ATTACHMENT 4

COMPUTATION COVER SHEET

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

Title of Computations: Geohazard Mitigation Design at ACP AP-1 MP 73.20 to 73.50

Computations by:	Signature		4/7/2017
	Printed Name	Mustafa Erten, Ph.D., P.E. (TX)	Date
	Title	Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		4/7/2017
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	Printed Name	Logan Brant, Ph.D., P.E. (WV)	Date
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Approval notes: Supports drawing set Revision D, dated March 2017

Revision History:

No.	Description	Date	By	Checked by	Approval
D	Preliminary – Not for Construction	4/26/2017	MBE	LCB	LCB
A	Interim Design	12/22/2016	MBE	RS	LCB

Written by:	**Mustafa Erten**	Date:	**4/7/2017**	Reviewed by:	**Logan Brant**	Date:	**5/3/2017**
Client:	**ACP**	Project:	**Geohazard Mitigation Design 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 and cut and fill volume calculations 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) profile and cross sections; (ii) subsurface stratigraphy and engineering parameters; (iii) stability assessment methodology; (iv) stability assessment calculations; and (v) cut-fill volume calculations.

PROFILE AND CROSS SECTIONS

Figure 1 is a plan of the slope showing the pipeline alignment, the selected Profile A extending between STA 4417+00 and STA 4445+50, Sections 1-1', 2-2', 3-3', and 4-4' and the locations of the test pits observed during the Order 1 Soil Survey near the area of interest [RETTEW and Geosyntec, 2016].

Figure 2 and Figure 3 (split in two parts) show the elevation profile along Profile A for the existing, temporary, and the final ground conditions and the locations of the test pits. Figure 4 and Figure 5 present the existing, temporary, and final ground surface slope inclination profiles and the angle between the pipeline alignment and the slope fall line, again split in two parts.

In Figure 6, the elevation and slope inclination profiles of the four sections are shown. These four sections were selected to capture the side slopes with the maximum slope inclinations along the site. Figure 6 also shows the limit of disturbance (LOD) on the plots for each section, since it is the main area of interest for geohazard mitigation.

SUBSURFACE STRATIGRAPHY AND ENGINEERING PARAMETERS

The estimated material properties and developed soil stratigraphy for this site can be found in Appendix A. The engineering properties of the native soil and the backfill material were conservatively assumed the same, although the backfill material may have large rock fragments as a result of excavation into bedrock while grading the existing ground. The increase in the rock fragment content would increase the shear strength of the backfill material. For the site soils, a cohesion (c') value of 150 psf was assigned for the temporary condition during trench excavation and pipeline installation. The cohesion

(c') was conservatively assumed as 0 for the final ground. We consider the use of $c' = 0$ for the final ground conservative because the soil is likely to exhibit some apparent cohesion caused by: i) root systems of the vegetation after right of way (ROW) restoration and pre-existing vegetation; and ii) interstitial water tension in the partially saturated soil.

Sack-crete bags may be used for rebuilding natural benches or restoring the existing ground surface at relatively steeper areas along this site. The unit weight of sack-crete bags can be assumed similar to typical cement material. In engineering practice, sack-crete bags can be used to construct almost vertical walls of limited height. Therefore, a friction angle of 60 degrees is considered representative.

Table 1 lists the engineering parameters that we used in the slope stability analysis for the site soils and the sack-crete bags.

Table 1. Engineering Parameters for Slope Stability Analysis

Soil Type	USCS	Total Unit Weight (pcf)	Effective Stress Cohesion (psf)	Effective Stress Friction Angle (°)
Silts and Sandy/ Gravelly Silt	ML	110	150 (for temporary ground) 0 (for final ground)	32
Backfill	---	110	150 (for temporary ground) 0 (for final ground)	32
Sack-crete	---	170	0	60

Note: The bedrock is assumed to have sufficient strength that it will not control the stability of the slope

STABILITY ASSESSMENT METHODOLOGY

Infinite Slope Stability Assessment

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 slopes with no groundwater (zero pore pressures), the factor of safety against sliding (FS) is calculated by the following equation:

$$FS = \frac{2 \cdot c'}{\gamma \cdot z \cdot sin(2\beta)} + \frac{\tan\phi'}{\tan\beta} \qquad \text{Equation 1}$$

where c' is the cohesion of the soil, z is the depth of soil, ϕ' is the soil friction angle and β is the slope inclination. For a cohesionless soil (c'=0) Equation 1 is simplified to the following form;

$$FS = \frac{\tan\phi'}{\tan\beta} \qquad \text{Equation 2}$$

Stability Assessment with 2-D Limit Equilibrium Approach

The 2-D slope stability analyses were performed using Spencer's method [Spencer, 1973], as implemented in the computer program SLIDE, version 7.023 [Rocscience, 2013].

STABILITY ASSESSMENT CALCULATIONS

The geohazard mitigation recommendations were developed after evaluating the slope inclination values within the limits of disturbance (LOD) and performing slope stability assessment. As shown in Figure 4 and Figure 5, the existing slope inclinations along the slope varies generally between 2 degrees and 26 degrees, although there exists relatively short segments of slope where the inclination locally can be 34 degrees to 50 degrees. The slope stability assessment was conducted for the temporary ground and the final ground surface conditions.

<u>Temporary Ground</u>

During the trench excavation and pipeline construction, the existing ground surface will be temporarily graded to an approximately planar working surface within the LOD and this surface is called "temporary ground".

As shown in Figure 4 and Figure 5, the slope inclination for the temporary ground along the pipeline alignment varies between 6 degrees and 22 degrees, except for the zone between STA 4434+00 and STA 4435+00, where the slope inclination is approximately 26 degrees. Figure 6 shows the elevation and the slope inclination profiles of the four sections which were selected to capture the steepest side slopes along the site. The stability was evaluated only for the areas inside the LOD, since the pipeline construction will not disturb the existing ground outside the LOD. As seen in Figure 6, among four sections, the maximum slope inclination for the temporary ground within the LOD is 31 degrees and observed on Section 3-3'. Stability assessment was conducted using this maximum slope inclination. The friction angle value of the soils (native or backfill) is estimated as 32 degrees along the slope. The infinite slope stability analysis using Equation 1 show that, using a conservative cohesion value of 150 psf and a soil depth of 10 ft, the minimum factor of safety against sliding on Section 3-3' within the LOD is;

$$FS = \frac{2 \cdot c'}{\gamma \cdot z \cdot \sin(2\beta)} + \frac{\tan\phi'}{\tan\beta} = \frac{2 \cdot 150\, psf}{110\, pcf \cdot 10\, ft \cdot \sin(2x31°)} + \frac{\tan(32°)}{\tan(31°)} = 1.35$$

Figure 7 shows the slope inclinations with color shadings within the (LOD). As shown with the yellow shading, the maximum slope inclination along the side slopes of the temporary ground surface is anticipated to be approximately 37 degrees. For the temporary case, since the soil will also have some cohesion, 37-degree slope inclination is considered appropriate. Using the same soil parameters as above, the infinite slope stability analysis using Equation 1 show that the factor of safety against sliding on side slopes is;

$$FS = \frac{2 \cdot c'}{\gamma \cdot z \cdot \sin(2\beta)} + \frac{\tan\phi'}{\tan\beta} = \frac{2 \cdot 150\, psf}{110\, pcf \cdot 10\, ft \cdot \sin(2x37°)} + \frac{\tan(32°)}{\tan(37°)} = 1.11$$

As shown in Figure 7, the slope inclination is typically less than 17 degrees and less than 30 degrees at the steepest areas along the planar work surface around the pipeline trench. Using the same soil parameters as above and assuming a 30-degree slope inclination, the infinite slope stability analysis using Equation 1 show that the factor of safety against sliding along the planar work surface is;

$$FS = \frac{2 \cdot c'}{\gamma \cdot z \cdot \sin(2\beta)} + \frac{\tan\phi'}{\tan\beta} = \frac{2 \cdot 150\, psf}{110\, pcf \cdot 10\, ft \cdot \sin(2x30°)} + \frac{\tan(32°)}{\tan(30°)} = 1.40$$

At the zones along the planar work surface where slope inclinations are smaller, the factor of safety against sliding will be even higher.

Final Ground

After pipeline construction, Forest Service requests to restore the existing ground surface. Geosyntec calls this restored surface "final ground". The average slope inclination for the final ground surface is estimated using the surface contours generated by smoothing the existing ground surface contours, assuming that local surface anomalies will be diminished during restoration and the slope inclinations will be more uniform along the LOD.

As shown in Figure 4 and Figure 5, the slope inclination of the final ground along the pipeline alignment is typically less than 22 degrees. The maximum inclination along the slope is approximately 31 degrees, except for the relatively small areas between STA 4434+70 to STA 4434+95, between STA 4438+40 to STA 4438+65, and between STA 4438+90 to STA 4439+10. Figure 8 shows the anticipated average slope inclination values for the final ground with color shadings. When the cohesion of the soils (native or backfill) is assumed zero, for the typical 22-degree and the maximum 31-degree slope inclinations and using the soil parameters given in Table 1, the factors of safety against sliding are calculated as;

$$FS = \frac{\tan\phi'}{\tan\beta} = \frac{\tan(32°)}{\tan(22°)} = 1.55 \quad \text{and} \quad FS = \frac{\tan\phi'}{\tan\beta} = \frac{\tan(32°)}{\tan(31°)} = 1.04$$

Figure 6 shows the elevation and the slope inclination profiles of the four sections which were selected to capture the steepest side slopes along the site. The stability was evaluated only for the areas inside the LOD, since the pipeline construction will not disturb the existing ground outside the LOD. As seen in Figure 6, among four sections, the maximum slope inclination for the final ground within the LOD is 29 degrees and observed on Section 1-1'. Stability assessment was conducted using this maximum slope inclination. For this slope inclination and using the soil parameters given in Table 1, the factor of safety against sliding is calculated as;

$$FS = \frac{\tan\phi'}{\tan\beta} = \frac{\tan(32°)}{\tan(29°)} = 1.13$$

Table 2 shows the summary of the factor of safety values calculated for the temporary ground and final ground surfaces.

Table 2 Summary of Factor of Safety Against Shallow Seated Sliding

Factor of Safety Against Shallow Seated Sliding				
Temporary Ground		**Final Ground**		
Planar Work Surface	**Side Slopes**	**Typical Slope Inclination (22 Degrees)**	**Maximum Slope Inclination (31 Degrees)**	**Side Slopes**
1.40	1.10 (1.35 on Section 3-3')	1.55	1.04	1.13

Note: Pipeline will be buried below bedrock surface, so will not be affected by shallow seated sliding.

The minimum recommended geohazard mitigation controls for this steep slope area are listed below. This list does not include the remaining conditional geohazard mitigation controls that may be required at the time of construction and restoration.

- Foam trench breakers (BIC Incremental Control No. 4A)
- Temporary and permanent slope breakers for surface drainage (BIC Incremental Control No. 5A)

GEOHAZARD MITIGATION AT LOCAL STEEP SLOPE AREAS

According to the available topographic information, the slope inclinations for some portions of the existing ground between STA 4434+70 to STA 4434+95, between STA 4438+40 to STA 4438+65, and between STA 4438+90 to STA 4439+10 are steeper than 32 degrees. As the first option, Geosyntec recommends grading these areas to a slope inclination of 31 degrees or less. If grading is not feasible, Geosyntec recommends the second option of restoring the existing ground surface in these areas with sack-crete bags or equivalent (BIC Incremental Control No. 10A).

Option A: Grading to 31 Degree Slope Inclination or Less

According to the available topographical information, grading these local steep slope areas to 31 degree slope inclination or less would result in up to 5 ft of soil cut between STA 4434+70 to STA 4434+95, and up to 2 ft of soil cut between STA 4438+40 to STA

4439+10. For the 31 degree slope inclination and using Equation 2, the factor of safety against sliding is calculated as;

$$FS = \frac{\tan\phi'}{\tan\beta} = \frac{\tan(32°)}{\tan(31°)} = 1.04$$

<u>Option B: Restoring Existing Ground Using Sack-crete Bags</u>

Geosyntec performed 2-D slope stability analyses to demonstrate the restoration of the existing ground could be achieved using sack-crete bags to provide sufficient stability.

From STA 4434+70 to STA 4434+95

Figure 9 shows the 2-D slope stability assessment for the local steep slope area if the existing ground is restored with sack-crete bags between STA 4434+70 and STA 4434+95. The analysis in Figure 9 was performed along the centerline and represented the case after pipeline construction and trench backfilling. The analysis focused on the shallow seated slips near the area which was backfilled with sack-crete bags. In this area, the whole trench was assumed to be backfilled with sack-crete bags and the bags were keyed into bedrock at the bottom of the trench. Outside the trench area within the LOD, Geosyntec considers that the temporary ground will be on bedrock. Therefore sack-crete bags are suggested to be used only above the temporary ground level between STA 4434+70 and 4434+95 outside the trench limits. For given conditions, the analysis demonstrates that the stability is achieved.

From STA 4438+40 to STA 4438+65 and From STA 4438+90 to STA 4439+10

Figure 10 shows the 2-D slope stability assessment for the two local steep slope areas between STA 4438+40 to STA 4438+65 and from STA 4438+90 to STA 4439+10 after the existing ground is restored with sack-crete bags. The analysis in Figure 10 was performed along the centerline and represented the case after pipeline construction and trench backfilling. In this analysis, the sack-crete bags were placed only above the temporary ground level within the LOD. Outside the trench area within the LOD, Geosyntec considers that the temporary ground will be on bedrock which would represent a more conservative case than the one presented in Figure 10; therefore, no additional analysis was performed. For given conditions, the analysis demonstrates that the stability is achieved.

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CUT-FILL VOLUME CALCULATIONS

The cut and fill volume calculations were conducted in order to estimate the volume of soil materials that will need to be stored or transported during different stages of the pipeline construction. The cut and fill volumes and the net volume ignoring bulking for the temporary ground conditions were estimated as given below by comparing the existing ground and the temporary ground surfaces in AutoCAD (Figure 11):

Cut Volume = 17,756 cu.yd.
Fill Volume = 17,805 cu.yd.
Net Volume = 49 cu.yd. (Soil Bulking Ignored)

The bulked cut volume was calculated by multiplying the cut volume with a net bulking factor of 1.4. This net bulking factor accounts for the net volume increase anticipated following excavation and subsequent replacement of fill. We consider 1.4 to be a conservative value when used for the anticipated mix of soil and sedimentary bedrock, based on comparison with recommendations published by FHWA [1988] and by WVDOT [1998] in their design directive (DD) 406.

The net fill volume that is required to be temporarily stored at site during grading for the temporary ground condition is calculated as:

(Bulked Cut Volume) – (Fill Volume) = (Excess Grade Spoils)
1.4 x 17,756 cu. yd. – 17,805 cu.yd. = 7,054 cu.yd.

The volume of soils that will be cut during trench excavation is calculated as 7,804 cu.yd. by comparing the temporary ground surface with and without a trench in AutoCAD. After applying a net bulking factor of 1.4, the bulked trench excavation volume is calculated as 10,926 cu.yd.

In Table 3, the estimated volumes of non-native trench backfill materials to be placed into the trench are summarized.

Table 3. Non-Native Trench Backfill Materials

Item	Quantity	Assumed Thickness / Length (ft)	Assumed Cross Sectional Area (ft^2)	Volume (cu.yd.)
Foam Trench Breakers	32	6	70	498
42-inch Pipe	1	2,850	10	1,056
			TOTAL (cu.yd.)	1,554

The net fill volume of excess ditch spoils created during trench excavation and pipeline installation is calculated as:

> (Bulked Trench Excavation Volume) + (Non-Native Trench Backfill Material Volume) – (Trench Volume) = (Excess Grade Spoils)
> 10,926 cu.yd. + 1,554 cu.yd. – 7,804 cu.yd. = 4,676 cu.yd.

As the work at this site is currently described, excess spoils should be spreading across the construction ROW and extra workspace during restoration. Further input and discussions with ACP and the Contractor will be essential. The net fill volume requiring spreading across the construction ROW and extra workspace during restoration is calculated as follows considering 7,054 cu.yd. of surplus soils temporarily stored during temporary grading and 4,676 cu.yd. of surplus soil in ditch spoils:

> (Cut/Fill Net Volume) + (Excess Grade Spoils) + (Excess Ditch Spoils) = (Total Excess Spoils)
> 49 cu.yd. + 7,054 cu.yd. + 4,676 cu.yd. = 11,779 cu.yd.

The total area of the LOD (construction ROW and the extra workspace along the slope) is calculated approximately 362,500 ft^2. If the final surplus material is evenly distributed within the LOD, the additional thickness of soil along the slope is calculated as:

> 11,779 cu.yd. x 27 ft^3/cu.yd. / 362,500 ft^2 = 0.88 ft

Table 4 presents the summary of cut and fill volume calculations.

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Table 4. Summary of Soil Volume Calculation Results

Stage	**Volume (cu.yd.)**
Net Fill Volume of Excess Grade Spoils Requiring Temporary Storage During Grading for Temporary Ground Condition	7,054
Net Fill Volume of Excess Ditch Spoils After Backfilling the Trench	4,676
Net Fill Volume Requiring Spreading Across Construction ROW and Extra Workspace during Restoration	11,779

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FIGURES

Written by:	**Mustafa Erten**	Date:	**4/7/2017**	Reviewed by:	**Logan Brant**	Date:	**5/3/2017**
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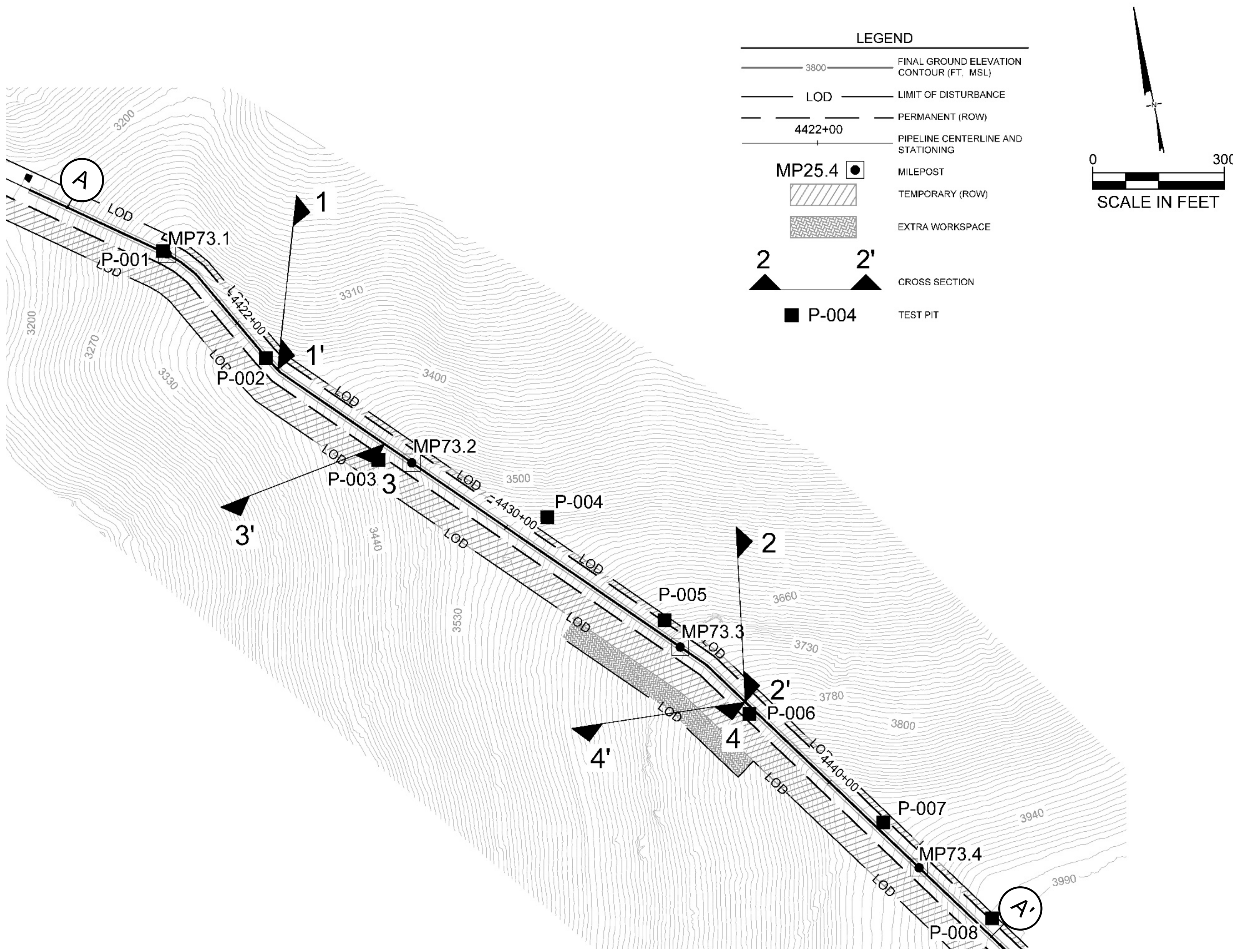

Figure 1. Plan View of the Existing Ground with the Test Pits and Profile A and Sections 1-1', 2-2', 3-3', and 4-4'

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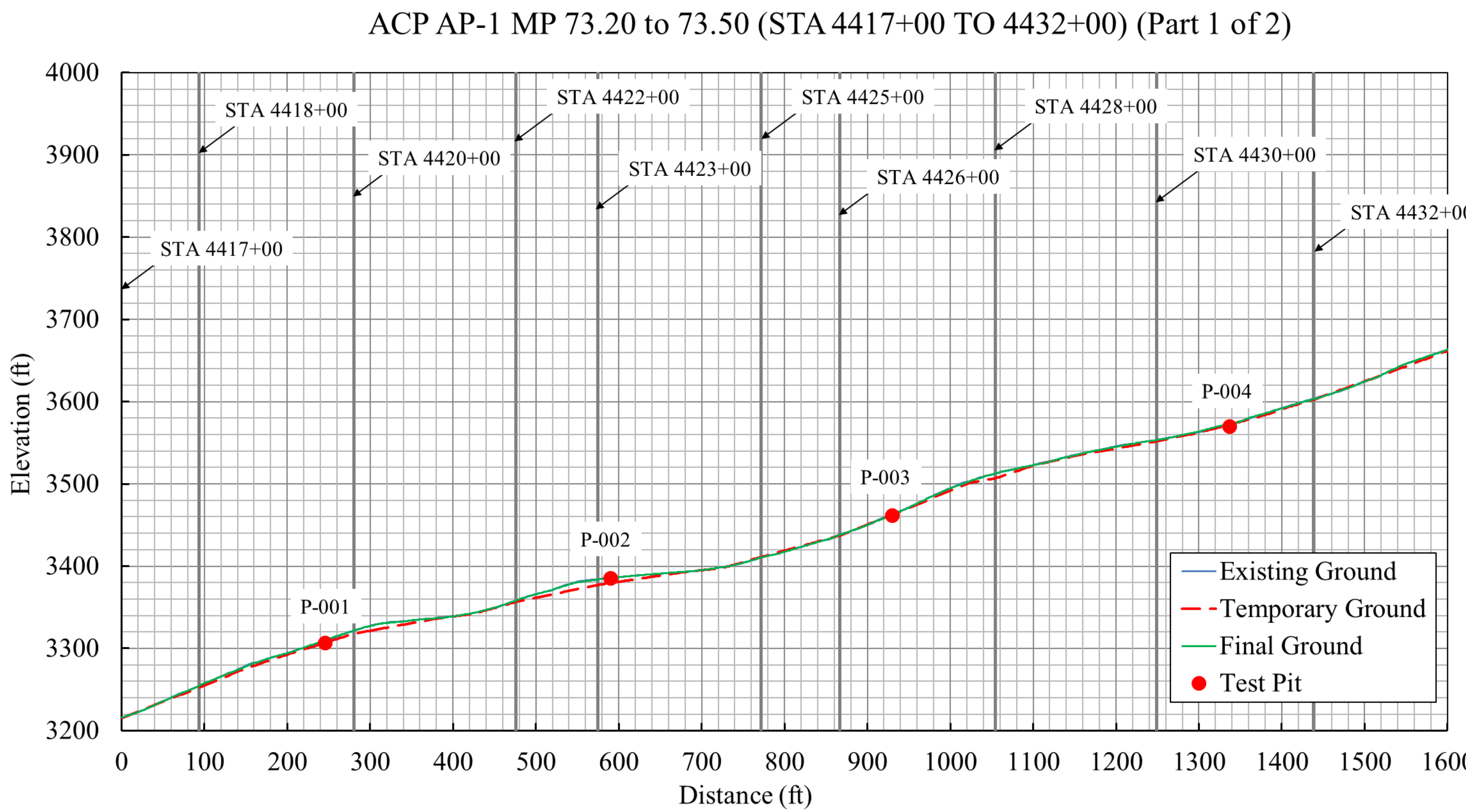

Figure 2. Elevation Profile of the Existing Ground, Temporary Ground, Final Ground and the Test Pit Locations (From STA 4417+00 to 4432+00)

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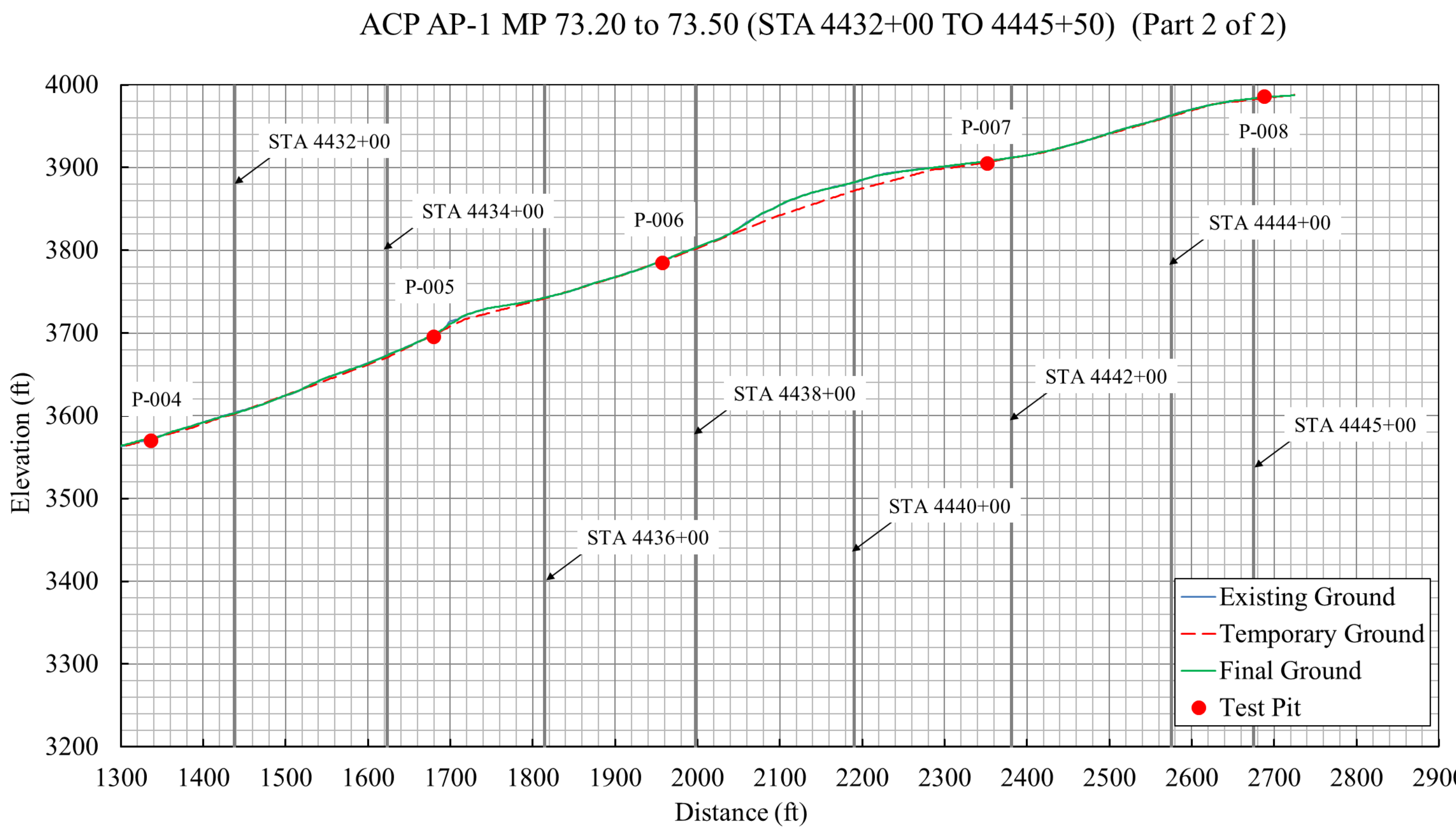

Figure 3. Elevation Profile of the Existing Ground, Temporary Ground, Final Ground and the Test Pit Locations (From STA 4432+00 to 4445+50)

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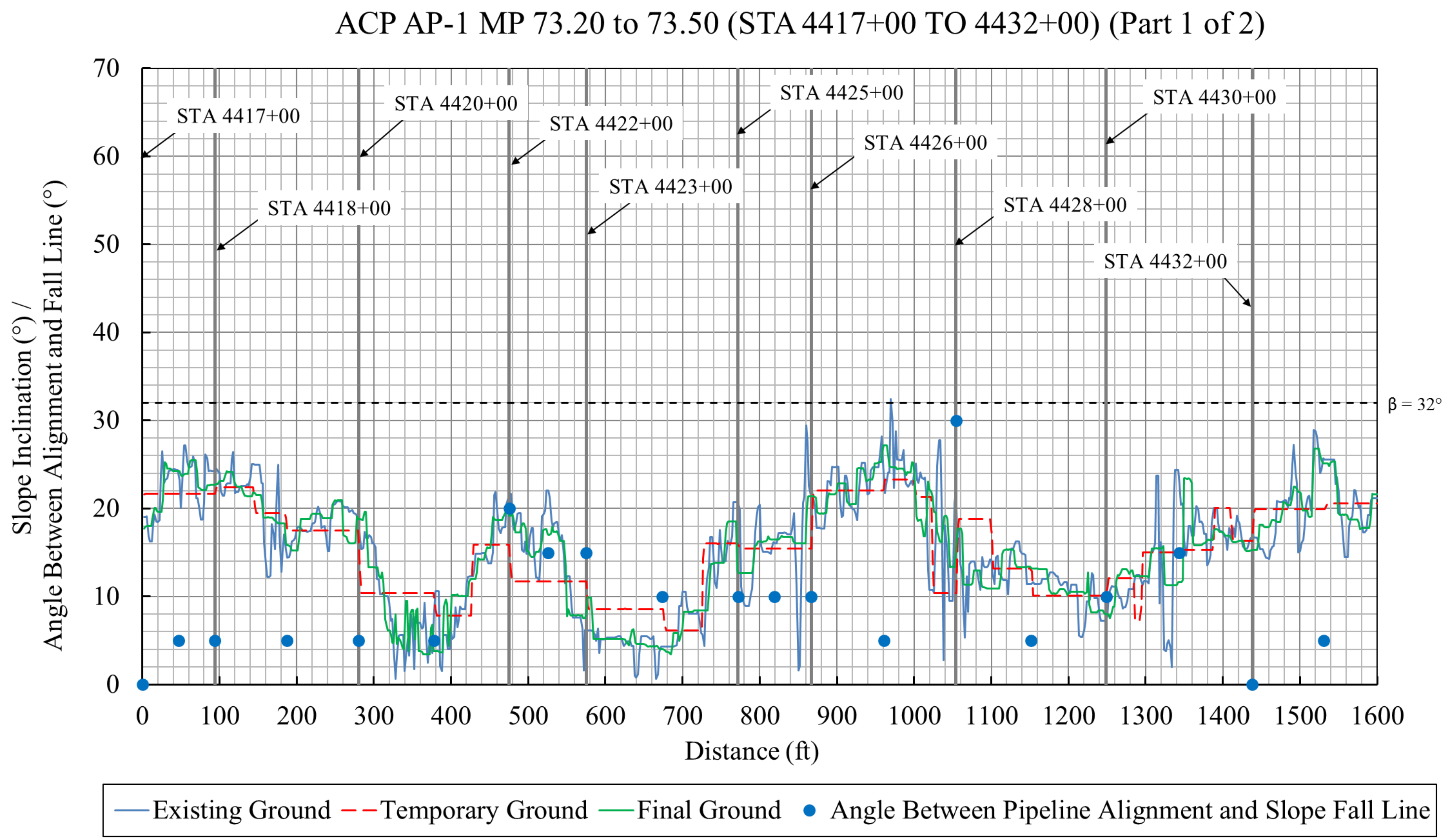

Figure 4. Slope Inclination Profile of the Existing Ground, Temporary Ground, Final Ground and the Angle Between Pipeline Alignment and Slope Fall Line (From STA 4417+00 to 4432+00)

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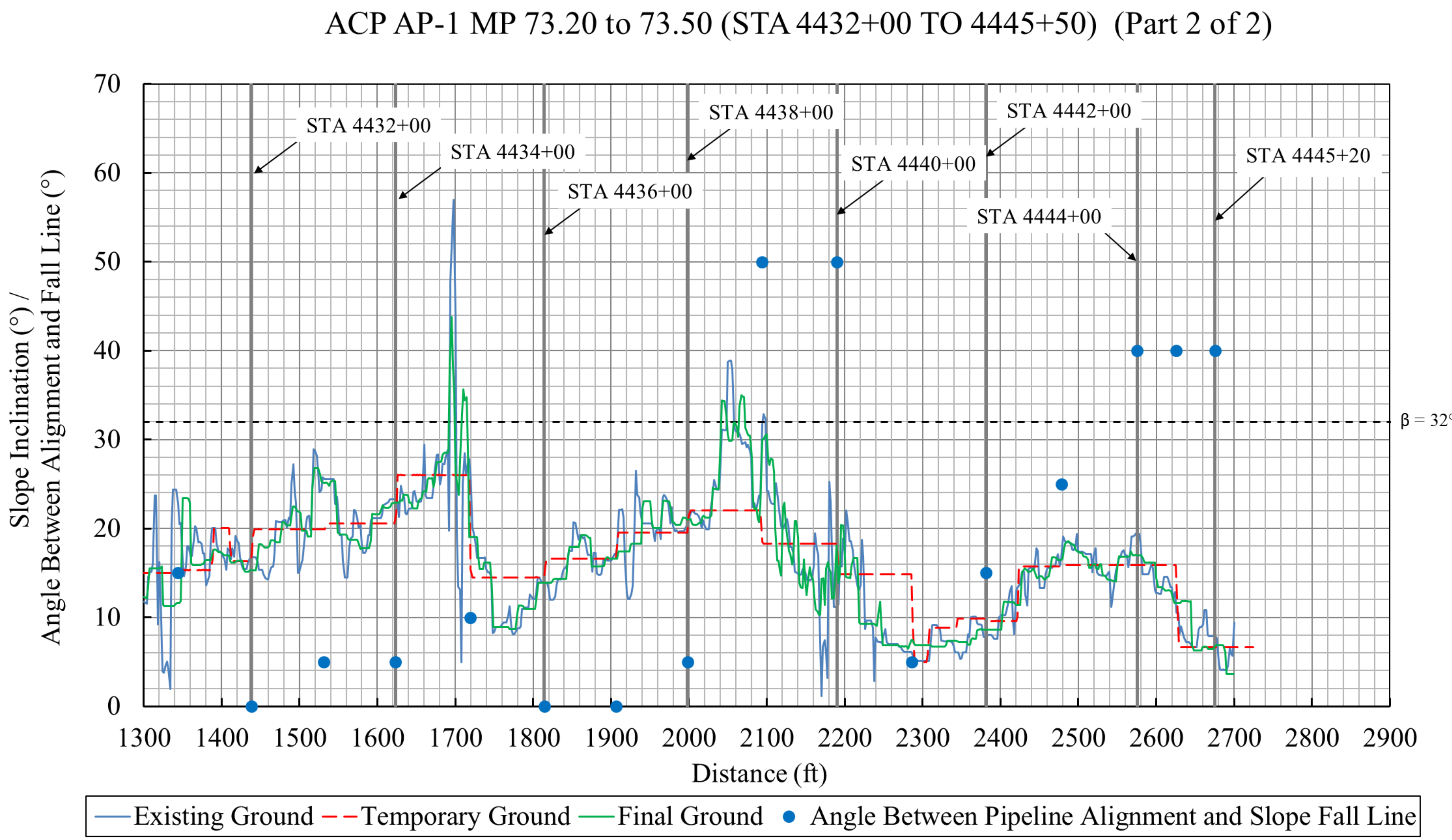

Figure 5. Slope Inclination Profile of the Existing Ground, Temporary Ground, Final Ground and the Angle Between Pipeline Alignment and Slope Fall Line (From STA 4432+00 to 4445+50)

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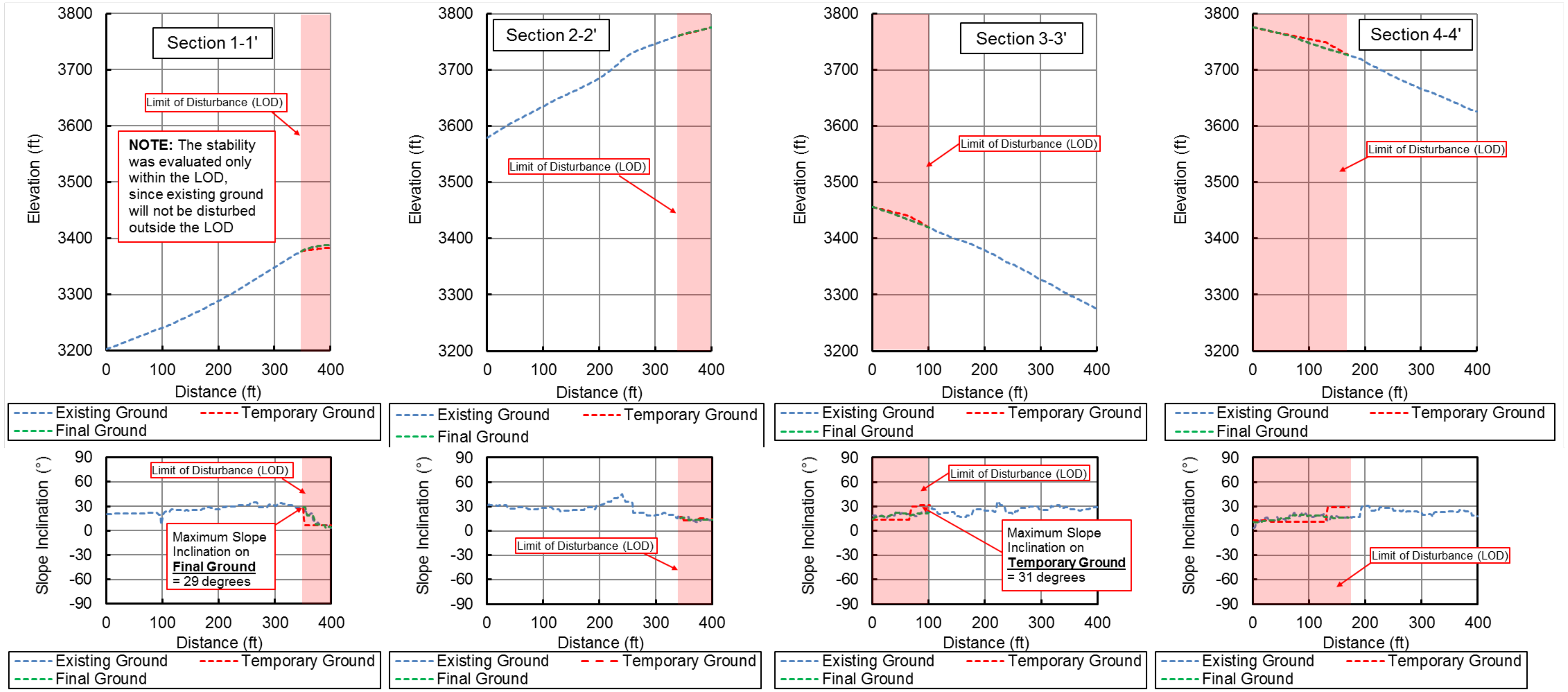

Figure 6. Elevation and Slope Inclination Profiles for Sections 1-1', 2-2', 3-3', and 4-4' for the Existing, Temporary, and Final Ground Surfaces

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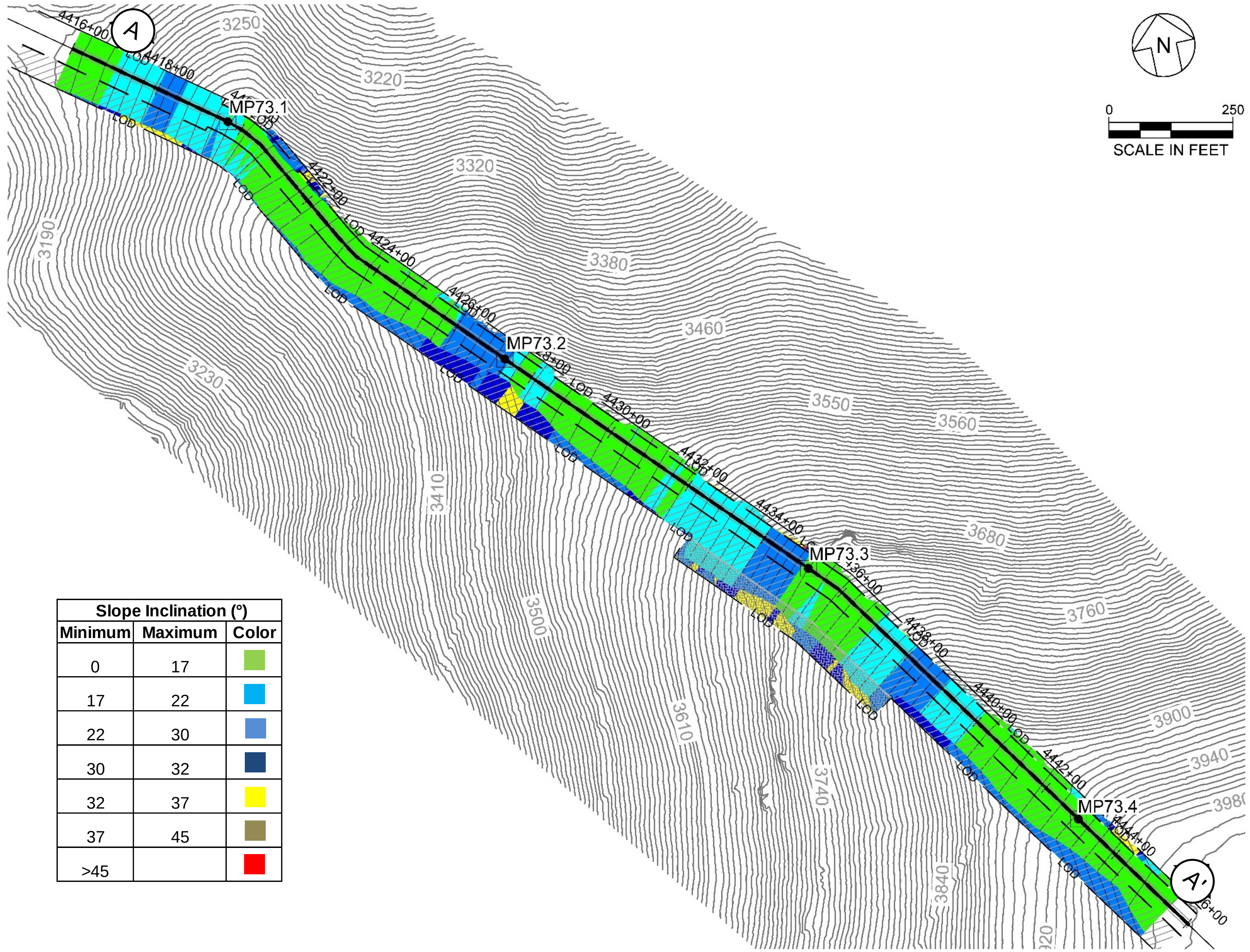

Slope Inclination (°)		
Minimum	**Maximum**	**Color**
0	17	
17	22	
22	30	
30	32	
32	37	
37	45	
>45		

Figure 7. Anticipated Average Slope Inclination Values for the Temporary Ground

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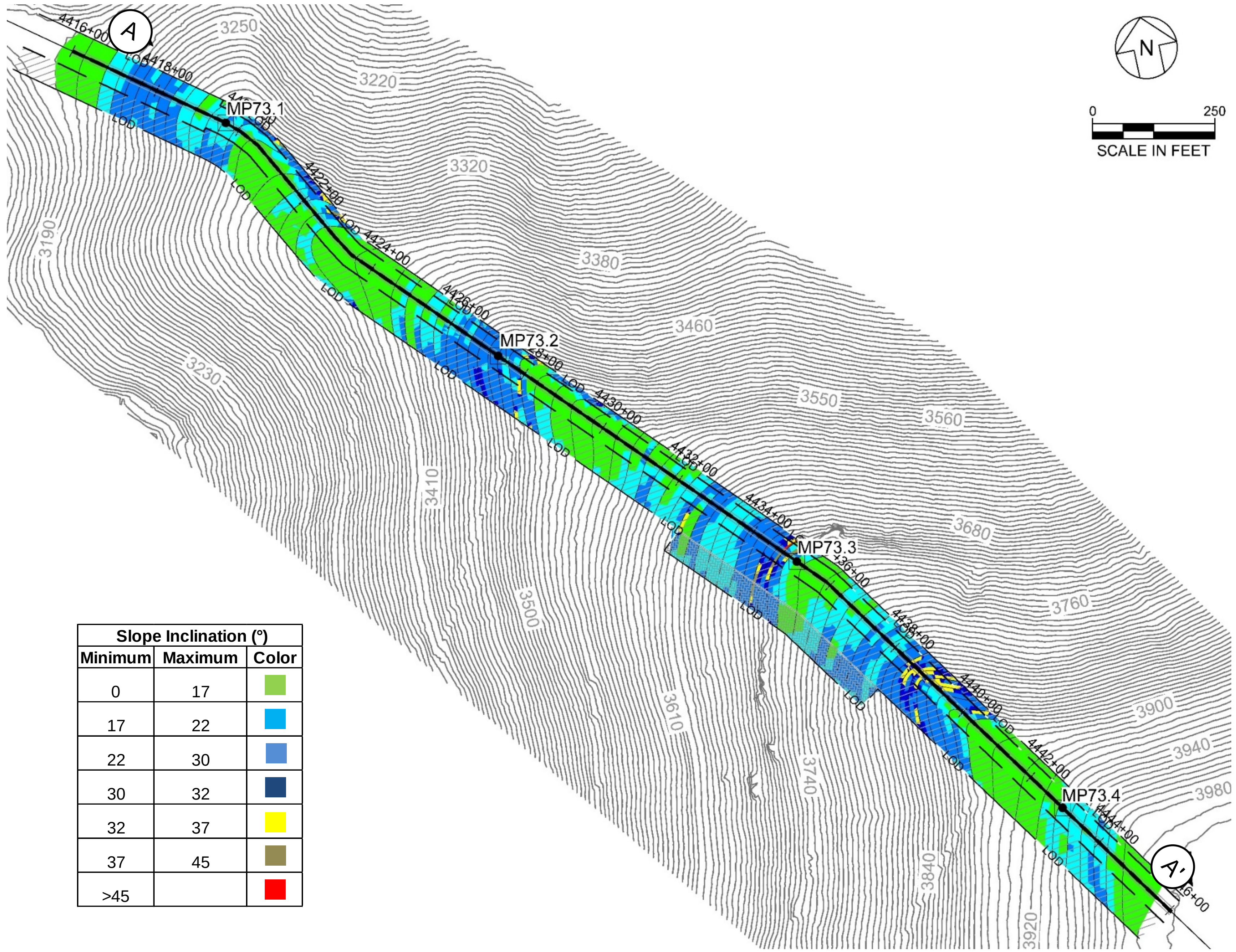
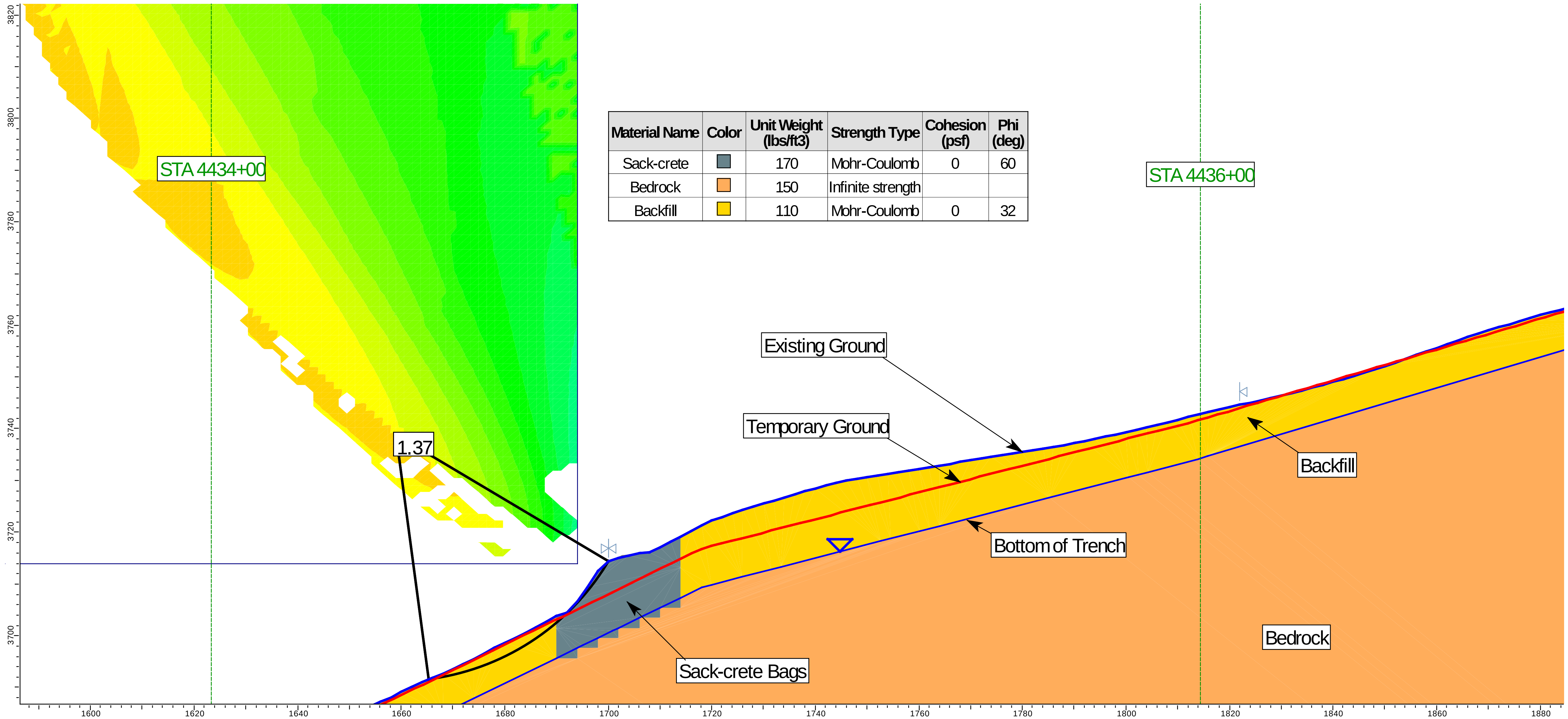


Slope Inclination (°)		
Minimum	Maximum	Color
0	17	
17	22	
22	30	
30	32	
32	37	
37	45	
>45		

Figure 8. Anticipated Average Slope Inclination Values for the Final Ground



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sack-crete		170	Mohr-Coulomb	0	60
Bedrock		150	Infinite strength		
Backfill		110	Mohr-Coulomb	0	32

Figure 9. Stability of the Steep Slope Area after Restoration of the Existing Ground with Sack-crete Bags Between STA 4434+70 to STA 4434+95

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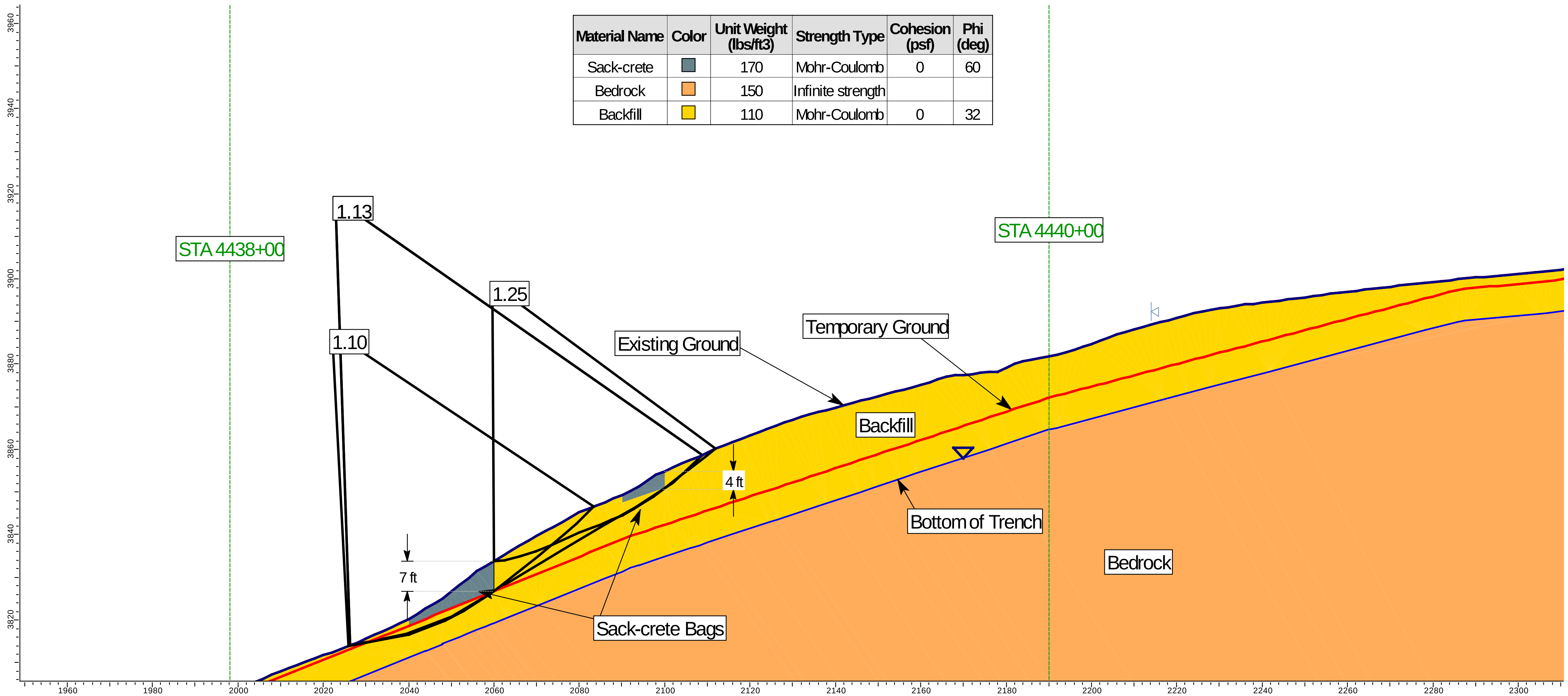

Figure 10 Stability of the Steep Slope Area after Restoration of the Existing Ground with Sack-crete Bags Between STA 4438+40 to STA 4438+65 and Between STA 4438+90 and STA 4439+10

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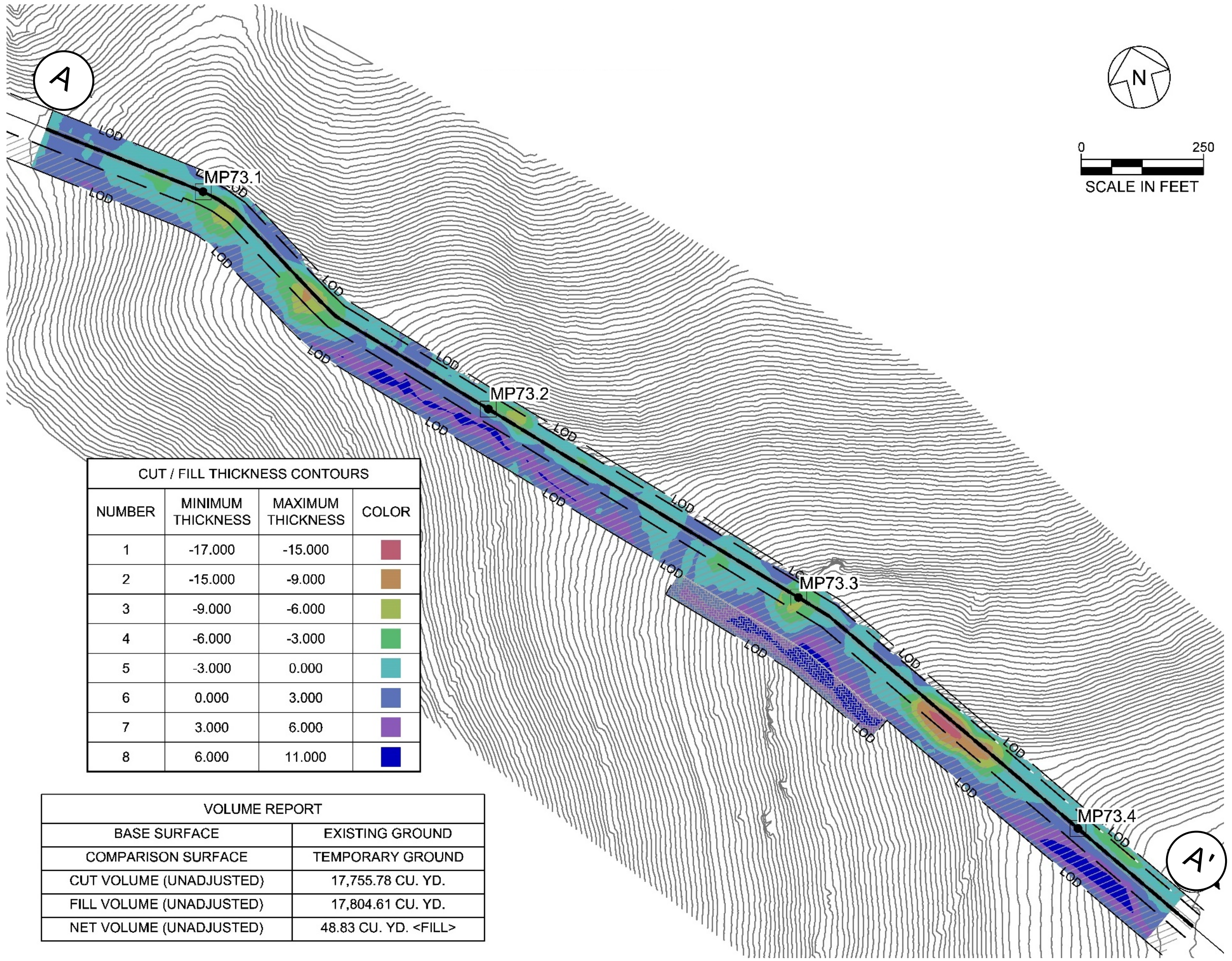

CUT / FILL THICKNESS CONTOURS			
NUMBER	MINIMUM THICKNESS	MAXIMUM THICKNESS	COLOR
1	-17.000	-15.000	
2	-15.000	-9.000	
3	-9.000	-6.000	
4	-6.000	-3.000	
5	-3.000	0.000	
6	0.000	3.000	
7	3.000	6.000	
8	6.000	11.000	

VOLUME REPORT	
BASE SURFACE	EXISTING GROUND
COMPARISON SURFACE	TEMPORARY GROUND
CUT VOLUME (UNADJUSTED)	17,755.78 CU. YD.
FILL VOLUME (UNADJUSTED)	17,804.61 CU. YD.
NET VOLUME (UNADJUSTED)	48.83 CU. YD. <FILL>

Figure 11. Cut-Fill Volume Calculations

Written by:	**Mustafa Erten**	Date:	**4/7/2017**	Reviewed by:	**Logan Brant**	Date:	**5/3/2017**
Client:	**ACP**	Project:	**Geohazard Mitigation Design at ACP AP-1 MP 73.20 to MP 73.50**	Project No.:	**TXG0007**	Task No.:	**013**

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 used 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) 73.20 to MP 73.50 (Site) in the Pocahontas County, West Virginia.

The information used to generate inputs to the design have been interpreted from the following sources:

- Ground reconnaissance conducted by Geosyntec Consultants (Geosyntec) on 20 April 2016;
- Test pits observed during Order 1 Soil Survey in June 2016;
- United States Department of Agriculture (USDA) soil surveys;
- United States Geological Survey (USGS) topographic and geologic maps; and
- Ground surface contours provided by GAI Consultants.

SURFACE TOPOGRAPHY

The slope at ACP AP-1 MP 73.20 to 73.50 rises approximately 770 feet (ft) in elevation over a horizontal distance of about 2,700 ft, for an average slope inclination of 29%. The proposed pipeline alignment is approximately parallel the ridge line.

Geomorphology

Geosyntec visited the Site on 20 April 2016 to conduct a ground reconnaissance survey. The reconnaissance survey identified a short moderate slope created by a cliff-forming outcrop of bedrock. The right of way traverses along a narrow ridge with steep slopes north and south of the centerline.

Regional Geology

The site lies within the northeastern margin of the Appalachian Plateau Physiographic Province of West Virginia, within the Kanawha Physiographic Section, also referred to as the Unglaciated Allegheny Plateau. This section exhibits high-elevation, low relief plateau-like morphology and is thoroughly dissected by streams with a dendritic drainage pattern and rugged topography [USGS 2017].

Geologic Formation

The site (in Pocahontas County) is locally underlain by an approximately 2,000-ft thick sequence of Late Paleozoic sedimentary rocks of the Mauch Chunk Group [USGS 1979]. The Mauch Chunk Group is composed of middle to late Mississippian-age red, green, and medium gray shale and sandstone with few thin limestone members. The formation contains dominately marine members with minor amounts of terrestrial members [USGS 1979, WVGES 1986].

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. This soil unit has a low to moderate shrink/swell capacity.

The Table 2 summarizes the results of the laboratory tests that were performed on soil samples collected from this soil unit by USDA.

Table 2 USDA Laboratory Test Results

Soil Unit	Soil (coverage)	Depth (in)	Liquid Limit	Plasticity Index	Gravel > 4.75mm (%)	Sand 4.75mm – 0.075mm (%)	Fines < 0.075mm (%)	Clay Content < 0.002mm (%)	USCS Symbol
CfC CfE CfF	Cateache (85%)	0-1	--	--	--	--	--	0	--
		1-3	20-30-40	4-10-15	15-30	0-40	45-70	0-15-27	CL, CL-ML, GC, GC-GM
		3-29	20-30-40	4-10-15	15-60	0-60	25-70	0-35-40	CL, CL-ML, GC, GC-GM
		29-33	20-30-40	4-10-15	40-80	0-50	10-40	0-35-40	GC, GC-GM,GP-GC

Note: liquid limit and plasticity index values provided are low – representative – high. Soils CfC, CfE, and CfF have identical engineering properties as reported by USDA.

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 3 summarizes the geotechnical engineering soil descriptions in selected test pits.

Table 3. 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).

Bedrock

The site is underlain by the Bluefield Formation of the Mauch Chunk Group 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 5 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 30 for the soil listed in Table 2, the corresponding ball-milled derived LL value was calculated as 40 using the relationship suggested in Stark et al. [2013].

Using Figure 1 (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.

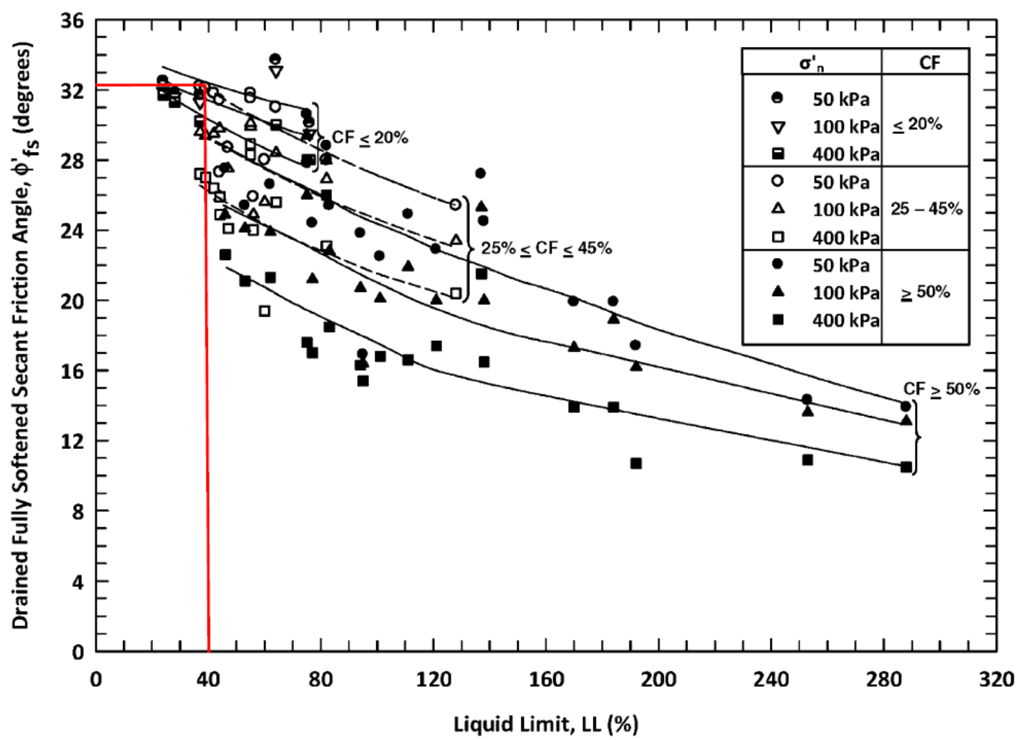

Figure 1. Empirical Correlation for Fully-Softened Drained Secant Friction Angle based on Ball-Milled Derived LL, CF, and σ'_n (Adapted from Stark et al. (2013).

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 sliding, 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

Groundwater

Based on the available information, Geosyntec assumed the groundwater at top of the bedrock below the soil layer for the purpose of geotechnical analyses. The groundwater level can fluctuate due to seasonal changes and periodic precipitations.

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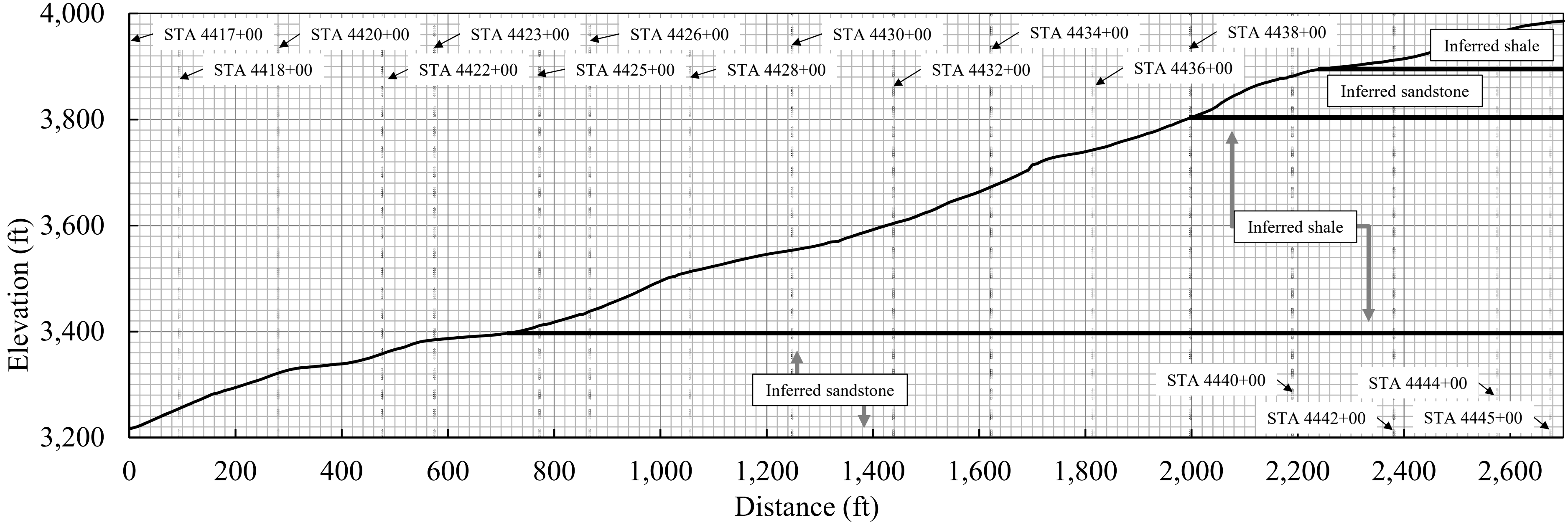

Notes:

1. Groundwater flow is anticipated to flow along the soil bedrock interface.
2. Profile alignment follows the centerline alignment of pipeline
3. Bedrock outcrops displayed are inferred units based on topographic expression, available LiDAR data, aerial photos, field photos, and field notes.
4. Where inferred bedrock outcrops are not observed in photographs, a veneer of soil or vegetation is assumed to obscure the outcrop.

Inferred Subsurface Profile
MP 73.20 to MP 73.50

ACP AP-1

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Houston, TX

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Attachment
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