Wells Truck Climbing Lane

Elko County

August 23, 2018

STATE OF NEVADA

DEPARTMENT OF TRANSPORTATION

MATERIALS DIVISION

GEOTECHNICAL SECTION

GEOTECHNICAL REPORT

WELLS TRUCK CLIMBING LANE

ELKO COUNTY

August 23, 2018

EA 73793

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1.0 INTRODUCTION

Navigating the Interstate 80 (I-80) corridor before summiting the Pequop Mountains, just east of Wells, in Elko County, NV is a stretch of winding mountainous terrain with steep grades. A site location map is presented in Appendix A, Figure 1. The eastbound lanes will be widened to accommodate a truck climbing lane that will alleviate congestion due to slow moving truck traffic. The proposed project extends approximately between Station “PE” 142+15.05 and Station “PE” 1171+26.47; where, the truck climbing lane lies roughly between Station “PEC” 628+89.37 and Station “PEC” 815+59.34 (a project location map is attached as Appendix A, Figure 2). The following report summarizes the results and recommendations from the geotechnical analysis of the proposed lane widening project.

2.0 PROJECT DESCRIPTION

Currently I-80 is a four-lane highway, consisting of two eastbound lanes and two westbound lanes, that traverse Nevada in an approximate east/west direction. The proposed truck climbing lane will create an additional eastbound lane that will require areas of both cut and fill. Cut slope inclinations reviewed in this report are modeled at a 1:1 (H:V) slope with a maximum vertical relief of 59 feet. Embankment fills are modeled at a 1.5:1 riprapped lined slope with a maximum vertical relief of 50 feet. While the total project scope includes mill and overlay pavement preservation and improvements to the Pequop Summit Interchange on-ramps, in addition to the construction of a new eastbound traffic lane; the following report is specific to the proposed cut and fill slopes necessary for the widening construction.

3.0 FIELD RECONNASSIANCE

A field reconnaissance at the proposed site location was conducted by NDOT’s Geotechnical Section on May 22, 2018. Field reconnaissance activities included observing the condition and performance of cut and embankment fill slopes, qualitatively estimating rock mass properties, and identifying areas of potential rockfall. A description of the individual embankment fill and rock cut slopes observed during our field reconnaissance is detailed below.

3.1 IN-SITU EMBANKMENT SLOPE

The embankment fill slope located approximately between Station “PEC” 664+00 and Station “PEC” 703+40 is constructed at a 1.5:1 inclination with riprap surfacing along the lower portion of the slope. A relatively small amount of established vegetation was observed along the embankment with a slightly denser vegetation pattern adjacent to the highway. A drainage channel located adjacent to the toe of the embankment, along with the embankment slopes, were dry at the time of our field reconnaissance. No evidence of global instabilities was noted during the field reconnaissance. Such signs of global instability include; settlement, cracking, bulging of the toe, or fresh/active raveling along the slope.

3.2 IN-SITU CUT SLOPES

At the time of the geotechnical field reconnaissance, cut slopes contained small quantities of erosional detritus at the toe of the slope and very little mature vegetation. The native surface located approximately between Station "PEC" 628+89.37 and Station "PEC" 765+00 consists of mature vegetation and sedimentary limestone and/or dolomite rock outcroppings.

Two in-situ cut slopes were observed during the field reconnaissance. The first is approximately located between Stations "PEC" 648+00 and "PEC" 654+20 and the second is approximately located between Stations "PEC" 729+00 and "PEC" 744+00; these slopes will be referred to as Cut Slope 1 and Cut Slope 2, respectively. No evidence of global instabilities was noted in either cut slope. Signs of global instability in cut slopes include; movement along dominant joints, cracking above the slope, bulging of the toe, or fresh/active raveling along the slope. Due to limited access to Cut Slope 2, conditions above the slope were not observed; however, no signs of instability along the slope face or along the toe were noted during the field reconnaissance. A detailed description of each cut slope is given in the following sections below.

3.2.1 CUT SLOPE 1: "PEC" 648+00 to "PEC" 654+20

Cut Slope 1 is characterized by a tabular limestone formation comprised of blocky jointing patterns. The dominant bedding consists of continuous, closely to moderately spaced, sub-horizontal planes that daylight into the slope face. Secondary widely spaced sub-vertical discontinuities were also observed. The maximum size detritus noted at the toe of the slope was three feet in diameter. Bedding along the eastern section of the slope was observed to be dipping at a slightly steeper angle than the western section. During our site visit in May a slight amount of moisture was observed above the slope and along the slope face; however, this was likely a seasonal seep due to snowmelt from the recent increase in temperature at the time of our visit. Overhangs, ranging on the order of several inches to two feet, were observed approximately 20 to 30 feet from the toe of the slope. A rockfall catchment mesh is present along the entire length of the slope, and was observed to be in good condition.

3.2.2 CUT SLOPE 2: "PEC" 729+00 to "PEC" 744+00

Cut slope 2 is also characterized by tabular limestone; however, jointing patterns were observed to be very blocky and very closely spaced. The dominant bedding was a closely spaced sub-horizontal plane. Three sets of secondary sub-vertical joints, spaced moderately to extremely wide, were also observed. The maximum size detritus at the toe of the slope was estimated to be eight inches. Proposed construction will not impact this cut slope; therefore, slope stability analyses have not been performed.

3.3 GEOLOGY AND SEISMICITY

The geology along the I-80 corridor, as it crosses the Pequop Mountain Range, is a combination of several units. Basins bounding the Pequop Mountains consist of Quaternary alluvial fan deposits with intersecting units of sandy silty wash deposits, derived from volcanic and sedimentary rock. The geology along the project corridor is defined by limestone and dolomite rock outcroppings, with areas of weathered colluvium derived from the above mentioned sedimentary parent outcroppings (Coats, 1987).

The project lies within the Basin and Range Province, also known as the Great Basin Province, which is typically characterized by large normal fault systems producing predominantly northern trending mountain ranges bounded by alluvial basins. Transecting I-80, within the project limits, is a northern section of the Independence Valley fault zone (see Appendix A, Figure 3 for a fault map). This is a well-defined normal fault with a slip rate of less than 0.2 mm/yr. A pseudo static design component is analyzed below in accordance with Section 11.5.4 of the American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) manual.

4.0 ANALYSES

It is important for slopes with significant vertical relief to be examined for potential global instabilities to be examined for potential rockfall launching mechanisms, and to verify the loads resulting from increased embankment fills do not pose settlement concerns. Results from these analyses are detailed below.

4.1 SLOPE STABILITY ANALYSES

Global stability analyses were conducted using SLIDE v.7.0, a two-dimensional limit equilibrium analysis program by Rocscience. Design parameters were estimated via the Mohr-Coulomb criterion for the fill slope and the Hoek-Brown strength criterion for the cut slope. A seismic coefficient of 0.075g was used in the pseudo static analysis, based on one half of the PGA as presented in the NDOT Structures Manual.

The AASHTO LRFD guidelines suggest a minimum Factor of Safety of 1.3 for static analyses; where, the National Highway Institutes (NHI) Circular No. 5 (FHWA-NHI-11-032) guidelines suggest a minimum Factor of Safety of 1.1 for pseudo static analyses. Factors of Safety from both static and pseudo static models are determined using the Spencer method.

4.1.1 EMBANKMENT FILL SLOPE STABILITY

Global stability analyses of the embankment fill slope were conducted using a circular failure method. The modeled fill section is representative of an embankment fill slope that is comprised of relatively large vertical relief (50 feet). The section also models a distributed load across the top of the 70 feet wide embankment, to represent vehicle loading.

Two embankment fill slope configurations have been modeled. The first incorporates a 2:1 slope, and the second incorporates a 1.5:1 slope with the slope protection consisting of a Class 700 riprap surfacing along the bottom nine feet of slope (measured parallel to the slope) and Class 300 riprap surfacing along the remaining slope. Material properties of the modeled fill are summarized in Table 1 below.

Table 1: Summary of Embankment Fill Material Properties

Material Property	Native Ground	Fill	Class 700 Riprap	Class 300 Riprap
Unit Weight (pcf)	125	125	145	145
Phi (deg)	34	34	45	45
Cohesion (psf)	25	100	0	0

Static analyses returned a minimum Factor of Safety above 1.3 and a minimum pseudo static Factor of Safety above 1.1. These results are summarized in Table 2 below.

Table 2: Summary of Results for Embankment Fill Global Stability Analyses

Global Stability Model	1.5:1 Slope	2:1 Slope
Static Minimum Factor of Safety	1.31	1.58
Pseudo Static Minimum Factor of Safety	1.14	1.33

Potential failure surfaces less than three feet deep are disregarded in this analysis, as they are considered surficial sloughing/raveling. Surficial sloughing/raveling may present a maintenance concern; however, it does not indicate potential global instabilities within the modeled slope. Detailed results from these analyses are provided in Appendix B.

4.1.2 CUT SLOPE STABILITY

During the field reconnaissance, the Unconfined Compressive Strength and dimensionless constants, m and s, of the rock mass were estimated using field index tests and observations of rock behavior in accordance with the *Rock Slopes Reference Manual* (FHWA HI-99-007). These values are presented below in Table 3.

Table 3: Summary of Cut Slope Rock Mass Characteristics

Rock Mass Property	Value
Unconfined Compressive Strength (UCS) (psi)	200
Unit Weight (psf)	145
m (dim)	0.575
s (dim)	0.00293

Global stability analyses of the modeled cut slope were conducted using a path search method for determining non-circular failure surfaces. The modeled cut slope is defined by relatively large vertical relief (60 feet) with undulating natural topography and a drainage ditch at the toe of the slope. This cut slope section is directly impacted by the widening construction and is modeled with a 1:1 inclination.

Table 4: Summary of Results for Cut Slope Analyses

Global Stability Model	Value
Static Minimum Factor of Safety	1.33
Pseudo Static Minimum Factor of Safety	1.17

The modeled cut slope returned a static and pseudo static Factor of Safety above the suggested minimums (Table 4). Detailed results from these analyses are provided in Appendix C.

4.2 POTENTIAL ROCKFALL HAZARD ANALYSIS

Dynamic properties of potential rockfall events were analyzed using the Colorado Rockfall Simulation Program (CRSP) v. 4.0, a statistical analysis program that uses the gravitational acceleration equations and the conservation of energy equations to determine the dynamic characteristics of potential rockfall events. Each simulation initiated 100 rockfall events along the slope face with each event consisting of a single rock, ½-foot, 1-foot, and 2-foot diameter in size. Rock destabilization was modeled with no initial velocity; thus, once mobilized, rocks were acting solely under the influence of gravity.

Without a rockfall barrier/fence, the 95% containment requirement presented in AASHTO cannot be met for rocks greater than ½-foot. If a 2½-foot high rockfall barrier is constructed along the proposed roadway edge, 99% containment is met for rocks up to 2-foot in diameter. A summary of results is presented in Table 5 below. Detailed results are attached as Appendix D.

Table 5: Summary of Results for Rockfall Hazard Analyses

Modeled Parameter	Rock Size (Diameter)		
	½-Foot	1-Foot	2-Foot
Catchment Type	Fence/Barrier Rail	Fence/Barrier Rail	Fence/Barrier Rail
Percent Contained (%)	>99	>99	>99
Max Bounce Height (ft)	---	1.06	2.31
Max Velocity (ft/sec)	---	11.87	23.72
Make Kinetic Energy (ft-lbs)	---	298	7,285

4.3 SETTLEMENT ANALYSIS

Settlement of the embankment fill slopes was analyzed using SETTLE3D, a three-dimensional vertical settlement and consolidation program by Rocscience. Using the Boussinesq stress computation method, a two-stage approach was employed to estimate the settlement resulting from the embankment widening. The first stage estimates the settlement from the existing embankment; where, the second stage estimates the settlement from the proposed widening construction. Model parameters are summarized in Table 6 below; where, detailed results are presented in Appendix E.

Table 6: Summary of SETTLE3D Model Parameters

Material Property	Value
Native Soil Unit Weight (psf)	125
Embankment Fill Unit Weight (psf)	125
Young’s Modulus (ksi)	10.4
Poisson’s Ratio (dim)	0.3

Model results indicate an additional estimated settlement of 1 to 1½ inches of settlement; therefore, settlement due to the increased loading conditions do not present a concern. Due to the granular nature of embankment fill material, elastic settlement is anticipated to occur relatively quickly during the construction process; with, minimal post construction settlement.

5.0 RECOMMENDATIONS

5.1 EMBANKMENT FILL SLOPES

Maximum recommended inclinations for embankment fill slopes are 2:1 or 1.5:1 with riprap surfacing protection along the entire slope. Subsidence of newly placed fills should be negligible, with estimated settlements between 1 to 1½ inches.

5.2 ROCK CUT SLOPES

Maximum recommended inclinations for rock cut slopes is 1:1. A rock catchment mesh is required to meet the 95% containment requirement presented in AASHTO. Rock catchment meshing should cover the entire exposed face of the cut slope.

5.3 SHRINK/SWELL FACTORS

Excavated bedrock material may be suitable for reuse as embankment fill material. Bedrock material excavated and recompacted in embankment fills should experience a quantity swelling of approximately 20%. Oversized material may be encountered and will need to be reprocessed to satisfy fill specifications outlined in the NDOT Standard Specifications for Road and Bridge Construction manual.

5.4 LIMITATIONS

Recommendations contained in this Geotechnical Report are based on information obtained from the proposed project plan set (at the date of this document), our field reconnaissance, global stability analyses, rockfall hazard analyses, settlement analyses, and observations from our Geotechnical Engineers. The nature and extent of subsurface variations may not be evident until construction takes place; therefore, this report may not fully quantify the natural variation of in-situ soil characteristics. If encountered construction conditions differ from those found in this report, or the scope is altered, the Geotechnical Section must be notified to evaluate in-situ conditions and/or new plan sets and provide additional recommendations, if necessary.

6.0 <u>REFERENCES</u>

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Hoek, E. and Brown, E.T. *Practical Estimates of Rock Mass Strength*. International Journal Rock Mechanics Mining Science, 34, 1165-1186, 1997.

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Munfakh et al. *Rock Slopes*, Reference Manual, Publication No. FHWA-NHI-99-007, NHI Course No. 13235 – Section 5, October 1998.

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U.S. Seismic Design Maps. https://earthquake.usgs.gov/designmaps/us/application.php. Accessed 1 March 2018.

APPENDIX A
Figures

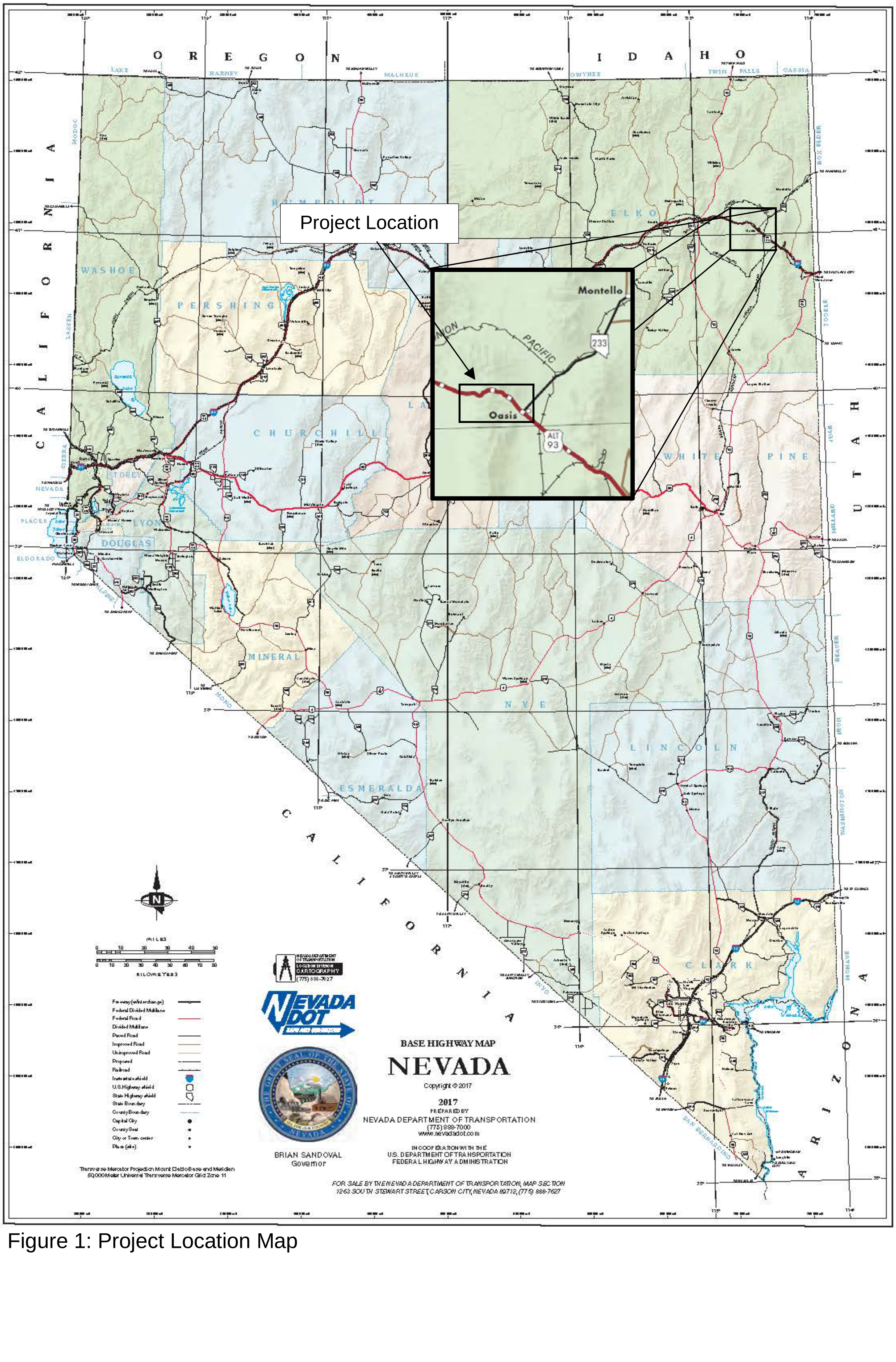


Figure 1: Project Location Map

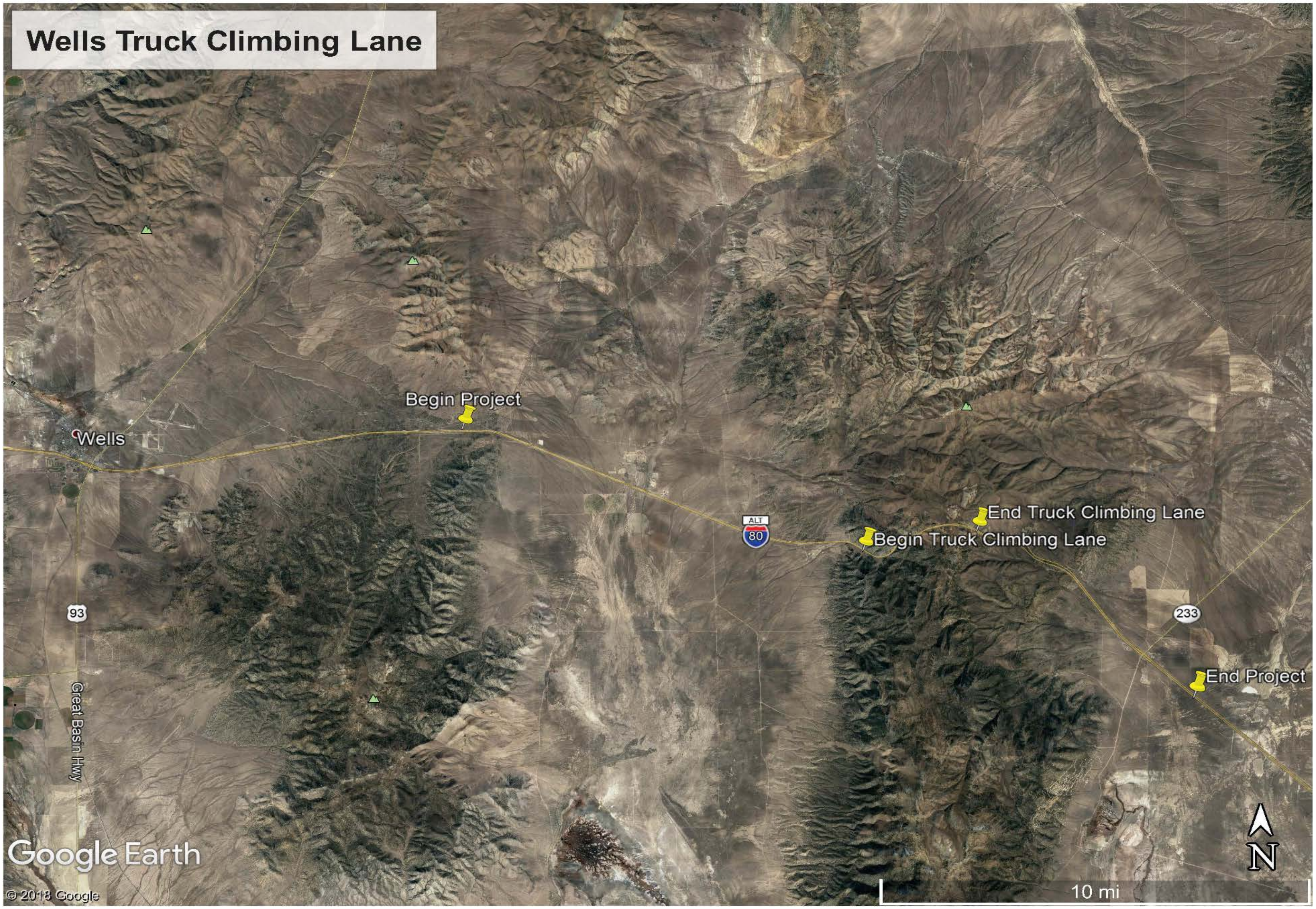


Figure 2a: Total Project Scope Location Map

Figure 2b: Truck Climbing Lane Location Map

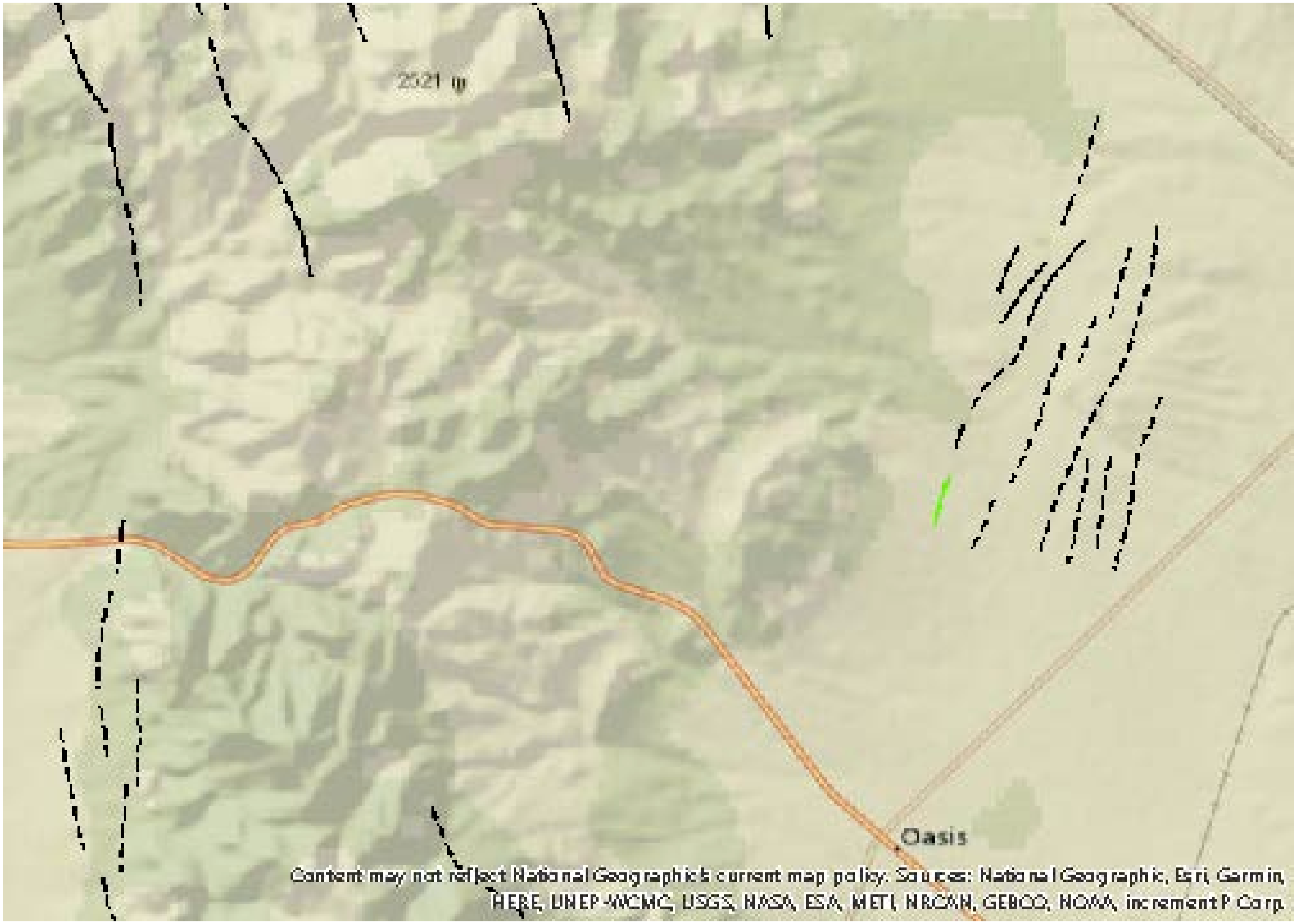
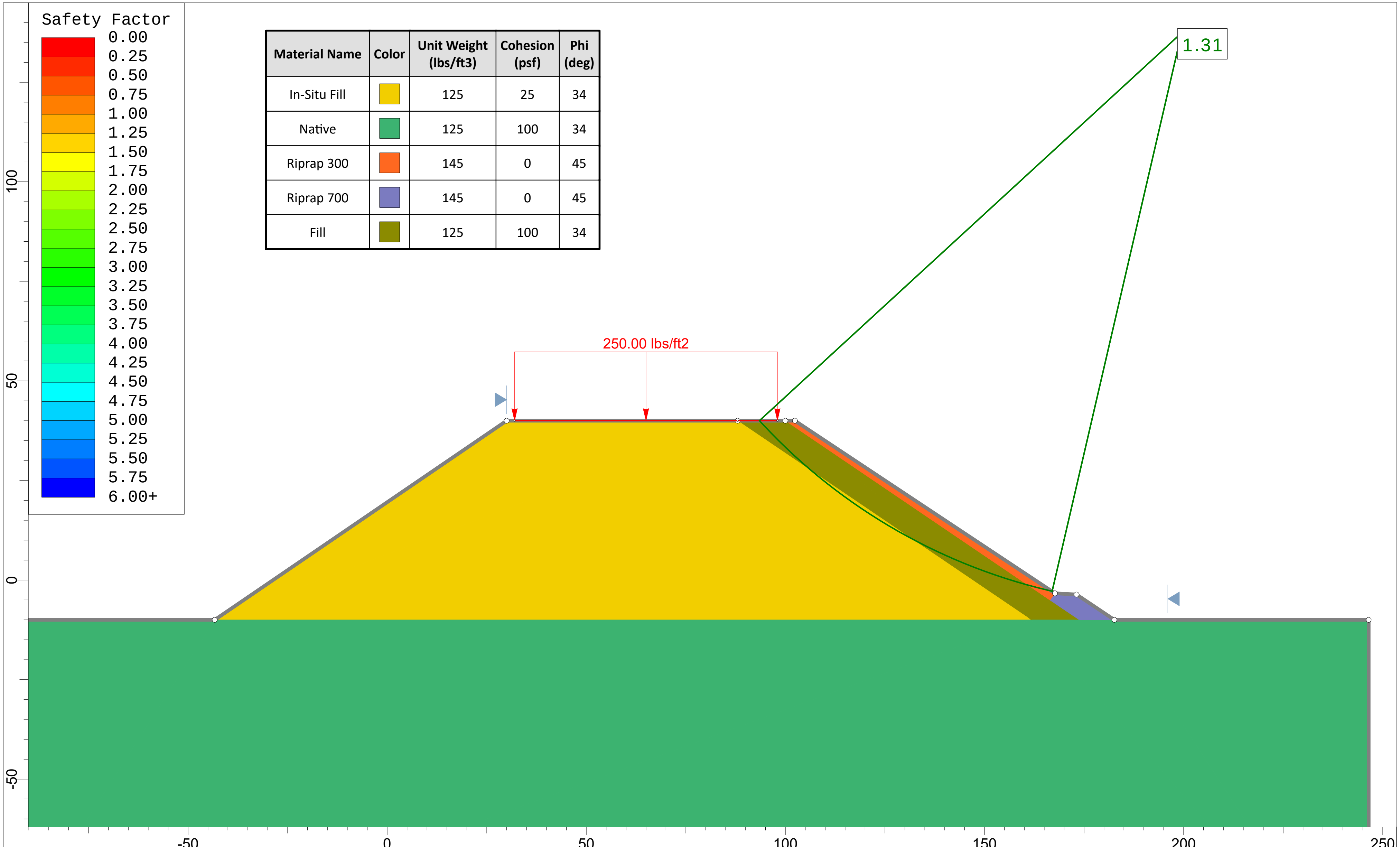


Figure 3: Fault Location Map.

APPENDIX B

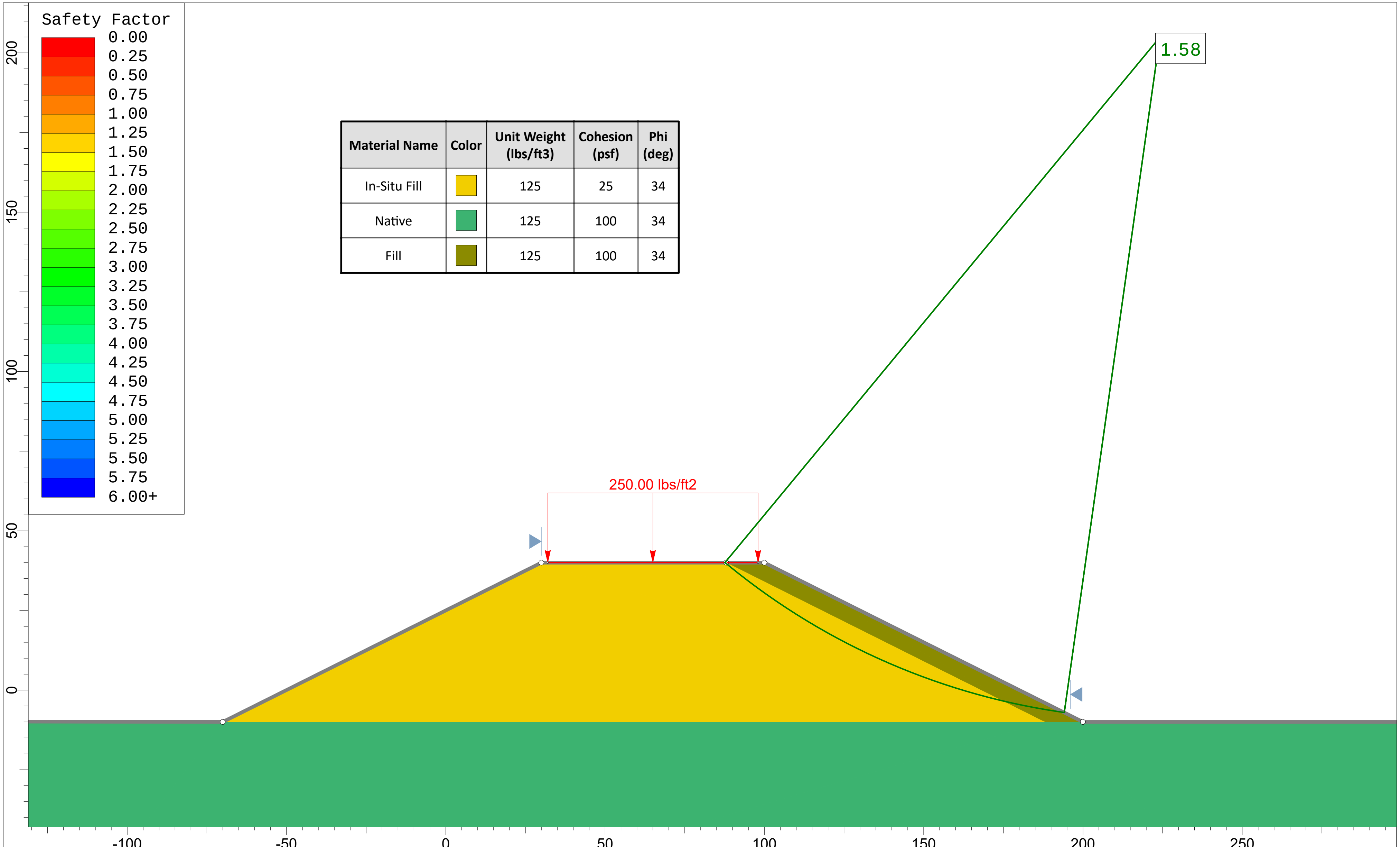
Global Stability Analyses: Embankment Fill



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
In-Situ Fill		125	25	34
Native		125	100	34
Riprap 300		145	0	45
Riprap 700		145	0	45
Fill		125	100	34

SLIDEINTERPRET 7.035

Project	Wells Truck Climbing Lane EA# 73793		
Analysis Description	Static Analysis of 1.5:1 Embankment Fill Slope: Station "PEC" 664+00 to "PEC" 703+00		
Drawn By	S. Jensen	*Scale* 1:400	*Company* NDOT
Date	6/11/2018, 2:07:58 PM	*File Name*	EmbankmentFill_1.5to1_Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
In-Situ Fill		125	25	34
Native		125	100	34
Fill		125	100	34

rocscience

SLIDEINTERPRET 7.035

Project	Wells Truck Climbing Lane EA# 73793
Analysis Description	Static Analysis of 2:1 Embankment Fill Slope: Station "PEC" 664+00 to "PEC" 703+00
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Scale	1:500
Company	NDOT
Date	6/11/2018, 2:07:58 PM
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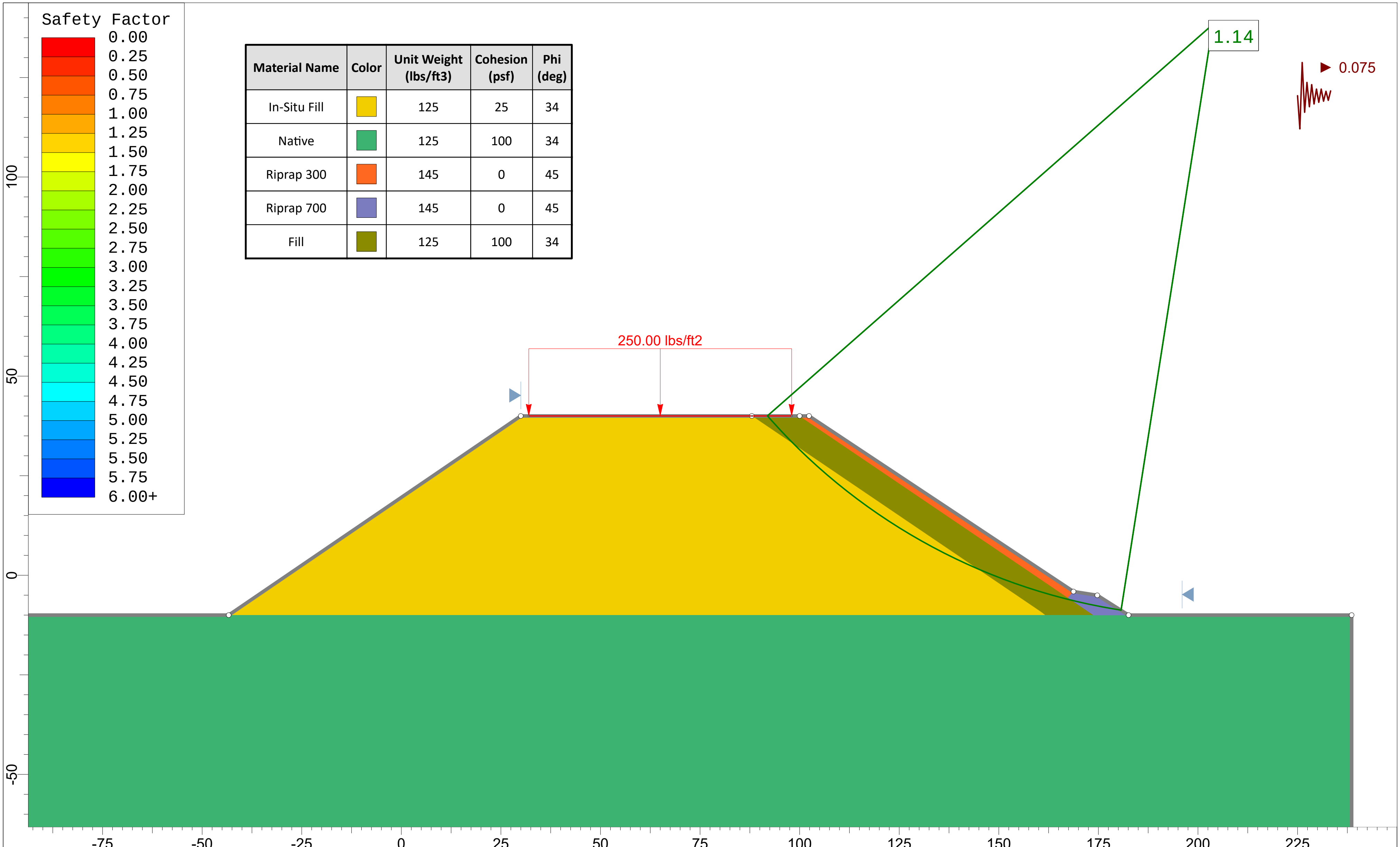

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
In-Situ Fill		125	25	34
Native		125	100	34
Riprap 300		145	0	45
Riprap 700		145	0	45
Fill		125	100	34

SLIDEINTERPRET 7.035

Project	Wells Truck Climbing Lane EA# 73793		
Analysis Description	Pseudo Static Analysis of 1.5:1 Embankment Fill Slope: Station "PEC" 664+00 to "PEC" 703+00		
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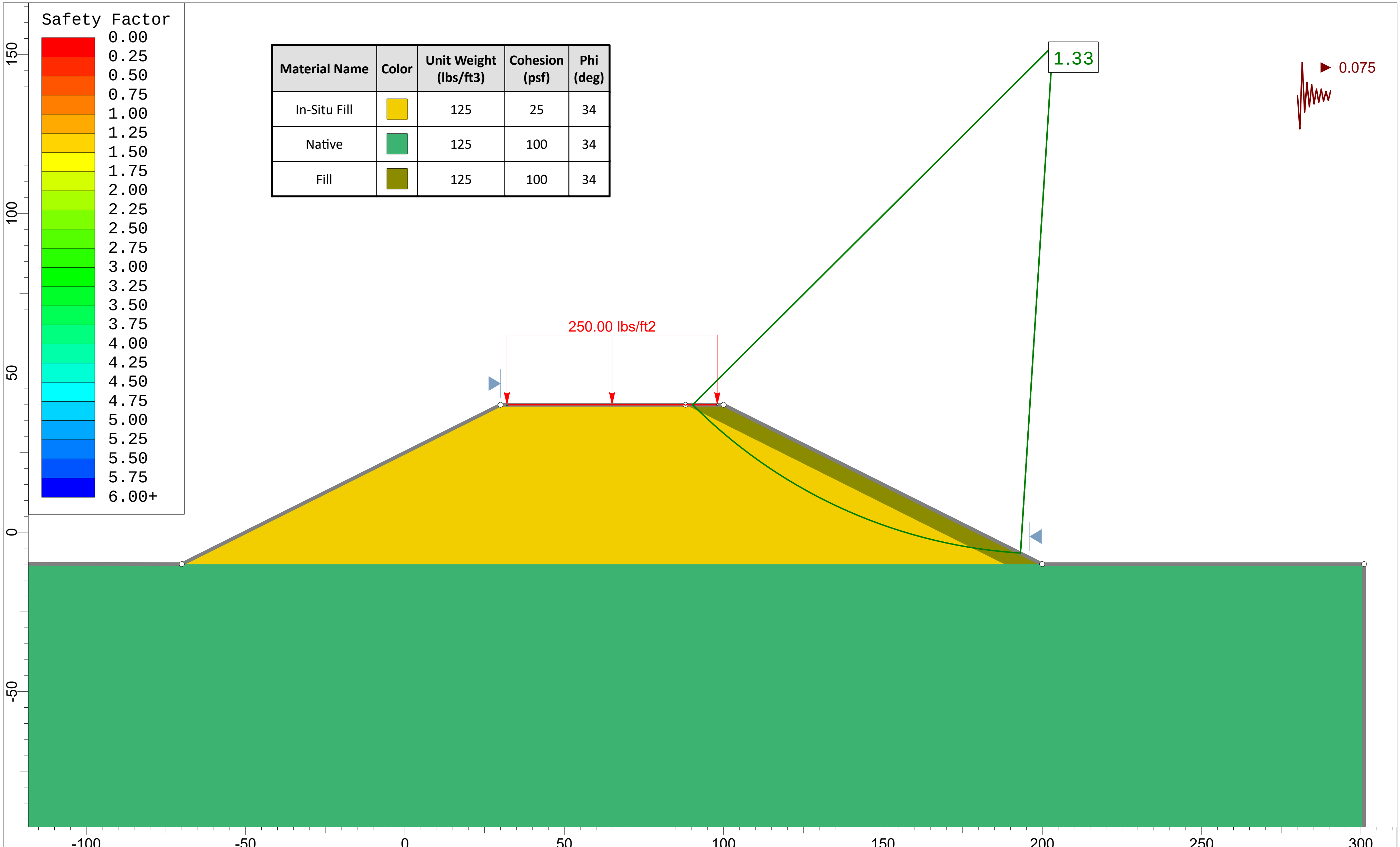

SLIDEINTERPRET 7.035

Project	Wells Truck Climbing Lane EA# 73793
Analysis Description	Pseudo Static Analysis of 2:1 Embankment Fill Slope: Station "PEC" 664+00 to "PEC" 703+00
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Company	NDOT
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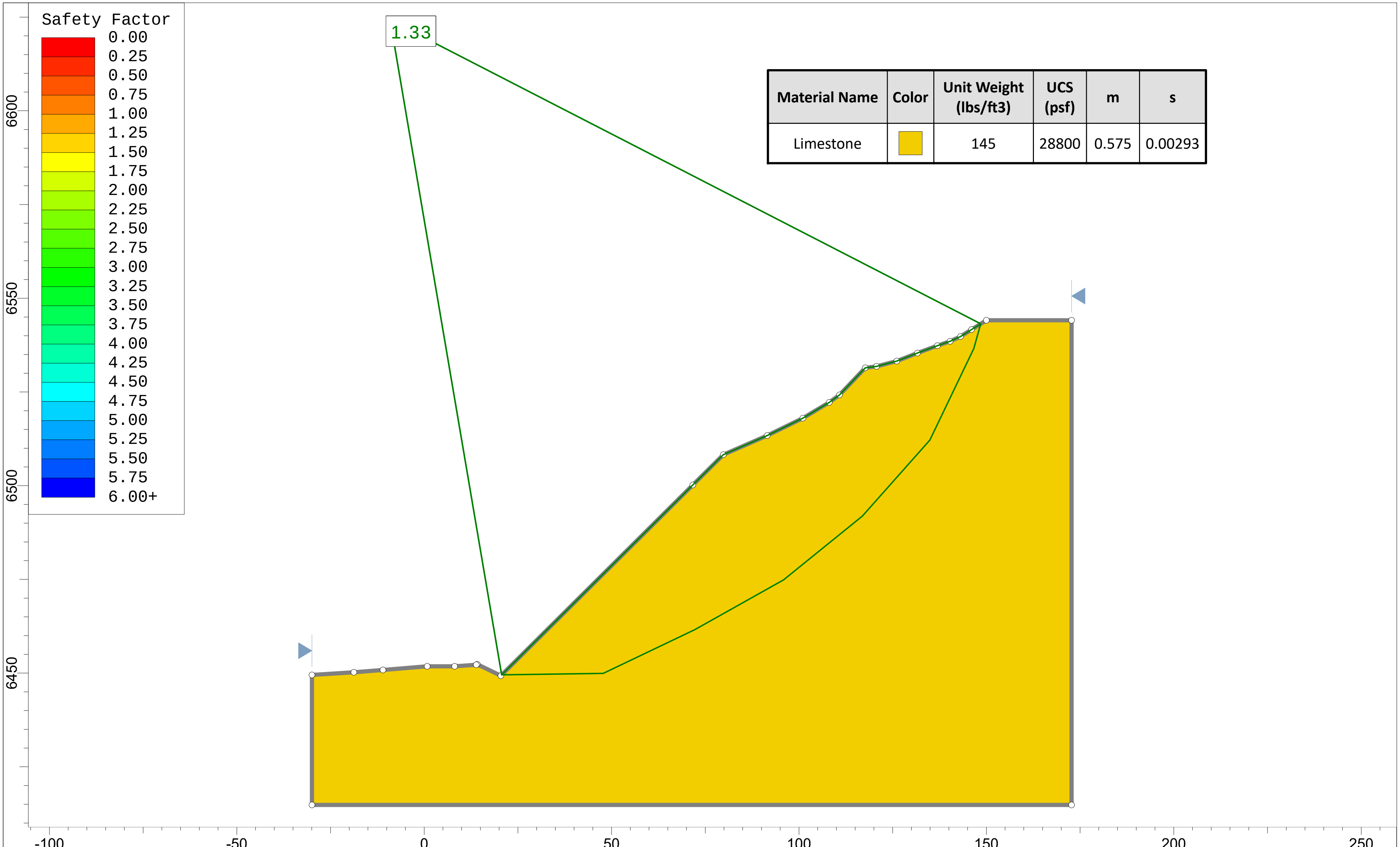
File Name	EmbankmentFill_2to1_PseudoStatic.slim

APPENDIX C

Global Stability Analyses: Cut Slope



Material Name	Color	Unit Weight (lbs/ft3)	UCS (psf)	m	s
Limestone		145	28800	0.575	0.00293

Project	Wells Truck Climbing Lane EA# 73793		
Analysis Description	Static Analysis of Cut Slope 1: Station "PEC" 648+00 to "PEC" 654+00		
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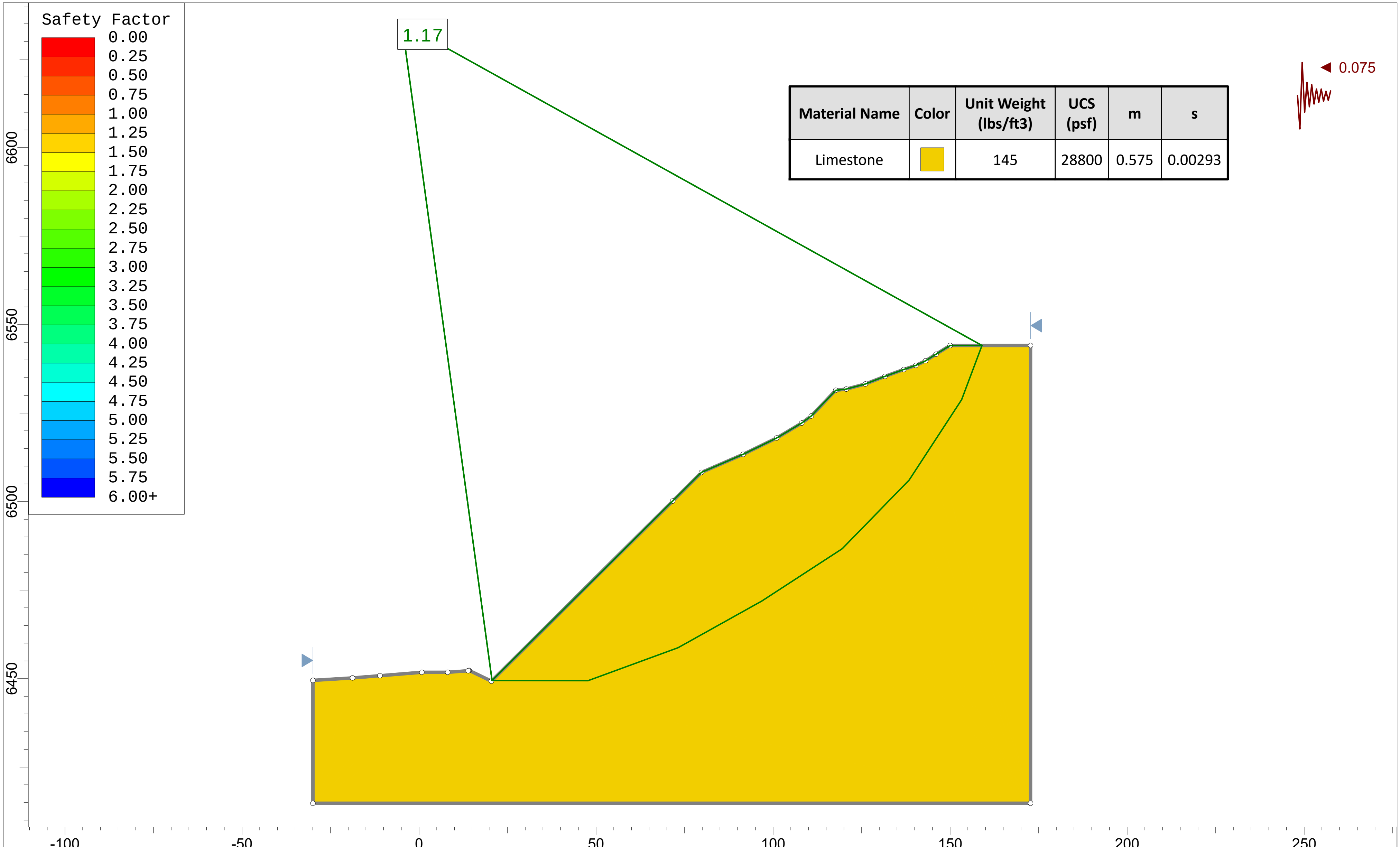
SLIDEINTERPRET 7.035



Material Name	Color	Unit Weight (lbs/ft3)	UCS (psf)	m	s
Limestone		145	28800	0.575	0.00293

SLIDEINTERPRET 7.035

Project	Wells Truck Climbing Lane EA# 73793
Analysis Description	Pseudo Static Analysis of Cut Slope 1: Station "PEC" 648+00 to "PEC" 654+00
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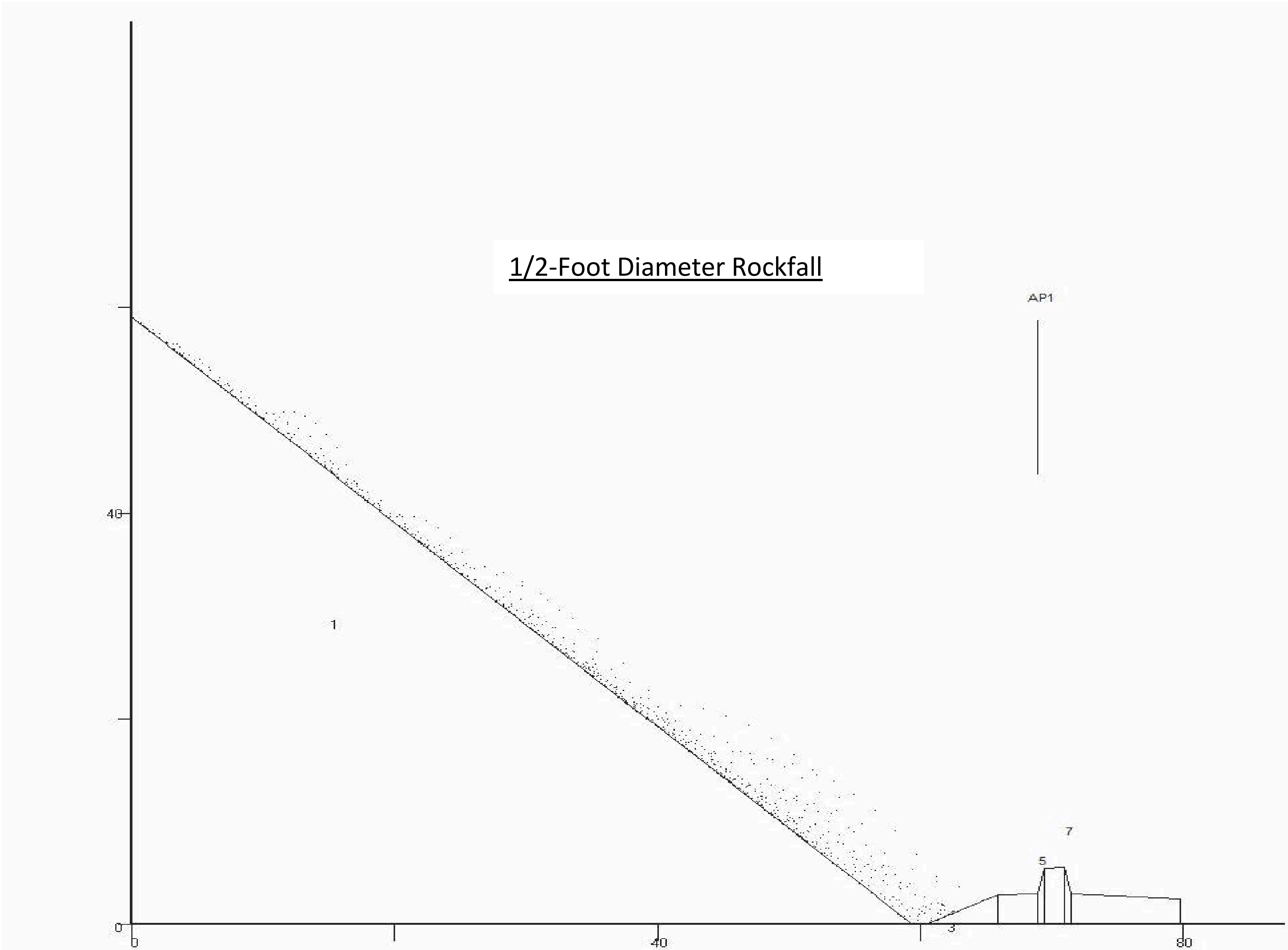
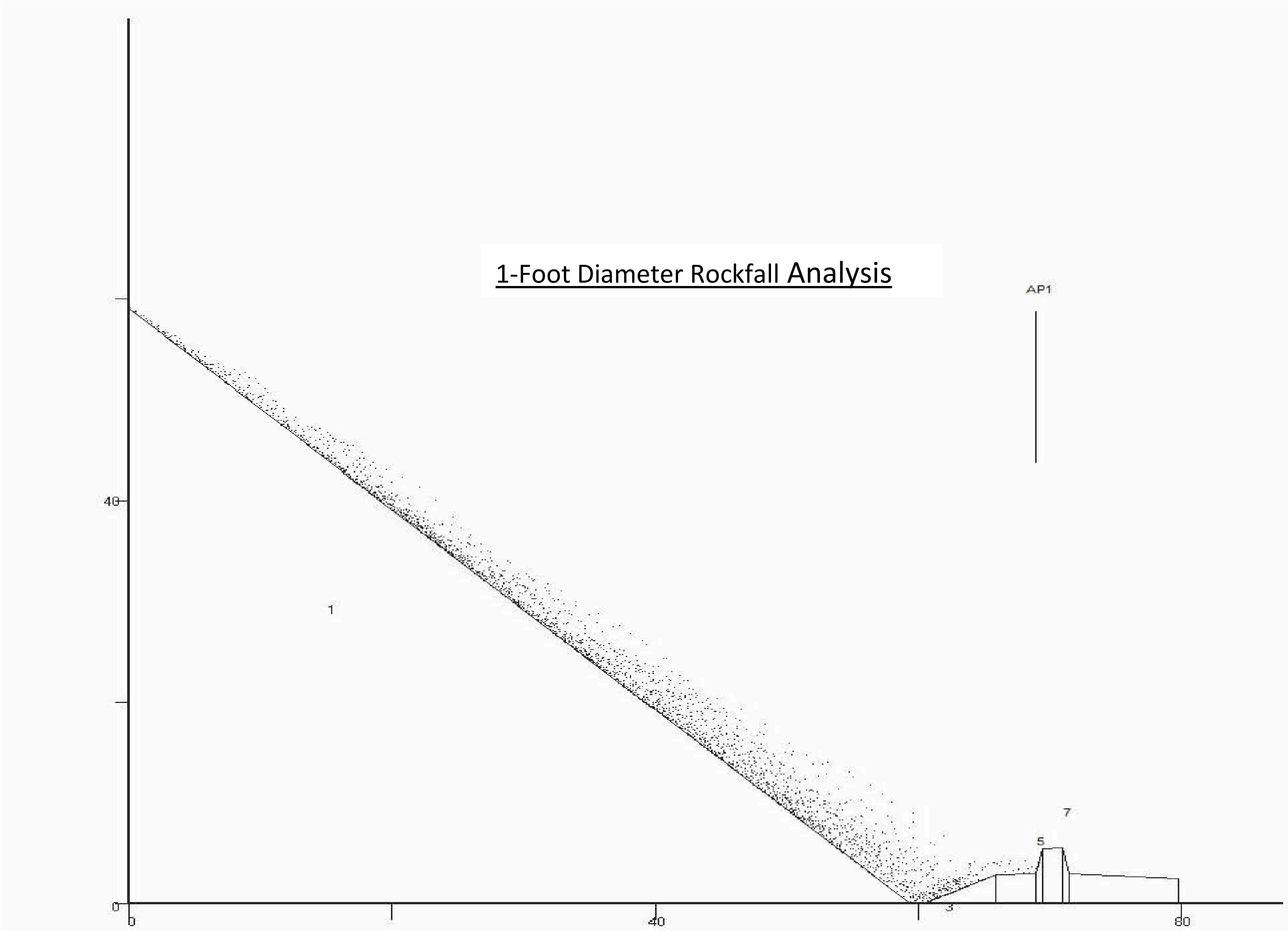
APPENDIX D
Rockfall Hazard Analyses


1/2-Foot Diameter Rockfall
AP1
40
1
7
5
0
3
0
40
80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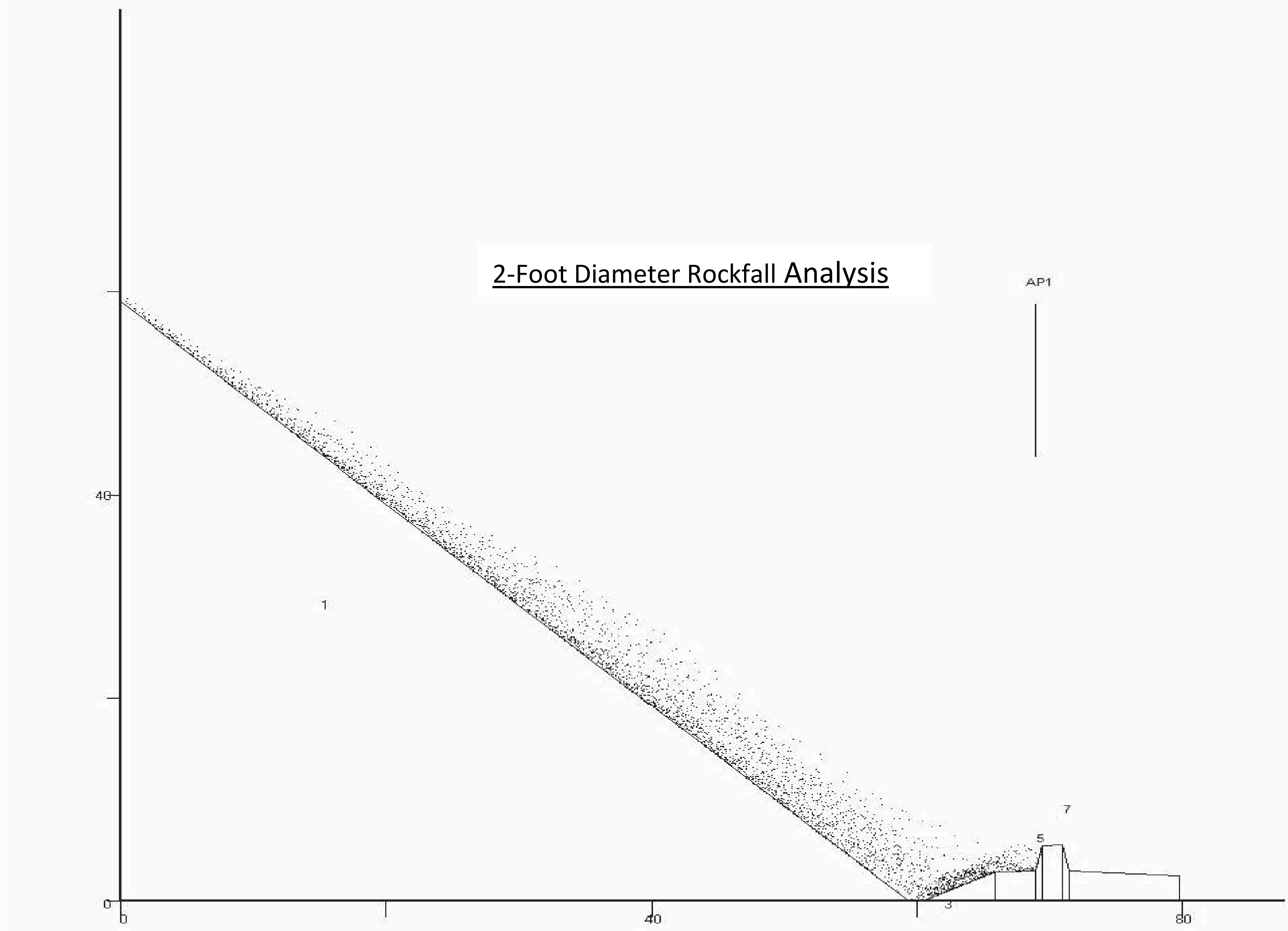
40
80


2-Foot Diameter Rockfall Analysis
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40
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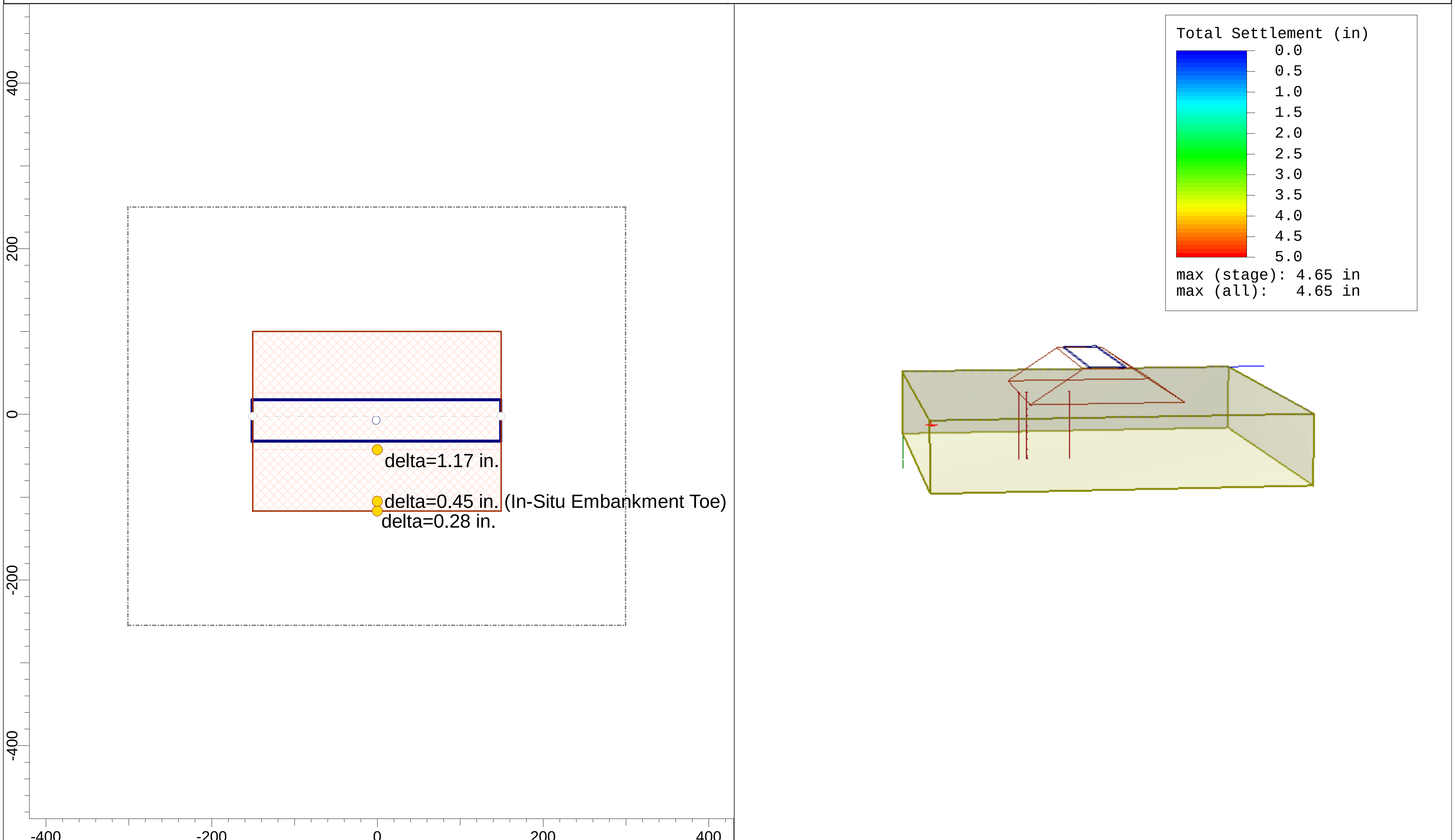
APPENDIX E

Embankment Fill Settlement Analysis

Stage 2 | Data Type: Total Settlement



Project	Wells Truck Climbing Lane EA# 73793		
Analysis Description	Embankment Fill Settlement Analysis		
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SETTLE3D 2.015