ATLANTIC COAST PIPELINE, LLC
ATLANTIC COAST PIPELINE

and

DOMINION TRANSMISSION, INC.
SUPPLY HEADER PROJECT

Supplemental Filing
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APPENDIX C

Revised Site Specific Geohazard Mitigation Design Drawings

Geosyntec consultants

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Memorandum

Date:	22 December, 2016
To:	Colin Olness, Dominion
Copies to:	Tony Rice, Geosyntec Seattle
From:	Logan Brant, Geosyntec Houston
Subject:	Revision C Updates to Site Specific Geohazard Mitigation Design Drawings MNF and GWNF Sites - Atlantic Coast Pipeline Geosyntec Project: TXG0007 / 013 / 1210

Following the 8 December, 2016 conference call meeting related to Atlantic Coast Pipeline (ACP) Site Specific Stabilization, Geosyntec Consultants, Inc. (Geosyntec) has revised the site specific geohazard mitigation design drawings developed for the two steep slope sites requested by the Forest Service, located along the ACP Segment AP-1 between Mileposts (MP) 73.20 to 73.50 (MNF) and MP 84.95 to 85.05 (GWNF). The revised drawings are identified as Revision C and are dated December 2016. The changes are largely intended to address comments received during the meeting by Tom Collins of the Forest Service.

Drawing Revisions

The following lists summarize the changes on each drawing incorporated into Revision C.

ACP AP-1 MP 73.20 to 73.50 – Plan and Profile A-A':

- Scales of Plan and Profile A-A' doubled from 1"=200' (11x17) to 1"=100' (11x17), in order to show the slopes in greater detail. This change required expanding the plan and profile to two drawings, with a matchline cut approximately mid-way on the slope.
- Revision C line added to the revision block.

ACP AP-1 MP 73.20 to 73.50 – Sections:

- Four additional sections were added to the drawing set, including one cut through the additional temporary work space. Previous Section B-B' is now C-C' (4428+50) and previous Section C-C' is now F-F' (STA 4439+00). The new sections include Section B-B' (STA 4425+00), Section D-D' (STA 4432+00), Section E-E' (STA 4435+50) and Section G-G' (STA 4442+50). The six sections are spaced at approximately 350 ft intervals along the alignment. This change required presenting the sections on three drawings instead of just one.
- Cutting and filling for ROW grading at individual sections has been revised to better approximate the balanced temporary grading planned for this area of the project.
- Note 3 revised to include sentence indicating that "Temporary cut / fill is for illustrative purposes."
- Revision C line added to the revision block.

ACP AP-1 MP 84.95 to 85.05 – Plan and Profile A-A':

- Marker for Section C-C' on Plan and Profile moved to STA 5351+00, in order to include additional temporary work spaces, located near the top of the slope.
- Revision C line added to the revision block.
- Drawing set expanded from 4 to 5 drawings, so drawing numbering now shows "of 5".

ACP AP-1 MP 84.95 to 85.05 – Detailed Plan and Profile X-X':

- Revision C line added to the revision block.
- Drawing set expanded from 4 to 5 drawings, so drawing numbering now shows "of 5".

ACP AP-1 MP 84.95 to 85.05 – Sections B-B' and C-C':

- Remove dimensions and label for outdated additional temporary work space in Section B-B'.
- Section C-C' moved to STA 5351+00, in order to include additional temporary work spaces, located near the top of the slope.
- Changed "Existing Ground" labels to "Existing / Final Ground" on sections.
- Changed the scale of the sections on this drawing to 1"-40' (11x17), in order to show wider limits at revised Section C-C'.
- Revision C line added to the revision block.

- Drawing set expanded from 4 to 5 drawings, so drawing numbering now shows “of 5”.

ACP AP-1 MP 84.95 to 85.05 – Sections D-D’ and E-E’:

- Changed “Existing Ground” labels to “Existing / Final Ground” on sections.
- Revision C line added to the revision block.
- Drawing set expanded from 4 to 5 drawings, so drawing numbering now shows “of 5”.

ACP AP-1 MP 84.95 to 85.05 – Soil Nail Specifications:

- Soil nail specifications now presented on new Drawing No. 5 of 5.

Additional Comments

We have reviewed the notes and meeting presentations from the 8 December, 2016 meeting that have been circulated by Galileo Project, LLC, and have comments that relate particularly to the “Tom Collins presentation”.

Tom Collins indicated that the “right-of-way (ROW) is 52’ wide”. Geosyntec’s drawings show a 53.5’ permanent ROW because we understand from Dominion that this 53.5’ is the actual ROW width agreed with the Forest Service.

Tom Collins indicated that the Forest Service wants to see accounting for mass balance and that the cross sections presented do not show filling in the temporary workspace or extra temporary workspace. Geosyntec has added cross sections to show temporary filling in the extra temporary workspace, but as previously discussed with Dominion, Geosyntec has not developed a detailed mass-balance based grading design and the profile and cross-sections remain illustrative.

Tom Collins indicated that the “planar bottom of cut” creates a potential slip surface for fill slope failure. Geosyntec has conducted infinite slope type stability analyses for material placed on both ground sloping across (perpendicular to) the ROW and ground sloping along the ROW (parallel to the pipeline). The analyses indicate that the factor of safety and a calculation package presenting the analyses is attached.

Tom Collins indicated that that more detail was required for the profile at the MNF site in order for the Forest Service review. The revised drawing set presents at an enlarged scale, with the profile on two sheets instead of one.

Tom Collins indicated that the Forest Service wants to see a narrative on how the construction is going to progress. We understand that Dominion Construction is preparing this.

Tom Collins indicated that the nature of the surface and the amount of materials to be placed need to be better shown in cross section. We have incorporated a cross section through the extra temporary workspace to illustrate the temporary placement of fill at the MNF site.

Tom Collins indicated that it would be helpful to know more about stump removal in the ROW and temporary workspace areas. We understand that Dominion Construction will include discussion about stump removal in the narrative that is being prepared.

Tom Collins indicated that restoration measures are not shown on the site specific design sheets. They are shown on the alignment sheets. The matter of where restoration details will be shown and not shown is the subject of ongoing discussion. Geosyntec's revised drawings do not currently incorporate final restoration details.

ATTACHMENTS

Rev C Drawings (5 sheets for MNF site and 5 sheets for GWNF site)

Slope Stability Calculation Packages for MNF and GWNF sites.

* * * * *

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **ACP** Project: **ACP Geohazard Analysis Program** Project/ Proposal No.: **TXG0007** Task No. **013**

Title of Computations: Slope Stability Assessment at ACP AP-1 MP 73.20 to MP 73.50

Computations by:	Signature		11/15/2016
	Printed Name	Mustafa Erten, Ph.D., P.E. (TX)	Date
	Title	Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		11/18/2016
	Printed Name	Rodolfo Sancio, Ph.D., P.E. (TX)	Date
	Title	Senior Principal	
Computations Checked by:	Signature		11/18/2016
	Printed Name	Rodolfo Sancio, Ph.D., P.E. (TX)	Date
	Title	Senior Principal	
Computations backchecked by: (originator)	Signature		12/22/2016
	Printed Name	Mustafa Erten, Ph.D., P.E. (TX)	Date
	Title	Engineer	
Approved by: (pm or designate)	Signature		12/22/2016
	Printed Name	Logan Brant, Ph.D., P.E. (WV)	Date
	Title	Senior Engineer	

Approval notes:

Revision History:

No.	Description	Date	By	Checked by	Approval

Written by:	Mustafa Erten	Date:	11/11/2016	Reviewed by:	Rodolfo Sancio	Date:	11/18/2016
Client:	ACP	Project:	Slope Stability at ACP AP-1 MP 73.20 to MP 73.50	Project No.:	TXG0007	Task No.:	013

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INTRODUCTION

Geosyntec Consultants, Inc. (Geosyntec) prepared this calculation package to present results of the slope stability assessment in the vicinity of MP 73.20 to MP 73.50 on the Atlantic Coast Pipeline (ACP) Segment AP-1 alignment. The slope is located in the Monongahela National Forest in Pocahontas County, West Virginia.

This calculation package is organized to present: (i) methodology; (ii) cross sections; (iii) subsurface stratigraphy and geotechnical parameters; and (iv) summary of slope stability analysis results.

METHODOLOGY

The stability of slopes in the vicinity of the pipeline alignment at ACP AP-1 MP 73.20 to MP 73.50 was analyzed using the infinite slope approach considering effective stresses (also known as a drained analysis). In this approach, the slope is assumed to extend infinitely and the slip surface is parallel to the slope surface [Duncan and Wright, 2005]. The infinite slope approach is considered appropriate at this location because the thickness of the potentially unstable materials is small compared to the longitudinal dimension of the slope. For a cohesionless material (i.e., $c' = 0$) and zero pore pressures, the factor of safety against sliding (FS) is independent of the depth of soil and calculated by the following equation:

$$FS = \frac{\tan \phi'}{\tan \beta} \qquad \text{Equation 1}$$

where ϕ' is the soil friction angle and β is the slope inclination.

CROSS SECTIONS

Figure 1 is a plan of the slope showing the pipeline alignment and the selected profile (A-A') and cross sections (D-D', E-E', F-F' and G-G')[1] that were used by Geosyntec for slope stability evaluation. Figure 2 shows the elevation profile along the pipeline path (Section A-A') and the location of the test pits made during the Order 1 Soil Survey near the area of interest [RETTEW and Geosyntec, 2016]. We selected Cross Sections D, E, F, and G in order to capture the steepest side slopes along the pipeline profile. In Figure 3, the elevation and inclination profiles of these cross sections are compared where

[1] These sections are also described herein as Section D, Section E, Section F and Section G.

positive slope inclination values indicate an upslope and negative values indicate downslope. Figure 3 also shows the permanent right of way (ROW) since it is the main area of interest for geohazard mitigation. The permanent ROW extends 26.75-ft offset on both sides of the pipeline centerline, with additional temporary ROW often extending beyond the permanent ROW.

SUBSURFACE STRATIGRAPHY AND GEOTECHNICAL PARAMETERS

The estimated material properties and developed soil stratigraphy for this site can be found in Appendix A. Since no ground water was observed at the site and the cohesion (c') was conservatively assumed as 0 psf, the only soil parameter that was used in the infinite slope stability analysis was the friction angle (ϕ') of the subsurface soil. Table 1 lists the geotechnical parameters that we used in the slope stability analysis.

Table 1. Geotechnical Soil Parameters

Soil Type	USCS	Total Unit Weight (pcf)	Effective Stress Cohesion (psf)	Effective Stress Friction Angle (°)
Silts and Sandy / Gravelly Silt	ML	110	0	32

Geosyntec considers appropriate the use of zero pore pressure in the stability analysis because of the enhanced surface water drainage measures that are being implemented through the incorporation of Best in Class (BIC) incremental controls [Atlantic Coast Pipeline, 2016]. We also consider the use of $c' = 0$ conservative because the soil is likely to exhibit some apparent cohesion caused by: i) root systems of the vegetation after ROW restoration and pre-existing vegetation; and ii) interstitial water tension in the partially saturated soil.

SUMMARY OF STABILITY ANALYSIS RESULTS

The slope inclination along the pipeline profile section was approximately in the range of 15% and 40% (Figure 2). The infinite slope analysis using Equation 1 on this cross section using a friction angle (ϕ') of 32° and zero pore pressures results in FS between 4.16 and 1.56. Since FS is larger than 1.0, no engineered site specific geohazard mitigation design is recommended.

Written by:	**Mustafa Erten**	Date:	**11/11/2016**	Reviewed by:	**Rodolfo Sancio**	Date:	**11/18/2016**
Client:	**ACP**	Project:	**Slope Stability at ACP AP-1 MP 73.20 to MP 73.50**	Project No.:	**TXG0007**	Task No.:	**013**

The side slope at Section E within the construction ROW approximately varies between 50 and 60% (Figure 3). The infinite slope analysis on this cross section using a friction angle (ϕ') of 32° and zero pore pressures results in FS between 1.25 and 1.04. Since FS is larger than 1.0, no geohazard mitigation at this area is considered necessary. Moreover, the disturbed portion of the ROW is relatively short.

Figure 3 shows that the slope inclination at Sections D, F, and G within the limits of permanent ROW is usually 20% to 40%. Due to relatively mild side slopes, no geohazard mitigation is considered necessary in these areas.

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Atlantic Coast Pipeline [2016]. Best In Class (BIC) Incremental Controls, Working-Draft, 3 November 2016.

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FIGURES

Written by:	Mustafa Erten	Date:	11/11/2016	Reviewed by:	Rodolfo Sancio	Date:	11/18/2016
Client:	ACP	Project:	Slope Stability at ACP AP-1 MP 73.20 to MP 73.50	Project No.:	TXG0007	Task No.:	013

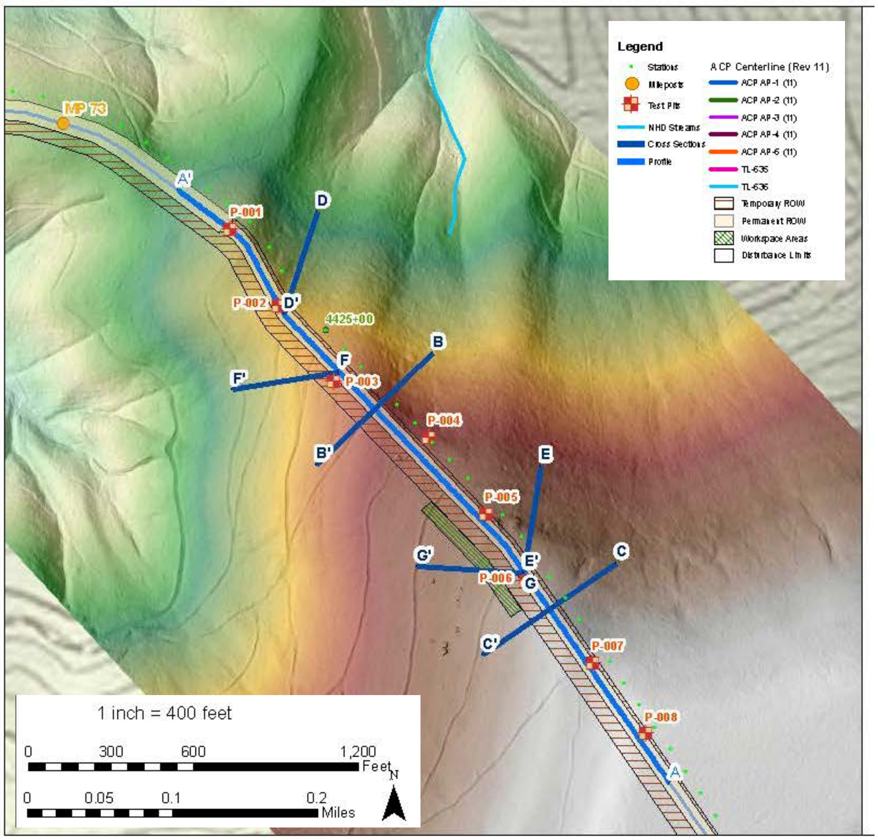

Figure 1. Location of Cross Sections and Profile Section

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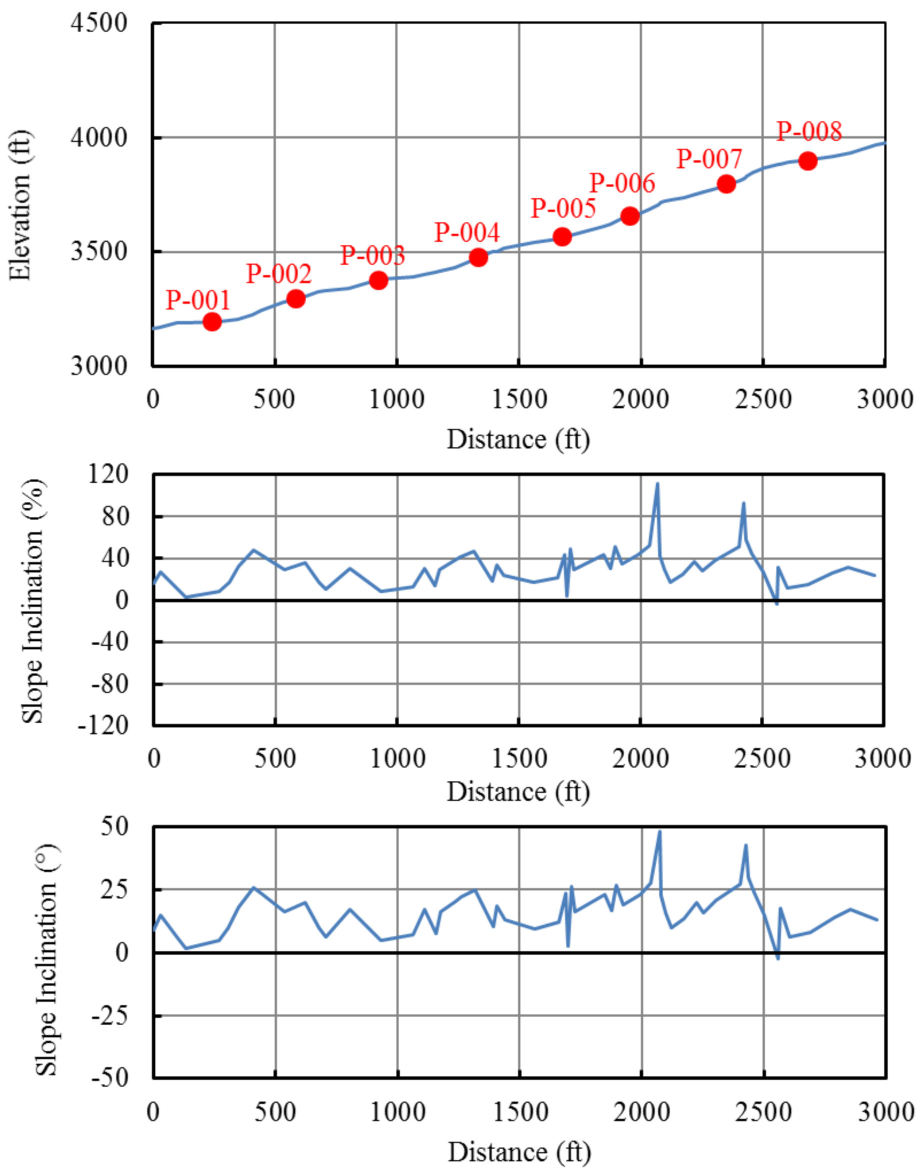

Figure 2. Elevation and Slope Inclination Profiles for Section A

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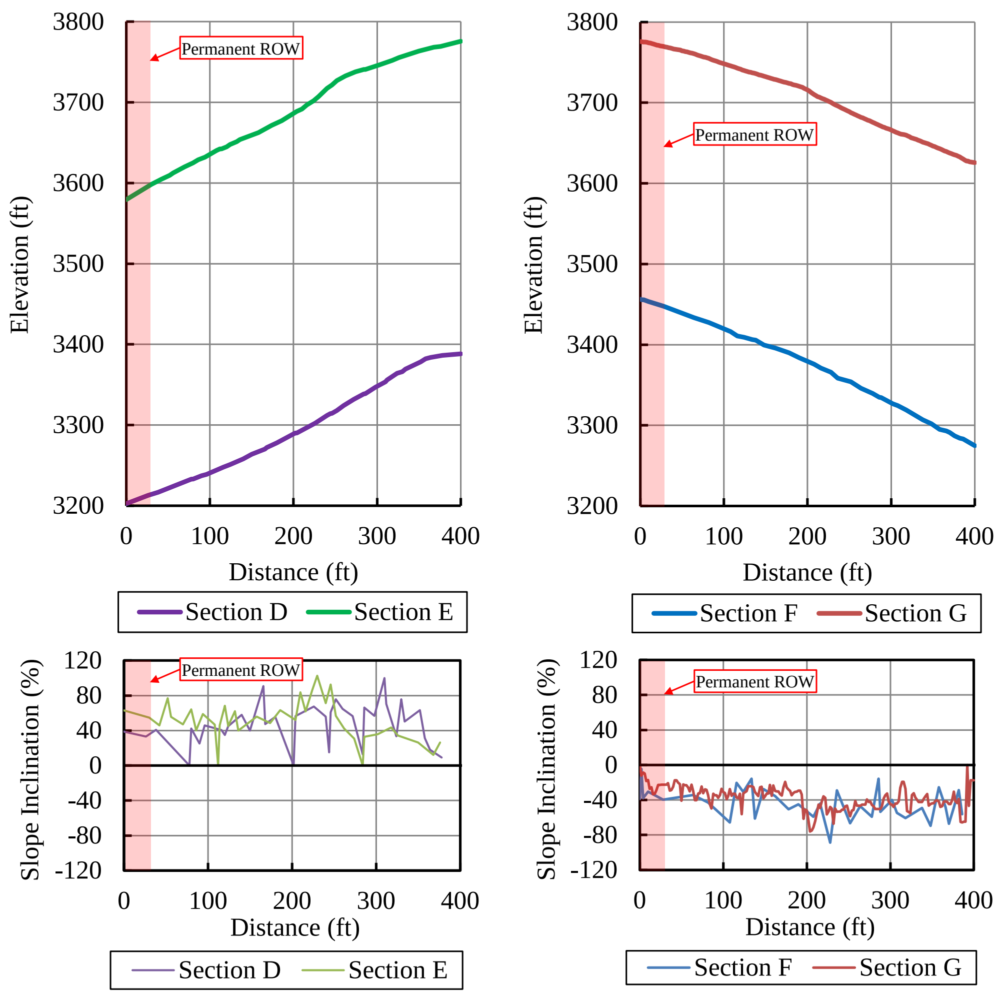

Figure 3. Elevation and Slope Inclination Profiles for Sections D, E, F, And G

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Appendix A

Geotechnical and Topographical Inputs for Site-Specific Geohazard Mitigation Design at ACP AP-1 MP 73.20 to MP 73.50

APPENDIX A - GEOTECHNICAL AND TOPOGRAPHICAL INPUTS FOR SITE-SPECIFIC GEOHAZARD MITIGATION DESIGN AT ACP AP-1 MP 73.20 TO MP 73.50

INTRODUCTION

This document summarizes and interprets the available geotechnical and topographical information for use as inputs in the site-specific slope stability analyses and geohazard mitigation design for pipeline construction and right-of-way restoration on the slope at the Atlantic Coast Pipeline (ACP) Segment AP-1 Milepost (MP) 73.20 to MP 73.50. This steep slope area is located in the Monongahela National Forest in Pocahontas County, West Virginia.

SURFACE TOPOGRAPHY

The proposed pipeline alignment at ACP AP-1 MP 73.20 to MP 73.50 is along the mountain ridge. The slope usually rises towards the southeast with slope inclination in the range of 15% (8.5°) and 40% (21.8°). At several locations, the slope inclination increases to approximately 60% (31°) for about 100 ft to 150 ft horizontal distance. Figure 1 shows the elevation profile of the steep slope area.

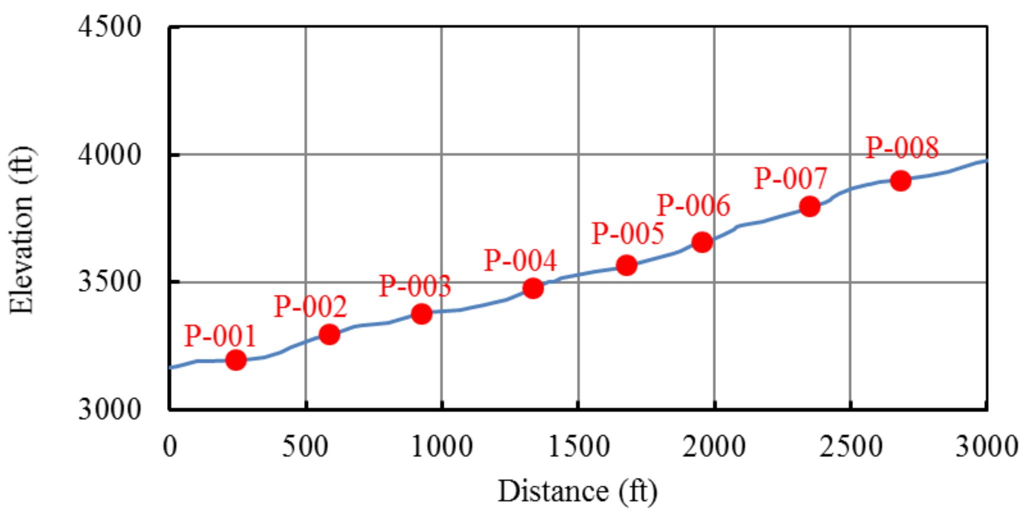

Figure 1. Elevation Profile

SUBSURFACE CONDITIONS

Available information on the subsurface conditions at the site is largely based on field observation and laboratory testing related to the Order 1 Soil Survey conducted by Geosyntec

Consultants and their subcontractors. For the soil survey, eight test pits were excavated in the vicinity of the slope that is the interest of this assessment [Rettew and Geosyntec, 2016].

Soil

The soil profiles at the test pit locations, which reached a maximum depth of 50 inches, were logged by soil scientists using the classification system in the Soil Survey Manual by U.S. Department of Agriculture [1993]. The soil profiles at all test pits were identified as predominantly silt loam with various proportions of rock fragments. The rock fragment content increased from 10 to 90 percent with depth. In this classification system, rock fragments are defined as any soil particle larger than 2 mm in diameter (the coarse/medium sand threshold used by geotechnical engineers).

Table 1 summarizes the USDA Soil Classifications and the percentage of the rock fragments of the soils in each test pit.

Table 1. USDA Soil Classification of Test Pits

Test Pit ID	USDA Soil Name	USDA Map Symbol	Rock Fragments	Depth to Bedrock (ft)
P-001	Cateache channery silt loam	CfF	5%-60%	4.2
P-002	Cateache channery silt loam	CfF	5%-50%	3.2
P-003	Cateache channery silt loam	CfF	10%-40%	4.2
P-004	Cateache channery silt loam	CfF	10%-90%	4.2
P-005	Cateache channery silt loam	CfF	20%-40%	N/A
P-006	Cateache channery silt loam	CfF	5%-25%	N/A
P-007	Cateache channery silt loam	CfE	5%-85%	N/A
P-008	Cateache channery silt loam	CfE	40%-80%	2.6

Note: N/A: Bedrock was not encountered at the test pit.

The review of USDA's database indicates that the CfF type of Cateache channery silt loams are mapped on the steep slope areas where the slope inclination was in the range of 35% and 55% and CfE type was mapped in the areas with slope inclinations of 15% to 35% (Soil Survey Staff, 2016). Cateache silt loam (both CfE and CfF) is composed of silt loam and loam with significant gravel content. According to the Unified Soil Classification System (USCS), this silt loam is similar to CL, ML, GC or GM. The gravel content shows an increasing trend with depth. The deeper horizons just on top of the bedrock are more likely to be classified as GC or GM. For this soil type, typically the liquid limit (LL) values vary between 20 and 40, with an average of 30, and plasticity index (PI) values vary between 4 and 15, with an average of 10.

Four of the eight test pits were also logged by a geotechnical engineer to record soil descriptions for engineering purposes. These descriptions were prepared in accordance with ASTM D2488. Group symbols based on the USCS were also developed for each soil. The geotechnical engineering description of the soils in all test pits were gravelly silt (ML) with varying gravel content. Table 2 summarizes the geotechnical engineering soil descriptions in selected test pits.

Table 2. Geotechnical Engineering Soil Descriptions of Test Pits

Test Pit ID	Geotechnical Engineering Soil Description	USCS Group
P-001	Silt with trace gravel	ML
P-003	Silt with little to few gravel	ML
P-004	Silt with little to some gravel	ML
P-005	Silt with few sand and gravel	ML

Geotechnical laboratory testing was not conducted on soil samples recovered from these specific test pits (P-001 to P-008). However, geotechnical laboratory index and classification testing was performed on soil samples collected from ten other test pits within the George Washington and Monongahela National Forests (one sample from each test pit as listed in Table 3). The index properties of the soil samples are generally consistent where fines contents are at or near 50% and LL values generally are in the range of 33 to 40, with one sample at 52. The PI values varied between 7 and 17. The visual characteristics of the samples tested in the laboratory were similar to the CfE and CfF soil units. The fine-grained portion (i.e., portion smaller than 0.75 mm) of eight out of 10 samples were classified as low plasticity silt (ML), and are considered to be similar to the fines portions of the soils encountered in the test pits at P-001 to P-008. The laboratory test results for those ten samples are shown in Table 3.

Table 3. Laboratory Test Results from Test Pits for Order 1 Soil Survey Program

Test Pit ID	USCS	USCS for Fines	Moisture Content (%)	Fines Content (%)	LL	PL	PI
P-047	ML	ML	14.9	59.4	34	25	9
P-091	GM	ML	14.4	42.2	NP	NP	NP
P-092	ML	ML	22.9	55.1	38	26	12
P-115	GM	ML	12.3	41.7	33	26	7
P-217	SM	MH	21.8	42.9	52	35	17
P-226	ML	ML	19.8	58.8	34	24	10
P-244	GM	ML	17.1	43.5	36	26	10
P-275	SM	ML	16.4	44.1	40	26	14
P-300	ML	ML	23.2	53.7	35	27	8
P-314	GC	CL	17.7	48.2	33	23	10

Note: Fines contents are percentage of soil particles smaller than 75 μm.

Bedrock

The site is underlain by the Bluefield Formation which is composed of red and green shale and sandstone with a few thin limestone lenses [Cardwell et al. 1968]. The test pit logs by soil scientists also confirm that the bedrock type that was encountered at the site was sedimentary rocks (usually sandstone or siltstone).

Bedrock was encountered in five out of eight test pits. In three test pits (P-001, P-003 and P-004) the bedrock was encountered at 4.2 ft below ground surface (bgs) (i.e., at the bottom of the test pits). At test pits P-002 and P-008, the bedrock depth was measured as 3.2 ft and 2.6 ft bgs, respectively. According to the USDA database, the bedrock depth that CfE and CfF types of silt loams is in the range of 2.8 ft to 3.1 ft, which is in general agreement with the field observations.

The bedding plane strike and dip was only measured at P-001 and P-008, since no clear bedrock surface or consistent bedrock alignment was identified in other locations. At test pit P-001, the ground surface inclination is 36% (19.8°), the bedrock dipped 4° into the slope. At test pit P-008, the ground surface inclination was 5% (2.9°) and the bedrock dipped 2° into the slope. Table 4 summarizes the bedrock observations in five test pits.

Table 4. Bedrock Observations in Test Pits

Test Pit ID	Bedrock Type	Bedrock Depth (ft)	Bedding Plane Dip	Bedding Plane Strike
P-001	Siltstone	4.2	4° S	S 64° E
P-002	Sandstone	3.2	Not measured	Not measured
P-003	Siltstone	4.2	Not measured	Not measured
P-004	Siltstone	4.2	Not measured	Not measured
P-008	Sandstone	2.6	2° N	N 74° W

Groundwater

The test pit logs prepared by soil scientists on June 20, 2016 reported that the ground water table (GWT) was 2 ft bgs and 2.7 ft bgs at test pits P-001 and P-003, respectively. According to D. Fenstermacher of RETTEW (personal communication, 11/30/2016), this observation is not based on the measurement of any standing water depth, but it is the identification of the redoximorphic features (redox) in the test pits profiles. The redox is formed in conditions of saturation and typically found in zones where the groundwater table fluctuates throughout the year, even if the groundwater table level is not present at the time of test pit observation. Geotechnical assessment of the test pits on the same day did not report any standing water.

RECOMMENDED DESIGN GEOTECHNICAL PARAMETERS

Geosyntec has estimated site-specific design parameters to support the geohazard mitigation design of the slope at ACP AP-1 MP 73.20 to MP 73.50.

Soil

The soil observed in the test pits are typically the product of in situ weathering of the parent rock (i.e., residual soil). These soils may therefore retain some cohesion. Additionally, they are partially saturated, thus exhibit apparent cohesion caused by interstitial pore water tension. Moreover, at shallow depths (e.g., < 2 ft), they also exhibit apparent cohesion caused by the root mat of deciduous trees, shrubs, and grasses. Upon saturation, however, the apparent cohesion caused by interstitial tension is likely to decrease or disappear. Also, the removal of vegetation to establish the right of way will decrease the effect of the root mat.

Stark et al. [2013] provides relationships to estimate the drained secant friction angle of fine grained soils as a function of clay fraction, effective confining pressure (σ_n'), and ball-milled derived liquid limit (LL) values for slope stability calculations. Using the average LL value of 36 for the soil listed in Table 3, the corresponding ball-milled derived LL value was calculated as 48 using the relationship suggested in Stark et al. [2013].

Using Figure 2 (adopted from Stark et al., 2013), the drained secant friction angle was estimated by Geosyntec to be 32 degrees for fully softened condition for clay content (CF) less than 20% and effective normal stress of 1044 psf (50 kPa), which is equivalent to about 10 ft of soil above the GWT (the friction angle increases as the confining stress decreases). As discussed by Stark et al. [2013], the selection of fully softened shear strength parameters would be proper for overconsolidated soils; however, they may be conservative for first-time slides, for which a cohesion term is appropriate.

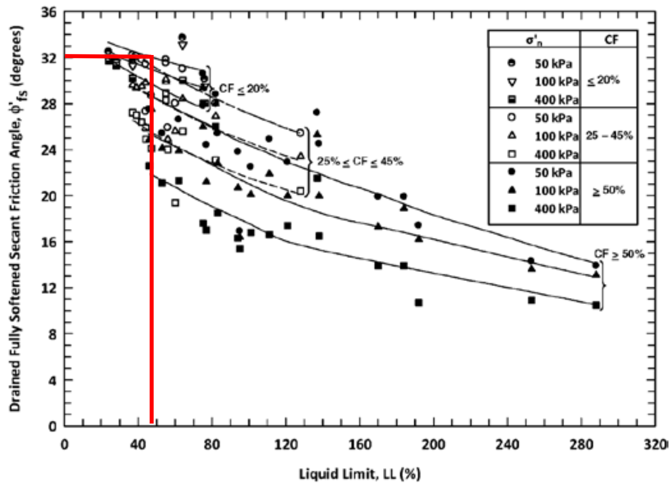

Figure 2. Empirical Correlation for Fully-Softened Drained Secant Friction Angle based on Ball-Milled Derived LL, CF, and σ'_n (Adapted from Stark et al. (2013).

Table 5 shows the selected total unit weight and drained shear strength parameters for the soil at this site. The selected total unit weight value is the upper range of typical value for ML soils, as given in Coduto [2001].

Bedrock

Since the bedrock strength is not believed to control the minimum factor of safety against slope stability, infinite strength was assigned for the bedrock. The total unit weight for bedrock was estimated for a typical sedimentary rock.

Table 5. Summary of Geotechnical Properties for Slope Stability Analysis

Soil Type	USCS Group	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (°)
Silts and Sandy / Gravelly Silt	ML	110	0	32
Bedrock	-	150	Infinite Strength	

Groundwater

Based on the available information, Geosyntec assumed the ground water at top of the bedrock below the soil layer for the purpose of geotechnical analyses. Ground water level can potentially fluctuate due to seasonal changes and periodic precipitations. However, given the enhanced drainage measures that will be implemented on steep slopes through implementation of the Best in Class (BIC) program, we consider appropriate to assume a condition of partial saturation in the soil profile.

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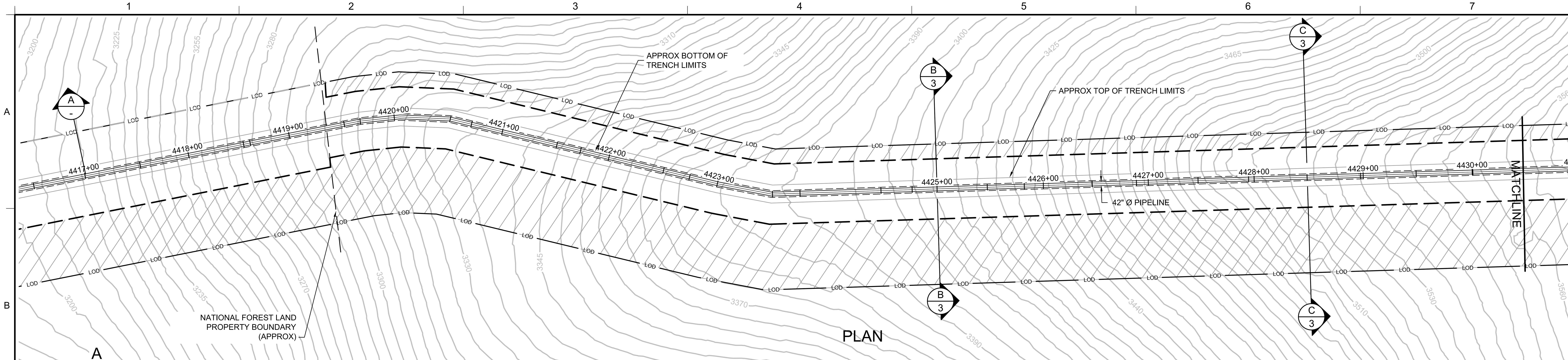
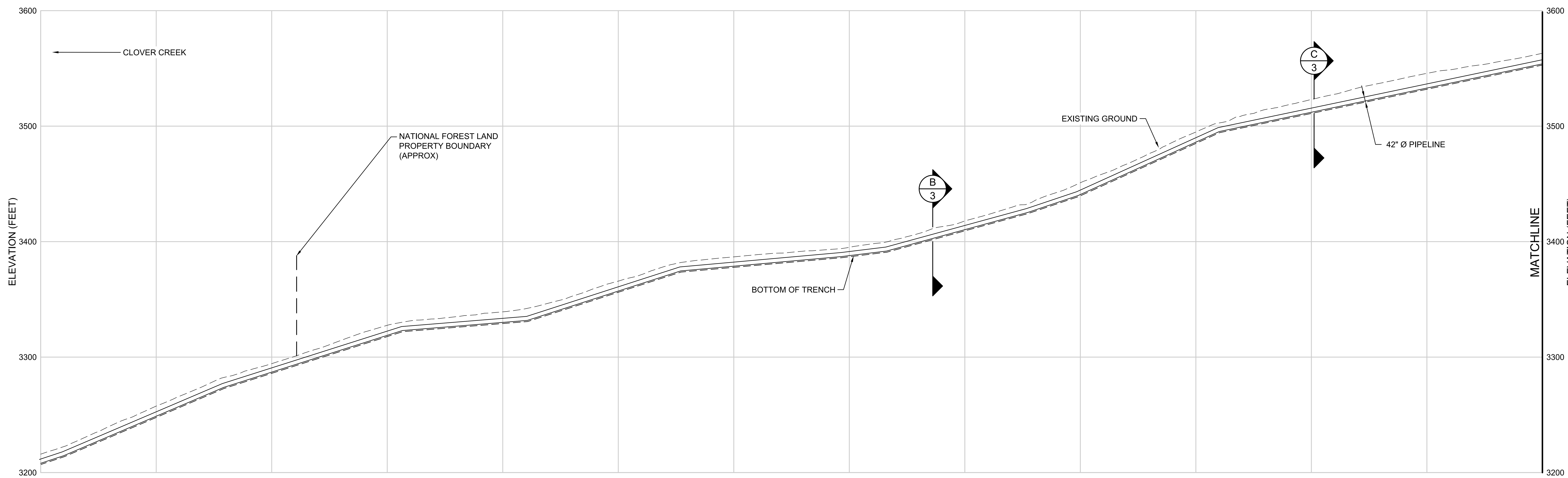


PLAN



PROFILE A -A'

LEGEND

2650	EXISTING GROUND ELEVATION CONTOUR (FT, MSL)
LOD	LIMIT OF DISTURBANCE
	PERMANENT (ROW)
4426+00	PIPELINE CENTERLINE AND STATIONING
	TEMPORARY (ROW)
	EXTRA WORK SPACE

0 50 100
SCALE IN FEET

NOTES:

1. MAPPING AND TOPOGRAPHY BASED ON UTM COORDINATE SYSTEM WITH NAD83 DATUM, ZONE 17, US SURVEY FOOT, CENTRAL MERIDIAN 81°W.
2. STATIONING SHOWN IS SLOPE STATIONING FOR ROUTE REV 11A (3D).
3. CONTOURS AND TOPOGRAPHIC FEATURES DERIVED FROM LIDAR DATA AND GPS SUB-METER GROUND SURVEY PERFORMED BY GAI CONSULTANTS, INC.
4. STREAM AND WETLAND DATA PROVIDED BY NRG/ERM.
5. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
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Atlantic Coast Pipeline

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TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN PLAN AND PROFILE A-A'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN MP 73.20 TO 73.50 (AP-1)

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DESIGN BY:	LCB/TR	DATE:	DECEMBER 2016
DRAWN BY:	JJV/KH	PROJECT NO.:	TXG0007.13
CHECKED BY:	LCB	FILE:	TXG000713D01
REVIEWED BY:	RS	DRAWING NO.:	
APPROVED BY:	TR		1 OF 5

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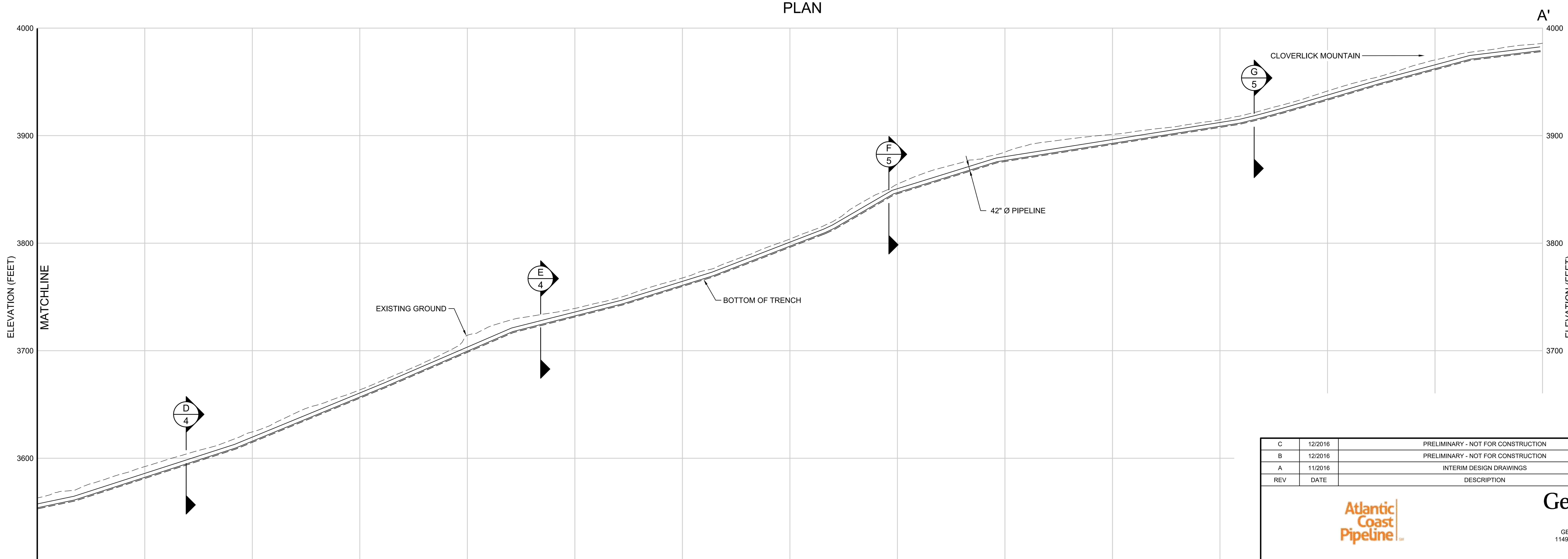

PROFILE A - A'

LEGEND

2650	EXISTING GROUND ELEVATION CONTOUR (FT, MSL)
LOD	LIMIT OF DISTURBANCE
— — —	PERMANENT (ROW)
4426+00	PIPELINE CENTERLINE AND STATIONING
	TEMPORARY (ROW)
	EXTRA WORK SPACE

0 50 100
SCALE IN FEET

NOTES:

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PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN MP 73.20 TO 73.50 (AP-1)

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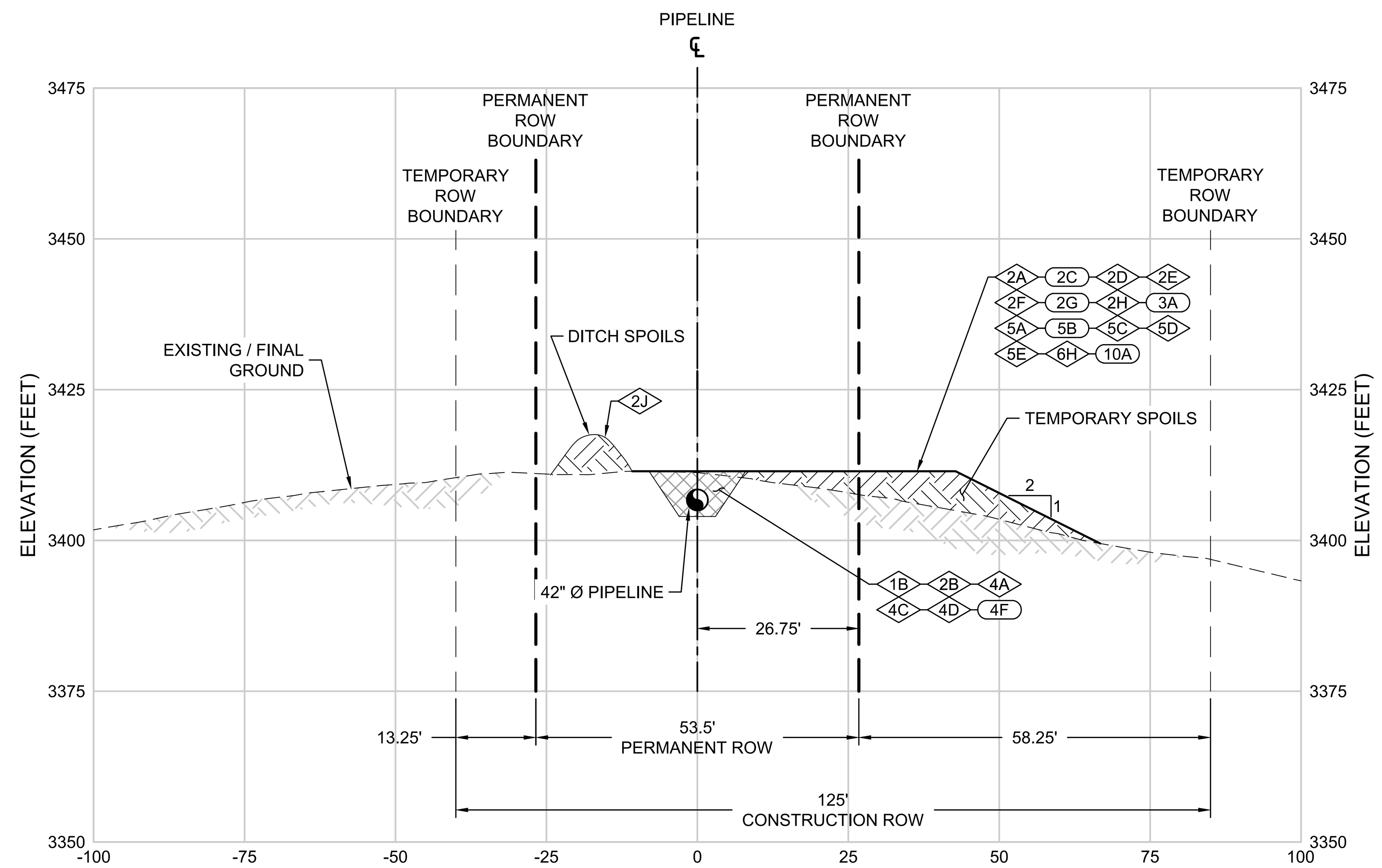
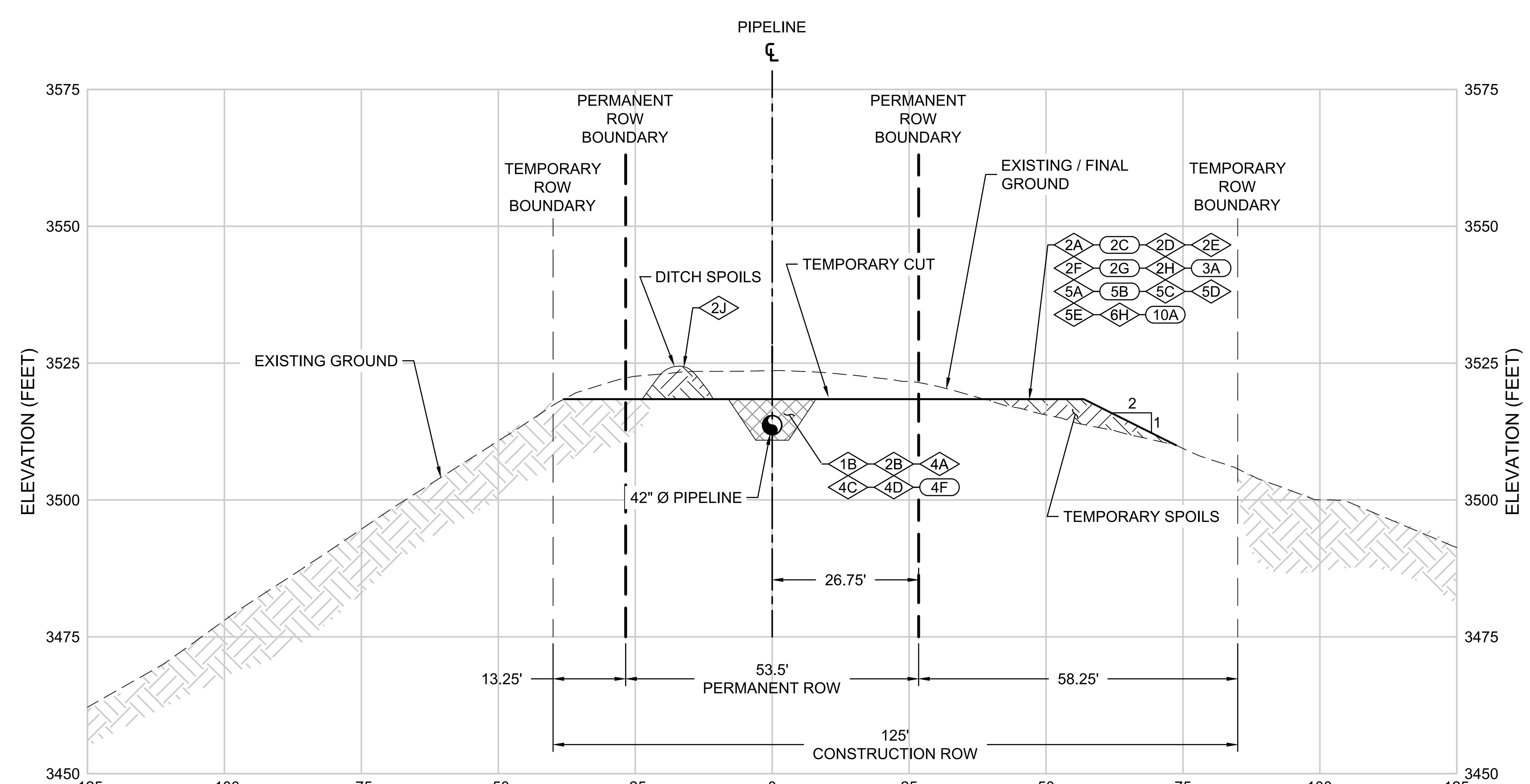
APPROVED BY:	TR		



SECTION B - B' (STA 4425+00)
DISTANCE (FEET)



SECTION C - C' (STA 4428+50)
DISTANCE (FEET)

BEST IN CLASS (BIC) INCREMENTAL CONTROLS

- 1B ENHANCED DRAIN (GERMAN DRAIN)
- 2A GRADING TEMPORARY ROW SURFACE
- 2B GRADING TRENCH WITH OUTBOARD WEDGE
- 2C COMPACT BACKFILL
- 2D DRY SOILS AND BACKFILL
- 2E REMOVE UNSUITABLE EXISTING SOILS AS BACKFILL
- 2F ROCK BACKFILL (WITH DRAIN)
- 2G GRADING TO MATCH EXISTING CONTOURS
- 2H GRADING TO MINIMIZE BACKFILL
- 2J SPOILS MANAGEMENT
- 3A TRACK DISTURBED SLOPES
- 4A TRENCH BREAKERS (FOAM AND SANDBAGS), MODIFIED SPACING
- 4C SACK-CRETE BREAKERS (STRUCTURAL BREAKER)
- 4D SLEEVE INTERFACE BETWEEN PIPELINE AND BREAKER
- 4F TRENCH BREAKER WITH DRAINAGE
- 5A SLOPE BREAKERS (TEMP AND PERMANENT), MODIFIED SPACING
- 5B SLOPE BREAKER ARMORED OUTLET
- 5C SLOPE BREAKERS WITH DIVERSION CHANNELS
- 5D ACCESS ROADS
- 5E TEMPORARY SLOPE BREAKER WITH DRAIN PIPE
- 6H TYP SURFACE WATER CONTROL LAYOUT
- 10A BENCH RE-CONSTRUCTION THROUGH NATURAL STEPS

NOTES:

1. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
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0 20 40
SCALE IN FEET

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HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
SECTIONS B-B' AND C-C'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN
MP 73.20 TO 73.50 (AP-1)

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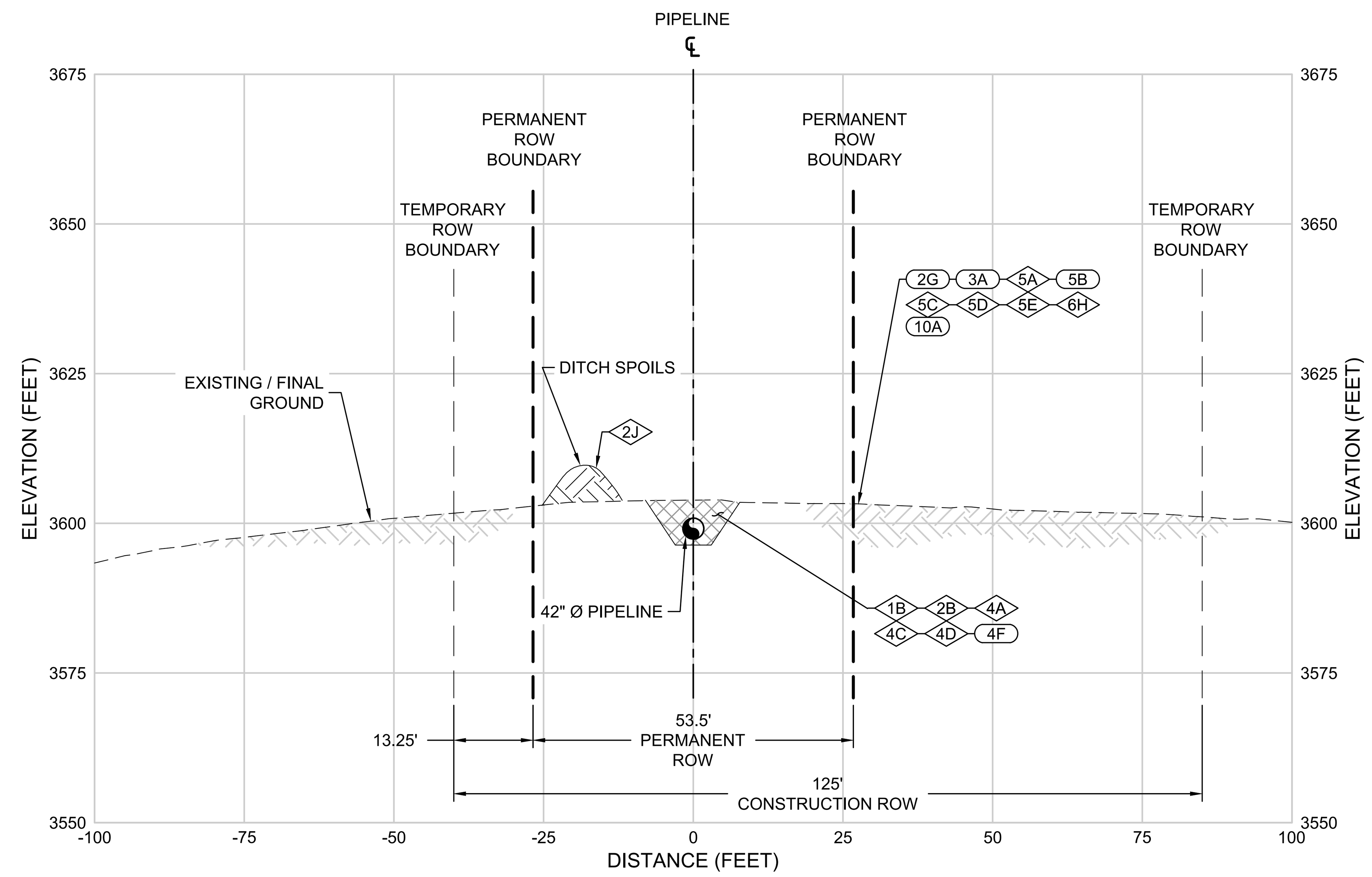

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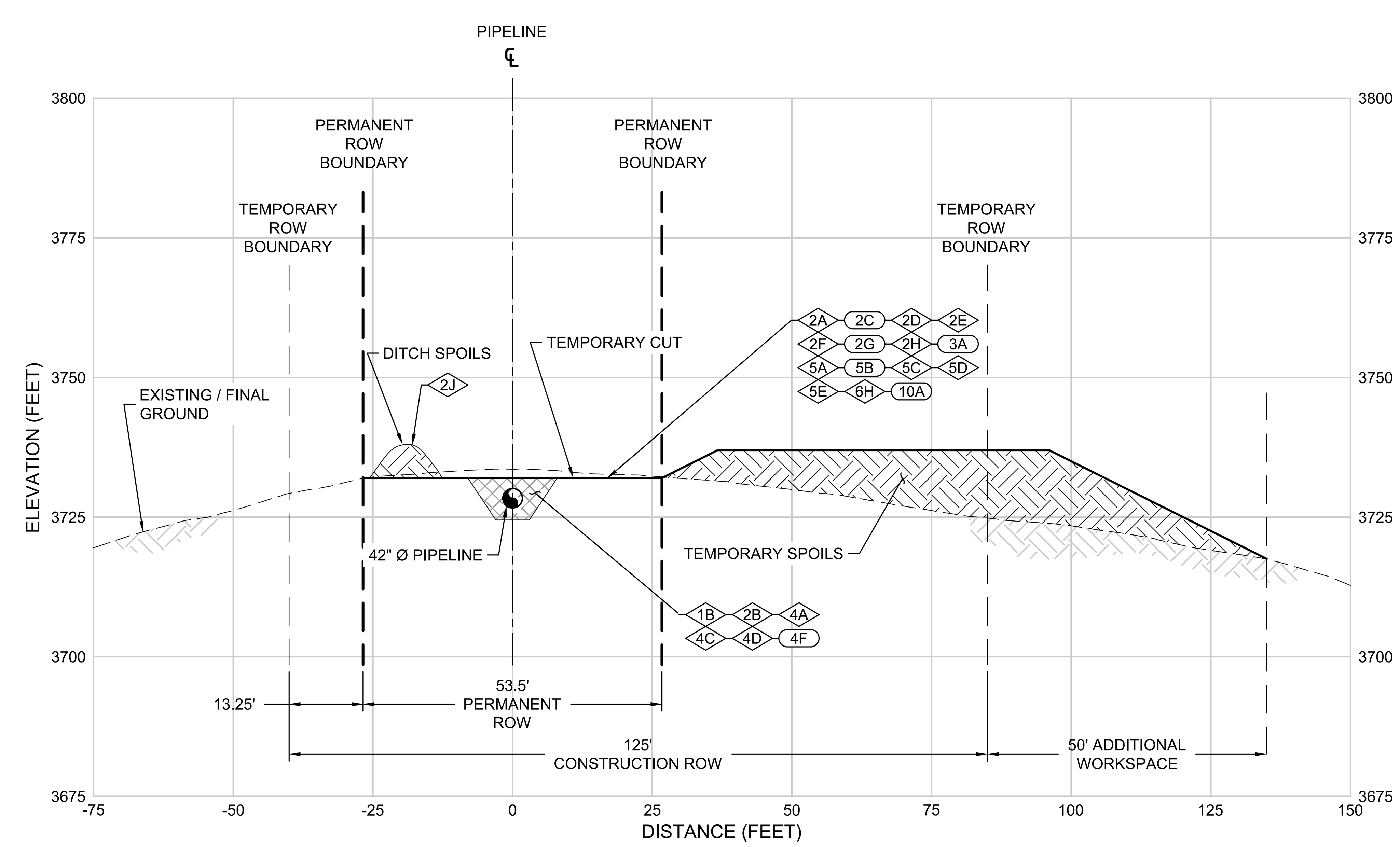

SECTION E - E' (STA 4435+50)

BEST IN CLASS (BIC) INCREMENTAL CONTROLS

- 1B ENHANCED DRAIN (GERMAN DRAIN)
- 2A GRADING TEMPORARY ROW SURFACE
- 2B GRADING TRENCH WITH OUTBOARD WEDGE
- 2C COMPACT BACKFILL
- 2D DRY SOILS AND BACKFILL
- 2E REMOVE UNSUITABLE EXISTING SOILS AS BACKFILL
- 2F ROCK BACKFILL (WITH DRAIN)
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HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
SECTIONS D-D' AND E-E'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN
MP 73.20 TO 73.50 (AP-1)

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REVIEWED BY:	RS	DRAWING NO.:	4 OF 5
APPROVED BY:	TR		

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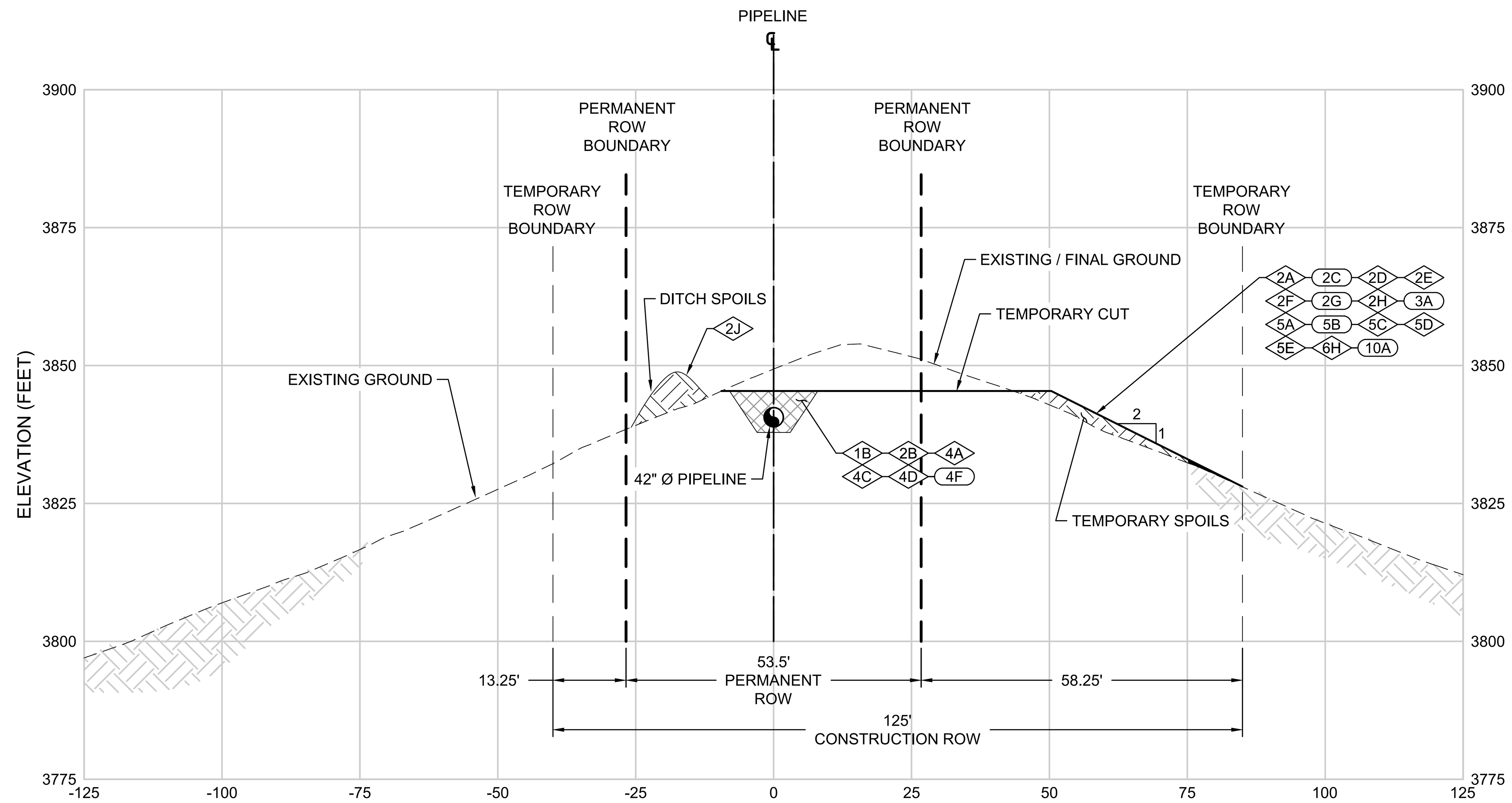
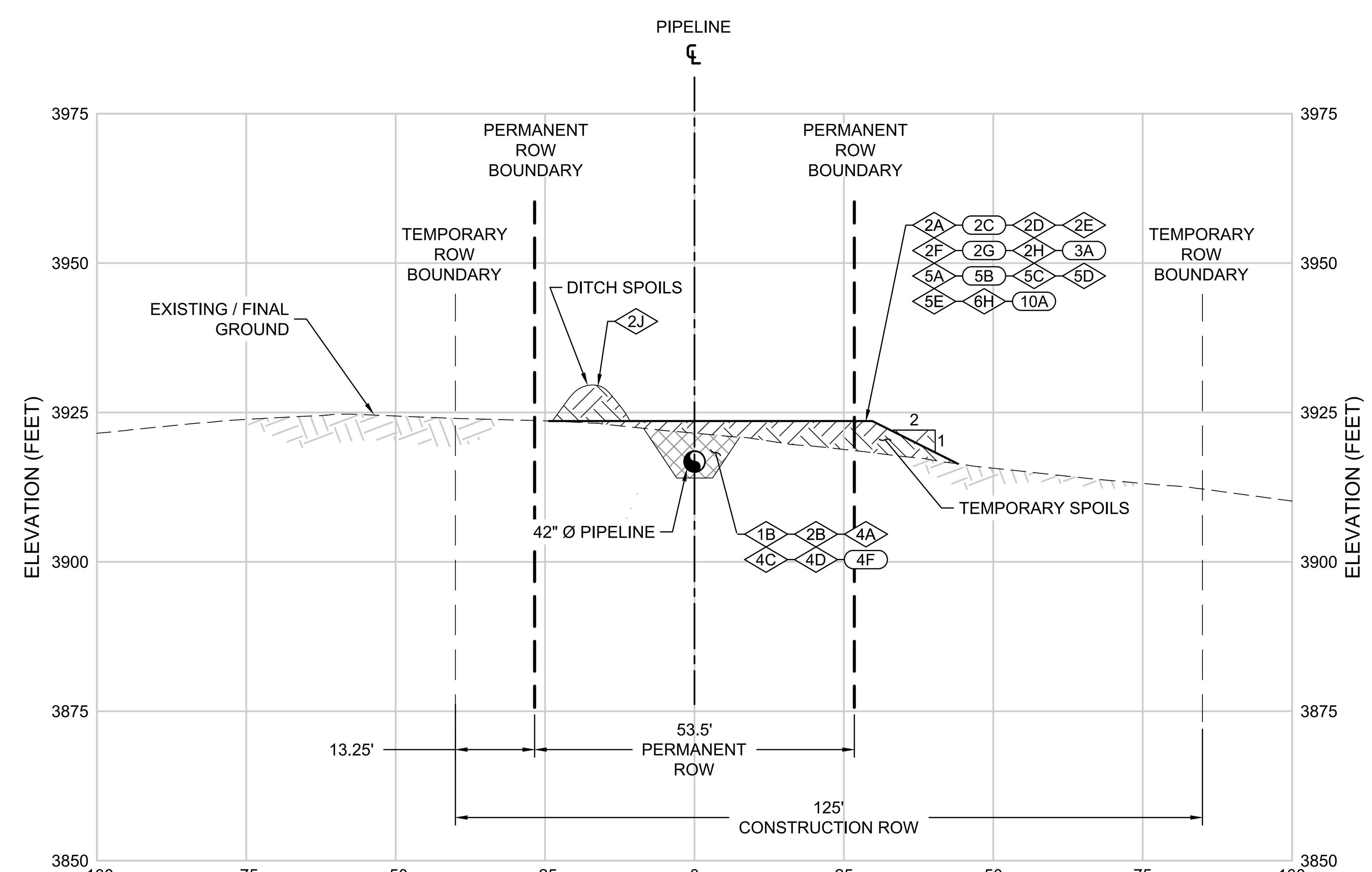


SECTION F - F' (STA 4439+00)
DISTANCE (FEET)



SECTION G - G' (STA 4442+50)
DISTANCE (FEET)

BEST IN CLASS (BIC) INCREMENTAL CONTROLS

1B	ENHANCED DRAIN (GERMAN DRAIN)
2A	GRADING TEMPORARY ROW SURFACE
2B	GRADING TRENCH WITH OUTBOARD WEDGE
2C	COMPACT BACKFILL
2D	DRY SOILS AND BACKFILL
2E	REMOVE UNSUITABLE EXISTING SOILS AS BACKFILL
2F	ROCK BACKFILL (WITH DRAIN)
2G	GRADING TO MATCH EXISTING CONTOURS
2H	GRADING TO MINIMIZE BACKFILL
2J	SPOILS MANAGEMENT
3A	TRACK DISTURBED SLOPES
4A	TRENCH BREAKERS (FOAM AND SANDBAGS), MODIFIED SPACING
4C	SACK-CRETE BREAKERS (STRUCTURAL BREAKER)
4D	SLEEVE INTERFACE BETWEEN PIPELINE AND BREAKER
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5C	SLOPE BREAKERS WITH DIVERSION CHANNELS
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TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN SECTIONS F-F' AND G-G'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN MP 73.20 TO 73.50 (AP-1)

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APPROVED BY:	TR		5 OF 5

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **ACP** Project: **ACP Geohazard Analysis Program** Project/ Proposal No.: **TXG0007** Task No. **013**

Title of Computations: Geohazard Mitigation Design of the Slope at ACP AP-1 MP 84.95 to 85.05

Computations by:	Signature		11/10/2016 (Date)
	Printed Name	Mustafa Erten, Ph.D., P.E. (TX)	
	Title	Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		11/18/2016 (Date)
	Printed Name	Rodolfo Sancio, Ph.D., P.E. (TX)	
	Title	Senior Principal	
Computations Checked by:	Signature		11/18/2016 (Date)
	Printed Name	Rodolfo Sancio, Ph.D., P.E. (TX)	
	Title	Senior Principal	
Computations backchecked by: (originator)	Signature		12/22/2016 (Date)
	Printed Name	Mustafa Erten, Ph.D., P.E. (TX)	
	Title	Engineer	
Approved by: (pm or designate)	Signature		12/22/2016 (Date)
	Printed Name	Logan Brant, Ph.D., P.E. (VA)	
	Title	Senior Engineer	

Approval notes:

Revision History:

No.	Description	Date	By	Checked by	Approval

Written by:	Mustafa Erten	Date:	11/10/2016	Reviewed by:	Rodolfo Sancio	Date:	11/18/2016
Client:	ACP	Project:	Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05	Project No.:	TXG0007	Task No.:	013

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LIST OF APPENDICES

Written by:	Mustafa Erten	Date:	11/10/2016	Reviewed by:	Rodolfo Sancio	Date:	11/18/2016
Client:	ACP	Project:	Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05	Project No.:	TXG0007	Task No.:	013

INTRODUCTION

Geosyntec Consultants (Geosyntec) prepared this calculation package to present the soil nail system design and the results of slope stability assessment performed for the site-specific geohazard mitigation of slope at MP 84.95 to 85.05 on the Atlantic Coast Pipeline (ACP) Segment AP-1 alignment. The slope is located in the George Washington National Forest in Highland County, Virginia.

The purpose of this calculation package is to evaluate the proposed geohazard mitigation including a slope stabilization system with soil nails.

This calculation package is organized to present: (i) methodology; (ii) cross sections; (iii) subsurface stratigraphy and geotechnical parameters; (iv) analysis results; and (v) summary and recommendation.

METHODOLOGY

The soil nails were designed for the steep slope area above the stream using the online dimensioning tool named ROVULUM®, which was developed by Geobrugg [GeoBrugg, 2016]. As mentioned in the user's manual, this tool is developed to design the slope stabilization system consisting of high tensile steel wire mesh, bearing plate, and nail rebars [GeoBrugg, 2016]. The approach that is implemented in this dimensioning tool is recommended for potential slippage of shallow soils up to approximately 6-ft depth. Geosyntec considered this methodology to be applicable for this site.

The stability of slopes above the steep slope area on the pipeline alignment at ACP AP-1 MP 84.95 to MP 85.05 was analyzed using the infinite slope approach considering effective stresses (also known as a drained analysis). In this approach, the slope is assumed to extend infinitely and the slip surface is parallel to the slope surface [Duncan and Wright, 2005]. The infinite slope approach is considered appropriate at this location because the thickness of the potentially unstable materials is small compared to the longitudinal dimension of the slope. For a cohesionless material (i.e., c' = 0) and zero pore pressures, the factor of safety against sliding (FS) is independent of the depth of soil and calculated by the following equation:

$$FS = \frac{\tan \phi'}{\tan \beta} \qquad \text{Equation 1}$$

where ϕ' is the soil friction angle and β is the slope inclination.

CROSS SECTIONS

Figure 1 is a plan of the slope showing pipeline alignment and the selected profile (Section D-D') that was used by Geosyntec for slope stability evaluation. Figure 2 shows the elevation profile along the pipeline path (Section D-D') and the location of the test pits made during the Order 1 Soil Survey near the area of interest [RETTEW and Geosyntec, 2016]. This profile section was selected to capture the maximum slope inclination near the proposed pipeline alignment and it is oriented approximately in the direction of the steepest slope above the stream.

SUBSURFACE STRATIGRAPHY AND GEOTECHNICAL PARAMETERS

The subsurface stratigraphy was developed using the information collected from the test pits described in Appendix A. The subsurface stratigraphy is summarized in Table 1.

Table 1. Subsurface Stratigraphy Along the Profile Section

Distance in Figure 2 (ft)	Depth (ft)	Material Type
0 - 650	0 - 2	Sandy/Gravelly Silt (ML)
	> 2	Bedrock (Sandstone/Shale)
650 - 1000	0 - 1.2	Silty Gravel (GM)
	> 1.2	Bedrock (Sandstone/Shale)

Table 2 lists the geotechnical parameters that were used in the slope stability analysis. The development of geotechnical parameters is discussed in Appendix A.

Written by:	**Mustafa Erten**	Date:	**11/10/2016**	Reviewed by:	**Rodolfo Sancio**	Date:	**11/18/2016**
Client:	**ACP**	Project:	**Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05**	Project No.:	**TXG0007**	Task No.:	**013**

Table 2. Selected Geotechnical Parameters

Soil Type	USCS	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (°)
Silts and Sandy / Gravelly Silt	ML	110	0	32
Silty Gravel	GM	130	0	40
Bedrock	-	150	Infinite Strength	

ANALYSIS RESULTS

Soil Nail System Design

Table 3 shows the input parameters that are implemented in soil nail dimensioning tool named ROVULUM®. The properties of the designed soil nail system are given in Table 4. Two sets of soil nail arrangements are presented in Table 4. The first set (8 ft by 8 ft spacing) is for the general area inside the limits of the permanent right of way (ROW) along the pipeline alignment. Since the soil nails cannot be installed within the footprint of the pipeline trench, which is approximately 16-ft wide, a second soil nail arrangement plan is designed. In the second plan, the density of the soil nails increased vertically on the slope (16-ft spacing) and decreased horizontally (4-ft spacing). The minimum soil layer thickness that could be entered into the dimensioning tool was 3-ft. Therefore, the soil nails were designed conservatively for 3-ft thick soil instead of 2-ft. The design parameters and recommended design details calculated by the dimensioning tool is given in Appendix B for both soil nail arrangements.

For the both soil nail arrangements, the mobilized tensile force in each soil nail was calculated to be 13.3 kips. This load is assumed to be carried by the bond between the bedrock and the grout around the nail. According to the Soil Nail Walls Reference Manual by FHWA [2015], the estimated bond strength between the weathered shale and grout using the rotary drilling method is a minimum of 15 psi. For a 6-inch hole, the bond strength would correspond to a capacity of 2.2 kips per lineal feet after applying a bond strength reduction factor of 0.65. Therefore, in order to achieve the minimum bond strength, a minimum 6.0-ft nail embedment into bedrock is required. Including the 2-ft thick soil layer above the bedrock, the total length for the soil nails is 8 ft.

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Client:	ACP	Project:	Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05	Project No.:	TXG0007	Task No.:	013

Table 3. Input Parameters for ROVULUM®

Soil Thickness:	3 ft
Soil Parameters:	c’ = 0 psf, φ’ = 32°
Slope Inclination:	50°
Pre-tension Force:	6.7 kips

Table 4. Soil Nail System Specification and Arrangements

	General	**Adjacent to Pipeline Trench**
Nail Type (Diameter):	GEWI D (32 mm)	GEWI D (32 mm)
Bearing Plate:	Spike Plate P66	Spike Plate P66
Mesh Type:	TECCO G65/4	TECCO G65/4
Distance Between Each Nail:	8 ft	4 ft
Out-of-Plane Spacing	8 ft	15 ft
Nail Angle from Horizontal:	30°	30°
Hole Diameter:	6 inch	6 inch
Mobilized Tensile Force in Each Nail:	13.3 lbs	13.3 lbs

Slope Stability for the Upslope Area

The average slope inclination along the pipeline profile section above the steep slope area is 60% (31°) (Figure 2). The infinite slope analysis using Equation 1 on this cross section using a friction angle (ϕ') of 32° and 40° with zero pore pressures results in FS between 1.04 and 1.40, respectively in ML and GM types of materials. Since FS is larger than 1.0, no engineered site specific geohazard mitigation at the upslope area is recommended.

SUMMARY AND RECOMMENDATION

This calculation package presents the results of analyses for geohazard mitigation of the steep slope area near ACP AP-1 MP 84.95 to 85.05. The summary of analyses results are presented below.

The soil nail arrangement within the general permanent limits of ROW and within the footprint of the trench area is summarized in Table 4. The length of soil nail was proposed to be 8-ft long, except adjacent to the pipeline trench.

- In the general area inside the limits of the permanent right of way (ROW), the soil nail with 8-ft by 8-ft spacing is proposed.
- In the area within the footprint of the pipeline trench, the soil nail with 15-ft by 4-ft spacing is proposed.

REFERENCES

Duncan, J. M. and Wright, S. G. [2005]. Soil strength and slope stability. 1st Edition. John Wiley & Sons.

GeoBrugg [2016], "Dimensioning tool for the TECCO / SPIDER slope and rock stabilization system Version 2016 – Software Manual".

RETTEW and Geosyntec Consultants [2016]. "Order 1 Soil Survey Atlantic Coast Pipeline Monongahela National Forest, WV and George Washington National Forest, VA", Submitted on August 1, 2016.

U.S. Department of Transportation Federal Highway Administration (FHWA) [2015], "Geotechnical Engineering Circular No. 7: Soil Nail Walls - Reference Manual", FHWA-NHI-14-007

Written by:	**Mustafa Erten**	Date:	**11/10/2016**	Reviewed by:	**Rodolfo Sancio**	Date:	**11/18/2016**

Client:	**ACP**	Project:	**Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05**	Project No.:	**TXG0007**	Task No.:	**013**

FIGURES

Written by:	**Mustafa Erten**	Date:	**11/10/2016**	Reviewed by:	**Rodolfo Sancio**	Date:	**11/18/2016**
Client:	**ACP**	Project:	**Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05**	Project No.:	**TXG0007**	Task No.:	**013**

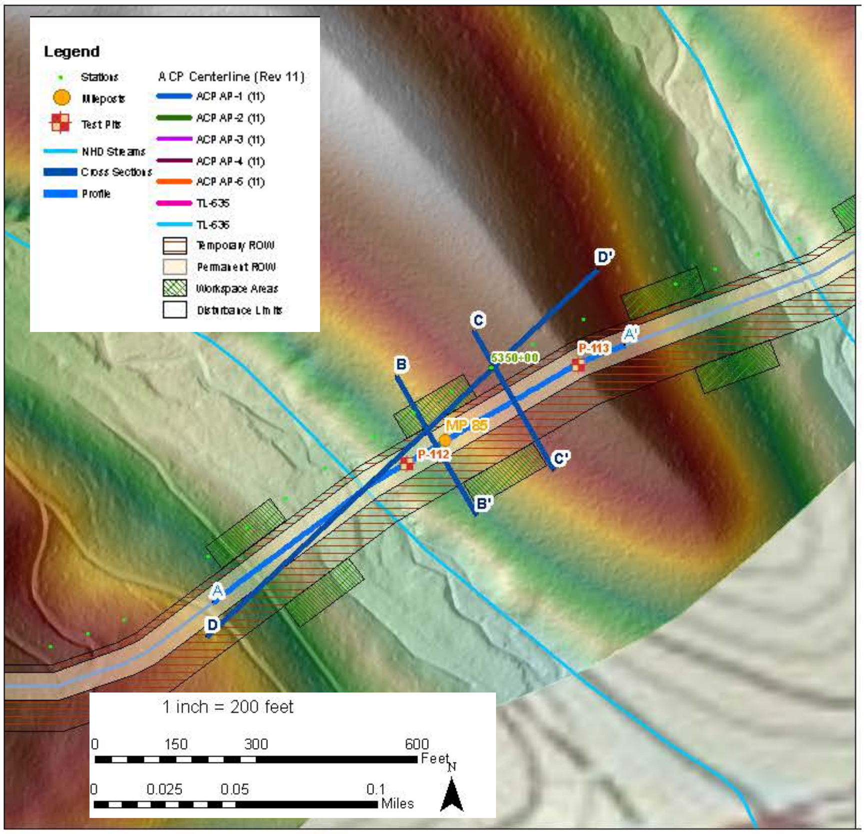

Figure 1. Location of Cross Sections and Profile Section

Written by:	Mustafa Erten	Date:	11/10/2016	Reviewed by:	Rodolfo Sancio	Date:	11/18/2016

Client:	ACP	Project:	Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05	Project No.:	TXG0007	Task No.:	013

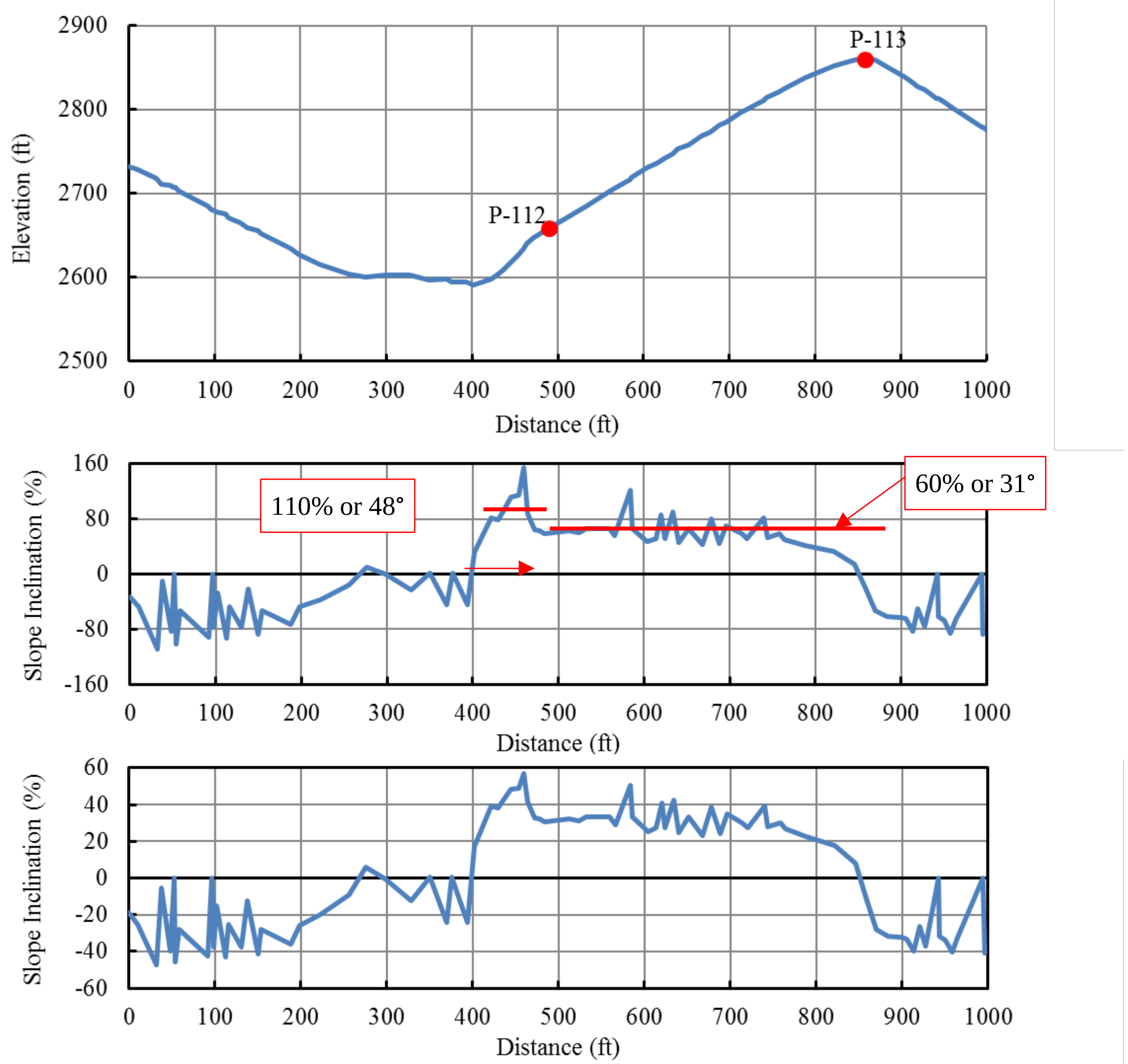

Figure 2. Elevation and Slope Inclination Profiles for Section D-D'

Written by:	**Mustafa Erten**	Date:	**11/10/2016**	Reviewed by:	**Rodolfo Sancio**	Date:	**11/18/2016**
Client:	**ACP**	Project:	**Geohazard Mitigation Design at ACP AP-1 MP 84.95 to 85.05**	Project No.:	**TXG0007**	Task No.:	**013**

Appendix A

Geotechnical and Topographical Inputs for Site-Specific Geohazard Mitigation Design at ACP AP-1 MP 84.95 to MP 85.05

APPENDIX A - GEOTECHNICAL AND TOPOGRAPHICAL INPUTS FOR SITE-SPECIFIC GEOHAZARD MITIGATION DESIGN AT ACP AP-1 MP 84.95 TO MP 85.05

INTRODUCTION

This document summarizes and interprets the available geotechnical and topographical information for use as inputs in the site-specific geohazard mitigation design for pipeline construction and right-of-way restoration on the slope at the Atlantic Coast Pipeline (ACP) Segment AP-1 Milepost (MP) 84.95 to MP 85.05. This steep slope area is located in the George Washington National Forest in Highland County, Virginia.

SURFACE TOPOGRAPHY

The slope at ACP AP-1 MP 84.95 to MP 85.05 rises approximately 250 ft in elevation over a horizontal distance of about 500 ft. A stream identified as “UNT to Townsend Draft” is adjacent to the toe of the slope. Above the stream, the slope projects upward for about 50 ft vertically at a slope inclination in the range of 80% (39º) to 130% (52º). Beyond that initial steep section, the slope flattens slightly and continues to rise relatively uniformly until reaching the crest of the slope with an average inclination of approximately 60% (31º). At the slope’s crest, there is a narrow ridge. On the opposite side of the ridge, another slope continues down the other side.

The proposed pipeline alignment is oriented approximately parallel to the direction of slope. Figure 1 shows the elevation profile of the area of interest.

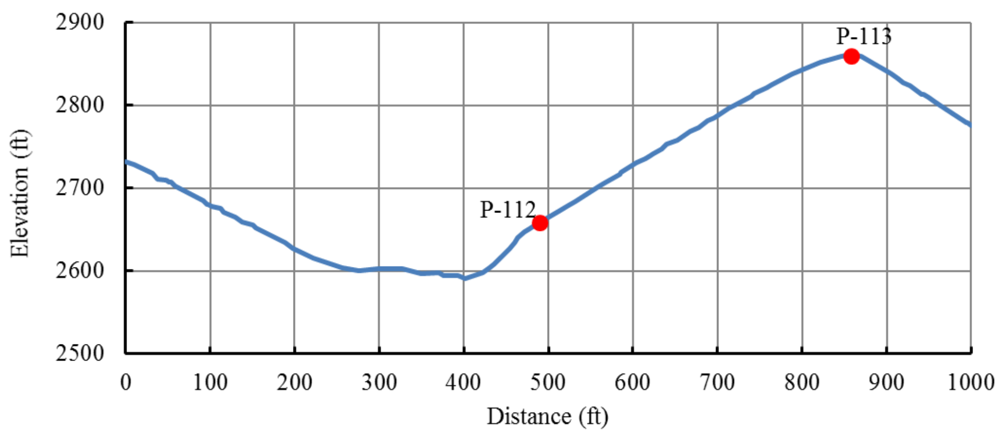

Figure 1. Elevation Profile

SUBSURFACE CONDITIONS

Available information on the subsurface conditions at the site is largely based on field observation and laboratory testing related to the Order 1 Soil Survey conducted by Geosyntec Consultants and their subcontractors. For the soil survey, two test pits were excavated in the vicinity of this slope that is the interest of this assessment [Rettew and Geosyntec, 2016]. Test Pit P-112 was located just above the steepest segment near the toe, in an area with a slope inclination of 62%. Test Pit P-113 was located on the narrow ridge at the crest of the slope.

Soil

The soil profiles at both test pit locations were logged by soil scientists using the classification system in Soil Survey Manual by U.S. Department of Agriculture [1993]. The soil profiles at all test pits were identified as predominantly silt loam with various proportions of rock fragments. The rock fragment content increased from 10 to 90 percent with depth. In this classification system, rock fragments are defined as any soil particle larger than 2 mm in diameter (the coarse/medium sand threshold used by geotechnical engineers).

Table 1 summarizes the USDA Soil Classifications and the percentage of the rock fragments of the soils in each test pit.

Table 1. USDA Soil Classification of Test Pits at ACP AP-1 MP 85

Test Pit ID	USDA Soil Name	USDA Map Symbol	Rock Fragments	Depth to Bedrock (ft)
P-112	Weikert-Berks-Rough Complex	55G	10%-85%	2.0
P-113	Weikert-Berks-Rough Complex	55G	40%-90%	1.2

The review of USDA's database indicates that the 55G type of Weikert-Berks-Rough complex are mapped on the steep slope areas where the slope inclination was in the range of 55% and 80%. Weikert-Berks-Rough complex is composed of silt or silty clay loam with significant gravel content. According to the Unified Soil Classification System (USCS), similar to CL, ML, SC, SM, GC, GM. For this soil type, typically the liquid limit (LL) values vary between 20 and 25, on average. The plasticity index (PI) values vary between 4 and 8, on average.

The same two test pits were also logged by a geotechnical engineer to record soil descriptions for engineering purposes. These descriptions were prepared in accordance with ASTM D2488. Group symbols based on the USCS were also developed for each soil. The geotechnical engineering description of the soil was gravelly silt (ML) at Test Pit P-112 and was silty gravel (GM) at Test Pit P-113.

Table 2 summarizes the Geotechnical Engineering Soil Descriptions in each test pit.

Table 2. Geotechnical Engineering Soil Descriptions of Test Pits

Test Pit ID	**Geotechnical Engineering Soil Description**	**USCS Group**
P-112	Gravelly Silt	ML
P-113	Silty Gravel	GM

Geotechnical laboratory testing was not conducted on soil samples recovered from these specific test pits (P-112 and P-113). However, geotechnical laboratory index and classification testing was performed on soil samples collected from ten other test pits within the George Washington and Monongahela National Forests (one sample from each test pit as listed in Table 3). The index properties of the soil samples are generally consistent. where fines contents are at or near 50% and LL values generally are in the range of 33 to 40, with one sample at 52. The PI values varied between 7 and 17. The fine-grained portion (i.e., portion smaller than 0.75 mm) of eight out of 10 samples were classified as low plasticity silt (ML), and are considered to be similar to the fines portions of the soils encountered in the test pits at P-112 and P-113. The laboratory test results for those ten samples are shown in Table 3.

Table 3. Laboratory Test Results from Test Pits for Order 1 Soil Survey Program

Test Pit ID	USCS	USCS for Fines	Moisture Content (%)	Fines Content (%)	LL (%)	PL (%)	PI (%)
P-047	ML	ML	14.9	59.4	34	25	9
P-091	GM	ML	14.4	42.2	NP	NP	NP
P-092	ML	ML	22.9	55.1	38	26	12
P-115	GM	ML	12.3	41.7	33	26	7
P-217	SM	MH	21.8	42.9	52	35	17
P-226	ML	ML	19.8	58.8	34	24	10
P-244	GM	ML	17.1	43.5	36	26	10
P-275	SM	ML	16.4	44.1	40	26	14
P-300	ML	ML	23.2	53.7	35	27	8
P-314	GC	CL	17.7	48.2	33	23	10

Bedrock

The site is underlain by the Chemung Formation (redefined as Foreknobs Formation) which is composed of shale and sandstone with a few thin, quartz-pebble conglomerates and red-beds. The test pit logs by soil scientists also confirm that the bedrock type that was encountered at the site was sedimentary rocks (usually sandstone or siltstone).

Bedrock was encountered at relatively shallow depths in both test pits, 2.0 ft bgs at Test Pit P-112 and 1.2 ft bgs at Test Pit P-113, respectively. The bedrock at these two test pits are sedimentary rock, including sandstone and shale. At Test Pit P-112, where the ground surface inclination was 62%, the bedrock dipped 6° into the slope. At Test Pit P-113, the bedrock was aligned with the shallow ground surface slope at the ridge top with a dip of 6°. Table 4 summarizes the bedrock observations in both test pits.

Table 4. Bedrock Observations in Test Pits

Test Pit ID	Bedrock Type	Bedrock Depth (ft)	Bedding Plane Dip	Bedding Plane Strike
P-112	Sandstone / Shale	2.0	6° N	N 37° W
P-113	Sandstone / Shale	1.2	6° S	S 71° E

Groundwater

Groundwater table (GWT) was not observed at test pits P-112 and P-113. Surface water was observed in the stream adjacent to the toe of the slope.

RECOMMENDED DESIGN GEOTECHNICAL PARAMETERS

Geosyntec has estimated site-specific design parameters to support the geohazard mitigation design of the slope at ACP AP-1 MP 84.95 to MP 85.05.

Soil

The soil observed in the test pits are typically the product of in situ weathering of the parent rock (i.e., residual soil). These soils may therefore retain some cohesion. Additionally, they are partially saturated, thus exhibit apparent cohesion caused by interstitial pore water tension. Moreover, at shallow depths (e.g., < 2 ft), they also exhibit apparent cohesion caused by the root mat of deciduous trees, shrubs, and grasses. Upon saturation, however, the apparent cohesion caused by interstitial tension is likely to decrease or disappear. Also, the removal of vegetation to establish the right of way will decrease the effect of the root mat. The selection of parameters for slope stability evaluation should thus consider these effects.

Stark et al. [2013] provides relationships to estimate the drained secant friction angle of fine grained soils as a function of the clay fraction, effective confining pressure (σ_n'), and ball-milled derived LL values for the slope stability calculations. Using the average LL value of 36 for the ML types of soils as encountered at this site, the corresponding ball-milled derived LL value was calculated as 48 using the relationship suggested in Stark et al. [2013].

Using Figure 2 (adopted from Stark et al., 2013), the drained secant friction angle for ML was estimated to be 32 degrees for fully softened condition for clay content (CF) less than 20%, and effective normal stress of 1044 psf (50 kPa). As discussed by Stark et al. [2013], the selection of fully softened shear strength parameters would be proper for overconsolidated soils; however, they may be conservative for first times slides, for which a cohesion term is appropriate.

Lambe and Whitman (1969) indicated that for the preliminary design, the drained friction angle of dense gravel would be in the range of 40° and 48°. A friction angle of 40° with 0 psf cohesion was selected as drained shear strength parameters for the silty gravel (GM).

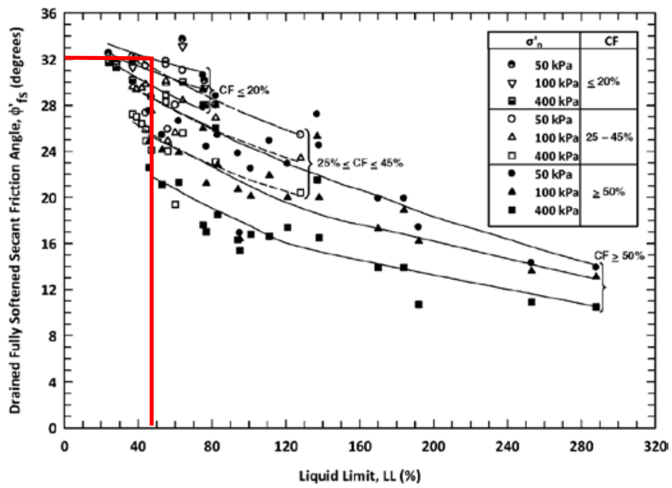

Figure 2. Empirical Correlation for Drained Fully Softened Secant Friction Angle Based on LL, CF, and σ'_n (Adapted from Stark et al. (2013).

Since the bedrock strength is not believed to control the minimum factor of safety against slope stability, infinite strength was assigned for the bedrock. The total unit weight for bedrock was estimated for a typical sedimentary rock.

Table 5 shows the assumed unit weight and shear strength parameters for soils. The unit weight values selected are upper bound typical values for ML and GM soils above groundwater table, as given in Coduto [2001].

Bedrock

Since the bedrock strength is not believed to control the minimum factor of safety against slope stability, infinite strength was assigned for the bedrock. The total unit weight for bedrock was estimated for a typical sedimentary rock.

Table 5. Selected Soil Design Parameters

Soil Type	USCS	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (°)
Silts and Sandy / Gravelly Silt	ML	110	0	32
Silty Gravel	GM	130	0	40
Bedrock	-	150	Infinite Strength	

Soil Nail Bond Strength

The ultimate bond strength between the grouted soil nail and the bedrock (weathered shale) was estimated to be 15 psi, based on recommendation in FHWA [2015]. Using a bond strength reduction factor of 0.65 Per FHWA [2015], the bond strength for the soil nail in the bedrock was calculated to be 2200 lbs per lineal ft for a 6-inch diameter soil nail hole.

Groundwater

Based on the available information, Geosyntec assumed the ground water at top of the bedrock below the soil layer for the purpose of geotechnical analyses. Ground water level can potentially fluctuate due to seasonal change and periodic precipitations. However, given the enhanced drainage measures that will be implemented on steep slopes through implementation of the Best in Class (BIC) program, we consider appropriate to assume a condition of partial saturation in the soil profile.

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Lambe T. W. and Whitmann R. V. [1969]. "Soil Mechanics," First Edition, John Wiley and Sons, Inc., 553p.

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Appendix B

Soil Nail System Design

Dimensioning of the slope stabilization system TECCO® / SPIDER® by the RUVOLUM® method

Project No.	**TXG0007**
Project Name	**ACP AP-1 MP85 - 16x4ft_Trench Area**
Date, Author	**Geosyntec - 10 Nov 2016**

Input quantities

Slope inclination	α	=	50.0 degrees
Layer thickness	t	=	3.00 ft
Friction angle ground (characteristic value)	Φ_k	=	32.0 degrees
Volume weight ground (characteristic value)	Υ_k	=	110.0 lbs/ft^3
Nail inclination to horizontal	Ψ	=	30.0 degrees
Nail distance horizontal	a	=	16.00 ft
Nail distance in line of slope	b	=	4.00 ft

Load cases

Streaming pressure considered			No
Earthquake considered			No
Coefficient of horizontal acceleration due to earthquake	ε_h	=	0.000 [-]
Coefficient of vertical acceleration due to earthquake	ε_v	=	0.000 [-]

Defaults and Safety Factors

Cohesion ground (characteristic value)	c_k	=	0.0 lbs/ft^2
Radius of pressure cone, top	ζ	=	6.0 in
Inclination of pressure cone to horizontal	δ	=	45.0 degrees
Slope-parallel force	Z_d	=	9.9 kips
Pretensioning force of the system	V	=	6.7 kips
Partial safety correction value for friction angle	Υ_Φ	=	1.25 [-]
Partial safety correction value for cohesion	Υ_c	=	1.25 [-]
Partial safety correction value for volume weight	Υ_Υ	=	1.00 [-]
Model uncertainty correction value	Υ_{mod}	=	1.10 [-]

Dimensioning quantities

Φ_d [degrees] = 26.6

c_d [lbs/ft^2] = 0.0

Υ_d [lbs/ft^3] = 110.0

Elements of the system

Applied mesh type	TECCO® G65/4		
Applied spike plate	TECCO® system spike plate P66		
Bearing resistance of mesh to selective, slope parallel tensile stress	Z_R [kips]	=	16.9
Bearing resistance of mesh to pressure stress in nail direction	D_R [kips]	=	83.2
Bearing resistance of mesh against shearing-off in nail direction	P_R [kips]	=	41.6
Applied nail type	GEWI D = 32 mm		
Taking into account rusting away	Yes		
Bearing resistance of nail to tensile stress	T_{Rred} [kips]	=	69.3
Bearing resistance of nail to shear stress	S_{Rred} [kips]	=	40.0
Cross-section surface of the applied nail with / without rusting away	A_{red} [²]	=	0.956

Proofs

Proof of the mesh against shearing-off at the upslope edge of the spike plate	Fulfilled
Proof of the mesh to selective transmission of the force Z onto the nail	Fulfilled
Proof of the nail against sliding-off of a superficial layer parallel to the slope	Fulfilled
Proof of the mesh against puncturing	Fulfilled
Proof of the nail to combined stress	Fulfilled

The given proofs concern the investigation of superficial instabilities. Additional investigations are required if there is a risk regarding global stability of the slope. If necessary the nail type and nail pattern have to be adapted.

Investigation of local instabilities between single nails

Proof of the mesh against shearing-off at the upslope edge of the spike plate

Maximum stress on the mesh for shearing-off in nail direction at the upslope edge of the spike plate (dimensioning level).	P_d [kips] =	0.0
Thickness of decisive sliding mechanism	t_{rel} [ft] =	1.95
Bearing resistance of the mesh against shearing-off in nail direction at the upslope edge of the spike plate (characteristic value).	P_R [kips] =	41.6
Resistance correction value for shearing-off of the mesh	Υ_{PR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh against shearing-off	P_R/Υ_{PR} [kips] =	27.7
Proof of bearing safety	$P_d \Leftarrow P_R/\Upsilon_{PR}$	Fulfilled

Proof of the mesh to selective transmission of the force Z onto the nail

Slope parallel force taken into account in the equilibrium considerations	Z_d [kips] =	10.1
Bearing resistance of the mesh to selective, slope-parallel tensile stress	Z_R [kips] =	16.9
Resistance correction value for selective, slope-parallel transmission of the force Z	Υ_{ZR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh to tensile stress	Z_R/Υ_{ZR} [kips] =	11.2
Proof of bearing safety	$Z_d \Leftarrow Z_R/\Upsilon_{ZR}$	Fulfilled

Investigation of slope-parallel, superficial instabilities

Proof of the nail against sliding-off of a superficial layer parallel to the slope

Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for positive influence of pretension V	Υ_{VI} [-] =	0.8
Dimensioning value of the applied pretensioning force by positive influence of V	V_{dI} [kips]=	5.4
Calculatorily required shear force at dimensioning level in function of V_{dI}	S_d [kips] =	6.7
Bearing resistance of the nail to shear stress	S_{Rred} [kips] =	40.0
Resistance correction value for shearing-off of the nail	Υ_{SR} [-] =	1.5
Dimensioning value of the bearing resistance of the nail to shear stress	S_{Rred}/Υ_{SR} [kips] =	26.7
Proof of bearing safety	$S_d \Leftarrow S_{Rred}/\Upsilon_{SR}$	Fulfilled

Proof of the mesh against puncturing

Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for negative influence of pretension V	Υ_{VII} [-] =	1.5
Dimensioning value of the applied pretensioning force by negative influence of V	V_{dII} [kips] =	10.1
Bearing resistance of the mesh to pressure stress in nail direction	D_R [kips] =	83.2
Resistance correction value for puncturing	Υ_{DR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh to pressure stress	D_R/Υ_{DR} [kips] =	55.5
Proof of bearing safety	$V_{dII} \Leftarrow D_R/\Upsilon_{DR}$	Fulfilled

Proof of the nail to combined stress

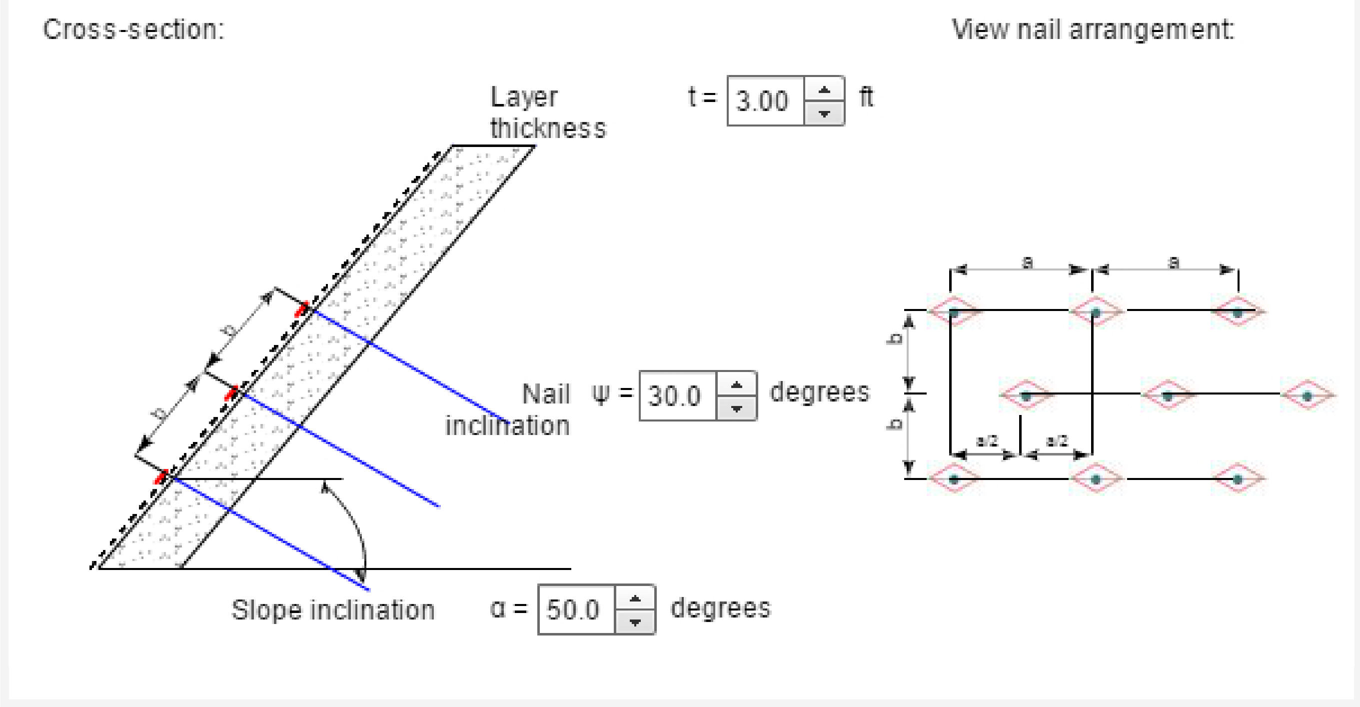
Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for positive influence of pretension V	Υ_{VI} [-] =	0.8
Dimensioning value of the applied pretensioning force by positive influence of V	V_{dI} [kips] =	5.4
Load factor for negative influence of pretension V	Υ_{VII} [-] =	1.5
Dimensioning value of the applied pretensioning force by negative influence of V	V_{dII} [kips] =	10.1
Calculatorily required shear force at dimensioning level in function of V_{dII}	S_d [kips] =	6.7
Maximum stress on the mesh for shearing-off	P_d [kips] =	0.0
Bearing resistance of the nail to tensile stress	T_{Rred} [kips] =	69.3
Bearing resistance of the nail to shear stress	S_{Rred} [kips] =	40.0
Resistance correction value for tensile stress	Υ_{TR} [-] =	1.5
Resistance correction value for shear stress	Υ_{SR} [-] =	1.5

Proof of bearing safety $([V_{dII}/(T_{Rred}/\Upsilon_{TR})]^2 + [S_d/(S_{Rred}/\Upsilon_{SR})]^2)^{0.5} \leq 1.0$	0.33	Fulfilled
Proof of bearing safety $([P_d/(T_{Rred}/\Upsilon_{TR})]^2 + [S_d/(S_{Rred}/\Upsilon_{SR})]^2)^{0.5} \leq 1.0$	0.25	Fulfilled

Minimal tensile strength in the nail for superficial instabilities

Dimensioning value of the static equivalent tensile force in the nail for determination of the nail length	T_d [kips] =	13.3

Bond Strength in Weathered Shale = 2200 lbs/ft
Tensile load in nail = 13.3 kips
Bond length of nail = 13,300 lbs / 2,200 (lbs/ft) = 6.0 ft



Dimensioning of the slope stabilization system TECCO® / SPIDER® by the RUVOLUM® method

Project No. **TXG0007**

Project Name **ACP AP-1 MP85**

Date, Author **Geosyntec - 10 Nov 2016**

Input quantities

Slope inclination	α	=	50.0 degrees
Layer thickness	t	=	3.00 ft
Friction angle ground (characteristic value)	Φ_k	=	32.0 degrees
Volume weight ground (characteristic value)	Υ_k	=	110.0 lbs/ft^3
Nail inclination to horizontal	Ψ	=	30.0 degrees
Nail distance horizontal	a	=	8.00 ft
Nail distance in line of slope	b	=	8.00 ft

Load cases

Streaming pressure considered			No
Earthquake considered			No
Coefficient of horizontal acceleration due to earthquake	ε_h	=	0.000 [-]
Coefficient of vertical acceleration due to earthquake	ε_v	=	0.000 [-]

Defaults and Safety Factors

Cohesion ground (characteristic value)	c_k	=	0.0 lbs/ft^2
Radius of pressure cone, top	ζ	=	6.0 in
Inclination of pressure cone to horizontal	δ	=	45.0 degrees
Slope-parallel force	Z_d	=	9.9 kips
Pretensioning force of the system	V	=	6.7 kips
Partial safety correction value for friction angle	Υ_Φ	=	1.25 [-]
Partial safety correction value for cohesion	Υ_c	=	1.25 [-]
Partial safety correction value for volume weight	Υ_Υ	=	1.00 [-]
Model uncertainty correction value	Υ_{mod}	=	1.10 [-]

Dimensioning quantities

Φ_d [degrees]	=	26.6
c_d [lbs/ft^2]	=	0.0
Υ_d [lbs/ft^3]	=	110.0

Elements of the system

Applied mesh type	TECCO® G65/4		
Applied spike plate	TECCO® system spike plate P66		
Bearing resistance of mesh to selective, slope parallel tensile stress	Z_R [kips]	=	16.9
Bearing resistance of mesh to pressure stress in nail direction	D_R [kips]	=	83.2
Bearing resistance of mesh against shearing-off in nail direction	P_R [kips]	=	41.6

Applied nail type	GEWI D = 32 mm		
Taking into account rusting away	Yes		
Bearing resistance of nail to tensile stress	T_{Rred} [kips]	=	69.3
Bearing resistance of nail to shear stress	S_{Rred} [kips]	=	40.0
Cross-section surface of the applied nail with / without rusting away	A_{red} [^{2}]	=	0.956

Proofs

Proof of the mesh against shearing-off at the upslope edge of the spike plate	Fulfilled
Proof of the mesh to selective transmission of the force Z onto the nail	Fulfilled
Proof of the nail against sliding-off of a superficial layer parallel to the slope	Fulfilled
Proof of the mesh against puncturing	Fulfilled
Proof of the nail to combined stress	Fulfilled

The given proofs concern the investigation of superficial instabilities. Additional investigations are required if there is a risk regarding global stability of the slope. If necessary the nail type and nail pattern have to be adapted.

Investigation of local instabilities between single nails

Proof of the mesh against shearing-off at the upslope edge of the spike plate

Maximum stress on the mesh for shearing-off in nail direction at the upslope edge of the spike plate (dimensioning level).	P_d [kips] =	0.0
Thickness of decisive sliding mechanism	t_{rel} [ft] =	2.55
Bearing resistance of the mesh against shearing-off in nail direction at the upslope edge of the spike plate (characteristic value).	P_R [kips] =	41.6
Resistance correction value for shearing-off of the mesh	Υ_{PR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh against shearing-off	P_R/Υ_{PR} [kips] =	27.7
Proof of bearing safety	$P_d \Leftarrow P_R/\Upsilon_{PR}$	Fulfilled

Proof of the mesh to selective transmission of the force Z onto the nail

Slope parallel force taken into account in the equilibrium considerations	Z_d [kips] =	10.1
Bearing resistance of the mesh to selective, slope-parallel tensile stress	Z_R [kips] =	16.9
Resistance correction value for selective, slope-parallel transmission of the force Z	Υ_{ZR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh to tensile stress	Z_R/Υ_{ZR} [kips] =	11.2
Proof of bearing safety	$Z_d \Leftarrow Z_R/\Upsilon_{ZR}$	Fulfilled

Investigation of slope-parallel, superficial instabilities

Proof of the nail against sliding-off of a superficial layer parallel to the slope

Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for positive influence of pretension V	Υ_{VI} [-] =	0.8
Dimensioning value of the applied pretensioning force by positive influence of V	V_{dI} [kips]=	5.4
Calculatorily required shear force at dimensioning level in function of V_{dI}	S_d [kips] =	6.7
Bearing resistance of the nail to shear stress	S_{Rred} [kips] =	40.0
Resistance correction value for shearing-off of the nail	Υ_{SR} [-] =	1.5
Dimensioning value of the bearing resistance of the nail to shear stress	S_{Rred}/Υ_{SR} [kips] =	26.7
Proof of bearing safety	$S_d \Leftarrow S_{Rred}/\Upsilon_{SR}$	Fulfilled

Proof of the mesh against puncturing

Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for negative influence of pretension V	Υ_{VII} [-] =	1.5
Dimensioning value of the applied pretensioning force by negative influence of V	V_{dII} [kips] =	10.1
Bearing resistance of the mesh to pressure stress in nail direction	D_R [kips] =	83.2
Resistance correction value for puncturing	Υ_{DR} [-] =	1.5
Dimensioning value of the bearing resistance of the mesh to pressure stress	D_R/Υ_{DR} [kips] =	55.5
Proof of bearing safety	$V_{dII} \Leftarrow D_R/\Upsilon_{DR}$	Fulfilled

Proof of the nail to combined stress

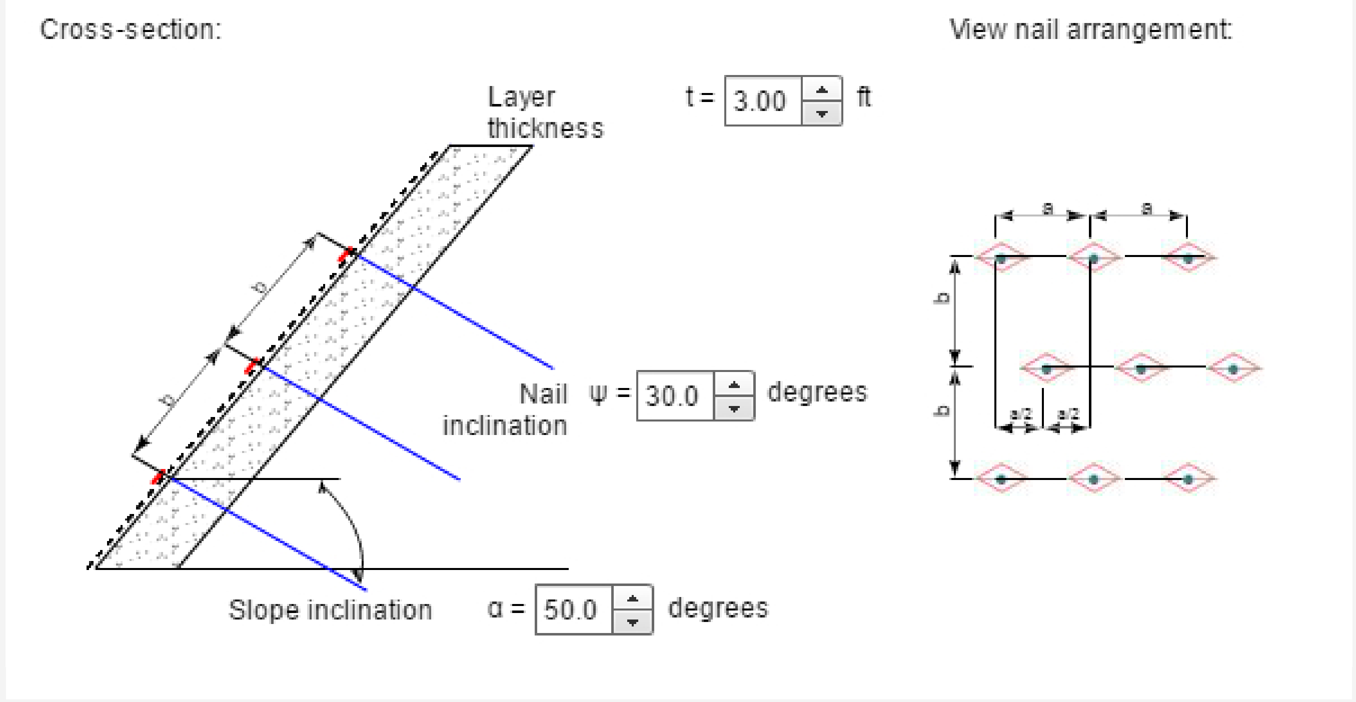
Pretensioning force effectively applied on nail	V [kips] =	6.7
Load factor for positive influence of pretension V	Υ_{VI} [-] =	0.8
Dimensioning value of the applied pretensioning force by positive influence of V	V_{dI} [kips] =	5.4
Load factor for negative influence of pretension V	Υ_{VII} [-] =	1.5
Dimensioning value of the applied pretensioning force by negative influence of V	V_{dII} [kips] =	10.1
Calculatorily required shear force at dimensioning level in function of V_{dII}	S_d [kips] =	6.7
Maximum stress on the mesh for shearing-off	P_d [kips] =	0.0
Bearing resistance of the nail to tensile stress	T_{Rred} [kips] =	69.3
Bearing resistance of the nail to shear stress	S_{Rred} [kips] =	40.0
Resistance correction value for tensile stress	Υ_{TR} [-] =	1.5
Resistance correction value for shear stress	Υ_{SR} [-] =	1.5

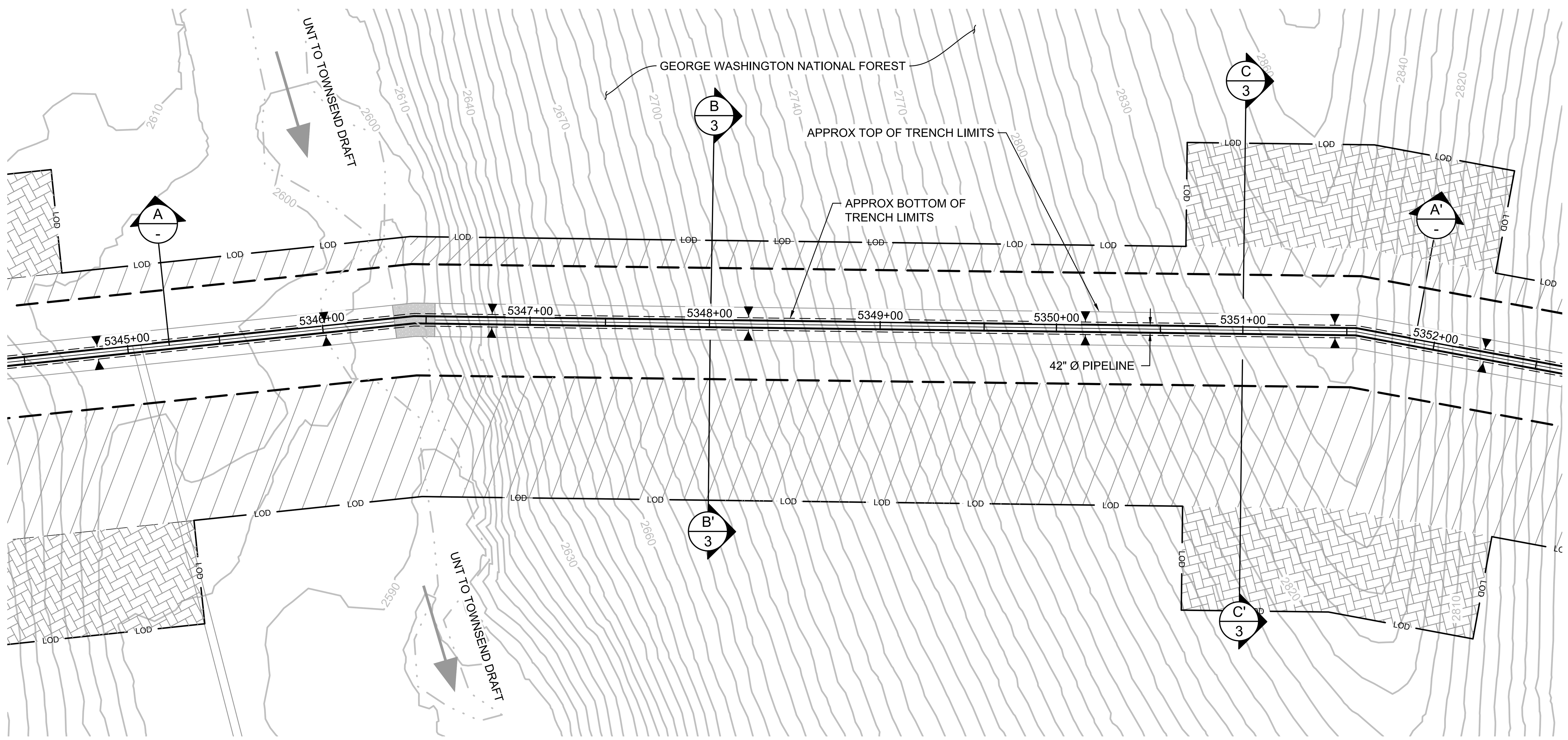
Proof of bearing safety $([V_{dII}/(T_{Rred}/\Upsilon_{TR})]^2 + [S_d/(S_{Rred}/\Upsilon_{SR})]^2)^{0.5} \leq 1.0$	0.33	Fulfilled
Proof of bearing safety $([P_d/(T_{Rred}/\Upsilon_{TR})]^2 + [S_d/(S_{Rred}/\Upsilon_{SR})]^2)^{0.5} \leq 1.0$	0.25	Fulfilled

Minimal tensile strength in the nail for superficial instabilities

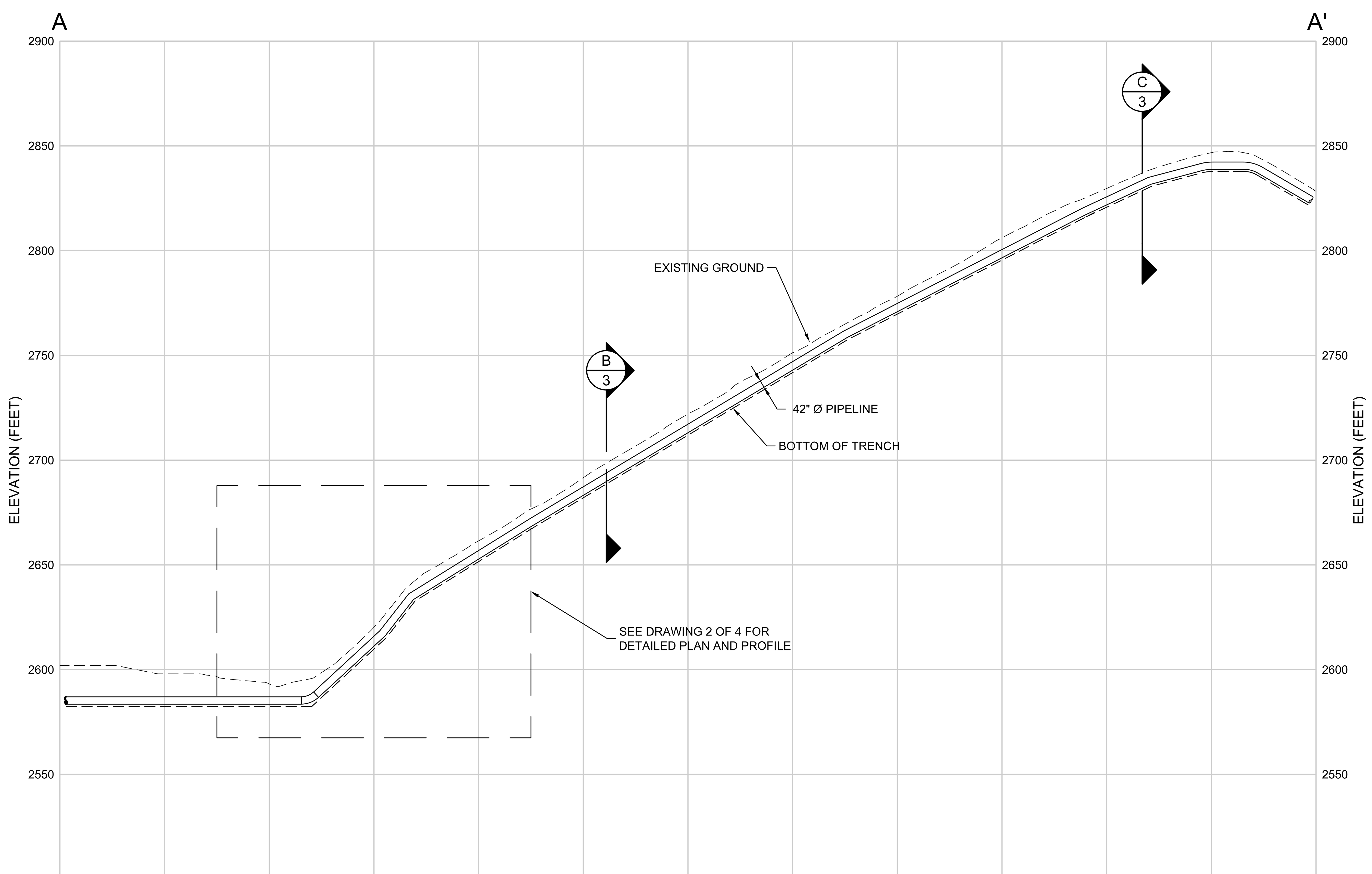
Dimensioning value of the static equivalent tensile force in the nail for determination of the nail length	T_d [kips] =	13.3

Bond Strength in Weathered Shale = 2200 lbs/ft
Tensile load in nail = 13.3 kips
Bond length of nail = 13,300 lbs / 2,200 (lbs/ft) = 6.0 ft





PLAN



PROFILE A - A'

LEGEND

- 2650 — EXISTING GROUND ELEVATION CONTOUR (FT, MSL)
- EXISTING STREAM LINE
- ACCESS ROAD
- LOD — LIMIT OF DISTURBANCE
- PERMANENT (ROW)
- 4426+00 — PIPELINE CENTERLINE AND STATIONING
- TEMPORARY (ROW)
- EXTRA WORK SPACE

N

NOTES:

1. MAPPING AND TOPOGRAPHY BASED ON UTM COORDINATE SYSTEM WITH NAD83 DATUM, ZONE 17, US SURVEY FOOT, CENTRAL MERIDIAN 81°W.
2. STATIONING SHOWN IS SLOPE STATIONING FOR ROUTE 11A (3D).
3. CONTOURS AND TOPOGRAPHIC FEATURES DERIVED FROM LIDAR DATA AND GPS SUB-METER GROUND SURVEY PERFORMED BY GAI CONSULTANTS, INC.
4. STREAM AND WETLAND DATA PROVIDED BY NRG/ERM.
5. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
6. VOLUMES, GRADES, ELEVATIONS AND QUANTITIES, WILL VARY DEPENDING ON SITE CONDITIONS ENCOUNTERED.
7. STANDARD EROSION AND SEDIMENT CONTROLS (NON-BIC) ARE SEPARATELY PROVIDED ON THE CONSTRUCTION ALIGNMENT SHEETS.

0 40 80
SCALE IN FEET

REV	DATE	DESCRIPTION	DRN	APP
C	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
B	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
A	11/2016	INTERIM DESIGN DRAWINGS	JJV / KH	TR

Atlantic Coast Pipeline

Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
11490 WESTHEIMER ROAD, SUITE 150
HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
PLAN AND PROFILE A-A'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN
MP 84.95 TO 85.05 (AP-1)

PRELIMINARY
NOT FOR CONSTRUCTION

DESIGN BY:	LCB/TR	DATE:	DECEMBER 2016
DRAWN BY:	JJV/KH	PROJECT NO.:	TXG0007.13
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REVIEWED BY:	RS	DRAWING NO.:	
APPROVED BY:	TR		1 OF 5

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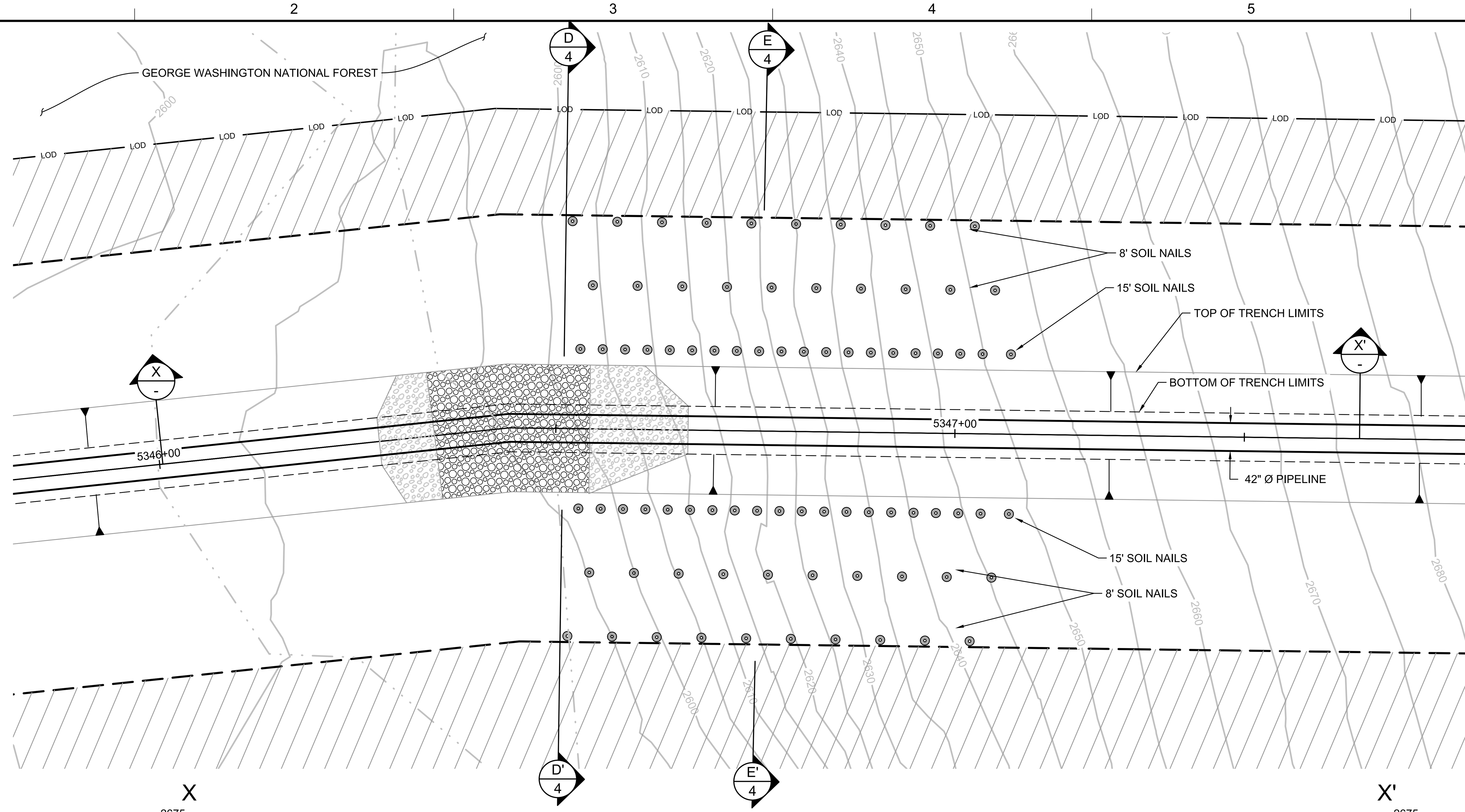

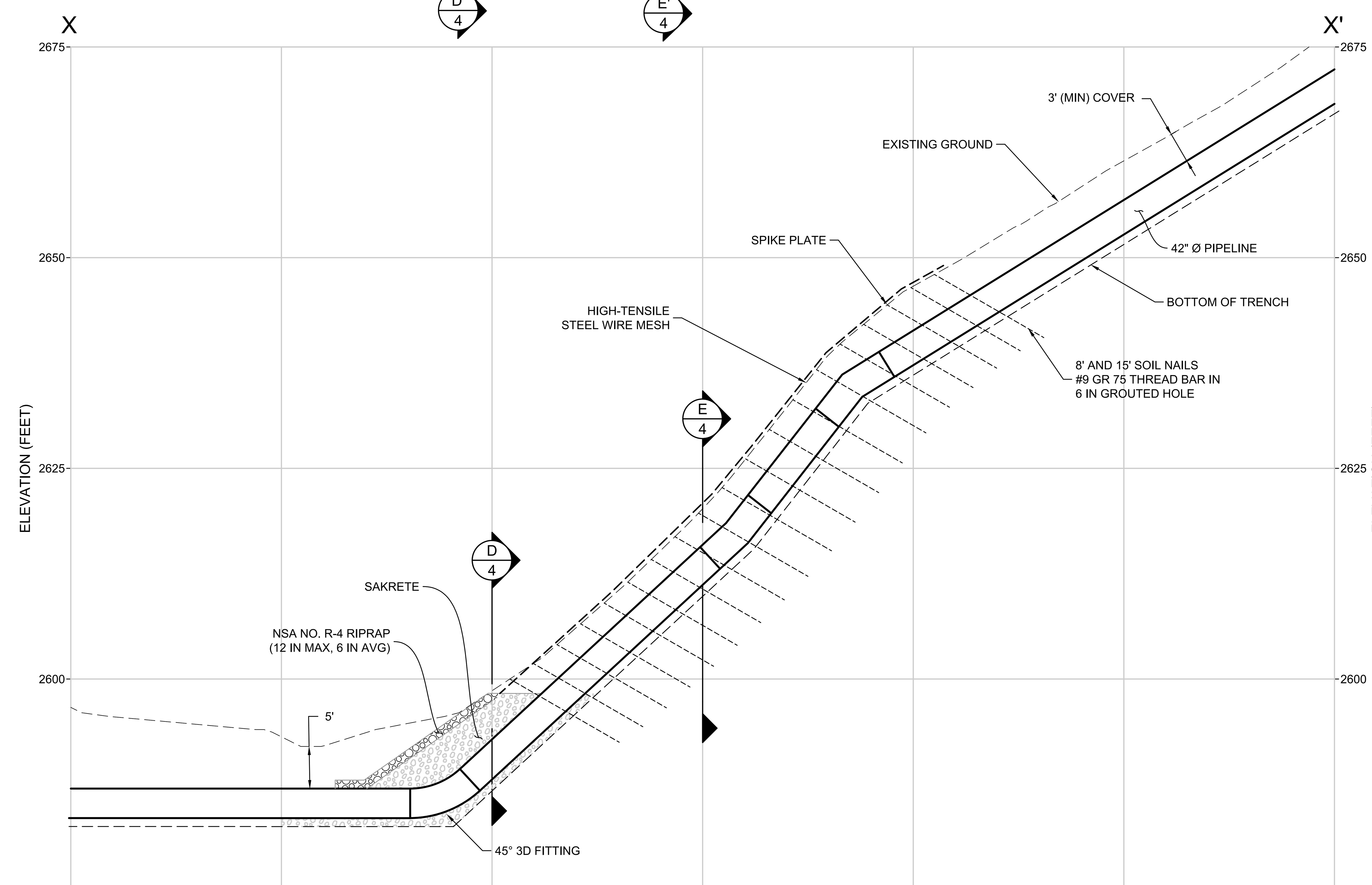

LEGEND

2650	EXISTING GROUND ELEVATION CONTOUR (FT, MSL)
	EXISTING STREAM LINE
LOD	LIMIT OF DISTURBANCE
	PERMANENT (ROW)
4426+00	PIPELINE CENTERLINE AND STATIONING
	TEMPORARY (ROW)

NOTES:

1. MAPPING AND TOPOGRAPHY BASED ON UTM COORDINATE SYSTEM WITH NAD83 DATUM, ZONE 17, US SURVEY FOOT, CENTRAL MERIDIAN 81°W.
2. STATIONING SHOWN IS SLOPE STATIONING FOR ROUTE 11A (3D).
3. CONTOURS AND TOPOGRAPHIC FEATURES DERIVED FROM LIDAR DATA AND GPS SUB-METER GROUND SURVEY PERFORMED BY GAI CONSULTANTS, INC.
4. STREAM AND WETLAND DATA PROVIDED BY NRG/ERM.
5. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
6. VOLUMES, GRADES, ELEVATIONS AND QUANTITIES, WILL VARY DEPENDING ON SITE CONDITIONS ENCOUNTERED.
7. STANDARD EROSION AND SEDIMENT CONTROLS (NON-BIC) ARE SEPARATELY PROVIDED ON THE CONSTRUCTION ALIGNMENT SHEETS.

0 10 20
SCALE IN FEET

REV	DATE	DESCRIPTION	DRN	APP
C	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
B	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
A	11/2016	INTERIM DESIGN DRAWINGS	JJV / KH	TR

Atlantic Coast Pipeline

Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
11490 WESTHEIMER ROAD, SUITE 150
HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN DETAILED PLAN AND PROFILE X-X'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN MP 84.95 TO 85.05 (AP-1)

PRELIMINARY NOT FOR CONSTRUCTION

DESIGN BY:	LCB/TR	DATE:	DECEMBER 2016
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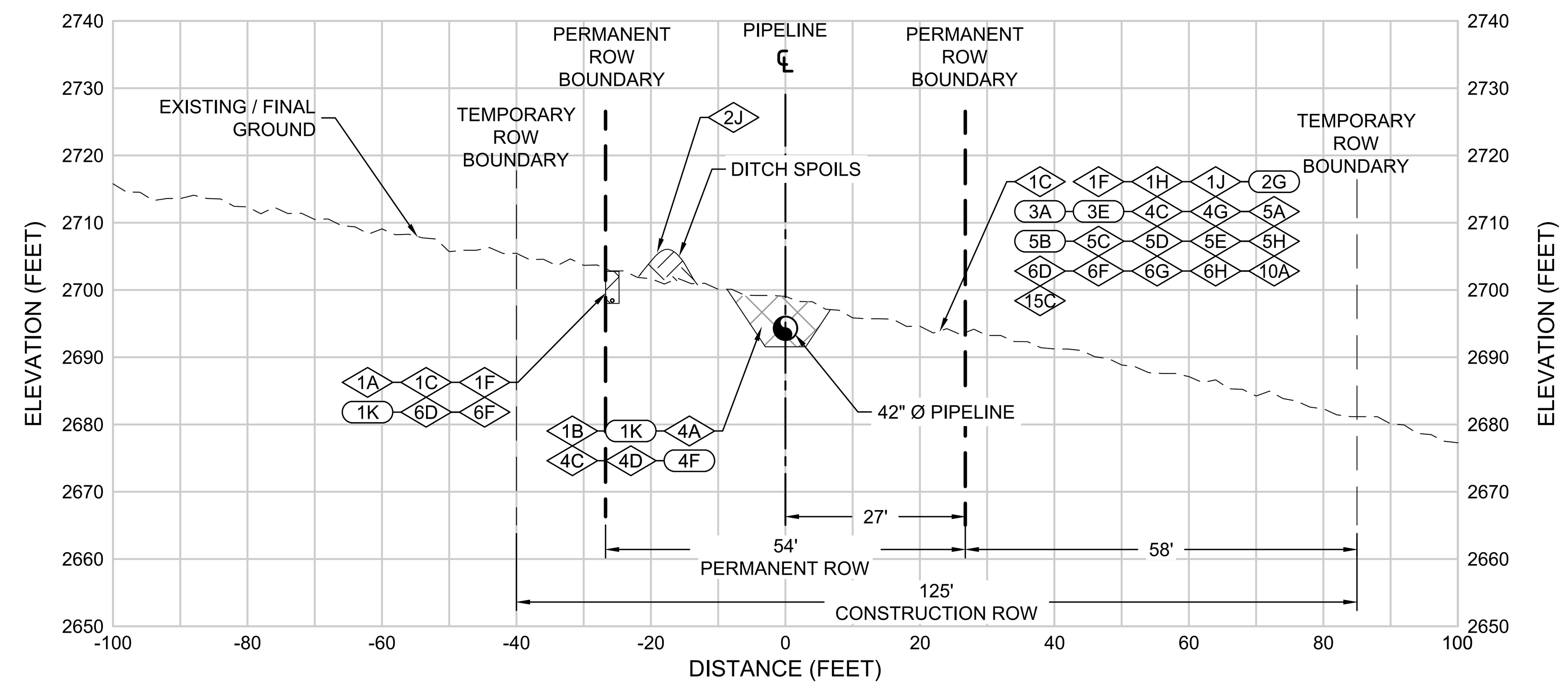
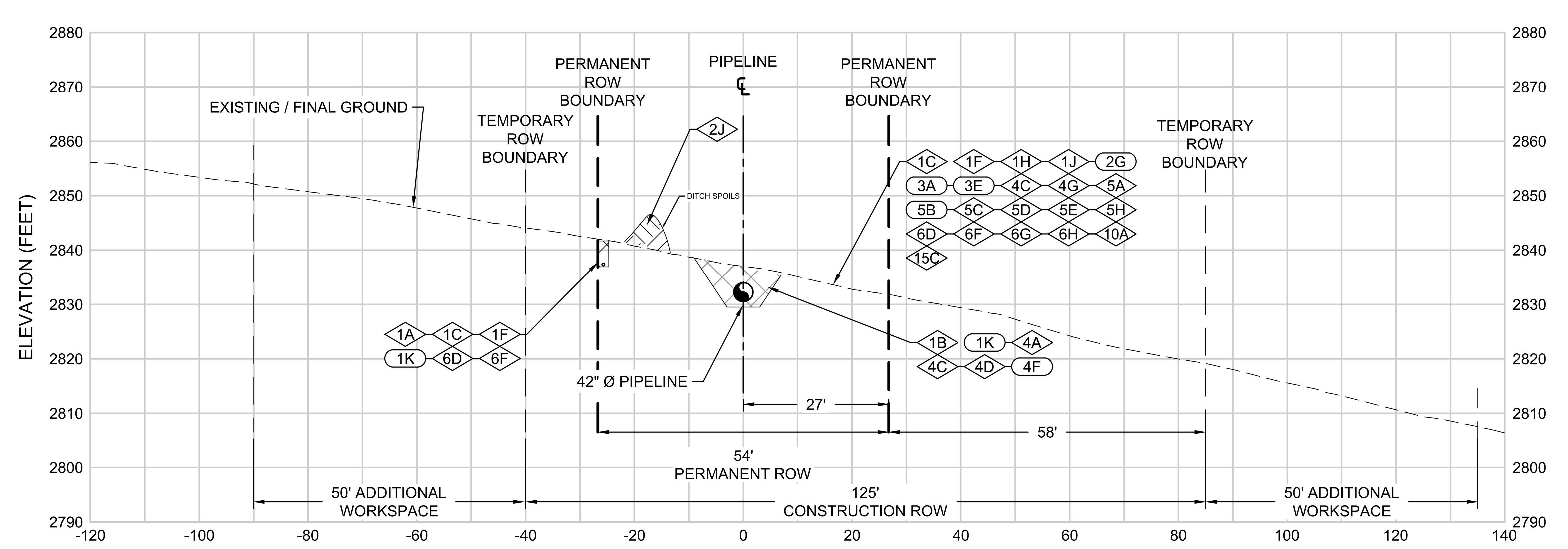
REVIEWED BY:	RS	DRAWING NO.:	
APPROVED BY:	TR		2 OF 5



SECTION B-B' (STA 5348+00)



SECTION C-C' (STA 5351+00)

BEST IN CLASS (BIC) INCREMENTAL CONTROLS

- 1B ENHANCED DRAIN (GERMAN DRAIN)
- 1C TARGETED SEEP DRAINS, AT INTERSEPTED SEEPS
- 1F ARMORED CHANNEL WITH DRAIN PIPE
- 1H STEEP CONVEYANCE CHANNEL
- 1I CHANGED SEEP CHARACTERISTICS
- 1J SINGLE TARGETED SEEP COLLECTOR
- 1K ENERGY DISSIPATION BASIN
- 2G GRADING TO MATCH EXISTING CONTOURS
- 2J SPOILS MANAGEMENT
- 3A TRACK DISTURBED SLOPES
- 3E COIR LOGS ON DISTURBED SLOPES
- 4A TRENCH BREAKERS (FOAM AND SANDBAGS), MODIFIED SPACING
- 4C SACK-CRETE BREAKERS (STRUCTURAL BREAKER)
- 4D SLEEVE INTERFACE BETWEEN PIPELINE AND BREAKER
- 4F TRENCH BREAKER WITH DRAINAGE
- 4G SACK-CRETE ARMOR WITH BREAKERS
- 5A SLOPE BREAKERS (TEMP AND PERMANENT), MODIFIED SPACING
- 5B SLOPE BREAKER ARMORED OUTLET
- 5C SLOPE BREAKERS WITH DIVERSION CHANNELS
- 5D ACCESS ROADS
- 5E TEMPORARY SLOPE BREAKER WITH DRAIN PIPE
- 5H SURFACE WATER DIVERSIONS
- 6D ARMORED CHANNEL
- 6F RIPRAP GRADATIONS
- 6G ARMORED V-SHAPED AND U-SHAPED CHANNELS
- 6H TYP SURFACE WATER CONTROL LAYOUT
- 10A BENCH RE-CONSTRUCTION THROUGH NATURAL STEPS
- 15C ACCESS TO REMOTE ROW LOCATIONS

NOTES:

1. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
2. VOLUMES, GRADES, ELEVATIONS AND QUANTITIES, WILL VARY DEPENDING ON SITE CONDITIONS ENCOUNTERED.
3. ACTUAL CUT/FILL CONFIGURATIONS MAY VARY DEPENDING ON ACTUAL SITE CONDITIONS.
4. STANDARD EROSION AND SEDIMENT CONTROLS (NON-BIC) ARE SEPARATELY PROVIDED ON THE CONSTRUCTION ALIGNMENT SHEETS.

0 20 40
SCALE IN FEET

REV	DATE	DESCRIPTION	DRN	APP
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B	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
A	11/2016	INTERIM DESIGN DRAWINGS	JJV / KH	TR

Atlantic Coast Pipeline

Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
11490 WESTHEIMER ROAD, SUITE 150
HOUSTON, TEXAS 77077

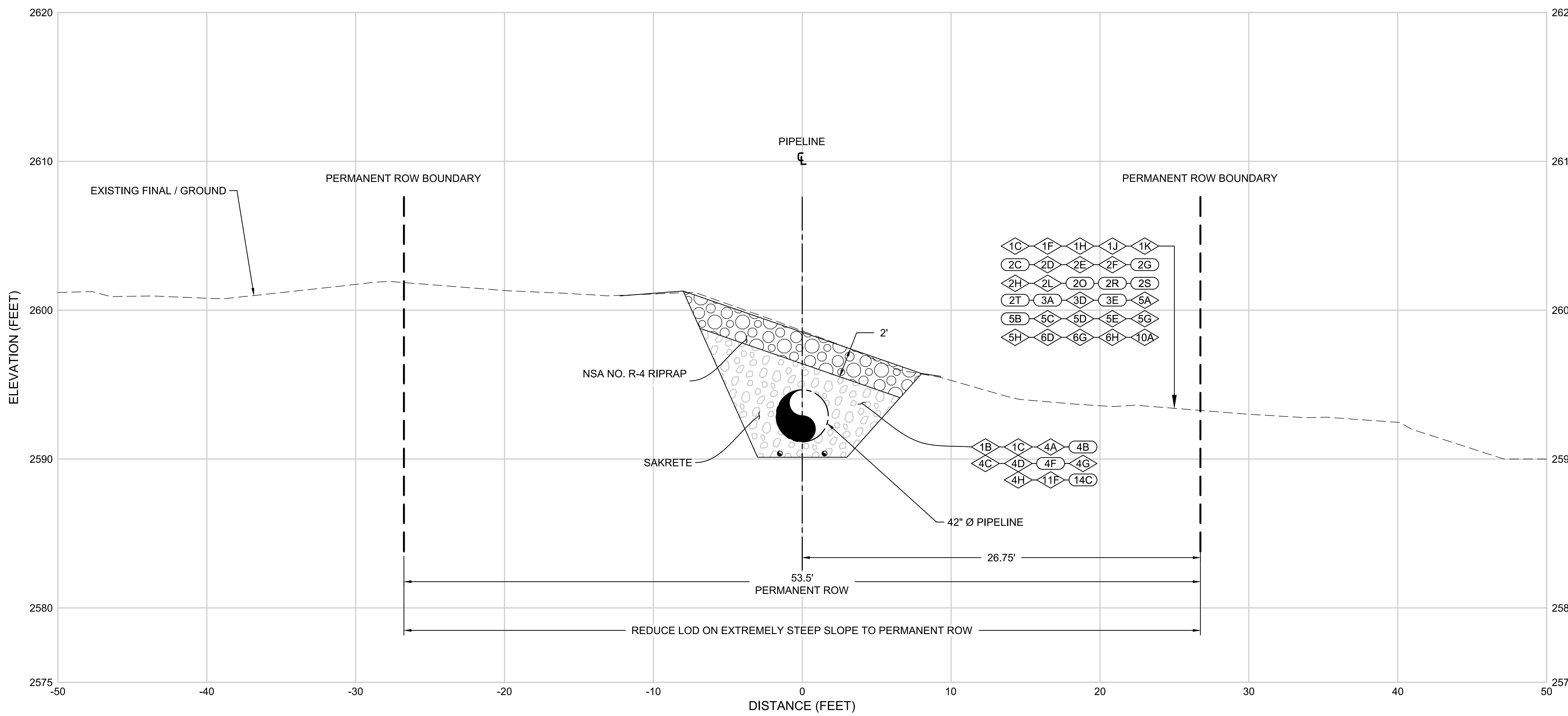
TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
SECTIONS B-B' AND C-C'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

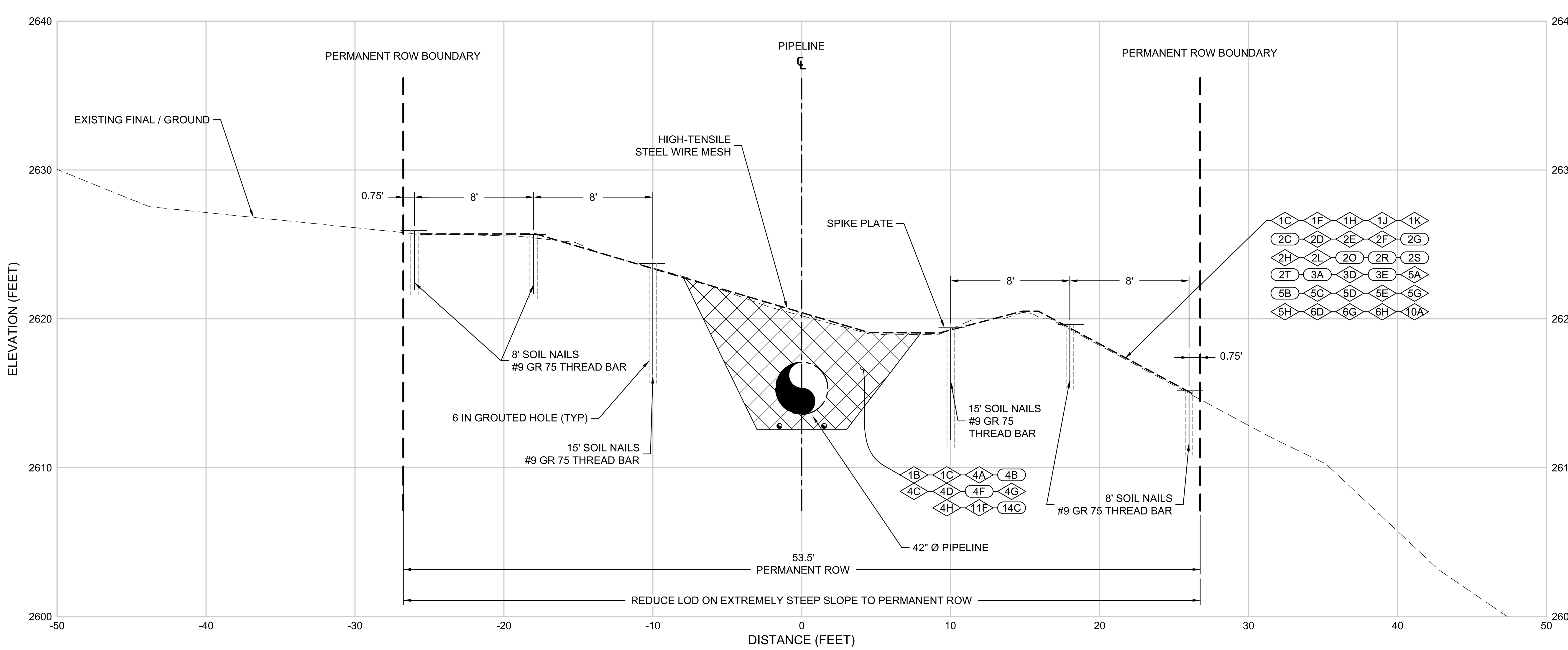
SITE: SITE SPECIFIC DESIGN
MP 84.95 TO 85.05 (AP-1)

PRELIMINARY
NOT FOR CONSTRUCTION

DESIGN BY: LCB/TR	DATE: DECEMBER 2016
DRAWN BY: JJV/KH	PROJECT NO.: TXG0007.13
CHECKED BY: LCB	FILE: TXG000713D04
REVIEWED BY: RS	DRAWING NO.:
APPROVED BY: TR	3 OF 5



SECTION D-D' (STA 5346+50)



SECTION E-E' (STA 5346+75)

BEST IN CLASS (BIC) INCREMENTAL CONTROLS

- 1B ENHANCED DRAIN (GERMAN DRAIN)
- 1C TARGETED SEEP DRAINS, AT INTERSEPTED SEEPS
- 1F ARMORED CHANNEL WITH DRAIN PIPE
- 1H STEEP CONVEYANCE CHANNEL
- 1I CHANGED SEEP CHARACTERISTICS
- 1J SINGLE TARGETED SEEP COLLECTOR
- 1K ENERGY DISSIPATION BASIN
- 2B GRADING TRENCH WITH OUTBOARD WEDGE
- 2C COMPACT BACKFILL
- 2D DRY SOILS AND BACKFILL
- 2E REMOVE UNSUITABLE EXISTING SOILS AS BACKFILL
- 2F ROCK BACKFILL (WITH DRAIN)
- 2G GRADING TO MATCH EXISTING CONTOURS
- 2H GRADING TO MINIMIZE BACKFILL
- 2L SOIL-NAIL WITH TECCO MESH
- 2O BENCH AND REGRADE WITH BACKFILL
- 2R TYP SECTION VIEW FILL WITH ROCK UNDER DRAIN
- 2S TYP PLAN VIEW FILL WITH ROCK UNDER DRAIN
- 2T TYP FILL WITH MULTIPLE ROCK CHANNELS
- 3A TRACK DISTURBED SLOPES
- 3D ROCK ARMORING ON DISTURBED SLOPES
- 3E COIR LOGS ON DISTURBED SLOPES
- 4A TRENCH BREAKERS (FOAM AND SANDBAGS), MODIFIED SPACING
- 4B TRENCH DAMS (FOAM BAGS OR FINE GRAINED SOILS)
- 4C SACK-CRETE BREAKERS (STRUCTURAL BREAKER)
- 4D SLEEVE INTERFACE BETWEEN PIPELINE AND BREAKER
- 4F TRENCH BREAKER WITH DRAINAGE
- 4G SACK-CRETE ARMOR WITH BREAKERS
- 5G NO WOOD CHIPS IN ROW
- 4H FLOWABLE FILL FOR TRENCH BACKFILL
- 5A SLOPE BREAKERS (TEMP AND PERMANENT), MODIFIED SPACING
- 5B SLOPE BREAKER ARMORED OUTLET
- 5C SLOPE BREAKERS WITH DIVERSION CHANNELS
- 5D ACCESS ROADS
- 5E TEMPORARY SLOPE BREAKER WITH DRAIN PIPE
- 5G NO WOOD CHIPS IN ROW
- 5H SURFACE WATER DIVERSIONS
- 6D ARMORED CHANNEL
- 6G ARMORED V-SHAPED AND U-SHAPED CHANNELS
- 6H TYP SURFACE WATER CONTROL LAYOUT
- 10A BENCH RE-CONSTRUCTION THROUGH NATURAL STEPS
- 11F AS-BUILT SURVEY TRENCH AND SLOPE BREAKERS
- 14C BLASTING PLAN(S)

NOTES:

1. FINAL CONFIGURATION OF ROW RESTORATION MEASURES TO BE DETERMINED BASED ON CONDITIONS ENCOUNTERED AT TIME OF CONSTRUCTION, AND MAY CHANGE OR VARY AND/OR INCORPORATE ADDITIONAL TYPICAL DETAILS TO MITIGATE SPECIFIC CONDITIONS.
2. VOLUMES, GRADES, ELEVATIONS AND QUANTITIES, WILL VARY DEPENDING ON SITE CONDITIONS ENCOUNTERED.
3. ACTUAL CUT/FILL CONFIGURATIONS MAY VARY DEPENDING ON ACTUAL SITE CONDITIONS.
4. STANDARD EROSION AND SEDIMENT CONTROLS (NON-BIC) ARE SEPARATELY PROVIDED ON THE CONSTRUCTION ALIGNMENT SHEETS.

0 5 10
SCALE IN FEET

REV	DATE	DESCRIPTION	DRN	APP
C	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
B	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB
A	11/2016	INTERIM DESIGN DRAWINGS	JJV / KH	TR

Atlantic Coast Pipeline

Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
11490 WESTHEIMER ROAD, SUITE 150
HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
SECTIONS D-D' AND E-E'

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN
MP 84.95 TO 85.05 (AP-1)

PRELIMINARY
NOT FOR CONSTRUCTION

DESIGN BY:	LCB/TR	DATE:	DECEMBER 2016
DRAWN BY:	JJV/KH	PROJECT NO.:	TXG0007.13
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APPROVED BY:	TR		

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GENERAL NOTES

1. STEEL WIRE MESH REINFORCEMENT SYSTEM INCLUDING STEEL TECCO WIRE MESH REINFORCEMENT (G65/4 MM), SPIKE PLATES (P66), AND OTHER FACING HARDWARE FABRICATED BY GEOBRUGG.
2. PERFORM NAIL TESTING IN ACCORDANCE WITH THE FHWA-NHI-14-007 (FEB 2015) AT SELECT SOIL NAILS. VERIFICATION TEST LOAD (VTL) = 27 KIPS. PROOF TEST LOAD (PTL) = 20 KIPS.
3. TOTAL LENGTH OF SOIL NAIL = 8 FT OR 15 FT, AS SPECIFIED IN DRAWINGS.
4. SOIL NAIL ORIENTATION = 30 DEGREES FROM THE HORIZONTAL.

SOIL NAIL VERIFICATION TESTING NOTES

1. PERFORM VERIFICATION TESTS ON SACRIFICIAL TEST NAILS WHICH WILL NOT BE INCORPORATED INTO THE PERMANENT WORK.
2. PERFORM NAIL TESTING ONLY AFTER THE GROUT HAS CURED FOR AT LEAST 72 HOURS AND ATTAINED AT LEAST 1,700 PSI (3-DAY) COMPRESSIVE STRENGTH.
3. THE REQUIRED 28-DAY COMPRESSIVE STRENGTH IS 3,000 PSI.
4. PROVIDE A BONDED LENGTH OF 6 FT.
5. PROVIDE EXTRA BAR LENGTH OUTSIDE THE DRILL HOLE TO ALLOW PROPER CONNECTION TO THE LOAD ASSEMBLY.
6. PRIOR TO GROUTING THE DESIGNATED VERIFICATION TEST NAILS, INSTALL FREE STRESSING SLEEVE AS INDICATED TO ENSURE FULL TRANSFER OF VTL TO DESIGN BOND ZONE DURING TESTING.
7. THE ALIGNMENT LOAD (AL) NECESSARY TO MAINTAIN POSITION OF THE STRESSING AND TESTING EQUIPMENT MUST NOT EXCEED 0.025 TIMES VTL. [SET DIAL GAUGES TO "0" AFTER THE ALIGNMENT LOAD HAS BEEN APPLIED].
8. IN CASE PULLOUT IS NOT ACHIEVED UP TO VTL, TEST LOADS LARGER THAN VTL MAY BE APPLIED TO ACHIEVE PULLOUT. MONITOR THE JACK LOAD WITH A LOAD CELL. PROVIDE THE ENGINEER WITH THE LOAD CELL CALIBRATION CURVE BEFORE START OF TEST.
9. PROVIDE A DIAL GAUGE CAPABLE OF MEASURING TO 1/1000" MOVEMENT.
10. PERFORM A MINIMUM OF THREE VERIFICATION TESTS ON INSTALLATION OF PRODUCTION NAILS TO VERIFY THE CONTRACTOR'S INSTALLATION METHODS AND NAIL PULLOUT RESISTANCE.
11. APPLY INCREMENTAL LOAD UP TO VTL IN ACCORDANCE WITH THE FOLLOWING SCHEDULE. RECORD SOIL NAIL MOVEMENTS AT EACH LOAD INCREMENT.

LOAD	HOLD TIME
AL	1 MINUTE
0.13 VTL	10 MINUTES
0.25 VTL	10 MINUTES
0.38 VTL	10 MINUTES
1.50 VTL	10 MINUTES
0.75 VTL (CREEP TEST)	60 MINUTES (RECORDED AT 1, 2, 4, 5, 6, 10, 20, 30, 50, 60 MINUTES)
0.88 VTL	10 MINUTES
1.00 VTL	10 MINUTES
AL	1 MINUTES

HOLD EACH LOAD INCREMENT FOR A TIME PERIOD SPECIFIED ABOVE.

12. MONITOR THE VERIFICATION TEST NAIL FOR CREEP AT THE 0.75VTL LOAD INCREMENT. MEASURE NAIL MOVEMENTS DURING CREEP PORTION OF THE TEST AND RECORED AT 1 MINUTE, 2, 4, 5, 6, 10, 20, 30, 50 AND 60 MINUTES. MAINTAIN LOAD DURING THE CREEP TEST WITHIN 2 PERCENT OF THE INTENDED LOAD.
13. THE ENGINEER WILL REVIEW ALL VERIFICATION TEST TO DETERMINE IF THE NAIL IS ACCEPTABLE. A NAIL WILL BE ACCEPTED IF THE FOLLOWING 3 CRITERIA ARE MET:

a. PULLOUT DOES NOT OCCUR AT LOADS LESS THAN 1.00 VTL;

b. THE TOTAL MEASURED MOVEMENT AT THE MAXIMUM TEST LOAD EXCEEDS 80 PERCENT OF THE THEORETICAL ELASTIC ELONGATION OF THE TEST NAIL UNBONDED LENGTH.

c. THE CREEP MOVEMENT BETWEEN THE 1-MINUTE AND 10-MINUTE READINGS AT 0.75 VTL IS LESS THAN 0.04 INCH, AND 6-AND 60-MINUTE READINGS AT 0.75 VTL IS LESS THAN 0.08 INCH. THE CREEP RATE IN LINEAR OR DECREASING THROUGHOUT THE CREEP TEST LOAD-HOLD PERIOD.

SOIL NAIL PROOF TESTING NOTES

1. UPON COMPLETION OF VERIFICATION TESTING, PERFORM PROOF TESTING AT SELECT LOCATIONS OR AS APPROVED BY THE ENGINEER. PROOF TEST A MINIMUM OF 5 PERCENT OF PRODUCTION NAILS.
2. SIMILAR TO VERIFICATION TESTING, PROVIDE A BONDED LENGTH OF 6 FT.
3. THE MAXIMUM PROOF TEST LOAD (PTL) IS 75 PERCENT OF THE PRODUCT OF THE BONDED LENGTH AND THE NOMINAL PULLOUT RESISTANCE PER UNIT LENGTH OF THE SOIL MASS.
4. APPLY INCREMENTAL LOAD UP TO PTL IN ACCORDANCE WITH THE FOLLOWING SCHEDULE. RECORD ALL SOIL NAIL MOVEMENT AT EACH LOAD INCREMENT. THE ALIGNMENT LOAD IS LESS OR EQUAL TO 0.025 TIMES PTL.

LOAD	HOLD TIME
AL	1 MINUTE
0.17 PTL	UNTIL MOVEMENT STABILIZES
0.33 PTL	UNTIL MOVEMENT STABILIZES
0.50 PTL	UNTIL MOVEMENT STABILIZES
0.67 PTL	UNTIL MOVEMENT STABILIZES
0.83 PTL	UNTIL MOVEMENT STABILIZES
1.00 PTL	UNTIL MOVEMENT STABILIZES (CREEP TEST, RECORDED AT 1, 2, 4, 5, 6, AND 10 MINUTES)
AL	1 MINUTE

5. EACH LOAD INCREMENT IS HELD FOR A MINIMUM PERIOD OF 10 MINUTES.
6. CREEP TESTS MUST BE PERFORMED AT THE MAXIMUM PROOF TEST LOAD (PTL). THE CREEP PERIOD MUST START AS SOON AS THE MAXIMUM PROOF TEST LOAD IS APPLIED AND THE NAIL MOVEMENT MUST BE MEASURED AND RECORDED AT 1, 2, 4, 5, 6 AND 10 MINUTES. WHERE THE NAIL MOVEMENT BETWEEN 1 MINUTE AND 10 MINUTES EXCEEDS 0.04", THE MAXIMUM TEST LOAD SHALL BE MAINTAINED FOR AN ADDITIONAL 50 MINUTES AND MOVEMENTS MUST BE RECORDED AT 20, 30, 50 AND 60 MINUTES.
7. THE ENGINEER WILL REVIEW ALL PROOF TESTS TO DETERMINE IF THE NAIL IS ACCEPTABLE. A NAIL WILL BE ACCEPTED IF THE FOLLOWING THREE CRITERIA ARE MET:

a. PULLOUT DOES NOT OCCUR AT LOADS LESS THAN 1.0 PTL.

b. THE TOTAL SOIL NAIL MOVEMENT MEASURED AT PTL IS GREATER THAN 80 PERCENT OF THE THEORETICAL ELASTIC ELONGATION OF THE UNBONDED LENGTH.

c. THE TOTAL CREEP MOVEMENT OF LESS THAN 0.04" MEASURED BETWEEN THE 1 AND 10 MINUTE READINGS OR A TOTAL CREEP MOVEMENT OF LESS THAN 0.08" IS MEASURED BETWEEN THE 6 AND 60 MINUTE READINGS AND THE CREEP RATE IS LINEAR OR DECREASING THROUGHOUT THE CREEP TEST HOLD PERIOD.

8. SUCCESSFUL PROOF TEST NAILS MEETING THE ABOVE TEST ACCEPTANCE CRITERIA MAY BE INCORPORATED AS PRODUCTION NAILS.

REV	DATE	DESCRIPTION	DRN	APP
C	12/2016	PRELIMINARY - NOT FOR CONSTRUCTION	JJV	LCB

Atlantic Coast Pipeline

Geosyntec consultants

GEOSYNTEC CONSULTANTS, INC.
11490 WESTHEIMER ROAD, SUITE 150
HOUSTON, TEXAS 77077

TITLE: GEOHAZARD MITIGATION SITE SPECIFIC DESIGN
SOIL NAIL SPECIFICATIONS

PROJECT: ATLANTIC COAST PIPELINE, REV 11a

SITE: SITE SPECIFIC DESIGN
MP 84.95 TO 85.05 (AP-1)

PRELIMINARY
NOT FOR CONSTRUCTION

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