602-114	16042	Fourth Lane	7/8/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-Y Ramp D Bridge/South Abutment	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
	45.0ft		Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%	SPT DATA (5 10 20 40 70)	WELL DIAGRAM
	40.0		40 - 42 Highly fractured sandstone, fine grained, frey, Feo coating on joints		40.0		100% / 65%		
	42.0		42 - 44 Sandstone, fine grained, grey, no joints						
	44.0		44 - 45 Shale, grey, intact, curved joint with slickensides at 44.0						
	45.0		Total Boring Depth 45.0ft						

SPT | CON'T | GRAB | SHELBY | CORE

H_2O DEPTH					
DATE					
TIME					

NOTES: CME 550, Backfilled with cuttings

DRAFT

FIG. A-6 (Page 2 of 2)

GEOLOGIC BORING LOG 160 DURANGO ADDENDUM 4.GPJ CO_DOT.GDT 7/17/08

GEOLOGICAL BORING LOG

DEPARTMENT OF TRANSPORTATION

BORING # **B-128**

PROJECT ID	SA	PROJECT NAME	DATE DRILLED
NH 1602-114	16042	Fourth Lane	7/9/08

ROUTE	COUNTY	STRUCTURE/BENT	LOCATION
US 160	La Plata	P-05-Y Ramp D Bridge/Center Pier	

TOP HOLE ELEV	TOTAL DEPTH	SURVEY INFO	GEOLOGIST/FOREMAN
	24.0ft		Laudeman/Blailes/Spahr

ELEV (ft)	DEPTH (ft)	LOG	DESCRIPTION	SAMPLE TYPE	DEPTH (ft)	SAMPLE ID	N-VALUE REC%/RQD%
			0 - 3 Fill, sandy clay, brown, recently placed for drill pad		0.0		0%
	3.0		3 - 5 Clayey sand, med. grained, brown, moist		4.0	128A CAL	10
	5.0		5 - 17 Sandstone, highly weathered, brown, fine grained, oxide staining on joint surfaces		4.0 4.1		20% 0%
					9.0		50% 0%
					14.0		90% 50%
	17.0		17 - 24 Shale, weathered, joints steeply inclined and uneven, FeO stained, grey to reddish brown where weathered		19.0		60% 25%
	24.0		Total Boring Depth 24.0ft				

SPT DATA: 5 10 20 40 70

WELL DIAGRAM: Set 1 inch piezometer

SPT	CON'T	GRAB	SHELBY	CORE

H_2O DEPTH	▽ 6.4
DATE	7/10/2008
TIME	07:15

NOTES: CME 550, Set 1 inch piezometer

DRAFT

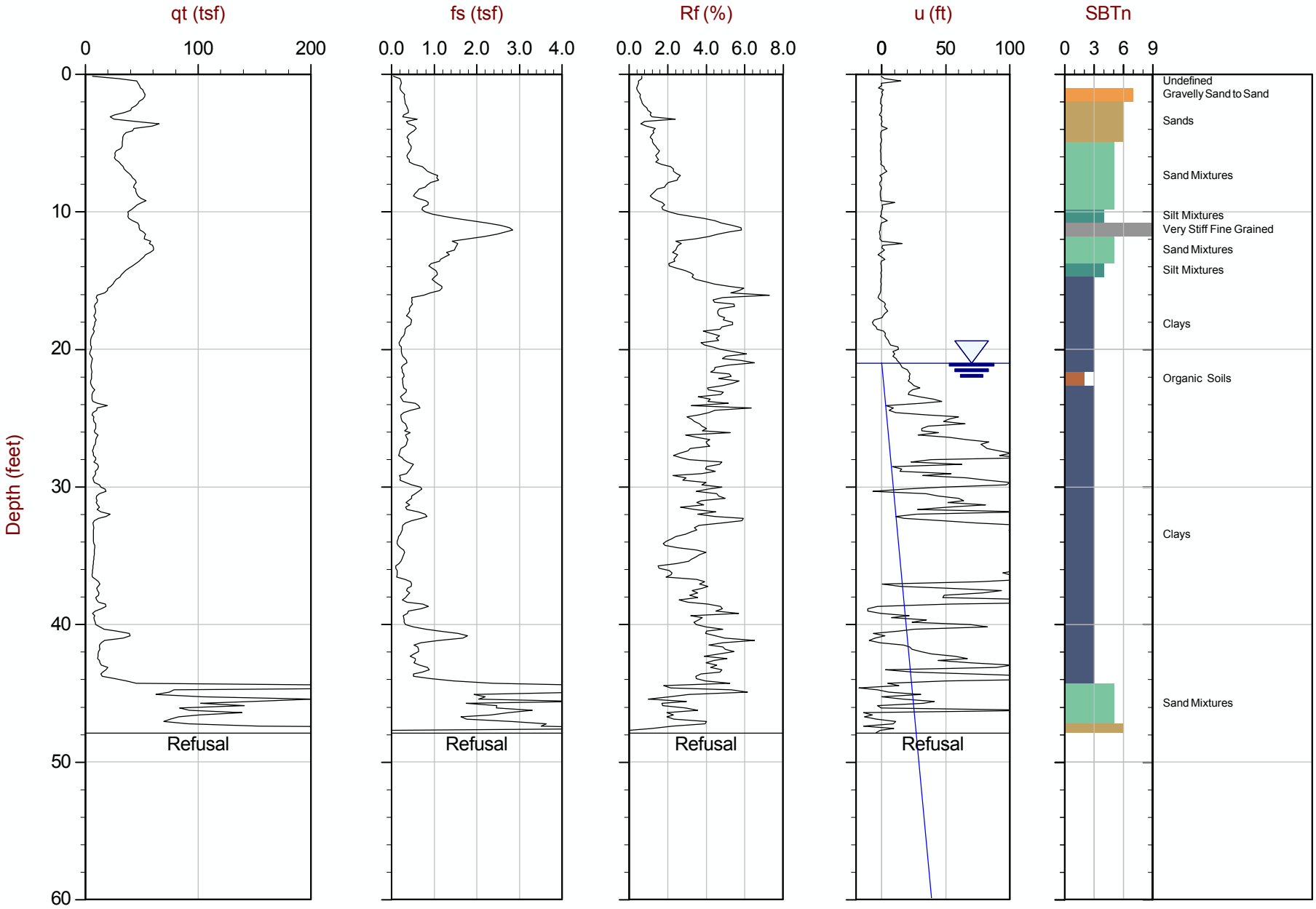
FIG. A-7

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 02:53
Site: US160 / US550

Sounding: CPT-4
Cone: STD 20T AD214



Max Depth: 14.600 m / 47.90 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

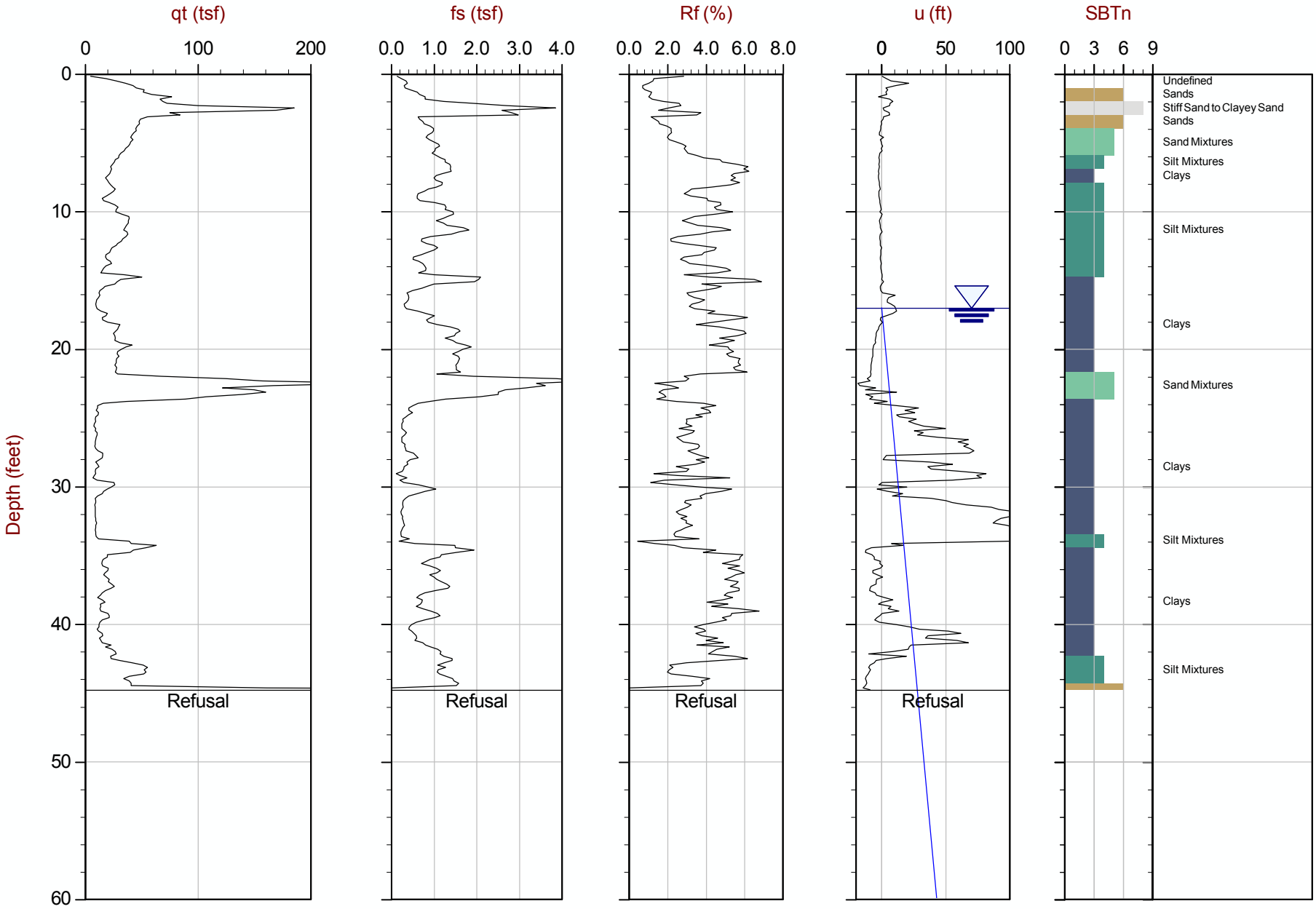
FIG. A-8

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 03:32
Site: US160 / US550

Sounding: CPT-5
Cone: STD 20T AD214



Max Depth: 13.650 m / 44.78 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP04.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

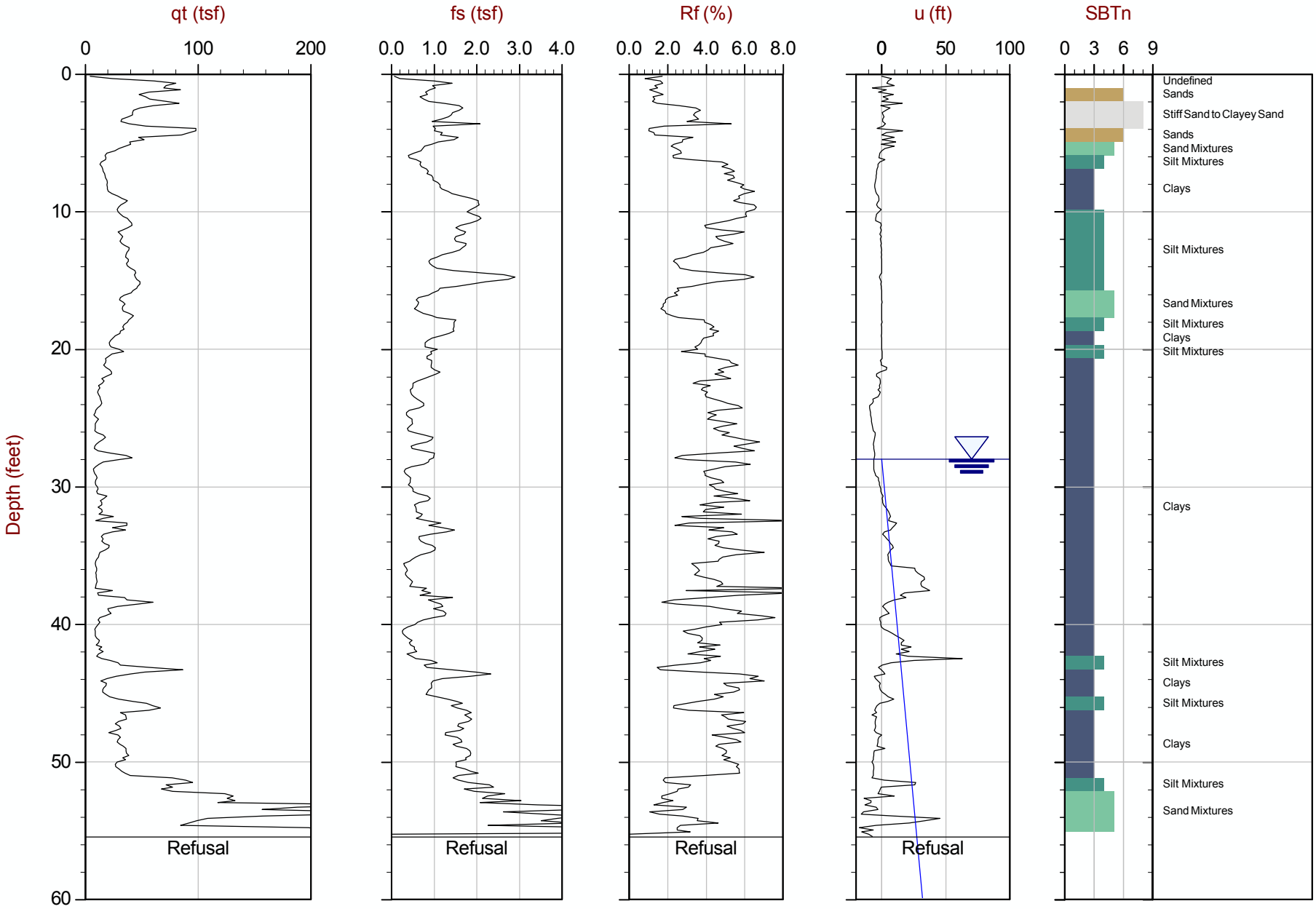
FIG. A-9

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 04:09
Site: US160 / US550

Sounding: CPT-6
Cone: STD 20T AD214



Max Depth: 16.900 m / 55.45 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP05.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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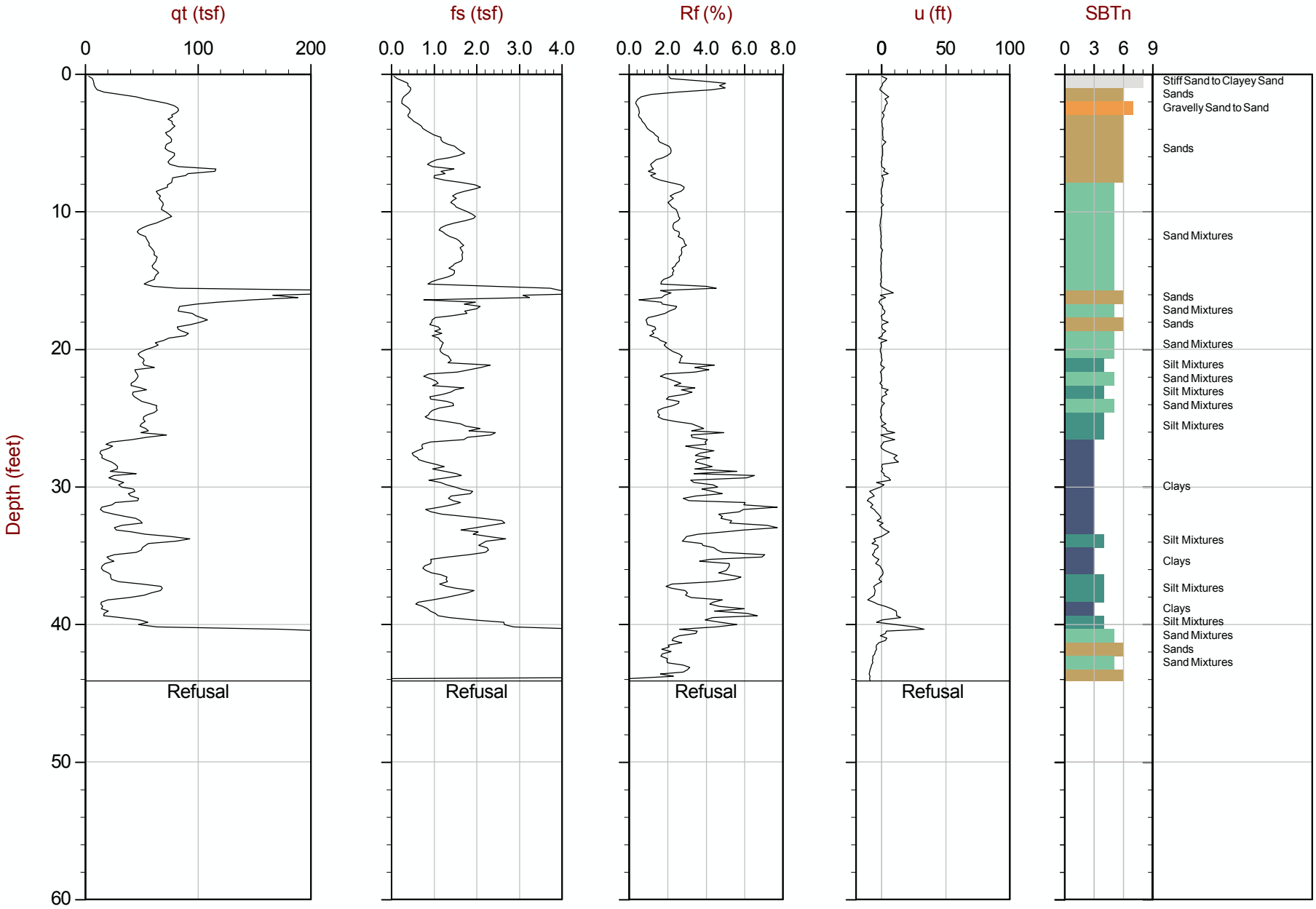
FIG. A-10

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 05:09
Site: US160 / US550

Sounding: CPT-7
Cone: STD 20T AD214



Max Depth: 13.450 m / 44.13 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP06.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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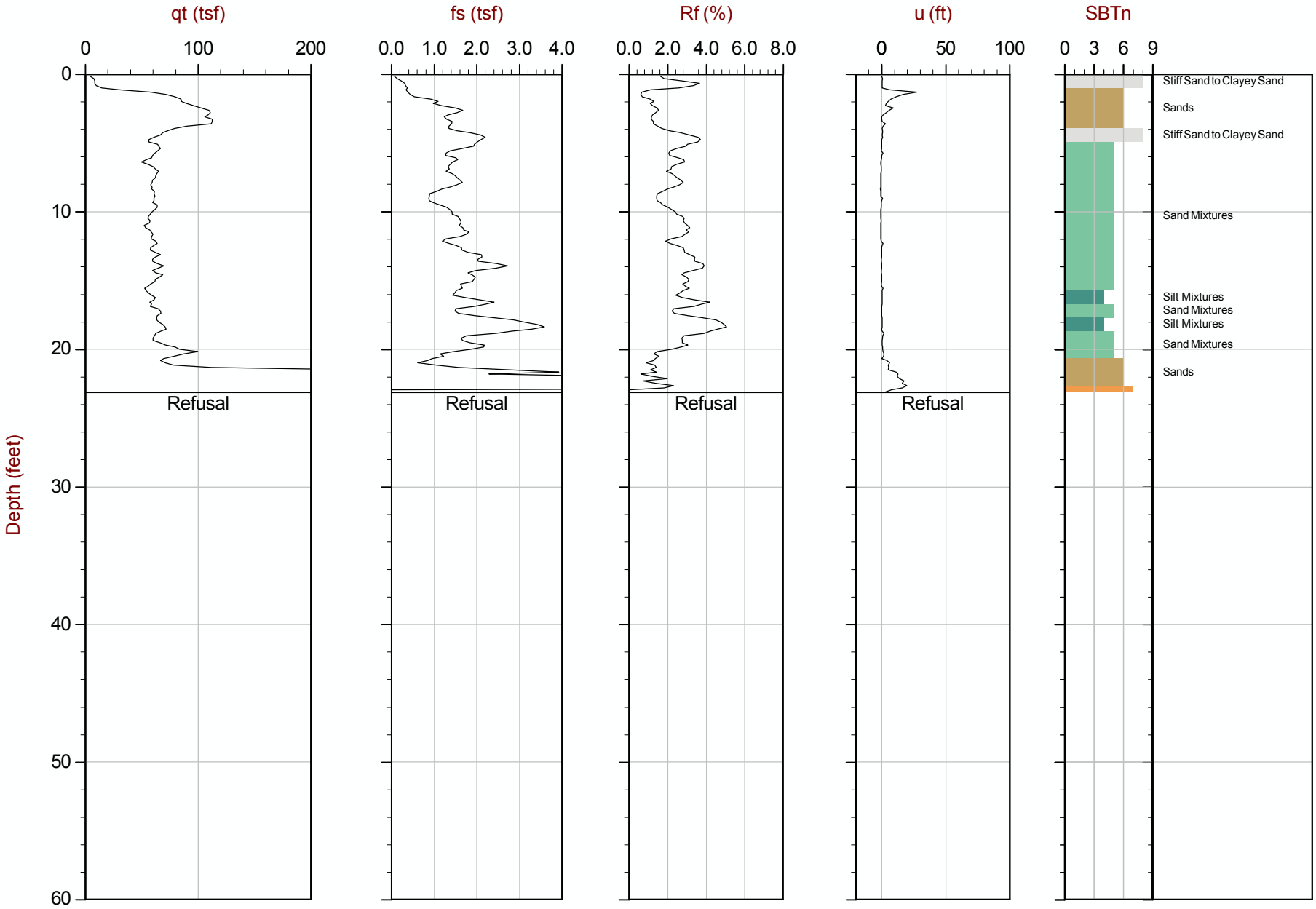
FIG. A-11

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 05:53
Site: US160 / US550

Sounding: CPT-8
Cone: STD 20T AD214



Max Depth: 7.050 m / 23.13 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP07.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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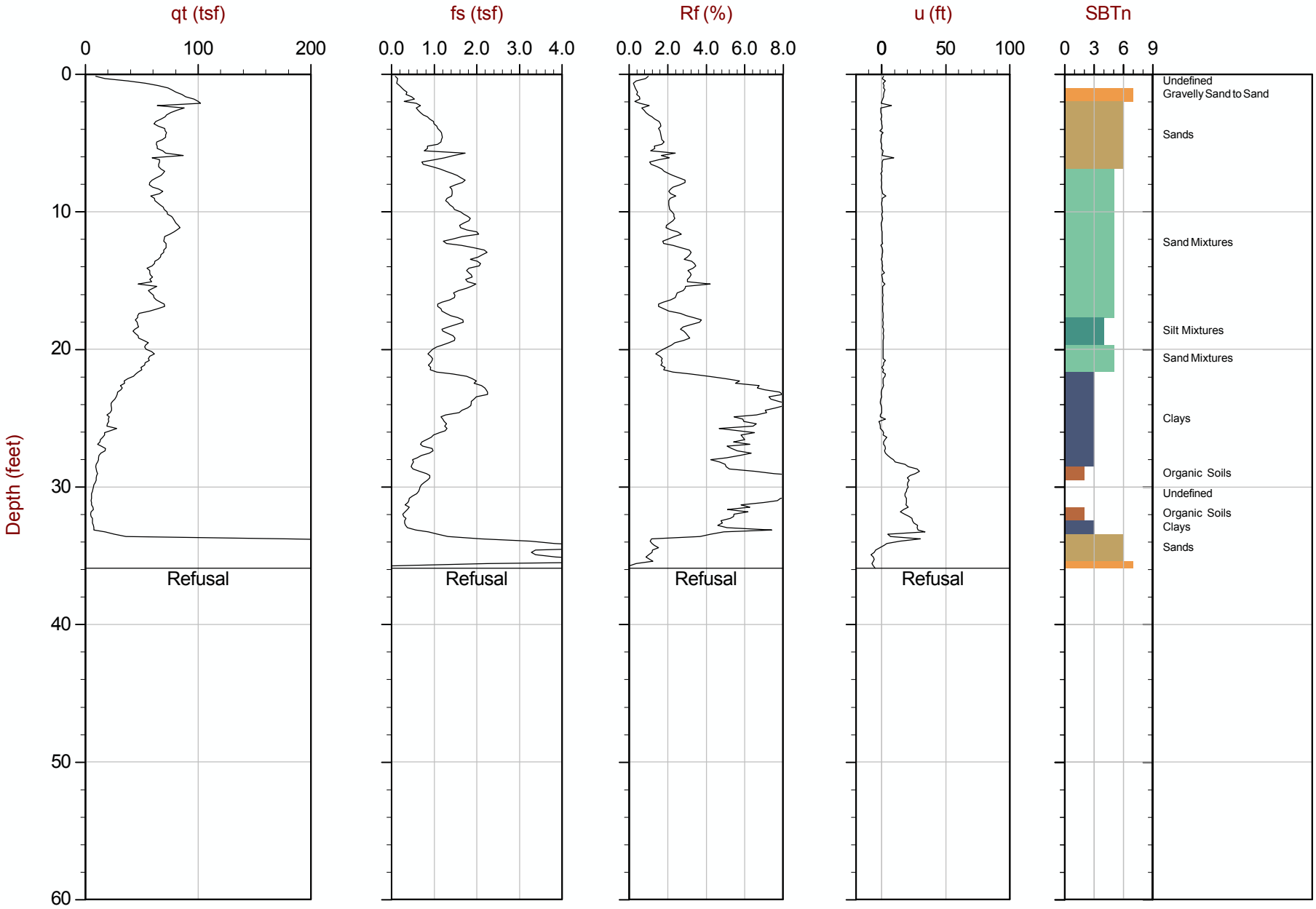
FIG. A-12

CONETEC

Shannon & Wilson

Job No: 08-959
Date: 07:15:08 06:22
Site: US160 / US550

Sounding: CPT-9
Cone: STD 20T AD214



Max Depth: 10.950 m / 35.92 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.300 m

File: 959CP08.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
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FIG. A-13