Part 18 of 19 of Appendices for Resource Report No. 13 LNG

J.7 - LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report

Document Number:	Description:	Revision:	Appendix:
USAL-FG-GRZZZ-00-002016-009	LNG Facilities Onshore Integrated Site Characterization And Geotechnical Engineering Report	Rev 0	Public

Confidential

LNG FACILITIES ONSHORE INTEGRATED SITE CHARACTERIZATION AND GEOTECHNICAL ENGINEERING REPORT

USAL-FG-GRZZZ-00-002016-009

Rev	Date	Revision Description	Originator	Reviewer / Endorser	Response Code	Approver
A	22-Dec-16	Issued for Review	S. Jung / E. Terrell / B. Binatli	P. Wong	2	A. S. Shethji T. A. Posey
0	3-Feb-17	Issued for Use	S. Jung / E. Terrell / B. Binatli			A. S. Shethji T. A. Posey

Document Control No.	Country	Facility	Originator	Discipline	Type	Sub-Type	Location	Sequence	Identifier
	US	AL	FG	G	R	ZZZ	00	002016	009

Alaska LNG™

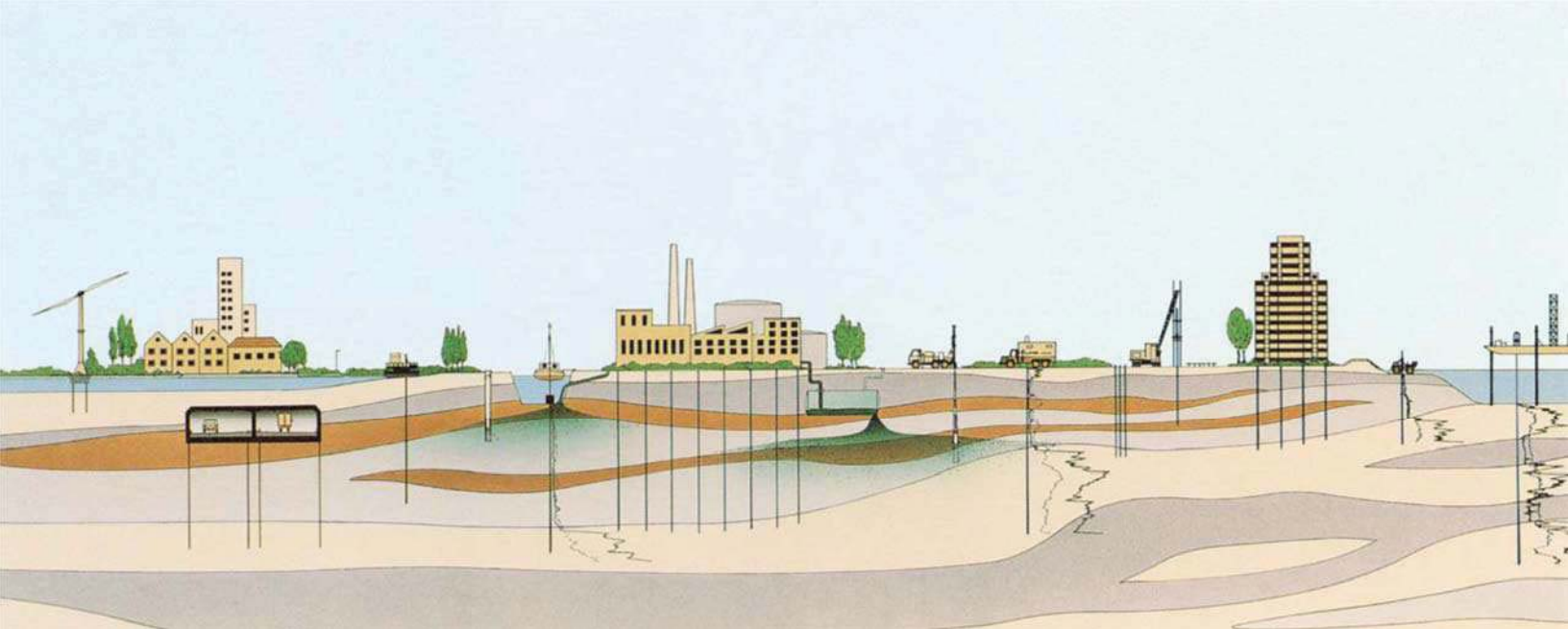
LNG FACILITIES ONSHORE INTEGRATED SITE CHARACTERIZATION AND GEOTECHNICAL ENGINEERING REPORT

ALASKA LNG PROJECT
NIKISKI, ALASKA

AKLNG REPORT NO. USAL-FG-GRZZZ-00-002015-009

REPORT NO. 04.1016001-12
EXXONMOBIL ALASKA LNG LLC (EMALL)
HOUSTON, TEXAS

Rev	Date	Revision Description	Originator	Reviewer	Approver
A	22-Dec-16	Issued for Review	S. Jung / E. Terrell / B. Binatli	A. S. Shethji	T. A. Posey
0	3-Feb-17	Issued for Use	S. Jung / E. Terrell / B. Binatli	A. S. Shethji	T. A. Posey



FUGRO CONSULTANTS, INC.

6100 Hillcroft (77081)
Houston, Texas 77274
Tel: (713) 369-5400
P.O Box 740010
Fax: (713) 369-5518

Report No. 04.10160001-12
February 3, 2017

ExxonMobil Alaska LNG LLC (EMALL)
10613 W. Sam Houston Pkwy N, Suite 500
Houston, TX, 77064

Attention: Mr. Patrick Wong
Geotechnical Engineer/Technical POC

LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report
Alaska LNG Project
Nikiski, Alaska

Fugro Consultants, Inc. (Fugro) is pleased to present this integrated site characterization and geotechnical engineering report for the onshore facilities of the Alaska LNG Project (AKLNG) located in Nikiski, Alaska. Our services were authorized under Service Work Order No. AKLNG-FUG-US-005, dated February 5, 2015 in accordance with the Service Agreement No. A2275592 between Fugro and ExxonMobil Global Services Company, dated October 29, 2012.

The AKLNG Project includes building a natural gas Liquefaction Plant (LNG) with a marine terminal on the south central Alaskan coast, a Gas Treatment Plant (GTP) at the North Slope of Alaska, and a gas pipeline that will carry treated gas from the GTP to the LNG Plant. This report addresses the various onshore facilities planned as a part of the proposed LNG Plant. Marine facilities of the project were addressed in a separate Fugro report (USAL-FG-GRZZZ-90-002015-012[1]). Fugro has been performing geophysical and geotechnical site investigations (G&G) for the proposed AKLNG Project since August 2014. Various studies performed by Fugro during 2014, 2015 and 2016 G&G programs include; topographic surveys, marine geophysical surveys (chirp sub-bottom profiler, boomer, air gun and seafloor mapping), marine geotechnical studies (borings, PS Logging and Vibracore sampling), geologic hazards studies, hydrogeologic studies (monitoring wells, observation wells, aquifer pump testing wells and continuous groundwater elevation readings), onshore geophysical (shallow and deep) and geotechnical investigations, and PSHA/Site response analysis.

[1] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-15, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012, Rev.0, dated September 06, 2016.

Integrated site characterization and engineering reports (USAL-FG-GRZZZ-00-000006-000[2] and USAL-FG-GRZZZ-00-002015-009[3]) were prepared based on the data collected during the 2014 and 2015 G&G programs. This report includes a brief overview of the previous studies conducted by Fugro and incorporates the data collected during the 2016 G&G program. Results of our engineering studies that include integration of the geotechnical and geophysical data, developing a site subsurface model, interpretation of soil design parameters, and providing geotechnical recommendations to support the proposed onshore facilities, are presented in this report. It should be noted that the recommendations provided in this report are based on data collected during the 2014, 2015 and 2016 G&G programs, therefore this report supersedes previously submitted Fugro reports USAL-FG-GRZZZ-00-000006-000 and USAL-FG-GRZZZ-00-002015-009.

[2] Fugro Consultants, Inc. (Fugro), 2016, Integrated Site Characterization and Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-12, AKLNG Document No. USAL-FG-GRZZZ-00-000006-000, Rev.1, dated April 18, 2016.

[3] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-14, AKLNG Document No. USAL-FG-GRZZZ-00-002015-009, Rev.0, dated August 30, 2016.

We appreciate the opportunity to be of service to ExxonMobil Alaska LNG LLC (EMALL). Please call us at (713) 369-5400 if you have any questions or comments concerning this report, or when we may be of further assistance.

Sincerely,
FUGRO CONSULTANTS, INC.
TBPE Firm Registration No. 299

Earnest Terrell, E.I.T.
Project Professional

Baret Binatli, E.I.T.
Project Professional

Sochan Jung, Ph.D., P.E. (Texas)
Senior Project Professional

Abhishek S. Shethji, P.E. (Texas)
Project Manager

Thomas A. Posey, P.E. (Alaska)
Engineering Manager

STATE OF ALASKA
49TH
07 February 2017
THOMAS A. POSEY
No. CE13107
REGISTERED PROFESSIONAL ENGINEER

Copies Submitted: Aconex Document Control System
SJ/BB/AS/TAP (R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\10_Reporting\-12 Onshore Integrated Report\2.Rev.0\USAL-FG-GRZZZ-00-002016-009-n_0 LNG Onshore Integrated Report.docx)

TABLE OF CONTENTS

Page

TABLE OF CONTENTS IV

LIST OF TABLES IX

ILLUSTRATIONS XII

APPENDICES XIV

1.0 INTRODUCTION 1-1
- 1.1 Project Description 1-1
- 1.2 Purposes and Scope 1-3
- 1.3 Report Format 1-4
- 1.4 Applicability of Report 1-7
- 1.5 Limitations 1-8
- 1.6 Unit Conversions and Datums 1-8

2.0 RELEVANT STUDIES 2-1
- 2.1 General 2-1
- 2.2 Previous Reports 2-1
- 2.3 Land Survey 2-3
- 2.4 Marine Geophysical Survey 2-4
- 2.5 Marine Geotechnical Reports 2-5
- 2.6 Field Geologic Mapping 2-6
- 2.7 Shallow Onshore Geophysical Surveys 2-7
  - 2.7.1 Electrical Resistivity Tomography Survey – Results and Conclusions 2-8
  - 2.7.2 Shallow Seismic Survey – Results and Conclusions 2-9
- 2.8 Deep Seismic Surveys 2-9
- 2.9 Onshore Geotechnical Exploration 2-10
- 2.10 Onshore Geologic Hazards 2-11
- 2.11 Probabilistic Seismic Hazard Assessment (PSHA) & Site Response 2-14
- 2.12 Hydrogeological Study 2-17
- 2.13 Cold Region Considerations 2-20

3.0 INTEGRATED SITE MODEL 3-1
- 3.1 General 3-1
- 3.2 Topography and Geomorphology 3-1
- 3.3 Subsurface Model 3-2

4.0 SITE PREPARATION 4-1
- 4.1 General 4-1
- 4.2 Low-Lying Areas 4-2

4.3 Ground Improvement 4-3
4.4 Site Preparation for Cut and Fill Areas 4-3
4.4.1 Reuse of On-site Material 4-3
4.4.2 Drainage Considerations 4-4
4.4.3 Cut and Fill Slopes 4-5
4.4.4 Earthwork Considerations 4-5
4.5 Site Grade Raise and Settlement 4-6

5.0 SELECTED DESIGN PROFILES AND INTERPRETED ENGINEERING PARAMETERS 5-7
5.1 General 5-7
5.2 Engineering Areas 5-7
5.3 Soil Design Parameters 5-8
5.3.1 Evaluation of Field and Laboratory Test Results 5-8
5.3.2 SPT N Values 5-10
5.3.3 Unit Weight 5-11
5.3.4 Mineralogy and Morphology 5-11
5.3.5 Particle Degradation 5-11
5.3.6 Friction Angle 5-12
5.3.7 Undrained Shear Strength 5-13
5.3.8 Compressibility 5-13
5.3.9 Frost Susceptibility 5-14
5.4 Selected Soil Design Profiles and Parameters 5-15

6.0 SETTLEMENT DUE TO SITE GRADE RAISE 6-1
6.1 General 6-1
6.2 Site Grade Raise Material 6-1
6.3 Method of Analysis 6-1

7.0 FOUNDATION TYPES AND FEASIBILITY 7-1
7.1 General 7-1
7.2 Foundation Types 7-2
7.3 Seismic Design Considerations 7-4
7.4 Foundation Protection and Frost Considerations 7-4

8.0 LNG TANK SHALLOW FOUNDATION RECOMMENDATIONS 8-1
8.1 Tank Loading and Settlement Criteria 8-1
8.2 Tank Pad Preparation 8-2
8.3 Soil Parameters 8-2
8.4 Mat Foundation Recommendations 8-4
8.4.1 Foundation Depth 8-4
8.4.2 Allowable Net Bearing Pressure 8-4
8.4.3 Resistance to Lateral Loads 8-4
8.4.4 Tank Stability 8-5

8.5 Settlement Estimate 8-5
8.6 Hydrotesting Recommendations 8-7
8.6.1 Survey Control 8-7
8.6.2 Loading 8-7
8.6.3 Data Evaluation 8-7

9.0 SHALLOW FOUNDATION RECOMMENDATIONS 9-1
9.1 Foundation Depths 9-1
9.2 Allowable Bearing Capacity 9-1
9.3 Resistance to Uplift Loads 9-2
9.4 Resistance to Overturning Loads 9-3
9.5 Resistance to Lateral Loads 9-3
9.6 Settlement Estimates 9-3
9.7 Coefficient of Subgrade Reaction for Mat Foundations 9-6
9.8 Insulation Recommendations 9-7

10.0 SLAB-ON-GRADE FOUNDATION RECOMMENDATIONS 10-1
10.1 General 10-1
10.2 Bearing Capacity Recommendations 10-1
10.3 Insulation Recommendations 10-1

11.0 DEEP FOUNDATION DESIGN RECOMMENDATIONS 11-1
11.1 General 11-1
11.2 Soil Parameters 11-2
11.3 Axial Capacity 11-2
11.4 Adfreeze Stresses 11-3
11.5 Negative Skin Friction 11-4
11.6 Settlement Considerations 11-5
11.7 Axial Group Effects 11-6
11.8 Lateral Capacity 11-6
11.9 Lateral Group Effects 11-7
11.10 Load Tests 11-8
11.11 Frost Considerations for Pile Caps & Grade Beams 11-9

12.0 BLUFF SLOPE STABILITY 12-1
12.1 General 12-1
12.1.1 Bluff Stability in Area 1 (Heavy Haul Road Area) 12-2
12.1.2 Bluff Stability in Area 2 (Storage Tank Area) 12-3
12.1.3 Bluff Stability in Area 3 (Outfall) 12-5
12.2 Soil Parameters 12-5
12.2.1 Soil Parameters – Area 1 (Heavy Haul Road) 12-5
12.2.2 Soil Parameters – Area 2 (Storage Tank Area) 12-5
12.2.3 Soil Parameters – Area 3 Outfall 12-6

12.3 Loading Conditions 12-7
12.4 Method of Analysis 12-8
12.4.1 Static Analyses 12-8
12.4.2 Pseudo-Static Analyses and Dynamic Analyses 12-8
12.5 Slope Stability Results 12-9
12.5.1 Static Slope Stability Results – Area 1 (Heavy Haul Road Area) 12-9
12.5.2 Pseudo-Static Slope Stability Results – Area 1 (Heavy Haul Road Area) 12-9
12.5.3 Static and Pseudo-Static Slope Stability Results – Area 2 (Storage Tank Area) 12-10
12.5.4 Dynamic Slope Stability Results – Area 2 (Storage Tank Area) 12-11
12.5.5 Static Slope Stability Results – Area 3 (Outfall) 12-13
12.5.6 Pseudo-Static Slope Stability Results – Area 3 (Outfall) 12-14
12.6 Slope Protection 12-14

13.0 BELOW-GRADE STRUCTURES 13-1
13.1 General 13-1
13.2 Lateral Earth Pressures 13-1
13.3 Excavation Considerations 13-2
13.4 Backfill 13-2

14.0 DYNAMIC SOIL PROPERTIES 14-1
14.1 General 14-1
14.2 Dynamic Shear Modulus 14-1
14.3 Material Damping and Poisson's Ratio 14-2
14.4 Modulus of Elasticity 14-4
14.5 Dynamic Spring Constants 14-4

15.0 PAVEMENT RECOMMENDATIONS 15-1
15.1 General 15-1
15.2 Site Preparation 15-1
15.3 Subgrade 15-2
15.4 Pavement Sections 15-2
15.5 Drainage 15-4
15.6 Heavy Haul Road 15-4

16.0 SOIL CORROSION POTENTIAL 16-1
16.1 General 16-1
16.2 Laboratory Test Results 16-1
16.3 Corrosion of Buried Steel 16-3
16.4 Degradation of Concrete 16-4

17.0 CONSTRUCTION CONSIDERATIONS 17-1
17.1 Site Preparation 17-1

17.2 Shallow Open-Cut Excavations 17-1
17.3 Fill Materials and Placement 17-2
17.3.1 Structural Fill 17-2
17.3.2 Select Fill 17-3
17.3.3 General Fill 17-3
17.3.4 Flowable Fill 17-4
17.4 Groundwater Considerations 17-4
17.5 Driven Pile Installation 17-5
17.5.1 Pile Drivability 17-5
17.5.2 Detailed Pile Driving Specification 17-8
17.5.3 Pile Driving Equipment 17-9
17.5.4 Driven Pile Installation Criteria 17-10
17.5.5 Pile Load Test Program 17-10
17.5.6 Dynamic PDA Testing 17-11
17.5.7 Pile Driving Records 17-11
17.6 Cast-in-Place Concrete Pile Installation 17-11
17.6.1 Contractor Experience and Equipment 17-11
17.6.2 Concrete Mix 17-12
17.6.3 Pile Installation and Monitoring 17-12
17.6.4 Crosshole Sonic Logging 17-13
17.7 Thermal Integrity Profiler 17-14
17.8 Construction Monitoring 17-14

LIST OF TABLES

Page

Table 1.1: Summary of Fugro Reports Developed for 2014, 2015 and 2016 G&G Programs 1-5
Table 1.2: Conversion Units 1-8
Table 2.1: PGA Horizontal Component at the Ground Surface (V_{s30} = 885 ft/s) 2-15
Table 2.2: PGA Vertical Component at the Ground Surface (V_{s30} = 885 ft/s) 2-16
Table 2.3: Climatological Data 2-21
Table 2.4: Estimated Maximum Depth of Frost Penetration 2-22
Table 3.1: Generalized Subsurface Model 3-5
Table 4.1: Engineering Areas 4-2
Table 5.1: Engineering Areas with Design Elevations 5-7
Table 5.2: Laboratory Tests on Composite Soil Samples – 2014 G&G program 5-9
Table 5.3: Laboratory Tests on Composite Soil Samples – 2015 G&G program 5-9
Table 5.4: Laboratory Tests on Composite Soil Samples – 2016 G&G program 5-10
Table 5.5: Summary of Direct Shear Tests and Correlated Friction Angles 5-12
Table 5.6: Selected Soil Design Parameters – Area 1 (Heavy Haul Road Area, HA1-HA1') 5-15
Table 5.7: Selected Soil Design Parameters – Area 2A (Storage Tank Area) 5-15
Table 5.8: Selected Soil Design Parameters – Area 2B (Storage Tank Area) 5-16
Table 5.9: Selected Soil Design Parameters – Area 2C (Storage Tank Area) 5-16
Table 5.10: Selected Soil Design Parameters – Area 3 (Condensate and Admin Area) 5-17
Table 5.11: Selected Soil Design Parameters – Area 4A (Process Train Area) 5-17
Table 5.12: Selected Soil Design Parameters – Area 4B (Process Train Area) 5-18
Table 5.13: Selected Soil Design Parameters – Area 5 (Utility Area) 5-18
Table 5.14: Selected Soil Design Parameters – Area 6 (Laydown Area) 5-19
Table 5.15: Selected Soil Design Parameters – Area 8 (ASRC Area, B-176) 5-19
Table 5.16: Selected Soil Design Parameters – Area 8 (ASRC Area, B-183) 5-20
Table 6.1: Soil Profile for Grade Raise Settlement Analysis – Area 2B 6-2
Table 6.2: Results of Settlement Analysis for Site Grade Raise – Area 2B 6-3
Table 7.1: Geotechnical Recommendation Sections 7-2

Table 8.1: Details of LNG Tanks 8-1
Table 8.2: FERC Guidelines for LNG Tank Settlements 8-1
Table 8.3: Selected Soil Design Parameters – Area 2A (North Tank) 8-3
Table 8.4: Selected Soil Design Parameters – Area 2B (South Tank) 8-3
Table 8.5: Results of Tank Settlement Analysis 8-6
Table 9.1: Allowable Net Bearing Pressure for Shallow Foundations 9-2
Table 9.2: Design Profile for Area 2B Settlement Evaluation 9-4
Table 9.3: Design Profile for Area 3 Settlement Evaluation 9-4
Table 9.4: Design Profile for Area 5 Settlement Evaluation 9-5
Table 11.1: Summary of Deep Foundation Recommendations 11-1
Table 11.2: Required Minimum Pile Penetrations for Lateral Capacity 11-7
Table 11.3: Recommended Reduction Factors for Lateral Group Effect 11-8
Table 12.1: Soil Parameters for Slope Stability Analysis – Area 1 (HA1, HA2 and HA3) 12-5
Table 12.2: Soil Parameters for Slope Stability Analysis – Area 2 (North Tank), DP2A-S (Bluff) 12-6
Table 12.3: Soil Parameters for Slope Stability Analysis – Area 2 (North Tank), DP2A-S (Beach)12-6
Table 12.4: Soil Parameters for Slope Stability Analysis – Area 3, DP3-S (Bluff) 12-7
Table 12.5: Soil Parameters for Slope Stability Analysis – Area 3, DP3-S (Beach) 12-7
Table 12.6: Summary of Static Slope Stability Analysis (Heavy Haul Road Cut) 12-9
Table 12.7: Summary of Pseudo-Static Slope Stability Analysis (Heavy Haul Road Cut) 12-10
Table 12.8: Summary of Static Bluff Slope Stability Analysis 12-10
Table 12.9: Summary of Pseudo-Static Bluff Slope Stability Analysis 12-11
Table 12.10: Comparison of the Estimated Absolute Total Horizontal Surface Displacements for the OBE Level within the Tank Location 12-12
Table 12.11: Comparison of the Estimated Absolute Relative Horizontal Displacements at the Surface per Quartile for the OBE Level 12-12
Table 12.12: Comparison of the Estimated Absolute Total Horizontal Surface Displacements for the SSE Level within the Tank Location 12-12
Table 12.13: Comparison of the Estimated Absolute Relative Horizontal Displacements at the Surface per Quartile for the SSE Level 12-13

Table 12.14: Summary of Static Slope Stability Analysis (Outfall) 12-13
Table 12.15: Summary of Pseudo-Static Slope Stability Analysis (Outfall) 12-14
Table 13.1: Lateral Earth Pressure Parameters – Areas 1 to 6 13-1
Table 14.1: Dynamic Soil Properties – Non-Frost Susceptible Granular Structural Fill Soils 14-2
Table 14.2: Dynamic Soil Properties – Area 2 14-2
Table 14.3: Dynamic Soil Properties – Area 3 14-3
Table 14.4: Dynamic Soil Properties – Area 4 14-3
Table 14.5: Dynamic Soil Properties – Area 5 14-4
Table 15.1: Typical Pavement Sections 15-2
Table 16.1: Summary of Corrosion Potential Testing – 2014 G&G program 16-1
Table 16.2: Summary of Corrosion Potential Testing – 2015 G&G program 16-2
Table 16.3: Summary of Corrosion Potential Testing – 2016 G&G program 16-2
Table 16.4: Corrosion Potential based on Resistivity, pH and Chloride Content 16-3
Table 16.5: Degradation of Concrete using Sulfate Ion Concentration 16-4
Table 17.1: Selected Soil Design Parameters – Pile Drivability Analysis 17-6
Table 17.2: Summary of Pile Drivability Results – Impact Hammers 17-8
Table 17.3: Summary of Pile Drivability Results – Vibratory Hammer ICE 1412 with Penetration Depth of 33-ft 17-8

ILLUSTRATIONS

	Plate
Vicinity Map	1
Overall Site Plan	2
Structure Layout Plan	3a
Plan of Explorations	3b
Interpreted Topsoil and Soft Soil Thickness	4a
Interpreted Gravel Thickness and Depth to Near Surface Gravel	4b
Summary of Vertical Electrical Sounding (VES) Profiles	5
Hydrogeology Groundwater Interpretation	6a
Interpretation of Groundwater Depth (Water Bearing Unit 1)	6b
Site Topography Map	7a
Low-Lying Areas	7b
Geologic Cross Section of the Site Area	8a
Integrated Site Schematic Model	8b
Integrated Subsurface Profiles – Areas 1 thru 6	9 thru 14
IMASW, Downhole, & SPT-derived Shear Wave Velocity Profiles	15
Cut and Fill Areas	16
Heavy Haul Road Area – 1	17a
Storage Tank Area – 2	17b
Condensate and Admin Area – 3	17c
Process Train Area – 4	17d
Utility Area – 5	17e
Laydown Area – 6	17f
Fines Content of Proposed Cut Materials	18a
Gradation of Proposed Cut Materials	18b
Maximum and Minimum Densities – All Areas	19
Plan of Design Cross Section Lines	20
Selected Design Profiles – Areas 1 thru 8	21a thru 21n
Parameter Plots for Area 1 (HA1)	22a thru 22e
Parameter Plots for Area 1 (HA2)	23a thru 23e
Parameter Plots for Area 1 (HA3)	24a thru 24f
Parameter Plots for Area 2a (DP2A)	25a thru 25f

Parameter Plots for Area 2a (DP2A-S)	26a thru 26f
Parameter Plots for Area 2b (DP2B)	27a thru 27f
Parameter Plots for Area 2c (DP2C)	28a thru 28f
Parameter Plots for Area 3 (DP3)	29a thru 29f
Parameter Plots for Area 3 (DP3-S)	30a thru 30f
Parameter Plots for Area 4a (DP4A)	31a thru 31e
Parameter Plots for Area 4b (DP4B)	32a thru 32e
Parameter Plots for Area 5 (DP5)	33a thru 33e
Parameter Plots for Area 6 (DP6)	34a thru 34e
Parameter Plots for Area 8 (DP8A – Boring B-176)	35a thru 35e
Parameter Plots for Area 8 (DP8B – Boring B-183)	36a thru 36d

APPENDICES

	Plate
Appendix A – LNG Tank Shallow Foundation Recommendations	
Schematic View of LNG Tank Foundations	A-1
Tank Stability	A-2 thru A-9
Tank Settlement	A-10 thru A-15
Appendix B – Shallow Foundation Recommendations	
Shallow Foundation Settlements	B-1 thru B-18
Computation of Bearing Pressures	B-19
Shallow Foundation Insulation Recommendations	B-20 thru B-23
Appendix C – Deep Foundation Recommendations	
Deep Foundation Design Parameters	C-1 and C-9
Ultimate Axial Capacity	C-10 thru C-33
Axial Load vs Pile Top Settlement	C-34 and C-41
Lateral Capacity	C-42 thru C-63
Pile Cap Insulation Recommendations	C-64
Appendix D – Bluff Slope Stability Recommendations	
Overall Bluff Cross Sections	D-1
Overall Bluff Slope Angles	D-2a thru D-2f
Slope Configurations & Loading Conditions	D-3a thru D-3d
Area 1 - Static Slope Stability Results	D-4 thru D-13
Area 1 - Pseudo-Static Slope Stability Results	D-14 thru D-18
Area 2 - Static Slope Stability Results	D-19 thru D-20
Area 2 - Pseudo-Static Slope Stability Results	D-21 and D-22
Area 3 - Static Slope Stability Results	D-23 thru D-28
Area 3 - Pseudo-Static Slope Stability Results	D-29 thru D-31
Appendix E – Dynamic (FLAC 2D) Slope Stability Results	
(USAL-FG-GRZZZ-00-002015-018 Rev.0)	E-1 thru E-187
Appendix F – Summary of Downhole Test Results	
Location of Downhole Seismic Tests	F-1
Shear Wave Velocity Test Results	F-2a thru F-2c
Shear Wave Velocity vs Elevation – Areas 2, 3, 4, 5	F-3a thru F-3d
Appendix G – Pile Drivability Analysis Results	
GRLWEAP Program Outputs	G- 1 thru G-40

1.0 INTRODUCTION

1.1 Project Description

The Alaska Gasline Development Corporation, BP Alaska LNG LLC, ConocoPhillips Alaska LNG Company, and ExxonMobil Alaska LNG LLC (Applicants) plan to construct one integrated liquefied natural gas (LNG) Project (Project) with interdependent facilities for the purpose of liquefying supplies of natural gas from Alaska, in particular from the Point Thomson Unit (PTU) and Prudhoe Bay Unit (PBU) production fields on the Alaska North Slope (North Slope), for export in foreign commerce and opportunities for in-state deliveries of natural gas.

The Natural Gas Act (NGA), 15 U.S.C. § 717a(11) (2006), and Federal Energy Regulatory Commission (FERC) regulations, 18 C.F.R. § 153.2(d) (2014), define “LNG terminal” to include “all natural gas facilities located onshore or in State waters that are used to receive, unload, load, store, transport, gasify, liquefy, or process natural gas that is ... exported to a foreign country from the United States.” With respect to this Project, the “LNG Terminal” includes the following: a liquefaction facility (Liquefaction Facility) in Southcentral Alaska; an approximately 804-mile gas pipeline (Mainline); a gas treatment plant (GTP) on the North Slope; an approximately 62-mile gas transmission line connecting the GTP to the PTU gas production facility (PTU Gas Transmission Line or PTTL); and an approximately 1-mile gas transmission line connecting the GTP to the PBU gas production facility (PBU Gas Transmission Line or PBTL). All of these facilities are essential to export natural gas in foreign commerce.

The new Liquefaction Facility would be constructed on the eastern shore of Cook Inlet just south of the existing Agrium fertilizer plant on the Kenai Peninsula, approximately 3 miles southwest of Nikiski and 8.5 miles north of Kenai (Plate 1). The Liquefaction Facility would include the structures, equipment, underlying access rights, and all other associated systems for final processing and liquefaction of natural gas, as well as storage and loading of LNG, including terminal facilities and auxiliary marine vessels used to support Marine Terminal operations (excluding LNG carriers [LNGCs]). The Liquefaction Facility would include three liquefaction trains combining to process up to approximately 20 million metric tons per annum (MMTPA) of LNG. Two 240,000-cubic-meter tanks would be constructed to store the LNG. The Liquefaction Facility would be capable of accommodating two LNG carriers. The size of LNGCs that the Liquefaction Facility would accommodate would range between 125,000–216,000-cubic-meter vessels.

EMALL contracted Fugro to investigate the site conditions of the onshore LNG facilities, marine LNG Terminal, and marine pipeline corridors. Overview of overall project facilities described above are presented on Plate 2. Completed onshore explorations (2014 through 2016) and the proposed LNG Facilities plant layout are presented on Plate 3a and Plate 3b.

A brief description of the various onshore facilities is provided below and were also provided to us on drawings with document numbers USAL-CB-LDLAY-00-000003-000 (Rev.6) dated July 30, 2016, and USAL-CB-CDDET-00-000088-001 through 015 (Rev.2) dated May 13, 2016, by AKLNG:

- LNG Storage Tank Area: LNG produced from the liquefaction process will be stored on the plant site in specially designed insulated tanks. Two 240,000-cubic-meter tanks would be constructed to store the LNG. The storage system will include product pumps for ship loading and a boil off compressor to return the vaporized LNG back to the liquefaction section. Current plans are to construct the LNG storage tank area on the existing bluff on the southwest part of the site. Sizes and loading conditions for the storage tanks were provided by AKLNG. The applied pressure on bearing surface along the perimeter (thickened edges) will be about 5,760 psf. The applied pressure on bearing surface across the entire floor portion of the tank will be 4,070 during hydrotest and 2,690 psf during operations.

- LNG Train Area: Three LNG trains are currently planned to process up to approximately 20 million metric tons per annum (MMTPA) of LNG. The APCI (Air Products and Chemicals Inc.) process, also called Propane Pre-Cooled Mixed Refrigerant Process (PPMR) will be utilized. Ancillary equipment/structures will include compressors, propane chillers, seawater coolers, condensers, pipe racks, horizontal and vertical vessels, substations, pumps, and other miscellaneous equipment. Detailed equipment weights were not available at the time of this report, however typical loads for the proposed equipment/structures within the train areas are expected to vary between 600 psf to 3,000 psf.

- Flare Area: This area will include ground flare systems that will act as a vapor relief system and is the primary safety feature of the proposed LNG Plant. The flare system will collect and dispose of hydrocarbon-containing streams, which are typically released during start-up and shutdown, and if necessary, during emergency conditions. Locations and sizes for the various equipment/structures planned in this area were provided by AKLNG. All-inclusive weights were not available at the time of this report, but typical loads within this area may vary between 400 psf and 1,800 psf.

- Buildings & Miscellaneous Structures: The construction of the proposed LNG Plant will also include construction of site infrastructure such as administrative buildings, laboratory building, fuel and water storage tanks, and chemical storage facilities, and workshops. Typical loads for the proposed structures within the buildings area are expected to vary between 100 psf and 600 psf.

- Camp Area: A camp will be erected to accommodate the workforce at the site during construction. Access around the facility will require construction of typical engineered roads constructed in accordance with standard engineering specifications. The proposed construction will require the use of cranes, excavators, trucks and other heavy machinery on

site. These will require construction of heavy haul roads around the site. Preliminary locations of the proposed roads are also shown on drawings provided by AKLNG.

This report documents the onshore geotechnical engineering recommendations based on the data collected during our 2014, 2015 and 2016 G&G programs conducted at the Alaska LNG site (Site) near Nikiski, Alaska (see Plate 1). Recommendations presented in this report are applicable to the layout presented and the design information provided at the time of this report.

A list of the reports (including the superseded reports) that are generated by Fugro as part of 2014, 2015 and 2016 G&G programs are presented in Table 1.1.

1.2 Purposes and Scope

The purpose of this integrated report was to develop a multi-disciplinary review of the data collected during 2014, 2015 and 2016 G&G programs and to integrate the data to develop a subsurface site model, identify geohazards and their potential impact on site development, and to provide geotechnical engineering recommendations. We accomplished these purposes by:

- Reviewing publicly available literature as well as historical reports provided by AKLNG.
- Reviewing various site investigation activities from 2014, 2015, and 2016 G&G programs and the interpretive work conducted to integrate the various data sets.
- Reviewing frost depths at the proposed project site and evaluating the frost susceptibility of the encountered subsurface soils.
- Integrating the geotechnical, geophysical, and geological data to provide interpretations of the subsurface conditions in the project area.
- Providing descriptions of the stratigraphy with interpreted subsurface engineering properties.
- Providing site preparation recommendations, including recommendations for fill materials.
- Evaluating suitable shallow and deep foundations types.
- Providing recommendations for LNG tank foundations.
- Providing shallow foundation recommendations including foundation depth, allowable bearing capacity, resistance to uplift and lateral loads, and modulus of subgrade reaction for mat foundations.
- Providing settlement analyses for recommended shallow foundation types and tank foundations. Providing general discussion on settlement of a single isolated pile and on pile-group settlement.
- Providing axial and lateral pile capacities.

- Providing slope stability analyses of anticipated cuts and fills and roadway and dike embankments and performing detailed static and dynamic slope stability evaluations on critical bluff sections and heavy haul road sections.
- Providing dynamic soil properties for vibratory equipment foundation design based on in-situ testing.
- Providing typical roadway sections for both flexible and rigid pavements based on Alaska Department of Transportation and Public Facilities (DOT&PF) specifications.
- Providing soil and groundwater corrosion potential (pH, chloride, sulfate, and electrical resistivity) based on the tests conducted on samples collected from the site.
- Performing preliminary pile drivability analyses to provide general comments on pile drivability.

We were neither asked nor authorized to evaluate environmental conditions at the site. However, our scope of work included subsurface water quality data collection, chemical analysis, and reporting. Consultation regarding potential chemical impact, sources of chemical impact, assessment of potential contamination migration, compliance with state and federal regulatory requirements, and geo-chemical environmental risk analyses were beyond the scope of our services.

1.3 Report Format

In general, Fugro followed the detailed reporting schedule provided in the Project Execution Plan for 2016 Onshore and Marine G&G Program[4]. Initially, separate reports were generated for various field and desktop studies required as per the proposed scope of work. Various relevant studies included:

1. Marine Geophysics Reports
2. Marine Geotechnical Data Reports
3. Geologic Field Mapping Reports
4. Onshore Geophysical Reports
5. Onshore Geotechnical Data Reports
6. Geologic Hazards Reports
7. Hydrogeologic & Groundwater Well Installation Reports, and
8. PSHA/Site Response Reports/Liquefaction Hazard Assessment Reports

Data collected from the above-mentioned studies was interpreted and combined to develop the integrated site subsurface model. This integrated site characterization and engineering report briefly discusses the various studies completed for this project and presents the results of our geological/geotechnical interpretation and geophysical integration, and analyses for only the onshore LNG facilities of the Project. Please note that this report supersedes 2014 and 2015 integrated site

[4] Fugro Consultants, Inc. (Fugro), 2016, Project Execution Plan for 2016 Onshore and Marine G&G Program, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-1, AKLNG Document No. USAL-FG-GPZZZ-00-002016-001, Rev.0, dated October 13, 2016.

characterization and engineering reports prepared for the onshore LNG facilities, and this report should be used in conjunction with the results of the various studies conducted at this site by Fugro.

A list of the reports (including the superseded reports) that are generated by Fugro as part of the 2014, 2015 and 2016 G&G programs are presented in Table 1.1. A copy of Table 1.1 is also separately submitted to AKLNG under document number USAL-FG-BRCTL-00-000001-000[5].

Table 1.1: Summary of Fugro Reports Developed for 2014, 2015 and 2016 G&G Programs

G&G Program	Report Title (Superseded Reports in Gray)	AKLNG Document Number	Fugro Report Number
2014	Project Execution Plan for 2014 Onshore and Marine G&G	USAL-FG-GPZZZ-00-000001-000	04.10140094-1
	Geologic Mapping Report	USAL-FG-GRZZZ-00-000001-000	04.10140094-2
	Marine Survey Report Pipeline Corridor Route 1	USAP-FG-GRZZZ-10-000001-000	04.10140094-3
	Marine Survey Report Pipeline Corridor Route 2	USAP-FG-GRZZZ-10-000002-000	04.10140094-4
	Marine Survey Report Nearshore LNG Facilities and Approach Channel	USAL-FG-GRZZZ-90-000003-000	04.10140094-5
	Probabilistic Seismic Hazard Analysis Report(i)	USAL-FG-GRHAZ-00-000001-000	04.10140094-6
	Geophysical Survey Report	USAL-FG-GRZZZ-00-000002-000	04.10140094-7
	Geotechnical Data Report Onshore LNG Facilities	USAL-FG-GRZZZ-00-000003-000	04.10140094-8
	Geologic Hazard Report(ii)	USAL-FG-GRHAZ-00-000002-000	04.10140094-9
	Hydrogeologic Report(iii)	USAL-FG-GRZZZ-00-000004-000	04.10140094-10
	Groundwater Monitoring Well Installation Report	USAL-FG-GRZZZ-00-000007-000	04.10140094-10A
	Liquefaction Potential Evaluation Report(iv)	USAL-FG-GRZZZ-00-000005-000	04.10140094-11
	Integrated Site Characterization and Engineering Report(v)	USAL-FG-GRZZZ-00-000006-000	04.10140094-12

[5] Fugro Consultants, Inc. (Fugro), 2016, A Roadmap to Fugro G&G Reports Covering Site Investigation Campaigns in 2014, 2015 & 2016, Alaska LNG Project, Nikiski, Alaska, AKLNG Document No. USAL-FG-BRCTL-00-000001-000, Rev.0, dated December 22, 2016.

G&G Program	Report Title (Superseded Reports in Gray)	AKLNG Document Number	Fugro Report Number
2015	Project Execution Plan for 2015 Onshore and Marine G&G Program	USAL-FG-GPZZZ-00-000002-000	04.10140334-1
	LNG Facilities Onshore Geologic Field Mapping Report	USAL-FG-GRZZZ-00-002015-004	04.10140334-2
	Pipeline Marine Geophysical Survey Report - Route 1	USAP-FG-GRZZZ-10-002015-013	04.10140334-3
	Pipeline Marine Geophysical Survey Report - Route 2	USAP-FG-GRZZZ-10-002015-014	04.10140334-4
	LNG Facilities Marine Geophysical Survey Report	USAL-FG-GRZZZ-90-002015-010	04.10140334-5
	LNG Facilities Probabilistic Seismic Hazard Analysis (PSHA) Report[(i)]	USAL-FG-GRHAZ-00-002015-001	04.10140334-6
	LNG Facilities Onshore Geophysical Survey Report	USAL-FG-GRZZZ-00-002015-005	04.10140334-7
	LNG Facilities Onshore Geotechnical Data Report	USAL-FG-GRZZZ-00-002015-006	04.10140334-8
	LNG Facilities Marine Geotechnical Data Report	USAL-FG-GRZZZ-90-002015-011	04.10140334-9
	LNG Facilities Geologic Hazard Report[(ii)]	USAL-FG-GRHAZ-00-002015-002	04.10140334-10
	LNG Facilities Onshore Groundwater Monitoring Well Installation Report	USAL-FG-GRZZZ-00-002015-007	04.10140334-11
	LNG Facilities Onshore Hydrogeologic Report[(iii)]	USAL-FG-GRZZZ-00-002015-008	04.10140334-12
	LNG Facilities Seismic Engineering Report[(iv)]	USAL-FG-GRZZZ-00-002015-003	04.10140334-13
	LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report[(v)]	USAL-FG-GRZZZ-00-002015-009	04.10140334-14
	LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report	USAL-FG-GRZZZ-90-002015-012	04.10140334-15
2016	Project Execution Plan for 2016 Onshore and Marine G&G Program	USAL-FG-GPZZZ-00-002016-001	04.10160001-1
	LNG Facilities Groundwater Quality Sampling and Testing Report – Event 1	USAL-FG-GRZZZ-00-002016-003	04.10160001-2

G&G Program	Report Title (Superseded Reports in Gray)	AKLNG Document Number	Fugro Report Number
2016	LNG Facilities Groundwater Quality Sampling and Testing Report – Event 2	USAL-FG-GRZZZ-00-002016-004	04.10160001-3
	LNG Facilities Aquifer Pump Test Well and Groundwater Observation Well Installation Report	USAL-FG-GRZZZ-00-002016-002	04.10160001-4
	LNG Facilities Onshore Geotechnical Data Report	USAL-FG-GRZZZ-00-002016-001	04.10160001-5
	LNG Facilities Onshore Hydrogeologic Report(iii)	USAL-FG-GRZZZ-00-002016-007	04.10160001-8
	LNG Facilities Seismic Engineering Report(iv)	USAL-FG-GRZZZ-00-002016-008	04.10160001-9
	Pipeline Marine Shallow Geotechnical Report	USAP-FG-GRZZZ-10-002016-011	04.10160001-10
	LNG Facilities Marine Survey Report	USAL-FG-GRZZZ-90-002016-010	04.10160001-11
	LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report(v)	USAL-FG-GRZZZ-00-002016-009	04.10160001-12
	LNG Facilities Rigs Tenders Wharf Siltation Survey Report	USAL-FG-CRZZZ-90-002016-001	04.10160001-13

Notes: (i) Fugro Report No. 04.10140334-6 supersedes Fugro Report No. 04.10140094-6.
(ii) Fugro Report No. 04.10140334-10 supersedes Fugro Report No. 04.10140094-9.
(iii) Fugro Report No. 04.10160001-8 supersedes Fugro Report Nos. 04.10140094-10 and 04.10140334-12.
(iv) Fugro Report No. 04.10160001-9 supersedes Fugro Report Nos. 04.10140094-11 and 04.10140334-13.
(v) Fugro Report No. 04.10160001-12 supersedes Fugro Report Nos. 04.10140094-12 and 04.10140334-14.

1.4 Applicability of Report

We have prepared this report for AKLNG for use in planning and design of the proposed Alaska LNG Project. The scope of the explorations and tests performed in this report are for the Alaska LNG Project only. The field programs and tests were selected or developed based on our understanding of the project as described in this report and our discussions with AKLNG representatives. If pertinent details of the project change or otherwise differ from discussion and information presented in this report, we request that we be notified and authorized to review and modify recommendations presented herein. The information presented in this report shall be used only in the areas explored during this study.

1.5 Limitations

Fugro makes no claim or representation concerning any activity or condition falling outside the specified purposes to which this report is directed. We have conducted our work using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. We intend for this report, including all illustrations, to be used in its entirety. The information presented in this report may not apply to locations not explored by borings or areas outside the project boundaries. Undisclosed variations in soil conditions may be observed in low-lying areas and outside the boring locations. The information presented in this report may not apply to locations not explored. This information should be made available to prospective users for information only, and not as a warranty of subsurface conditions.

1.6 Unit Conversions and Datums

The recommendations presented herein are based on the Imperial Unit System. Table 1.2 provides a quick reference for conversion from Imperial Units to SI.

Table 1.2: Conversion Units

From SI System	To Imperial System	Multiply by
Kilo Newtons – kN	Kips – k	0.224809
Mega Newtons – MN	Kips – k	224.809
Kilo Newtons/meter^2 – kN/m^2 (kPa)	Pounds/feet^2 - psf	20.885
Kilo Newtons/meter^3 – kN/m^3	Pounds/feet^3 - pcf	6.3659
Meters – m	Feet – ft	3.2808
Millimeters – mm	Inches – in.	0.03937

Geodetic data references used in this report are specified in the document USAKE-EX-GSZZZ-00-0001 Geodetic Parameters, dated May, 2015. All coordinates are reported in Zones AK3 AK4 AK5 North, NAD83 (NSRS 2007), and are in feet. Topographic elevations for onshore areas are referenced to NAVD88. It should be noted that the marine geophysical survey is referenced to Mean Lower Low Water (MLLW). The following conversion formula is used to convert the elevations from NAVD88 to MLLW:

Elevation, in ft (MLLW) = Elevation, in ft (NAVD88) + 7.32 ft

Please note that this conversion formula is only applicable at Nikiski Area. Elevations presented in this report, and the corresponding illustrations and engineering plates are all referenced to the NAVD88 datum, unless noted otherwise.

2.0 RELEVANT STUDIES

2.1 General

Our scope of work included performing field and desktop studies to support the proposed development and creating an integrated site characterization model. Previous studies that were performed at or near the proposed project site were also considered as a part of the current scope of work. Data available from various studies conducted as a part of the current scope of work was combined and integrated with the previous studies (including some performed by others), where possible to better understand the geological and hydrogeological conditions at the site. Data collected and interpretations associated with various studies are briefly discussed in the following subsections.

2.2 Previous Reports

AKLNG provided various studies performed by others, some of which were relevant to the proposed project site and were considered while developing our recommendations. Additionally, Fugro has conducted several other studies, including nearshore and onshore investigations for ConocoPhillips and site screening studies and geohazard assessments of several potential sites in Alaska. Following is the list of documents that were reviewed while developing recommendations for the current study:

- Fugro-McClelland Marine Geosciences, Inc. (FMMG), 2012a, Geologic Study Potential LNG Plant Sites, South Central Alaska, Report No. 0201-7173-2, prepared for ConocoPhillips, Inc., AKLNG Document No. USAKL-FG-GRHAZ-40-0001, Rev.1, dated December 21, 2012.
- Fugro-McClelland Marine Geosciences, Inc. (FMMG), 2012b, Geologic Hazard Study, Tsunami and Surface Deformation Assessment of Potential LNG Plant Sites, South Central Alaska, Phase 1B desktop study – Addendum to report No. 0201-7173-2, prepared for ConocoPhillips, Inc., AKLNG Document No. USAKL-FG-GRHAZ-40-0002, Rev.0, dated December 21, 2012.
- Fugro Consultants, Inc., 2014a, Investigation planning report for Pre-FEED Onshore and Nearshore Geotechnical, Geophysical Survey, and Analysis Services, Alaska LNG Project, Nikiski, Alaska USA, prepared for ExxonMobil Development Company, Houston, TX Fugro project #04.79227300, AKLNG Document No. USAL-FG-GYZZZ-00-000001-000, Rev.1, dated February, 21, 2014.
- Fugro Consultants, Inc., 2014b, Desktop Review and Compilation of Existing Investigations, Nikiski area, Alaska, version 1.1, prepared for ExxonMobil Development Company, Houston, TX, Fugro project #04.79227300, AKLNG Document No. USAL-PL-GRZZZ-00-000001-000, Rev.1.1, dated February 26, 2014.

- Fugro Gulf, Inc., Consulting Engineers and Geologists, 1976, Soils and Foundation Investigation Construction Dock – Pacific Alaska LNG Project, Cook Inlet, Alaska, Report No. 75-093, prepared for Fluor Ocean Services Inc., submitted June 24, 1976.
- Fugro Gulf, Inc., Consulting Engineers and Geologists 1975, Soils and Foundation Investigation Pacific Alaska LNG Terminal, Cook Inlet, Alaska, Report No. 75-036-1, prepared for Fluor Ocean Services Inc., AKLNG Document No. 75-036-1, Rev.0, dated September 4, 1975.
- McClelland Engineers, Inc., Soil & Foundation Consultants, 1967, Soil and Foundation Investigation Proposed L.N.G. Dock Facility, Cook Inlet, Alaska, Report No. 6796-1 dated May 31, 1967 prepared for Phillips Petroleum Company
- McClelland Engineers, Inc., Soil & Foundation Consultants, 1967, Soil and Foundation Investigation Proposed L.N.G. Dock Facility, Cook Inlet, Alaska, Report No. 6796-2, prepared for Phillips Petroleum Company, AKLNG Document No. USAL-NA-GRSIT-90-000001-000, Rev.0, dated June 23, 1967.
- Worley Analytical Services, 2006, Site Investigation Report, Agrium Kenai Nitrogen Operations Plant, prepared for Agrium Us., Inc., dated January 3, 2006.
- URS Corporation, Inc., 2001, Completion Report and Baseline Groundwater Results, BP Gas to Liquids Test Facility Mile 20.5, North Kenai Road, Nikiski, Alaska, ADEC Wastewater Permit 0023-DB066, prepared for British Petroleum (BP) Exploration (Alaska) and Kvaerner Process Ex Alaska, Project No. 53-17298029.00, dated November 20, 2001.
- URS Corporation, Inc., Dames & Moore, 2000, Final Report of Geotechnical Investigation BP GTL-TF Project prepared for British Petroleum (BP) Alaska, D&M Job No. 17298-029-189, dated September 6, 2000.
- Kent and Sullivan Environmental Consultants Inc., 1997, Characterization Report, Pirm Extension Area Tesoro Alaska Refinery Project, prepared for Tesoro Alaska Petroleum Corporation, Project No. 01-19, dated November 20, 1997.
- Kent and Sullivan Environmental Consultants Inc., 1996, 1995/96 Winter Investigation Report, Kenai Pipeline Terminal, Nikiski, Alaska, Volume I of II, prepared for Tesoro Alaska Petroleum Corporation, Project No. 01-02.2, dated July 24, 1996.
- Dalton, Olmsted & Fugand Sullivan Environmental Consultants Inc., 1996, Site Characterization Analysis, Unocal Chemicals Plant Kenai, Alaska, prepared for Union Oil Company of California Unocal Agricultural Products, dated June, 1996.
- ENSR Consulting and Engineering, 1993, Site Characterization Report Volume I of II, Kenai Pipe Line Company Terminal Nikiski, Alaska, prepared for Kenai Pipe Line Company, Document No. 1420-072-0625, dated June 1993.

- Geomega Inc., 2006, Chevron Kenai Former Refinery 2006 Site Characterization Report, prepared for Chevron Environmental Management Company, Document No. C0341A, dated July 29, 1993.

2.3 Land Survey

The land surveying services were provided by qualified and licensed Alaska surveyors JOA Surveying Services, Inc. (JOA) (2014 G&G program) and McLane Consulting, Inc. (MCI) (2015 and 2016 G&G programs).

JOA's services consisted of performing as-built surveys at each completed onshore boring, monitoring well, and permanent survey monument at the end of the 2014 field program. MCI survey services included accuracy analysis of available LIDAR based topographic data sets, collection of conventional topographic data, collection of as-built data for existing structures and utilities, and survey support for 2015 G&G program. The primary data collection method utilized was Global Positioning System (GPS) Real Time Kinematics (RTK) with supplemental data via total station systems. Details of the completed survey by MCI are provided in AKLNG Document No. USAL-FG-GRZZZ-00-002015-006[6]. Additionally, MCI performed as-built surveys at each completed onshore exploration location at the end of both 2015 and 2016 field programs.

LiDAR data collected by airborne surveys extended the area of high resolution topographic data to the 5-mile site radius for use in geomorphic and geologic analysis. LiDAR data were collected from May to September 2008[7] by the United States Geological Survey (USGS) for Kenai Peninsula Borough (KPB) and in 2012[8] & 2014[9] by Fugro. The LiDAR data were integrated with the MCI topographic survey of the LNG facilities study area in ArcGIS to create a comprehensive topographic model.

As part of the 2016 G&G program, MCI also performed a siltation survey at the ASRC (Arctic Slope Regional Corporation) Rig Tenders marine terminal dock located north of the LNG facilities study area (Plate 3b). The vicinity of the ASRC dock receives deposition of significant quantities of sand and gravel along the south and southwest seawalls due to tidal activities. The owner of the ASRC dock, ASRC Energy Services Inc., has undertaken dredging activities several times a year during

[6] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[7] U.S. Geological Survey (USGS), 2008, National Elevation Dataset, 1/9 arc second (2.5 meters), LiDAR derived digital elevation model, EROS Data Center, https://lta.cr.usgs.gov/LIDAR_digitalelevation, Data obtained Sept 2014 from USGS.
[8] Fugro-McClelland Marine Geosciences, Inc. (FMMG), 2012, Geologic Study Potential LNG Plant Sites, South Central Alaska, Report No. 0201-7173-2, prepared for ConocoPhillips, Inc., AKLNG Document No. USAKL-FG-GRHAZ-40-0001, Rev.1, dated December 21, 2012
[9] Fugro Consultants, Inc. (Fugro), 2015, Marine Survey Report Nearshore LNG Facilities and Approach Channel, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140094-5, AKLNG Document No. USAL-FG-GRZZZ-90-000003-000, Rev.0, dated April 29, 2015.

low tide cycles to remove the excess material deposition. The purpose of this survey was to evaluate the sediment accumulation/removal and rate of siltation/erosion in the tidal areas around ASRC dock. After conducting the siltation survey, MCI concluded that beach areas north of the wharf experienced overall erosion while the areas south of the wharf experienced overall deposition. Detailed information about the siltation survey are presented in AKLNG Document No. USAL-FG-CRZZZ-90-002016-001[10].

2.4 Marine Geophysical Survey

Marine geophysical survey was performed in 2014 and 2015 along two proposed pipeline corridors (Route 1 and Route 2) crossing Cook Inlet and at the marine terminal study area (Plate 2). The principal objective of the marine survey in 2014 and 2015 was to identify and characterize the seafloor features and hydrographic conditions at these locations. The results of the surveys conducted in 2014 are presented in AKLNG Document Nos. USAP-FG-GRZZZ-10-000001-000[11], USAP-FG-GRZZZ-10-000002-000[12] and USAL-FG-GRZZZ-90-000003-000[13]. The results of the surveys conducted in 2015 are presented in AKLNG Document Nos. USAP-FG-GRZZZ-10-002015-013[14], USAP-FG-GRZZZ-10-002015-014[15] and USAL-FG-GRZZZ-90-002015-010[16].

A separate marine integrated site characterization report for the proposed nearshore facilities was developed and the data available from these studies was integrated into the overall GIS database and our recommendations are provided in AKLNG Document No. USAL-FG-GRZZZ-90-002015-

[10] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Rigs Tenders Wharf Siltation Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-13, AKLNG Document No. USAL-FG-CRZZZ-90-002016-001, Rev.0, dated October 10, 2016.
[11] Fugro Consultants, Inc. (Fugro), 2015, Marine Survey Report Pipeline Corridor Route 1, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140094-3, AKLNG Document No. USAP-FG-GRZZZ-10-000001-000, Rev.0, dated April 29, 2015.
[12] Fugro Consultants, Inc. (Fugro), 2015, Marine Survey Report Pipeline Corridor Route 2, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140094-4, AKLNG Document No. USAP-FG-GRZZZ-10-000002-000, Rev.0, dated April 25, 2015.
[13] Fugro Consultants, Inc. (Fugro), 2015, Marine Survey Report Nearshore LNG Facilities And Approach Channel, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140094-5, AKLNG Document No. USAL-FG-GRZZZ-90-000003-000, Rev.0, dated April 29, 2015.
[14] Fugro Consultants, Inc. (Fugro), 2016, Pipeline Marine Geophysical Survey Report - Route 1, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140334-3, AKLNG Document No. USAP-FG-GRZZZ-10-002015-013, Rev.1, dated June 08, 2016.
[15] Fugro Consultants, Inc. (Fugro), 2016, Pipeline Marine Geophysical Survey Report - Route 2, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140334-4, AKLNG Document No. USAP-FG-GRZZZ-10-002015-014, Rev.0, dated June 08, 2016.
[16] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Geophysical Survey Report, Alaska LNG Project, Cook Inlet, Alaska, Report No. 04.10140334-5, AKLNG Document No. USAL-FG-GRZZZ-90-002015-010, Rev.0, dated June 15, 2016.

012[17]. Nearshore LIDAR data collected during this survey was utilized to evaluate the existing bluff geomorphology and stability and will be discussed in subsequent sections of this report.

The 2016 G&G program included additional geophysical survey at the marine terminal study area to assess the bathymetry and identify the seafloor features at the two proposed dredge disposal areas and at the ASRC dock area. Results of these surveys are presented in AKLNG Document No. USAL-FG-GRZZZ-90-002016-010[18].

2.5 Marine Geotechnical Reports

Various marine geotechnical investigations were conducted during 2015 and 2016 G&G programs. The 2015 Marine G&G program consisted of drilling 25 marine soil borings and acquiring PS Suspension logging data within 9 selected borings. The 2016 G&G program consisted of performing vibracore and grab sampling activities along the two proposed pipeline corridors, the marine terminal study area (including the existing ASRC dock area) and the dredge disposal area (see Plate 2) in order to explore and evaluate the nearsurface soil conditions. The investigation at the marine terminal study area and dredge disposal area also included collecting samples for biological (benthic) testing and water quality measurements.

Laboratory tests were performed in the field and the laboratory to evaluate pertinent index and geotechnical engineering properties of nearsurface soils. The results of the field and the laboratory tests conducted in 2015 are presented in AKLNG Document No. USAL-FG-GRZZZ-90-002015-011[19].

Marine geotechnical engineering recommendations based on the data collected in 2015 are presented in AKLNG Document No. USAL-FG-GRZZZ-90-002015-012[20]. The results of the field and the laboratory tests conducted in 2016 are presented in AKLNG Document Nos. USAP-FG-GRZZZ-10-002016-011[21] and USAL-FG-GRZZZ-90-002016-010[22] for the pipeline area and the marine terminal study area, respectively.

[17] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-15, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012, Rev.0, dated September 06, 2016.
[18] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-11, AKLNG Document No. USAL-FG-GRZZZ-90-002016-010, Rev.0, dated November 09, 2016.
[19] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-9, AKLNG Document No. USAL-FG-GRZZZ-90-002015-011, Rev.0, dated June 20, 2016.
[20] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-15, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012, Rev.0, dated September 06, 2016.
[21] Fugro Consultants, Inc. (Fugro), 2016, Pipeline Marine Shallow Geotechnical Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-10, AKLNG Document No. USAP-FG-GRZZZ-10-002016-011, Rev.0, dated September 08, 2016.
[22] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-11, AKLNG Document No. USAL-FG-GRZZZ-90-002016-010, Rev.0, dated November 09, 2016.

In the marine terminal study area, erosion by waves and currents has removed the Killey deposits, resulting in a sea floor underlain primarily by Moosehorn and older Pleistocene deposits. These consist of fine-grained glacially-derived subestuarine sediments, sandy and gravelly glacial outwash deposits, and poorly-sorted bouldery to gravelly glacial till deposits.

The Quaternary deposits overlie an erosional surface cut on gently folded Tertiary marine sediments, a surface referred to as the Quaternary/Tertiary unconformity. Immediately below the unconformity lies the Pliocene Sterling Formation, the youngest in the sequence of Tertiary formations filling the Cook Inlet basin. The Sterling Formation sediments are described as well-sorted, fine to coarse-grained sands that are rich in volcanic lithic fragments, quartz, and feldspars (McElmoyl, 2013). They may include conglomerate beds, thin seams of coal, and clays. Borings drilled in the marine terminal area penetrated the Moosehorn and older Pleistocene deposits and Sterling formation deposits. Locally, currents have deposited waves and ridges of sand and silt, forming a mantle of very young sediment over the older deposits. Please see *Section 3.3 Subsurface Model* of this report for details about the geologic setting of the site.

2.6 Field Geologic Mapping

Geologic field reconnaissance and mapping was performed September 8 to 12, 2014 and from May 27 to June 2, 2015 to observe and document geologic and geomorphic features to assess potential geologic hazards within a 5-mile radius of the project site (AKLNG Document Nos. USAL-FG-GRZZZ-00-000001-000[23] and USAL-FG-GRZZZ-00-002015-004[24]). Due to limited site access, most field mapping was performed in public access areas. The major activities and findings of the geologic field mapping are as follows:

<u>Surface Faulting</u>

- A detailed review of LIDAR and RTK topographic survey data was conducted to look for indications of faults and lineaments, and to identify targets for field inspection.
- A field survey was conducted of the prominent contact between sandy outwash deposits (Killey) and underlying fine-grained deposits (Moosehorn) exposed in the coastal bluff to evaluate evidence of surface faulting. Approximately 3,000 feet of the 9,300 feet of coastal bluff exposes this contact; the remainder is covered with colluvium and vegetation.
- The exposed contact exhibits two types of geometry: (1) planar and horizontal at elevations of 45 to 60 ft, and (2) gently downwarped to elevations as low as 35 feet.

[23] Fugro Consultants, Inc. (Fugro), 2014, Geologic Mapping Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-2, AKLNG Document No. USAL-FG-GRZZZ-00-000001-000, Rev.0, dated November 18, 2014.
[24] Fugro Consultants, Inc. (Fugro), 2015, LNG Facilities Onshore Geologic Field Mapping Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-2, AKLNG Document No. USAL-FG-GRZZZ-00-002015-004, Rev.0, dated August 21, 2015.

- No positive evidence for faulting of this contact was observed. The contact, where exposed, is laterally continuous, precluding the presence of faulting in these reaches of the bluff. The presence or absence of surface faulting could not be evaluated where the contact was covered.

Coastal Erosion

- A traverse of the base of the coastal bluffs indicated that multiple erosion processes are responsible for continued bluff retreat, including: wave erosion, gullying, raveling, sloughing of vegetation, debris flows, and slumping. Coastal erosion begins with wave erosion at the toe of the bluff, followed by displacement of material from the over steepened slope. No evidence of large deep-seated, rotational slumps was observed during the field activities. All slides, including slumps, are relatively shallow and involve materials from the bluff face above sea level.
- The surveyed stratigraphic contact exposed in the coastal bluff is gently downwarped in the vicinity of Lovers Loop Road. At the center of the downwarp, springs emanate from the contact and a large recent debris flow scar extends to the bluff crest and is the source of an actively building debris fan on the beach. Concentration of groundwater flow at the center of the downwarp is likely a major factor in the occurrence of this debris flow. An adjacent homeowner noted the debris flow occurred after an earthquake.
- Accessible segments (approximately 800 feet) of the crest of the bluff were inspected for evidence of tension cracks that may indicate incipient deep sliding. No evidence for tension cracks was observed. The bluff crest appears to fail by shallow landsliding and raveling. The geomorphology of the bluff crest is fairly linear and generally is not characterized by large arcuate scarps. An exception is the scarp of the large debris flow described above. It is judged unlikely that large slump failures occur that extend to the bluff crest. The evidence for large slumps is further evaluated in this report using the collected LiDAR data in *Section 12.0 Bluff Slope Stability*.

2.7 Shallow Onshore Geophysical Surveys

Shallow onshore geophysical survey (Plate 3b) consisting of seismic refraction along 8 lines was conducted as a part of the 2014 G&G program (AKLNG Document No. USAL-FG-GRZZZ-00-000002-000[25]). The main objective of the refraction survey was to aid in the geologic hazard evaluation and evaluate any major anomalies that may exist between the borings. In 2015, primary objectives of the shallow geophysical survey were to attempt to verify the shallow stratigraphy (in

[25] Fugro Consultants, Inc. (Fugro), 2015, Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-7, AKLNG Document No. USAL-FG-GRZZZ-00-000002-000, Rev.0, dated April 20, 2015.

particular, thickness of organic topsoil), map any gravel deposits that could potentially be used for construction, and to image any significant geotechnical and geologic variations or structures.

Based on the information available from the 2014 G&G program, the Electrical Resistivity Tomography (ERT) and shallow seismic methods (Seismic Refraction Tomography, SRT, and Interferometric Multi-channel Analysis of Surface Waves, IMASW) were selected for a trial survey (Plate 3b). Based on data acquired during the trial geophysical survey, it was determined that the Electrical Resistivity Tomography method displayed the best sensitivity to changes in the shallow stratigraphy (in the upper 20 to 40 feet of the subsurface) across the site and yielded encouraging potential for the mapping of organic topsoil thickness and potential gravel deposits.

The 2015 shallow onshore geophysical survey (Plate 3b) results are documented in AKLNG Document No. USAL-FG-GRZZZ-00-002015-005[26], which incorporates the data obtained during 2014 G&G program. Following is the brief summary of the results:

2.7.1 Electrical Resistivity Tomography Survey – Results and Conclusions

Through comparison with the geotechnical borings, test pits and hand augers, a series of correlations between the modeled resistivity values and the shallow stratigraphy have been developed. Generally, across the majority of the site, the modeled Electrical Resistivity Tomography data displays a surface layer with variable thickness, with resistivity values in the 500 to 5,000 Ω-m range. This layer correlates well with topsoil (organic, silty, clayey sandy soils), with notable increases in its thickness being observed where appreciable silty strata were found (such as on profile EW-3 in the vicinity of boring B-92). It has been noted that in most instances, the resistivity values within this layer display lateral variability across relatively short distances, which is interpreted to result from both changes in the composition of the surficial soils and changes in the surface conditions on site (such as presence or absence of vegetation and local topographic variations, which potentially result in variations in soil moisture). There are also occurrences where this surface layer is not apparent within the modeled data. The integrated evaluation of borings along with modeled resistivity values was used to develop the topsoil thickness over the entire project area as shown on Plate 4a.

At depth, the Electrical Resistivity Tomography data displays values in excess of 20,000 Ω-m across significant portions of the site. Such high values can be associated with air-filled void-spaces and hence relatively dry and granular materials and have therefore been associated with regions displaying relatively more appreciable concentrations of gravels. A preliminary integration of available borings and modeled resistivity values was used to develop a map where appreciable concentrations of gravel may be encountered as shown on Plate 4b. Depth to appreciable concentrations of gravel is also shown on Plate 4b.

[26] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-7, AKLNG Document No. USAL-FG-GRZZZ-00-002015-005, Rev.0, dated June 10, 2016.

In some localized areas, anomalously low resistivity values have been identified below 200 Ω-m, which can be correlated with clayey soils. These were sometimes verified with available borings and can sometimes be correlated to low-lying areas of the site and are discussed in further sections of the report.

2.7.2 Shallow Seismic Survey – Results and Conclusions

In general, the IMASW and seismic reflection methods produced very useful results, and accomplished the goals of the shallow seismic surveys. The IMASW profiles provide good imaging of sedimentary layering in the Killey and Moosehorn deposits within the 30 to 80 feet depth range. Planar impedance contrasts match well with the positions of fine-grained beds logged in boreholes, indicating that these beds have relatively high shear-wave velocities. Tracing planar impedance contrasts laterally may delineate the tops of these fine-grained beds. This information was useful for correlations of bedding between boreholes and ultimately for the understanding of geologic structure.

Shallow seismic reflection in 2015 was used solely on the beach survey lines where the lack of soft topsoil allowed greater penetration of seismic waves from the accelerated weight drop (AWD) source. We were successful in imaging deep geologic structure to depths of 1,400 feet. The resulting data matched well with the nearby marine seismic reflection data, and confirmed the presence of dipping Tertiary marine beds unconformably overlain by Quaternary glacial and glacially-derived sediments. Additionally, geophysical properties useful for engineering analysis were also obtained from shallow seismic data. Therefore, 10 profiles of Vs and Vp were generated within the LNG facilities area. These, along with downhole measurements in boreholes, provide a robust dataset for geotechnical calculations and are discussed in further sections of the report.

2.8 Deep Seismic Surveys

The purpose of the 2015 deep onshore geophysical survey was to provide deep subsurface geologic data, including the general subsurface characteristics of the project area, and to identify faults, folds and other geologic structures for geohazard evaluation. The deep seismic reflection surveys were successful in their goal of imaging the deep geologic structure beneath the site. Bedding planes in the Tertiary sediments are clearly resolved to depths of more than 24,000 feet, the approximate top of the Mesozoic basement. The unconformity at the top of the Tertiary sediments and base of the Quaternary is well imaged. These data fill in the gap between the industry and 2015 marine seismic reflection data to the west side of the site and industry seismic reflection data to the east of the site, and enabled the construction of a robust structural geologic model of the site area.

The deep onshore geophysical survey data show planar continuous reflectors throughout the Tertiary sedimentary strata, demonstrating absence of new or previously unknown faults beneath the site and within the five-mile site radius.

2.9 Onshore Geotechnical Exploration

Onshore geotechnical study for the 2014, 2015 and 2016 G&G programs consisted of drilling a total of 130 onshore soil borings (including beach borings), performing 35 downhole compression and shear wave velocity tests, 14 test pits, 9 vertical electrical soundings (VES), 31 hand augers, and installing 20 monitoring wells as shown on Plate 3b. Based on the information obtained from the above listed explorations, field geologic mapping, geophysical surveys, and desktop studies (by Fugro and others) we have identified three geologic units that underlie the project study area within the upper 300 feet. The characteristics of each geologic unit are summarized in Table 3.1.

- The topsoil is composed of an organic mat, or web of plant roots and humus, just beneath the ground surface and is underlain by loose silt containing volcanic ash. The topsoil ranges in thickness from 0 to 6.5 ft, forming a continuous mantle over the Killey deposits. The topsoil thickens in swales and depressions, thins across the outwash plain, and may be absent in areas of prior grading.
- The Killey glacial outwash deposits (17,500 to 18,500 years before present) consist of sands, laminated sands and silts, coarse sands, and subrounded to rounded gravels, gravelly sands and sandy gravels, extending from the base of the topsoil to depths of about 40 to 80 feet.
- The Moosehorn deposits underlie the Killey deposits and consist of glacially-derived subestuarine deposits of Moosehorn age (18,500 to 32,000 years before present). The Moosehorn deposits contain clayey sands with gravel, silty sands, sandy silts, silts, lean clays and silty clays. The Moosehorn deposits, together with the underlying older Pleistocene deposits, extend to a maximum explored depth of about 300 ft.

Detailed field and laboratory test results completed during the 2014, 2015 and 2016 G&G programs are presented in Fugro reports AKLNG Document Nos. USAL-FG-GRZZZ-00-000003-000[27], USAL-FG-GRZZZ-00-002015-006[28] and USAL-FG-GRZZZ-00-002016-001[29], respectively.

Hand auger explorations were performed for ground truthing and correlation with electrical resistivity tomography (ERT) and seismic surveys. Preliminary ERT data showed anomalous areas that required further verification. Thus, hand auger explorations were performed with target depths of 5 to 6 feet or refusal in gravel / cobbles, whichever occurred first. The hand auger used was 8-feet long rod with a 12-inch long auger and 3-inch inner diameter. Samples were logged by a field geologist, and collected for potential laboratory testing. The hand auger holes were backfilled with cuttings and natural material upon completion of the explorations.

[27] Fugro Consultants, Inc. (Fugro), 2015, Geotechnical Data Report Onshore LNG Facilities, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-8, AKLNG Document No. USAL-FG-GRZZZ-00-000003-000, Rev.0, dated May 08, 2015.
[28] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[29] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.

The downhole compression and shear wave velocity testing was conducted in selected boring locations to estimate the velocity at which both compression and shear waves propagate through the subsurface. Through correlation with geotechnical data, these velocity profiles were used to infer the (low strain) engineering properties (Young's modulus, shear and bulk modulus values) of the subsurface materials (assuming isotropic conditions). The calculated compression wave velocities tend to be between 1,000 and 3,500 ft/s at depths shallower than the observed groundwater surface and are likely to be associated with the comparatively less dense near surface soils. Below the observed groundwater surface, the compression wave velocities are notably higher, in the range of 4,000 to 8,000 ft/s. Such elevated velocities are commonly observed in saturated soils as the compression wave tends to predominantly travel within the pore fluid, rather than through the soil matrix. It should be noted that only shear wave velocities were used during selection of parameters for our engineering analyses. The test results are discussed in *Section 14.0 Dynamic Soil Properties* of this report.

VES tests (E-1 through E-9) involve determining a vertical profile of the apparent resistivity of the subsurface, which is commonly used to assist with the design of cathodic protection and electrical grounding systems.

Data acquisition logs from the Vertical Electrical Soundings, details about the equipment, data acquisition parameters and test locations, along with tabulated and graphical plots of the recorded data from the tests are presented in the AKLNG Document No. USAL-FG-GRZZZ-00-002016-001[30].

For the majority of the tests, the data is considered to be of good quality as the measurements recorded with the array in two perpendicular orientations display a close correlation. In general, the apparent resistivity values range from 1,000 to 30,000 Ω.ft, which is both in line with textbook resistivity values for the shallow geological conditions and previous work conducted at the project site (AKLNG Document No. USAL-FG-GRZZZ-00-002015-005[31]). A summary of the 9 VES test results is presented on Plate 5 with an average trend line. It should be noted that the trend line does not take into consideration the data collected at E-5, E-6 and E-9, due to erroneous data and possible subsurface metallic structures that would act as subsurface electrical conductors (AKLNG Document No. USAL-FG-GRZZZ-00-002016-001).

2.10 Onshore Geologic Hazards

The above-mentioned studies, previous reports, and 2014-2015 geotechnical, geophysical, and geologic data were reviewed to perform an assessment of the geologic hazards that may impact the

[30] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.
[31] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-7, AKLNG Document No. USAL-FG-GRZZZ-00-002015-005, Rev.0, dated June 10, 2016.

siting or design of the proposed facility. LNG Facilities Geologic Hazard Report (AKLNG Document No. USAL-FG-GRHAZ-00-002015-002[32]) provides complete documentation of these assessments. Geologic hazards assessed include surface faulting, folding and tilting, effects of earthquake-related strong ground motions, tsunami inundation, coastal erosion, volcanic hazards, seafloor erosion and sedimentation, slope failure, fluid expulsion features, sea level change, and anthropogenic hazards. These results are summarized below:

Geologic and Tectonic Setting

- The LNG site is located within the active convergent margin between the North American Plate and the subducting Pacific Plate. The region is characterized by high rates of seismicity and relatively frequent moderate to great (>M8) earthquakes. The subducting Pacific Plate underlies the site and is the primary source for great earthquakes in the region. The LNG site is located in the center of the Cook Inlet Basin, a forearc basin filled with more than 20,000 feet of Tertiary sediments. East-west compression has caused folding and blind thrust faulting of these sediments. Tertiary sediments are unconformably overlain by several hundred feet of Quaternary glacial deposits.
- Seismic reflection data from the oil and gas industry and marine and onshore seismic reflection data collected during the 2015 Geotechnical and Geophysical Program were used to interpret the geometry and locations of fold and faults, any part of which lies within 5 miles of the site. The LNG site is located in a synclinal flat between the Middle Ground Shoal Anticline to the west, and the Kenai-Cannery Loop monocline to the east. Each of these folds is cored by a blind thrust fault, located 4 to 5 miles away on either side of the LNG site center. Based on interpretation of 2015 seismic reflection data, no tectonic fault (neither blind or surficial) is present beneath the onshore and/or marine LNG facilities areas.
- Onshore and marine LNG sites are underlain by Quaternary glacial and glacially-derived deposits, consisting of sand, silt, gravel, boulders, and clay. Onshore, sandy glacial outwash deposits 40 to 80 feet thick from the Killey glacial advance (17,500 to 18,500 years ago) overlie finer grained subestuarine deposits of the Moosehorn glacial advance (18,500 to 32,000 years ago). In the marine area, the Killey and upper Moosehorn deposits have been eroded away, and the sea floor is underlain by Moosehorn and older Pleistocene deposits.

Hazard Assessments

- Surface faulting hazard was evaluated through lineament analysis, review of information regarding known surface faults, mapping of stratigraphic contacts in coastal bluff exposures, and analysis of shallow and deep geophysical data. The surface faulting hazard to the LNG facility is considered low. There is very low hazard of tectonic fault rupture, as seismic reflection data show no tectonic faults are present beneath the site. The hazard of lateral spreading from strong ground shaking is low but cannot be ruled out. Surface faults are

[32] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Geologic Hazard Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-10, AKLNG Document No. USAL-FG-GRHAZ-00-002015-002, Rev.1, dated October 05, 2016.

documented in the Nikiski area outside the LNG facilities area. These structures displace Quaternary deposits only, and could be either the result of strong ground shaking, or may have formed by glacial processes. These faults show no geomorphic evidence of recent displacement.

- The hazard of tectonic tilting and folding is considered very low. Geomorphic evidence for rates of tilting was evaluated through examination of paleo-shorelines along kettle lakes, profiles of the glacial outwash plain, and evaluation of the slope of the Tertiary unconformity from marine seismic reflection data. Observed geologic rates of tilting are very low and do not pose a hazard to the site.
- Strong ground motions are considered a potential ground deformation hazard to the site. Strong ground motion from the 1964 M9.2 Great Alaskan Earthquake caused ground fissuring, liquefaction, and slope failure throughout the Kenai Lowland. Geotechnical studies, summarized in *Section 1.3 Report Format*, assess liquefaction potential and slope stability on a site-specific basis.
- Tsunami inundation hazard is considered to be low for the onshore facilities and moderate for the marine facilities. Tsunamis at the site could be generated by a great earthquake like the 1964 event, by a flank collapse of Augustine volcano, or by a submarine landslide in the Cook Inlet. Tsunami modeling, conducted in 2015 and updated in 2016, simulates a maximum wave height of 39 feet (MLLW) at the highest astronomical tide from the landslide source, with significantly lower heights from the other two sources[33,34]. This wave would not reach the onshore LNG facilities, located on the bluff at elevation approximately 130 feet (MLLW), and would not reach the base of the pile caps on the marine trestle at minimum elevation of 42 feet. The wave may, however, increase erosion of the coastal bluff.
- Coastal erosion is a potential hazard to the facilities. The coastal bluffs in the LNG facilities area are 100 to 120 feet in elevation (NAVD88), and slope 35 to 40 degrees up from the beach to the crest of the bluff. The bluffs erode as a result of waves undercutting of the base of the bluff, followed by shallow land sliding, raveling, and gullying of the bluff face. Long term rates of bluff retreat are estimated at approximately two feet per year, with as much as 50 feet occurring during a powerful storm event.
- Volcanic ash fall is a potential hazard to the onshore and marine facilities. The Nikiski site is located across Cook Inlet from the Aleutian Range, a chain of active volcanoes. Five historical eruptions have deposited ash in the Nikiski area since 1976. Ash fall thickness may vary from a fraction of an inch to several inches. Ash-fall events can hamper plant operations, damage

[33] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Geologic Hazard Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-10, AKLNG Document No. USAL-FG-GRHAZ-00-002015-002, Rev.1, dated October 05, 2016.
[34] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Seismic Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-13, AKLNG Document No. USAL-FG-GRZZZ-00-002015-003, Rev.0, dated June 20, 2016.

machinery, slow or halt vehicle transportation, ship traffic, and aviation, and impact human health.

Additionally, as a part of the geologic hazard analysis, a seismic source model was developed for the site as input to a probabilistic seismic hazard assessment (AKLNG Document No. USAL-FG-GRHAZ-00-002015-001[35]). The seismic source model, also referred to as a seismic source characterization, included: (1) delineation of active fault traces within 200 km of the site, (2) geometric approximation of each fault plane, and (3) characterization of each fault in terms of fault length, depth, dip of fault plane, sense of slip, slip rate, and maximum earthquake magnitude. The seismic source model developed was based on information obtained through review of the seismotectonic setting and historical seismicity. Characterization of the blind thrust faults nearest the site was based on Fugro interpretation of industry seismic reflection data and 2015 project geophysical data[36].

Based on deaggregation by magnitude and distance, for both short and longer structural periods, it was concluded that the seismic hazard at the project site is controlled by three scenarios:

a) larger magnitude earthquakes, between M 9.0 to 9.2, at distances between about 31 to 62 mi (50 to 100 km) attributed to the megathrust events on S2 interface (the plate boundary at Prince William Sound),

b) intermediate to large magnitude events (i.e., M 7.0 to 8.0) at distances between about 31 to 92 mi (50 to 150 km) associated with the intermediate depth (31 to 50 mi (or 50 to 80 km)) and deep intraslab subduction earthquakes, and

c) intermediate to large magnitude earthquakes (i.e., M 7.0 to 7.5) at distances about 3.1 to 6.2 miles (5 to 10 km) associated with the shallow depth (< 31 mi (or 50 km)) from the project site.

2.11 Probabilistic Seismic Hazard Assessment (PSHA) & Site Response

The Alaska LNG Project site is located at the active convergent margin between the North American Plate (Southern Alaska block) and the Pacific Plate. The project site is centered in the Cook Inlet basin, which is characterized by high rates of seismicity, and relatively frequent moderate to great earthquakes. Site-specific seismic hazard analyses were conducted to estimate the severity of ground motions that may affect the onshore and marine structures which will be developed as a part of the LNG project. Those results were used to conduct liquefaction potential evaluation for the

[35] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Probabilistic Seismic Hazard Analysis (PSHA) Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-6, AKLNG Document No. USAL-FG-GRHAZ-00-002015-001, Rev.0, dated May 05, 2016.
[36] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-7, AKLNG Document No. USAL-FG-GRZZZ-00-002015-005, Rev.0, dated June 10, 2016.

onshore and marine areas. Detailed results, including maps of estimated liquefaction-induced settlements, are provided in AKLNG Document Nos. USAL-FG-GRHAZ-00-002015-001[37] and USAL-FG-GRZZZ-00-002016-008[38]. A brief summary of our findings and results is provided below:

Based on the information gathered from available geotechnical data for project facilities (including the in-situ shear wave velocity measurements, Interferometric Multichannel Analysis of Surface Waves and Standard Penetration Tests), PSHA was conducted at ground surface with a time-weighted average shear wave velocity (V_s) in the top 100 feet (approximately 30 meters) (i.e., 100 ft/integral from 0 to 100 ft of dz/Vs(z)) of 885 ft/s (270 m/s).

Near-surface design spectra were developed in accordance with the requirements of National Fire Protection Agency (NFPA) 59A (2006) and ASCE 7-05 guidelines for project facilities. According to NFPA 59A (2006), two levels of ground motions are developed for: Operational Basis Earthquake (OBE) corresponding to 475-year return period, and a more severe Safe Shutdown Earthquake (SSE), which is the same as Maximum Considered Earthquake (MCE) per ASCE 7-05. According to ASCE 7-05, two levels of design ground motions are developed for: Maximum Considered Earthquake (MCE) and Design Earthquake (DE). Similar results per NFPA 59A (2013) are provided in the Appendix C of AKLNG Document No. USAL-FG-GRHAZ-00-002015-001. The estimated near-surface horizontal Peak Ground Acceleration (PGA) values at different shaking levels are reported in Table 2.1.

Table 2.1: PGA Horizontal Component at the Ground Surface (V_{s30} = 885 ft/s)

NFPA 59A (2006)		ASCE 7-05	
Operating Basis Earthquake (OBE), (g)	Safe Shutdown Earthquake (SSE), (g)	Maximum Considered Earthquake (MCE), (g)	Design Earthquake (DE), (g)
0.528	0.897	0.897	0.598

The vertical response spectra were developed by applying vertical to horizontal (V/H) spectral ratios to the ground surface horizontal response spectra. The estimated near-surface vertical peak ground acceleration values at different shaking levels are reported in Table 2.2.

[37] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Probabilistic Seismic Hazard Analysis (PSHA) Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-6, AKLNG Document No. USAL-FG-GRHAZ-00-002015-001, Rev.0, dated May 05, 2016.
[38] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Seismic Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-9, AKLNG Document No. USAL-FG-GRZZZ-00-002016-008, Rev.0, dated December 16, 2016.

Table 2.2: PGA Vertical Component at the Ground Surface (V_{s30} = 885 ft/s)

NFPA 59A (2006)		ASCE 7-05	
Operating Basis Earthquake (OBE), (g)	Safe Shutdown Earthquake (SSE), (g)	Maximum Considered Earthquake (MCE), (g)	Design Earthquake (DE), (g)
0.407	0.691	0.691	0.461

Liquefaction potential evaluation was conducted in accordance with the requirements of both NFPA 59A (2006) and ASCE 7-05 for onshore and marine area based on the available geotechnical data from: a) 25 onshore borings presented in the Geotechnical Data report (AKLNG Document No. USAL-FG-GRZZZ-00-000003-000[39]), b) 62 onshore borings presented in the Onshore Geotechnical Data report (AKLNG Document No. USAL-FG-GRZZZ-00-002015-006[40]) and (3) 43 onshore borings performed during 2016 field investigations and presented in the Onshore Geotechnical Data report (AKLNG Document No. USAL-FG-GRZZZ-00-002016-001 [41]). Liquefaction triggering hazard assessment using the NCEER (Youd et al., 2001) and the Boulanger and Idriss (2014) empirical relationships was performed for the 130 onshore borings.

Based on these evaluations, generally, continuous potentially liquefiable soil layers were not identified at the site except surficial granular soils in the top 3 feet at the beach area (borings B-117, B-120, B-121, B-122, B-123, B-124, B-125, B-126, B-127, B-190 and B-191) which are expected to liquefy for both OBE and SSE shaking levels. Especially for beach boring B-191, the extent of liquefaction is expected to reach 9 ft depth. For the rest of borings, limited and localized liquefaction is estimated during the OBE and SSE shaking levels. For example, sublayers of coarse-grained soils (sand with gravel) in the depth range of approximately 20 to 25 feet were found to be potentially liquefiable in borings B-28 and B-39 (6 and 4 feet thick, respectively), for both SSE and OBE events, however these borings are located outside the locations of the planned structures. Some localized pockets of potentially liquefiable coarse-grained soils were also identified in borings B-17, B-43, B-50, B-61, B-64, B-67, B-76, B-146, B-160 and B-166. In addition, boring B-176 which is close to the coast and located on the alternative heavy haul road area north from the project site, contains a potentially liquefiable soil layer from approximately 18 to 34 ft depth. Liquefaction triggering hazard assessment was also performed for OBE and SSE shaking levels per NFPA 59A (2013) requirements. Results are similar to the SSE / MCE event per NFPA 59A (2006).

[39] Fugro Consultants, Inc. (Fugro), 2015, Geotechnical Data Report Onshore LNG Facilities, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-8, AKLNG Document No. USAL-FG-GRZZZ-00-000003-000, Rev.0, dated May 08, 2015.
[40] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[41] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.

Liquefaction-induced reconsolidation settlements are estimated in the range of approximately 0 to 0.5 inches for the onshore borings during both the OBE and the SSE / MCE shaking level, with negligible estimated settlements at most locations. The estimated settlements are in the range of 0.8 to 1.8 inches for borings B-28, B-39, B-61, and B-76. However, these borings are located outside the locations of the planned structures. The estimated settlements for boring B-176 are higher than 3 inches, but boring B-176 is located on the alternative heavy haul road area north from the project site. The estimated settlements at the beach area (borings B-117, B-120, B-121, B-122, B-123, B-124, B-125, B-126, B-127, B-190 and B-191) are in the range of 0.6 to 1.9 inches during both the OBE and the SSE / MCE shaking. However, these values will be reduced significantly if the top 3 feet surficial granular soils at the beach area are dredged or densified.

A screening assessment of the liquefaction susceptibility of fine-grained soils for the onshore borings was performed based on the procedures of Bray and Sancio (2006). Results showed that some sublayers classified as sandy silts (ML) and lean clays (CL) might be susceptible to liquefaction. But, these layers are thin and discontinuous and hence, soil liquefaction will likely be localized and discontinuous if it occurs[42].

2.12 Hydrogeological Study

The hydrogeological study components of Fugro's site investigations completed in 2014, 2015, and 2016 (Fugro Report No. 04.10160001-8 43) included observing geologic and hydrogeologic conditions encountered in 130 onshore borings, and during installation and monitoring of twenty-six (26) groundwater monitoring wells, four (4) observation wells (OW), and three (3) aquifer pump test (APT) wells. In addition, the hydrogeologic study included reviewing groundwater quality data collected in 2016. It should be noted that long-duration (approximately 10 day) pump tests designed to stress possible source water aquifers were planned but removed from the study due to time constraints on the regulatory approval process for upland discharge of pump test waters.

Data collected and reviewed to date provides a means to develop an understanding of aquifer characteristics to a depth of about 437 feet below existing site grade. A summary of our findings such as water bearing units, artesian conditions, and tidal influence on various water bearing units is presented below.

- Groundwater has been observed in 3 distinct water bearing units below the site. The first encountered water bearing unit (Water Bearing Unit 1) is observed to be unconfined and lies above the Killey-Moosehorn transition zone. Water Bearing Units 2 and 3 are present within the Moosehorn geologic unit. Water Bearing Unit 2 is confined or semi-confined, lies immediately beneath the Killey-Moosehorn transition zone, and extends to a competent clay

[42] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Seismic Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-9, AKLNG Document No. USAL-FG-GRZZZ-00-002016-008, Rev.0, dated December 16, 2016
[43] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Hydrogeologic Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-8, AKLNG Document No. USAL-FG-GRZZZ-00-002016-007, Rev.A, dated November 21, 2016.

layer varying from approximately elevation -100 to -150 feet (NAVD88). Water Bearing Unit 3 is confined and was encountered as a separate unit below approximately elevation -150 feet (NAVD88) and extended to the total drilled depth of APT-3 at elevation -320 feet (NAVD88).

- Water elevation contours for Water Bearing Units 1 and 2 are separately shown on Plate 6a of this report. Water elevation contours are interpreted based on September 2016 static water depth measurements of wells screened within a given Water Bearing Unit. Where dry well conditions were encountered, the depth to a static water level was inferred to be equivalent to a depth of 5 feet above the interpreted Kiley/Moosehorn contact at a given well. Plate 6b provides an interpretation of depth to water measurements based on data from Water Bearing Unit 1 wells only and groundsurface topography.
- Water within Water Bearing Unit 1 occurs within geologic materials which are both laterally and vertically discontinuous. Due to the lenticular, discontinuous nature of these deposits, groundwater levels within this unit appear somewhat sporadic across the site, and dry well conditions have been observed.
- Artesian groundwater conditions are observed in select wells installed into Water Bearing Units 2 and 3, with the groundwater level in these wells measured above the aquitard at the Killey/Moosehorn transition zone. However, no flowing artesian conditions have been encountered at any of the well locations installed as part of studies completed to date.
- Artesian groundwater conditions were observed during drilling of two beach borings completed during the 2015 field campaign. At these locations, B-117 and B-136 artesian conditions were encountered at elevations of approximately -84 feet (NAVD88) and -78 feet, respectively. These two borings are located in the southwestern area of the site investigation program. It appears that the widely varied, lenticular, and heterogeneous properties of the sediments encountered in these borings are acting as confining layers, controlling the presence and nature of the artesian conditions. An artesian condition was observed to be flowing (where enough hydraulic pressure exists to push the groundwater up to the ground surface) at boring B-136. Artesian conditions were also observed in one offshore boring, MB-24. At this location, a flowing artesian condition was encountered at an elevation of approximately -100 feet (NAVD88).
- Water Bearing Unit 1 is observed not to be tidally influenced. Water Bearing Units 2 and 3 are observed to be tidally influenced at various degrees of correlation. In general, a higher degree of correlation exists closer to the coastline. A stronger correlation between Cook Inlet tides and groundwater level fluctuation was noted within well APT-3 (Water Bearing Unit 3) than at co-located well APT-1 (Water Bearing Unit 2).
- Groundwater elevations in several Water Bearing Units 1 and 2 wells exhibited a response to a January 24, 2016 magnitude 7.1 earthquake located in the Iniskin area of Alaska, approximately 60 miles west of Homer, based upon water level data retrieved from the wells. The change in groundwater level response was greatest at the location of well MW-

74A, which was noted to drop approximately 3 feet, then rebounded and sustained an approximately 2 feet rise above its pre-earthquake level within the span of about 1 hour. It appears that the water level in this well returned to within its normal fluctuation within a period of about 4 months.

- Groundwater elevations have been observed to vary seasonally and annually at the site as observed best by reviewing conditions within Water Bearing Unit 1 wells. The groundwater elevations within Water Bearing Unit 1 wells declined 2 to 5 feet from December 2014 through October 2015, at which time elevations are observed at their lowest. Groundwater levels reach their highest elevations in February 2016, at which time groundwater elevations are noted to begin their seasonal decline.
- Groundwater is inferred to flow in a west/southwest direction at a gradient of approximately 0.008 to 0.01 feet per feet (ft/ft) in Water Bearing Unit 1. The inferred groundwater flow in Water Bearing Unit 2 is to the west/southwest direction at a gradient of approximately 0.008 to 0.06 ft/ft. The groundwater gradient is noted to steepen as it approaches the Cook Inlet. There is insufficient data upon which to provide comment regarding water flow and direction in Water Bearing Unit 3.
- Although there is likely communication between water within Water Bearing Unit 1 and Water Bearing Unit 2 at the Site, there was no measurable decrease in water elevation in nearby Water Bearing Unit 1 wells during the development water removal from wells screened within Water Bearing Unit 2. This may be attributable to the limited nature of the pumping activities, the presence of a relatively competent aquitard separating Water Bearing Unit 1 and Water Bearing Unit 2 in this area, or a combination of these or other possible factors.
- It is evident that good hydraulic communication within Water Bearing Unit 2 exists between the locations of wells APT-1, OW-2, and OW-4. There was no discernable water elevation response at well MW-82A (located approximately 1,450 feet northeast of well APT-1); indicating that there is an impediment to hydraulic communication somewhere between the locations of wells APT-1 and MW-82A and/or the lack of response is a reflection of the shortened development activities which did not stress the aquifer system sufficiently.
- During development water removal from well APT-2, the water elevation recorded within well OW-4 (located approximately 500 feet north of well APT-2) was observed to quickly respond to the water removal activities. Groundwater elevations within well OW-2 (located approximately 1,225 feet north-northwest of well APT-2) did not show a discernable response to well APT-2 groundwater development activities; indicating that there may be an impediment to hydraulic communication somewhere between the locations of wells APT-2 and OW-2, and/or the lack of response is a reflection of the shortened development activities which did not stress the aquifer system sufficiently.
- During the installation and development of wells APT-1 and APT-3, no sequences of lithology conducive to high groundwater yield were encountered. Observations during development suggest a lack of recharge which significantly limits pumping rates at this location. This may

be reflective of geologic processes and conditions found within the northeast-southwest trending geological feature found in this area.

- Both naturally occurring and anthropogenic sources of groundwater quality degradation and variation are present in the study area. Groundwater in the proposed well field area is impacted by concentrations of antimony, arsenic and trichloroethene that exceed the concentrations allowed by the regulatory water quality standards. The source(s) of the antimony and arsenic are likely naturally occurring sediments, whereas the trichloroethene source is an unknown anthropogenic source.
- The quality of water also varies between the 3 observed water bearing units. Total arsenic concentrations within Water Bearing Unit 2 and Water Bearing Unit 3 are up to two orders of magnitude greater than concentrations within Water Bearing Unit 1. pH values measured in Water Bearing Unit 1 well samples suggest that groundwater in that unit is slightly acidic to neutral; pH values measured in Water Bearing Unit 2 and Water Bearing Unit 3 groundwater suggest that those waters are neutral to alkaline.

The Site hydrogeology is a direct result of the complex depositional environment, and subsurface groundwater conditions vary significantly, both vertically and laterally. Although some hydrogeological parameters of Water Bearing Units 1, 2, and 3 have been observed, evaluated, and documented herein, a full-scale long duration (approximately 10 day) aquifer pump test, as originally planned, is necessary to properly evaluate properties of the water source aquifers below the Site and their ability to meet the demand based on the LNG project design criteria.

The location of the LNG project production well field will also need to be reconsidered given the presence of the chemical contaminant trichloroethene in Water Bearing Units 2 and 3 in the currently proposed well field area. Water removal from the currently proposed well field area during well development activities generated contaminated water which was unsuitable for upland discharge. Continuous pumping of water for long term use may move the contaminated water and require additional treatment prior to use. As a result, Fugro recommends that the location of the production well field be moved to another location. Areas further to the north, beyond the subsurface geologic structure, may provide better water quality and capacity.

2.13 Cold Region Considerations

Lying between the Cook Inlet and the Kenai Mountains, Nikiski has a transitional climate which may be characterized as variable with influence of both maritime and continental climate regimes. The temperature ranges from extremes of about -47°F to 87°F with an annual mean of 34°F. The mean monthly temperature ranges from about 12.5°F in January to 54.7°F in July. A summary of climatological data obtained from the Kenai Municipal Airport, and the Anchorage International Airport is presented in Table 2.3.

Table 2.3: Climatological Data

Location (Station)	Kenai Municipal Airport	Anchorage International Airport
Period of Record	1952-2014	1952-2014
Mean Annual Temperature (F°)	34.3	36.1
Mean Maximum Temperature (F°)	42.5	43.1
Mean Minimum Temperature (F°)	26.1	29.4
Record High Temperature (F°)	87.0	85.0
Record Low Temperature (F°)	-47.0	-34.0

Source: Western Regional Climate Center (WRCC)

The following air freezing indices were calculated from average monthly temperature data from Kenai Municipal Airport. The average air freezing index (AFI) for the period of record (1952 – 2014) for Kenai is 2,125 °F-days. The maximum AFI of 3,524 °F-days was recorded in the winter of 1955/1956. In the last 30 years, the maximum recorded AFI was 2,789 °F-days in 2011/2012. The Design AFI is often defined as the average of the three coldest AFIs in the past 30 years, which is 2,678 °F-days for Kenai. However, the extreme AFI for the period of record is often used for critical or frost sensitive structures. For reference, an AFI of 3,500 °F-days is commonly used for design in the Anchorage area which has a climate very similar to Kenai.

Permafrost. Permafrost is defined as soil, rock or sediment that remains at or below 0° C, in other terms frozen, at least two consecutive years. A typical classification recognizes continuous permafrost (underlying 90-100 percent of the landscape); discontinuous permafrost (50-90 percent); and sporadic permafrost (0-50 percent). In the Northern Hemisphere, regions in which permafrost occurs occupy approximately 25 percent (23 million km²) of the land area. In the discontinuous and sporadic zones permafrost distribution is complex and patchy, and permafrost-free terrain is common. The thickness of permafrost varies from less than 3 ft to more than 5,000 ft.

In the state of Alaska, regions of continuous permafrost, discontinuous permafrost, sporadic permafrost, and no-permafrost exist. However, the project site is located at Kenai Peninsula near Nikiski, Alaska which is a non-permafrost region and therefore permafrost considerations are not required.

Depth of Seasonal Frost. The depth of seasonal frost penetration at any given location is a function of climate, soil properties, and groundwater conditions. The following characterizes the potential depth of maximum annual frost penetration (following ARMY TM 5-852-41[44]) expected at the project site as a function of surface conditions.

[44] Joint Departments of the Army and the Air Force, USA, 1983, Arctic and Subarctic Construction, Foundations for Structures, Technical Manual TM 5-852-4/AFM 88-19, Chapter 4.

Table 2.4: Estimated Maximum Depth of Frost Penetration

Surface Condition	Estimated Depth of Seasonal Frost Penetration (ft)	
	Average (AFI=2,100°F-days)	Extreme (AFI=3,500°F-days)
Turf	4 to 6	7 to 9
Paved and Gravel Surfaces	6 to 8	9 to 11
Behind Retaining Wall	7 to 9	10 to 12

Groundwater at the site was only encountered at depths lower than El. +90 ft. Based on the evaluations of climate (using AFI of 2,100°F), water table, and frost susceptibility of soils (discussed in later *Section 5.3.9 Frost Susceptibility* of this report) the frost penetration at the project site is estimated at about 7 ft (Table 2.4), which applies to the depth below final grade.

Frost susceptibility was determined by performing frost heave tests on coarse grained soil samples, collected as part of 2015 and 2016 G&G programs, from the top 20 ft of AKLNG site. Frost heave laboratory tests were performed by Dr. Jean-Marie Konrad, professor of civil engineering at Laval University, Quebec City, Canada.

- A total of 7 samples were tested in 2015, in order to evaluate the frost susceptibility of the soils located within LNG facilities study area. Two of the samples were reconstituted such that the fines (percent passing No.200 sieve) were about 21 and 32 percent. The maximum heave observed in the test samples was about 2.4 percent for the sample with fines content of 32 percent at vertical stress of about 0.5 psi.
- A total of 8 samples were tested in 2016, at the proposed locations of the critical structures. One sample was reconstituted such that the fines (percent passing No.200 sieve) was about 20 percent. The maximum heave observed in the test samples was about 0.24 percent for the sample with fines content of 20 percent at vertical stress of about 0.5 psi.

Based on the performed tests listed above, the surficial granular soils at the site can be classified as low to non-frost susceptible (AKLNG Document No. USAL-FG-GRZZZ-00-002016-001[45]).

We expect that the bearing capacity and movement of foundations that are placed below the frost penetration depth will not be affected by the frost-heave phenomenon. Buried pipes in areas of frost susceptible soils should be placed either below the frost penetration depth of 7 ft or at shallower depths with properly designed thermal insulation. Spread footings should be placed at a depth of 7 ft or at a minimum depth of 4 ft with properly designed thermal insulation. Spread footings may bear on existing soils at a depth of 4 ft without thermal insulation, provided the soils to a depth of at least 7 ft are non-frost susceptible or the footings can tolerate frost-related heave. Frost related heave of spread footings supported at 4 ft over frost susceptible soils will be dependent on the fines content

[45] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.

of the bearing material. Frost heave of up to ¾ inch can be expected, assuming the fines content of bearing soils is not greater than 32 percent. Lightly-loaded mat foundations and slabs-on-grade, constructed without thermal insulation, and supported in on frost susceptible soils may experience more than 1-inch heave depending on the loading and fines content of bearing soils.

3.0 INTEGRATED SITE MODEL

3.1 General

Results from various studies (*Section 2.0*) were analyzed and interpreted, and all applicable data have been integrated to develop a surface and subsurface model, or an integrated site model. Integrated geoscience studies provide a comprehensive approach to data collection and analysis. The GIS database created for the Alaska LNG Project (utilizing ESRI's ArcGIS suite of software) is used for the organization, integration, analysis, storage, retrieval and display of geophysical, geologic, and geotechnical data.

This section discusses the integrated site model and describes the relationship between subsurface geophysical data, physical properties observed in geotechnical borings, in-situ and laboratory test results, and features identified in geologic mapping and the literature.

3.2 Topography and Geomorphology

Topography of the onshore LNG Facilities study area consists of a gently sloping plain broken by swaths of northeast-trending kettle lakes and depressions, and bounded on the west by coastal bluffs at the shore of the Cook Inlet. The surface of the plain is underlain by glacial outwash debris that forms alluvial fan extending south-southwest across the site from the moraine ridges and hills at the northern tip of the Kenai Peninsula. Plate 7 shows the topography within the LNG facilities study area. The existing site grade within the footprint of the main onshore facilities is relatively flat, varying between El. +140 ft to north/northwest to about El. +120 ft south/southwest, consistent with the downslope direction of the outwash fan. Swaths of northeast-trending kettle lakes and closed depressions (the largest of which include Bernice Lake and Island Lake) have bottom elevations as low as El. +80 ft. These bands of kettle holes and lakes within the outwash plain are aligned with former glacial ice fronts, and are prominent in the southern portion of the LNG facilities study area. Two privately owned gravel pits (one on the north and one centrally located) are visible as depressions with bottom elevations of El. +80 ft.

The site is bounded on the west side by coastal bluffs that rise to about El. +140 ft on the north and El. +100 feet to the south with the toe of the bluff at an approximate elevation of El. +20 ft. Buffeted by storm waves and subjected to a 20 to 25 feet tidal fluctuations, the coastal bluff evaluations (AKLNG Document No. USAL-FG-GRHAZ-00-002015-002[46]) indicate the bluff to be slowly eroding by raveling, wave undercutting, gullying, and shallow land sliding.

[46] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Geologic Hazard Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-10, AKLNG Document No. USAL-FG-GRHAZ-00-002015-002, Rev.1, dated October 05, 2016.

3.3 Subsurface Model

Using ArcGIS, a subsurface site model was developed based on the results of relevant studies that describe the relationship between site geology, subsurface geophysical data, physical properties observed in geotechnical explorations, hydrogeology, in-situ and laboratory test results, and topographic features identified in geologic mapping. The studies used in developing the subsurface model with respective summaries are presented in *Section 2.0 Relevant Studies*. A brief summary of the model is presented below as a series of statements that reflects current understanding of the site geologic history, geomorphology, stratigraphy, and hydrogeology.

- The site lies on the Kenai Peninsula, located on the North American Plate, above the subducting Pacific Plate. The site stratigraphy consists of Mesozoic basement rock, overlain by approximately 20,000 feet of Tertiary marine sedimentary rocks of the Cook Inlet Basin, which is in turn overlain by 300 to 500 feet of Quaternary (Pleistocene) glacial sediments. The upper 25 to 85 feet of Quaternary sediments consist of stratified glacial outwash deposits (sand, gravelly sand, and silty sand) that form a gently sloping outwash plain. Holocene deposits discontinuously overlie the Late Pleistocene deposits.

- Seismic reflection data, from industry and project-generated marine and onshore deep seismic surveys, show the site to be located in a synclinal flat on the east limb of the Middle Ground Shoal Anticline. Bedding in the Sterling Formation dips gently to the southeast (Plate 8a). Folds on either side of the syncline are cored by blind thrust faults, which do not reach the surface. Seismic reflection data show continuous planar reflectors in the Tertiary strata beneath the LNG facilities study area, indicating an absence of tectonic surface faulting beneath the site.

- On the outwash plain, Holocene deposits form a veneer up to 5 feet thick of loess, tephra, and organic-rich topsoil. Within the kettles, Holocene deposits may be tens of feet thick, consisting of silt, clay, peat, diatomaceous earth, and tephra overlying the Killey deposits. The Killey deposits sit on subestuarine deposits of the Moosehorn glacial sediments (deposited 18,500 to 32,000 years ago), which acts as an aquitard over much of the site (Plate 8b). A brief summary of these three strata and their properties are summarized in Table 3.1.

- On the seafloor, waves, ridges, and sheets of loose sand, gravel, and silt deposited by modern currents and waves discontinuously mantle the sea floor, tidal flats, and beach (marine Unit 4, presented on Plate 8b). Throughout the marine terminal area, the Killey deposits and upper beds of the Moosehorn deposits have been removed by erosion and the seafloor is underlain by the lower beds of the Moosehorn deposits and older Pleistocene deposits (marine Units 2 and 3). (Plate 8b). These Pleistocene deposits, 100 to 200 feet thick in the marine area, consist of glacially derived fluvial and subestuarine deposits and minor

glacial till. The Pleistocene deposits overlie an erosional surface, or unconformity, cut on gently folded Tertiary sediments of the Pliocene Sterling Formation (Plates 8a and 8b). The base of the Moosehorn deposits is not well defined, hence is shown by a dashed line.

- A preliminary integration of available borings and modeled resistivity (ERT) values was used to develop order of magnitude estimate of topsoil and gravel thicknesses within the LNG Facilities Study Area project boundary. We have adopted the terminology "order of magnitude estimate" to account for the potential inaccuracies associated with interpolating data across the relatively large spacing between the ERT lines and geotechnical borings. Following interpolations were developed:
    - Topsoil: Interpolated thickness of topsoil (including clays and silts) generally ranges from 0 to 6.5 ft. Based on the ERT data and geotechnical borings (B-107 and B-137) Topsoil thicknesses may be up to 12 ft in localized areas (see Plate 4a). Additionally, anomalies in some of the low-lying areas indicate interpreted thickness of clayey/silty soils with thickness of up to and about 30 ft in these areas (see hatched zones on Plate 4a). Based on the field observations, Topsoil does not exist within the developed areas such as roads, quarries, industrial plants, lay-down areas, gas stations, etc. Major parts of the LNG Facilities Study Area where Topsoil is absent are also approximately marked on Plate 4a.
    - Gravel: Interpolated thicknesses of gravel show appreciable concentrations may be encountered within the top 30 ft as shown on Plate 4b. Depth to appreciable concentrations of gravel is also shown on Plate 4b.
- Shallow seismic surveys provided information on sedimentary layering in the Killey and Moosehorn deposits within the 30 to 80 ft depth range. Also, the survey provided some soil parameter estimates useful for engineering analyses.
- The hydrogeology study provided a groundwater model for the site. The depth-to-water in the upper Killey deposits (Water Bearing Unit 1) is at about El.+40 near the bluff and rising to about El.+100 on the eastern edge of the project site.
- Borehole data and coastal bluff exposures suggest that the stratigraphy of the Killey deposits beneath outwash plain areas differs somewhat from the stratigraphy beneath the zones of kettles. Beneath the outwash plain, the deposits are stratified and relatively planar. Individual layers of alluvium may be elongated in the downslope direction (southwest) and lenticular in the cross-slope direction. The basal contact of the Killey deposits is also relatively planar. Within the bands of kettles, stratigraphic marker beds dip down and follow the shape of the depression or may be truncated at its margin. The basal contact is depressed downward as well.

- The Killey/Moosehorn contact was identified in boreholes, primarily on the basis of the contrast in grain size, and secondarily on a contrast in stiffness or blow count, or shear wave velocity contrast on IMASW geophysical profiles. Based on the borings drilled onshore on top of the bluff during the 2014, 2015 and 2016 studies, the elevation of the Killey/Moosehorn contact averaged 49 feet, and ranged from 15 to 73 feet (NAVD88). Shallow seismic (IMASW) surveys show the contact to be planar to undulating in shape. The Moosehorn deposits are finer than the Killey deposits and the transition zone from Killey to Moosehorn units can be observed as an increase in fines content and shear wave velocity. Additional discussion of the nature of this contact follows:

  The contact between the Killey and Moosehorn deposits is a prominent stratigraphic marker throughout the study area; however, the identification of this contact in boreholes is not straightforward in all cases. Cross-sections (Plates 9 through 14) illustrate a “best estimate” of the position of contact, with dashed lines and queries to indicate uncertainty.

  Due to the discontinuous nature of the dataset and uncertainties in precise identification of the contact, a robust and detailed view of the geometry of the Killey-Moosehorn is not possible. A general view, however, shows that this contact underlies the entire LNG Facilities study area, is a clear planar boundary in some areas, and locally exhibits depressions and downwarps, often coincident with depressed relief in the surface topography. A brief summary of the location of these three strata (topsoil, Killey, and Moosehorn) and their properties are summarized in Table 3.1.

- Shear wave velocity profiles from shallow seismic survey (IMASW), downhole-seismic tests, and SPT derived shear waves were plotted to evaluate the overall velocity change with elevation and type of material encountered (Plate 15).

Table 3.1: Generalized Subsurface Model

Strata	Elevation (ft) (Max Top to Min Bottom)	Depth (ft) (Min Top to Max Bottom)	Geologic Unit[3]	Geotechnical Soil Type	Blow Count SPT (N)	% Gravel	% Sand	% Fines	S_u (ksf)
I	137.5 to 92	0 to 6.5[1]	Topsoil[1]	Organic, Silty, Clayey, Sandy Soil	0 to 12	Up to 40	20 to 70	40 to 90	-
II	137.0 to 15.0	0 to 99.0	Killey Glacial Outwash Deposits [2]	Clayey Sand, Sand with Silt and Gravel, Sand, Gravel with Silt and Sand	10 to Ref	Up to 50[4]	30 to 90[5]	1 to 12[6]	5.0 to 7.5[7]
III[3]	72.5 to -180.0	22.0 to 300.5	Moosehorn and older Pleistocene Deposits	Clayey Sand, Sand with Silt, Silty Sand, Sandy Silt, Silt, Lean Clay, Silty Clay	19 to Ref	Up to 30[4]	10 to 95[8]	5 to 95[8]	1.5 to 17.6

Notes: [1] Topsoil may be absent in areas of prior grading. A combination of thicker Topsoil and soft silt and/or clays were encountered in borings B-107 and B-137, and are also interpreted based on ERT data (See Plate 4a).

[2] Killey deposits were not encountered in 11 beach borings and Killey-Moosehorn contact was not differentiated at the ASRC area borings B-176 through B-183 (See Fugro Report No. 04.10160001-5).

[3] In some of the borings, an impermeable aquitard transition layer, consisting of lean clays, silty clays and silts are observed at the contact of the Moosehorn and the Killey deposits.

[4] There are some exceptions to the reported maximum gravel percentage in both Strata II and III. Layers with higher gravel percentages are encountered in few borings. However, generally the gravel percentages are up to 50 and 30 in Strata II and III, respectively.

[5] There are some exceptions to the reported maximum sand percentages in both Strata II and III. Layers with lower sand and higher gravel percentages are encountered in some borings. Also, layers with lower sand and higher fines contents are encountered in few borings, generally at depths close to the contacts of Topsoil-Killey and Killey-Moosehorn. However, generally the sand percentages were in the ranges of 30 to 90 in Strata II.

[6] Generally, percentage of fines (passing No.200) was limited to 12 in Stratum II. However, clay lenses and silt pockets with fines content higher than 12% are also observed in this Stratum.

[7] There are limited clay layers and undrained shear strength test data in the Killey Formation. Clays are encountered generally close to the interpreted Killey-Moosehorn contacts.

[8] Broad ranges for sand and fines contents are reported within the Moosehorn Formation due to the characteristics of this formation (i.e. distinct granular and fine soil layers exist within this formation).

4.0 SITE PREPARATION

4.1 General

AKLNG provided us with proposed design grade elevations in various areas (Drawings USAL-CB-LDLAY-00-000003-000 (Rev.6) dated July 30, 2016, USAL-CB-CDDET-00-000088-001 through 015 (Rev.2) dated May 13, 2016 and a 3D-Model in MicroStation format received on March 22, 2016) of the project site. Fugro generated maps (Plates 16, and 17a through 17f) presenting the approximate cut and fill thicknesses proposed for the LNG facilities study area, using the above-mentioned drawings provided by AKLNG. It should be noted that cut and fill plans for the alternate heavy haul road located within the existing developed area (ASRC) were not available at the time of this report.

Based on the existing topography and the proposed design grade elevations, the project site will require significant cut and fill in some areas as shown on Plate 16. Subsurface conditions are generally uniform other than in some low-lying areas or some isolated areas (discussed in later sections). Therefore, based on the proposed final finished grades in each area, groundwater conditions, and the layout of the proposed structures, we divided the LNG facilities study area into 8 sub-areas for developing our engineering recommendations (Table 4.1). Plates 16, and 17a through 17f show the cut and fill areas within each designated area and generalized final grade in each area for engineering purposes.

It should be noted that the 3D-Model provided to us on March 22, 2016 did not incorporate the significant changes in elevations in the Areas 2, 3, 4 and 5; therefore, cut and fill for Areas 2, 3, 4, and 5 were generated using single design grade elevations selected for this report (as shown on Plate 16), with the following exceptions:

1. Area 5 pond earthwork was modeled using the elevations provided in the 3D-Model for the previous pond location (see USAL-CB-LDLAY-00-000003-000 Rev.3, dated April 14, 2016). This area is designated as “A” on Plate 16.
2. The earthwork on the bluff associated with the trestle structure (marine LNG facilities) was modeled using the cross section shown on USAL-CH-JDFND-90-000020-006 Rev.1, dated January 22, 2016. This area is designated as “B” on Plate 16.
3. The earthwork on the bluff associated with the outfall structure (Area 3) was modeled using assumed slope of 3H:1V and the plan dimensions shown on USAL-CB-CDDET-00-000088-015 Rev.2, dated May13, 2016. This area is designated as “C” on Plate 16.

The following sections present information on low-lying areas, ground improvement techniques, cut and fill considerations, reuse of on-site soils, earthwork considerations, settlement due to site grade raise, and cut and fill slope recommendations.

Table 4.1: Engineering Areas

Engineering Areas	Sub-Areas	Major Structure(s)
Area 1	-	Heavy Haul Road from MOF
Area 2	2A 2B 2C	LNG Tank (North) LNG Tank (South) Trestle, Pipe Racks
Area 3	-	LNG Admin Complex, Condensate and Compressors
Area 4	4A 4B	LNG Train No.3 LNG Trains Nos.1&2
Area 5	-	Gas Treating, Firewater Storage
Area 6	-	Laydown Yard, Quarters, Miscellaneous Structures
Area 7	-	Camp and Miscellaneous Structures
Area 8	-	Alternative Heavy Haul Road

4.2 Low-Lying Areas

The terrain within the project site includes localized low-lying areas such as kettle lakes, closed depressions, and man-made gravel pits with elevations as low as El. +80 ft (see Site Topography Map, Plate 7a). Wet and boggy areas were observed during our field exploration around the lakes and other low-lying areas. These low-lying areas are localized and are primarily in the southern portion of Area 3 (Condensate and Admin Area) and Area 5 (Utility Area) and possibly in the southern portion of Area 4 (Process Train Area). ERT survey identified low resistivity areas within the low-lying areas that were interpreted as clay and/or silt. The kettle lake/low-lying area near Boring B-101, with ground elevation about El. +95 ft, is estimated to have about 30 ft or more of silt and/or clay. Two other locations, between B-39 and B-104 and between HA-110 and HA-111 shown on Plate 4a, were interpreted to have clay/silt thickness of about 15 ft. Soft cohesive soils or loose silts may exist in these and other low-lying areas. Additional field exploration, sampling, and testing may be required to determine the type and consistency (strength and density) of soils in these low-lying areas. Exploration through borings, test pits, and hand augers and ERT survey should be used to further define these areas prior to or during earthwork activities.

Ground improvement should be considered if the silts/clays present in these low-lying areas are found to be marginal/weak and sensitive or critical structures will be constructed there.

4.3 Ground Improvement

In-situ or other ground improvement could be considered in localized low-lying areas (Site Topography Map, Plate 7a) of the site that may include marginal/weak soils below the topsoil. Removal of very soft clays and/or loose silts and replacement with granular fill material should be reviewed for cost effectiveness. Typically, removal and replacement of material to a depth of about 8 ft is cost-effective over other ground improvement options. Ground improvement could be considered as an alternative to removal and replacement. The effectiveness of ground improvement techniques will vary depending on the type, consistency, and depth/thickness of encountered marginal/weak soils. Sampling and testing will be required of soils in these low-lying areas to determine the type (clays/silts) and consistency (strength and density) of soils, and to evaluate ground improvement options.

We recommend consideration be given to removal and replacement or soil mixing as a means of ground improvement, if soft clays or loose silts are encountered below topsoil in the low-lying areas. If loose granular soils are encountered in these areas, ground improvement techniques including roller compaction, dynamic compaction, dynamic replacement, stone columns, preloading by surcharge, rapid impact compaction and/or soil mixing could be considered. Each of the above-mentioned techniques has limitations based on soil type encountered (percent fines passing No. 200 sieve), the depth to groundwater within the zone of improvement, and the potential depth of improvement. Ground improvement techniques should be evaluated, once the type, consistency, and depth of marginal/weak soils in low-lying area have been explored and evaluated.

4.4 Site Preparation for Cut and Fill Areas

Design recommendations for site grade raise and site grade cut for the various areas are provided in this section. Additionally, recommendations for re-use of on-site soils, drainage considerations, and permanent cut and fill slopes are also included in this section.

4.4.1 Reuse of On-site Material

The topsoil depth across the site generally ranges between 0 and 6.5 ft. Loose silts and soft clays were encountered below topsoil at several locations (see Plate 4a), typically within the low-lying areas. The maximum depth of these loose silts and soft clays at explored boring locations was up to 9 ft below-grade (B-107). In general, the organic content of topsoil at the site is higher than 5% and the topsoil is frost susceptible, and therefore topsoil does not meet the requirements for general or structural fill. We also do not recommend using peat or any soils with high organics for backfilling any foundation areas; therefore, topsoil will need to be removed prior to major earthwork of cut and fill at the site. Topsoil should not be reused as fill, but can be utilized in landscaping. Soft clays and loose silts encountered below topsoil are expected to be frost susceptible. We recommend clays and silts, if encountered, below topsoil be removed and replaced with non-frost susceptible soils to a depth of at least 7 ft below final grade. If soft clays and loose silts are encountered at a depth

greater than 7 ft, ground improvement techniques as described in *Section 4.3* may be considered. Additionally, foundation bearing materials at this site should be non-frost susceptible to prevent frost-heave action under foundations.

The material obtained during the lowering of site grade, once topsoil and surficial silts/clays are removed, is expected to consist of granular soils, mainly sands and gravelly sands. The fines content of the granular soils within excavation depths generally ranges from about 3 to 18 percent. In localized areas, the fines content of granular soils within excavation depth is up to 32 percent (Plate 18a). Frost susceptibility tests performed on the samples collected from site indicated that the granular soils with up to 20 percent fines are non-frost susceptible, provided the fines consist of granitic rock flour[47,48]. Therefore, on-site granular soils with fines content of up to 20 percent may be used as non-frost susceptible fill soils. The remaining excavated material, provided the fines are less than 30 percent, may be used as structural fill soils (provided the fines consist of granitic rock flour). Additionally, the granular soils meet or are within range of DOT&PF C-1 specifications and hence can be used as base material for construction of roads. Material gradations of these specific soils with DOT&PF specification for upper bound (UB) and lower bound (LB) base course material (C-1)[49] are presented on Plate 18b. We recommend the materials to be separated and stockpiled for reuse as needed. Material not meeting non-frost susceptible fill, structural fill, or base material may be used as general fill. Plate 19 presents the results of minimum and maximum density tests conducted on all areas. Non-frost susceptible granular structural fill material should be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D4253 and D4254) or 95 percent of the maximum dry density (as determined by ASTM D1557) at a moisture content within 1 percent dry to 3 percent wet of the optimum moisture content.

4.4.2 Drainage Considerations

All surface drainage measures should be designed to positively direct water away from foundations. Runoff should also be collected and diverted away from structures. The finished surface level of the site should be graded with falls away from the structures and their foundations. Grading around the proposed structures should be such that future ponding or standing water around the foundations does not occur.

[47] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[48] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.
[49] Alaska Department of Transportation and Public Facilities (DOT&PF), 2015, Standard Specification for Highway Construction, Table 703-2, p.400, http://www.dot.state.ak.us

4.4.3 Cut and Fill Slopes

The proposed grading plans will require either cut or fill in a majority of the site areas. Therefore, there will be some permanent cut slopes outside the footprint of the operational area. Additionally, fill slopes are expected at containment berms. Cut and/or fill slopes are expected for various ponds planned at the site. Our recommendations for permanent cut and fill slopes are provided in this section.

Based on the information provided by AKLNG, we understand that the proposed final grade in the process areas will typically be about El. 116 or El. 125 (see Plate 16). Containment berms/dikes are expected to be up to 10 ft tall, and the site will have several ditches in process areas. Permanent slopes with heights of up to 5 ft constructed of on-site granular material or structural fill may be designed with slopes of 2-horizontal to 1-vertical or flatter. Slopes with heights greater than 5 ft and less than 10 ft, may be designed with 2.5-horizontal to 1-vertical or flatter. Geogrid reinforced slopes may be employed if steeper slopes are needed/preferred. The above recommendations do not apply to detention/retention ponds with liquid level fluctuations.

The exposed slopes should be protected from erosion by diverting surface runoff using shallow berms or channels. Where erosion due to wind or water is a concern, the slopes should be protected with shotcrete, asphalt, erosion control mats, concrete cloth, riprap, geocell or one of the other similar solutions. If vegetation is allowed on the slopes, a layer of woven vegetated geotextile may be considered. If the slope will require equipment access, consideration should be given to flattening the permanent granular fill slopes to 3-horizontal to 1-vertical. Slopes subject to any type of surcharge loading or greater than 10 ft in height should be analyzed on a case by case basis.

Our recommendations for the large planned cut in the bluff for the Heavy Haul Road that provides access to the MOF are discussed in *Section 12.0 Bluff Stability.*

4.4.4 Earthwork Considerations

Recommendations for grade raise and cut are discussed below:

- Establish site drainage to prevent surface water from entering areas to receive cut or fill. Our recommendations for drainage are provided in *Section 4.4.2 Drainage Considerations* of this report.
- Clear and grub all debris, vegetation, organic material, and other deleterious material within the areas to be cut or receive fill. Any loose material should also be removed before placing any fill material.
- The thickness of the topsoil encountered in the borings generally varies between 0 and 4 feet. Topsoil, surficial loose silts, and soft clays should be removed to a depth of at least 7 ft, preferably 10 ft, where construction of paving is anticipated. Loose silts/soft clays

encountered beneath any planned structures should be removed entirely. Ground improvement may be needed as mentioned in *Section 4.3 Ground Improvement*.

- Perform site grade cut and stockpile the material per recommendations presented in *Section 4.4.1 Reuse of On-site Material*.
- Proof-roll the areas where grade raise is required with a heavy (20 ton) vibratory roller to expose loose or weak zones. At least three passes should be made during proof-rolling. Consecutive passes should be made perpendicular to each other. Loose or weak zones indicated during proof-rolling should be over-excavated. Areas of subgrade in which pumping or significant deflections are observed should also be removed. We recommend that a geotechnical engineer or a qualified representative be present on-site to observe the exposed subgrade prior to placement of any new fill material.
- Site grade can be raised by using the stockpile or imported granular structural fill and placing in 8 to 10-inch-thick loose lifts within the footprints of the proposed foundations. Site grade outside the foundation footprint, or in areas where no foundations are planned can be raised using general fill. Please refer to *Section 17.3 Fill Material and Placement* of this report for recommendations regarding placement and compaction of various fill materials.

4.5 Site Grade Raise and Settlement

Grade raise of up to 30 ft is expected in certain localized areas of the site. The grade raise will result in settlements within the underlying foundation soils and within the newly placed fill. Subsurface soils are predominantly granular with an intermittent very stiff to hard clay layer at a depth of about 50 ft below existing grade in some areas. The majority (more than 90 percent) of the anticipated settlement is elastic in nature and will occur during or immediately after construction and is not of significance for design of the structures (i.e. immediate settlement). Settlement due to site grade raise and methods used to assess the settlements are presented in *Section 6.0 Settlement Due to Site Grade Raise*.

5.0 SELECTED DESIGN PROFILES AND INTERPRETED ENGINEERING PARAMETERS

5.1 General

The generalized subsurface cross-sections (Plates 9 through 14) developed from the integration of geophysical and geotechnical data illustrate the homogeneity of the subsurface conditions across the site, except in localized low-lying areas. Our discussion of selected soil design profiles including our design methodology, and our interpreted engineering parameters are provided in this section.

5.2 Engineering Areas

The site design grade elevation in the areas of structures, apart from the dikes/ponds and drainage ditches, is generally either El.+125 ft or El.+116 ft. The ground water elevation across the site varies and ranges from El.+40 ft at the bluff to about El.+100 ft on the eastern end of the site. Based on the proposed final finished grades in each area, groundwater conditions, and taking into account the layout of the proposed structures, we divided the project site into 8 areas (Plates 20, 21a through 21n) for developing our engineering recommendations. The various areas with associated structures, design grade and design groundwater elevations are summarized in Table 5.1.

Table 5.1: Engineering Areas with Design Elevations

Engineering Areas	Sub-Areas	Major Structure(s)	Design Grade Elevation, ft (NAVD88)	Groundwater Elevation, ft (NAVD88)	Plates
Area 1	-	Heavy Haul Road from MOF	27 to 125	60	Plates 21a-c
Area 2	2A 2B 2C	LNG Tank (North) LNG Tank (South) Trestle, Pipe Racks	125	70	Plates 21d-f
Area 3	-	LNG Admin Complex, Condensate and Compressors	116	70	Plates 21g-h
Area 4	4A 4B	LNG Train No.3 LNG Trains Nos.1&2	125	85	Plate 21i
Area 5	-	Gas Treating, Firewater Storage	116	85	Plate 21j
Area 6	-	Laydown Yard, Quarters, Miscellaneous Structures	122	90	Plate 21k
Area 7	-	Camp and Miscellaneous Structures	100 - 145	Varies	Plate 20
Area 8	-	Alternative Heavy Haul Road	Not Available	Varies	Plates 21l-n

5.3 Soil Design Parameters

The 2014, 2015 and 2016 field and laboratory testing programs were designed to evaluate pertinent engineering properties of the foundation soils. The laboratory test results performed on the soil samples collected from the onshore explorations performed as part of the 2014, 2015, and 2016 G&G programs are presented in AKLNG Document Nos. USAL-FG-GRZZZ-00-000003-000[50], USAL-FG-GRZZZ-00-002015-006[51] and USAL-FG-GRZZZ-00-002016-001[52]. We developed our strength and compressibility parameters for the soils based on the available field and laboratory test results. Soil parameters such as unit weights, friction angles, and elastic modulus are required to perform our engineering analysis and to develop recommendations for the proposed shallow and deep foundations. The first subsection provides our general approach in evaluating field and laboratory tests data to develop the soil design parameters. Subsequent subsections provide detailed discussions of field and laboratory tests including correlations of properties based on published literature. Selected soil design parameters are presented in *Section 5.4 Selected Soil Design Profiles and Parameters*.

5.3.1 Evaluation of Field and Laboratory Test Results

The soils at the project site are predominantly granular consisting of sands and gravelly sands. Therefore, a Standard Penetration Test (SPT) split-barrel (and when necessary 3-in outside-diameter oversize split-barrel) sampler was used to collect the majority of the samples in both Killey and Moosehorn units. Clay layers were encountered at deeper depths, but use of Shelby tubes for collecting samples was precluded at times due to the clay's very stiff to hard consistency. SPT N values were predominantly used to estimate in-situ densities and soil stiffness. We also performed laboratory tests such as maximum and minimum density, direct shear and direct simple shear on granular soil samples to evaluate friction angles and other engineering parameters.

For clayey samples collected from Shelby tubes, we developed soil compressibility (consolidation) parameters based on the laboratory consolidation test results. We also reviewed literature correlations for the compressibility of the clayey soils.

Please note that due to the difficulties encountered in recovering gravelly soils, the quantity of the samples collected was sometimes not sufficient to perform certain laboratory tests (i.e. minimum and maximum density, direct simple shear tests (DSS), corrosion series tests, particle degradation tests

[50] Fugro Consultants, Inc. (Fugro), 2015, Geotechnical Data Report Onshore LNG Facilities, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-8, AKLNG Document No. USAL-FG-GRZZZ-00-000003-000, Rev.0, dated May 08, 2015.
[51] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[52] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.

and frost heave tests). In these cases, we used composite samples to perform the laboratory tests as shown in Table 5.2.

Table 5.2: Laboratory Tests on Composite Soil Samples – 2014 G&G program

Boring ID	Samples Tested (IDs)	Depth Range (ft)	Laboratory Tests(1),(2),(3),(4)
B-14	S-7 & S-8	14.5-19.0	Min-Max Density, DSS
B-15	S-3 & S-4	4.0-8.5	Corrosion Series Tests
B-18	S-13 & S-14	26.5-30.0	Particle Degradation
B-23	S-10 & S-11	22.0-26.5	Min-Max Density, DSS
B-24	S-9 & S-10	20.0-24.5	Particle Degradation
B-26	S-19 & S-20	59.0-66.0	Corrosion Series Tests
B-26	S-27 & S-28	89.0-95.0	Min-Max Density, DSS
B-27	S-16, S-17 & S-18	49.5-61.5	Corrosion Series Tests
B-28	S-8 & S-9	18.0-22.0	Min-Max Density, DSS
B-32	S-20 & S-21	74.5-81.5	Corrosion Series Tests
B-34	S-7 & S-8	14.0-18.5	Min-Max Density, DSS
B-38	B-33, B-34 & B-35	98.5-119.8	Min-Max Density, DSS
B-40	S-3, S-4 & S-5	4.0-9.5	Min-Max Density
B-40	S-7 & S-8	12.0-15.0	Particle Degradation
B-45	S-14 & S-15	44.5-51.5	Corrosion Series Tests
B-47	S-18 & S-19	54.0-61.0	Corrosion Series Tests

Notes:

(1) Min-Max Density = Minimum and Minimum Density Tests (ASTM D 4253 & ASTM 4254)
(2) DSS = Direct Simple Shear Test (ASTM D 6528)
(3) Corrosion series tests include pH, soluble sulfate and soluble chloride concentration, and electrical resistivity tests (ASTM G51, ASTM D516, ASTM D512, ASTM G57-78)
(4) Particle degradation test included the compaction of the soil samples in Harvard Miniature Compaction Mold. Samples were sieved prior to and after compaction to assess the potential for particle degradation.

Table 5.3: Laboratory Tests on Composite Soil Samples – 2015 G&G program

Boring/Test Pit ID	Samples Tested (IDs)	Depth Range (ft)	Laboratory Tests(1),(2),(3)
B-69	S-2 & S-3	2.0-6.0	Min-Max Density, DSS
B-80	S-4, S-5 & S-6	6.5-13.5	Min-Max Density, DSS
B-85	S-7 & S-10	14.0-26.0	DSS
B-88	S-8 & S-9	16.8-21.3	Min-Max Density, DSS
B-91	S-5, S-6 & S-7	9.0-16.0	Min-Max Density
B-104	S-6, S-7 & S-8	11.5-18.5	Min-Max Density, DSS
B-132	S-9, S-10 & S-11	19.0-31.0	Min-Max Density, DSS
TP-01	S-3	2.0-3.0	FH (06)
TP-06	S-3	2.0-3.0	FH (07)

Notes:

(1) Min-Max Density = Minimum and Minimum Density Tests (ASTM D 4253 & ASTM 4254)
(2) DSS = Direct Simple Shear Test (ASTM D 6528)
(3) FH = Frost Heave Tests (derived from ASTM D 5918). FH (06) and FH (07) were conducted with original soil sample mixed with additional fines in order to achieve target fines contents of 20% and 30%, respectively.

Table 5.4: Laboratory Tests on Composite Soil Samples – 2016 G&G program

Boring ID	Samples Tested (IDs)	Depth Range (ft)	Laboratory Tests(1),(2)
B-147	S-2, S-3 & S-4	2.5-9.5	FH (01)
B-148	S-2, S-3 & S-4b	2.5-8.7	FH (01)
B-149	S-4	6-8.5	FH (01)
B-152	S-2b, S-3 & S-4	2.8-8.5	FH (02)
B-156	S-2b, S-3a, S-3c & S-4	2.5-8.5	FH (03)
B-157	S-2b, S-3, S-4 & S-5	2.5-11.0	FH (03)
B-169	S-3 & S-4	4.0-11.0	FH (04)
B-170	S-2, S-3 & S-4	2.0-8.5	FH (05)
B-171	S-3	4.0-6.0	FH (05)
B-172	S-2, S-3 & S-4	2.0-8.5	FH (06)
B-197	S-3 & S-4	4.0-8.5	FH (07)

Notes:
(1) FH = Frost Heave Tests (derived from ASTM D 5918)
(2) Number in parenthesis indicates FH test number.

5.3.2 SPT N Values

The SPT N_{field} values provide information for granular soils that can be correlated with the in-situ density and friction angle. It also provides useful information on consistency and stiffness for clayey soils. SPT N_{field} vs Elevation, and SPT N_{60} (SPT N_{field} value corrected for field procedures and apparatus), and SPT $(N_1)_{60}$ (SPT N_{60} value corrected for overburden) vs Elevation plots for each Area are presented in Plates 22 through 36 of the report. For more detailed discussion on SPT N value energy corrections, please refer to the AKLNG Document Nos. USAL-FG-GRZZZ-00-000003-000[53], USAL-FG-GRZZZ-00-002015-006[54] and USAL-FG-GRZZZ-00-002016-001[55]. In some cases, over-size particles (i.e., gravel-size particles in the soil matrix) can get stuck in the regular SPT spoon resulting in an increase in the blow count numbers. Due to the lack of reliable correlations in the literature, SPT N values were not explicitly corrected for gravel size particles. The SPT N_{field} values within the encountered natural sands and gravelly sands/silty sands indicate generally medium-dense to very dense soils.

[53] Fugro Consultants, Inc. (Fugro), 2015, Geotechnical Data Report Onshore LNG Facilities, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-8, AKLNG Document No. USAL-FG-GRZZZ-00-000003-000, Rev.0, dated May 08, 2015.
[54] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.
[55] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-5, AKLNG Document No. USAL-FG-GRZZZ-00-002016-001, Rev.0, dated November 11, 2016.

5.3.3 Unit Weight

In order to calculate in-situ effective vertical stress, effective unit weights are required. Total unit weights are routinely measured on cohesive soil samples during most laboratory tests. Undisturbed granular soil samples cannot be recovered by SPT sampling techniques, therefore, direct measurement of unit weight was not possible. We performed maximum and minimum dry density tests on oven dried granular soil samples. The SPT N_{field}, SPT N_{60}, and SPT $(N_1)_{60}$ values were reviewed and a representative design SPT N value was selected for each layer. Correlations (lower bound) after Bowles (1977) as adopted in Caltrans geotechnical manual[56] by California Department of Transportation were used in selecting design unit weights based on selected SPT $(N_1)_{60}$ value for each layer.

5.3.4 Mineralogy and Morphology

Mineralogical and morphological characteristics of soil particles affect their strength characteristics and degradation potential. In order to determine the mineralogical composition of sands within Killey unit, X-Ray Diffraction (XRD) analysis was performed on selected soil samples. Results of the XRD analysis indicate that the Killey sands are composed of quartz, plagioclase, and clay minerals with approximate ranges of 42 to 62 percent, 23 to 37 percent, and 9 to 11 percent, respectively. Plagioclase is a mineral of feldspar family. The presence of 23 to 37 percent plagioclase in the samples indicate the Killey unit sands are not typical quartz sands. Mohs scale hardness of plagioclase (6 to 6.5) is slightly lower than quartz (7). The mineral surface roughness is also less than quartz which may result in lower friction angles than typical quartz sand. However, as morphological factors prevail, this effect is insignificant.

5.3.5 Particle Degradation

Standard compaction tests were performed to estimate the particle degradation potential of sand and gravel size particles of Killey unit. Samples of sand were compacted in Harvard Miniature Compaction Mold. Sieve analyses were performed prior to and after the compaction. Similarly, a representative sample which included gravel and sand size particles of Killey Formation was compacted in a Standard Proctor Mold. The standard test method specifies compaction in three layers. However, due to limited quantity of sample, the test was performed on a single layer (1/3rd of Proctor mold). Sieve analyses tests performed before and after compaction tests revealed very similar gradation curves indicating insignificant breakdown of sand-size particles. Based on these test results, we believe that the engineering properties of the sands and gravels will not be significantly affected by anticipated construction activities, i.e. mass earthworks and foundation installation.

[56] California Department of Transportation (Caltrans), 2014, Caltrans Geotechnical Manual, Chapter *Soil Correlations (March 2014)*, Page 3 and 5, http://www.dot.ca.gov/hq/esc/geotech/geo_manual/manual.html

5.3.6 Friction Angle

Direct shear and direct simple shear tests were performed on sandy soils compacted at their estimated in-situ densities to obtain strength parameters of granular soils. Table 5.5 summarizes the average SPT N_{field}, SPT N_{60} and SPT $(N_1)_{60}$ values for elevations at which the direct shear and direct simple shear tests were performed. Also, presented in Table 5.5 is the friction angle obtained using empirical correlation after Bowles (1977) as adopted in Caltrans geotechnical manual[57] by California Department of Transportation. Since the direct shear test method shears the sand on a predefined horizontal plane, the results of direct simple shear tests were given more weight. In general, the direct simple shear test results were similar to the correlated friction angles (lower bound) per Bowles (1977). Hence, we used empirical correlation per Bowles (1977) for selection of design values of friction angles. The SPT profiles were developed for each of the areas at the project site. The average SPT $(N_1)_{60}$ values were selected for each layer and correlations per Bowles (1977) was used to develop design friction angles for analyses.

Table 5.5: Summary of Direct Shear Tests and Correlated Friction Angles

Boring No.	Sample Elevation (ft)(1)	SPT(2) N_{ave}	SPT(2) $N_{60,ave}$	SPT(2) $(N_1)_{60,ave}$	Direct Shear	Direct Simple Shear	Bowles (1977)(3)
					ϕ	ϕ	ϕ
B-11	74.4	49	64	23	39.1	-	33
B-14	116.9	47	55	32	-	34.9	35
B-18	107.3	30	39	33	42.3	-	36
B-23	100.6	24	29	24	-	33.1	34
B-24	105.9	40	52	29	47.6	-	35
B-26	94.5	22	29	22	37.1	-	33
B-26	34.5	99	99	68	-	32.5	40
B-28	98.9	17	21	21	-	30.9	33
B-32	66.5	58	68	28	-	33.7	35
B-34	111.8	49	61	37	-	33.3	36
B-38	108.0	17	23	24	-	31.8	34
B-39	-5.6	99	99	70	41.7	-	40
B-42	72.4	99	99	71	-	31.3	40
B-43	66.1	57	76	35	-	30.6	36
B-45	102.6	45	59	53	-	34.3	38
B-69	117.9	26	39	65	-	48.6	39
B-77	85.0	38	57	42	42.3	-	37
B-77	-4.5	99	99	64	33.7	38.7	39
B-80	117.4	25	37	42	-	39.6	37
B-85	103.5	33	49	51	-	36.9	38
B-85	63.8	99	99	65	-	38.7	39
B-85	39.3	99	99	70	38.5	-	40

[57] California Department of Transportation (Caltrans), 2014, Caltrans Geotechnical Manual, Chapter *Soil Correlations (March 2014)*, Page 1 and 2, http://www.dot.ca.gov/hq/esc/geotech/geo_manual/manual.html

Boring No.	Sample Elevation (ft)(1)	SPT(2) N_{ave}	SPT(2) $N_{60,ave}$	SPT(2) $(N_1)_{60,ave}$	Direct Shear	Direct Simple Shear	Bowles (1977)(3)
					ϕ	ϕ	ϕ
B-85	-21.5	99	99	60	36.8	40.5	39
B-88	75.2	52	77	58	-	38.7	39
B-97	59.6	99	99	84	38.0	-	40
B-104	76.7	64	95	70	-	44.4	40
B-121	-9.6	71	99	99	42.1	38.7	40
B-122	5.2	32	48	58	-	42.8	39
B-122	-19.5	33	43	43	-	36.9	37
B-122	-54.5	29	43	28	-	40.0	35
B-123	-55.3	99	99	94	37.8	-	40
B-132	96.7	82	99	75	-	40.0	40
B-176	20.1	39	65	99	35.2	-	40
B-176	7.6	9	15	14	-	38.7	31
B-189	105.0	35	58	62	34.2	-	39
B-189	-11.0	83	99	59	37.5	-	39
B-189	-51.0	61	99	40	-	36.9	37
B-190	9.6	28	45	76	-	41.8	40
B-191	-0.5	39	63	99	32.1	-	40
B-191	-15.5	41	66	74	-	36.9	40
B-192	77.2	99	99	77	40.5	-	40
B-192	67.2	65	99	42	-	38.7	37
B-192	37.2	73	99	56	37.6	-	39
B-192	-17.8	71	99	47	-	38.7	38
B-192	-37.8	94	99	60	38.1	-	39
B-195	38.5	99	99	83	-	36.9	40

Notes:
1. Elevations are referenced to NAVD88.
2. SPT N_{ave}= average SPT (N) blow count value, SPT $N_{60,ave}$= average energy corrected blow count (60 percent of the theoretical energy), SPT $(N_1)_{60,ave}$ = both energy (60 percent) and effective overburden energy corrected average blow count
3. Lower bound of SPT correlation for friction angle by Bowles (1977) is limited to 40 deg.
4. Clay properties encountered in this layer are discussed separately under *Section 5.3.7 & 5.3.8*.

5.3.7 Undrained Shear Strength

The undrained shear strengths for lean clay layers were estimated by SPT N_{field} values, hand-held pocket penetrometer, and unconsolidated-undrained (UU) triaxial compression tests, then conservatively assessed between lower one-third and average values. The clays at this site are overconsolidated, very stiff to hard with an undrained shear strength generally above 4 ksf.

5.3.8 Compressibility

The project site is predominantly underlain by granular soils (sands/silty sands/gravelly sands). The typical strain rate of soils for static loading is about 1 percent or more. Young's Modulus (E_s) values at a strain rate of about 1 percent are required to estimate the compressibility of these granular soils. We relied on the downhole compression and shear wave data, which provides small strain Young's

Modulus, to estimate elastic modulus of soils for settlement analyses. A reduction of 90 percent was applied to the Young's Modulus obtained from the downhole test results to estimate the Elastic Modulus of soils for static settlement analyses per literature (Massarsch, 2004)[58] and industry standards. The downhole compression and shear wave test data is not reliable at shallower depths, 2 times the distance between the boring and impact source, which is about 20 ft. Hence, in the upper 20 ft, we used a combination of the Young's Modulus obtained from the downhole test results and the AASHTO method (2004)[59] to estimate the elastic modulus in granular soils.

Very stiff to hard clayey soils with low compressibility potential were encountered at depths deeper than 50 ft below ground surface. Typically, the clay layer thicknesses vary between 5 ft and 10 ft, with maximum thickness of about 25 ft. Atterberg limit tests (liquid limits (LL), liquidity index (LI), and plasticity index (PI)) were performed on selected cohesive soil samples to aid in soil classification. One dimensional consolidation tests were performed on the clayey soils to determine the consolidation parameters. Design values were selected from lab test results. In general, overconsolidated clays at depths greater than 50 ft are expected to contribute little to settlement and have minimal influence on shallow foundation behavior.

5.3.9 Frost Susceptibility

In addition to freezing weather conditions, the potential for the soils to undergo seasonal frost heave is determined by two factors, 1) the presence of water and 2) the frost susceptibility of the soils. Groundwater at the site was only encountered at depths lower than El. +100 ft. Based on the U.S. Army Corps of Engineers (USACE) Frost Design Classification[60], the soils that have more than 3 percent passing 0.02 mm opening sieve are potentially frost susceptible. Based on hydrometer test results, the granular soils at the project site generally contain 3 to 18 percent passing 0.02 mm opening sieve. Thus, the granular soils have a frost classification of non-frost susceptible to moderate frost susceptibility per USACE frost design classifications.

In order to evaluate the frost susceptibility classifications using guidelines set by USACE, further advanced frost susceptibility tests were performed during 2015 and 2016 G&G programs on 15 select samples collected from the site as mentioned in *Section 2.13 Cold Region Considerations*. Three of the samples were reconstituted such that the fines (percent passing No.200 sieve) were about 20, 21 and 32 percent. The maximum heave observed in the test samples was about 2.4 percent for the sample with fines content of 32 percent at vertical stress of about 0.5 psi. Based on the test results and encountered soils, the soils at the site can be classified as low to non-frost susceptible.

[58] Massarsch, K.R. (2004), "Deformation Properties of Fine-Grained Soils from Seismic Tests", Keynote lecture, *International Conference on Site Characterization*, ISC'2, 19-22 Sept. 2004, Porto, 14 p.
[59] AASHTO (2004 with 2006 Interims), "AASHTO LFRD Bridge Design Specifications", 3rd Ed., Washington, D.C.
[60] U.S. Army Corps of Engineers (USACE), 1984, "Engineering and Design – Pavement Criteria for Seasonal Frost Conditions – Mobilization Consruction", Engineer Manual EM 1110-3-138, April 9, 1984, Page 2-8.

5.4 Selected Soil Design Profiles and Parameters

Soil borings located within the delineated Areas 1 through 6, and Area 8 (B-176 and B-183) are plotted to develop design cross sections and foundation design recommendations. Based on evaluation of the laboratory test results and field tests presented in the earlier sections, and our engineering judgment, design parameters were selected for the eight areas and are presented on Plates 21a through 21n. SPT plots and laboratory test data with the selected parameters for Areas 1 through 6 and 8 are presented on Plates 22 through 36. Detailed discussion of selection of design parameters is presented in previous sections.

Table 5.6: Selected Soil Design Parameters – Area 1 (Heavy Haul Road Area, HA1-HA1')

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)
125 to 105	0 to 15	Silty Sand with Gravel	118	-	34	25
105 to 90	15 to 30	Silty Sand/Sand with Silt and Gravel	120	-	35	30
90 to 60	30 to 60	Silty Sand/Sand with Silt and Gravel	120	-	35	30
60 to 40	60 to 80	Silty Sand/Sand	60.6	-	36	35
40 to 10	80 to 110	Silty Sand/ Sandy Silt/Clay	62.6	4.0 (0.3 (4))	23 (3)	-
10 to -80 (3)	110 to 200	Silty Sand/Sandy Silt	62.6	-	36	40
-80 to -160	200 to 280	Silty Sand/ Sandy Silt/Clay	62.6	5.0 (0.3 (4))	23 (3)	-
-160 and below	280 and below	Silty Sand/ Sand with Silt and Gravel	72.6	-	38	50

Notes:
1. Groundwater table was estimated at 65 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Three different profiles were used for slope stability analyses based on the section analyzed. The variations in soil profiles were below El.-25.
4. Drained parameters for clay soils were used for long term slope stability analysis.

Table 5.7: Selected Soil Design Parameters – Area 2A (Storage Tank Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4 (0.3(4))	23(4)	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to -20	85 to 145	Sand	62.6	-	36	40	1350

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
-20 to -180	145 to 305	Clay	62.6	6 to 12 (0.3(4))	23(4)	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 55 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data
4. Drained properties of Clay layers were used based on literature and laboratory test results.

Table 5.8: Selected Soil Design Parameters – Area 2B (Storage Tank Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4 (0.3(4))	23(4)	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to 0	85 to 125	Sand	62.6	-	36	40	1350
0 to -15	125 to 140	Clay	62.6	5 (0.3(4))	23(4)	-	Cc= 0.10, Cr=0.009, OCR=3
-15 to -50	140 to 175	Sand	62.6	-	36	40	1350
-50 to -180	175 to 305	Clay	62.6	7 to 12 (0.3(4))	23(4)	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 55 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data
4. Drained properties of Clay layers were used based on literature and laboratory test results.
5.

Table 5.9: Selected Soil Design Parameters – Area 2C (Storage Tank Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 50	55 to 75	Sand/Gravel	60.6	-	36	35	1200
50 to 40	75 to 85	Clay	60.6	4 (0.3(4))	23(4)	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to 20	85 to 105	Sand	62.6	-	36	40	1350

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
20 to 0	105 to 125	Clay	62.6	5 (0.3(4))	23(4)	-	Cc= 0.10, Cr=0.009, OCR=3
0 to -40	125 to 165	Sand	62.6	-	36	40	1350
-40 to -180	165 to 305	Clay	62.6	7 to 12 (0.3(4))	23(4)	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 55 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data
4. Drained properties of Clay layers were used based on literature and laboratory test results.

Table 5.10: Selected Soil Design Parameters – Area 3 (Condensate and Admin Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
116 to100	0 to 16	Sand/Gravel	118	-	34	25	600
100 to 80	16 to 31	Sand/Gravel	120	-	35	30	700
80 to 70	31 to 41	Clay	123	4	-	-	Cc= 0.15, Cr=0.01, OCR=5
70 to 50	41 to 61	Sand/Gravel	62.6	-	36	40	1150
50 to 40	61 to 76	Clay	62.6	4	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to -15	76 to 131	Sand	65.6	-	37	45	1300
-15 to -80	131 to 196	Clay	65.6	5 to 8	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 46 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data.

Table 5.11: Selected Soil Design Parameters – Area 4A (Process Train Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
125 to105	0 to 20	Sand/Gravel	118	-	34	25	600
105 to 90	20 to 35	Sand/Gravel	120	-	35	30	800
90 to 85	35 to 40	Sand/Gravel	125	-	36	40	1050
85 to 55	40 to 70	Sand/Gravel	62.6	-	36	40	1050
55 to 45	70 to 80	Clay	62.6	5	-	-	Cc= 0.13, Cr=0.01, OCR=4.5

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
45 to -5	80 to 130	Sand	65.6	-	37	45	1300
-5 to -75	130 to 200	Sand	65.6	-	37	45	1500

Notes:
1. Groundwater table was estimated at 40 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data.

Table 5.12: Selected Soil Design Parameters – Area 4B (Process Train Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
125 to105	0 to 20	Sand/Gravel	118	-	34	25	600
105 to 90	20 to 35	Sand/Gravel	120	-	35	30	800
90 to 85	35 to 40	Sand/Gravel	125	-	36	40	1050
85 to 55	40 to 70	Sand/Gravel	62.6	-	36	40	1050
55 to 45	70 to 80	Clay	62.6	5	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
45 to 20	80 to 105	Sand	65.6	-	37	45	1300
20 to -5	105 to 130	Clay	65.6	5	-	-	Cc= 0.10, Cr=0.009, OCR=3
-5 to -70	130 to 195	Sand	65.6	-	37	45	1500

Notes:
1. Groundwater table was estimated at 40 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data.

Table 5.13: Selected Soil Design Parameters – Area 5 (Utility Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
116 to105	0 to 11	Sand/Gravel	118	-	34	25	600
105 to 90	11 to 26	Sand/Gravel	120	-	35	30	800
90 to 85	26 to 31	Sand/Gravel	123	-	36	35	900
85 to 70	31 to 46	Sand/Gravel	60.6	-	36	35	900
70 to 60	46 to 56	Clay	62.6	4	-	-	Cc= 0.15, Cr=0.01, OCR=5

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
60 to 40	56 to 76	Sand	62.6	-	36	40	1000
40 to 25	76 to 91	Clay	65.6	5	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
25 to -5	91 to 121	Sand	65.6	-	37	45	1100
-5 to -50	121 to 166	Clay	65.6	5 to 6.5	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 31 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data.

Table 5.14: Selected Soil Design Parameters – Area 6 (Laydown Area)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							E_s, ksf
122 to 90	0 to 32	Structural Sand Fill/Gravel with Sand/Sand with Silt and Gravel	120	-	35	30	800
90 to 65	32 to 57	Silty Sand/Sand with Silt and Gravel	60.6	-	36	35	1050
65 to 40	57 to 82	Silty Sand/Clayey Sand/Clay	65.6	(4.5)	37	45	1150
							Cc= 0.15, Cr=0.01, OCR=3

Notes:
1. Groundwater table was estimated at 32 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data.

Table 5.15: Selected Soil Design Parameters – Area 8 (ASRC Area, B-176)

Elevation, ft	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)
27 to 7	0 to 20	Existing Fill: Sand with Silt and Gravel	118	-	30	10
7 to -7	20 to 34	Existing Fill or Natural Very Loose to Loose Sand with Silt and Gravel	55.6	(0.2)(3)	27	5

Elevation, ft	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)
-7 to -73	34 to 100	Dense to Very Dense Sand with Silt and Gravel	62.6	-	36	40
-73 to -103	100 to 130	Stiff to very Stiff Clay and Gravelly Clay	62.6	4.0	-	-
-103 to -147	130 to 174	Dense to Very Dense Silty and Gravelly Sand	62.6	-	36	40

Notes:
1. Groundwater table was estimated at 20 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. The residual shear strength of the liquefiable layer is estimated to be 0.2 ksf based on Idriss and Boulanger (2008).

Table 5.16: Selected Soil Design Parameters – Area 8 (ASRC Area, B-183)

Elevation, ft	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)
99 to 85	0 to 14	Medium Dense to Dense Silty and Gravelly Sand	118	-	34	25
85 to 60	14 to 39	Dense Silty and Gravelly Sand and Sand	125	-	36	40
60 to 15	39 to 84	Dense Silty and Gravelly Sand and Sand	62.6	-	36	40
15 to -1	84 to 100	Dense to Very Dense Silty Sand	65.6	-	37	45

Notes:
1. Groundwater table was estimated at 39 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.

6.0 SETTLEMENT DUE TO SITE GRADE RAISE

6.1 General

The project site will receive fill generally varying between 0 ft to 20 ft (and up to 30 ft in low-lying areas) above existing grade to match the assumed rough grade elevation in the respective engineering areas (Plate 16). The grade raise will result in settlements within the underlying foundation soils and within the newly placed fill. Subsurface soils are predominantly granular with an intermittent very stiff to hard clay layer at a depth of about 50 ft below existing grade in some areas. The majority of the settlements will occur during construction and will vary across the grade raise area.

This subsection presents the methods used in our settlement analysis and our computed settlements. Please note that we have not calculated settlements for low-lying areas where elevations are as low as El. +80 ft (See *Section 4.2 Low-Lying Areas*) and significant amounts of clay and/or silt is expected based on ERT survey. Type, consistency (shear strength and density) and compressibility information of these clays/silts is not available at this time. Sampling in these areas should be performed to collect samples for consistency and compressibility testing. Settlement analyses can be performed once this information is obtained, and possible ground improvement methods, if needed, can be evaluated.

6.2 Site Grade Raise Material

Site grade raise will be performed using stockpiled on-site cut material or imported material meeting structural fill and/or non-frost susceptible fill recommendations. The material will be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D4253 and D4254), or 95 percent of the maximum dry density (as determined by ASTM D1557) at a moisture content within 1 percent dry to 3 percent wet of the optimum moisture content. For fill material, we assumed average geotechnical properties based on the properties of in-situ soils at similar elevations in order to simplify development of recommendations for each area.

6.3 Method of Analysis

We have divided the project site into “Engineering Areas” based upon anticipated use, and final grade elevations. The soil profiles vary from one area to another area. The majority (more than 90 percent) of the anticipated settlement within each area will be elastic in nature, and is expected to occur immediately after construction and is not of significance for design of the structures (i.e. immediate settlement). Hence, we used one profile (Area 2B) to compute settlement to provide an order of magnitude assessment of settlements due to site grade raise. Thus, the results of our analyses for Area 2B are also applicable for Areas 2 to 7. Soil parameters used in the settlement analyses are presented in Table 6.1.

Table 6.1: Soil Profile for Grade Raise Settlement Analysis – Area 2B

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to 0	85 to 125	Sand	62.6	-	36	40	1350
0 to -15	125 to 140	Clay	62.6	5	-	-	Cc= 0.10, Cr=0.009, OCR=3
-15 to -50	140 to 175	Sand	62.6	-	36	40	1350
-50 to -180	175 to 305	Clay	62.6	7 to 12	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 55 ft (El. 70 ft) below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data

We assumed that the fill areas will be constructed using granular non-frost susceptible fill, structural fill or general granular fill where settlement within the fill material will also occur relatively quickly. The magnitude of settlement within fill will be dependent on the type, thickness, and compaction effort of the fill materials used to raise grade. Settlement within properly compacted granular fill soils due to self-weight is expected to be about 0.5 to 1 percent of fill height and will mostly be completed during construction.

We calculated settlements due to 5 ft, 10 ft, 15 ft, and 20 ft grade raise. The unit weight of the fill material that will be used to raise the site grade was assumed to be about 120 pcf (corresponding to approximately 80 percent relative density for recompacted on-site Killey formation soils); therefore, four surcharge pressures of 0.6 ksf, 1.2 ksf, 1.8 ksf, and 2.4 ksf were applied on top of the existing overburden grade for 5 ft, 10 ft, 15 ft, and 20 ft of grade raise, respectively. For our analysis, a plan area of about 1000 ft x 1000 ft was considered for each of the proposed grade raise areas. The existing groundwater table is assumed at about 55 ft below the existing ground surface (i.e. El. 70 ft, NAVD88) for grade raise settlement analysis. We performed settlement analyses using Settle3D 2009 (2010), developed by Rocscience Inc. This program uses a Boussinesq theory for stress distribution in soils and consolidation theory to estimate consolidation settlements in clayey soils and elastic compression (i.e. immediate settlement) in granular soils.

Results of the settlement analysis for the cases of 5 ft, 10 ft, 15 ft, and 20 ft of fill heights are presented in Table 6.2.

Table 6.2: Results of Settlement Analysis for Site Grade Raise – Area 2B

Grade Rise, ft	Total Settlements(1), in.	Immediate Settlement(1), In.	Additional Settlement after 6 months, in.
5 ft raise (0.6 ksf)	2 to 2.5	1.5 to 2	< 0.5
10 ft raise (1.2 ksf)	4 to 5	3.5 to 4	< 0.5
15 ft raise (1.8 ksf)	6.5 to 7.5	6 to 7	< 0.5
20 ft raise (2.4 ksf)	9 to 10	8.5 to 9.5	< 0.5

Notes:
1. These are the estimated values. Actual values could be outside the range of values given in the table above.
2. The settlement estimates provided under this table do not include settlements within the fill soils. As mentioned earlier, settlement within properly compacted granular fill soils due to self-weight is expected to be about 0.5 to 1 percent of fill height and will mostly be completed during construction.

7.0 FOUNDATION TYPES AND FEASIBILITY

7.1 General

A foundation system should satisfy two basic and independent design criteria. First, the bearing pressure transmitted to the foundation soils should not exceed the allowable net soil bearing capacity, which includes an appropriate safety factor. The allowable net bearing pressure for shallow foundations is a function of, among other items, the bearing surface, the strength of the foundation soils, the location of the foundation, the shape of the foundation, and the recommended factor of safety. Second, settlements during the operating life should not be of a magnitude that will cause damage to the structures or their utility connections. Settlements are a function of, among other things, the underlying soil conditions, the foundation size and shape, and the applied pressures of the foundation elements.

Loading conditions, including the anticipated foundations for the proposed structures in each of the areas, were provided to us under Document No. RFI USAI-PS-BPDCC-00-009 dated December 23, 2014 and Drawing Nos. USAL-PL-CARFI-00-000002-000 Attachment A through E, dated December 27, 2014. Based on this information, we understand the typical structural loads to be as follows:

- LNG Train Area – 600 psf to 3,000 psf
- LNG Storage Tank Area – 4,070 psf to 5,760 psf
- Ground Flare Area – 400 psf to 1,800 psf
- Buildings/Miscellaneous structures - 100 psf to 600 psf

Based on the subsurface conditions encountered during our field investigation, all of the proposed equipment/structures can likely be placed on shallow foundations consisting of shallow spread footings and mat foundations, as long as proper site preparation (see *Section 4.0*) has been performed and seismic considerations are met. Based on the granular soil conditions encountered, the proposed shallow foundation design will be governed by settlement rather than bearing capacity. Based on geotechnical considerations, it is our opinion that the proposed equipment/structures will not require deep foundations. However, if the settlements calculated in this report are not within the tolerable limits for a specific structure, or if lateral loads, and/or seismic requirements govern foundation design, then deep foundations should be considered.

Low-lying areas were explored by hand augers to limited depth and ERT survey during 2015 G&G Program. The surface elevation in these low-lying areas can be as low as El.+80 ft per site topography map (Plate 7). The available information identified areas with clay/silt to depths of up to 30 ft. Borings were not drilled in these areas to collect soils samples for testing. Hence, the

consistency and compressibility of these soils are not known at this time. These soils, if loose/soft, should be either removed and replaced or improved prior to the placement of foundations.

Table 7.1 provides summary of various areas and each section that will provide foundation recommendations in those areas.

Table 7.1: Geotechnical Recommendation Sections

Engineering Areas	Major Structure(s)	Sections
Area 1	Heavy Haul Road from MOF	Section 9.0 Shallow Foundation Recommendations Section 15.0 Pavement Recommendations
Area 2	LNG Tank Area	Section 8.0 LNG Tank Shallow Foundation Recommendations Section 9.0 Shallow Foundation Recommendations Section 10.0 Slab-on-Grade Recommendations Section 11.0 Deep Foundation Recommendations
Area 3	LNG Admin Complex, Condensate and Compressors	Section 9.0 Shallow Foundation Recommendations Section 10.0 Slab-on-Grade Recommendations Section 11.0 Deep Foundation Recommendations
Area 4	LNG Trains	Section 9.0 Shallow Foundation Recommendations Section 10.0 Slab-on-Grade Recommendations Section 11.0 Deep Foundation Recommendations
Area 5	Gas Treating, Firewater Storage	Section 9.0 Shallow Foundation Recommendations Section 10.0 Slab-on-Grade Recommendations Section 11.0 Deep Foundation Recommendations
Area 6	Laydown Yard	Section 10.0 Slab-on-Grade Recommendations
Area 7	Camp and Miscellaneous	Section 10.0 Slab-on-Grade Recommendations
Area 8	Alternative Heavy Haul Road	Section 11.0 Deep Foundation Recommendations Section 15.0 Pavement Recommendations

This section further discusses our recommendations related to various foundation types that may be used to support the proposed structures. Seismic design and liquefaction considerations, as well as foundation protection and frost considerations are also discussed.

7.2 Foundation Types

Based on the subsurface conditions encountered, we expect that the proposed equipment/structures can be supported on shallow foundations consisting of spread footings or mat foundations. Deep foundations can also be utilized. The following foundation conditions are expected:

1. Shallow Foundations bearing on in-situ soils,
2. Shallow Foundations bearing on non-frost susceptible structural fill soils,

3. Deep foundations consisting of steel pipe piles (and drilled shafts proposed for the Area 8) installed through fill or existing soils and tipped within Moosehorn Formations (sands/sandy gravels).

Based on review of subsurface conditions and local practices, typical shallow foundations that may be used for the project site will include spread, strip, and mat foundations. We have performed our analysis for footing widths varying between 3 ft and 20 ft. In our analyses, footings having widths less than about 15 ft were considered as spread footings and footings with widths greater than 15 ft were considered as mat foundations. The proposed heavily loaded LNG tanks may also be supported on shallow rigid mat foundations with thickened edges after following the site preparation recommendations provided in *Section 4.0 Site Preparation*. It should be noted that for cases where multiple foundations will be constructed in close proximity, a bathtub excavation may be considered for efficient construction. Bathtub excavation techniques should be reviewed during the final design stage of the project.

As an alternative to shallow foundations, the proposed equipment/structures may be supported on deep foundations. The selection of deep foundation type depends on various factors, such as; subsurface conditions that will govern installation and constructability, climate, local availability, schedule, and cost. Most of the site is currently underlain by medium dense to dense sands and gravelly sands in at least the top 30 ft below the final grade. The gravel content of these soils generally varies between 30 to 50 percent. Higher SPT blowcounts were observed within these layers, and we generally started encountering SPT refusal (N_{field} values greater than 50 blows/ft) below 30 ft, indicating the presence of dense to very dense sands and gravelly sands. Cast-in-place concrete piles are not ideal in these granular soils, especially due to the low fines content and difficulty in keeping the auger holes open. Casing may be used during installation to prevent caving of material. However, use of casing will increase the installation costs substantially. Additionally, if installed during winter months, the cast-in-place concrete piles will introduce difficulties during pouring and curing of concrete, and therefore are not a commonly used deep foundation type in the Nikiski area. Driven piles would be better suited for this site from a design and schedule point of view. However, precast concrete driven piles are not easy to drive through the very dense to dense gravelly and sandy soils encountered at this site, and the piles may be damaged due to higher driving stresses and may reach early refusal. Therefore, we recommend the use of either steel H-piles or open–ended steel pipe piles to support the proposed equipment/structures on deep foundations. Even though H-piles are an acceptable option, we understand that they are not commonly used in the Nikiski area. H-piles may also be more expensive than pipe piles. Therefore, our report primarily presents recommendations for open-ended steel pipe piles. We have developed recommendations for 16-, 18-, and 24-inch open ended pipe piles. Additionally, AKLNG requested Fugro to provide cast-in-place pile recommendations that may be used to support the bridge along the proposed alternative heavy haul road. Therefore, recommendations for 24-, and 36-inch cast-in-place concrete piles are included only for Area 8.

Drivability and refusal depth will be very important factors for driven pipe piles. Therefore, we performed a feasibility-level drivability study using upper bound design parameters assuming different types of driving equipment (i.e., hammers). Our drivability study results are discussed in detail in *Section 17.5.1 Pile Drivability*. It should be noted that smaller open-ended steel pipe piles are not recommended due to the possibility of early refusal and difficulties in driving that may result in pile damage.

7.3 Seismic Design Considerations

Near-surface design spectra for Operational Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) levels were developed according to NFPA 59A (2006) requirements in addition to the Maximum Considered Earthquake (MCE) and the Design Earthquake (DE) that were developed in accordance with ASCE 7-05/IBC (2009). The estimated near-surface horizontal Peak Ground Acceleration (PGA) values at different shaking levels are provided in *Section 2.11 Probabilistic Seismic Hazard Assessment (PSHA) & Site Response*.

Liquefaction potential evaluation was conducted in accordance with the requirements of both NFPA 59A (2006) and ASCE 7-05/IBC (2009) for onshore areas. Based on these evaluations, continuous potentially liquefiable soil layers were not identified at the site, except surficial granular soils in the top 3 feet at the beach area, and the top 9 ft at beach boring B-191. Onshore borings B-28, B-39, B-61, and B-76 which might present higher settlements are located outside the locations of the planned structures. For the rest of the onshore borings (located within Areas 1 through 7), the current data indicates limited and localized liquefactions at depth (unlikely to reach surface) during the OBE and SSE shaking levels. Therefore, liquefaction-induced strength reductions (such as p-multipliers) were not applied in developing the deep foundation recommendations for Areas 1 through 7. Boring B-176 located within Area 8 (on the existing ASRC dock) contains potentially liquefiable soil layer from approximately 18 to 34 ft depth. Therefore, a p-multiplier was used for pile capacity calculations at the B-176 boring location.

7.4 Foundation Protection and Frost Considerations

Fifteen (15) samples of the near surface soils at the site were tested for frost susceptibility. Based on the test results, the gravelly sands at the site can be classified as low to non-frost susceptible. The maximum heave observed in the test samples was about 2.4 percent. We expect that the bearing capacity and settlement of foundations that are placed below the frost penetration depth will not be affected by the frost-heave phenomenon.

Buried pipes in areas of frost susceptible soils should be placed either below the frost penetration depth of 7 ft or at shallower depths with properly designed thermal insulation. Spread footings in frost susceptible soils should be placed at a depth of 7 ft or at a minimum depth of 4 ft with properly designed thermal insulation. Spread footings may bear on existing soils at a depth of 4 ft without insulation provided the soils to a depth of at least 7 ft are non-frost susceptible or provided the

footings can tolerate frost related heave. Frost related heave of spread footings supported at 4 ft over frost susceptible soils will be dependent on the fines content of the bearing material. Frost heave of up to ¾ in, can be expected assuming the fines content of bearing soils is not greater than 32 percent. Lightly loaded mat foundations and slabs-on-grade, if constructed without thermal insulation and supported within on-site frost susceptible soils, may experience more than 1-in. heave depending on the loading and fines content of bearing soils. Insulation techniques, if needed, are discussed in following sections. Spread footings and mat foundations supported in areas with properly compacted non-frost susceptible fill soils to depths greater than 7 ft do not need insulation as frost heave is not expected.

8.0 LNG TANK SHALLOW FOUNDATION RECOMMENDATIONS

We understand that two field-fabricated storage tanks will be constructed in the LNG Tanks area (Area 2). Current plans are to support the proposed LNG tanks (designated as the North Tank and the South Tank) on seismic base isolator foundations, which consist of tank bottoms supported on isolators, underlain by shallow mat foundation consisting of reinforced concrete slab with thickened edges.

Recommendations for the shallow mat foundations were based on field and laboratory test data obtained during the geotechnical field investigation performed during the 2014, 2015 and 2016 G&G programs.

This section presents our recommendations on LNG tank shallow mat foundations, including bearing capacity and settlement estimates for static conditions only. Since non-liquefiable soils are anticipated in this area, we do not expect liquefaction-induced vertical settlements beneath the tank. Our estimated settlements due to seismic conditions are discussed at the end of this section. Recommendations for hydrotesting are also presented in this section. Recommendations provided in this section assume that the proposed tank areas are prepared following the recommendations provided in *Section 4.0 Site Preparation*.

8.1 Tank Loading and Settlement Criteria

Details about the tank size and loading conditions were provided to us by AKLNG and are summarized in Table 8.1.

Table 8.1: Details of LNG Tanks

Tanks	Tank Sizes Dimensions		Net Applied Pressure, psf					
	Diameter ft	Height ft	End of Construction		Hydrotesting		Service/Operating	
			Wall	Floor	Wall	Floor	Wall	Floor
Tank (North & South)	366	160	5760	350	5760	4070	5760	2690

We understand that the settlement criteria for LNG tanks will be as per FERC guidelines and are summarized in Table 8.2.

Table 8.2: FERC Guidelines for LNG Tank Settlements

Settlement Type	FERC Guidelines	Maximum Settlement Criteria Calculated (inches) based on FERC Guidelines
		Tank (North & South)
Center to Edge	1:300	7.3
Along periphery – 20 ft arc	1:500	0.5
Planar Tilt	0.002 radians	8.8

8.2 Tank Pad Preparation

Based on the information provided by AKLNG, current plans are to support the LNG Tanks on seismic base isolator foundations, which consist of tank bottoms supported on isolators, underlain by thick mat with thickened edges. A schematic showing the proposed tank foundation is presented on Plate A-1 of Appendix A. The bottom of the foundation is expected to be at about El. +113 ft at thickened edges, and at about El. +115 ft in the interior portion of the mat foundation. The site grade within tank footprints ranges from about El. +116 to El. +125 ft. Therefore, we do not expect significant fill material will be required to raise grade in the tank area. We recommend that prior to construction, topsoil, unsuitable soils or irregularities should be removed and, if needed, replaced with non-frost susceptible fill to ensure a competent subgrade and to provide a more uniform support for the foundations. Non-frost susceptible soils should be used to backfill against the mat foundation as shown Plate A-1 of Appendix A. As mentioned under *Section 4.4.1Reuse of On-site Material,* majority of the on-site soils are non-frost susceptible, and hence can be used as a backfill in this area. Additional recommendations for fill materials and placement requirements are presented in *Section 17.3 Fill Materials and Placement* of this report. Visual inspection of the subgrade immediately prior to construction by a qualified geotechnical engineer/geologist is also recommended to identify any anomalies or unforeseen conditions, in which case various ground improvement or additional cut and fill mitigations can be considered as required to provide a good supporting condition for the start of tank construction.

8.3 Soil Parameters

2016 G&G program was designed to explore the soils underneath the proposed LNG tanks. Hence, five (5) borings (one at the center, four at the perimeter) were proposed for each tank. Due to the land access constraints, only two (2) borings (B-195 and B-197) were drilled within the approximate footprint of the North Tank and to supplement one (1) boring (B-198) was drilled at a distance of approximately 150 ft from the east edge of the tank; whereas all five (5) borings (B-146 through B-150) were successfully performed within the footprint of the South Tank (Plate 3a and Plate 20). Additionally, borings, test pits, downhole tests and ERT/Shallow Seismic surveys were performed within Area 2 of the project to provide supporting information. Recommendations for the LNG tank foundation were developed based on the borings drilled within the footprints and in the vicinity of the proposed tanks. Following our review of the soil conditions encountered in these borings, Area 2 was further sub-divided into the following areas:

1. Area 2A: North Tank,
2. Area 2B: South Tank, and
3. Area 2C: Trestle and other support structures.

The soil parameters used for tank stability and settlement analysis are summarized in Table 8.3 (Area 2A) and Table 8.4 (Area 2B). The soil parameters selected for Area 2C are presented in *Section 5.4 Selected Soil Design Profiles and Parameters* of this report.

Table 8.3: Selected Soil Design Parameters – Area 2A (North Tank)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4 (0.3(4))	23(4)	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to -20	85 to 145	Sand	62.6	-	36	40	1350
-20 to -180	145 to 305	Clay	62.6	6 to 12 (0.5(4))	27(4)	-	Cc= 0.10, Cr=0.009, OCR=3
-180 and below	305 and below	Sand	65.6	-	37	45	2500

Notes:
1. Groundwater table was estimated at 55 ft below final grade (i.e. El. 70 ft).
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data
4. Drained properties of Clay layers were selected based on literature and laboratory test results.

Table 8.4: Selected Soil Design Parameters – Area 2B (South Tank)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4 (0.3(4))	23(4)	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to 0	85 to 125	Sand	62.6	-	36	40	1350
0 to -15	125 to 140	Clay	62.6	5 (0.5(4))	27(4)	-	Cc= 0.10, Cr=0.009, OCR=3
-15 to -50	140 to 175	Sand	62.6	-	36	40	1350
-50 to -180	175 to 305	Clay	62.6	7 to 12 (0.5(4))	27(4)	-	Cc= 0.10, Cr=0.009, OCR=3
-180 and below	305 and below	Sand	65.6	-	37	45	2500

Notes:
1. Groundwater table was estimated at 55 ft below final grade (i.e. El. 70 ft).
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from literature and consolidation test data
4. Drained properties of Clay layers were selected based on literature and laboratory test results.

8.4 Mat Foundation Recommendations

A properly designed and constructed reinforced concrete mat foundation along with a thickened edge will support and distribute the tank load. The following sub-sections provide our recommendations for foundation depth, tank stability, resistance to uplift loads, and resistance to lateral loads for the proposed tank foundation.

8.4.1 Foundation Depth

Based on the information provided by AKLNG, the foundation system is identical to a mat foundation in principle as shown on Plate A-1. Current plans are to support the LNG Tanks on seismic base isolator foundations, which consist of tank bottoms supported on isolators, underlain by thick mat with thickened edges as shown on Plate A-1. The bottom of the foundation is expected to be at about El. +113 ft at thickened edges, and at about El. +115 ft in the interior portion of the mat foundation. Recommendations for fill materials and placement requirements are presented in *Section 17.0 Construction Considerations* of this report.

8.4.2 Allowable Net Bearing Pressure

The foundation system should provide an adequate factor of safety against shear failure of the subgrade and the settlement should be within the tolerance criteria. Mat foundation design based on recommendations presented in this report for loading conditions in Table 8.1 will have a factor of safety of at least 3 against bearing capacity failure for the above-mentioned loading. Please note that the factor of safety presented herein only applies to foundations/loading placed on level ground surface. Current plans are to construct the tank (nearest edge) approximately 464 ft away from the crest of the bluff. The factor of safety against bearing failure is at least 3 for the tank constructed 464 ft from the crest of the bluff. Please refer to *Section 12.0 Bluff Slope Stability* that presents analyses of slope configuration that also incorporate the tank loading.

8.4.3 Resistance to Lateral Loads

Resistance of mat foundations to lateral loads can be provided by friction or adhesion at the base of the foundation. For concrete foundations poured in good contact with existing subgrade soils, an ultimate coefficient of friction of 0.55 can be used for calculation of lateral resistance[61]. We recommend that the calculated lateral resistance of mat foundations be reduced by a factor of 2 to compute the allowable lateral resistance of the foundation. For the lateral earth pressures acting on the vertical faces of the foundations, please refer to *Section 13.0 Below Grade Structures*.

[61] Foundations & Earth Structures, NAVFAC, Design Manual 7.02, Department of the Navy, September 1986, Chapter 3, Table 1.

8.4.4 Tank Stability

The two most common modes of failure associated with storage tanks are base shear (bearing) failure, and edge shear (bearing) failure. These mechanisms of failure are briefly described below.

- Base Shear: The entire tank acts as a single unit, with the entire base of the tank undergoing downward movement, usually accompanied by some rotation from vertical.
- Edge Shear: Edge shear is the most common mode of bearing failure for storage tanks supported on shallow foundations. In this mode, the near surface soils shear, allowing a small section of the tank to distort.

The following loading conditions were analyzed for our stability analyses:

Static Condition (Short-term Undrained) -- The short-term, or undrained, case models the condition soon after construction in which the pore pressures generated during construction and operation have not dissipated. Analyses for this condition involve the use of undrained shear strength parameters. This condition was analyzed only for hydrotesting load cases.

Static Condition (Long-Term Drained) -- The long-term, or drained, case models the condition in which the pore pressures generated during construction and operation have dissipated. Analyses for this condition involve the use of drained shear strength parameters. This condition was analyzed for operating load cases.

For our stability analyses of the tanks, we used slope stability computer program Slide Version 6.0[62] to evaluate the factors of safety with respect to base and edge shear modes of failure. A minimum factor of safety of 1.3 is recommended for the stability of the tanks both in edge and base shear failures. The parameters used in tank stability and settlement are presented on Table 8.3 and Table 8.4 and results of our stability analyses, including the factor of safety for hydrotesting and operating conditions are presented on Plates A-2 through A-9 in Appendix A. The factor of safety values for base shear type failure at hydrotesting and operating load conditions were greater than 3. The factor of safety values for the edge shear type failure at hydrotesting and operating load conditions were greater than 2.4 (Please refer to Plates A-2 through A-9 in Appendix A for the factor of safety values calculated).

8.5 Settlement Estimate

We evaluated the settlements for the proposed storage tanks in the LNG Tanks area based on the tank size and loads in Table 8.1 of this report. Settlement tolerance criterion in accordance with FERC Guidelines (Table 8.2) was used to evaluate the proposed tank foundations. Total and differential tank movements resulting from settlement of the foundation soils due to the sustained loads are calculated to be within tolerable performance limits specified by FERC. The results of our

[62] Slide – 2D limit equilibrium slope stability for soil and rock slopes. Rocscience Inc. Version 6.0.

settlement analyses are summarized in Table 8.5 and provided on Plates A-10 through A-15 in Appendix A.

Based on the borings performed in this area, the estimates of differential settlements (center to edge) are lower than the settlement criteria presented in Table 8.2. We anticipate that during the operations of the facility, the tank levels will change frequently and will not stay at the full height. For our calculations, we assumed that hydrotesting loads are sustained for about two weeks and operating loads are applied for a long duration (30 years). We also analyzed an end of construction case for the LNG Tanks. Our analyses were performed assuming the loads are applied immediately and construction/loading times were not considered. The majority (about 90 percent) of the estimated settlements from the granular layers are expected to occur soon after the application of the hydrotesting load. Due to the characteristics of granular layers on-site, we will only expect to have small amount of immediate settlement during operations. The results of the settlement analysis are summarized in Table 8.5.

Additionally, post-earthquake settlements are not expected to be significant and based on the seismic slope stability assessment using horizontal time histories, the mean vertical settlements at the tank locations are expected to be on the order of 1 inch for OBE level and 3 inches for the SSE level.

Table 8.5: Results of Tank Settlement Analysis

Cases	End of Construction Settlements (in)(1)		Hydrotesting Settlements (in)(1),(2)		Long-Term Settlements (in)(1),(3)	
	Center	Edge	Center	Edge	Center	Edge
North Tank	1 to 2	3 to 4	8 to 9	8.5 to 9.5	8.5 to 9.5	9 to 10
South Tank	1 to 2	3 to 4	8.5 to 9.5	9 to 10	9 to 10	9.5 to 10.5

Note:
1. These are the estimated values; however actual values could be outside the range given in the table above.
2. 15 days of hydrotesting load and 2 days of unloading period were assumed.
3. Long-term settlements are the settlements generated during 30 years of operation after the completion of tank construction. Long-term settlements were calculated by adding consolidation settlements of clay over 30 years to the hydrotesting settlements.
4. It should be noted that the remaining FERC criteria such as settlements along the periphery and planar tilt were not evaluated as uniform soil conditions were encountered. However, additional tank specific borings should be performed especially for the North Tank to confirm uniform subgrade conditions.

8.6 Hydrotesting Recommendations

The proposed tanks should be tested before being placed in service according to current API Specifications[63]. This testing generally consists of filling the tanks with water after construction, under controlled conditions to:

- Check the competency of the construction,
- Verify the ability of the foundation materials to carry the loads imposed by the tank and contents, and
- Reduce the amount of settlement that each tank will experience after it is placed in service.

We recommend implementation of a controlled, stage-loaded hydrotesting program for the proposed tanks, accompanied by a movement-monitoring program. The following paragraphs discuss our recommendations for hydrotesting.

8.6.1 Survey Control

We recommend that a series of reference points be established around each tank shell in order to monitor the movements of the tank during hydrotesting. API recommends at least 8 reference points be established around the perimeter of a tank, but with spacing no more than about 30 ft apart. Elevations of these points should be obtained prior to, during, and after hydrotesting. Elevation readings should be taken to the nearest millimeter. Readings should be referenced to a benchmark well away from the zone of influence of each tank (*i.e.* at least one tank diameter).

8.6.2 Loading

We suggest that, initially, water for hydrotesting be added to the tanks to reach one-fourth of the full tank height. This water level should be maintained until settlement observations indicate the rate of settlement has decreased significantly (at least 24 hours). Additional stage-loading increments should include the one-half, three-fourths mark, and the maximum test fluid level per API 625. Each level should be maintained until settlement readings are smaller than the range provided in Table 8.2. If excessive settlement rates are observed, smaller loading increments may be required.

8.6.3 Data Evaluation

Settlement readings should be analyzed as the hydrotesting proceeds to assess total settlement, differential settlement, local slope, tank tilt, and out-of-plane distortion for the tanks. The project engineer and/or the tank manufacturer should establish criteria for acceptable settlement prior to

[63] API Standard 620, (2013), Design and Construction of Large, Welded, Low-Pressure Storage Tanks, American Petroleum Institute, 12th edition.

hydrotesting. If settlements start approaching unacceptable limits, tank filling should be halted and not resumed until the situation is reviewed and evaluated by the structural engineer and/or tank contractor. Settlement observations should be obtained frequently to develop the data necessary to assess tank performance, preferably once daily. The Geotechnical Engineer of Record should be given the opportunity to review and evaluate the settlement data during hydrotesting of the completed tanks for foundation performance assessment.

9.0 SHALLOW FOUNDATION RECOMMENDATIONS

Recommendations for spread footings and mat foundations including foundation depth, allowable net bearing pressure, resistance to uplift loads, resistance to lateral loads, settlements, coefficient of subgrade reaction, and frost considerations are provided in this section. Spread footings, as used in this report, refer to square and rectangular footings used to support a single column load. Mat foundations are typically defined as monolithically constructed shallow foundation units that support multiple geometric lines of column and/or wall loads. The recommendations presented in this section are applicable to Areas 1 through 5. Area 6 and Area 7 will be used as laydown area and camp area, respectively; so our recommendations provided under *Section 10.0 Slab-on-Grade Recommendations* can be used for temporary storage of equipment and camp area structures.

From a geotechnical perspective, the performance of a shallow foundation should provide an adequate factor of safety against failure of the foundation soils and reduce the potential for excessive settlements due to stressing of the underlying foundation soils. Shallow foundations should be designed such that their applied bearing pressures do not exceed the allowable net bearing pressure of the underlying soils.

9.1 Foundation Depths

We recommend that the proposed shallow foundations be placed at a minimum depth of 4 ft below lowest adjacent grade. Please refer to *Section 7.4 Foundation Protection and Frost Considerations* for additional information. Insulation may be used to mitigate frost related heave, if near-surface fine-grained soils remain after site preparation. Our insulation recommendations are provided in Appendix B.

9.2 Allowable Bearing Capacity

The allowable bearing capacity for shallow foundations is a function of, among other items, the strength of the foundation soils, the depth of the foundation, the shape of the foundation, and the recommended factor of safety. It should be noted that shallow foundation design should also satisfy serviceability (short-term and long-term settlements). The allowable bearing capacity at this site will be limited by settlement/serviceability and not due to ultimate bearing capacity. Table 9.1 provides bearing capacities with factors of safety (F.S.) greater than 2 for total loading and greater than 3 for sustained loading.

Sustained loading conditions as described in this report refer to the combination of dead and, in some cases, a portion of the sustained live loads. Total loading conditions as described in this report refer to properly factored dead and live loads. Transient loading conditions refer to the combination of properly factored dead, live, and infrequent transient (*e.g.*, wind) loads.

The presented allowable net bearing pressures are for shallow footings and mat foundations bearing on undisturbed, natural competent granular soils or on properly compacted non-frost susceptible,

granular fill soils, without organic or other deleterious material. The design engineer should select an allowable net bearing pressure, considering settlement criteria (Please refer to Plates B-1 through B-18 in Appendix B for the estimated settlement of shallow foundations) established for the proposed structures and equipment, located within Areas 2 through 5. Please note that the design of thrust blocks for piping will be similar to those of below-grade structures presented in *Section 13.0 Below Grade Structures*.

Table 9.1: Allowable Net Bearing Pressure for Shallow Foundations

Areas	Type of Foundation (1)	Foundation Depth (2)	Loading Conditions(4) (psf)		
			Sustained	Total	Transient
			(F.S.(3) of >3.0)	(F.S. of >2.0)	(F.S. of >1.5)
Areas 1 to 5	Spread/Mat Foundations	4 ft	5,000	7,500	10,000
	Strip Footings	4 ft	5,000	7,500	10,000
Areas 1 to 5	Spread/Mat Foundations	7 ft	8,000	12,000	16,000
	Strip Footings	7 ft	8,000	12,000	16,000

Notes:

1. The allowable bearing capacities presented are applicable for spread footings (widths/diameters and lengths less than about 15 ft), strip footings (widths of at least 1.5 ft), and mat foundations (widths/diameters and lengths greater than about 15 ft) supported on natural dense granular soils.
2. Spread and strip footings and mat foundations should have an embedment of at least 4 ft below final site grade.
3. F.S. refers to factor of safety.
4. Refer to report text below for additional information relating to loading conditions.
5. For dynamically-loaded foundations, it is common to use an allowable bearing capacity equal to 50 percent of the allowable net bearing pressure for static loads. It should be noted that this method provides an estimate of the bearing capacity for dynamically-loaded foundations and does not address the dynamic response of the foundations.

Foundations should be proportioned so that the maximum contact pressure under various load combinations does not exceed the allowable net bearing pressure given previously. Net bearing pressure and gross bearing pressure are defined on Plate B-19 in Appendix B. To calculate values of W_e (weight of soil located above base of foundation excavation and lowest adjacent grade), W_s (weight of soil located above foundation), and W_f (weight of foundation) on Plate B-19, use total unit weights of 118 pcf for soil and 150 pcf for reinforced concrete above the long-term design subsurface water level elevation.

9.3 Resistance to Uplift Loads

The ultimate uplift capacity of spread footings and strip footings are limited to the weight of the footing plus the weight of any soil directly above the footing. We recommend a total unit weight of 118 pcf for soil and 150 pcf for concrete to compute the ultimate uplift capacity. Buoyancy is not expected for shallow foundations at the site. We recommend that the calculated ultimate uplift capacity for

spread and strip footings be reduced by a factor of safety of 1.5 to compute the allowable uplift capacity.

9.4 Resistance to Overturning Loads

The design of spread footings and strip footings subjected to overturning moments should consider a trapezoidal distribution of contact pressure between the base of the foundation and the foundation soil such that full soil contact and positive pressure is maintained under the foundation. The maximum contact pressure should not exceed the allowable net bearing pressure for the foundation soil. Please note that very-lightly loaded structures subject to high lateral loads may result in negative contact pressures. Those cases should be evaluated by a geotechnical engineer on a case-by-case basis and may be deemed acceptable, after evaluation as an exception to the positive contact pressure requirement.

9.5 Resistance to Lateral Loads

Resistance of spread footings and mat foundations to lateral loads can be provided by friction at the base and passive earth pressure acting on the front of the foundation. For design purposes, the resistance due to friction and passive soil pressure can be assumed to develop simultaneously.

For concrete foundations poured in good contact with the granular structural fill, a coefficient of friction of 0.55 may be used between concrete and structural fill[64]. For this calculation, only the dead weight of a structure and its foundation should be used. The following soil parameters can be used in the computation of coefficient of passive earth pressures using the following equation:

$$K_p = (1 + \sin \phi)/(1 - \sin \phi) = 3.54$$

where: K_p = coefficient of passive earth pressure, and

ϕ = soil friction angle, 34 degrees.

Passive resistance within the top 2 feet should be neglected unless area paving or other impervious barrier is provided that extends out from the footing laterally at least 5 feet in all directions. We recommend that the calculated lateral resistance and passive earth resistance of spread footings and mat foundations be reduced by a factor of 2 to compute the allowable resistances of the foundations. We do not recommend increasing allowable values for various loading conditions.

9.6 Settlement Estimates

The majority (more than 90 percent) of the anticipated settlement within each area will be elastic in nature, and is expected to occur immediately after construction and is not of significance for design

[64] Naval Facilities Engineering Command (NAVFAC), Foundations & Earth Structures, Design Manual 7.02, Department of the Navy, September 1986, Chapter 3, Table 1.

of the structures (i.e. immediate settlement). Hence, we selected conservative profiles that will yield higher order settlements to provide an order of magnitude assessment of settlements. We performed settlement analyses for individual shallow foundations for Areas 2 and 4 using Area 2B profile presented below.

Table 9.2: Design Profile for Area 2B Settlement Evaluation

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1) pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es ,ksf
125 to 100	0 to 25	Sand/Gravel	118	-	34	25	600
100 to 80	25 to 45	Sand/Gravel	120	-	35	30	850
80 to 70	45 to 55	Sand/Gravel	123	-	36	35	1200
70 to 60	55 to 65	Sand/Gravel	60.6	-	36	35	1200
60 to 40	65 to 85	Clay	60.6	4	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to 0	85 to 125	Sand	62.6	-	36	40	1350
0 to -15	125 to 140	Clay	62.6	5	-	-	Cc= 0.10, Cr=0.009, OCR=3
-15 to -50	140 to 175	Sand	62.6	-	36	40	1350
-50 to -180	175 to 305	Clay	62.6	7 to 12	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 55 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from consolidation test data.

We performed settlement analyses for individual shallow foundations for Areas 1 and 3 using Area 3 profile presented below.

Table 9.3: Design Profile for Area 3 Settlement Evaluation

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
116 to100	0 to 16	Sand/Gravel	118	-	34	25	600
100 to 80	16 to 31	Sand/Gravel	120	-	35	30	700
80 to 70	31 to 41	Clay	123	4	-	-	Cc= 0.15, Cr=0.01, OCR=5
70 to 50	41 to 61	Sand/Gravel	62.6	-	36	40	1150
50 to 40	61 to 76	Clay	62.6	4	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
40 to -15	76 to 131	Sand	65.6	-	37	45	1300

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
-15 to -80	131 to 196	Clay	65.6	5 to 8	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 46 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from consolidation test data.

We performed settlement analyses for individual shallow foundations for Area 5 using Area 5 profile presented below.

Table 9.4: Design Profile for Area 5 Settlement Evaluation

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Main Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle, degrees	$N_{1,60}$ (2)	Compressibility Parameters (3)
							Es, ksf
116 to105	0 to 11	Sand/Gravel	118	-	34	25	600
105 to 90	11 to 26	Sand/Gravel	120	-	35	30	800
90 to 85	26 to 31	Sand/Gravel	123	-	36	35	900
85 to 70	31 to 46	Sand/Gravel	60.6	-	36	35	900
70 to 60	46 to 56	Clay	62.6	4	-	-	Cc= 0.15, Cr=0.01, OCR=5
60 to 40	56 to 76	Sand	62.6	-	36	40	1000
40 to 25	76 to 91	Clay	65.6	5	-	-	Cc= 0.13, Cr=0.01, OCR=4.5
25 to -5	91 to 121	Sand	65.6	-	37	45	1100
-5 to -50	121 to 166	Clay	65.6	5 to 6.5	-	-	Cc= 0.10, Cr=0.009, OCR=3

Notes:
1. Groundwater table was estimated at 31 ft below final grade.
2. SPT N values were corrected for overburden and hammer efficiency, $N_{1,60}$.
3. Strain based Cc and Cr values were estimated from consolidation test data.

We used the computer program Settle3D 3.0, developed by Rocscience Inc. This program uses consolidation theory to estimate consolidation settlements in clayey soils and the Boussinesq theory to calculate immediate settlement in sandy soils. Given a pressure applied to the ground by a foundation, the program then uses the soil parameters to evaluate the change in thickness of individual layers and computes the overall movement of the foundation at the selected locations.

We considered square footings, strip footings, and mat foundations in our analyses. We computed the settlements for shallow foundations constructed in open-cut excavations and placed on natural soils and properly compacted fill material with an embedment depth of 4 ft and 7 ft below proposed final grade. The computed curves showing the applied pressure versus foundation settlement values are presented in Appendix B on Plates B-1 through B-6 for Areas 2 and 4; on Plates B-7 through B-12 for Area 3, and Plates B-13 through B-18 for Areas 5.

In our analyses, the foundations were assumed to act as isolated foundations, that is, the clear spacing between foundations is at least the width of the largest foundation. Two or more foundations placed closer than about the width of the largest foundation will probably experience greater settlements than those presented on Plates B-1 through B-18 in Appendix B. If foundation spacing is less than the width of the largest foundation and it appears that settlement will be critical, we recommend that a detailed settlement analysis be performed on a case-by-case basis. Large groups of foundations may also require a detailed group settlement analysis. Specific information regarding the foundation sizes, bearing depths, loads, and spacing is required to perform detailed settlement analysis.

Foundations settlements presented on Plates B-1 through B-18 do not include settlements calculated due to site grade raise. The settlements due to site grade raise are presented in *Section 6.0 Settlement Due to Site Grade Raise*. Most of the settlement due to site grade raise is expected to occur prior to construction of the foundations. If the calculated settlements are not within the tolerable limits, the selected bearing pressures should be reduced such that the settlements estimated based on Plates B-1 through B-18 will reduce within tolerable limits.

9.7 Coefficient of Subgrade Reaction for Mat Foundations

The design of mat foundations involves the distribution of various magnitudes of point loads across the foundation. Typical analyses require a coefficient of subgrade reaction, which is defined as the ratio between the pressure at any given point on the surface of contact and the deformation produced by the load application at that point. Input dimensions of the value of the coefficient of subgrade reaction for specific computer programs should be reviewed carefully. Some programs require a subgrade value for a 1-ft by 1-ft plate while others require the subgrade value for the actual foundation size. We are providing a coefficient of subgrade reaction value for a 1-ft by 1-ft plate. Terzaghi[65] also provides equations for adjusting the coefficient of subgrade reaction for a 1-ft by 1-ft plate to actual foundation dimensions as follows:

$$k = k_1 [(B + 1) / (2B)]^2$$

where: k = coefficient of subgrade reaction for actual foundation

[65] Terzaghi, Karl (1955), "Evaluation of Coefficients of Subgrade Reaction", Geotechnique, London, Vol 5, No. 4, pp. 297-326.

dimension, (pci)

k_1 = coefficient of subgrade reaction for a 1-ft x 1-ft plate, (pci)

B = foundation width, (ft)

Based on published correlations for similar materials, the modulus of subgrade reaction for a 1-ft by 1-ft plate can be taken as 150 pci, for undisturbed cut and properly placed and well compacted existing granular soils or non-frost susceptible fill. This value for the modulus of subgrade reaction may be used for both vertical and horizontal directions.

9.8 Insulation Recommendations

Spread Foundations for Heated Buildings. Spread foundations constructed above frost depth should be properly insulated if frost heave is expected due to frost susceptible soils. Insulation is not needed if the soils below the slab-on-grade are non-frost susceptible to a depth of at least 7 ft below final grade. Insulation for heated buildings may also be considered for energy efficiency and to provide protection for utility lines from freezing. Recommendations on insulation details for slab-on-grade foundations for heated buildings are presented on Plate B-20.

For heated buildings, a recommended minimum thickness of rigid polystyrene insulation (e.g. Styrofoam HI) of 2 in. should be installed per Plate B-20. It should be noted that rigid polystyrene insulation is prone to damage by construction equipment and, therefore, needs proper care during placement and adequate cover to support traffic loading and prevent damage. It is also susceptible to hydrocarbon attack and should generally not be used where there is a risk of hydrocarbon spills. Where there is potential for hydrocarbon attack, other forms of insulation such as cellular concrete should be used. The insulation should also be checked for expected loading conditions during and after construction. The compressive strength of the rigid insulation should be 3 times (or greater than) the design bearing pressure of the slab to avoid overstressing the rigid insulation.

Spread Foundations for Unheated Buildings. Insulation for unheated buildings may be considered as preventive measure if frost susceptible soils are present below slab-on-grade within frost depth. Typical details for insulation under unheated slab-on-grade foundations are shown on Plate B-21. The minimum recommended distance (L) of rigid insulation from the edge of slab is 8 ft. A minimum thickness of 6.5 in of rigid polystyrene insulation (e.g. Styrofoam HI) is recommended. All structural granular fill should be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D4253 and D4254). The compressive strength of the rigid insulation should be 3 times (or greater than) the design bearing pressure of the slab to avoid overstressing the rigid insulation.

10.0 SLAB-ON-GRADE FOUNDATION RECOMMENDATIONS

10.1 General

The following sections provide recommendations for the design of slab-on-grade foundations. The recommendations are applicable to Areas 2 through 6, and are also applicable for the camp area (Area 7). Slab-on-grade foundations are expected to be supported in non-frost susceptible fill soils and possibly on in-situ granular soils. Frost considerations are presented in *Section 7.4 Foundation Protection and Frost Considerations*. Positive drainage away from the slab-on-grade foundations must be maintained throughout the life of the structures.

10.2 Bearing Capacity Recommendations

Slab-on-grade foundations should be proportioned so that the maximum contact pressure under dead loads, sustained live loads, and transient loads does not exceed the allowable net bearing pressure for the foundation soil or insulation, if present. We recommend an allowable net bearing pressure of 1,500 psf for slab-on-grade foundations bearing on the existing natural soils or compacted non-frost susceptible soils at depths of at least 1.5-ft below-grade. This allowable net bearing pressure includes a factor of safety of at least 3 with respect to ultimate values. Net bearing pressure, as used in this report, is defined on Plate B-19 in Appendix B.

10.3 Insulation Recommendations

Slabs-on-Grade Foundations for Heated Buildings. Slab-on-grade foundations constructed above frost depth should be properly insulated if frost heave is expected due to frost susceptible soils. Insulation is not needed if the soils below the slab-on-grade are non-frost susceptible to a depth of at least 7 ft below final grade. Insulation for heated buildings may also be considered for energy efficiency and to provide protection for utility lines from freezing. Recommendations on insulation details for slab-on-grade foundations for heated buildings are presented on Plate B-22.

For heated buildings, a recommended minimum thickness of rigid polystyrene insulation (e.g. Styrofoam HI) of 2 in. should be installed per Plate B-22. It should be noted that rigid polystyrene insulation is prone to damage by construction equipment and, therefore, needs proper care during placement and adequate cover to support traffic loading and prevent damage. It is also susceptible to hydrocarbon attack and should generally not be used where there is a risk of hydrocarbon spills. Where there is potential for hydrocarbon attack, other forms of insulation such as cellular concrete should be used. The insulation should also be checked for expected loading conditions during and after construction. The compressive strength of the rigid insulation should be 3 times (or greater than) the design bearing pressure of the slab to avoid overstressing the rigid insulation.

Slabs-on-Grade Foundations for Unheated Buildings. Insulation for unheated buildings may be considered as preventive measure if frost susceptible soils are present below slab-on-grade within

frost depth. Typical details for insulation under unheated slab-on-grade foundations are shown on Plate B-23. The minimum recommended distance (L) of rigid insulation from the edge of slab is 8 ft. A minimum thickness of 6.5 in of rigid polystyrene insulation (e.g. Styrofoam HI) is recommended. All structural granular fill should be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D4253 and D4254). The compressive strength of the rigid insulation should be 3 times (or greater than) the design bearing pressure of the slab to avoid overstressing the rigid insulation.

11.0 DEEP FOUNDATION DESIGN RECOMMENDATIONS

11.1 General

As an alternative to shallow foundations, all moderately to heavily loaded equipment/structures can be supported on deep foundations. For this study, we have developed deep foundation recommendations for the proposed onshore structures. Deep foundation recommendations for the proposed marine structures (e.g. Trestle and MOF) are discussed in the Aconex Document No. USAL-FG-GRZZZ-90-002015-012[66]. Following open-ended steel pipe pile sizes were analyzed for this study:

- 16-in diameter open-ended steel pipe pile with 0.375-in wall thickness,
- 18-in diameter open-ended steel pipe pile with 0.5-in and 0.625-in wall thickness, and
- 24-in diameter open-ended steel pipe pile with 0.5-in and 0.625-in wall thickness.

Additionally, per AKLNG's request, we have developed deep foundation recommendations for the following cast-in-place concrete piles for Area 8 (Separately based on Borings B-176 and B-183) only:

- 24-in diameter cast-in-place concrete pile, and
- 36-in diameter cast-in-place concrete pile.

This section presents our recommendations for the proposed pile foundations as well as method of analysis, soil parameters, static axial capacity, minimum pile penetration depths, group effects, load tests and lateral capacities.

The pile capacity and settlement curves computed for the above-listed piles are presented in Appendix C (on plates that are summarized in Table 11.1) of this report for Areas 2, 3, 4, 5, and 8. The "penetration" shown on the ultimate axial pile capacity curves correspond to the assumed design grade elevations, also summarized in Table 11.1.

Table 11.1: Summary of Deep Foundation Recommendations

Engineering Areas	Sub-Areas	Design Grade Elevation, ft (NAVD88)	Axial Capacity	Settlement	Lateral Capacity
Area 2(1)	2A 2B 2C	125	C-10 thru C-13	C-34	C-42 thru C-45
Area 3	-	116	C-14 thru C-17	C-35	C-46 thru C-49(3)

[66] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-15, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012, Rev.0, dated September 06, 2016.

Engineering Areas	Sub-Areas	Design Grade Elevation, ft (NAVD88)	Axial Capacity	Settlement	Lateral Capacity
Area 4(2)	4A 4B	125	C-18 thru C-21	C-36	C-50 thru C-53
Area 5	-	116	C-22 thru C-25	C-37	C-46 thru C-49(3)
Area 8	B-176 B-183	27 99	C-26 thru C-29 C-30 thru C-33	C-38 and C-39 C-40 and C-41	C-54 thru C-57 C-58 thru C-63

Notes:
1. Area 2 deep foundation recommendations are valid for Area 2A, 2B and 2C.
2. Area 4 deep foundation recommendations are valid for Area 4A and 4B.
3. Area 3 and Area 5 lateral capacity curves are identical.

11.2 Soil Parameters

The design soil parameters for deep foundations were selected based on soil data obtained from field and laboratory tests as discussed in *Section 5.0 Selected Soil Design Profiles and Interpreted Engineering Parameters* of this report. Deep foundation recommendations provided here in are applicable to Areas 2 through 5 and Area 8. Soil parameters used for axial capacity and lateral capacity analyses are presented on Plates C-1 through C-9 in Appendix C, respectively.

11.3 Axial Capacity

We computed the static and seismic axial capacity, in both compression and tension, for open-ended steel pipe piles and the cast-in-place concrete piles based on the selected soil design parameters (see Plates C-1 through C-9). We computed the geotechnical axial pile capacity of individual, isolated driven open-ended steel pipe piles and cast-in-place concrete piles using computer programs/methods APILE v2015 / API RP 2A (1986)[67] and SHAFT v2012 / FHWA (1988)[68], respectively. In these methods, the ultimate compression capacity of a pile for a given penetration was computed as the sum of the skin friction acting on the pile wall and the end bearing.

For the open-ended steel pipe piles, coring and plugging conditions were evaluated in the computations of the ultimate axial capacity. The coring condition generally controls as an open-end pipe pile is initially driven into the ground. When the pile is coring, relative movement between pile and soil occurs both on the outside and inside of the pile wall. The end bearing area is only the cross-sectional area of steel. As the pile is driven deeper into the soil, the soil friction on the inside of the pile wall increases until a soil plug forms. When the pile plugs, the soil plug moves downward with the pile during driving. Pile resistance is mobilized only on the outer wall. The end bearing area during plugging is the gross cross-sectional area of the pile. The weights of the pile and soil plug, if applicable, are neglected in the computations.

[67] American Petroleum Institute, 1986, Recommended Practice for Planning, Designing, and Construction Fixed Offshore Platforms, API Recommended Practice 2A-WSD (RP 2A-WSD), 21st Ed., API, Washington, D.C.
[68] Reese, L. C. and M.W. O'Neill, 1988, Drilled Shafts: Construction Procedures and Design Methods, Prepared for U. S. Department of Transportation, Federal Highway Administration, Office of Implementation McLean, Virginia.

For the cast-in-place concrete piles, the ultimate compression capacity of a pile for a given penetration was computed as the sum of the skin friction acting on the pile and the end bearing defined as the capacity that would be achieved at a pile tip settlement of 5% percent of the pile base diameter (FHWA, 1988).

The design frost penetration depth at the project site was taken as 7 ft below the final grade. For geotechnical axial capacity in compression and tension, we neglected the soil strength in the upper 5 ft below final grade to account for construction disturbance and freeze-thaw conditions.

Fugro conducted earthquake engineering studies to estimate the severity of ground motions and the soil liquefaction potential that may affect the performance of the proposed foundations (AKLNG Document No. USAL-FG-GRZZZ-00-002016-008[69], also *Section 2.11 Probabilistic Seismic Hazard Assessment (PSHA) & Site Response* of this report). Based on the findings, boring B-176 (Area 8) contains a potentially liquefiable granular soil layer from approximately 18 to 34 ft depth from the existing grade level. As the estimated groundwater table is at 20 ft below the existing grade, we have modeled the soils between 20 ft and 34 ft as liquefiable and estimated a residual shear strength of 0.2 ksf based on Idriss and Boulanger (2008)[70] (see Plate C-8 for B-176 pile capacity design parameters).

The ultimate axial capacity at a given penetration may be obtained directly from the appropriate curve. For the open-ended steel pipe piles the pile capacity curves are limited to 60 ft below assumed rough grade based on our drivability analysis that is discussed separately in *Section 17.5.1 Pile Drivability*. Whereas, the maximum pile lengths for the cast-in-place concrete piles of 24-in and 36-in diameter, proposed for Area 8 (B-183), were calculated as 80 ft and 70 ft, respectively, for an available soil profile of 100 ft below the design grade and based on the assumption of an influence zone of 10D (10 times diameter of the pile) below the pile tip.

The capacities presented in Appendix C are ultimate capacities and should be reduced by an appropriate factor of safety to obtain allowable capacities for the design of steel pipe piles. We recommend a factor of safety of at least 2 for compression loads (transient and sustained) and transient uplift loads. A factor of safety of 3 is recommended for sustained uplift loads.

11.4 Adfreeze Stresses

In general, adfreeze (uplift or jacking due to frost) forces will be imposed on the piles and pile caps that are subjected to ground freezing in frost-susceptible soils. There is limited information in the literature dealing with measurement of adfreeze stresses on piles due to frost heave. The adfreeze

[69] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Seismic Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-9, AKLNG Document No. USAL-FG-GRZZZ-00-002016-008, Rev.0, dated December 16, 2016.
[70] Idrss, IM and RW Boulanger, 2008, Soil Liquefaction During Earthquakes, EERI Monograph MNO-12, EERI, Oakland, CA.

stresses depend on many factors, such as soil type, water content, soil temperature, salinity etc. The Canadian Foundation Engineering Manual, 4th Edition, 2006 (Section 13.5.1) cites the studies completed by Penner [71] and Penner and Goodrich[72] related to adfreeze stresses on steel and concrete piles, and provides a range of adfreeze stress values between 9.4 psi and 21.8 psi. Russian Standard SNiP[73] provides a range of adfreeze stresses between 5.8 psi and 16 psi. The surficial granular soils at the site are expected to be low to non-frost susceptible based on test results. The adfreeze stress, if any, is expected to be low to negligible based on the current available data. If the existing soils are non-frost susceptible or are replaced with structural fill (*Section 17.3.1 Structural Fill*) within the frost zone (i.e., 7 ft below the rough grade), or are heated foundations with a pile cap, then those piles are not expected to experience any adfreeze stresses/forces due to frost-heave action.

11.5 Negative Skin Friction

Negative skin friction typically developed when piles are driven or bored through soft and compressible soils and there is site grade raise. Site grade raise planned in certain areas of this project may result in additional downdrag (negative skin friction) on the piles installed in the fill area. Due to the existing medium dense to very dense granular soils and the stiffness of the on-site soils we do not expect negative skin friction and associated downdrag forces to be developed. However, we recommend that the piles be installed after backfilling the entire area and then the individual pile caps be constructed. In order to reduce the effects of negative skin friction and piping/utility distress, we recommend waiting for as much time as possible prior to driving piles and making final piping/utility connections.

It should also be noted that an appreciable soil settlement, if occurs, within the pile foundation due to liquefaction and strong shaking can subject the entire pile foundation to downdrag forces. As mentioned earlier, generally, continuous potentially liquefiable soil layers were not identified within the borings drilled in Areas 1 through 7 (*Section 2.12*). However, Boring B-176 which is close to the coast and located in Area 8 of the project site, contains a potentially liquefiable soil layer from approximately 18 to 34 ft depth. Calculations were performed for the open-ended steel pipe piles proposed for the B-176 location, as a result of the potential liquefaction in the event of earthquake[74].

[71] Penner, E., 1974, Uplift Forces on Foundations in Frost Heaving Soils, Canadian Geotechnical Journal, Vol 11, No 3,
[72] Penner and Goodrich, L.E., 1983, Adfreeze Stresses on Steel Pipe Piles, Thompson, Monitoba. Proceedings, 4th International Conference on Permafrost. National Academy Press, Washington, D.C. pp.261-285.
[73] Russian Standard SNiP. SP. 24.13330.2011. Pile Foundations, pp. 84.
[74] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Seismic Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10160001-9, AKLNG Document No. USAL-FG-GRZZZ-00-002016-008, Rev.0, dated December 16, 2016.

Numerous techniques have been presented to evaluate the magnitude of negative skin friction. We estimated the magnitude of static negative skin friction based on Fellenius (2004)[75] and Fellenius and Siegel (2008)[76]. In Fellenius method (2004), the neutral plane is defined based on the dead load given (here assumed as the half of the ultimate capacity of the piles), together with pile geometry and its unit shaft friction and end bearing values. This theory assumes that the neutral plane will be formed at a depth at which the pile settlement and soil settlement are equal. The skin friction above neutral plane will be regarded as negative skin friction (downdrag load) induced on pile due to settlement of soil around pile. The induced downdrag load will not have any impact on ultimate pile capacity. However, the piles with downdrag load will experience additional settlement. Pile settlement is equal to the soil settlement at the neutral plane plus the elastic compression of the pile segment above the neutral plane. Based on our calculations, neutral planes are developed below the liquefiable layer, at 42 ft and 43 ft below the existing ground surface, for 16-in and 18-in open-ended steel pipe piles, respectively.

Fellenius and Siegel (2008) suggests that the effect of liquefaction is limited to loss of negative skin friction in the liquefied soils and the liquefaction occurring above the static neutral plane has minor effect on the axial capacity of the pile.

11.6 Settlement Considerations

We estimated settlement of single, isolated open-ended steel pipe piles and isolated cast-in-place concrete piles using computer programs APILE v2015 / API RP 2A (1986) and SHAFT v2012 / FHWA (1988), respectively. We calculated the side load transfer versus pile displacement (t-z) and tip load transfer versus tip displacement (q-z). Elastic shortening of piles is also included in the load-settlement curves. The computed ultimate axial load versus pile top settlement curves for open-ended steel pipe piles and cast-in-place concrete piles for the pile lengths indicated in *Section 11.3 Axial Capacity*, are presented in Appendix C.

Pile groups will likely settle more than an individual pile subject to the same load per pile. The settlement of pile groups is dependent on several variables including dimension of the group, the pile length, the sustained structural load, and the compressibility characteristics of the foundation soils. We also expect that pile group settlement will be the same as the single pile settlement under the same per pile load, if the pile spacing is greater than 7 times the pile diameter. If pile group settlement controls the design of a specific structure, detailed settlement analysis should be performed on a case-by-case basis.

[75] Fellinius, B.H., 2004. Unified design of piled foundations with emphasis on settlement analysis. Honoring George G. Goble – "Current Practice and Future Trends in Deep Foundations" Geo-Institute Geo Trans Conference, Los Angeles, July 27-30, 2004. ASCE Geotechnical Special Publiciation, GSP 125, pp. 253 – 275.
[76] Fellenius, B.H. and Siegel, T.C., 2008, Pile Drag Load and Downdrag in a Liquefaction Event. Journal of Geotechnical and Geoenvironmental Engineering, 134(9): 1412-1416.

11.7 Axial Group Effects

The overall allowable axial load carrying capacity of a large group of piles can be less than the sum of the individual allowable capacities. A reduction in the individual pile capacity, to allow for group effects, is usually not necessary for piles having a sufficient center-to-center spacing. The reduction in individual capacity depends on many factors including the configuration of the group, number of piles in the group, pile size, the depth of installation, and the pile spacing. We expect that axial group effects will be relatively small provided that center-to-center spacing is at least 3 pile diameters.

11.8 Lateral Capacity

We evaluated the lateral resistance of 16-, 18-, and 24-inch diameter steel pipe piles with wall thicknesses of 0.375-, 0.5-, and 0.625-inch, respectively as well as 24-, and 36-inch cast-in-place concrete piles. The effective pile wall thicknesses of the steel pipe piles were reduced by approximately 0.06 in. to account for corrosion (Please refer to *Section 16.0 Soil Corrosion Potential* for discussion of corrosion allowance). The following pile parameters were used in our analyses:

- Young's Modulus of steel pipe piles is equal to 29,000 ksi;
- Young's Modulus of cast-in-place concrete piles is equal to 3,600 ksi for uncracked conditions and 1,260 ksi for cracked conditions;
- Length of the open-ended steel pipe piles was assumed to be up to 60-ft;
- Length of the cast-in-place concrete piles were assumed to be up to 80-ft for the 24-in dia piles, and 70-ft for the 36-in dia piles,
- Lateral loads were applied at ground surface which is the pile top;
- Both free and fixed head conditions were considered in our single pile analyses; and
- Axial load was applied on top of the piles.

We performed lateral capacity analyses using the computer program LPILE Plus 6.0.12 (2010) (LPILE), developed by Ensoft, Inc. This program uses finite difference numerical techniques to compute lateral deflections and bending moments induced in a pile due to lateral and axial loads applied at the top of the pile. The pile-soil system is modelled as a series of finite segments that represent the pile and the soil. Soil resistance is calculated using p-y curves developed from a distribution of input soil unit weights and strength parameters specific to the subsurface conditions in this area.

As mentioned in *Section 11.3 Axial Capacity*, we considered earthquake conditions in our analyses and at B-176 (Area 8) the soils between 20 ft and 34 ft were modelled as liquefiable. In order to implement the effects of the liquefiable soils into the lateral capacity analyses, conducted using

LPILE for the proposed pile sections at B-176, we estimated a p-multiplier of 0.1 based on Liu and Dobry (1995)[77] (see Plate C-8 for B-176 pile capacity design parameters).

Our results of lateral capacities for the above-mentioned piles are presented on Plates C-42 through C-63. These results are applicable for piles installed to minimum depths shown in the following table:

Table 11.2: Required Minimum Pile Penetrations for Lateral Capacity

Pile Diameter, inches	Minimum Pile Penetration, ft		
	Areas 2 to 5	Areas 8 (B-176)	Areas 8 (B-183)
16-inch Steel Pipe Pile	-	38(1)	25
18-inch Steel Pipe Pile	32	38(1)	28
24-inch Steel Pipe Pile	37	-	-
24-inch Concrete Pile	-	-	32
36-inch Concrete Pile	-	-	42

Notes:
1. Minimum pile penetrations of these piles at B-176 location are controlled by earthquake conditions (i.e. liquefaction).

The above table gives the minimum pile penetration requirements based on our calculations of a second inflection point shown during our lateral capacity analyses. Additional penetration may be required to satisfy the axial capacity or other requirements. Individual, isolated driven piles and cast-in-place concrete piles are representative of a free-head condition. A group of piles structurally tied together with a rigid concrete cap is more closely representative of a fixed-head condition. On the basis of the final foundation design, the structural engineer should determine the most representative condition.

11.9 Lateral Group Effects

Lateral load capacity of a single isolated pile is generally based on acceptable lateral deflection. In general, for the same lateral deflection, the lateral load carrying capacity of a pile within a group will be less than that of an individual pile. Leading row piles generally experience less reduction in lateral capacity as compared to trailing row piles for the same head deflection criteria. The pile group arrangement and more importantly, the center-to-center spacing between adjacent piles have a significant impact on lateral group effects. Piles spaced greater than about 7 diameters, center-to-center, generally have limited group effects.

For lateral group analyses, we evaluated 2x1, 2×2, 4×4, 6×6, and 10×10 pile groups using the computer program Group 7.0 (2006) developed by Ensoft. We assumed in our analyses that piles within a pile group are rigidly connected to the pile cap (fixed-head condition). In our group analyses, we used the API Sand method to calculate p-y curves (soil resistance versus deflection) for granular material. The center-to-center pile spacing was varied from 2.5 D to 7 D, where D is the pile

[77] Liu, L. and Dobry, R., 1995, Effect of Liquefaction on Lateral Response of Piles by Centrifuge Model Tests., National Center for Earthquake Engineering Research (NCEER) Bulletin, 9(1), 7–11.

diameter. Reduction factor is dependent on the geometry of the lateral loading and pile group. It is the ratio of the lateral capacity of a pile group to the lateral capacity of the individual pile multiplied by the number of piles in that group and therefore it is an average value. Lateral group reduction factors presented below (Table 11.3) are the average reduction factors for different pile group sizes and varying center-to-center spacing. The lateral group effect can be neglected when the center-to-center spacing is equal to or greater than 7 times the diameter of the pile. Using the reduction factor presented in Table 11.3, the lateral capacity of a pile group can be computed as: Pile group lateral capacity = Single pile lateral capacity x Number of Piles x Reduction Factor.

Table 11.3: Recommended Reduction Factors for Lateral Group Effect

Pile Group Configuration	Center-to-Center Spacing			
	2.5 D	3 D	5 D	≥ 7 D
2 X 1 (Loading along Short Axis)	0.90	0.95	1.0	1.0
2 X 1 (Loading along Long Axis)	0.70	0.75	0.90	1.0
2 x 2	0.60	0.70	0.90	1.0
4 x 4	0.58	0.70	0.90	1.0
6 x 6	0.55	0.70	0.90	1.0
10 x 10	0.50	0.65	0.90	1.0

Notes:
1. D refers to individual pile diameter.
2. Lateral loads are applied at the top of pile group.

If group effects are expected to be critical in the design, we will be pleased to provide a detailed group effect analysis on a case-by-case basis for specific groups and loading conditions.

11.10 Load Tests

Based on the relatively uniform soil conditions encountered at the site and our experience, pile load tests are recommended if the number of piles to be installed at this site exceeds 250 total. We recommend that static load tests (if performed) be in general accordance with ASTM D1143-07[78] (Procedure A: Quick Test) and ASTM D3966-07[79] (Procedure A: Standard Loading) for compression and lateral loading, respectively. Load testing should include both static axial capacity testing and dynamic pile testing, depending on the types of piles selected for the final design. Dynamic pile testing should be performed on 2 to 5 percent of the production piles, in general accordance with ASTM D4945-12[80]. The advantage of performing a load test can be realized when the ultimate capacity of the tested pile exceeds the calculated static pile capacity. In this case, we can re-evaluate the soil parameters to correlate them with the pile load test results and re-calculate the ultimate axial

[78] ASTM D1143 / D1143M-07(2013), Standard Test Methods for Deep Foundations Under Static Axial Compressive Load, ASTM International, West Conshohocken, PA, 2013, www.astm.org
[79] ASTM D3966 / D3966M-07(2013), Standard Test Methods for Deep Foundations Under Lateral Load, ASTM International, West Conshohocken, PA, 2013, www.astm.org
[80] ASTM D4945-12, Standard Test Method for High-Strain Dynamic Testing of Deep Foundations, ASTM International, West Conshohocken, PA, 2012, www.astm.org

pile capacity. The increased ultimate axial capacity can result in shorter pile lengths, shorter pile installation times, and a potential cost savings. Load testing should be conducted in the presence of an experienced geotechnical engineer. In static load testing, each test pile should be loaded to produce enough movement at the top of the pile to determine the ultimate capacity or at least 3 times the design load. In dynamic load testing, we recommend that dynamic load testing be correlated with the static compression load testing. The wait period between end of initial driving and PDA restrike testing should therefore be similar to the wait period of static load testing. Alternatively, if the PDA restrike testing cannot be performed within the timeframe as recommended above, it could be performed during the beginning of the test pile program after waiting 30 minutes and again after waiting 24 to 48 hours from end of initial driving. A pile set-up analysis can be performed using PDA data from initial drive and two restrike tests to estimate the compression load capacity at the time of static compression load test[81]. We recommend that the Geotechnical Engineer of Record be retained to review the results of load tests to: 1) evaluate the load/unload-displacement response of the piles, 2) evaluate the ultimate capacity of test piles, and 3) compare measured capacities and deflections with design criteria.

11.11 Frost Considerations for Pile Caps & Grade Beams

Where piles are used to support equipment or structures, concrete pile caps or grade beams may be required on top of the piles. Where the pile caps and grade beams lie above the seasonal frost line, precautions may need to be taken to prevent heaving in frost susceptible soils, if present. A typical recommended construction schematic for accommodating frost heave, if any, under the pile caps and grade beams in unheated areas is shown on Plate C-64.

[81] Bullock, P.J., 2012, The Easy Button for Driven Pile Setup: Dynamic Testing, Research to Practice in Geotechnical Engineering Congress 2008, New Orleans, Louisiana, United States

12.0 BLUFF SLOPE STABILITY

12.1 General

The site is bounded on the west side by a coastal bluff that rises to about El.120 ft, ranging from El. +140 ft on the north to El. +100 feet to the south, with the toe of the bluff at an approximate elevation of El. +20 ft. Although purposely limited by engineering and layout considerations, several structures are planned along the bluff as shown on Plate 3a and 3b, therefore, the integrity and stability of the bluff is critical to the project. Preliminary layout indicates the following critical structures along the bluff:

- Area 1 plans are to cut the bluff to accommodate the Heavy Haul Road from the Material Offloading Facility (MOF) to the Plant area (Note: this is non-load-bearing and temporary in nature);
- Area 2 will include LNG Tanks and is planned to keep the existing bluff, and;
- Area 3 will include an outfall structure.

Previous work and field evidence collected during the 2014 and 2015 geologic field mapping (documented in AKLNG Document Nos. USAL-FG-GRZZZ-00-000001-000[82], USAL-FG-GRZZZ-00-002015-004[83], and USAL-FG-GRHAZ-00-002015-002[84]) indicate that the bluff is eroding by a combination of wave erosion, which undermines the toe of the slope, followed by landsliding, raveling, and gullying on the face of the bluff.

Estimates of the annual rate of coastal bluff retreat vary, and are based on long-term data. Kenai Peninsula Borough (KPB) estimated rates of coastal erosion in 100-meter (328-feet) segments along the west coast of the Kenai Peninsula (Kenai Peninsula Borough, 2013)[85,86]. Within the area of the planned onshore LNG facilities, estimated rates for most segments are 1 to 3 feet per year, with one segment estimated at around 5 feet per year. A comparison of aerial photography from 1980 and 2012 in the LNG onshore facilities area, presented in AKLNG Document No. USAL-FG-GRHAZ-00-

[82] Fugro Consultants, Inc. (Fugro), 2014, Geologic Mapping Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-2, AKLNG Document No. USAL-FG-GRZZZ-00-000001-000, Rev.0, dated November 18, 2014.
[83] Fugro Consultants, Inc. (Fugro), 2015, LNG Facilities Onshore Geologic Field Mapping Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-2, AKLNG Document No. USAL-FG-GRZZZ-00-002015-004, Rev.0, dated August 21, 2015.
[84] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Geologic Hazard Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-10, AKLNG Document No. USAL-FG-GRHAZ-00-002015-002, Rev.1, dated October 05, 2016.
[85] Atwater, B. F., Trumm, D. A., Tinsley, J. C., Stein, R. S., Tucker, A. B., Donahue, D. J., and Jull, A. J. T., and Payen, L. A., 1990, Alluvial plains and earthquake recurrence at the Coalinga anticline, *in* Rymer, M. J., and Ellsworth, W. L., eds., The Coalinga, California, Earthquake of May 2, 1983, U. S. Geological Survey Professional Paper 1497, p. 273-297.
[86] Rymer, M. J., Kendrick, K., J., Lienkaemper, J. L., and Clark, M. M., 1990, The Nuñez fault and its surface rupture during the Coalina earthquake sequence, *in* Rymer, M. J., and Ellsworth, W. L., eds., The Coalinga, California, Earthquake of May 2, 1983, U. S. Geological Survey Professional Paper 1497, p.299-318.

002015-002, yielded annual rates of up to 2 feet per year. Coastal bluff retreat is an episodic process, both spatially and temporally, and average rates do not reflect the magnitude of erosion in a single event.

To assess recent patterns of coastal erosion, 2012 KPB Lidar elevation data was compared to the recently obtained 2015 Fugro Lidar elevation data. The evaluation focused on mapping the face of the coastal bluff in the 2012 and the 2015 data sets to identify locations of discernible change in the bluff profile within the site perimeter as shown on Plate D-1 in Appendix D. GIS tools were used to evaluate the slopes along the bluff alignment within the project boundary and are shown Plates D-2a through D-2f in Appendix D. As shown on Plates D-2a through D-2f, the bluff face slopes generally vary between 35 to 50 degrees to horizontal, and during the 2014/2015 field geologic mapping, they were found to be mostly bare or covered with a thin veneer of grass and shrubs. Sloughing of the shallow rooted vegetation and the associated root mat was observed discontinuously along the coastal bluffs. Debris flows were apparent in several locations along the coast (Plates D-2a through D-2f). Shallow, rotational slumps were observed in a small number of locations along the coastal bluffs, and were often associated with higher observed rates of groundwater seepage at the contact between the Killey and the late Moosehorn deposits.

Based on our review of the slopes along the bluff, we believe that the slope of bluff will continue undergoing shallow slump failures. The slope cross sections were developed utilizing the 2015 Fugro Lidar Data as the source file and proposed cut elevations provided by AKLNG. Current plans from AKLNG include moving the LNG onshore facilities sufficiently inland (e.g., east) away from the bluff so as to reduce any long term bluff erosion impact. This will minimize the need for bluff stabilization and erosion mitigation measures.

A number of erosion mitigation methods were noted during field reconnaissance[87]. In the vicinity of the existing piers (e.g. Tesoro Nikiski, Kenai LNG, Agrium), erosion protection structures such as seawalls (consisting of steel sheet piles and gabion structures) were observed and appear to have been effective at significantly slowing the rates of erosion of the bluff.

The following sections discuss the slope configurations, method of analysis, and results of our stability evaluations for Areas 1 through 3.

12.1.1 Bluff Stability in Area 1 (Heavy Haul Road Area)

AKLNG provided Fugro with proposed cut and associated elevations to accommodate the Heavy Haul Road (HHR) from MOF (bottom of the bluff) to the plant (top of bluff). The proposed cut will result in various slope cross-sections with varying slope angles in localized areas. We selected 3 cross-sections as shown on Plate 20 for our analyses (HA1, HA2 and HA3). Slope and loading

[87] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Geologic Hazard Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-10, AKLNG Document No. USAL-FG-GRHAZ-00-002015-002, Rev.1, dated October 05, 2016.

configurations are presented on Plate D-3a. We assumed a maximum uniform loading of 1,200 psf spread across the road width to analyze critical scenarios. Both local and global slope stability was analyzed for static and dynamic conditions. Results of the conducted analyses are presented in *Section 12.5 Slope Stability Results*.

12.1.2 Bluff Stability in Area 2 (Storage Tank Area)

The proposed development and loads associated with the LNG Tank area may result in additional slope instability. The existing bluff has a slope of about 1.4H:1V with height of about 100 ft. Static and dynamic stability analyses have been performed for the LNG tanks by Fugro, since 2014. It should be noted that the analyses were performed based on the G&G data and the structural layout that were available during each G&G program. Below is a list of the structural layout drawings that were provided by AKLNG and used for the configurations of the slope stability models:

1. 2014 G&G program: Preliminary Overall Plot Plan Layout provided in package USAL-PL-CARFI-00-000002-00 (Appendix B).
2. 2015 G&G program: USAL-CB-LDLAY-00-000003-000 Rev.3, dated April 14, 2016.
3. 2016 G&G program: USAL-CB-LDLAY-00-000003-000 Rev.6, dated June 30, 2016.

Based on the 2014 G&G engineering analyses presented in AKLNG Document No. USAL-FG-GRZZZ-00-000006-000[88], it was recommended that the bluff slope to be regraded to 3H:1V and the distance of the tanks from the bluff (i.e. tank edge to the crest of the regraded slope) be at least 300 ft. However, the estimated factors of safety from the pseudo-static slope stability analyses were less than 1.0 for the representative cross-section with three slope configurations (i.e., As-is, 2H:1V and 3H:1V) and three different load conditions (i.e., no additional load, load corresponding to 5 ft grade raise, and load corresponding to 5 ft grade raise and LNG Tank loads). The seismic slope displacements were estimated using simplified empirical methods (i.e simplified co-seismic displacement estimates using the methods suggested by Bray and Travasarou (2007)[89] and Rathje and Antonakos (2011)[90] and dynamic displacement analysis based on Newmark (1965)[91] were performed for the 3H:1V slope configuration). These displacement values were considered approximate, due to the inherent limitations of the simplified models in simulating a complex dynamic phenomenon. Hence, more representative two dimensional dynamic slope stability analyses (i.e.

[88] Fugro Consultants, Inc. (Fugro), 2016, Integrated Site Characterization and Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-12, AKLNG Document No. USAL-FG-GRZZZ-00-000006-000, Rev.1, dated April 18, 2016.
[89] Bray J.D. and Travasarou T., 2007, Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, V. 133, No. 4, pp. 381-392.
[90] Rathje E.M. and Antonakos G., 2011, A Unified Model for Predicting Earthquake-induced Sliding Displacement of Rigid and Flexible Slopes, Engineering Geology, 122, pp 51-60.
[91] Newmark, N. M., 1965, Effects of Earthquakes on Dams and Embankments, Geotechnique, Vol. 15, No. 2, pp 139-160.

FLAC[92]) had been recommended in the AKLNG Document No. USAL-FG-GRZZZ-00-000006-000 to provide more reliable estimates of slope displacements during earthquake shaking.

More data was collected during the 2015 G&G program in order to refine the previously performed engineering analyses. Based on the new data, preliminary FLAC analyses were conducted using the 2014 G&G load configurations at tank offset distances of 200 ft and 250 ft from the crest of the existing bluff and regraded bluff slope 2H:1V (See Plate D-3b, Preliminary Cases). The results of these analyses are attached to the addendum document USAL-FG-GRZZZ-00-002015-018[93]. This addendum is also presented in Appendix E of this report.

Considering the results of the 2014 and 2015 G&G study and preliminary FLAC analyses described above, plans were to regrade the bluff slope to 3H:1V and set the tanks on top of the bluff offset at 272 ft from the crest of the regraded slope. However, based on the updated structural layout drawings listed above and further discussions with AKLNG on the analysis results, it was recommended that the bluff slope not to be regraded but to be left as-is. Therefore, analyses were performed maintaining the existing bluff as-is, with tanks on top of the bluff set at a minimum distance of 272 ft from the crest of the theoretically regraded slope of 3H:1V (i.e. 427 ft from the crest of existing slope) as shown on Plate D-3b (referred to as Case 1). Associated static, pseudo-static, and dynamic (i.e FLAC) results were presented in AKLNG Document No. USAL-FG-GRZZZ-00-002015-009[94].

Additionally, Fugro performed pseudo-static, and dynamic (i.e FLAC) analyses for steeper theoretically regraded slope (2.5H:1V). For these analyses the tanks on top of the bluff were set at a minimum distance of 220 ft from the theoretical slope of 2.5H:1V (i.e. 327 ft from the crest of existing slope) as shown on Plate D-3b (referred to as Case 2) and the results were submitted in the addendum USAL-FG-GRZZZ-00-002015-018 (Appendix E).

During the 2016 G&G program, more borings were performed within the footprints and in the close proximity of the LNG tanks. This report presents the results of the updated static and pseudo-static slope stability analyses performed keeping the existing bluff as is. The tanks were seton top of the bluff at distance of 300 ft from the crest of the theoretically regraded slope of 3H:1V (i.e. 464 ft from the crest of existing slope) based on the current drawings (USAL-CB-LDLAY-00-000003-000 Rev.6, dated June 30, 2016), which is more conservative with respect to loading and driving stresses than the FLAC models Case 1 and Case 2. The loading conditions used in our analyses are also provided on Plate D-3c.

[92] Itasca, 2011, FLAC User's Manual.
[93] Fugro Consultants, Inc. (Fugro), 2016, Addendum to 2015 LNG Facilities Onshore Integrated Site Characterization and Engineering Report , Alaska LNG Project, Nikiski, Alaska, AKLNG Document No. USAL-FG-GRZZZ-00-002015-018 , Rev.0, dated November 30, 2016.
[94] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-14, AKLNG Document No. USAL-FG-GRZZZ-00-002015-009, Rev.0, dated August 30, 2016.

Results of the conducted analyses are presented in *Section 12.5 Slope Stability Results*.

12.1.3 Bluff Stability in Area 3 (Outfall)

AKLNG has provided the location of the proposed outfall structure that will be located within Area 3. We selected one cross-section as shown on Plate 20 for our analyses. Slope and loading configuration is presented on Plate D-3d. Both local and global slope stability was analyzed for static and dynamic conditions. Results of the conducted analyses are presented in *Section 12.5 Slope Stability Results*.

12.2 Soil Parameters

12.2.1 Soil Parameters – Area 1 (Heavy Haul Road)

The subsurface conditions for slope stability in Area 1 (Heavy Haul Road) were based on the 6 soil borings located on top of the bluff and the beach borings and are summarized in table below.

Table 12.1: Soil Parameters for Slope Stability Analysis – Area 1 (HA1, HA2 and HA3)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight(1), pcf	Undrained Shear Strength, ksf	Friction Angle degrees
125 to 105	0 to 20	Silty Sand with Gravel	118	-	34
105 to 90	20 to 35	Silty Sand/Sand with Silt and Gravel	120	-	35
90 to 60	35 to 65	Silty Sand/Sand with Silt and Gravel	120	-	35
60 to 40	65 to 85	Silty Sand/Sand	60.6	-	36
40 to 10	85 to 115	Silty Sand/Sandy Silt/Clay	62.6	4.0 (0.3 (3))	23 (3)
10 to -80 (2)	115 to 205	Silty Sand/Sandy Silt	62.6	-	36
-80 to -160	205 to 285	Silty Sand/Sandy Silt/Clay	62.6	5.0 (0.3 (3))	23 (3)
-160 and below	285 and below	Silty Sand/ Sand with Silt and Gravel	72.6	-	38

Notes:
1. Groundwater table was estimated at 65 ft below final grade.
2. Three different profiles were used for slope stability analyses based on the section analyzed. The variations in soil profiles were below El.-25 ft.
3. Drained parameters for clay soils were used for long term slope stability analysis.

12.2.2 Soil Parameters – Area 2 (Storage Tank Area)

The subsurface conditions for slope stability in Area 2 (Storage Tank Area, Bluff and Beach) were based on the soil borings drilled and geophysical data at the top of the bluff and the beach. A total of 15 borings were performed on top of the bluff and a total of 9 borings were drilled in the beach

area during 2014 and 2015 G&G programs. The beach boring data indicates that the clay layers at about the same elevation in the beach area are less stiff than the clays in borings drilled on top of the bluff. Therefore, two profiles, one for top of the bluff and another for the beach area, were used in the slope stability analyses as shown on Plate 21e. The two profiles are presented in the following tables.

Table 12.2: Soil Parameters for Slope Stability Analysis – Area 2 (North Tank), DP2A-S (Bluff)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight, pcf	Undrained Shear Strength, ksf	Friction Angle degrees
121 to 100	0 to 21	Sand/Gravel	118	-	34
100 to 80	21 to 41	Sand/Gravel	120	-	35
80 to 60	41 to 61	Sand/Gravel	123	-	36
60 to 40	61 to 81	Clay	60.6	4 (0.3 (1))	(23 (1))
40 to -20	81 to 141	Silty Sand/ Sandy Silt	62.6	-	36
-20 to -185	141 to 306	Clay	62.6	6 - 12 (0.5 (1))	(27 (1))
-185 and below	306 and below	Sand w/ Gravel	72.6	-	38

Notes:
1. Drained parameters for clay soils were used for long term slope stability analysis.
2. Undrained parameters for clay soils were used for seismic slope stability analysis.

Table 12.3: Soil Parameters for Slope Stability Analysis – Area 2 (North Tank), DP2A-S (Beach)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight, pcf	Undrained Shear Strength, ksf	Friction Angle degrees
0 to -20	121 to 141	Silty Sand/ Sandy Silt	55.6	-	34
-20 to -55	141 to 176	Clay/Silt	62.6	3.5 (0.5 (1))	(20 (1))
-55 to -80	176 to 201	Silty Sand/ Sandy Silt	62.6	-	36
-80 to -135	201 to 256	Clay	62.6	4 (0.75 (1))	(28 (1))
-135 to -185	256 to 306	Clay	67.6	5 (0.75 (1))	(28 (1))
-185 and below	306 and below	Sand w/ Gravel	72.6	-	38

Notes:
1. Drained parameters for clay soils were used for long term slope stability analysis.
2. Undrained parameters for clay soils were used for seismic slope stability analysis.

12.2.3 Soil Parameters – Area 3 Outfall

The subsurface conditions for slope stability in Area 3 (Outfall) were based on the 4 soil borings located on top of the bluff and the beach borings and are summarized in table below.

Table 12.4: Soil Parameters for Slope Stability Analysis – Area 3, DP3-S (Bluff)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight, pcf	Undrained Shear Strength, ksf	Friction Angle degrees
112 to 100	0 to 12	Sand/Gravel	118	-	34
100 to 80	12 to 32	Sand/Gravel	120	-	35
80 to 70	32 to 42	Clay	123	4 (0.3 (1))	(23 (1))
70 to 50	42 to 62	Sand/Gravel	62.6	-	36
50 to 40	62 to 72	Clay	60.6	4 (0.3 (1))	(23 (1))
40 to -15	72 to 127	Silty Sand/ Sandy Silt	65.6	-	37
-15 to -185	127 to 297	Clay	65.6	5 - 12 (0.5 (1))	(27 (1))
-185 and below	297 and below	Sand w/ Gravel	72.6	-	38

Notes:
1. Drained parameters for clay soils were used for long term slope stability analysis.
2. Undrained parameters for clay soils were used for seismic slope stability analysis.

Table 12.5: Soil Parameters for Slope Stability Analysis – Area 3, DP3-S (Beach)

Elevation, ft (NAVD88)	Depth below Finished Grade, ft	Soil Type	Effective Unit Weight, pcf	Undrained Shear Strength, ksf	Friction Angle degrees
0 to -15	112 to 127	Silty Sand/ Sandy Silt	55.6	-	34
-15 to -55	127 to 167	Clay/Silt	62.6	3.5 (0.5 (1))	(20 (1))
-55 to -80	167 to 192	Silty Sand/ Sandy Silt	62.6	-	36
-80 to -135	192 to 247	Clay	62.6	4 (0.75 (1))	(28 (1))
-135 to -185	247 to 297	Clay	67.6	5 (0.75 (1))	(28 (1))
-185 and below	297 and below	Sand w/ Gravel	72.6	-	38

Notes:
1. Drained parameters for clay soils were used for long term slope stability analysis.
2. Undrained parameters for clay soils were used for seismic slope stability analysis.

12.3 Loading Conditions

Static and dynamic loading conditions were analyzed for this report. The following loading conditions were analyzed for our stability analyses:

Static Condition (Short-term Undrained) – The short-term, or undrained, case models the condition soon after construction in which the pore pressures generated during construction and operation have not dissipated. Analyses for this condition involve the use of undrained shear strength parameters as tabulated in the previous section.

Static Condition (Long-Term Drained) – The long-term, or drained, case models the condition in which the pore pressures generated during construction and operation have dissipated. Analyses for this condition involve the use of drained shear strength parameters as tabulated in the previous section.

Dynamic Condition – OBE and SSE level ground motions were applied to the slope cross-section to estimate slope displacements of the top of bluff and LNG tanks. Analyses for this condition involve the use of undrained shear strength parameters.

12.4 Method of Analysis

12.4.1 Static Analyses

Our global stability analyses were conducted with the aid of the computer program *Slide Version 6.0*[95]. The computer program *Slide* generates trial failure surfaces through a designed slope and evaluates the factor of safety for each trial failure surface. The program allows a large number and shape of potential shear surfaces to be investigated to determine the most critical failure surfaces for each of the analyzed slope configurations. We used *Slide* to evaluate the stability of the slopes using the Spencer method (Slide Version 6.0). This method uses two-dimensional limit equilibrium analysis to determine the factor of safety for the slope.

The computed factor of safety (FS) is the ratio of forces resisting failure to the forces driving failure. A factor of safety of 1.0 or less implies the slope is unstable, while a factor of safety greater than 1.0 implies the slope is stable. There is no universally accepted code or standard for acceptable slope stability factors of safety against slope failure. Guidelines for factor of safety can be found in publications such as USACE EM 1110-2-1913[96] and FERC (2007)[97]. Following are the factors of safety selected for this study:

For foundations static loading adjacent to or atop slopes, F.S. ≥1.5

For permanent or sustained static loading, F.S.≥1.4

For pseudo-static seismic loading, F.S. ≥1.0

12.4.2 Pseudo-Static Analyses and Dynamic Analyses

The simplified limit-equilibrium slope stability evaluation was performed using Slide (Rocscience, 2009), utilizing the idealized soil profiles presented in Section 12.2. The Spencer (1967) method was selected for the pseudo-static analysis evaluation. Results presented herein consider the same cases presented for the static slope stability evaluations.

[95] Slide – 2D limit equilibrium slope stability for soil and rock slopes. Rocscience Inc. Version 6.01.
[96] USACE. (2000). EM 1110-2-1913, Design and Construction of Levees, Washington, DC.
[97] Federal Energy Regulation Commission (FERC), 2007, Draft Seismic Design Guidelines and Data Submittal Requirements for LNG Facilities.

Additionally, as mentioned earlier, two dimensional, dynamic, total-stress, slope stability analyses were performed in time domain with FLAC (Itasca, 2011) to assess the seismic slope displacement hazard at the bluff (e.g., for Area 2 only).

12.5 Slope Stability Results

12.5.1 Static Slope Stability Results – Area 1 (Heavy Haul Road Area)

The bluff will be cut in Area 1 to accommodate the Heavy Haul Road (HHR) from MOF to the plant. The HHR is expected to be about 150 ft wide. We assumed a surcharge load of 1,200 psf from the Self Propelled Modular Transporter (SPMT)'s and other heavy haul road vehicles, based on past projects. We analyzed three sections as shown on Plates 21a through 21c. The summary of the global stability analysis performed on existing slopes in static conditions is provided in table below.

Table 12.6: Summary of Static Slope Stability Analysis (Heavy Haul Road Cut)

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
Section HA1-HA1'	short-term	2.53	1.5	D-4
Section HA1-HA1'	long-term	2.09	1.5	D-5
Section HA2-HA2'	short-term	2.70	1.5	D-6, 8
Section HA2-HA2'	long-term	2.64	1.5	D-7, 9
Section HA3-HA3'	short-term	2.13	1.5	D-10, 12
Section HA3-HA3'	long-term	1.65	1.5	D-11, 13

The slope stability of the planned cut to accommodate the HHR is greater than the required factor of safety at the three analyzed sections based on static slope stability analyses.

12.5.2 Pseudo-Static Slope Stability Results – Area 1 (Heavy Haul Road Area)

Pseudo-static analyses are performed based on FERC (2007) and Blake et al. (2002) [98] recommendations. These recommendations suggest an equivalent seismic coefficient k_{eq} is a function of: i) the maximum horizontal acceleration at the site for a soft rock site condition, and ii) a factor f_{eq} related to the seismicity of the site in terms of magnitude and distance. The maximum horizontal acceleration is provided in the Probabilistic Seismic Hazard Analysis Report for the project (AKLNG Document No. USAL-FG-GRHAZ-00-002015-001). The factor f_{eq} is provided in a figure form at Blake et al. (2002) as a function of magnitude and the maximum horizontal acceleration. For preliminary purposes, the horizontal response spectra developed at the ground surface from the PSHA report (AKLNG Document No. USAL-FG-GRHAZ-00-002015-001) were conservatively used as the input for the maximum horizontal acceleration at the soft rock site. We understand that the structures for this area fall within Seismic Category II and III per FERC (2007). These structures are

[98] Blake T.F., Hollingsworth R.A., Stewart J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center, USC.

designed based on the Design Earthquake (DE) which is defined as 2/3 of the Maximum Considered Earthquake (MCE) per ASCE 7-05. The lateral seismic coefficients k_{eq} used equals to 0.36g (i.e., 0.60 times the PGA for the Design Earthquake (DE) ground shaking level [AKLNG Document No. USAL-FG-GRHAZ-00-002015-001] for an earthquake scenario of magnitude 8.0 based on a seismic slope displacement threshold of 2 in [5 cm]). The summary of the global stability analysis performed on existing slopes in pseudo-static conditions is provided in Table 12.7. As noted, the calculated factors of safety are below the recommended values (Table 12.7). Given the HHR is a temporary structure, we recommend performing detailed analyses during the design stage of this project, if needed.

Table 12.7: Summary of Pseudo-Static Slope Stability Analysis (Heavy Haul Road Cut)

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
Section HA1-HA1'	Pseudo-Static	0.66	1.0	D-14
Section HA2-HA2'	Pseudo-Static	0.64	1.0	D-15, 16
Section HA3-HA3'	Pseudo-Static	0.66	1.0	D-17, 18

12.5.3 Static and Pseudo-Static Slope Stability Results – Area 2 (Storage Tank Area)

For static loading conditions, we assumed the following loading conditions: 1) surcharge load due to fill (500 psf) starting at an offset distance of about 179 ft from the crest of the bluff and 2) surcharge load due to tank (5760 psf) starting at an offset distance of about 464 ft from the crest of the bluff and extending a distance of about 366 ft inwards, i.e. away from the slope (diameter of the tank) based on the drawings shown on AKLNG Document No. USAL-CB-LDLAY-00-000003-000 Rev.6, dated June 30, 2016. The results of the static slope analysis are provided on Plates D-19 and D-20 in Appendix D. The results indicate that the calculated failure plane would not affect the proposed tank slab foundation and therefore, natural bluff slope could be left as-is (as described in earlier *Section 12.1.2 Bluff Stability in Area 2 (Storage Tank Area)*). Summary of static global stability analyses is provided in Table 12.8.

Table 12.8: Summary of Static Bluff Slope Stability Analysis

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
Tank offset by 427 ft from bluff crest	short-term	1.08	1.5	D-19
Tank offset by 427 ft from bluff crest	long-term	0.91	1.5	D-20

The results of the pseudo-static analysis for the OBE level are presented in Plate D-21. The depicted failure envelopes are those with safety factor (FS) less than one. Similar results for SSE level are presented in Plate D-22. Based on the failure surfaces of the pseudo-static analyses, the possible failure zones (i.e. with seismic slope displacements higher than 2 in [5 cm]) reach the examined tank offset location. Summary of pseudo-static global stability analyses is provided in Table 12.9. Since the pseudo-static factor of safety is less than 1.0 using the method outlined by Blake et al. (2002), dynamic displacement analyses are required according to FERC (2007).

Table 12.9: Summary of Pseudo-Static Bluff Slope Stability Analysis

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
Tank offset by 427 ft from bluff crest	OBE level	0.65	1.0	D-21
Tank offset by 427 ft from bluff crest	SSE level	0.41	1.0	D-22

12.5.4 Dynamic Slope Stability Results – Area 2 (Storage Tank Area)

Dynamic slope stability analyses were performed for one representative geotechnical cross-section at a critical location for the coastal bluffs of the Alaska LNG project. Analyses were performed using the finite difference software FLAC (Itasca, 2011) for the baseline case for:

(i) Two design levels (OBE and SSE levels per NFPA 59A 2006),

(ii) Five design ground motions (two horizontal components each set),

(iii) Including polarity effects,

(iv) The existing (1.4H:1V) slope configuration

(v) Two tank offset distances of 427 ft (Case 1) and 327 ft (Case 2) from the existing crest.

Case 1 and Case 2 FLAC results are presented for the estimated permanent displacements in terms of total displacement at the surface of the model relative to the model base as well as permanent relative displacement between various reference points of the tank. The term “absolute” is used to indicate the total or relative displacement at the surface of the model without sign (positive or negative direction of the model). Thus, the permanent extreme (largest or smallest) value is reported as the absolute, whether for total displacement or relative displacement between two reference points. Additional details are provided in Appendix E.

The estimated range of absolute total horizontal near surface displacements at tank offset distance from crest of the existing slope, as well as at quarter, half and total tank width location per tank offset distance are summarized in Table 12.10 for OBE level. The estimated range of absolute relative horizontal near surface displacements per quartile of the tank (1st quartile being the one towards the slope) per tank offset distance are summarized in Table 12.11 for OBE level. Similar results are tabulated in Table 12.12 and Table 12.13 for SSE level, respectively. See the schematic presented on Plate 44 of Appendix E for the definitions of quartile.

Table 12.10: Comparison of the Estimated Absolute Total Horizontal Surface Displacements for the OBE Level within the Tank Location

OBE Level								
Location (ft)	Minimum Displacement in ft (inch)		Maximum Displacement in ft (inch)		Median Displacement in ft (inch)		Mean Displacement in ft (inch)	
	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest
Edge of Tank	0.0 (0)	0.0 (0)	1.5 (18)	1.2 (15)	0.0 (1)	0.0 (1)	0.1 (2)	0.1 (2)
Quarter Tank Width	0.0 (0)	0.0 (0)	1.1 (14)	1.0 (12)	0.0 (1)	0.0 (1)	0.1 (1)	0.1 (1)
Half Tank Width	0.0 (0)	0.0 (0)	1.1 (14)	1.0 (12)	0.0 (1)	0.0 (1)	0.1 (1)	0.1 (1)
Total Tank Width	0.0 (0)	0.0 (0)	0.9 (11)	0.7 (8)	0.0 (1)	0.0 (1)	0.1 (1)	0.1 (1)

Table 12.11: Comparison of the Estimated Absolute Relative Horizontal Displacements at the Surface per Quartile for the OBE Level

OBE Level								
Location (ft)	Minimum Displacement in ft (inch)		Maximum Displacement in ft (inch)		Median Displacement in ft (inch)		Mean Displacement in ft (inch)	
	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest
1st Quartile (towards slope)	0.0 (0)	0.0 (0)	0.4 (5)	0.2 (3)	0.0 (0)	0.0 (0)	0.0 (0)	0.0 (0)
2nd Quartile	0.0 (0)	0.0 (0)	0.1 (1)	0.1 (1)	0.0 (0)	0.0 (0)	0.0 (0)	0.0 (0)
3rd Quartile	0.0 (0)	0.0 (0)	0.1 (1)	0.1 (1)	0.0 (0)	0.0 (0)	0.0 (0)	0.0 (0)
4th Quartile	0.0 (0)	0.0 (0)	0.0 (0)	0.3 (3)	0.0 (0)	0.0 (0)	0.0 (0)	0.0 (0)

Table 12.12: Comparison of the Estimated Absolute Total Horizontal Surface Displacements for the SSE Level within the Tank Location

SSE Level								
Location (ft)	Minimum Displacement in ft (inch)		Maximum Displacement in ft (inch)		Median Displacement in ft (inch)		Mean Displacement in ft (inch)	
	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest
Edge of Tank	0.0 (0)	0.0 (0)	5.7 (68)	5.1 (61)	0.6 (7)	0.8 (10)	1.1 (14)	1.1 (14)
Quarter Tank Width	0.0 (0)	0.0 (0)	5.5 (65)	5.0 (60)	0.3 (3)	0.4 (5)	1.1 (13)	0.9 (11)
Half Tank Width	0.0 (0)	0.1 (1)	5.2 (63)	4.9 (58)	0.3 (3)	0.3 (4)	1.0 (12)	0.9 (11)
Total Tank Width	0.0 (0)	0.0 (0)	5.2 (63)	4.2 (50)	0.2 (2)	0.2 (3)	0.9 (11)	0.8 (10)

Table 12.13: Comparison of the Estimated Absolute Relative Horizontal Displacements at the Surface per Quartile for the SSE Level

SSE Level								
Location (ft)	Minimum Displacement in ft (inch)		Maximum Displacement in ft (inch)		Median Displacement in ft (inch)		Mean Displacement in ft (inch)	
	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest	427ft Tank Offset Distance from Crest	327ft Tank Offset Distance from Crest
1st Quartile (towards slope)	0.0 (0)	0.0 (0)	0.8 (10)	1.1 (13)	0.1 (1)	0.1 (1)	0.2 (2)	0.2 (3)
2nd Quartile	0.0 (0)	0.0 (0)	0.5 (6)	1.2 (14)	0.0 (1)	0.1 (1)	0.1 (1)	0.1 (2)
3rd Quartile	0.0 (0)	0.0 (0)	0.4 (5)	0.9 (11)	0.0 (0)	0.0 (0)	0.1 (1)	0.1 (2)
4th Quartile	0.0 (0)	0.0 (0)	1.0 (12)	0.6 (7)	0.0 (0)	0.0 (0)	0.1 (2)	0.1 (2)

We understand that the above tabulated horizontal displacement values will be used by the tank designer during the final design stages. Based on the results, the estimated near surface permanent absolute horizontal displacements at the tank location when the tank is located 427 ft from the crest for the 1.4H:1V slope analyzed at OBE level range between 0 to 18 inches with an average absolute horizontal displacement of approximately 2 inches and a median of 1 inch. This range increases at the SSE level to about 0 to 68 inches with an average absolute horizontal displacement of approximately 14 inches and a median of 7 inches. These values reduce at the quarter, half and total tank locations. Detailed information on two dimensional dynamic slope stability analyses, results and conclusions are provided in Appendix E.

12.5.5 Static Slope Stability Results – Area 3 (Outfall)

For static loading conditions, we assumed a surcharge load due to fill (500 psf) starting at an offset distance of about 197 ft from the crest of the existing bluff. We analyzed one case shown on Plate 21h. The summary of the global stability analysis performed on existing and cut slopes in static conditions is provided below, in Table 12.14.

Table 12.14: Summary of Static Slope Stability Analysis (Outfall)

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
1.3H:1V (existing)	short-term	1.63	1.5	D-23
1.3H:1V (existing)	long-term	1.07	1.4	D-24
2H:1V (cut)	short-term	2.05	1.5	D-25
2H:1V (cut)	long-term	1.30	1.4	D-26
2.5H:1V (cut)	short-term	2.37	1.5	D-27
2.5H:1V (cut)	long-term	1.46	1.4	D-28

The slope stability for 2.5H:1V cut for outfall in Area 3 is greater than the required factor of safety based on static slope stability analyses.

12.5.6 Pseudo-Static Slope Stability Results – Area 3 (Outfall)

Pseudo-static analyses are performed based on FERC (2007) and Blake et al. (2002) recommendations. More details can be found in *Section 12.5.2 Pseudo-Static Slope Stability Results – Area 1 (Heavy Haul Road Area).* The summary of the global stability analyses performed on existing and cut slopes in pseudo-static conditions is provided in Table 12.15. As the calculated factors of safety are below the recommended values (Table 12.15), we understand that detailed analyses will be performed during the design stage of this project.

Table 12.15: Summary of Pseudo-Static Slope Stability Analysis (Outfall)

CASE	Loading Conditions	Calculated FS	Recommended FS	Plate No.
1.3H:1V (existing)	Pseudo-Static	0.82	1.0	D-29
2H:1V (cut)	Pseudo-Static	0.84	1.0	D-30
2.5H:1V (cut)	Pseudo-Static	0.85	1.0	D-31

12.6 Slope Protection

The bluff is constantly eroding and is steep at certain locations (greater than 45° slope angle) that will likely result in shallow surficial failure surfaces. Proper slope protection would need to be provided to prevent the continued development of shallow surficial failures.

Shallow surface sloughing along the slope faces should be expected for each slope configuration due to natural processes, and some form of mitigation or maintenance would be required over the life of the structures to arrest the natural processes. As noted above in *Section 12.1 General*, existing protection measures are in service and appear to be effective in reducing erosion. Exposed slopes could be protected from erosion by diverting surface runoff using berms or channels. Where erosion due to wind or water is a concern, the slopes could be protected with shotcrete, asphalt, soil cement slurry, geocells, gabions, Reno mattresses, sheetpile, riprap or a layer of geomat in addition to diverting the surface water.

13.0 BELOW-GRADE STRUCTURES

13.1 General

We understand that below-grade structures such as sumps, basins and some shallow retaining structures may be planned as a part of the proposed AKLNG project. At this time, we expect the below-grade structures will be located within the top 20 feet below final grade, and will likely encounter granular soils. Therefore, this section presents our recommendations for the proposed below-grade structures, with embedment of less than 20 ft, including lateral earth pressures, excavation considerations, groundwater control, and buoyancy considerations. We do not expect groundwater seepage into excavations within the top 20 feet. Groundwater, if encountered, should be maintained below the bottom of the excavation using lateral ditches, sumps and pumps. It should be noted that excavations deeper than 20 ft below final grades need to be evaluated separately and specifically designed on a case-by-case basis.

13.2 Lateral Earth Pressures

The proposed below-grade structures will experience lateral earth pressures resulting from a combination of soil pressure and any surcharge loads. The groundwater table at the proposed project site is deeper than 20 feet, and we do not expect hydrostatic pressures acting on the below-grade structures, provided water is prevented from entering behind the wall. Our recommended soil parameters for the design of the below-grade structures for Areas 1 to 6 are summarized in Table 13.1.

Table 13.1: Lateral Earth Pressure Parameters – Areas 1 to 6

Areas	Soil Type	Depth	Total Unit Weight, pcf	Soil Shear Strength Parameters		Earth Pressure Coefficients		
				Cohesion (c), psf	Friction Angle (ϕ)(1), degrees	K_0(2)	K_a(3)	K_p(4)
1 to 6	NFS/natural granular soils	0-20	118	0	34	0.44	0.28	3.54

Note:
1. ϕ = design friction angle,
2. K_0 = coefficient of earth pressure at-rest
3. K_a = coefficient of active earth pressure
4. K_p = coefficient of passive earth pressure

Soil pressures behind the walls will impose a triangular stress distribution, while surcharge loads will impose a rectangular stress distribution. Lateral earth pressures resulting from surcharge loads may be calculated using the at-rest earth pressure coefficient presented in Table 13.1. Coefficient of earth pressure at rest should be used to compute pressure on rigid walls from retained soil. The coefficient of active earth pressure should be used to compute pressures on flexible walls from

retained soil. Coefficient of passive earth pressure can be used to compute the soil pressures on the resisting side. A factor of safety of at least 2 should be used in computing allowable passive earth pressures.

13.3 Excavation Considerations

Excavations for below-grade structures should be performed in accordance with the recommendations presented in *Section 17.2 Shallow Open-Cut Excavations* and should not intrude into the stress influence zone of foundations (i.e., 3V:1H stress distribution extending below the foundation) for proposed structures. If proposed below-grade structures are located in close proximity to the proposed foundation locations, use of temporary construction sheeting may be required. For design of the stormwater and wastewater basins with soil slopes, we recommend permanent side slopes no steeper than 3-horizontal to 1-vertical. The permanent side slopes should be evaluated on a case-by-case basis if surcharge loads are expected.

13.4 Backfill

Backfill behind reinforced concrete walls for below-grade structures should consist of non-frost susceptible fill to at least 7 ft below the proposed final grade. Below 7 ft, the fill may consist of granular fill. We recommend that non-frost susceptible fill soils behind the external walls of the structures should extend for a horizontal distance of at least 7 ft from the walls. Backfill should be selected and placed in accordance with the recommendations presented in *Section 17.3 Fill Materials and Placement*. Compacted backfill adjacent to the below-grade structure walls (within a distance equal to about 3.5 ft) should be placed using hand-operated compaction equipment and 6-inch thick loose lifts. Care should be taken to not over-compact the backfill behind walls.

14.0 DYNAMIC SOIL PROPERTIES

14.1 General

Foundations for vibrating machines should be designed for dynamic response using foundation information along with dynamic soil properties. For this purpose, a total of 35 (8 in 2014, 18 in 2015 and 9 in 2016) downhole tests were performed for this study and the locations are shown on Plate F-1 in Appendix F.

Additionally, the Interferometric Multi-Channel Analysis of Surface Waves (IMASW) method was used to collect shallow seismic data along 14 semi-continuous lines totaling 55,600 feet of data acquisition, and resulting in 47,391 feet of processed output length. The data were collected in three separate campaigns, September – November 2014, a trial survey in May-June of 2015, and a production survey in July of 2015. The data collection, data processing, and results of the 2014 shallow seismic surveys were presented in AKLNG Document No. USAL-FG-GRZZZ-00-000002-000[99] and results of the 2015 shallow seismic surveys were presented in AKLNG Document No. USAL-FG-GRZZZ-00-002015-005[100]. Shear wave velocities derived from downhole tests were used for developing the dynamic soil properties as the IMASW method provides velocity that is the average across the length of the geophone spread.

The measured shear wave (S-wave) velocities from the downhole tests together for each boring are presented on Plates F-2a through F-2c in Appendix F of this report. Shear wave velocity profiles for each area are also plotted on Plates F-3a through F-3d. The sections below provide our discussions on calculated dynamic shear modulus, material damping and Poisson's ratio, and dynamic spring constants based on the field downhole tests.

14.2 Dynamic Shear Modulus

The shear modulus of soils under dynamic loads is affected by several factors. The most important variables that affect soil response are: 1) amplitude of dynamic strain, 2) mean effective stress, 3) void ratio, 4) grain size characteristics and soil structure, 5) stress history, 6) frequency of vibration, and 7) degree of saturation.

The dynamic shear modulus can be computed using the following equation:

$$G_{max} = \gamma_t (V_s)^2 / (144)\, g;$$

where, G_{max} = dynamic shear modulus, ksi;

[99] Fugro Consultants, Inc. (Fugro), 2015, Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140094-7, AKLNG Document No. USAL-FG-GRZZZ-00-000002-000, Rev.0, dated April 20, 2015.
[100] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geophysical Survey Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-7, AKLNG Document No. USAL-FG-GRZZZ-00-002015-005, Rev.0, dated June 10, 2016.

V_s = shear wave velocity, ft/sec;

γ_t = total unit weight of soil, kcf; and

g = acceleration due to gravity, 32.2 ft/sec^2

The dynamic shear modulus should be used in dynamic analyses where the shear strain amplitudes are less than 10^{-3} percent. Reductions to the shear moduli should be applied for values of shear strain greater than 10^{-3} percent. The shear modulus varies non-linearly with displacement amplitude for large displacement amplitudes. The magnitude of reduction for the shear modulus for high displacement amplitudes cannot be determined without performing additional laboratory testing such as resonant column tests.

14.3 Material Damping and Poisson's Ratio

Material damping ratios may be assumed to range from 0.02 to 0.04 for silt and sand (Arya et al., 1979) [101]. Material damping ratios for clay may be assumed to range from 0.02 to 0.05 (Arya et al., 1979). For shear strain amplitudes greater than about 10^{-3} percent, increases to the material-damping ratio may be applied. Typical Poisson's ratios can be taken as 0.3 to 0.4 for sands and silts, and 0.35 to 0.5 for partially saturated and saturated clays. The following properties may be used for properly compacted non-frost susceptible structural fill (*Section 17.3.1 Structural Fill*).

Table 14.1: Dynamic Soil Properties – Non-Frost Susceptible Granular Structural Fill Soils

Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
New Fill	300 to 900	2.3 to 20.6	0.02 to 0.04	0.3 to 0.4

Based on the soil borings, downhole test results, and our literature review, we recommend the following range of dynamic soil properties for Areas 2 to 5.

Table 14.2: Dynamic Soil Properties – Area 2

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
125 to 120	0 to 5	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	150 to 450	0.6 to 5.2	0.02 to 0.04	0.3 to 0.4
120 to 100	5 to 25	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	400 to 1300	4.0 to 43.0	0.02 to 0.04	0.3 to 0.4

[101] Arya, S., O'Neill, M., and Pincus, G. (1979). Design of Structures and Foundations for Vibrating Machines, Gulf Publishing Company, Houston, Texas.

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
100 to 80	25 to 45	Dense to Very Dense Gravelly Sand / Sand with Silt	800 to 1400	16.6 to 50.7	0.02 to 0.04	0.3 to 0.4
80 to 25	45 to 100	Very Dense Silty Sand/ Sand/Clayey Sand, Very Stiff to Hard Clay	900 to 1500	21.5 to 59.7	0.02 to 0.05	0.35 to 0.5

Table 14.3: Dynamic Soil Properties – Area 3

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
116 to 100	0 to 16	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	200 to 500	1.0 to 6.4	0.02 to 0.04	0.3 to 0.4
100 to 85	16 to 31	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	650 to 1150	10.9 to 34.2	0.02 to 0.04	0.3 to 0.4
85 to 55	31 to 61	Dense to Very Dense Gravelly Sand / Sand with Silt	850 to 1300	19.3 to 45.2	0.02 to 0.04	0.3 to 0.4
55 to 16	61 to 100	Very Dense Silty Sand/ Sand/Clayey Sand, Very Stiff to Hard Clay	900 to 1300	22.4 to 46.7	0.02 to 0.05	0.35 to 0.5

Table 14.4: Dynamic Soil Properties – Area 4

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
125 to 115	0 to 10	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	150 to 500	0.6 to 6.4	0.02 to 0.04	0.3 to 0.4
115 to 105	10 to 20	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	300 to 1100	2.3 to 30.8	0.02 to 0.04	0.3 to 0.4
105 to 90	20 to 35	Medium Dense to Very Dense Gravelly Sand / Sand with Silt	400 to 1200	4.1 to 37.3	0.02 to 0.04	0.3 to 0.4
90 to 75	35 to 50	Dense to Very Dense Gravelly Sand / Sand with Silt	700 to 1300	13.2 to 45.6	0.02 to 0.04	0.3 to 0.4

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
75 to 55	50 to 70	Dense to Very Dense Gravelly Sand / Sand with Silt	900 to 1300	22.4 to 46.7	0.02 to 0.04	0.3 to 0.4
55 to 45	70 to 80	Very Dense Silty Sand/ Sand/Clayey Sand, Very Stiff to Hard Clay	900 to 1600	22.4 to 70.7	0.02 to 0.05	0.3 to 0.5
45 to 25	80 to 100	Very Dense Silty Sand/ Sand/Clayey Sand, Very Stiff to Hard Clay	1000 to 1350	27.6 to 50.3	0.02 to 0.05	0.3 to 0.5

Table 14.5: Dynamic Soil Properties – Area 5

Elevation, ft (NAVD88)	Depth Below Finished Grade, ft	Soil Description	Shear Wave Velocity, ft/sec	Shear Modulus, ksi	Damping Ratio	Poisson's Ratio
116 to 105	0 to 11	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	250 to 700	1.6 to 12.5	0.02 to 0.04	0.3 to 0.4
105 to 85	11 to 31	Medium Dense to Very Dense Gravelly Sands and Sands with Silt	250 to 1250	1.6 to 40.8	0.02 to 0.04	0.3 to 0.4
85 to 50	31 to 66	Dense to Very Dense Gravelly Sand / Sand with Silt	700 to 1150	13.2 to 35.7	0.02 to 0.04	0.3 to 0.4
50 to 16	66 to 100	Very Dense Silty Sand/ Sand/Clayey Sand, Very Stiff to Hard Clay	800 to 1300	17.7 to 46.7	0.02 to 0.05	0.3 to 0.5

14.4 Modulus of Elasticity

The modulus of elasticity, E, at low strain values may be computed using the following equation:

$$E = 2G(\nu + 1);$$

where, G = dynamic shear modulus; and

ν = Poisson's ratio

14.5 Dynamic Spring Constants

Dynamic spring constants are highly dependent on the *in-situ* dynamic soil properties as well as the physical properties of the foundation element. Dynamic response of foundations is also dependent

on the configuration of the foundation system, *i.e.,* the dynamic response of an individual foundation element may vary greatly from that of a group of foundation elements. Typically, we evaluate dynamic spring constants using frequency independent methods after detailed information on foundation loads and design has been provided.

15.0 PAVEMENT RECOMMENDATIONS

15.1 General

We understand that the proposed AKLNG Project development within different areas will include construction of paved primary (main) and secondary roads, access ways, service roads, heavy haul roads and rigid pavement sections. AKLNG requested Fugro to provide flexible and rigid pavement geotechnical recommendations for the various roads.

The roads are expected to carry moderate to heavy traffic loads in accordance with HS20 loading conditions. Our review of provided road alignments indicated that the proposed roads may be located in both cut and fill areas of the project site. Therefore, the detailed design of pavements for this project should consider possible settlement/deflections due to traffic and the load of the fill material used to raise site grade. It is anticipated that the settlements due to grade raise will be completed by the time the roadways are surfaced. The site grade raise settlement estimates are provided in *Section 6.0 Settlement Due to Site Grade Raise*. We assumed in our pavement recommendations that grading activities in any road section across the site will be performed following the recommendations provided in *Section 4.0 Site Preparation* of this report.

We recommend that consideration be given for delaying the installation of paved roads until general construction for this project is well advanced or nearly completed. After grading and preparing the subgrade, a predetermined thickness of sub-base should be placed and coated with a layer of stabilizing agent to protect the surface and reduce dusting. Periodic maintenance of this layer must be performed during construction to protect the underlying subgrade. Following the final stages of construction, the aggregate sub-base should be properly graded and dressed (cleaned) before receiving final treatment. The upper few inches may be replaced or the thickness increased and the final surface pavement constructed.

In addition to the proposed roads, current plans are to construct a Heavy Haul Road (HHR) to transport heavy equipment or modules. Based on the preliminary information, we understand that the proposed HHR will be constructed either in Area 1 or Area 8 (Plates 3a and 3b). The details about the transporter, size, and loading conditions were not available at the time of this report, therefore, general construction recommendations are provided at this time.

This section discusses preliminary flexible and rigid pavement recommendations within the proposed cut and fill areas.

15.2 Site Preparation

The performance of the pavement ultimately depends on the subgrade. For any pavements planned within the proposed fill areas, we recommend that the subgrade for the roadways should consist of

non-frost susceptible granular fill. Areas that are located at higher elevations will be cut to match the proposed final grades. The subgrade within the proposed cut areas predominantly consists of existing sands and gravelly sands. Based on the laboratory test results, the existing subgrade soils at the project site are low to non-frost susceptible. Any areas identified as wet/weak or frost susceptible should be over-excavated and replaced with properly compacted non-frost susceptible granular fill. Our recommendations for gradation, placement, and compaction of on-site material that will be used as fill material are presented in *Section 17.3 Fill Material and Placement* of this report.

15.3 Subgrade

The proposed grade raise at this site could be on the order of 20 ft in localized areas. The proposed cut at this site will be on the order of 20 ft in localized areas, except the HHR area. The subgrade is expected to consist of low to non-frost susceptible soils overlying the medium dense to dense soils or well compacted granular fill soils. CBR testing was performed on several samples and presented in AKLNG Document No. USAL-FG-GRZZZ-00-002015-006[102]. Specific traffic type and frequency was not provided at the time of this report. Typical pavement sections for various primary roads, secondary roads, and access ways are presented in following section.

15.4 Pavement Sections

A detailed pavement design based on specific traffic and design conditions (*e.g.* design life, loads, frequency, etc.) was beyond the scope of this study. The following table provides typical pavement sections based on our experience with similar projects and subsurface conditions. These sections are based on typical loading conditions and pavement life expectancy. We do not recommend asphaltic concrete pavements in heavy truck areas because of the potential for shoving and rutting. Additionally, we have provided typical recommendations for heavy haul roads. Table 15.1 provides typical pavement sections based on our experience with similar projects and subsurface conditions.

Table 15.1: Typical Pavement Sections

Pavement Type and Recommended Use	Material	Thickness, inches	DOT&PF Reference or Specification
Unbound Gravel Roads (temporary construction roads, heavy haul roads)	Crushed Aggregate Base over	8	Grading C-1 or D-1
	Geogrid (1)	-	Tensar BX-1200 or equivalent
	Prepared Subgrade	18	Non-frost susceptible soils

[102] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Onshore Geotechnical Data Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-8, AKLNG Document No. USAL-FG-GRZZZ-00-002015-006, Rev.1, dated June 10, 2016.

Pavement Type and Recommended Use	Material	Thickness, inches	DOT&PF Reference or Specification
Flexible Pavement Primary & Secondary Roads (100,000 to 1,000,000 ESALs(2))	Hot-Mix Asphaltic Concrete over	3	Type II, Class A STE-1 Asphalt Tack
	Crushed Aggregate Base over	6	Grading C-1 or D-1
	Geogrid (1)	-	Tensar BX-1200 or equivalent
	Prepared Subgrade	18	Non-frost susceptible soils
Flexible Pavement Access Ways (Low Volume – less than 100,000 ESALs	Hot-Mix Asphaltic Concrete over	2	Type II, Class A STE-1 Asphalt Tack
	Crushed Aggregate Base over	6	Grading C-1 or D-1
	Geogrid (1)	-	Tensar BX-1200 or equivalent
	Prepared Subgrade	18	Non-frost susceptible soils
Rigid Pavement (primary roads, light-duty parking/traffic, area paving, and road shoulders)	Portland Cement Concrete (Flexural strength ≥ 550 psi) over	6	ACI
	Crushed Aggregate Base over	6	Grading C-1 or D-1
	Prepared Subgrade	18	Non-frost susceptible soils

Notes:
1. Utilization of geogrid will provide better performance of the subgrade resulting in less maintenance.
2. ESAL = Equivalent Single Axle Load

Select Base Course and Sub-base course. The crushed base course and sub-base course should be in accordance with DOT&PF Standard Specification for Highway Construction[103] Sections 301 through 308.

Asphalt. Hot mix asphaltic concrete (HMAC) should be placed in accordance with DOT&PF Standard Specification for Highway Construction Sections 401 and 702.

Rigid Pavement. The concrete should have a compressive strength of 4,000 psi or greater at 28 days (at least 550 psi flexural strength) and be placed in accordance with American Concrete Institute (ACI) guidelines[104]. Reinforcement and joint spacing for the concrete section should be evaluated by the Structural Engineer of Record for the project. Actual traffic conditions may warrant

[103] Alaska Department of Transportation and Public Facilities (DOT&PF), 2015, Standard Specification for Highway Construction, http://www.dot.state.ak.us
[104] ACI 325.9R, Guide for Construction of Concrete Pavements, American Concrete Institute, 2015, https://www.concrete.org

a higher concrete compressive strength. We recommend that the Portland Cement Concrete pavement be steel reinforced, Plain Jointed Concrete Pavement (PJCP) and/or roller compacted such that that the concrete slabs have sufficient joints to allow for contraction and expansion of the concrete.

15.5 Drainage

The importance of drainage to the proper operation and function of any pavement cannot be overemphasized. The pavement and subgrade surface should be raised above adjacent grade and properly sloped into drainage inlets or lateral ditches. Water should not be allowed to pond on or adjacent to the pavement whereby the subgrade may become saturated. If the pavement sublayers do become saturated, the bearing capacity will be greatly reduced and the useful life of the pavement will be decreased. Periodic inspections and repair of joints and cracks in pavement sections should be performed as part of facility maintenance.

15.6 Heavy Haul Road

Based on the layout of the HHR (Plates 3a and 3b) and site grading operations (Plate 16 and Plate 17a), we understand that majority of the HHR section located in Area 1 will be located in the cut area. The proposed cut will generally expose competent soils. Normal site preparations provided in *Section 4.0 Site Preparation* will be required in the cut areas. We understand that HHR may also be planned at Area 8 as an alternative and the existing gravel road will be resurfaced (or graded) if needed and used as the alternative HHR. Grading plans for Area 8 were not available at the time of this report. Generally, the stability of the slopes of the haul road is a major concern particularly when the haul road is located within the fill areas and loads are applied close to the edge of the bluff or a slope. Our slope stability recommendations for HHR in Area 1 are provided under Section 12.0 Bluff Slope Stability. However, as no grade change is planned for Area 2, we did not perform any stability analyses. High pressures that develop beneath the tires located away from the edge are assumed to be critical for potential localized bearing capacity failures. Therefore, we recommend performing bearing capacity analyses for a maximum applied pressure beneath the tires for the proposed haul road sections once the details on the transporter are available.

Haul roads require base and subbase sections to provide resistance to rutting, prevent bearing capacity failure, and slope stability failures. This section provides our general recommendations that will assist with the design of the proposed heavy haul roads for both cut and fill areas of the project.

Specific construction considerations and recommendations for the proposed haul roads are provided in the following:

- The exposed subgrade should be proof-rolled, where feasible, with a heavy (20 ton or more) rubber tired vehicle to expose loose or weak zones. At least three passes should be made during proof-rolling. Loose or weak zones indicated during proof-rolling should be over-

excavated and backfilled with well compacted crushed stone. Areas of the subgrade in which pumping or significant deflections are observed should also be removed. If significant rutting or local failures are noticed during proof testing, an additional layer of at least 6 inches of crushed stone should be placed on top of the proposed layer of crushed stone. The exposed subgrade should then be brought back to design grade using the structural fill (*Section 17.3.1 Structural Fill*).

- Our recommendations are based on the assumption that the materials used for the embankment, sub-base and base are relatively uniform in properties and composition through the entire road. Tires may sink 1 to 3 inches in some areas and localized areas experiencing more than 3 inches of rutting must be backfilled with compacted crushed stone prior to allowing any further traffic. It should be understood that localized repairs /maintenance, especially after transportation of each module or piece of equipment, may be required.
- We cannot over emphasize the need for strict quality control / quality assurance testing during construction. We recommend full-time observation and testing by a geotechnical engineer during construction.
- The effects of planned heavy loading on existing underground utilities, drains, or pipelines where applicable should also be evaluated before transporting heavy modules.
- Water should not be allowed to pond on or adjacent to the haul road whereby the subgrade may become saturated. If the subgrade soils do become saturated, the bearing capacity can be greatly reduced, increasing the risk of severe rutting and failure of the road section
- The crushed stone aggregate should be sufficiently sound and hard so that they do not break down in the compaction processes or during the transportation of the heavy equipment.

16.0 SOIL CORROSION POTENTIAL

16.1 General

Steel and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried steel and concrete elements should be designed to resist corrosion and degradation based on accepted practices. Soil pH, resistivity, and chloride and sulfate ion concentration tests were performed on a total of 50 samples between the 2014 and 2016 studies. Sample test results for 2014, 2015, and 2016 G&G programs are separately presented in this report. These tests were performed on samples from varying depths to assess the corrosion potential on buried steel and concrete. Our laboratory test results are discussed in the following sections. The potential for corrosion of buried steel and degradation of buried concrete based on a comparison of the laboratory test results with published guidelines is also discussed. The guidelines used to evaluate corrosion and degradation potential is also presented.

16.2 Laboratory Test Results

The results of our laboratory corrosion potential testing are summarized as follows.

Table 16.1: Summary of Corrosion Potential Testing – 2014 G&G program

Boring	Depth (ft)	Material Description	pH	Resistivity (ohm-cm)	Ion Concentration, ppm	
					Chloride (Cl^-)	Sulfate (SO_4^-)
B-11	12	Sand w/gravel	6.8	26,325	<100	<100
B-15	6.0	Sand w/gravel	6.5	21,900	<100	<100
B-17	63.3	Sand w/silt	7.1	6,200	<100	128
B-26	61.0	Sand w/silt	7	5,457	<100	166
B-27	51.5	Sand w/silt & gravel	5.6	4,763	<100	276
B-38	3.5	Sand w/silt & gravel	5.7	12,000	<100	190
B-38	66.5	Sand w/silt	7.3	3,434	<100	235
B-39	4.0	Sand w/silt & gravel	5.5	11,770	<100	169
B-39	81.5	Lean Clay	7.2	3,701	<100	145
B-40	2.5	Sand w/silt	5.9	43,870	<100	<100
B-40	71.5	Sand w/silt and gravel	6.4	4,664	<100	132
B-43	6.0	Sand w/silt and gravel	6.2	32,475	<100	<100
B-45	51.5	Sand w/silt and gravel	6.8	5,534	<100	146
B-50	6.0	Sand w/silt and gravel	5.8	16,125	<100	112
B-50	91.5	Sand w/silt	7.3	4,367	<100	104
B-10	71.3	Silt	8.2	4,200	<100	<100
B-18	66.8	Sand w/silt & gravel	7.8	5,858	<100	189
B-32	81.5	Sand w/silt	8.6	4,686	<100	159
B-42	3.5	Sand w/silt & gravel	5.9	34,320	<100	<100
B-42	66.5	Silty Sand	5.4	2,802	<100	370
B-47	61.0	Sand w/silt	6.6	9,309	<100	113

Table 16.2: Summary of Corrosion Potential Testing – 2015 G&G program

Boring	Depth (ft)	Material Description	pH	Resistivity (ohm-cm)	Ion Concentration, ppm	
					Chloride (Cl^-)	Sulfate (SO_4^-)
B-69	10	Sand w/silt & gravel	5.6	23,595	<100	<100
B-72	1	Gravel w/silt & sand	5.8	46,515	<100	<100
B-75	6.5	Sand w/silt & gravel	6.2	53,625	<100	128
B-87	4	Sand w/silt & gravel	5.2	33,255	<100	<100
B-93	11	Sand w/silt & gravel	6.3	16,613	<100	<100
B-95	2.5	Silty sand	4.9	10,082	<100	<100
B-97	1	Sandy silty clay	4.7	2,398	<100	<100
B-102	30	Sand w/silt & gravel	4.6	2,550	<100	664
B-104	75	Sand w/silt & gravel	7.1	9,761	<100	<100
B-110	2.5	Silt w/sand	6.8	11,825	<100	<100
B-122	5	Sand w/silt and gravel	7.5	Insufficient sample	<100	<100
B-124	3	Lean clay	7.7	2,190	<100	122
B-131	7.5	Silty sand w/gravel	6.7	20,805	<100	<100

Table 16.3: Summary of Corrosion Potential Testing – 2016 G&G program

Boring	Depth (ft)	Material Description	pH	Resistivity (ohm-cm)	Ion Concentration, ppm	
					Chloride (Cl^-)	Sulfate (SO_4^-)
B-146	7	Sand w/silt & gravel	6.4	19,083	<100	<100
B-146	12	Sand w/gravel	6.8	21,803	<100	<100
B-148	7.8	Sand w/silt & gravel	6.1	Insufficient sample	<100	<100
B-152	13.5	Sand w/silt & gravel	6.3	57,070	<100	<100
B-153	7	Sand w/silt & gravel	6.1	36,315	<100	<100
B-157	16	Sand w/silt & gravel	6.3	22,300	<100	<100
B-158	9.5	Sand w/silt	6.3	28,535	<100	<100
B-162	5.5	Sand w/silt & gravel	4.6	5,843	<100	220
B-165	6	Silty sand w/gravel	4.5	10,035	<100	<100
B-169	11	Sand w/silt & gravel	6.4	46,095	<100	<100
B-171	3.5	Sand w/gravel	5.4	33,075	<100	<100
B-173	6.5	Sand w/silt & gravel	5.7	36,383	<100	<100
B-189	11	Sand w/silt & gravel	6.3	42,803	<100	<100
B-195	7	Sand w/silt & gravel	6.3	34,960	<100	<100
B-197	11	Sand w/silt & gravel	6.5	48,150	<100	<100
B-198	6	Sand w/gravel	6.5	52,675	<100	<100

16.3 Corrosion of Buried Steel

The corrosion potential of steel is influenced by electrical resistivity, chloride ion concentration, and pH. Corrosion of steel is more likely in soil environments with low resistivity, high chloride ion concentrations, or low pH. For example, it is not necessary to have a resistivity between 0 and 1000 ohm-cm and a pH between 0 and 4.5 to indicate a "Very High" potential for corrosion potential. Table 16.4 presents general guidelines for estimating the corrosion potential of steel as a function of chloride ion concentration, pH, and electrical resistivity[105].

Table 16.4: Corrosion Potential based on Resistivity, pH and Chloride Content

Resistivity, ohm-cm	Chloride Content, ppm	pH	Corrosion Potential
0 – 1,000	>500	0 – 4.5	Very High
1,000 – 2,000		4.5 – 5.5	High
2,000 – 5,000	<500	5.5 – 6.5	Moderate
> 5,000		> 6.5	Mild

Each of the columns in the above table should be used independent of the others for estimating corrosion potential. The results indicate that the potential for corrosion due to chloride ion concentration is *mild to moderate* and the potential due to pH is *mild to high* in tested samples. Based on the electrical resistivity results, the corrosion potential for buried steel is also *mild to moderate* in tested soils. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

Based on the U.S. Army Corps of Engineers (USACE) Design Guidelines[106], yearly corrosion rates of steel pipe piles range between 0.00047 and 0.002 inch per year. We also researched other sources. The USACE reference mentions that Florida's Department of Transportation[107] requires an additional sacrificial steel thickness of 0.075 inch for slightly aggressive conditions and 0.15 inch for moderately aggressive conditions. Whereas, California Department of Transportation (Caltrans)[108] uses corrosion rate of 0.001 inch per year for steel piling exposed to corrosive soils in soil embedded zone. Caltrans considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. For the purpose of this study, we considered a corrosion allowance of 0.06 inch for the steel pipe piles based on corrosion rate of 0.0012 inch per year and structural service life of 50 years. We recommend that a corrosion engineer be consulted to review the corrosion allowance used in this study and/or to recommend appropriate protective measures.

[105] Palmer, J.F. (1974), "Soil Resistivity Measurements and Analysis", Materials Performance, Vol.13
[106] HSDDRS Design Guidelines-Waiver for Steel Piles Corrosion Protection, United States Army Corps of Engineers, Army Corporation, Vicksburg Mississippi, D.C.
[107] Structural Design Guidelines, Version 1, January 2017, FDOT Structures Manual, Florida Department of Transportation.
[108] Corrosion Guidelines, Version 2.0, November 2012, California Department of Transportation.

16.4 Degradation of Concrete

The degradation of concrete is caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate compounds which cause cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete. Sulfate concentrations in soil can be used to evaluate the need for protection of concrete based on the following table[109].

Table 16.5: Degradation of Concrete using Sulfate Ion Concentration

Sulfate Ion Concentration, ppm	Aggressiveness
>20,000	Very Severe
2,000 to 20,000	Severe
1,000 to 2,000	Moderate
<1,000	Mild

The results of sulfate tests on the selected soil samples from this study indicate the potential for deterioration of concrete, based on sulfate ion concentrations, is *mild* across the site. We recommend that a materials engineer be consulted to determine if protective measures are warranted.

[109] ACI Building Code Requirements for Structural Concrete (ACI 318-08), Section 4.2, (2007).
.

17.0 CONSTRUCTION CONSIDERATIONS

The following subsections provide our construction-related recommendations for site preparations, shallow open-cut excavations, fill materials and placement, groundwater considerations, driven pile installation, cast-in-place concrete piles, and construction monitoring.

17.1 Site Preparation

Site preparation should include clearing and stripping all topsoil, debris, vegetation, organic material (if present), and other deleterious material below the footprint of structures, paving or fill. The exposed subgrade should be proof-rolled, where feasible, with a heavy (20 ton or more) rubber tired (or steel drum with vibration turned off) vehicle to expose loose or weak zones. At least three passes should be made during proof-rolling. Consecutive passes should be made perpendicular to each other. Loose or weak zones indicated during proof-rolling should be over-excavated. Areas of the subgrade in which pumping or significant deflections are observed should also be removed. The exposed subgrade should then be brought to design grade using granular properly compacted non-frost susceptible fill. In areas where proof-rolling is not feasible, probing or density checks should be performed by the geotechnical engineer or a qualified representative to verify soil competence. We recommend that a geotechnical engineer or a qualified representative be present on-site to observe the exposed subgrade prior to placement of any fill material. Our detailed site preparation recommendations for the cut and fill areas are provided in *Section 4.4 Site Preparation for Cut and Fill Areas* of this report.

17.2 Shallow Open-Cut Excavations

Shallow open-cut excavations will be needed to install shallow foundations proposed to support the equipment/structures. The bottom of all shallow open-cut excavations for foundations at a minimum depth of 4 ft should expose either well compacted non-frost susceptible structural fill (*Section 17.3.1 Structural Fill*) or existing medium dense to dense in-situ non-frost susceptible granular soils. The bottom of the excavations should be "dry" and free of loose soil. If poor, loose, or weak soils are encountered at proposed foundation locations, we recommend that the soils be over-excavated and replaced with flowable fill or well-compacted non-frost susceptible structural fill.

We recommend that a qualified geotechnical engineer observe shallow foundation excavations immediately prior to placing the concrete. The engineer should compare the soils exposed with those encountered in the soil test borings and document the results. Any significant differences should be brought to the attention of the Engineer of Record.

Foundation soils exposed by the excavations should be protected from disturbance due to construction activities. We recommend that individual foundations be placed the same day the excavation is completed. If this is impractical, a seal slab of lean concrete should be placed over the

base of the excavation. We recommend that foundations be completed promptly to prevent wetting or drying of the foundation soils. Good surface drainage away from the excavations should be established to prevent surface runoff from either flooding the excavations or ponding around the completed foundations. Groundwater control/dewatering for shallow foundations will not be required at this site due to deep groundwater table.

Shallow open-cut excavations should be designed in accordance with all applicable local, state, and federal trenching regulations. We also recommend that excavation safety systems should follow the Federal OSHA requirements for excavations presented in 29 CFR Part 1926 (Revised July 1996), Subpart P, *Excavations*. Based on our interpretation of the OSHA regulations, we consider that the existing soils at the site should be classified as Type C soils. Slopes of temporary excavations in Type C soils should be no steeper than 1.5-horizontal to 1-vertical in all areas. Based on our analysis of the current soil conditions observed in the 2014, 2015, and 2016 G&G programs, 2-horizontal to 1-vertical slopes will be stable with satisfactory factor of safety. If sloughing and/or caving is experienced in the surficial soils, we recommend the slopes should be cut back to a stable condition.

17.3 Fill Materials and Placement

Fill materials will be required to raise the grade below the foundations or as backfill around the foundations. Unsuitable soils that are over-excavated below the non-insulated foundations should be replaced with non-frost susceptible fill at least to a depth of 7 ft below final grade. Backfill around foundations should also consist of non-frost susceptible fill. Other site grade raise areas (with no planned structures) will require general fill to match the proposed final grades.

Based on information available, the excavated material below surficial topsoil and weak clays / silts will generally consist of granular soils with fines content on the order of 3 to 18 percent. The fines content of granular soils in localized areas is up to 32 percent. The topsoil and surficial soft clays / loose silts will not be suitable for fill material. We recommend that the earthworks contractor cull the unsuitable topsoil and soft clays / loose silts, and stockpile underlying non-frost susceptible and granular soils at various predefined locations across the site for re-use. Our recommendations for classifying the structural fill, select fill, general fill, and flowable fill are provided in the following subsections.

17.3.1 Structural Fill

Structural fill defined herein consists of non-frost susceptible granular soils. According to U.S. Army Corps of Engineers Document No. TM-5-852-3[110], well-graded (SW) or poorly-graded (SP) sands, containing less than 3 percent fines (defined as material finer than 0.02 mm) and well-graded (GW)

[110] Arctic and Subartic Construction Runway and Road Design, Document No. TM-5-852-3, October 1954, United States Army Corps of Engineers, Army Corporation, Washington, D.C.

or poorly-graded (GP) gravel containing less than 1.5 percent fines (defined as material finer than 0.02 mm) are considered non-frost susceptible material. Frost susceptibility tests of in-situ natural granular soils indicated that the soils with less than 20 percent fines are non-frost susceptible, provided the fines consist of granitic rock flour material and not clay minerals.

Non-frost susceptible granular structural fill should be placed in 8-inch to 10-inch thick loose lifts and should be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D4253 and D4254). We recommend using hand-operated compaction equipment and 6-inch thick loose lifts in confined areas and immediately adjacent to utility lines. Smaller equipment and thinner lifts are recommended when compacting next to walls as excessive compaction/energy could induce lateral pressures that are greater than the design pressures.

17.3.2 Select Fill

Select fill may be used as backfill below the non-frost susceptible structural fill (7 ft below design grade) and should consist of a clean sand fill with 20 to 30 percent material passing the No. 200 sieve (Plate 18a), provided the fines consist of granitic rock flour material and not clay minerals. It should be noted that any existing soils that will be stockpiled to use as structural sand fill will require additional laboratory testing to confirm clay content, and evaluate the compaction and compressibility characteristics. Structural sand fill should be placed in 8-inch to 10-inch thick loose lifts and should be compacted to a relative density of at least 80 percent as determined by the relative density test methods (ASTM D 4253 and 4254). We recommend using hand-operated compaction equipment and 6-inch thick loose lifts in confined areas and immediately adjacent to utility line. Care should be taken not to over compact next to below-grade structures. Excessive compaction could induce lateral pressures that are greater than the design pressures.

17.3.3 General Fill

General fill (Plate 18a) defined herein should consist of 30 to 50 percent material passing the No. 200 sieve. General fill does not have to be non-frost susceptible as it should only be placed outside of areas of structures, foundations, and rigid pavements. Soils to be used as general fill should be free of concrete, rubble, excessive organics, and contaminants. On-site soils with minimal processing are suitable as general fill. It should be noted that when minimally processed on-site material is used as a general fill, the volume might be reduced significantly depending upon the compaction level and the in-situ condition of the soil. General fill should be placed in loose lifts not exceeding 8-inches to 10-inches in thickness. General fill should be placed at a workable moisture content, and then uniformly compacted to a relative density of at least 70 percent as determined by the relative density test methods (ASTM D4253 and D4254).

Classification of in-situ soils for use as fill materials will require careful observation of cut operations at the project site, stockpiling, sampling from stockpiles and laboratory testing of these soils. We

recommend that sampling from stockpiles and laboratory assignments are performed with the supervision of a field geotechnical engineer. We also recommend that fill placement and compaction operations are monitored continuously and field density tests (e.g., sand cone or nuclear gauge tests) are performed to ensure the compaction quality control.

17.3.4 Flowable Fill

Flowable fill may be used as backfill against foundations or for places where loose or unsuitable soils are exposed. In general, sandy soils containing less than about 30 percent fines[111] are suitable in flowable fill. The upper poorly graded sands encountered at this site are generally suitable as flowable fill if it is free from deleterious materials. Flowable fill should meet the following recommendations [112,113].

- sandy soils used for flowable fill should contain no more than 30 percent fines,
- the soil and cement should be thoroughly mixed,
- proportioning of the sandy soil and cement should be sufficient to produce at the least an unconfined compressive strength of 100 to 200 psi at 7 days while remaining workable for placement,
- maximum strength should be suitable for future removal by excavation, if anticipated
- the flowable fill should not be placed when ambient air temperature is below 40 degrees Fahrenheit,
- placement of the flowable fill should completely fill the space to be backfilled,
- the flowable fill should be consolidated using standard ASTM procedures for soil-cement materials; if a fluid consistency is used for the soil-cement slurry, an “elephant trunk” vibrator may be used to consolidate and remove air voids from the mixture, and
- backfill over the flowable fill should not be placed until after initial set (4 to 6 hours) has been achieved for the flowable fill.

17.4 Groundwater Considerations

Based on the monitoring well records the groundwater levels within the Water Bearing Unit 1 vary between elevations El. +96 ft (NAVD88) to El. +75 ft. (NAVD88). Excavations below a depth of about 4 ft (below grade) as discussed in this report will generally encounter granular soils across the site. The existing granular soils at the site are generally permeable, and the surface runoff and seepage of perched water, if any, in to excavations is expected to drain through permeable soils.

[111] Howard, A.K., “Soil-Cement Slurry Pipe Embedment”, SP 150-6.
[112] US Bureau of Reclamation, Geotechnical Branch Training Manual No. 7 - Chapter 9 “Soil-Cement Pipe Bedding”, Denver, CO, 1989.
[113] ACI Committee 229 Report (229R-99), “Controlled Low-Strength Materials”, 2005.

Therefore, we do not anticipate groundwater seepage into shallow excavations. However, if seepage is encountered, it may be controlled using a system of sumps and pumps.

The proposed cut in Area 1 for the Heavy Haul Road (HHR) will intersect with the existing groundwater table. Plates 21a through 21c show the estimated groundwater table with proposed cuts at three cross-sections. The soils exposed during excavation are generally permeable. Longitudinal (parallel to HHR) deep ditches or ditches with underdrains (slotted pipe underdrains or french drains) should be designed and utilized to intercept and lower the groundwater table to at least 3 ft below planned road elevation for construction and performance of the haul road. Transverse underdrains may also be utilized as part of a properly designed drainage system. If wet-digging is not feasible, adding well-points and pumping should be considered.

17.5 Driven Pile Installation

General recommendations for driven pile installation are included in this section. We have included discussions of pile drivability, pile driving specifications, pile driving equipment, installation methods, pile driving records, static load testing and dynamic PDA load testing. Sheetpile walls planned to support the proposed MOF construction are addressed under our marine integrated report, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012[114].

17.5.1 Pile Drivability

It should be noted that dense to very dense granular soil layers containing will be encountered in the driven pile area(s). The subsurface soil conditions have the potential to make pile driving difficult and challenging. The pile driving contractor should study the site soil conditions and should select appropriate pile driving equipment and installation methods to reduce potential problems and to avoid pile and/or equipment damage.

We recommend that pile driving contractor complete detailed wave equation analyses and pile drivability studies to select suitable pile driving hammer(s) and pile installation methods. Preliminary pile drivability studies by Fugro indicate that compressive driving stresses will be high. The pile driving leads, fixed or swinging style, should maintain good alignment between the hammer and the pile during driving to avoid high eccentric hammer blows which could increase compression driving stresses and result in pile damage.

As stated above, Fugro completed a limited/preliminary pile drivability study to model the driving of selected open-end steel pipe piles. The purposes of the preliminary pile drivability study were to evaluate: 1) the suitability of various steel pipe sizes (diameters, wall thicknesses and yield strengths); and 2) the suitability of various diesel hammer sizes (rated energy). Our preliminary

[114] Fugro Consultants, Inc. (Fugro), 2016, LNG Facilities Marine Integrated Site Characterization and Geotechnical Engineering Report, Alaska LNG Project, Nikiski, Alaska, Report No. 04.10140334-15, AKLNG Document No. USAL-FG-GRZZZ-90-002015-012, Rev.0, dated September 06, 2016.

study included five different pile diameter and wall thickness combinations and four different diesel hammers to determine the various penetration depths.

Static capacity analyses were completed to establish axial capacity versus depth curves for each pile size. Specifically, we used the skin friction and end-bearing values from upper bound static axial capacity curves for Area 4B. The groundwater table was assumed at about 40 ft below existing grade. Upper bound axial pile capacities were calculated for the preliminary pile drivability study, using the following parameters (Table 17.1) selected from upper bound values of Bowles (1977) correlations published in Caltrans Manual[115].

Table 17.1: Selected Soil Design Parameters – Pile Drivability Analysis

Depth below Finished Grade, ft	SPT $N_{1,60}$(2)	Effective Unit Weight(1),(2), pcf	Friction Angle, degrees(2)
0 to 20	55	150	43
20 to 35	60	150	44
35 to 40	70	150	45
40 to 60	70	87.6	45

Notes:
1. Groundwater table was estimated at 40 ft below final grade.
2. Parameters from upper bound of Bowles (1977) in Area 4B were selected.

The pile driving contractor will select the pile driving equipment and pile installation methods that will be used for this project. The selection should be based on the results of their final pile drivability study which will need to be submitted and approved by the Owner and the Engineer of Record prior to the start of the pile driving. Pile drivability was performed using the computer program GRLWEAP (Wave Equation Analysis of Pile Driving, Version 2010) developed by PDI, Inc. The pile driving hammers used in Fugro's preliminary analyses were determined based on our review of the selected pile sizes, and our experience with similar soil conditions. The selected hammers are commonly used in the industry to drive open-end steel pipe piles. Drivability analyses were performed on the following steel pile diameters and wall thickness combinations:

- 16-inch diameter by 0.375-inch wall thickness
- 18-inch diameter by 0.5-inch wall thickness
- 18-inch diameter by 0.625-inch wall thickness
- 24-inch diameter by 0.5-inch wall thickness
- 24-inch diameter by 0.625-inch wall thickness

[115] California Department of Transportation (Caltrans), 2014, Caltrans Geotechnical Manual, Chapter Soil Correlations (March 2014), Page 3 and 5, http://www.dot.ca.gov/hq/esc/geotech/geo_manual/manual.html

Four open-end diesel pile driving hammers were included in the preliminary drivability study. Hammer size (rated energy) is typically selected based on the soil resistance to driving, but also on the pile's cross-sectional steel area. Hammer energies ranged from 74,419 ft-lbs up to 202,900 ft-lbs. Due to the preliminary nature of our study, we did not analyze any single acting/external combustion hammers or any hydraulic hammers, although both types of hammers could be used. The four diesel hammers included in our preliminary study are as follows:

1. APE Model D30-42 hammer: (Rated Energy=74,419 ft-lbs, 6,615 lb ram, 11.25 ft stroke), 16-inch and 18-inch diameter piles.
2. Berminghammer Model B5505 hammer: (Rated Energy=105,900 ft-lbs, 9,200 lb ram, 11.5 ft stroke),16-inch and 18-inch diameter piles.
3. APE Model D46-52 hammer: (Rated Energy=114,109 ft-lbs, 10,143 lb ram, 11.25 ft stroke), 24-inch diameter piles.
4. Berminghammer Model B6505 hammer: (Rated Energy=202,900 ft-lbs, 17,640 lb ram, 11.5 ft stroke), 24-inch diameter piles.

Hammer manufacturers typically provide a "refusal" criterion to avoid hammer and/or pile damage. The "refusal" criterion is given as a maximum number of blows per foot. Failure to follow these recommended limits will often void the warranty of the pile driving equipment. In our preliminary pile drivability study, we established the pile refusal depth based on limiting the hammer blows to 120 blows per foot used by American Piledriving Equipment Inc. (APE) standard practice[116]. Refusal depths, maximum blow counts, maximum strokes, maximum driving compressive stresses along the piles, and the compressive stresses at end-of-initial driving (EOID) obtained from the drivability analyses are summarized in Table 17.2. GRLWEAP output files including the cushion parameters are also presented on Plates G-1 through G-30 in Appendix G of this report. For all the cases analyzed, the driving stresses at the end of initial driving (EOID) were below 90 percent of the yield stress of steel which is considered to be the upper limit for compressive stresses. However, it should be noted that several piles are estimated to experience maximum compressive stresses higher than the 90 percent of the steel yield strength (see Table 17.2).

There are four methods generally used to remediate high compressive driving stresses. The four methods include: 1) increasing the steel yield strength; 2) increasing the steel area (wall thickness); 3) controlled predrilling of the pile locations prior to driving; and 4) initially seating the pile using a vibratory driver. These methods can be used individually or in combination. Use of the vibratory driver and pre-drilling will require additional construction equipment and personnel. The installation will require sequencing between vibratory driving or pre-drilling with the impact hammer driving. The

[116] American Pile Driving Equipment, "Refusal Criteria for Diesel and Hydraulic Impact Hammers" https://www.apevibro.com/ver2/refusaldiesel.asp

sequencing will increase the pile installation time and may require adjustments to the coordination of the field operations.

Table 17.2: Summary of Pile Drivability Results – Impact Hammers

Pile Diameter × Thickness (in)	Hammer Type	Max. Penetration Depth (ft)	Blow count at Max. Penetration Depth (blows/ft)(1)	Max. Stroke (ft)	Max. Compressive Stress (ksi)(2)	Compressive Stress at EOID (ksi)(2)
16 x 0.375	D30-42	52	117	10.3	40.9	38.0
	B5505	55	117	8.5	41.7	38.8
18 x 0.500	D30-42	60	111	12.2	40.0	38.9
	B5505	60	107	9.8	38.8	37.3
18 x 0.625	D30-42	60	112	10.8	35.6	35.0
	B5505	60	98	9.0	34.2	34.1
24 x 0.500	D46-42	21	119	9.8	41.7	37.6
	B6505	35	116	8.1	41.4	37.0
24 × 0.625	D46-42	50	119	12.0	41.5	39.8
	B6505	60	118	9.3	40.8	37.7

Note: 1. Maximum blow count should be limited to 120 blows / ft.
2. Maximum compressive stress should generally be less than 40.5 ksi (i.e. 90 percent of yield strength) for ASTM A252 Grade 3 steel pipe piles. Appendix G provides more detailed information of stress distribution along a pile.
3. Upper bound parameters for Area 4B were used for the analysis.

Vibratory drivers can be used to seat the piles in the granular soils, but driving to design penetration must be accomplished with an impact hammer. Table 17.3 presents preliminary drivability analyses based on predicted penetration rates for a plugged pile and the ICE 1412 vibratory driver at 33-ft penetration. The penetration rates increase to refusal below 33-ft penetration as the sands become denser. Drivability analysis results with a vibratory driver are also presented on Plates G-31 through G-40 in Appendix G of this report.

Table 17.3: Summary of Pile Drivability Results – Vibratory Hammer ICE 1412 with Penetration Depth of 33-ft

Pile Diameter × Thickness (in)	Penetration Rate (ft/sec)	Max. Driving Stress (ksi)
16 x 0.375	36	15.8
18 x 0.500	21	12.7
18 x 0.625	15	10.7
24 x 0.500	88	11.5
24 × 0.625	62	9.5

17.5.2 Detailed Pile Driving Specification

Detailed pile driving specifications should be prepared by the design engineer in conjunction with the geotechnical consultant. The specifications should cover the project requirements for furnishing and

installing the piles including the scope of work, necessary submittals, piling details, equipment requirements, installation requirements and tolerances, capacity evaluation, and construction records. The specification should require the contractor to submit a complete package detailing the proposed piling equipment and installation procedures for approval prior to mobilization to the site. The complete package should also include the results of the wave equation analyses to evaluate the proposed pile driving hammer and cushion system prior to approval for mobilization to the site.

We recommend that the specification establish pile driving criteria to clearly define the required pile capacities, pile penetrations, and final driving resistance for acceptance. The results of the wave equation analysis should be used to establish the pile driving criteria. Requirements for pile load tests and capacity evaluation should be stated. The Geotechnical Engineer of Record should be consulted during the development of the pile driving criteria. The specification should require the contractor to notify the engineer of any changes to the pile driving equipment and methods so that the pile driving criterion can be adjusted, if necessary. We recommend that the geotechnical engineer or their qualified representative be on-site during pile driving activities to observe pile installation. Remedial measures should be presented to address piles not achieving the specified criteria, out of tolerance piles, or piles with questionable driving records.

17.5.3 Pile Driving Equipment

Production piles should be driven using a hammer of adequate size in as nearly a continuous operation as feasible, without interruption if possible. Pile driving hammers may be hydraulic, diesel, steam, or air operated. The use of a drop hammer, with a light ram and a large stroke, should not be allowed since this type of hammer can produce exceedingly high and damaging stresses during driving. As high compressive driving stresses are anticipated, protective driving shoes are recommended in these soils to prevent pile tip damage during driving.

The results of the wave equation analysis should be used to demonstrate that the proposed hammer has sufficient energy to install the piles to the required capacity and/or penetration, and that the hammer is properly cushioned to avoid structurally damaging the piles. It is recommended that spare hammers are available on-site to ensure continuous pile driving. To avoid damaging the pile and/or pile driving equipment, refusal criteria should be determined and agreed upon by all parties involved prior to the start of actual pile driving. Pile driving should be stopped whenever the “refusal” criterion is observed. The “refusal” blow count records should be immediately transmitted to the Engineer of Record for review and approval or remedial measures.

Pile driving equipment may generate excessive noise levels. Federal Highway Administration (FHWA) documents typical pile driver noise levels as “95 to 101 dBA, slow” (measured at 50 ft from the loudest side of the equipment) based on the specification limit of the equipment and their actual emission measurements[117]. A number of mitigation techniques are available to reduce the

[117] Federal Highway Administration, 2006, Construction Noise Handbook, https://www.fhwa.dot.gov/environment/noise/construction_noise/

construction noise levels, such as use of shields and dampening methods. Detailed assessment of noise levels, sensitive receptors and applicable mitigation measures should be performed during the selection of the pile driving equipment.

17.5.4 Driven Pile Installation Criteria

Production piles should be installed to a design blow count criterion based on the axial pile capacity versus penetration curves presented in Appendix C. In addition, a minimum penetration criterion should be used to ensure sufficient lateral capacity. The selection of a particular length and particular criteria depends on the pile size, available length and capacity requirements. The minimum hammer blow count criterion should be selected based on the results of detailed wave equation analyses, dynamic PDA testing (dynamic load tests) and static compression load tests.

Driven piles can experience an increase or decrease in capacity with time after the end of continuous driving. Increase in capacity with time is called “setup”. In contrast, decrease in capacity with time is called “relaxation”. Setup is more significant when piles are driven into cohesive soils, but this can also be experienced by piles driven into cohesionless soils. Relaxation is less common compared to setup with time. However, relaxation can occur with dense to very dense cohesionless soils. Thus, we recommend restriking (see paragraph below) selected production piles periodically to verify that the pile capacity do not decrease with time. This can be a good practice item with little to no delay time during driven pile installation.

We recommend surveying the production piles to detect possible vertical (commonly referred to as heave) and/or horizontal movement that can result from soil displacement when driving adjacent piles. Piles which heave after driving adjacent piles should be redriven to at least their original penetration and final driving resistance. We do not recommend the use of jetting to aid pile installation at this site without the consultation of the Geotechnical Engineer of Record.

17.5.5 Pile Load Test Program

A pile load test program should be performed prior to the start of the production pile driving work. The purposes of the pile load test program should be to verify the axial and lateral capacity of the piles and to assist in evaluating the production pile installation criterion, installation methods and pile driving equipment. Typically, the pile load test program should include: 1) the driving of several “unloaded” or probe piles across the site; 2) dynamic PDA testing during initial driving to evaluate hammer performance, driving stresses and possible damage; 3) dynamic PDA testing during restrike driving to evaluate long-term axial capacity and provide a correlation with static compression load tests; 4) static compression load tests; and 5) static lateral load tests. The number of static load tests to be performed should be determined by the Engineer of Record and the Geotechnical Engineer based on the uniformity of the site soil conditions and the variability of the pile driving. The results of the probe pile driving will aid in the evaluation of soil and pile capacity variations across the site. Static load testing should be performed in the areas with lower bound axial capacities and

adjacent to the existing soil borings. Additional details about the dynamic and static load tests can be found in Section 11.10 Load Tests.

17.5.6 Dynamic PDA Testing

We recommend that design pile capacities also be verified during production pile installation by dynamic methods utilizing a Pile Driving Analyzer (PDA). The frequency of PDA testing should be determined based on the risk assessment analysis performed by the Engineer of Record. The PDA can estimate hammer performance, driving stresses, hammer-to-pile alignment, pile damage, and pile capacity during driving. Restrike testing monitored with the PDA in combination with the computer program CAPWAP, (**CA**se **P**ile **W**ave **A**nalysis **P**rogram) can provide an estimate of the pile capacity increase and aid in predicting long-term pile capacity.

17.5.7 Pile Driving Records

An accurate and detailed driving log should be kept by an independent inspector during production driving operations. The log should provide a complete record of hammer blows per foot of penetration from the initial to the final blow for each pile installed. The record for each pile should also include the driving date, pile information, hammer information, cushion type, size and condition information, hammer and compressor operation information, ground and pile tip elevations, records of pre-drilling, dynamic PDA re-strike driving tests, and notes on installation delays, problems, or unusual occurrences. The pile driving records should be provided to the Geotechnical Engineer of Record to review and identify possible anomalies.

17.6 Cast-in-Place Concrete Pile Installation

The proper installation of cast-in-place concrete piles (CIPC, also known as drilled shaft or drilled piers) is dependent upon the site soil conditions, construction procedures and equipment. Installation factors have a greater effect on the performance of cast-in-place piles compared to other types of deep foundations. The installation methods and installation equipment vary depending on the selected CIPC contractor. Detailed pile installation specifications specifically addressing the special concerns of CIPC piles, the site soil conditions and ground water conditions should be prepared as part of the construction package. The pile specifications should be in general accordance with the ACI 336.1 Specification for the Construction of Drilled Piers[118] prepared by the American Concrete Institute (ACI).

17.6.1 Contractor Experience and Equipment

The selected contractor should have relevant project experience with installation of similar types of piles in similar subsurface soil conditions; welding, installation and withdrawal of the steel casing

[118] ACI 336.1, Specification for the Construction of Drilled Piers, American Concrete Institute, 2001, https://www.concrete.org

liners; placement of reinforcing steel; as well as experience in working in cold climate and handling the special concrete mixes and admixtures. The contractor's equipment should be acceptable per the project requirements and specifications. Our experience indicates that not all contractors have the specialized equipment and experience required to successfully install CIPC piles and particularly for installation of pile diameters 24-inch and larger. We strongly recommend that the contractor's qualifications and experience records be reviewed as part of the selection process.

17.6.2 Concrete Mix

The pile contractor should be able to demonstrate that a concrete mix can be furnished to conform to ACI 301[119]. The results of a test mix design or test results from a previous project are generally required to be submitted prior to approval of the concrete mix for construction. During construction, the required tests on concrete include slump and temperature on site in accordance with ASTM C143 / C143M[120] and ASTM C1064[121], and compressive strength of standard-cured cylinders prepared in accordance with ASTM C31/ C31M[122] and tested in accordance with ASTM C39[123]. If the concrete pours are planned during cold weather (i.e. air temperature below 40°F), special measures should be in place as per ACI 306.1[124]. ACI recommends taking a minimum of one set of four concrete cylinders per drilled pier, for compression testing at 28 days.

17.6.3 Pile Installation and Monitoring

The pile installation and monitoring should be in general accordance with the ACI 336.1. Experience shows that the walls of the shafts excavated in these types of soils, especially those below the depth-to-water, will collapse unless supported by casing and/or heavyweight slurry. The use of temporary casing requires that the Deep Foundation Contractor be careful when pulling the auger to remove cuttings from the excavation. A slightly oversized casing is sometimes used to reduce chances for snagging and lifting the casing during cleanout operations. The CIPC pile installation should be observed by the Geotechnical Engineer-of-Record to verify that, among other things:

- Subsurface conditions are as anticipated from the soil borings.
- CIPC pile s are constructed to the proper diameter and penetration, and are within allowable tolerance for plumbness.
- Reinforcing steel and concrete are properly placed.

[119] Specifications for Structural Concrete, American Concrete Institute, 2010, https://www.concrete.org
[120] ASTM C143 / C143M-15a, Standard Test Method for Slump of Hydraulic-Cement Concrete, ASTM International, West Conshohocken, PA, 2015, www.astm.org.
[121] ASTM C1064 / C1064M-12, Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete, ASTM International, West Conshohocken, PA, 2012, www.astm.org
[122] ASTM C31 / C31M-15ae1, Standard Practice for Making and Curing Concrete Test Specimens in the Field, ASTM International, West Conshohocken, PA, 2015, www.astm.org
[123] ASTM C39 / C39M-16b, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, ASTM International, West Conshohocken, PA, 2016, www.astm.org
[124] ACI 306.1-90, Standard Specification for Cold Weather Concreting, American Concrete Institute, 2002.

These critical items are fundamental to the proper installation and behavior of the CIPC piles in accordance with our recommendations. The following installation techniques will aid in the successful construction of CIPC piles. We anticipate that bentonite slurry will be used.

- For tremie placed concrete, the clear spacing between rebar or behind the rebar cage should be at least 8 times the maximum size of the coarse aggregate or 5 inches, whichever is larger.
- Centralizers on the rebar cage should be installed to keep the case properly positioned.
- Cross bracing of a reinforcing cage may be used when fabricating, transporting, and lifting. However, experience has shown that cross bracing can contribute to the development of voids in a concrete shaft. Therefore, we recommend removing the cross bracing prior to lowering the reinforcing cage into the open shaft.
- We anticipate that slurry methods and casing will be used and recommend that the concrete be limited to a slump no greater than 8 inches. Concrete should be placed from the bottom of the shaft using a tremie pipe or chute, and proper tremie concrete placement procedures. The concrete should be designed to achieve the required strength at the slump indicated.

Concrete tremied into a shaft with slurry should always maintain a hydrostatic pressure greater than either the surrounding groundwater level or slurry in the excavation. The slurry should have a marsh funnel viscosity of between 28 and 50 seconds. The slurry should have a specific gravity between 1.03 (8.6 lb/gal.) and 1.15 (9.6 lb/gal.) at the time of concrete placement. In addition, the sand content in the slurry should be less than 4 percent immediately prior to placement of the concrete. If casing is used, a sufficient head of concrete to replace the volume of the casing is required before pulling the casing. Proper cure and protection of the pile tops should be implemented to prevent moisture loss and temperature extremes in accordance with ACI 301.

The successful completion of CIPC pile excavations will depend largely on the suitability of the drilling equipment and skill of the operator. The CIPC pile contractor should try to reduce the time the excavation is open. The Contractor should schedule the sequence of operations so each excavation can be completed, reinforcing steel placed, and the concrete poured in a continuous, rapid, and orderly manner. The Contractor should not place piles adjacent to each other until the first pile sets. CIPC pile spaced close than about 6 to 8 feet (clear spacing) should be placed at least 12 hours apart and preferably on alternate days. CIPC pile excavation should **not** be left open overnight.

17.6.4 Crosshole Sonic Logging

A non-destructive technique currently available to observe completed cast-in-place piles is Crosshole Sonic Logging (CSL). We believe it is essential that a qualified Geotechnical Engineer observe the installation of cast-in-place piles using CSL testing to provide a higher degree of confidence that the piles are properly installed. Steel access pipes (1.5 to 2.0-inch diameter) are tied to the inside of the reinforcing steel cage and installed into the pile shaft prior to concrete placement. Following concrete placement, a transmitter probe is inserted to the bottom of one

access pipe and a receiver probe is inserted to the bottom of an adjacent access pipe. The probes are lifted to the pile top and the travel time and signal strength of ultrasonic pulses are recorded. Defects such as caving, necking or soil inclusions in the pile can be detected by evaluating non-uniformity in the collected data. Use of the CSL testing at the start of construction to evaluate alternate installation procedures and to help establish an acceptable production pile installation procedure is particularly beneficial.

17.7 Thermal Integrity Profiler

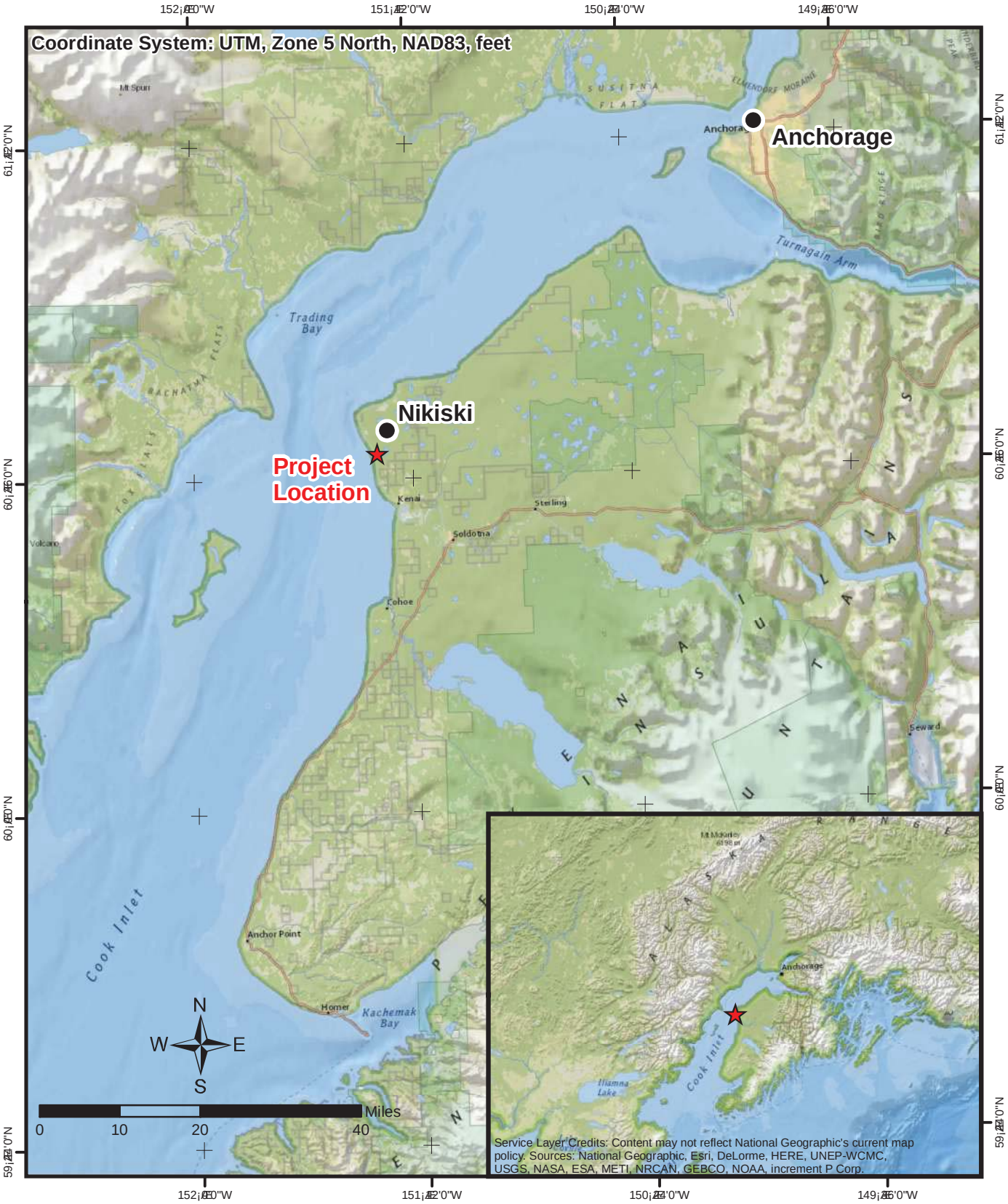
Another more recent non-destructive technique currently available to observe completed cast-in-place piles is the Thermal Integrity Profiler (TIP). Cables with equally spaced temperature sensors are attached to the inside of the reinforcing steel cage prior to concrete placement similar to CSL testing. The temperature data is collected and variations in temperature versus depth are an indication of non-uniform construction. Defects such as caving, necking or soil inclusions in the pile can be detected using sonic integrity logging. Use of the TIP testing at the start of construction to evaluate alternate installation procedures and to help establish an acceptable production pile installation procedure is particularly beneficial.

17.8 Construction Monitoring

We strongly recommend that a geotechnical engineer, or qualified representative, be present on-site to evaluate the suitability of subgrade soil for foundation support and to observe foundation installation and testing. On-site observations may also aid in recognizing and reconciling other unanticipated soil or groundwater conditions and ensure that design recommendations are appropriate and properly implemented during construction. During the foundation excavation and construction phases, we can provide material testing and construction surveillance to: (1) observe compliance with the design concepts, specifications, and recommendations; (2) observe subsurface conditions during construction; and (3) perform quality control tests.

* * *

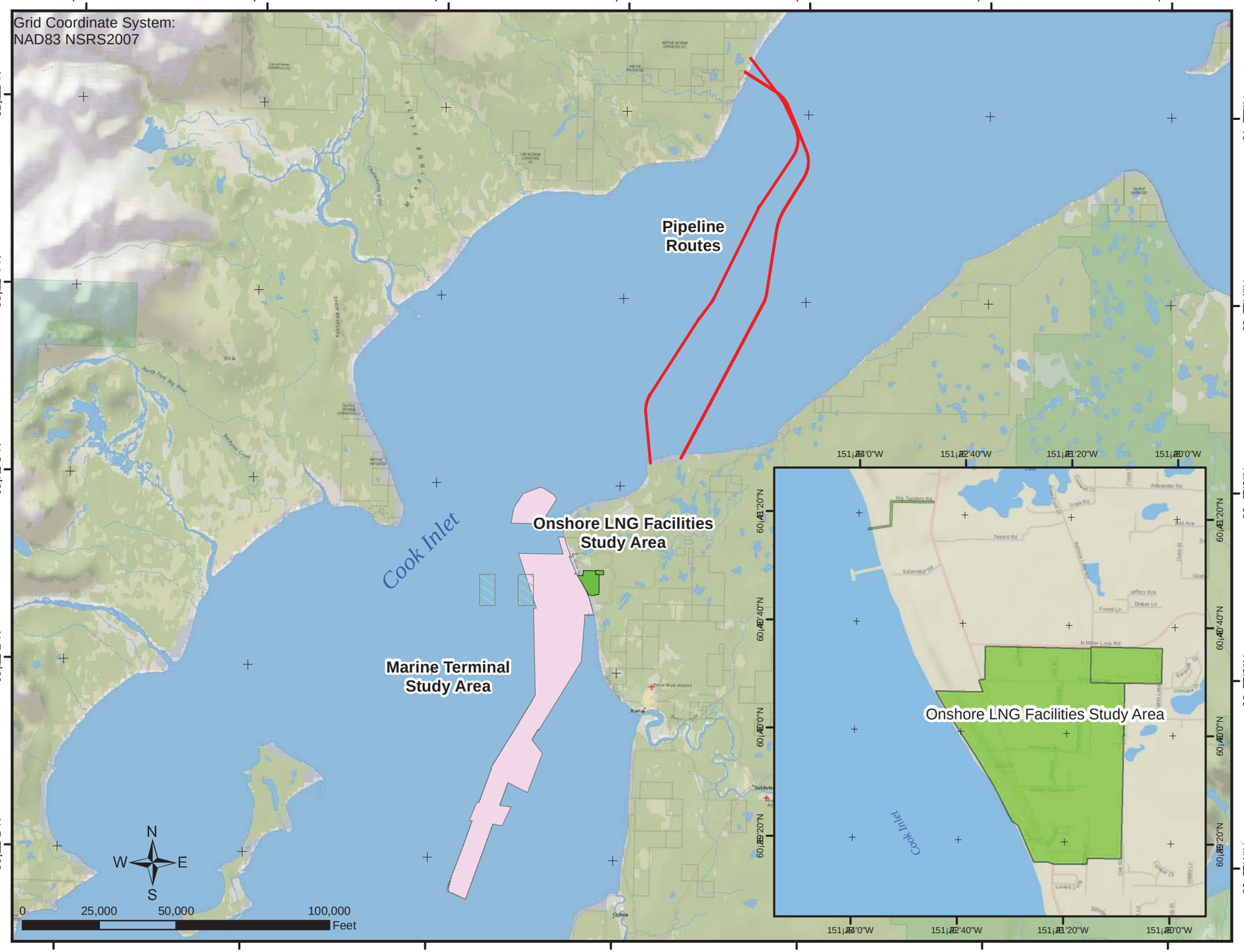
ILLUSTRATIONS

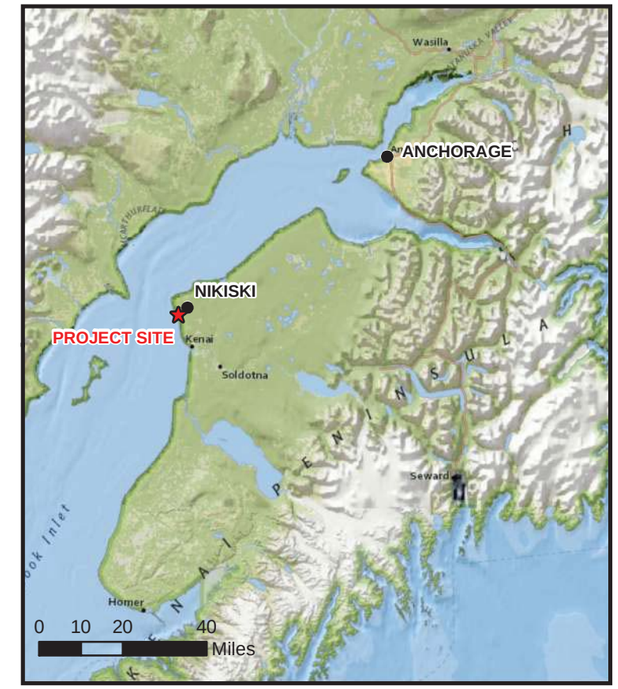


VICINITY MAP
ONSHORE FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 1

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-01_VicinityMap.mxd, 11/18/2016, kimk





KEY MAP

LEGEND

- Onshore LNG Facilities Study Area
- Marine Terminal Study Area
- Dredge Disposal Area
- Pipeline Routes

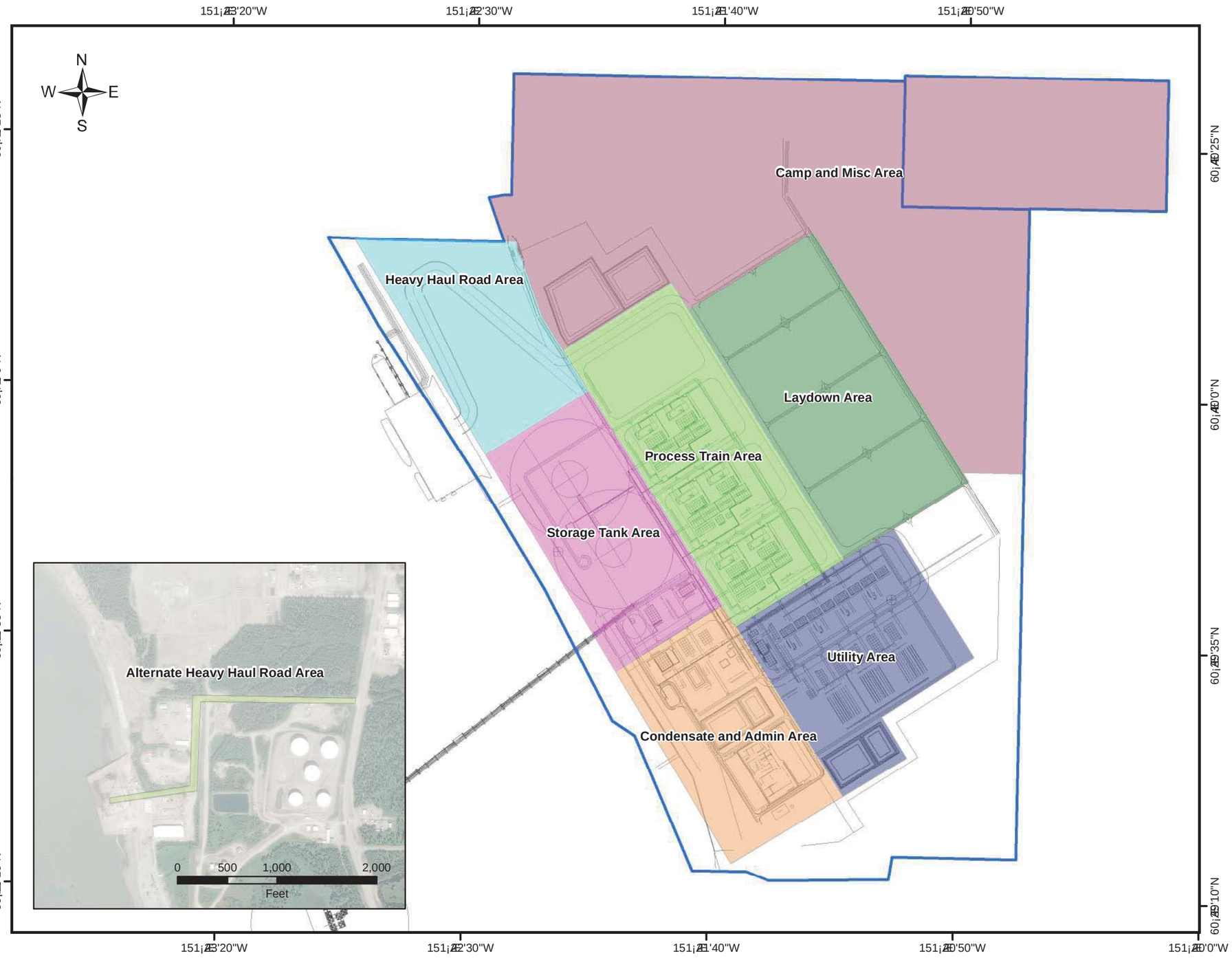
NOTE:

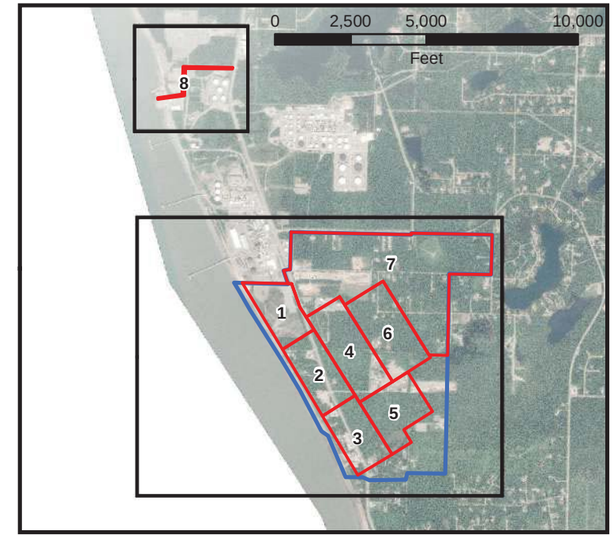
1. Onshore LNG Facilities Study Area boundary provided by AKLNG.

OVERALL SITE PLAN
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 2

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-02_OverallSitePlan.mxd, 1/17/2017, kimk





KEY MAP

Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area

NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.

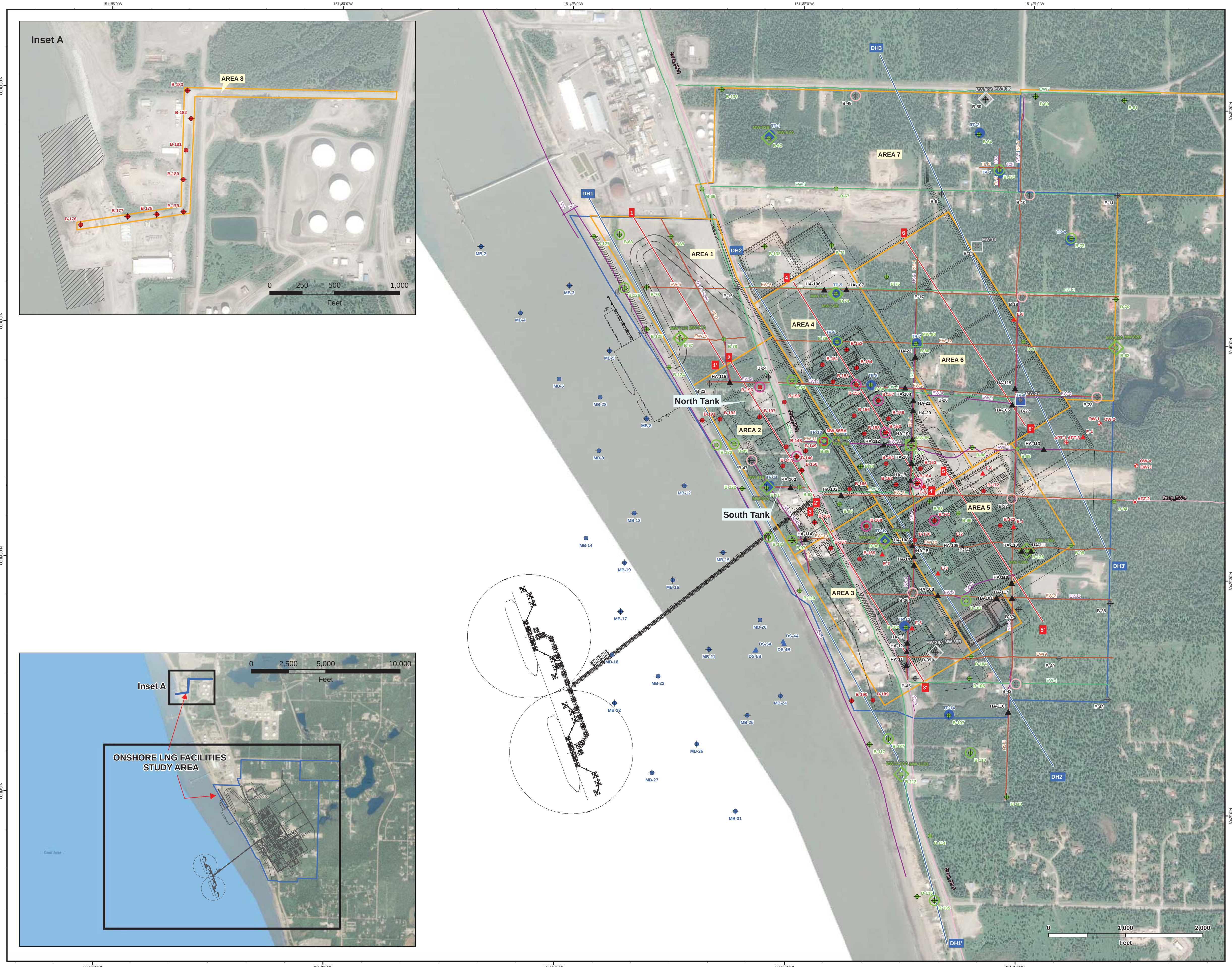
0 750 1,500 3,000
Feet

Grid Coordinate System: NAD83 NSRS2007

STRUCTURE LAYOUT PLAN
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 3a

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-03_StructureLayoutPlan.mxd, 12/15/2016, kimk



LEGEND

- Onshore LNG Facilities Study Area
- DH1 — Cross Section Lines - Downhole Seismic
- Engineering Area
- 1 — Cross Section Lines - Integrated Geophysical and Stratigraphy Profiles
- Siltation Survey Area (McLane, 2016)

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- APT Well (2016)
- Observation Well (2016)
- VES Location (2016)
- Hand Auger (2015)
- Test Pit (2015)
- Marine Boring (2015)
- Drive Sampling (2015)
- Monitoring Well - Shallow (2016)
- Monitoring Well - Shallow (2015)
- Monitoring Well - Shallow&Deep (2015)
- Monitoring Well - Shallow (2014)
- Monitoring Well - Shallow&Deep (2014)
- Downhole Geophysics (2016)
- Downhole Geophysics (2015)
- Downhole Geophysics (2014)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

Borings	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007) Longitude, deg	Latitude, deg	DH[1]	MW[2]	TP[3]	Borings	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007) Longitude, deg	Latitude, deg	DH[1]	MW[2]	TP[3]
B-9	132.25	100.50	-151.35640930	60.67185419				B-105	95.54	101.00	-151.35368353	60.65469558			
B-10	132.80	103.00	-151.35002130	60.67188199	1			B-107	96.04	101.00	-151.35545580	60.65337380			1
B-11	136.36	101.30	-151.34366490	60.67192943				B-110	117.07	155.70	-151.35348741	60.65205636	1		
B-14	133.40	110.70	-151.35375740	60.67003682		1		B-112	109.09	101.00	-151.35846843	60.65124872		2	
B-15	122.96	101.00	-151.37105710	60.66836028				B-113	112.53	113.50	-151.35080438	60.65052782			
B-17	131.21	101.50	-151.35723250	60.66845974				B-114	114.99	126.00	-151.35629452	60.64907498			
B-18	131.25	101.50	-151.35041260	60.66826252	1			B-117	7.60	99.30	-151.36078845	60.65228123			
B-23	124.63	186.50	-151.37288540	60.66494561				B-120	15.41	101.00	-151.36601528	60.65771551			
B-24	120.40	101.00	-151.36864010	60.66523854				B-121	16.38	120.40	-151.36825549	60.65946757	1		
B-26	125.54	100.50	-151.35535980	60.66481307				B-122	16.52	101.00	-151.37037895	60.66120614			
B-27	124.35	101.50	-151.35039120	60.66461107		1		B-123	15.78	139.00	-151.37209000	60.66279044	1		
B-28	118.89	103.00	-151.34486220	60.66475620	1			B-124	13.68	78.50	-151.37584730	60.66551173			
B-32	122.99	101.50	-151.35084730	60.66109265	1			B-125	14.43	101.00	-151.37751496	60.66684480			
B-34	111.83	101.00	-151.35361470	60.65951893				B-126	14.43	127.00	-151.37914509	60.66831013	1		
B-36	111.70	103.00	-151.35789050	60.65768300	1			B-127	15.98	101.00	-151.38141047	60.67012169			
B-37	106.27	27.00	-151.35022180	60.65717918				B-131	133.25	91.00	-151.35225744	60.67274798	1		1
B-38	123.47	151.30	-151.34359640	60.65746298				B-132	122.90	111.00	-151.36911316	60.66986806			
B-39	95.51	151.50	-151.35615280	60.65559414		2		B-133	125.63	105.50	-151.37244708	60.67540767			
B-40	120.84	101.50	-151.34721580	60.65547709				B-135	112.66	150.50	-151.35586806	60.64679349	1		
B-42	118.85	101.50	-151.35029920	60.65453131	1			B-136	13.42	94.00	-151.35727753	60.64689374			
B-43	121.14	102.80	-151.34362450	60.65405368				B-137	116.64	115.50	-151.35941780	60.65248997	1		
B-45	109.09	101.50	-151.35759480	60.65465742				B-138	103.22	146.00	-151.34969539	60.65926678		2	
B-47	119.73	151.00	-151.36972090	60.66227016	1			B-146	111.74	217.30	-151.36648264	60.66243655	1		
B-49	127.76	101.50	-151.36274610	60.67526763	1			B-147	119.69	200.30	-151.36747798	60.66209855			
B-50	131.66	151.50	-151.35335930	60.67524580		2		B-148	122.26	206.10	-151.36726271	60.66278039			
B-60	135.49	81.00	-151.34970518	60.67539889				B-149	126.30	223.00	-151.36585008	60.66265279			
B-61	137.58	96.00	-151.34332584	60.67530860				B-150	110.97	200.20	-151.36610352	60.66195569			
B-62	127.66	101.30	-151.36898112	60.67372335		2	1	B-151	125.85	201.00	-151.36476522	60.66571099			
B-64	133.91	86.00	-151.35383276	60.67397590			1	B-152	125.79	150.40	-151.36303312	60.66626550			
B-66	127.44	105.00	-151.37372952	60.67185480				B-153	126.81	150.60	-151.36397783	60.66511871			
B-67	124.49	110.50	-151.36405465	60.67196390				B-154	127.46	151.00	-151.36228714	60.66562743			
B-68	122.47	202.00	-151.37961780	60.67016196	1			B-155	126.76	151.00	-151.36228399	60.66502805	1		
B-69	123.86	105.50	-151.37590717	60.67015134				B-156	124.84	151.00	-151.36239760	60.66393399			
B-71	116.20	101.00	-151.36427266	60.66996116				B-157	124.51	151.00	-151.36066599	60.66448827	1		
B-72	135.28	86.30	-151.34710070	60.67027771	1		1	B-158	120.63	150.50	-151.36163461	60.66329807			
B-73	122.34	111.00	-151.37753990	60.66852118				B-159	118.84	150.30	-151.35991084	60.66385187			
B-74	111.99	151.00	-151.36389233	60.66824205	1	2	1	B-160	115.35	150.20	-151.36011160	60.66335236	1		
B-75	129.87	101.50	-151.36028017	60.66890221				B-161	105.03	156.00	-151.36001832	60.66225349			
B-76	130.47	101.00	-151.34362252	60.66829367				B-162	111.45	151.00	-151.35926363	60.66151597			
B-77	116.01	202.00	-151.37510864	60.66654928	1	2		B-163	104.85	150.90	-151.35752400	60.66210614			
B-78	118.09	101.00	-151.37192900	60.66654928				B-164	105.43	151.00	-151.35773753	60.66157129	1		
B-79	123.61	101.00	-151.36374290	60.66653811	1		1	B-165	103.54	150.30	-151.36510331	60.66011521			
B-80	130.85	100.50	-151.35803312	60.66653133		1	1	B-166	100.68	150.70	-151.36262002	60.66132084			
B-81	128.79	106.00	-151.35027766	60.66667672				B-167	111.91	150.30	-151.36390727	60.65921073			
B-82	116.34	150.50	-151.34337253	60.66650909		2	1	B-168	118.89	176.00	-151.36134711	60.66001705	1		
B-83	126.03	106.00	-151.36693500	60.66514200	1			B-169	122.82	151.00	-151.36179473	60.65885340			
B-84	127.53	100.80	-151.36123975	60.66503400			1	B-170	123.01	171.00	-151.35783392	60.65955956			
B-85	119.50	202.00	-151.37102452	60.66284303				B-171	116.53	151.00	-151.35643036	60.66026570	1		
B-86	124.85	101.00	-151.36453497	60.66300368	1	3	1	B-172	119.91	160.50	-151.35294376	60.66135351			
B-87	106.46	96.30	-151.35823947	60.66291477	1	1		B-173	121.02	171.00	-151.35167947	60.66013946			
B-88	93.98	101.30	-151.35380723	60.66288908				B-176	27.14	210.20	-151.39778916	60.68727972	1		
B-89	126.64	101.30	-151.35055076	60.66287057				B-177	29.39	51.00	-151.39577948	60.68747575			
B-90	110.48	101.00	-151.36189643	60.66213101				B-178	34.20	51.00	-151.39453464	60.68753301			
B-91	117.54	100.60	-151.36864403	60.66129503		2	1	B-179	43.66	51.00	-151.39339076	60.68760076			
B-92	118.03	99.80	-151.36624365	60.66135905				B-180	77.76	76.00	-151.39342768	60.68827819			
B-93	118.69	101.00	-151.35681706	60.66095091				B-181	87.19	76.50	-151.39334051	60.68889704			
B-94	127.39	101.00	-151.34346988	60.66106965				B-182	85.60	50.50	-151.39314642	60.68956559			
B-95	96.73	101.00	-151.36332415	60.66078281				B-183	98.76	101.10	-151.39333025	60.69015403			
B-96	97.65	101.00	-151.35473390	60.66054881				B-189	119.97	201.00	-151.36061126	60.65385926			
B-97	109.47	203.00	-151.36669900	60.65946066	1			B-190	20.56	97.10	-151.36215168	60.65382001			
B-98	115.67	101.00	-151.35997475	60.65951243		2	1	B-191	20.50	114.20	-151.37337267	60.66365831			
B-99	123.82	151.00	-151.34652491	60.65948995				B-192	123.19	239.30	-151.37209739	60.66371093			
B-101	94.98	152.00	-151.35402409	60.65744590	1			B-195	119.97	226.00	-151.36924661	60.66487249	1		
B-102	97.95	101.00	-151.35833603	60.65645887			1	B-197	122.74	249.60	-151.36922005	60.66381855			
B-104	95.22	105.90	-151.35258202	60.65547421				B-198	117.99	300.50	-151.36747007	60.66435767			

Notes:

[1]DH = Downhole Seismic Test [2]MW = Monitoring Well [3]TP = Test Pit

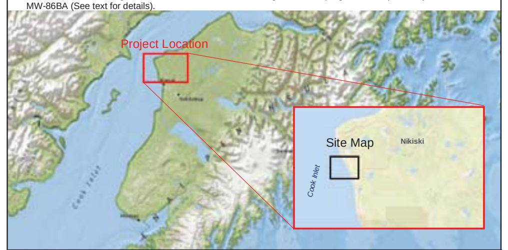
Monitoring Wells	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007) Longitude, deg	Latitude, deg
MW-14	133.70	56.00	-151.35376440	60.67001489
MW-27	124.30	57.00	-151.35037410	60.66459539
MW-39A	95.50	147.00	-151.35615710	60.65557926
MW-39B	95.30	40.00	-151.35613450	60.65560508
MW-50A	132.60	146.00	-151.35331470	60.67523060
MW-50B	132.20	55.50	-151.35336650	60.67526072
MW-62A	127.36	145.40	-151.36900760	60.67372740
MW-62B	127.63	47.00	-151.36896370	60.67373020
MW-74A	111.70	159.00	-151.36390300	60.66825110
MW-74B	111.53	81.00	-151.36390170	60.66822290
MW-77A	116.31	146.00	-151.37507200	60.66655740
MW-77B	116.25	61.00	-151.37506340	60.66654210
MW-80	130.99	61.70	-151.35796510	60.66660890
MW-82A	118.58	150.00	-151.34354410	60.66651740
MW-82B	119.22	52.00	-151.34358260	60.66654600
MW-86A	124.44	146.00	-151.36452040	60.66301340
MW-86B	124.48	42.00	-151.36451240	60.66299700
MW-87	106.91	52.50	-151.35826080	60.66293920
MW-91A	117.27	146.00	-151.36859480	60.66133530
MW-91B	117.31	64.00	-151.36858380	60.66131500
MW-98A	122.17	116.00	-151.36000880	60.65949110
MW-98B	122.26	43.00	-151.36003290	60.65952300
MW-112AA	115.55	119.00	-151.35844950	60.65126000
MW-112B	115.76	61.00	-151.35847780	60.65124200
MW-138A	103.22	146.80	-151.34969540	60.65926680
MW-138B	103.33	43.30	-151.34973370	60.65927620
APT-1	117.09	138.00	-151.34706092	60.66315390
APT-2	126.99	137.00	-151.34195457	60.66107404
APT-3	115.83	437.00	-151.34698760	60.66312928
OW-1	109.56	79.00	-151.34463703	60.66385170
OW-2	109.09	150.00	-151.34460741	60.66382579
OW-3	128.26	70.00	-151.34191656	60.66235654
OW-4	128.29	137.00	-151.34194479	60.66238108
MW-86BA	124.73	66.00	-151.36456512	60.66299257

Hand Augers	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007) Longitude, deg	Latitude, deg
HA-11	104.53	2.8	-151.3582393	60.65512561
HA-12	93.98	4.3	-151.3582455	60.65560946
HA-13	97.62	2.3	-151.3582056	60.65592101
HA-14	121.19	1.6	-151.3578491	60.65868729
HA-15	121.09	3.2	-151.3578434	60.65898341
HA-17	116.12	2.5	-151.3582332	60.66167632
HA-18	108.69	5.5	-151.3581713	60.66230348
HA-19	106.92	6.9	-151.3581512	60.66313564
HA-20	119.04	2.6	-151.3581436	60.66416493
HA-21	123.32	2.8	-151.3581355	60.66451619
HA-22	130.06	2.8	-151.3580616	60.66606764
HA-100	97.71	4.3	-151.3560797	60.65762929
HA-101	88.78	2.0	-151.3518266	60.65757639
HA-102	104.23	4.0	-151.3632097	60.6611113
HA-103	117.69	3.0	-151.3668833	60.66134432
HA-104	124.01	3.8	-151.3587445	60.6649634
HA-105	124.7	2.2	-151.3509738	60.66444029
HA-106	118.92	1.8	-151.3647449	60.66838243
HA-107	118.88	3.7	-151.3631376	60.66840602
HA-108	123.01	1.5	-151.3579906	60.65936729
HA-109	111.12	1.5	-151.3543773	60.65935399
HA-110	100.45	4.0	-151.3500841	60.65926121
HA-111	102.15	2.0	-151.3494033	60.65927181
HA-112	106.13	2.0	-151.3602101	60.662931
HA-113	125.64	2.4	-151.3486347	60.66288061
HA-114	105.52	2.7	-151.365746	60.65950795
HA-115	121.16	3.3	-151.371427	60.6650298
HA-116	109.59	5.0	-151.350813	60.65354365
HA-117	90.06	3.0	-151.3507027	60.65758828
HA-118	94.46	2.0	-151.3507445	60.65811632
HA-119	123.19	2.0	-151.350797	60.66501537

Test Pits	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007) Longitude, deg	Latitude, deg
TP-01	127.2	8	-151.36890605	60.67382958
TP-02	133.08	8	-151.35371804	60.67405024
TP-03	134.43	5	-151.35223907	60.67264108
TP-04	134.56	8	-151.34703124	60.67029871
TP-05	111.72	8	-151.36376102	60.66825015
TP-06	122.7	8	-151.36376305	60.66658642
TP-07	130.61	8	-151.35798402	60.66653967
TP-08	127.05	8	-151.36120253	60.66502775
TP-09	123.74	8	-151.35036700	60.66453403
TP-10	124.36	8	-151.36432332	60.66300351
TP-11	117.18	8	-151.36840024	60.66138217
TP-12	122.53	8	-151.35995040	60.65957943
TP-13	97.98	8	-151.35843823	60.65650048
TP-14	97.02	11	-151.35507881	60.65339243

Notes:

1. Base imagery provided by EXP Geomatics (EXP).
2. Onshore LNG Facilities Study Area provided by AKLNG.
3. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
4. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc. (2015, 2016).
5. Groundwater monitoring well MW-86B was abandoned during 2016 G&G program and replaced by MW-86BA (See text for details).



This document may only be used for the purpose for which it was commissioned and in accordance with the terms of engagement for that commission. Unauthorized use of this document in any form whatsoever is prohibited.

GEODETIC INFORMATION

PROJECTION:	Transverse_Mercator
FALSE EASTING:	1640416.666666666
FALSE NORTHING:	0.0
CENTRAL MERIDIAN:	-150.0
SCALE FACTOR:	0.9999
LATITUDE OF ORIGIN:	54.0
LINEAR UNIT:	US Foot
SPHEROID:	GRS 1980
SEMI-MAJOR AXIS:	6,378,137.000
SEMI-MINOR AXIS:	6,356,752.314140356
INVERSE FLATTENING:	298.257222101

N W E S

Coordinate Grid: NAD 1983 (NSRS 2007), Degrees Minutes Seconds

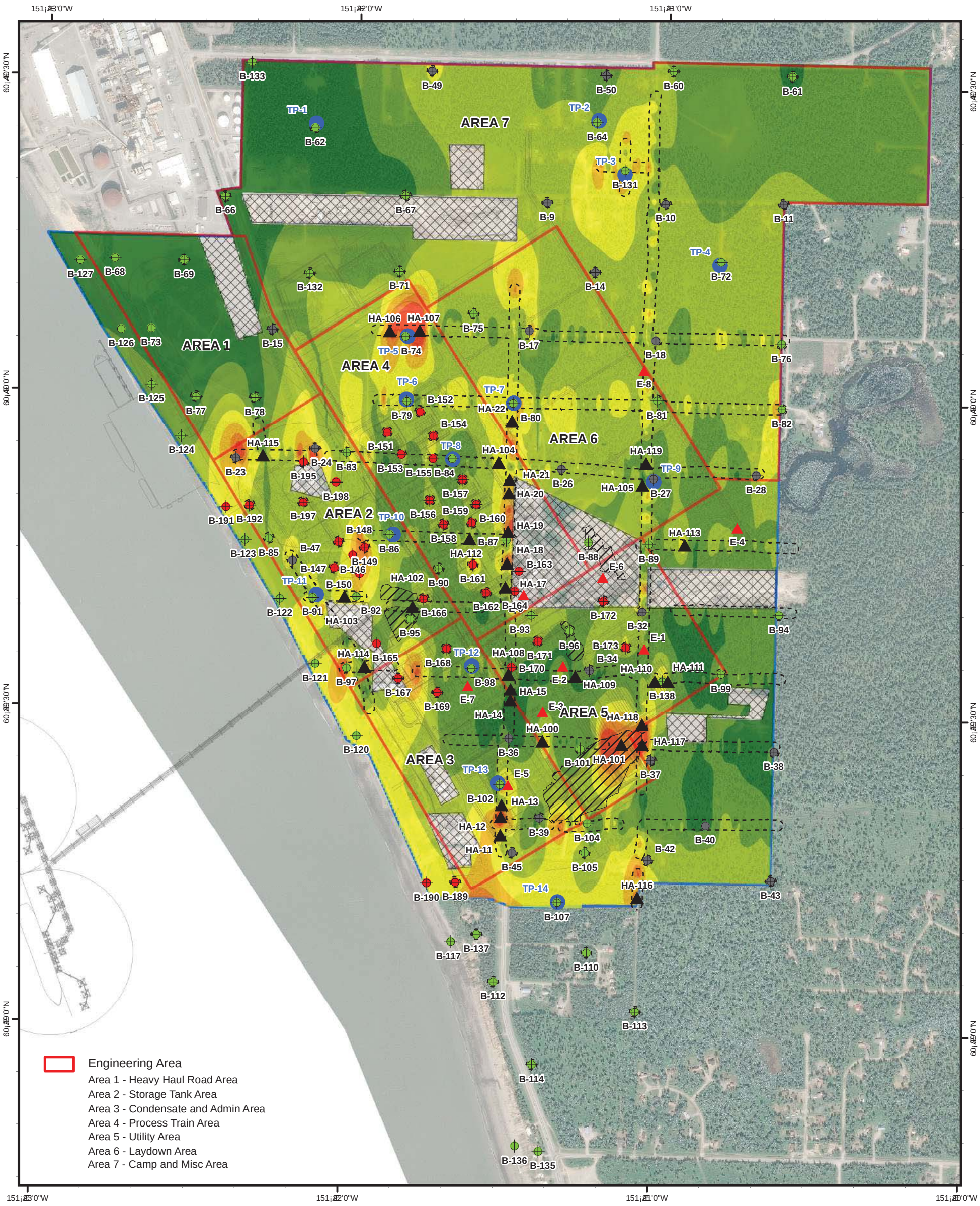
FUGRO CONSULTANTS, INC.
6100 Hillcroft Ave.
Houston, Texas 77081
Tel: 713.369.5400
Fax: 713.369.5518
www.fugroconsultants.com

Alaska LNG

PLAN OF EXPLORATIONS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

REV NO:	DATE:	DESCRIPTION:	DRAWN:	CHKD:	APPR:
0	11/29/2016	Onshore LNG Facilities Plan of Exploration	KIMK	BB	

JOB NUMBER: 04.10160001-12 DRAWING NO: PLATE 3b



LEGEND

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Boring (2016)
- Boring (2015)
- Boring (2014)
- VES Location (2016)
- Test Pit (2015)
- Hand Auger (2015)
- Low-Lying Areas with possible loose slits/soft clays.
- Developed Area with No Topsoil

Topsoil Thickness (ft)

- 0.07 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- 4.0 - 5.0
- 5.0 - 6.0
- 6.0 - 12

0 500 1,000 2,000 Feet

NOTES:

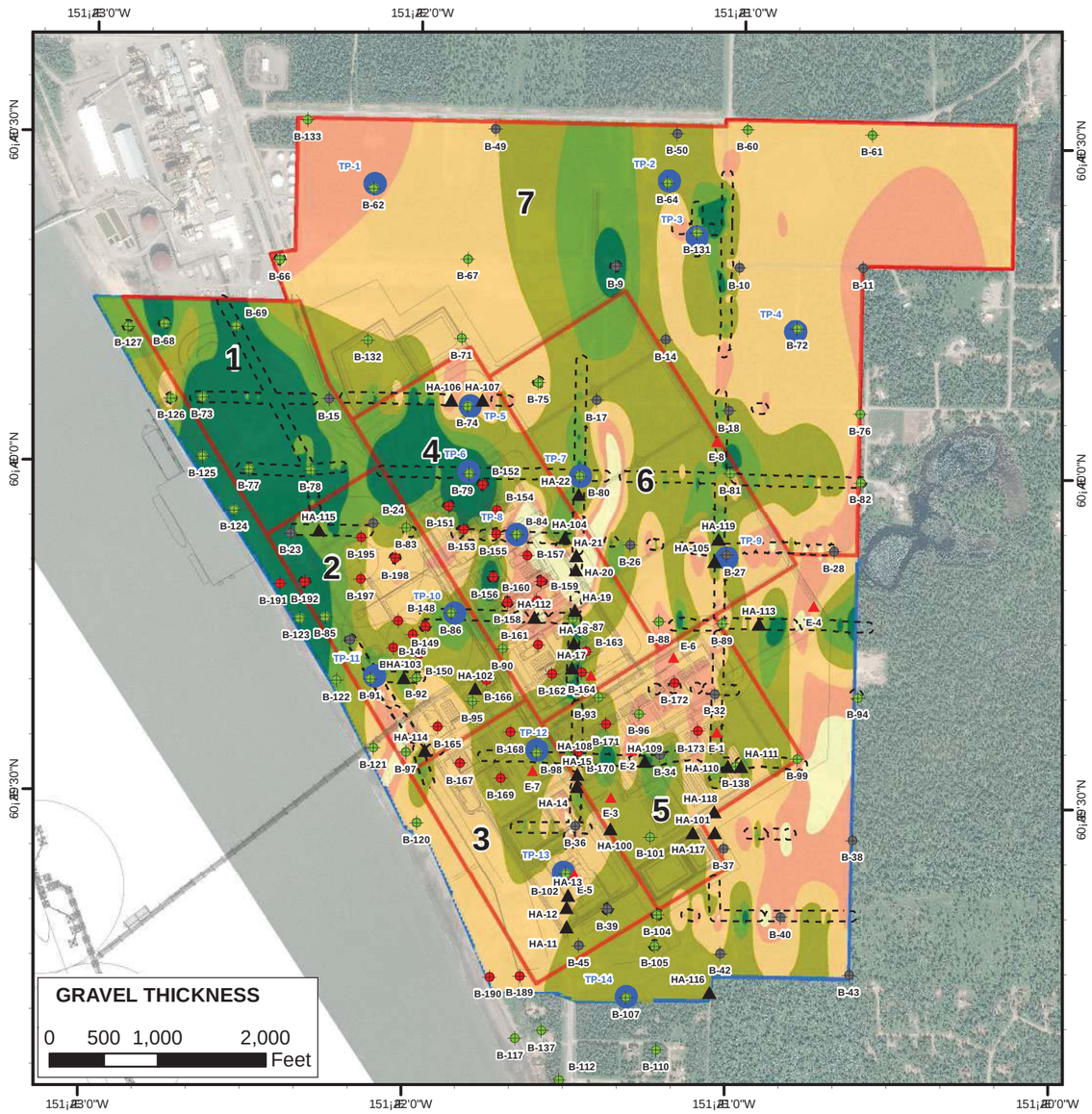
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated on June 30, 2016.
3. Topsoill thickness is approximate and is based on borings and ERT data.
4. Black dashed-polygons represent 50ft radius buffer zones of individual ERT and borings used for interpretation.
5. Interpolation among various data sets is approximate and is based on boring data and limited ERT survey information (100 ft wide area along ERT survey lines only).
6. This plate should be used for information only and not as warranty of subsurface conditions.

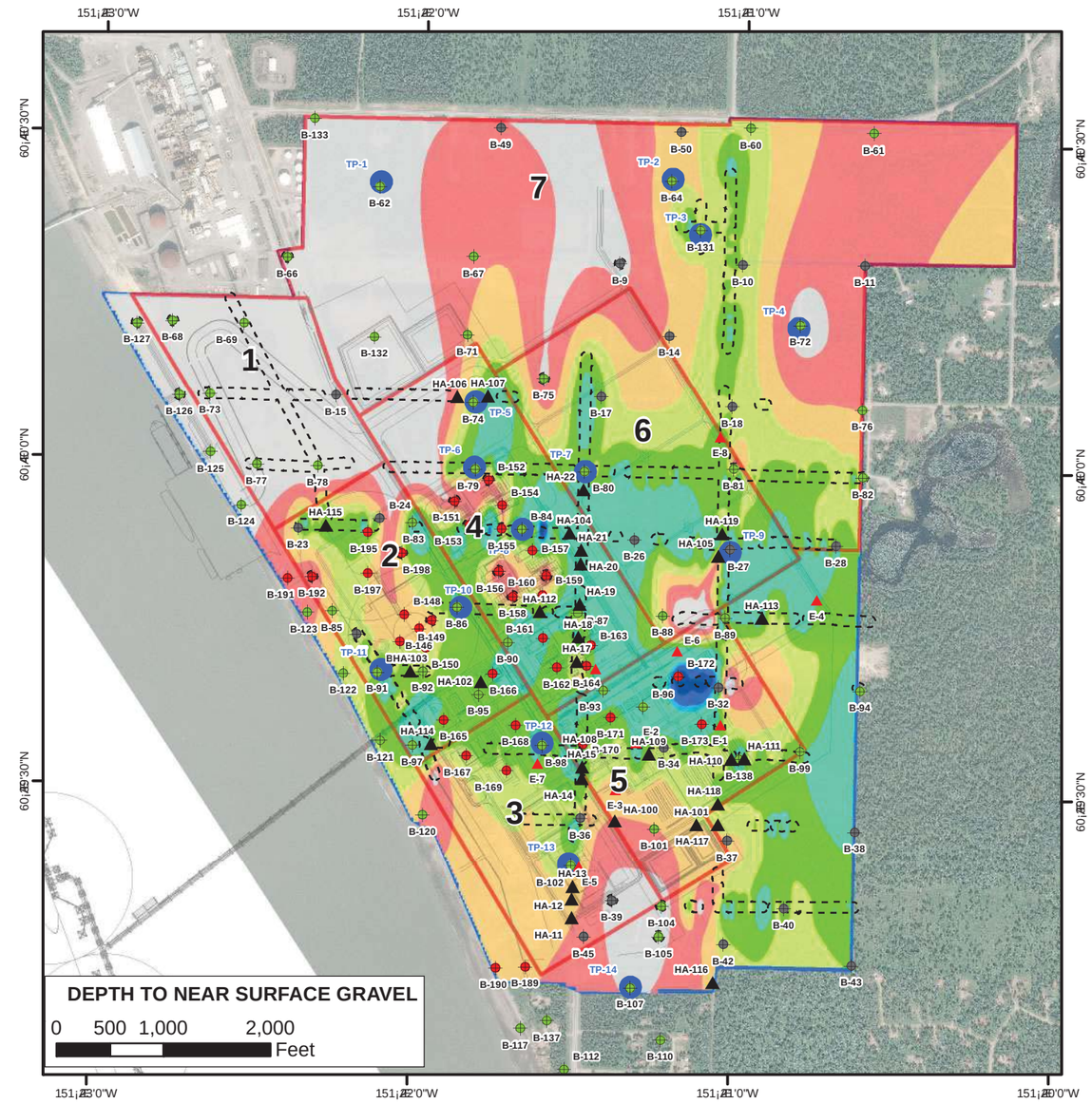
INTERPRETED TOPSOIL AND SOFT SOIL THICKNESS

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 4a

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-4a_InterpretedTopSoilThickness.mxd, 1/31/2017, kink





LEGEND

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 - Heavy Haul Road Area
- Area 2 - Storage Tank Area
- Area 3 - Condensate and Admin Area
- Area 4 - Process Train Area
- Area 5 - Utility Area
- Area 6 - Laydown Area
- Area 7 - Camp and Misc Area
- Boring (2016)
- Boring (2015)
- Boring (2014)
- VES Location (2016)
- Test Pit (2015)
- Hand Auger (2015)

Gravel Thickness (ft)

- 0 - 3
- 3 - 6
- 6 - 9
- 9 - 12
- 12 - 15
- 15 - 42

Depth to Gravel (ft)

- 0 - 3
- 3 - 6
- 6 - 9
- 9 - 12
- 12 - 15
- 15 - 18
- 18 - 21
- 21 - 24
- 24 - 42

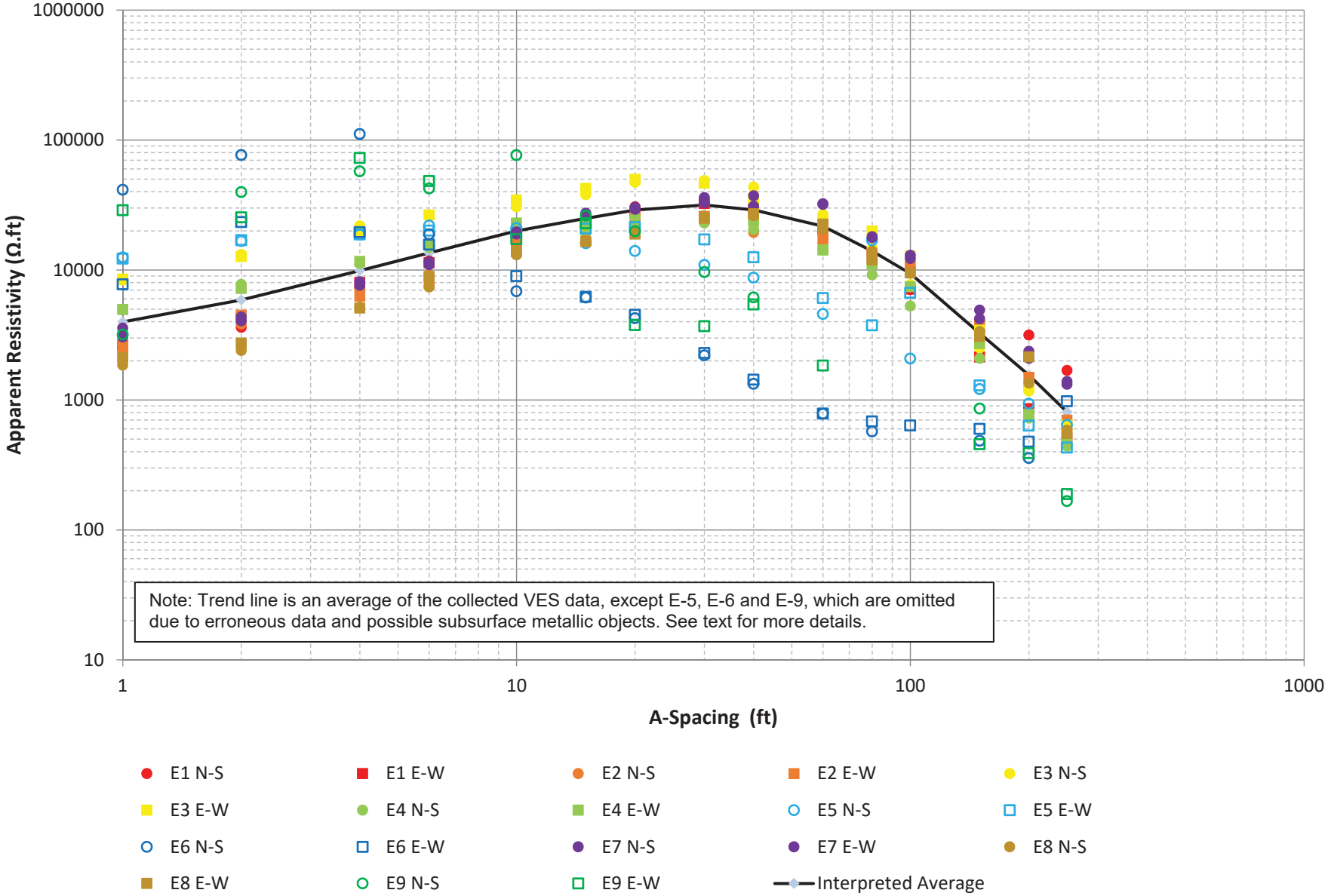
NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated on June 30, 2016.
3. Gravel thickness and depth to near surface gravel interpretation are approximate and are based on borings and ERT data.
4. Black dashed-polygons represent 50ft radius buffer zones of individual ERT and borings used for interpretation.
5. Interpolation among various data sets is approximate and is based on boring data and limited ERT survey information (100 ft wide area along ERT survey lines only).
6. This plate should be used for information only and not as warranty of subsurface conditions.
7. To see the estimated thickness of gravel, please refer to Plates 10 through 15.

INTERPRETED GRAVEL THICKNESS AND DEPTH TO NEAR SURFACE GRAVEL
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

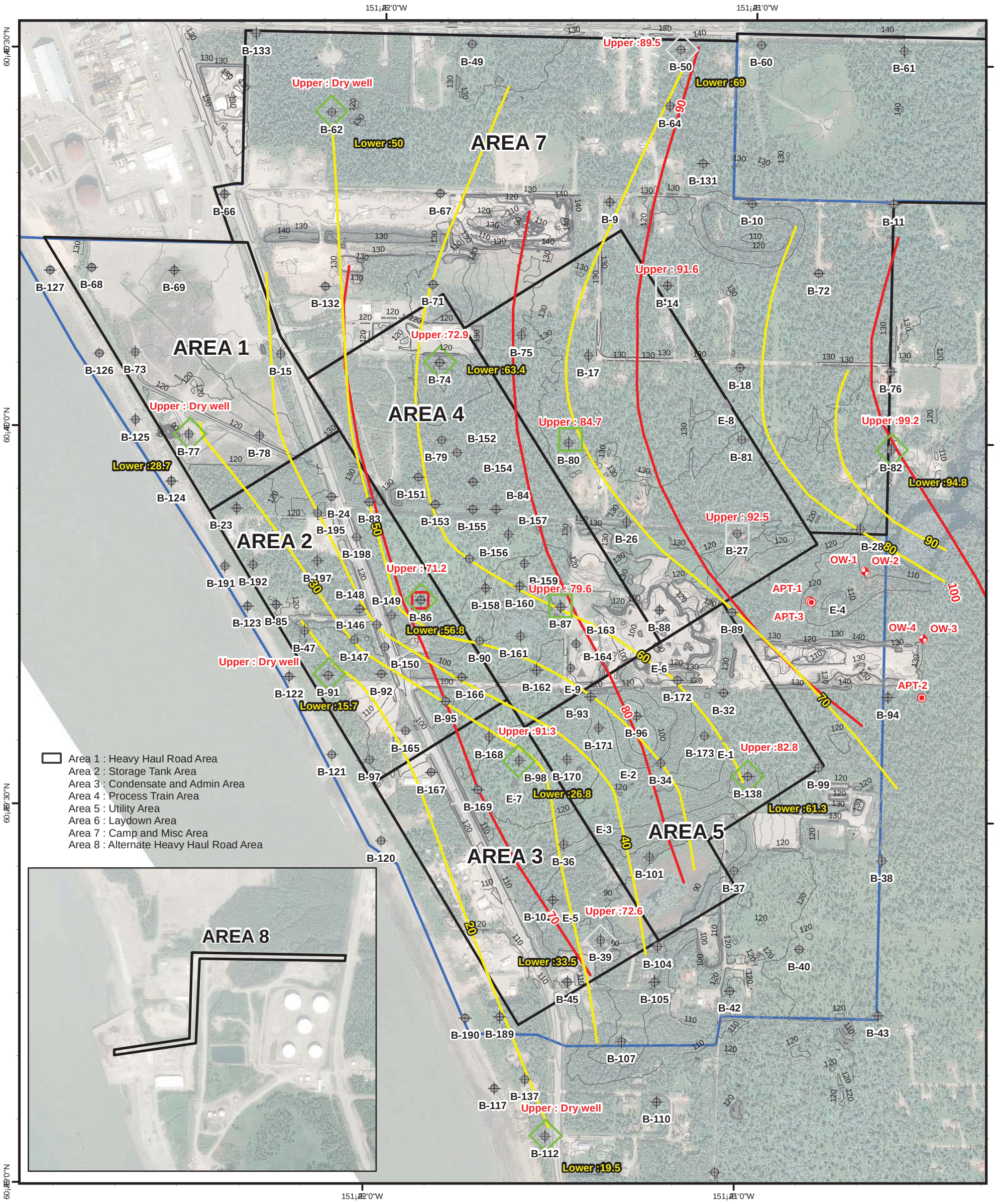
PLATE 4b

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-4b_InterpolatedGravelThickness.mxd, 1/17/2017, kimk



SUMMARY OF VERTICAL ELECTRICAL SOUNDING (VES) PROFILES

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



LEGEND

- Onshore LNG Facilities Study Area
- Boring (2014, 2015, 2016)
- VES Location (2016)
- APT Well (2016)
- Observation Well (2016)
- Monitoring Well - Shallow (2016)
- Monitoring Well - Shallow (2015)
- Monitoring Well - Shallow&Deep (2015)
- Monitoring Well - Shallow (2014)
- Monitoring Well - Shallow&Deep (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: Contour lines (McLane, 2015)
Major Contour interval is 10 feet
Minor Contour interval is 2 feet

Groundwater Well Water Elevations (NAVD88, Feet)

Upper : Shallow Well (Unit 1)
Lower : Deep Well (Unit 2)

Groundwater Elevation Contours (NAVD88, Feet)

Water Bearing Unit 1
Water Bearing Unit 2

NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Groundwater interpretations are based on the water level readings from groundwater wells on 9/22/16 at 6pm.

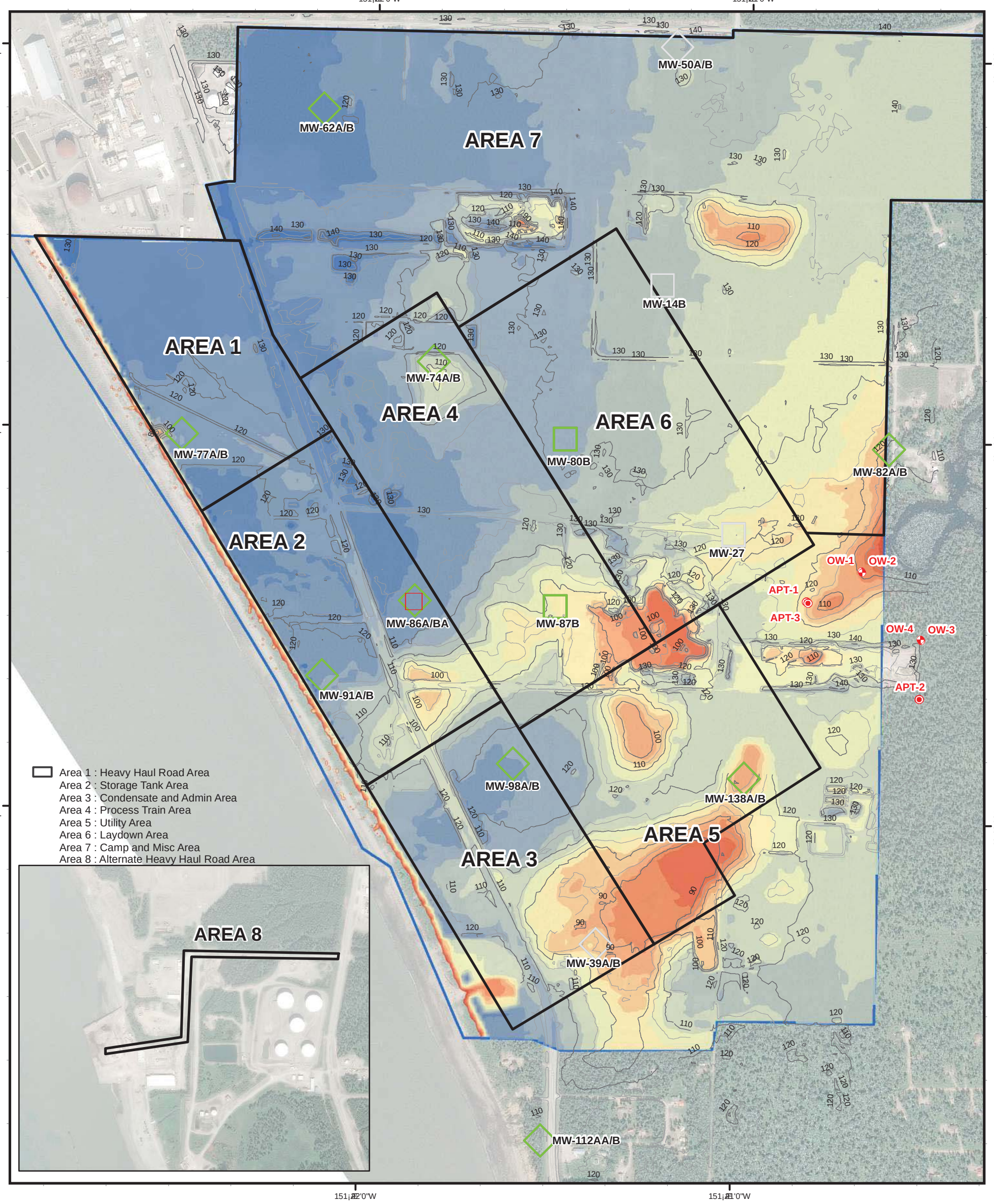
0 500 1,000 2,000 Feet

HYDROGEOLOGY GROUNDWATER INTERPRETATION

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 6a

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-06_HydroGeologyGWTInterpretation.mxd, 1/27/2017, kimk



LEGEND

- Onshore LNG Facilities Study Area
- APT Well (2016)
- Observation Well (2016)
- Monitoring Well - Shallow (2016)
- Monitoring Well - Shallow (2015)
- Monitoring Well - Shallow&Deep (2015)
- Monitoring Well - Shallow (2014)
- Monitoring Well - Shallow&Deep (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: Contour lines (McLane, 2015)
Major Contour interval is 10 feet
Minor Contour interval is 5 feet

Depth to First Encountered Water Surface (September 2016, ft)

0 - 5	35 - 40
5 - 10	40 - 45
10 - 15	45 - 50
15 - 20	50 - 55
20 - 25	55 - 60
25 - 30	60 - 65
30 - 35	

NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Groundwater interpretations are based on the water level readings from groundwater wells on 9/22/16 at 6pm.

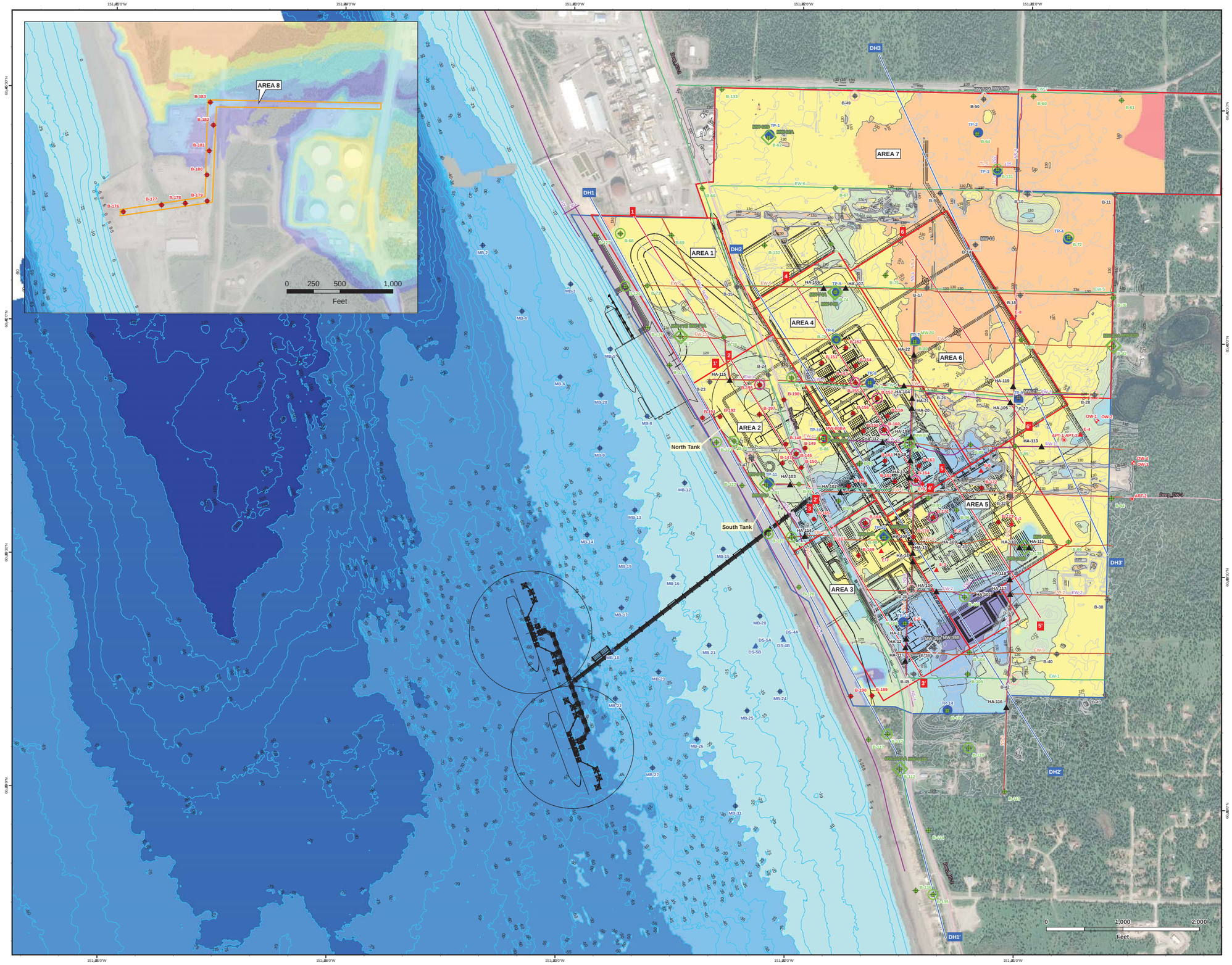
0 500 1,000 2,000 Feet

INTERPRETATION OF GROUNDWATER DEPTH (WATER BEARING UNIT 1)

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 6b

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\99_WORKING\01162017_DepthToGroundWaterInterpretation_ver2\Plate-6b_DepthToGroundWaterInterpretation_ver2\2017, kimk



LEGEND

- Onshore LNG Facilities Study Area
- Engineering Area

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- APT Well (2016)
- Observation Well (2016)
- VES Location (2016)
- Hand Auger (2015)
- Test Pit (2015)
- Marine Boring (2015)
- Drive Sampling (2015)
- Monitoring Well - Shallow (2016)
- Monitoring Well - Shallow (2015)
- Monitoring Well - Shallow&Deep (2015)
- Monitoring Well - Shallow (2014)
- Monitoring Well - Shallow&Deep (2014)
- Downhole Geophysics (2016)
- Downhole Geophysics (2015)
- Downhole Geophysics (2014)

CROSS SECTION LINES

- Cross Section Lines - Downhole Seismic
- Cross Section Lines - Integrated Geophysical and Stratigraphy Profiles

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet
- 80 - 90ft
- 90 - 100ft
- 100 - 110ft
- 110 - 120ft
- 120 - 130ft
- 130 - 140ft
- 140 - 180ft

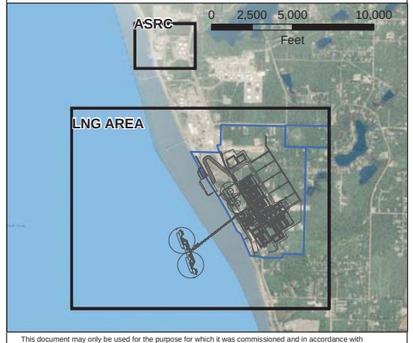
BATHYMETRY (MLLW, Feet)

Source: Multibeam (Fugro, 2014 and 2015)

- Contour interval is 1 feet
- -149 ~ -125
- -125 ~ -100
- -100 ~ -75
- -75 ~ -50
- -50 ~ -25
- -25 ~ 0

Notes:

1. Base imagery provided by EXP Geomatics (EXP).
2. Onshore LNG Facilities Study Area provided by AKLNG.
3. Onshore Facilities plot plan is provided by CB&I/CH!YODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
4. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc. (2015, 2016).
5. Groundwater monitoring well MW-86B was abandoned during 2016 G&G program and replaced by MW-86BA (See text for details).
6. Topographic source data in LNG Facilities Study Area is provided by McLane Consulting, Inc. (2015).
7. Topographic source data in ASRC area is from Kenai Peninsula Borough LiDAR collected in 2008.
8. Bathymetry contour lines based on data collected by Fugro (2014 & 2015).



This document may only be used for the purpose for which it was commissioned and in accordance with the terms of engagement for that commission. Unauthorized use of this document in any form whatsoever is prohibited.

GEODETIC INFORMATION

PROJECTION:	Transverse_Mercator
FALSE EASTING:	1640416.666666666
FALSE NORTHING:	0.0
CENTRAL MERIDIAN:	-150.0
SCALE FACTOR:	0.9999
LATITUDE OF ORIGIN:	54.0
LINEAR UNIT:	US Foot
SPHEROID:	GRS 1980
SEMI-MAJOR AXIS:	6,378,137.000
SEMI-MINOR AXIS:	6,356,752.314140356
INVERSE FLATTENING:	298.257222101

N W E S

Coordinate Grid: NAD 1983 (NSRS 2007), Degrees Minutes Seconds

FUGRO CONSULTANTS, INC.
6100 Hillcroft Ave.
Houston, Texas 77081
Tel: 713.369.5400
Fax: 713.369.5518
www.fugroconsultants.com

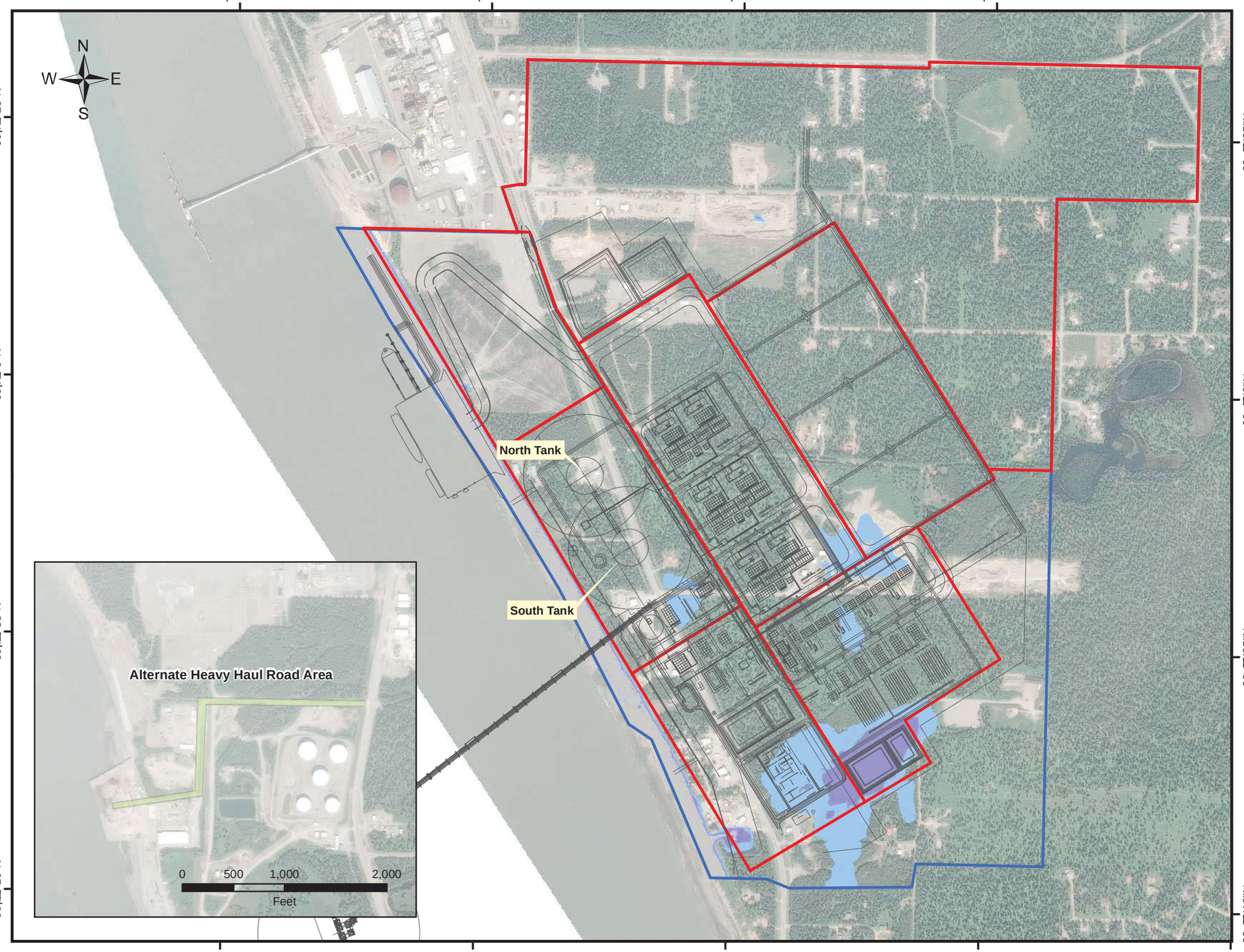
Alaska LNG

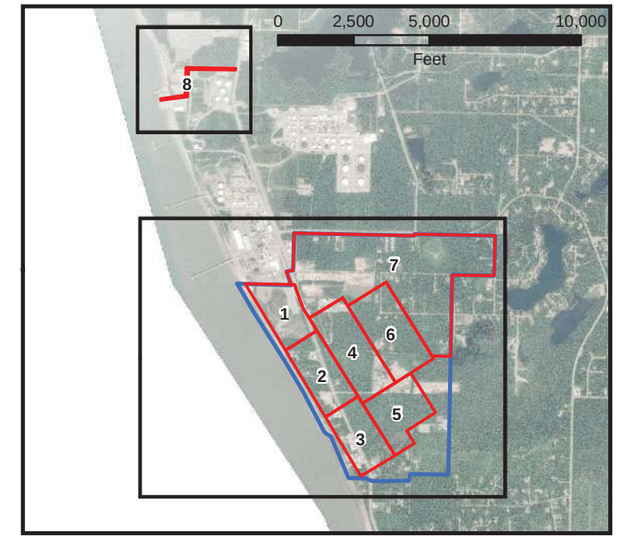
SITE TOPOGRAPHY MAP
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

REV NO.	DATE:	DESCRIPTION:	DRAWN:	CHKD:	APPR:
0	11/30/2016	Site Topography Map	KIMK	BB	

JOB NUMBER: 04.10160001-12

DRAWING NO. PLATE 7a





KEY MAP

Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Engineering Area

TOPOGRAPHY (NAVD88, Feet)

- 80 - 90ft
- 90 - 100ft

NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. Topographic source data in LNG Facilities Study Area is provided by McLane Consulting, Inc. (2015).

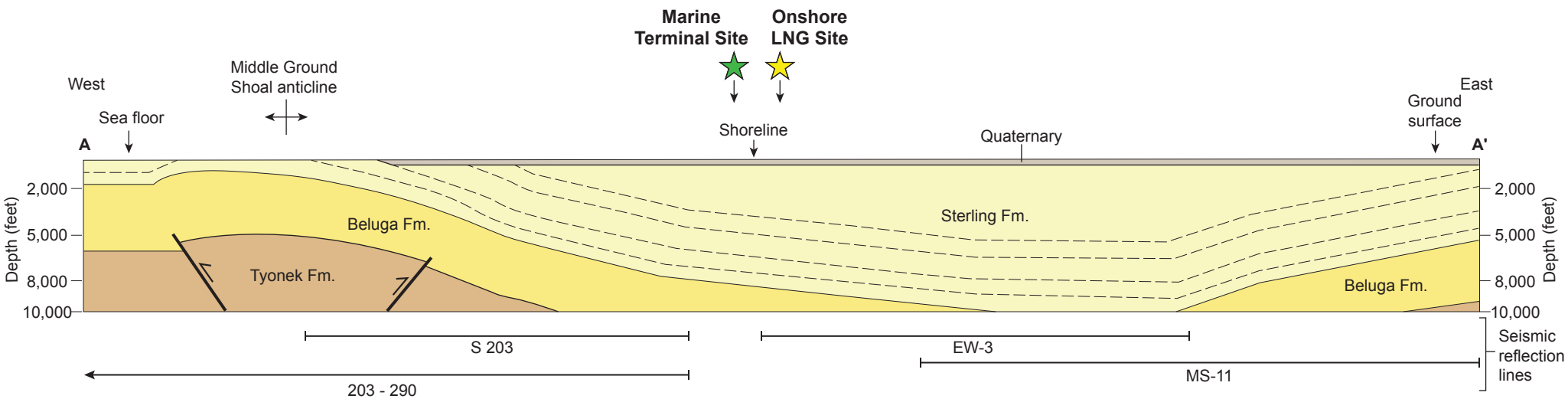
0 750 1,500 3,000
Feet

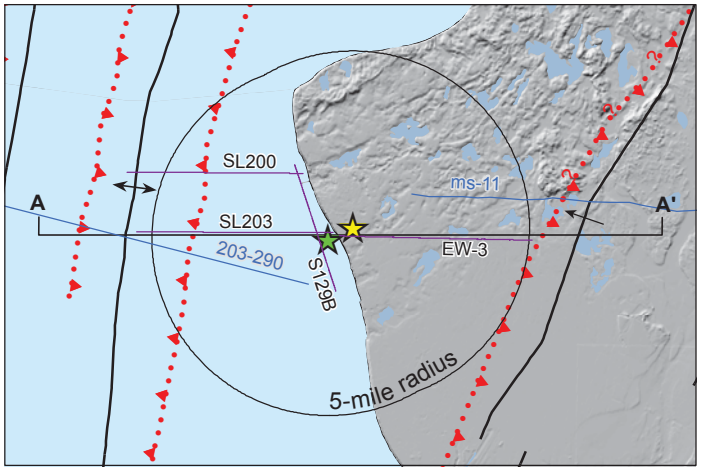
Grid Coordinate System: NAD83 NSRS2007

LOW-LYING AREAS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 7b

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-07b_SiteTopographyMap.mxd, 1/31/2017, kimk





Legend

- Blind thrust fault, teeth on upper plate, queried where uncertain
- Monocline axis
- Anticline axis
- Onshore LNG site center
- Marine terminal site center
- A — A' Structural cross section line
- SL200 Seismic line, project
- 203-290 Seismic line, industry

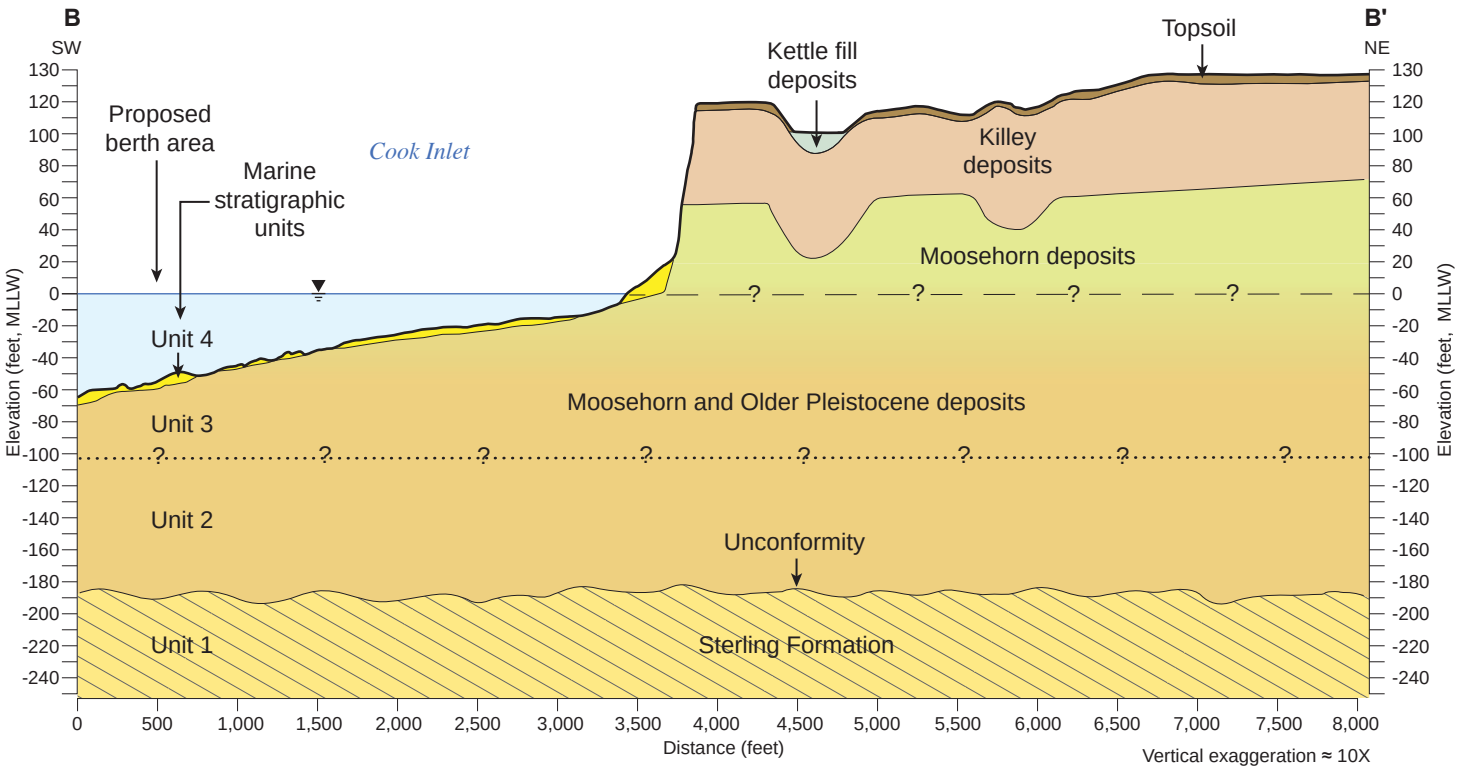
0 — 2 mi.
0 — 5,000 — 10,000 feet

No vertical exaggeration

GEOLOGIC CROSS-SECTION OF THE SITE AREA

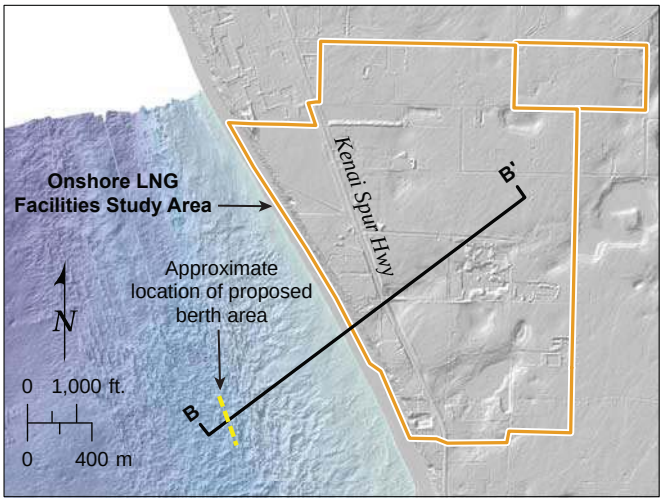
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 8a



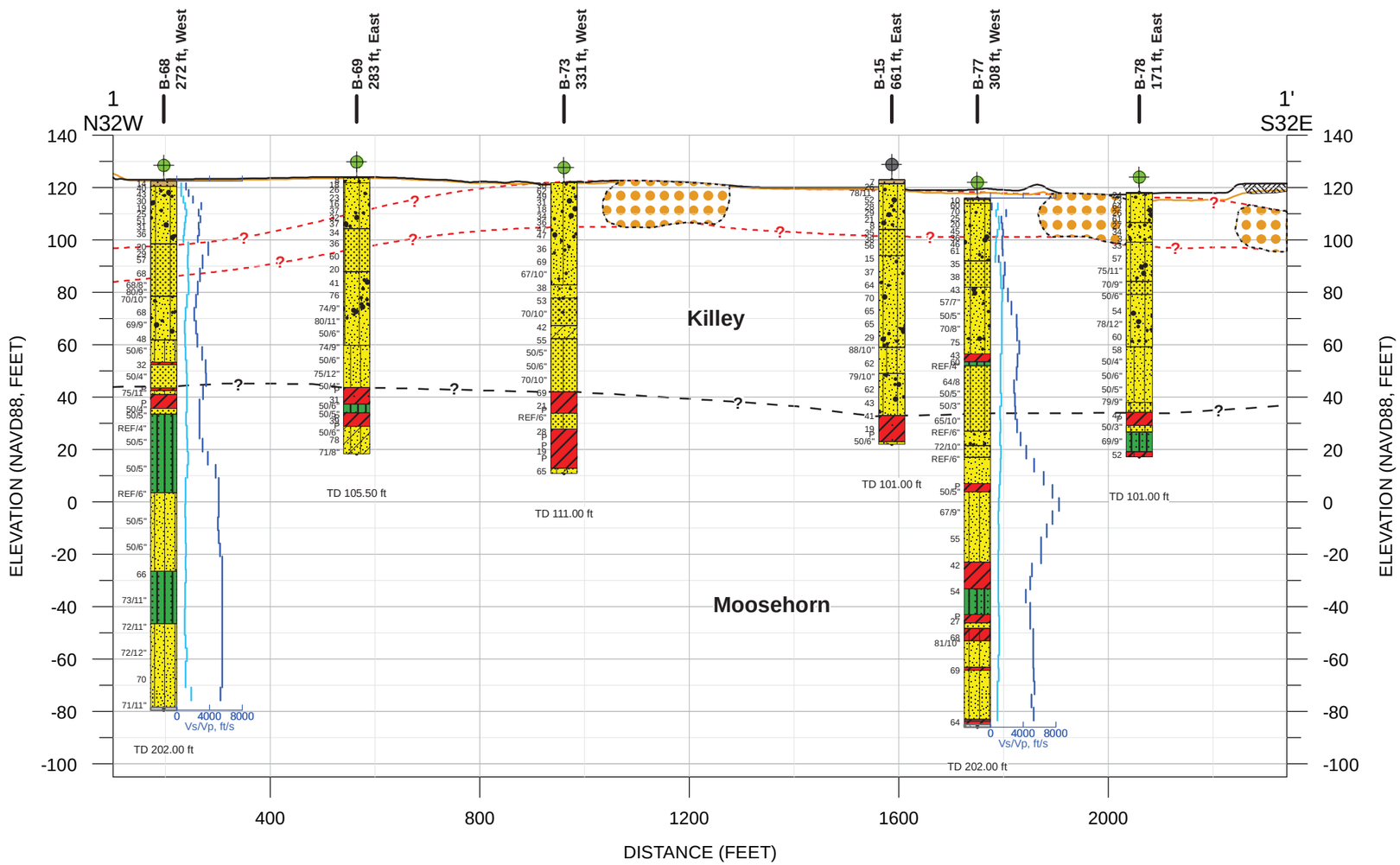
Notes:
1. Dashed line marks the top of Pleistocene deposits that have not been dated or formally assigned to a glacial stade.
2. Dotted line extends the contact between Units 2 and 3 identified from marine geophysical data, assuming that the contact would continue beneath the bluff. The contact could not be confirmed by onshore geophysical data due to lack of high resolution imaging in this depth interval.

Period	Epoch	Unit name	Marine unit #	Description
Quaternary	Holocene (modern)	Beach and sea floor sediment	4	Sand, silt, and gravel deposited by modern currents and waves
	Late Pleistocene to Holocene (17,500 to Present)	Topsoil	Not present	Roots, humus, loess, and tephra
		Kettle fill deposits	Not present	Peat, tephra, diatomaceous earth
	Late Pleistocene (18,500 to 17,500 yr)	Killey deposits	Not present	Fluvial glacial outwash of sand, gravel, and minor silt
	Late Pleistocene (32,000 to 18,500 yr)	Moosehorn deposits	Not present	Subestuarine glacially derived silt, clay, sand, and gravel
	Pleistocene (2 million to 18,500 yr)	Moosehorn and older Pleistocene deposits	2, 3	Glacial and glacially derived silt, clay, sand, and gravel
Tertiary	Pliocene	Sterling Formation	1	Fluvial sandstone, siltstone, mudstone, and coal



INTEGRATED SITE SCHEMATIC MODEL
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 8b



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation, NAVD88, Feet
- Topsoil plus clays/silts (symbolized only for 100ft wide area along ERT survey lines)
- Gravel (symbolized only for 100ft wide area along ERT survey lines)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY with Sand (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- SILT with Sand (ML)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND (SW)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Silty SAND (SM)
- Topsoil

50 ft
250 ft
Vertical Exaggeration = 5.0X

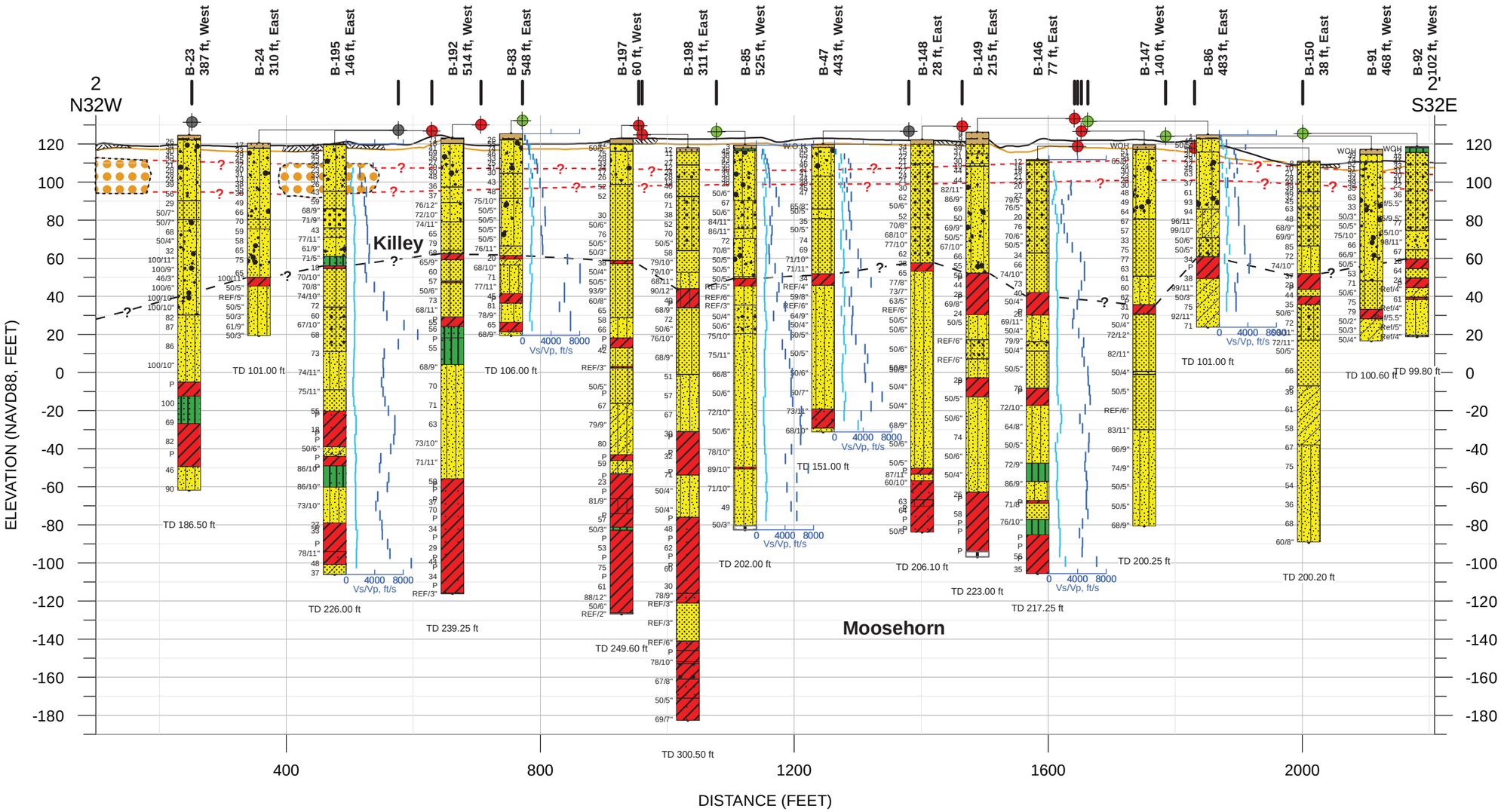
TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:
1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

INTEGRATED SUBSURFACE PROFILE - AREA 1
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\041000\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-09_IntegratedGeophysical_Xsect1.mxd, 1/17/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation, NAVD88, Feet
- Topsoil plus clays/silts (symbolized only for 100ft wide area along ERT survey lines)
- Gravel (symbolized only for 100ft wide area along ERT survey lines)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Silty CLAY with Sand (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Gravelly Lean CLAY (CL)
- Silt (ML)
- SILT with Sand (ML)
- Sandy, Gravelly SILT (ML)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Clay (SP-SC)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Gravelly, Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL with Silt (GP-GM)
- Sandy GRAVEL (GP)
- Sandy Well-Graded GRAVEL (GW)
- Sandy, Silty GRAVEL (GM)

TUBE AND SPT SAMPLES

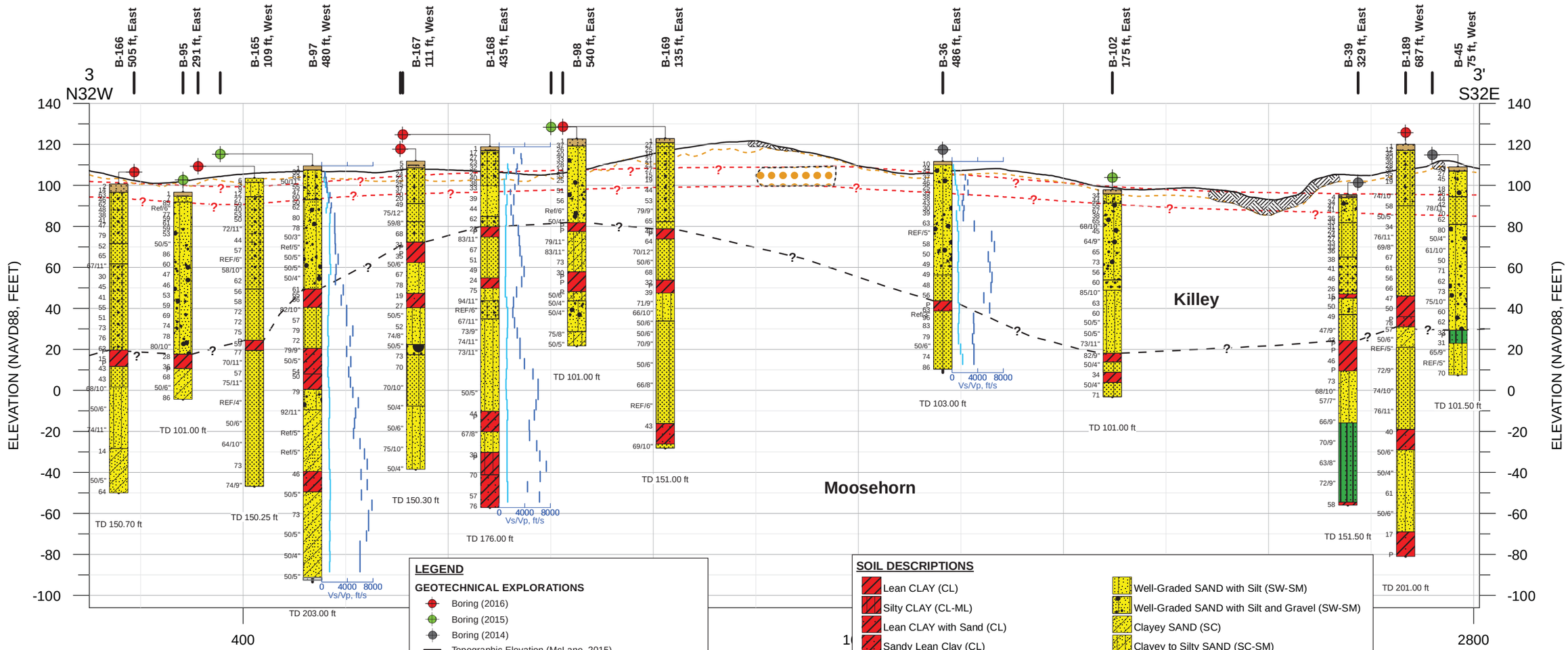
P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:
1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

60 ft
180 ft
Vertical Exaggeration = 3.0X

INTEGRATED SUBSURFACE PROFILE - AREA 2
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReportMXD\Plate-10_IntegratedGeophysical_Xsed2.mxd, 1/17/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)
- Topsoil plus clays/silts (symbolized only for 100ft wide area along ERT survey lines)
- Gravel (symbolized only for 100ft wide area along ERT survey lines)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL with Silt and Sand (GW-GM)

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

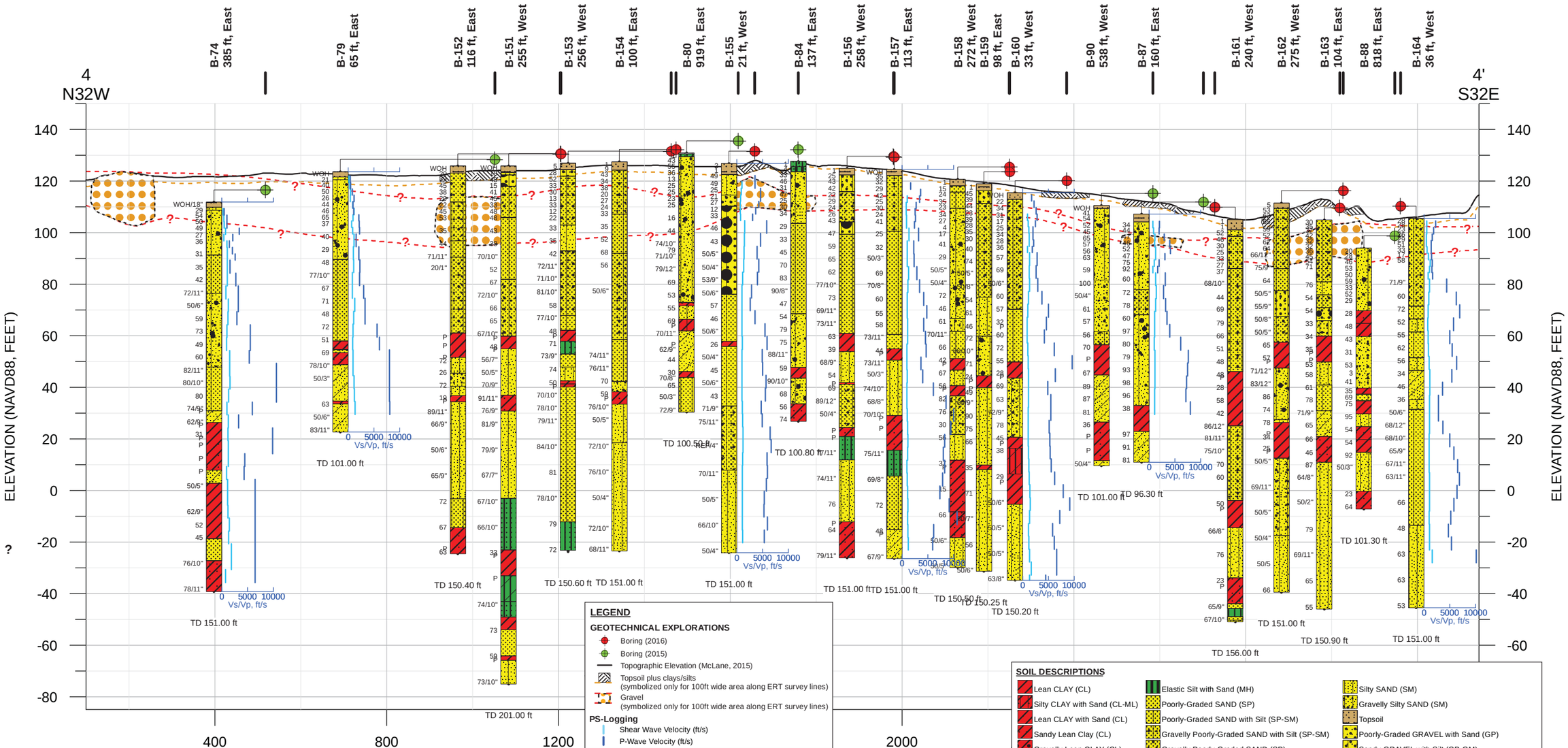
NOTES:

1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

50 ft
200 ft
Vertical Exaggeration = 4.0X

INTEGRATED SUBSURFACE PROFILE - AREA 3
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-11_IntegratedGeophysical_Xsect3.mxd, 1/17/2017, kimk



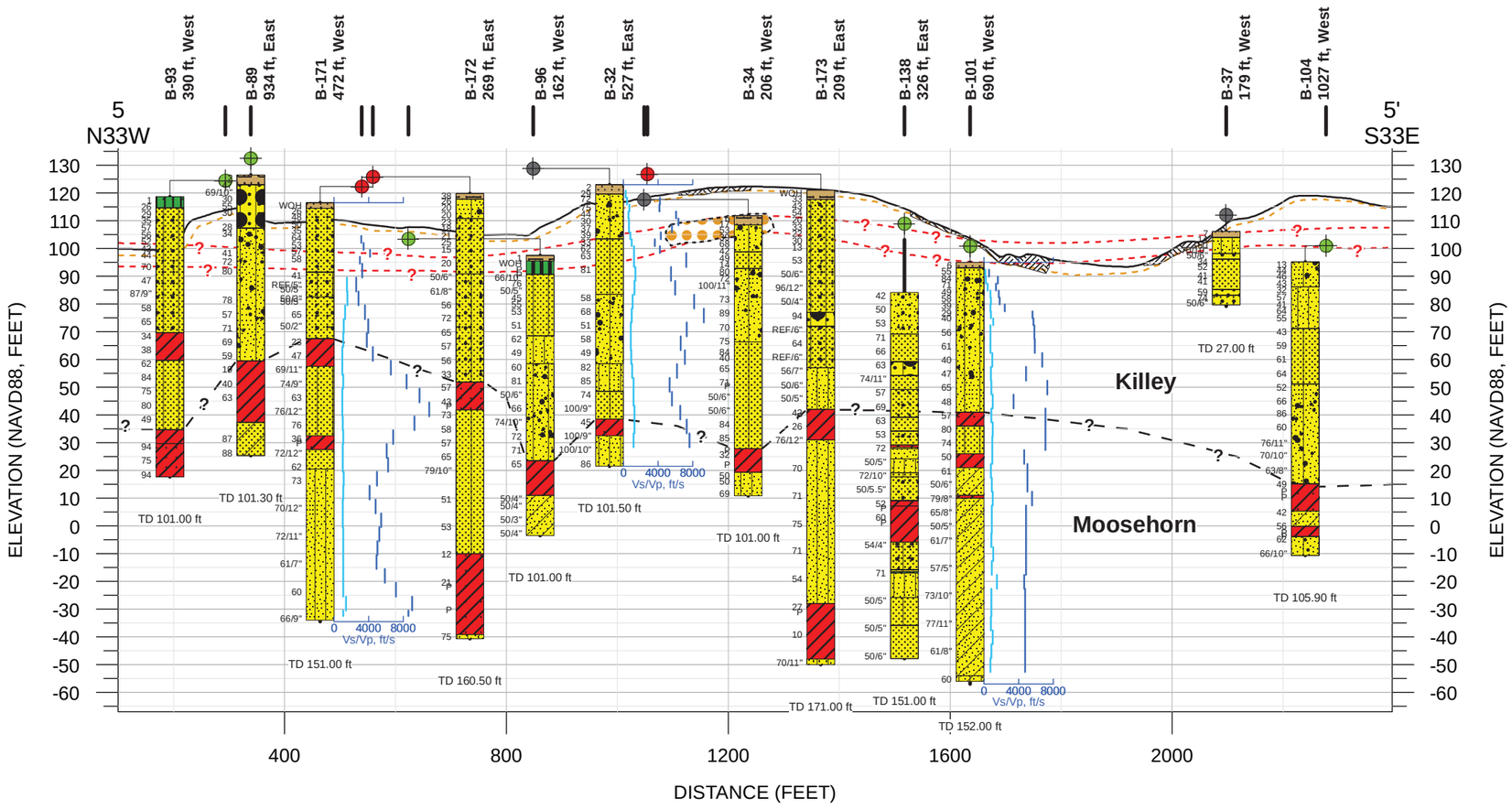
TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:
1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

INTEGRATED SUBSURFACE PROFILE - AREA 4
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-12_IntegratedGeophysical_Xsect4.mxd, 1/17/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)
- Topsoil plus clays/silts (symbolized only for 100ft wide area along ERT survey lines)
- Gravel (symbolized only for 100ft wide area along ERT survey lines)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- SILT with Sand (ML)
- Sandy SILT (ML)
- Elastic Silt (MH)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Clay (SP-SC)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND (SW)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL (GP)
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL with Silt and Sand (GW-GM)

TUBE AND SPT SAMPLES

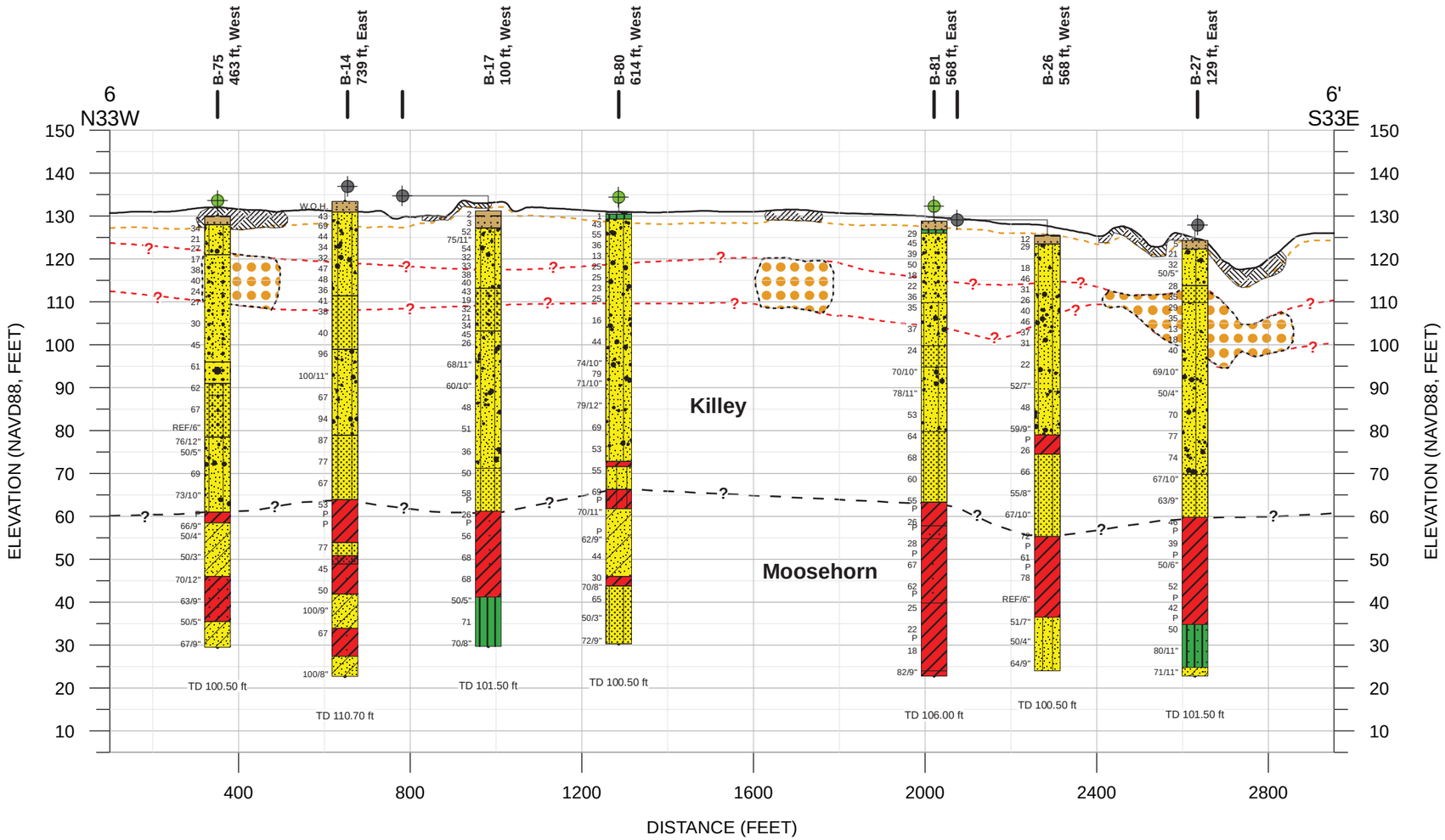
P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:
1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

50 ft
250 ft
Vertical Exaggeration = 5.0X

INTEGRATED SUBSURFACE PROFILE - AREA 5
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-13_IntegratedGeophysical_Xsec5.mxd, 1/17/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)
- Topsoil plus clays/silts (symbolized only for 100ft wide area along ERT survey lines)
- Gravel (symbolized only for 100ft wide area along ERT survey lines)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY with Sand (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Gravelly Lean CLAY (CL)
- Silt (ML)
- SILT with Sand (ML)
- Gravelly SILT (ML)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Well-Graded SAND with Clay (SW)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Topsoil
- Sandy Well-Graded GRAVEL (GW)
- Low-Plasticity Organic (OL)

30 ft
300 ft
Vertical Exaggeration = 10.0X

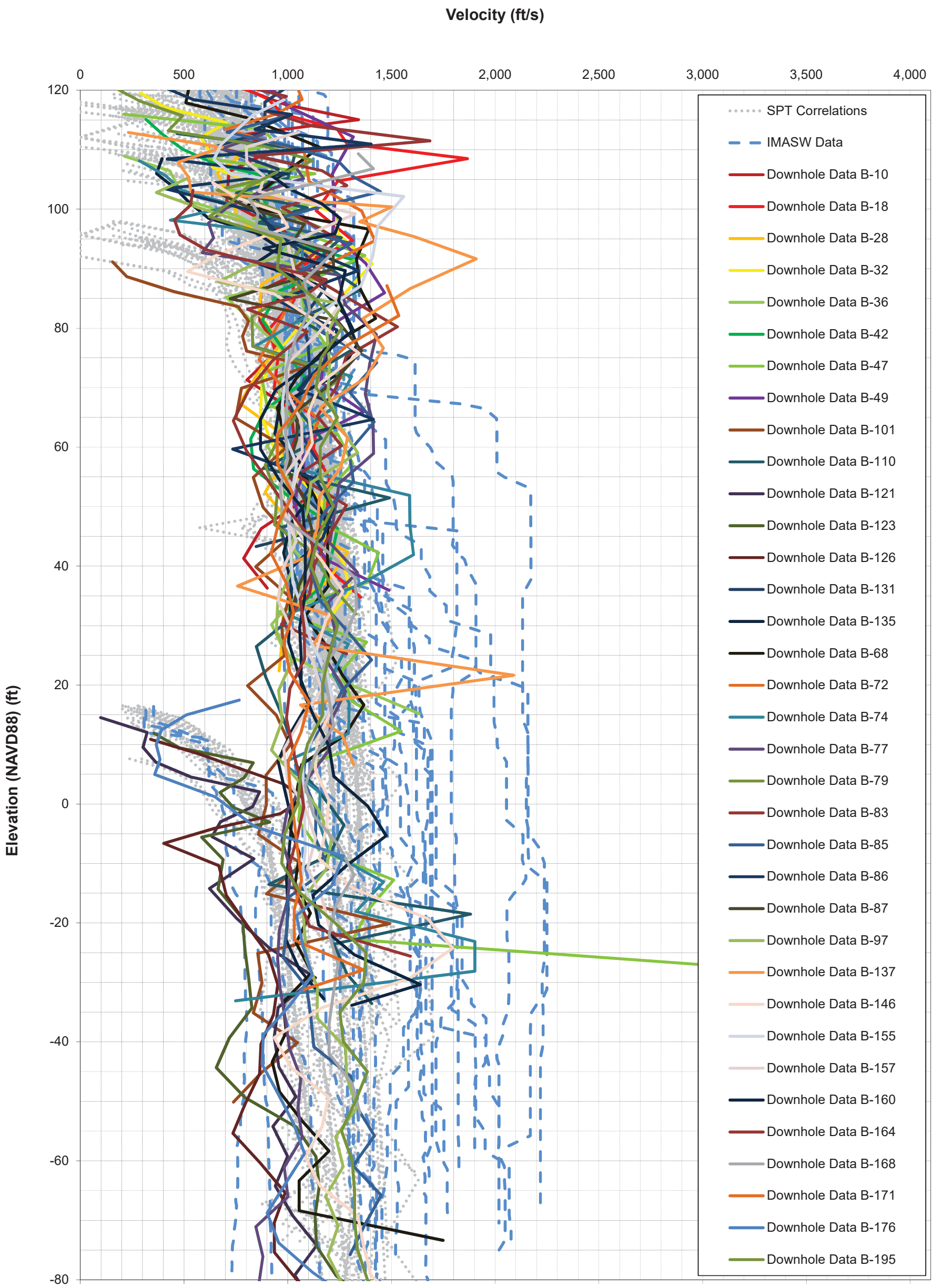
TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

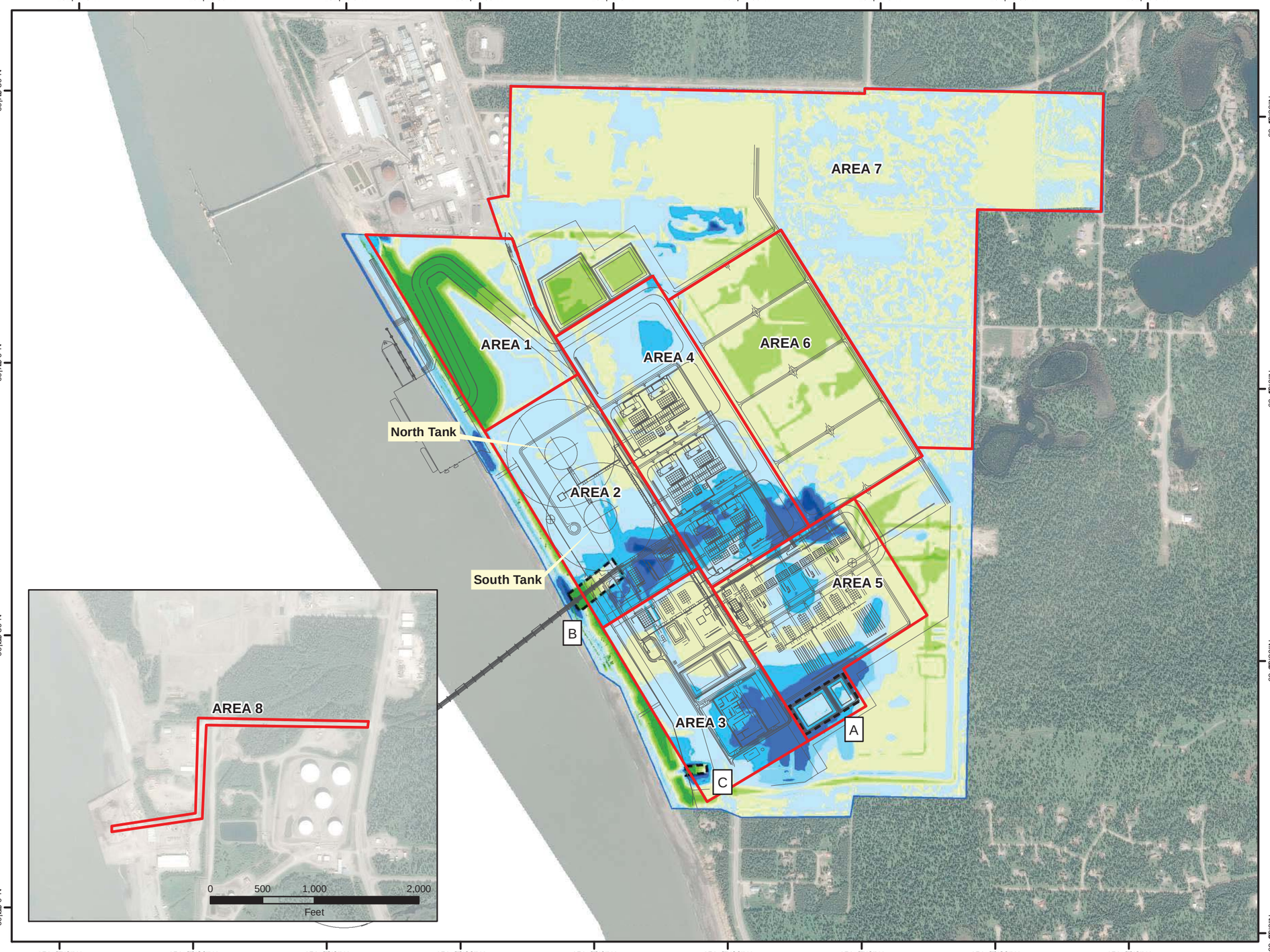
NOTES:
1. Topographic source data is provided by McLane Consulting, Inc.(2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the topsoil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 3b for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

INTEGRATED SUBSURFACE PROFILE - AREA 6
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-14_IntegratedGeophysical_Xsect6.mxd, 1/17/2017, kimk



IMASW, DOWNHOLE AND SPT-DERIVED SHEAR WAVE VELOCITY PROFILES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



Legend

Onshore LNG Facilities Study Area

Onshore Facilities Plot Plan

Design Areas (Design Grade Elevations, NAVD88)

Area 1 : Heavy Haul Road Area (27 ft - 124 ft)
Area 2 : Storage Tank Area (125 ft)
Area 3 : Condensate and Admin Area (116 ft)
Area 4 : Process Train Area (125 ft)
Area 5 : Utility Area (116 ft)
Area 6 : Laydown Area (122 ft)
Area 7 : Camp and Misc Area (100 ft to 145 ft)
Area 8 : Alternate Heavy Haul Road Area (N/A)

A : Area 5 Pond Earthwork (See note 5)
B : Area 2 Trestle Earthwork (See note 5)
C : Area 3 Outfall Earthwork (See note 5)

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 500 1,000 2,000
Feet

CUT AND FILL AREAS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-XX_CutAndFillAreas.mxd, 2/6/2017, kimk

Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

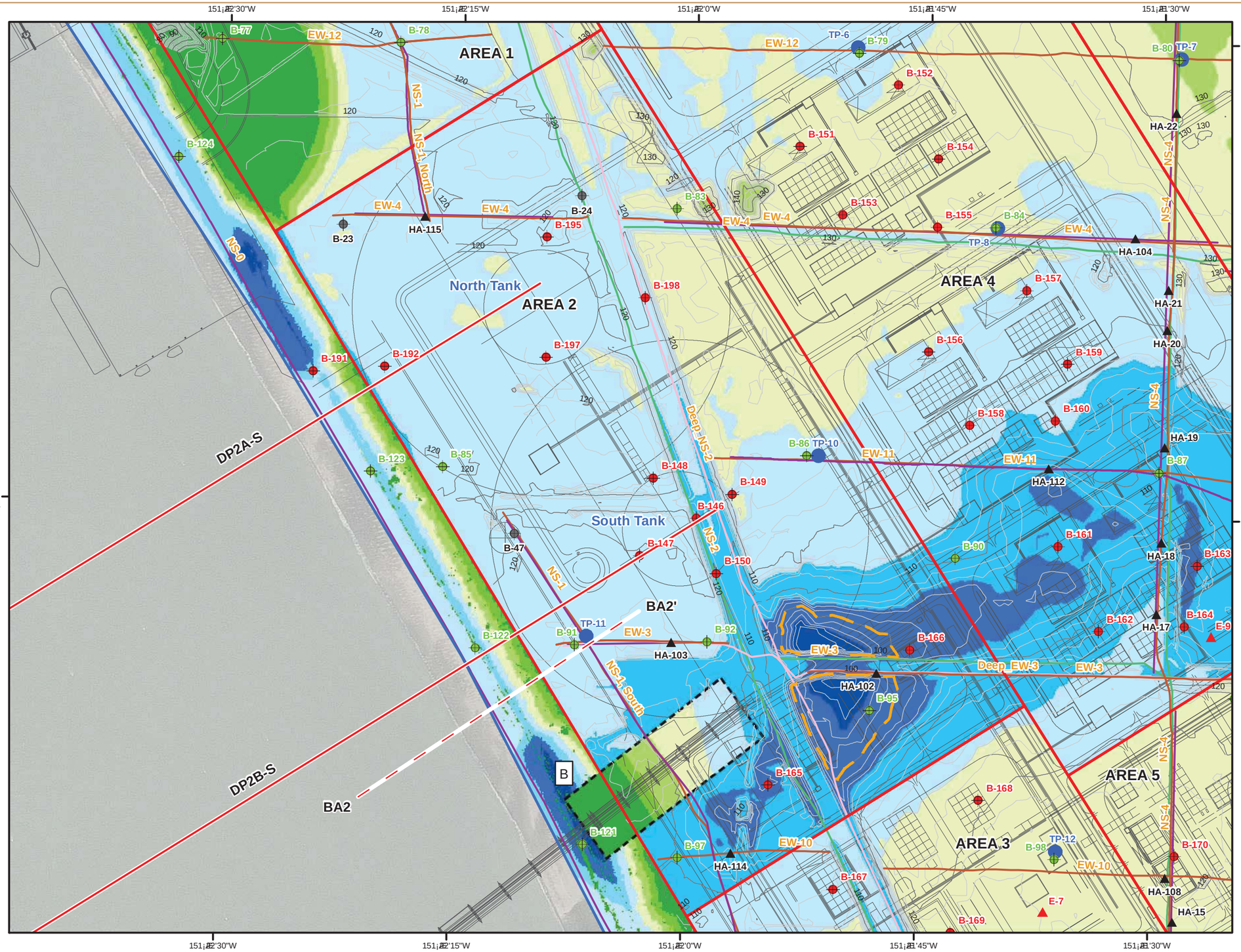
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

HEAVY HAUL ROAD AREA - 1
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17a

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-17a_CutAndFill_Area1.mxd, 2/6/2017, kimk



Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- A : Area 5 Pond Earthwork (See note 5)
- B : Area 2 Trestle Earthwork (See note 5)
- C : Area 3 Outfall Earthwork (See note 5)
- Low-Lying Areas with possible loose silts/soft clays
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

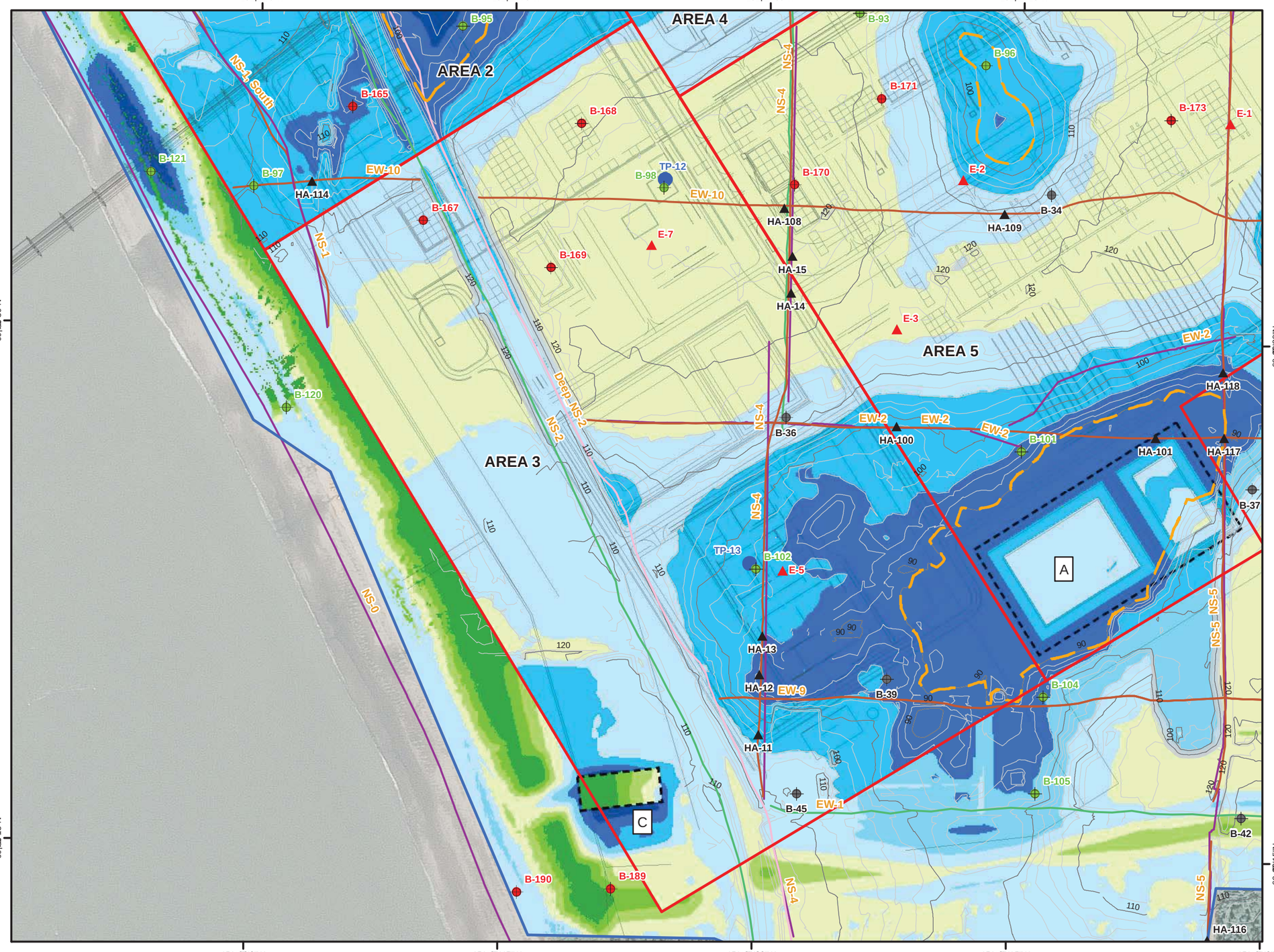
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

STORAGE TANK AREA - 2
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17b

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-17b_CutAndFill_Area2.mxd, 2/6/2017, kimk



Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- A : Area 5 Pond Earthwork (See note 5)
- B : Area 2 Trestle Earthwork (See note 5)
- C : Area 3 Outfall Earthwork (See note 5)
- Low-Lying Areas with possible loose silts/soft clays
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

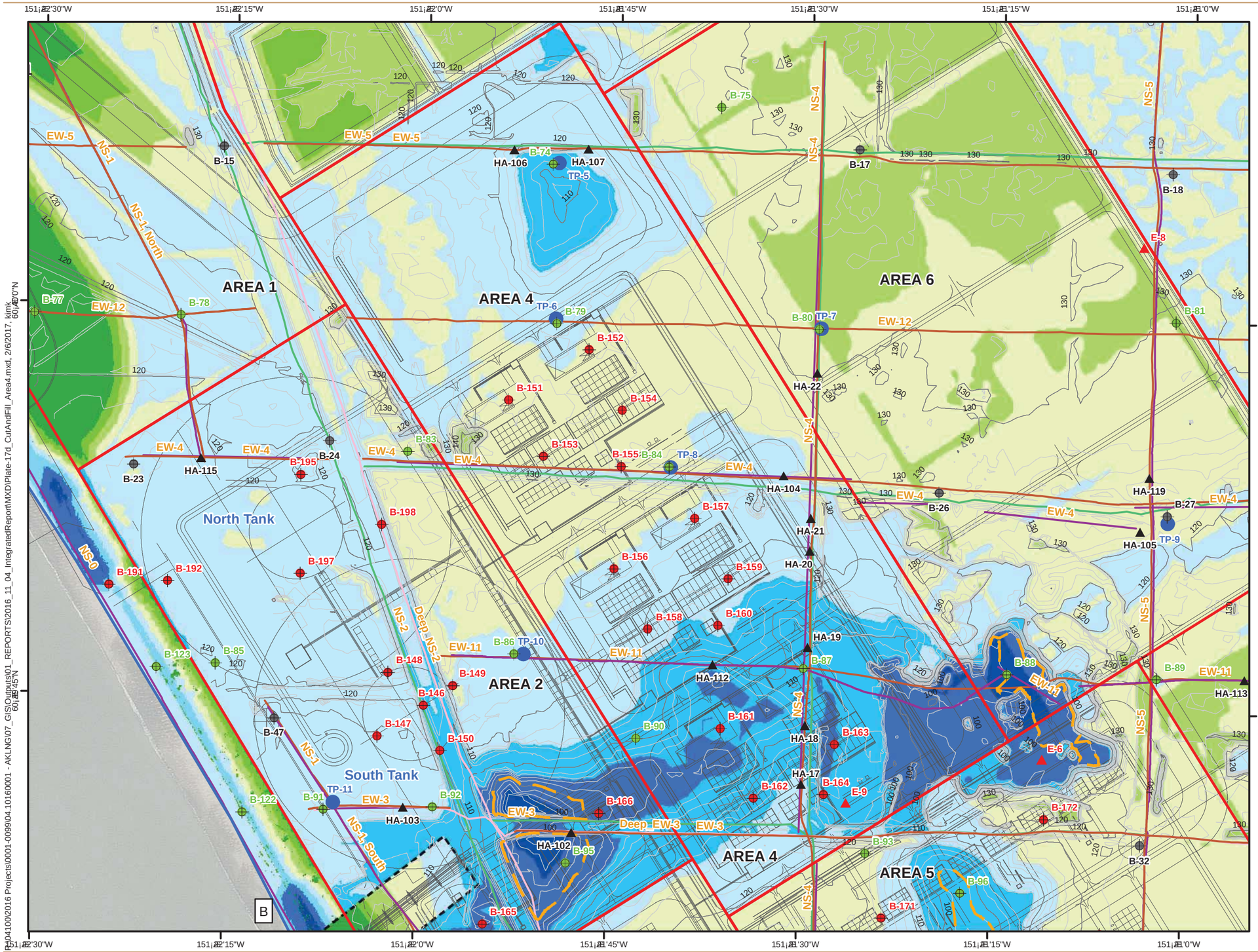
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

CONDENSATE AND ADMIN AREA - 3
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17c

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-17b_CutAndFill_Area2.mxd, 2/6/2017, kimk



Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- A : Area 5 Pond Earthwork (See note 5)
- B : Area 2 Trestle Earthwork (See note 5)
- C : Area 3 Outfall Earthwork (See note 5)
- Low-Lying Areas with possible loose silts/soft clays
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

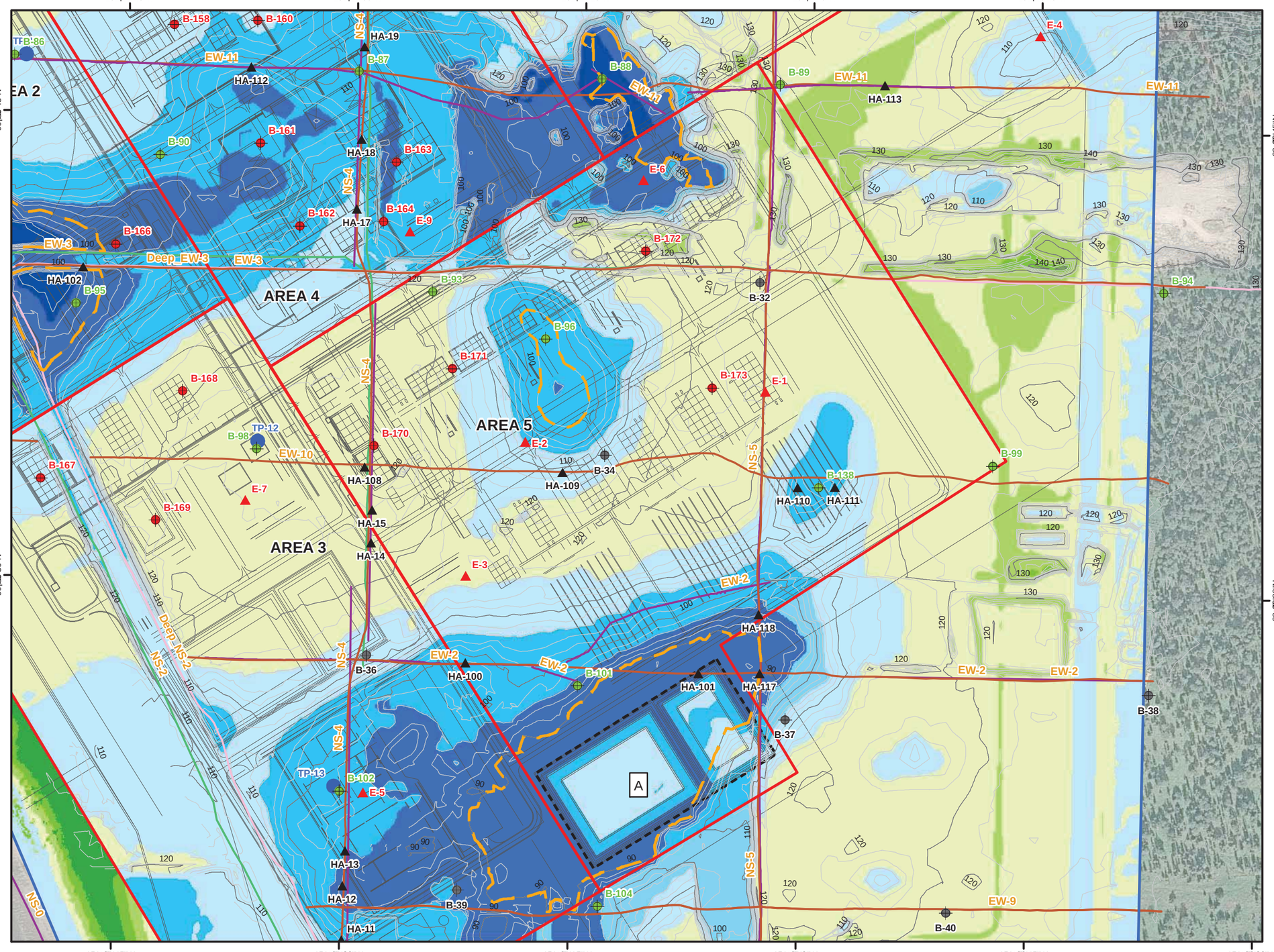
NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

PROCESS TRAIN AREA - 4
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17d



Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- A : Area 5 Pond Earthwork (See note 5)
- B : Area 2 Trestle Earthwork (See note 5)
- C : Area 3 Outfall Earthwork (See note 5)
- Low-Lying Areas with possible loose silts/soft clays
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

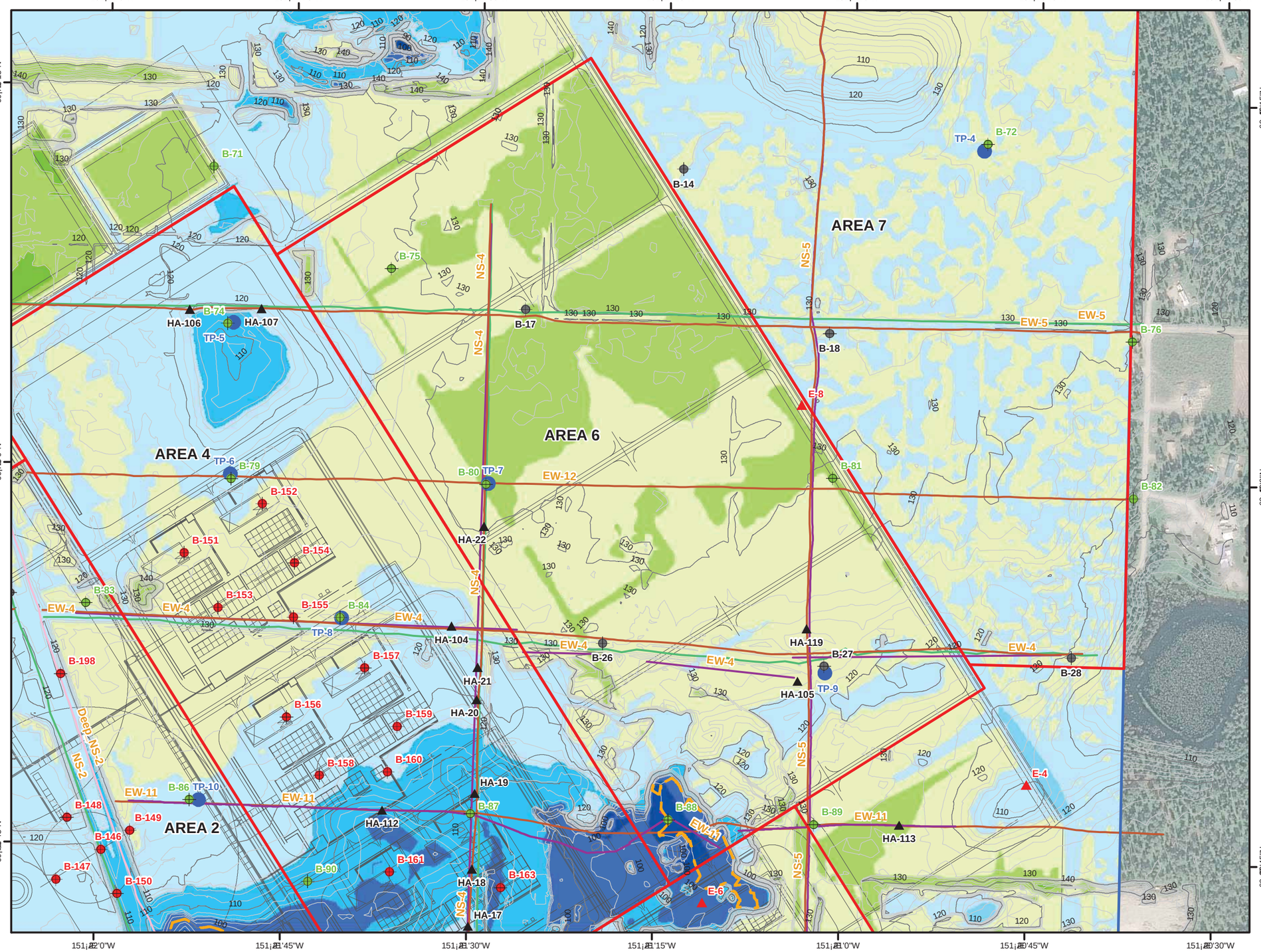
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

UTILITY AREA - 5
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17e

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-17e_CutAndFill_Area5.mxd, 2/6/2017, kimk



Legend

- Onshore LNG Facilities Study Area
- Onshore Facilities Plot Plan
- Area 1 : Heavy Haul Road Area
- Area 2 : Storage Tank Area
- Area 3 : Condensate and Admin Area
- Area 4 : Process Train Area
- Area 5 : Utility Area
- Area 6 : Laydown Area
- Area 7 : Camp and Misc Area
- Area 8 : Alternate Heavy Haul Road Area
- Low-Lying Areas with possible loose silts/soft clays
- Boring (2016)
- Boring (2015)
- Boring (2014)
- Test Pit (2015)
- Hand Auger (2015)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

TOPOGRAPHY (NAVD88, Feet)

Source: McLane, 2015

- Major Contour interval is 10 feet
- Minor Contour interval is 2 feet

Fill Thickness	Cut Thickness
> 30 ft	0 - 10 ft
20 ~ 30 ft	10 - 20 ft
10 ~ 20 ft	20 - 30 ft
0 ~ 10 ft	> 30 ft

NOTES:

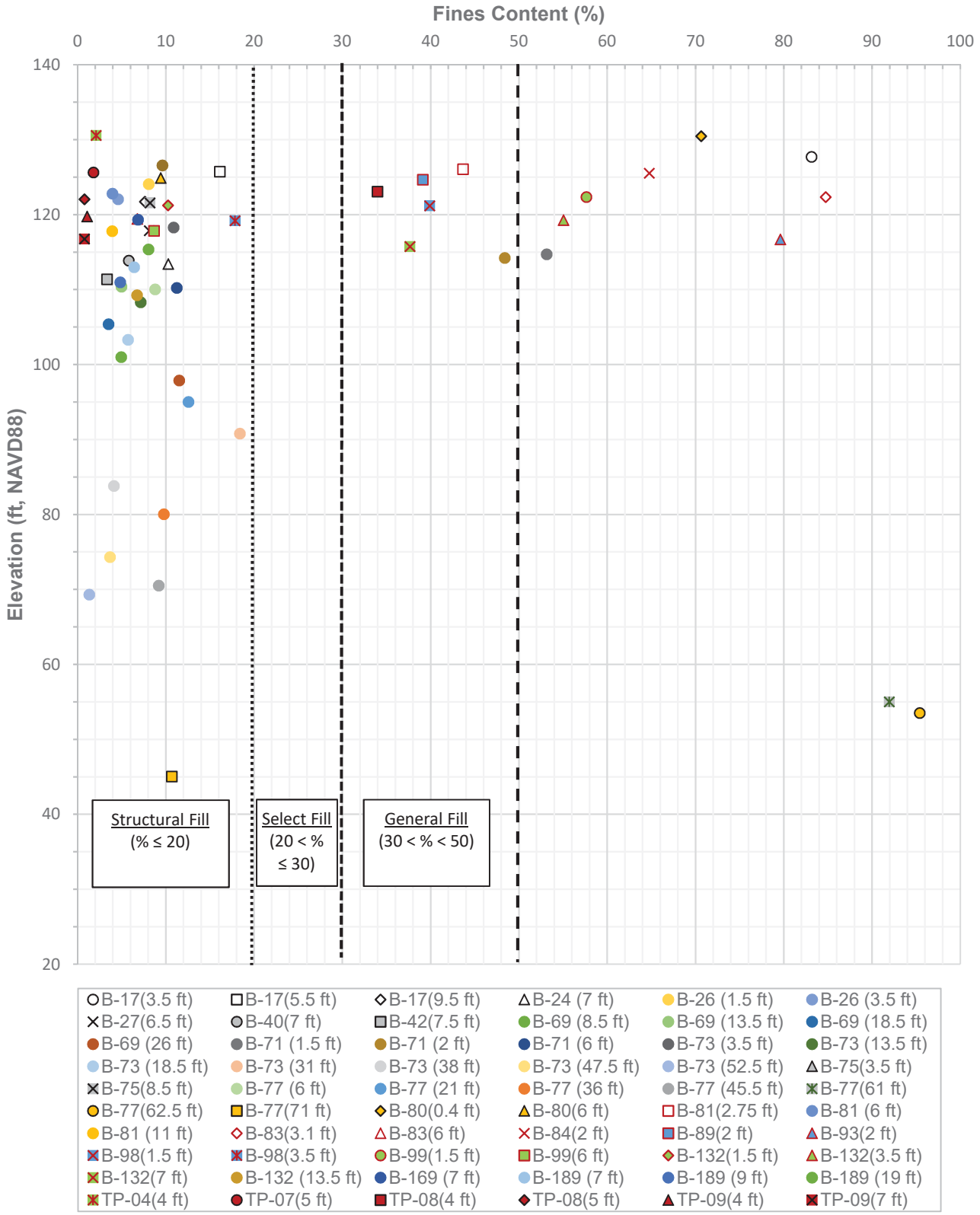
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawing USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
3. 3D site model provided by CB&I/CHIYODA on March 28, 2016.
4. Finished design grade elevations are provided in each area by CB&I/CHIYODA on drawings USAL-CB-CDDET-00-000088-001 through 015 dated May 13, 2016. Based on the finished grade elevations, Fugro selected a generalized design grade elevation per area for engineering recommendations.
5. Cut and fill thickness were calculated based on a combination of topography data presented on Plate 7, the selected design grade elevations for Areas 2 through 5 and 3D model provided by CBI& CHIYODA for Area 1, 6 and 7. No grading information is available for Area 8. See text Section 4.0 Site Preparation for details.
6. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

0 175 350 700 Feet

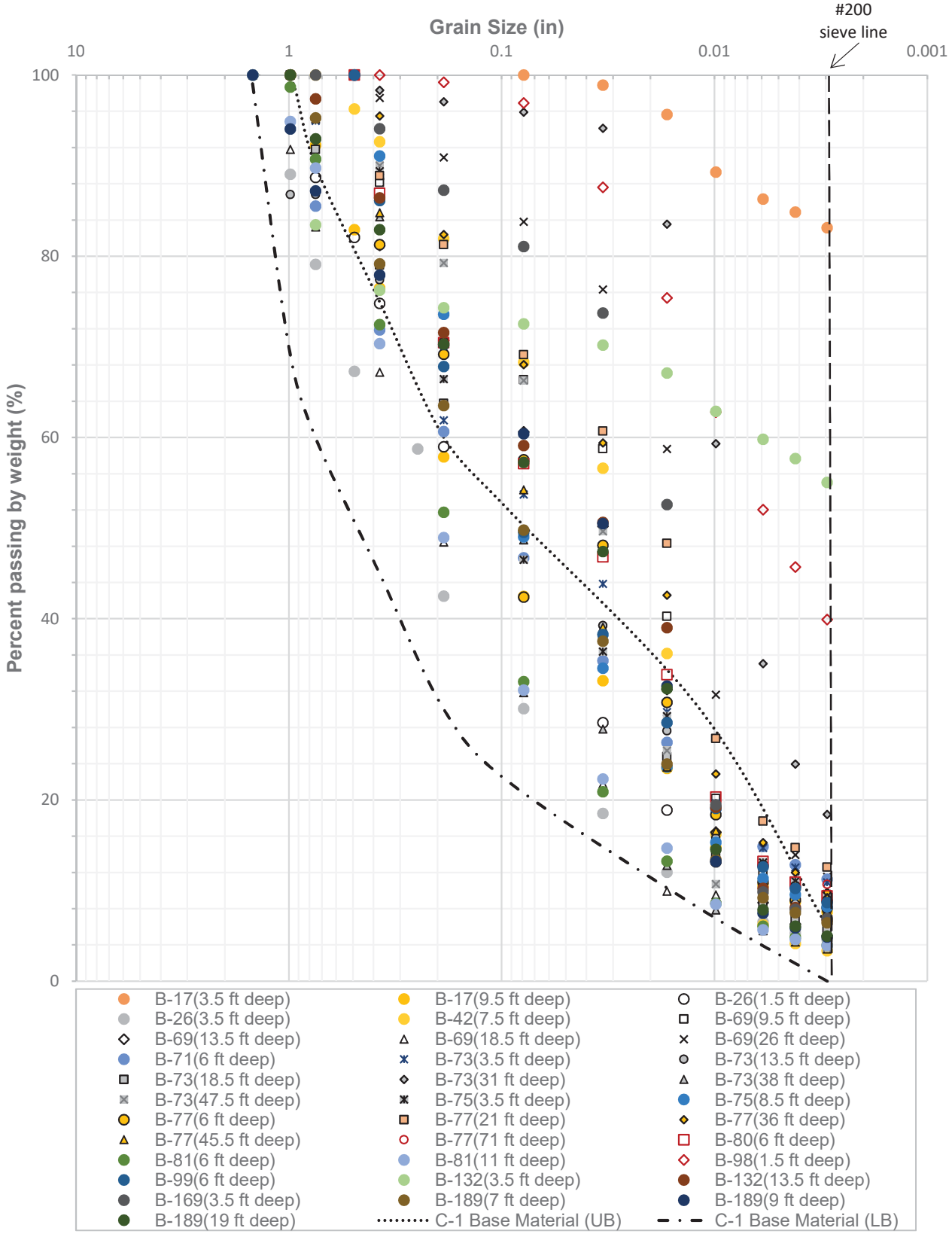
LAYDOWN AREA - 6
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 17f

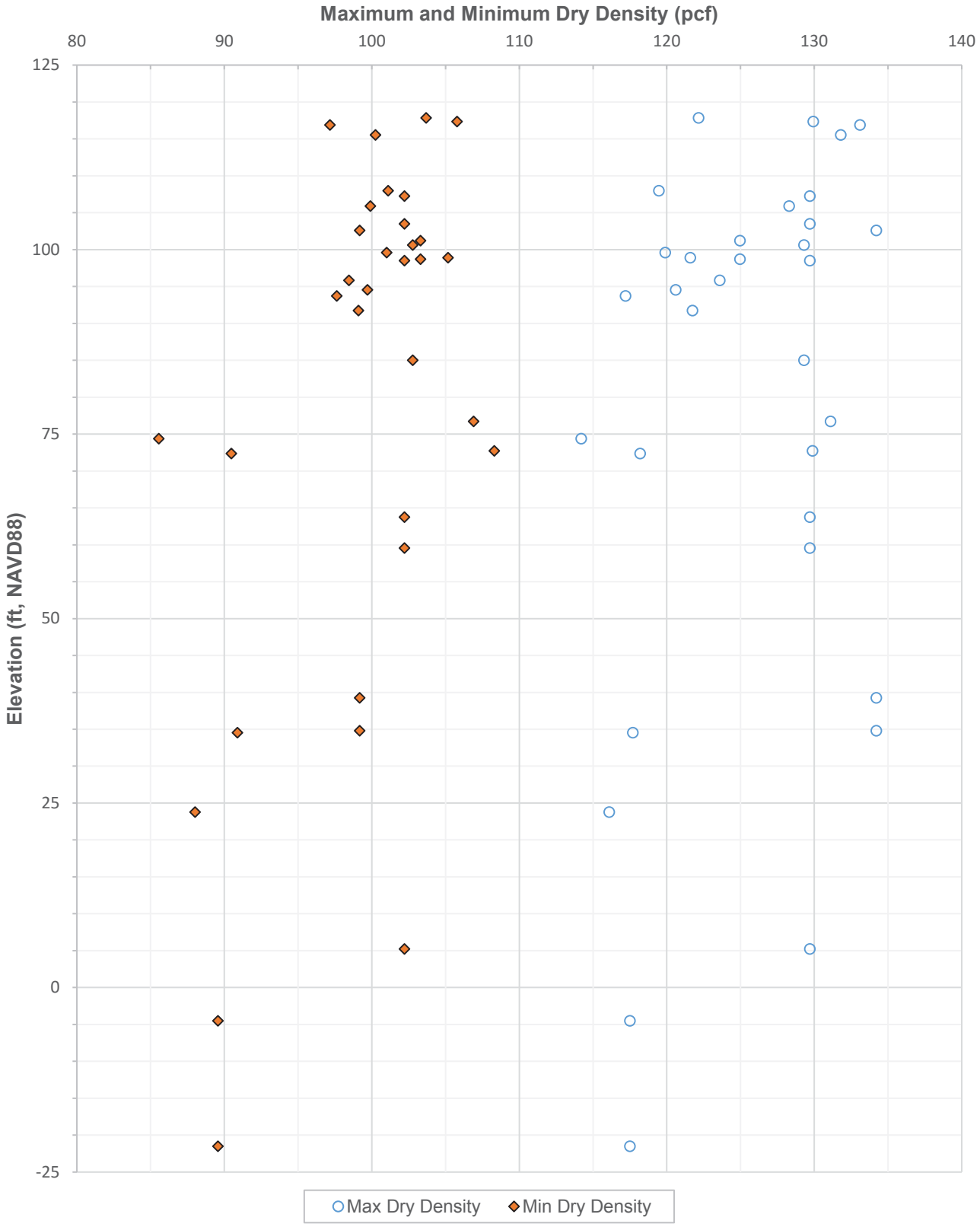
R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Plate-17f_CutAndFill_Area6.mxd, 2/6/2017, kimk



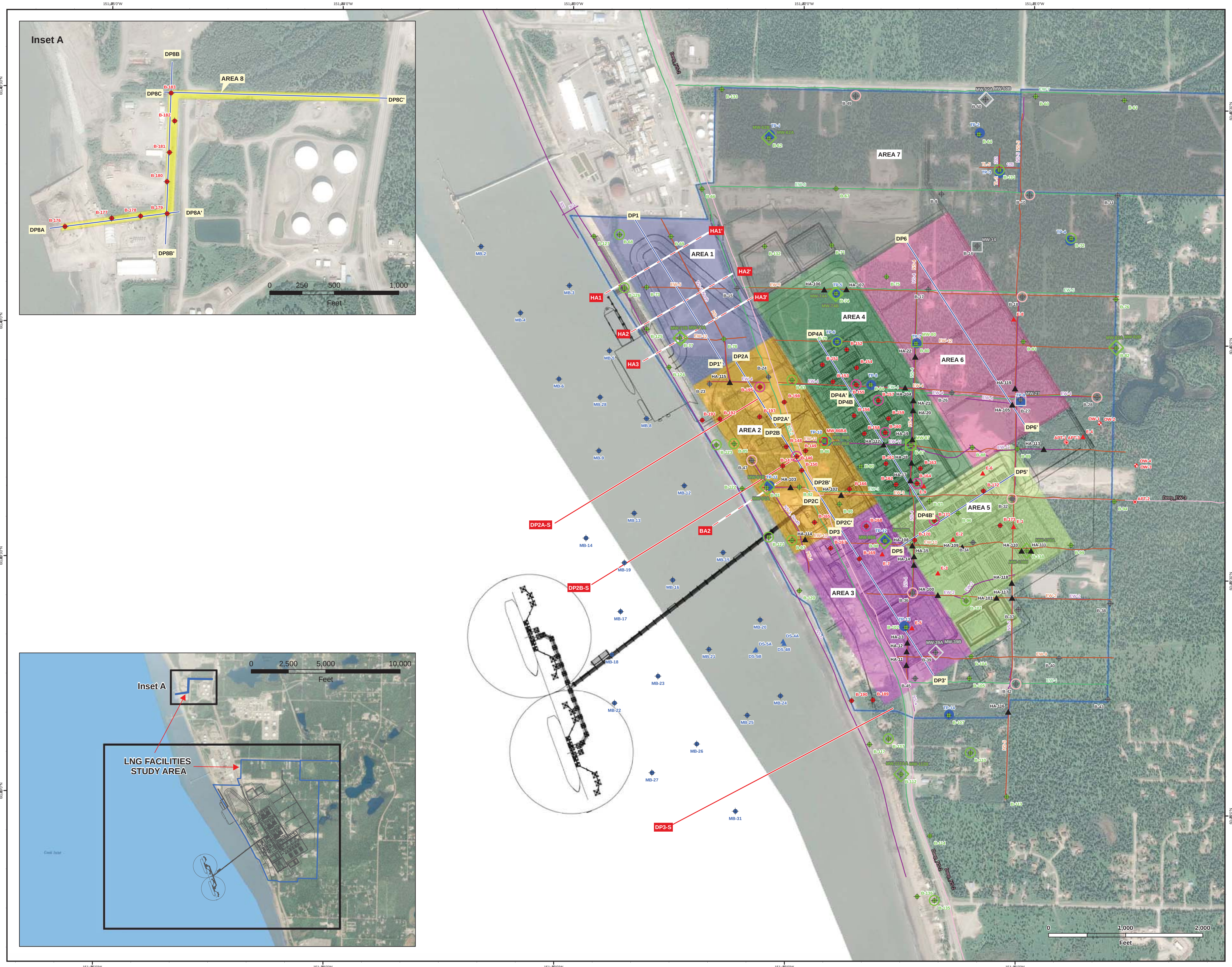
FINES CONTENT OF PROPOSED CUT MATERIALS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



GRADATION OF PROPOSED CUT MATERIALS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



MAXIMUM AND MINIMUM DENSITIES - ALL AREAS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



LEGEND

Onshore LNG Facilities Study Area
Cross Section Lines - Design Profiles
Cross Section Lines - Slope Stability Profiles (2016)
Cross Section Lines - Slope Stability Profiles (2015)

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- APT Well (2016)
- Observation Well (2016)
- VES Location (2016)
- Hand Auger (2015)
- Test Pit (2015)
- Marine Boring (2015)
- Drive Sampling (2015)
- Monitoring Well - Shallow (2016)
- Monitoring Well - Shallow (2015)
- Monitoring Well - Shallow&Deep (2015)
- Monitoring Well - Shallow (2014)
- Monitoring Well - Shallow&Deep (2014)
- Downhole Geophysics (2016)
- Downhole Geophysics (2015)
- Downhole Geophysics (2014)

GEOPHYSICAL SURVEY LINES

- Deep Seismic (2015)
- ERT (2015)
- Shallow Seismic (2015)
- Shallow Seismic (2014)

Borings	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007)		DH(1)	MW(2)	TP(3)	Borings	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007)		DH(1)	MW(2)	TP(3)
			Longitude, deg	Latitude, deg							Longitude, deg	Latitude, deg			
B-9	132.25	100.50	-151.35640930	60.67185419				B-105	95.54	101.00	-151.35368353	60.65469558			
B-10	132.80	103.00	-151.35002130	60.67188199	1			B-107	96.04	101.00	-151.35545580	60.65337380			1
B-11	136.36	101.30	-151.34366490	60.67192943				B-110	117.07	155.70	-151.35348741	60.65205636	1		
B-14	133.40	110.70	-151.35375740	60.67003682		1		B-112	109.09	101.00	-151.35846843	60.65124872		2	
B-15	122.96	101.00	-151.37105710	60.66836028				B-113	112.53	113.50	-151.35080438	60.65052782			
B-17	131.21	101.50	-151.35723250	60.66845974				B-114	114.99	126.00	-151.35629452	60.64907498			
B-18	131.25	101.50	-151.35041260	60.66826252	1			B-117	7.60	99.30	-151.36078845	60.65228123			
B-23	124.63	186.50	-151.37288540	60.66494561				B-120	15.41	101.00	-151.36601528	60.65771551			
B-24	120.40	101.00	-151.36864010	60.66523854				B-121	16.38	120.40	-151.36825549	60.65946757	1		
B-26	125.54	100.50	-151.35535980	60.66481307				B-122	16.52	101.00	-151.37037895	60.66120614			
B-27	124.35	101.50	-151.35039120	60.66461107		1		B-123	15.78	139.00	-151.37209000	60.66279044	1		
B-28	118.89	103.00	-151.34486220	60.66475620	1			B-124	13.68	78.50	-151.37584730	60.66551173			
B-32	122.99	101.50	-151.35084730	60.66109265	1			B-125	14.43	101.00	-151.37751496	60.66684480			
B-34	111.83	101.00	-151.35361470	60.65951893				B-126	14.43	127.00	-151.37914509	60.66831013	1		
B-36	111.70	103.00	-151.35789050	60.65768300	1			B-127	15.98	101.00	-151.38141047	60.67012169			
B-37	106.27	27.00	-151.35022180	60.65717918				B-131	133.25	91.00	-151.35225744	60.67274798	1		1
B-38	123.47	151.30	-151.34359640	60.65746298				B-132	122.90	111.00	-151.36911316	60.66986806			
B-39	95.51	151.50	-151.35615280	60.65559414		2		B-133	125.63	105.50	-151.37244708	60.67540767			
B-40	120.84	101.50	-151.34721580	60.65547709				B-135	112.66	150.50	-151.35586806	60.64679349	1		
B-42	118.85	101.50	-151.35029920	60.65453131	1			B-136	13.42	94.00	-151.35727753	60.64689374			
B-43	121.14	102.80	-151.34362450	60.65405368				B-137	116.64	115.50	-151.35941780	60.65248997	1		
B-45	109.09	101.50	-151.35759480	60.65465742				B-138	103.22	146.00	-151.34969539	60.65926678		2	
B-47	119.73	151.00	-151.36972090	60.66227016	1			B-146	111.74	217.30	-151.36648264	60.66243655	1		
B-49	127.76	101.50	-151.36274610	60.67526763	1			B-147	119.69	200.30	-151.36747798	60.66209855			
B-50	131.66	151.50	-151.35335930	60.67524580		2		B-148	122.26	206.10	-151.36726271	60.66278039			
B-60	135.49	81.00	-151.34970518	60.67539889				B-149	126.30	223.00	-151.36585008	60.66265279			
B-61	137.58	96.00	-151.34332584	60.67530860				B-150	110.97	200.20	-151.36610352	60.66195569			
B-62	127.66	101.30	-151.36898112	60.67372335		2	1	B-151	125.85	201.00	-151.36476522	60.66571099			
B-64	133.91	86.00	-151.35383276	60.67397590			1	B-152	125.79	150.40	-151.36303312	60.66626550			
B-66	127.44	105.00	-151.37372952	60.67185480				B-153	126.81	150.60	-151.36397783	60.66511871			
B-67	124.49	110.50	-151.36405465	60.67196390				B-154	127.46	151.00	-151.36228714	60.66562743			
B-68	122.47	202.00	-151.37961780	60.67016196	1			B-155	126.76	151.00	-151.36228399	60.66502805	1		
B-69	123.86	105.50	-151.37590717	60.67015134				B-156	124.84	151.00	-151.36239760	60.66393399			
B-71	116.20	101.00	-151.36427266	60.66996116				B-157	124.51	151.00	-151.36066599	60.66448827	1		
B-72	135.28	86.30	-151.34710070	60.67027771	1		1	B-158	120.63	150.50	-151.36163461	60.66329807			
B-73	122.34	111.00	-151.37753990	60.66852118				B-159	118.84	150.30	-151.35991084	60.66385187			
B-74	111.99	151.00	-151.36389233	60.66824205	1	2	1	B-160	115.35	150.20	-151.36011160	60.66335236	1		
B-75	129.87	101.50	-151.36028017	60.66890221				B-161	105.03	156.00	-151.36001832	60.66225349			
B-76	130.47	101.00	-151.34362252	60.66829367				B-162	111.45	151.00	-151.35926363	60.66151597			
B-77	116.01	202.00	-151.37510864	60.66654928	1	2		B-163	104.85	150.90	-151.35752400	60.66210614			
B-78	118.09	101.00	-151.37192900	60.66654928				B-164	105.43	151.00	-151.35773753	60.66157129	1		
B-79	123.61	101.00	-151.36374290	60.66653811	1		1	B-165	103.54	150.30	-151.36510331	60.66011521			
B-80	130.85	100.50	-151.35803312	60.66653133		1	1	B-166	100.68	150.70	-151.36262002	60.66132084			
B-81	128.79	106.00	-151.35027766	60.66667672				B-167	111.91	150.30	-151.36390727	60.65921073			
B-82	116.34	150.50	-151.34337253	60.66650909		2	1	B-168	118.89	176.00	-151.36134711	60.66001705	1		
B-83	126.03	106.00	-151.36693500	60.66514200	1			B-169	122.82	151.00	-151.36179473	60.65885340			
B-84	127.53	100.80	-151.36123975	60.66503400			1	B-170	123.01	171.00	-151.35783392	60.65955956			
B-85	119.50	202.00	-151.37102452	60.66284303				B-171	116.53	151.00	-151.35643036	60.66026570	1		
B-86	124.85	101.00	-151.36453497	60.66300368	1	3	1	B-172	119.91	160.50	-151.35294376	60.66135351			
B-87	106.46	96.30	-151.35823947	60.66291477	1	1		B-173	121.02	171.00	-151.35167947	60.66013946			
B-88	93.98	101.30	-151.35380723	60.66288908				B-176	27.14	210.20	-151.39778916	60.68727972	1		
B-89	126.64	101.30	-151.35055076	60.66287057				B-177	29.39	51.00	-151.39577948	60.68747575			
B-90	110.48	101.00	-151.36189643	60.66213101				B-178	34.20	51.00	-151.39453464	60.68753301			
B-91	117.54	100.60	-151.36864403	60.66129503		2	1	B-179	43.66	51.00	-151.39339076	60.68760076			
B-92	118.03	99.80	-151.36624365	60.66135905				B-180	77.76	76.00	-151.39342768	60.68827819			
B-93	118.69	101.00	-151.35681706	60.66095091				B-181	87.19	76.50	-151.39334051	60.68889704			
B-94	127.39	101.00	-151.34346988	60.66106965				B-182	85.60	50.50	-151.39314642	60.68956559			
B-95	96.73	101.00	-151.36332415	60.66078281				B-183	98.76	101.10	-151.39333025	60.69015403			
B-96	97.65	101.00	-151.35473390	60.66054881				B-189	119.97	201.00	-151.36061126	60.65385926			
B-97	109.47	203.00	-151.36669900	60.65946066	1			B-190	20.56	97.10	-151.36215168	60.65382001			
B-98	115.67	101.00	-151.35997475	60.65951243		2	1	B-191	20.50	114.20	-151.37337267	60.66365831			
B-99	123.82	151.00	-151.34652491	60.65948995				B-192	123.19	239.30	-151.37209739	60.66371093			
B-101	94.98	152.00	-151.35402409	60.65744590	1			B-195	119.97	226.00	-151.36924661	60.66487249	1		
B-102	97.95	101.00	-151.35833603	60.65645887			1	B-197	122.74	249.60	-151.36922005	60.66381855			
B-104	95.22	105.90	-151.35258202	60.65547421				B-198	117.99	300.50	-151.36747007	60.66435767			

Notes:

(1)DH = Downhole Seismic Test (2)MW = Monitoring Well (3)TP = Test Pit

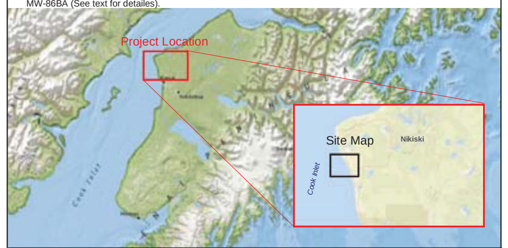
Monitoring Wells	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007)	
			Longitude, deg	Latitude, deg
MW-14	133.70	56.00	-151.35376440	60.67001489
MW-27	124.30	57.00	-151.35037410	60.66459539
MW-39A	95.50	147.00	-151.35615710	60.65557926
MW-39B	95.30	40.00	-151.35613450	60.65560508
MW-50A	132.60	146.00	-151.35331470	60.67523060
MW-50B	132.20	55.50	-151.35336650	60.67526072
MW-62A	127.36	145.40	-151.36900760	60.67372740
MW-62B	127.63	47.00	-151.36896370	60.67373020
MW-74A	111.70	159.00	-151.36390300	60.66825110
MW-74B	111.53	81.00	-151.36390170	60.66822290
MW-77A	116.31	146.00	-151.37507200	60.66655740
MW-77B	116.25	61.00	-151.37506340	60.66654210
MW-80	130.99	61.70	-151.35796510	60.66660890
MW-82A	118.58	150.00	-151.34354410	60.66651740
MW-82B	119.22	52.00	-151.34358260	60.66654600
MW-86A	124.44	146.00	-151.36452040	60.66301340
MW-86B	124.48	42.00	-151.36451240	60.66299700
MW-87	106.91	52.50	-151.35826080	60.66293920
MW-91A	117.27	146.00	-151.36859480	60.66133530
MW-91B	117.31	64.00	-151.36858380	60.66131500
MW-98A	122.17	116.00	-151.36000880	60.65949110
MW-98B	122.26	43.00	-151.36003290	60.65952300
MW-112AA	115.55	119.00	-151.35844950	60.65126000
MW-112B	115.76	61.00	-151.35847780	60.65124200
MW-138A	103.22	146.80	-151.34969540	60.65926680
MW-138B	103.33	43.30	-151.34973370	60.65927620
APT-1	117.09	138.00	-151.34706092	60.66315390
APT-2	126.99	137.00	-151.34195457	60.66107404
APT-3	115.83	437.00	-151.34698760	60.66312928
OW-1	109.56	79.00	-151.34463703	60.66385170
OW-2	109.09	150.00	-151.34460741	60.66382579
OW-3	128.26	70.00	-151.34191656	60.66235654
OW-4	128.29	137.00	-151.34194479	60.66238108
MW-86BA	124.73	66.00	-151.36456512	60.66299257

Hand Augers	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007)	
			Longitude, deg	Latitude, deg
HA-11	104.53	2.8	-151.3582393	60.65512561
HA-12	93.98	4.3	-151.3582455	60.65560946
HA-13	97.62	2.3	-151.3582056	60.65592101
HA-14	121.19	1.6	-151.3578491	60.65868729
HA-15	121.09	3.2	-151.3578434	60.65898341
HA-17	116.12	2.5	-151.3582332	60.66167632
HA-18	108.69	5.5	-151.3581713	60.66230348
HA-19	106.92	6.9	-151.3581512	60.66313564
HA-20	119.04	2.6	-151.3581436	60.66416493
HA-21	123.32	2.8	-151.3581355	60.66451619
HA-22	130.06	2.8	-151.3580616	60.66606764
HA-100	97.71	4.3	-151.3560797	60.65762929
HA-101	88.78	2.0	-151.3518266	60.65757639
HA-102	104.23	4.0	-151.3632097	60.6611113
HA-103	117.69	3.0	-151.3668833	60.66134432
HA-104	124.01	3.8	-151.3587445	60.6649634
HA-105	124.7	2.2	-151.3509738	60.66444029
HA-106	118.92	1.8	-151.3647449	60.66838243
HA-107	118.88	3.7	-151.3631376	60.66840602
HA-108	123.01	1.5	-151.3579906	60.65936729
HA-109	111.12	1.5	-151.3543773	60.65935399
HA-110	100.45	4.0	-151.3500841	60.65926121
HA-111	102.15	2.0	-151.3494033	60.65927181
HA-112	106.13	2.0	-151.3602101	60.662931
HA-113	125.64	2.4	-151.3486347	60.66288061
HA-114	105.52	2.7	-151.365746	60.65950795
HA-115	121.16	3.3	-151.371427	60.6650298
HA-116	109.59	5.0	-151.350813	60.65354365
HA-117	90.06	3.0	-151.3507027	60.65758828
HA-118	94.46	2.0	-151.3507445	60.65811632
HA-119	123.19	2.0	-151.350797	60.66501537

Test Pits	As-Built Elevation (NAVD88, ft)	Depth, ft	As-Built Coordinates (NAD83 - NSRS2007)	
			Longitude, deg	Latitude, deg
TP-01	127.2	8	-151.36890605	60.67382958
TP-02	133.08	8	-151.35371804	60.67405024
TP-03	134.43	5	-151.35223907	60.67264108
TP-04	134.56	8	-151.34703124	60.67029871
TP-05	111.72	8	-151.36376102	60.66825015
TP-06	122.7	8	-151.36376305	60.66658642
TP-07	130.61	8	-151.35798402	60.66653967
TP-08	127.05	8	-151.36120253	60.66502775
TP-09	123.74	8	-151.35036700	60.66453403
TP-10	124.36	8	-151.36432332	60.66300351
TP-11	117.18	8	-151.36840024	60.66138217
TP-12	122.53	8	-151.35995040	60.65957943
TP-13	97.98	8	-151.35843823	60.65650048
TP-14	97.02	11	-151.35507881	60.65339243

Notes:

1. Base imagery provided by EXP Geomatics (EXP).
2. Onshore LNG Facilities Study Area provided by AKLNG.
3. Onshore Facilities plot plan is provided by CB&I/CHIYODA on drawings USAL-CB-LDLAY-00-000003-000 Rev.6 dated June 30, 2016.
4. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc. (2015, 2016).
5. Groundwater monitoring well MW-86B was abandoned during 2016 G&G program and replaced by MW-86BA (See text for detailes).



This document may only be used for the purpose for which it was commissioned and in accordance with the terms of engagement for that commission. Unauthorized use of this document in any form whatsoever is prohibited.

GEODETIC INFORMATION

PROJECTION:	Transverse_Mercator
FALSE EASTING:	1640416.666666666
FALSE NORTHING:	0.0
CENTRAL MERIDIAN:	-150.0
SCALE FACTOR:	0.9999
LATITUDE OF ORIGIN:	54.0
LINEAR UNIT:	US Foot
SPHEROID:	GRS 1980
SEMI-MAJOR AXIS:	6,378,137.000
SEMI-MINOR AXIS:	6,356,752.314140356
INVERSE FLATTENING:	298.257222101

N W E S

Coordinate Grid: NAD 1983 (NSRS 2007), Degrees Minutes Seconds

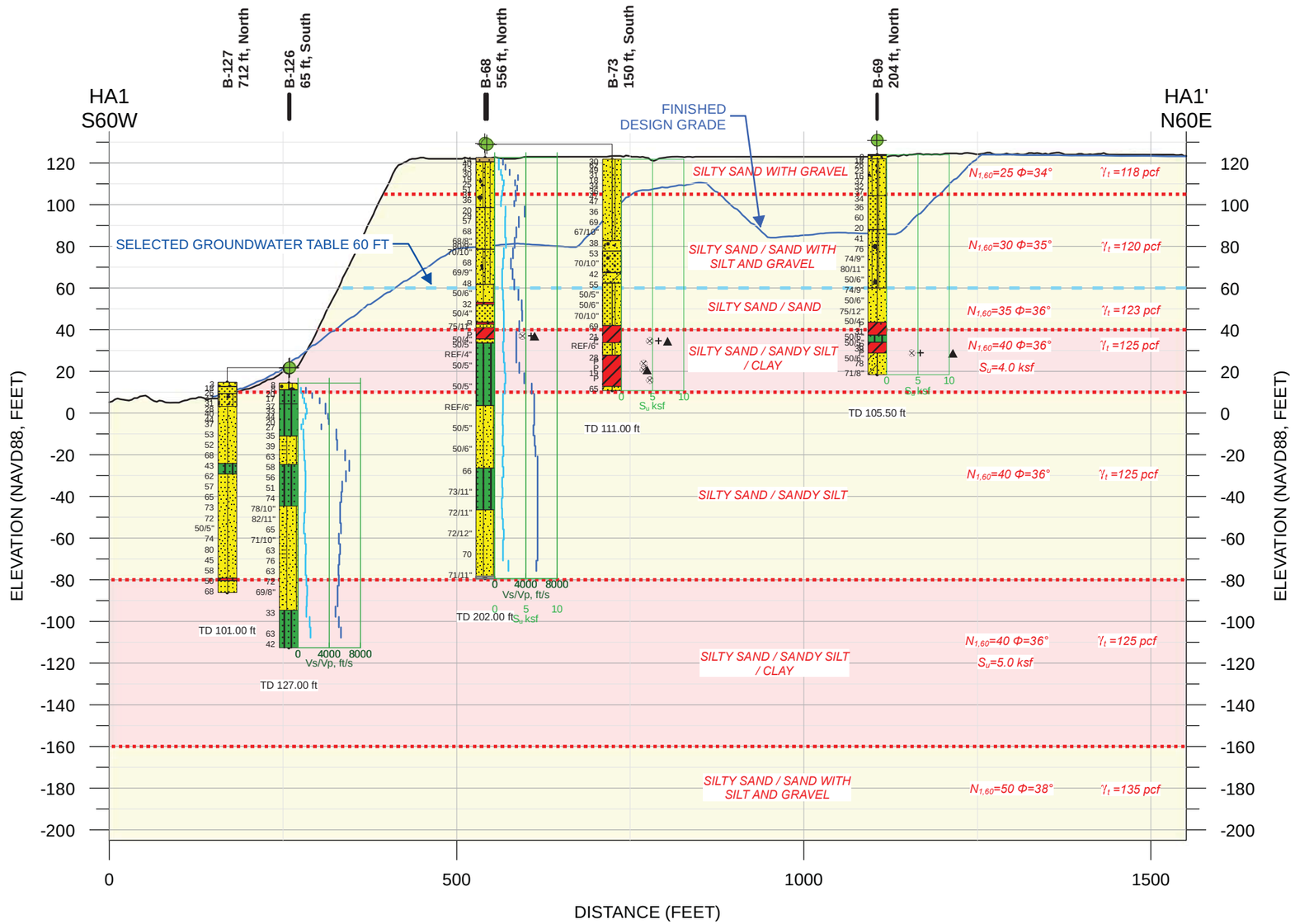
FUGRO CONSULTANTS, INC.
6100 Hillcroft Ave.
Houston, Texas 77081
Tel: 713.369.5400
Fax: 713.369.5518
www.fugroconsultants.com

Alaska LNG.

PLAN OF DESIGN CROSS SECTION LINES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

REV NO:	DATE:	DESCRIPTION:	DRAWN:	CHKD:	APPR:
0	11/29/2016	Design Cross Section Lines	KIMK	BB	

JOB NUMBER: 04.10160001-12 DRAWING NO.: PLATE 20



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2015)
- Topographic Elevation (KPB, 2008)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean Clay with Sand (CL)
- Sandy Lean CLAY (CL)
- Topsoil
- Silty Gravel with Sand (GM)
- Silt (ML)
- Sandy Silt (ML)
- Sandy Silt with Gravel (ML)
- Clayey Sand (SC)
- Silty Sand (SM)
- Silty Sand with Gravel (SM)
- Sand (SP)
- Sand with Gravel (SP)
- Sand with Silt and Gravel (SP-SM)
- Sand (SW)
- Sand with Gravel (SW)
- Sand with Silt and Gravel (SW-SM)

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

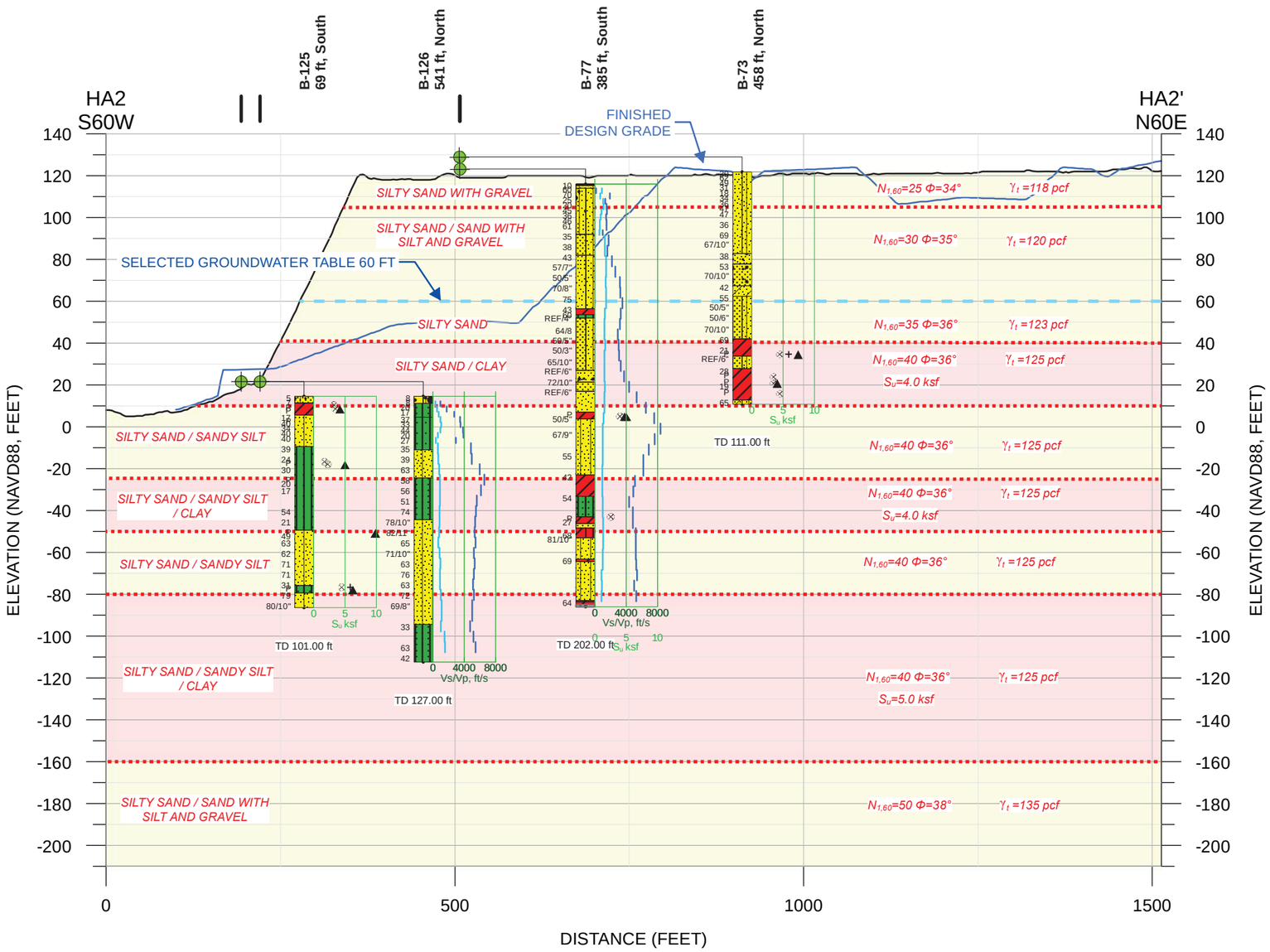
1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

60 ft
180 ft
Vertical Exaggeration = 3.0X

SELECTED DESIGN PROFILE - AREA 1
CROSS SECTION HA1 - HA1'
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21a

R:\041000\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21a_Xsect-HA1.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2015)
- Topographic Elevation (KPB, 2008)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean Clay with Gravel (CL)
- Sandy Lean CLAY (CL)
- Lean Clay with Sand (CL)
- Topsoil
- Silty Gravel with Sand (GM)
- Gravel (GP)
- Sandy Silt (ML)
- Silt with Sand (ML)
- Clayey Sand (SC)
- Silty Sand (SM)
- Silty Sand with Gravel (SM)
- Sand (SP)
- Sand with Gravel (SP)
- Sand with Silt and Gravel (SP-SM)
- Sand (SW)
- Sand with Gravel (SW)
- Sand with Silt and Gravel (SW-SM)

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

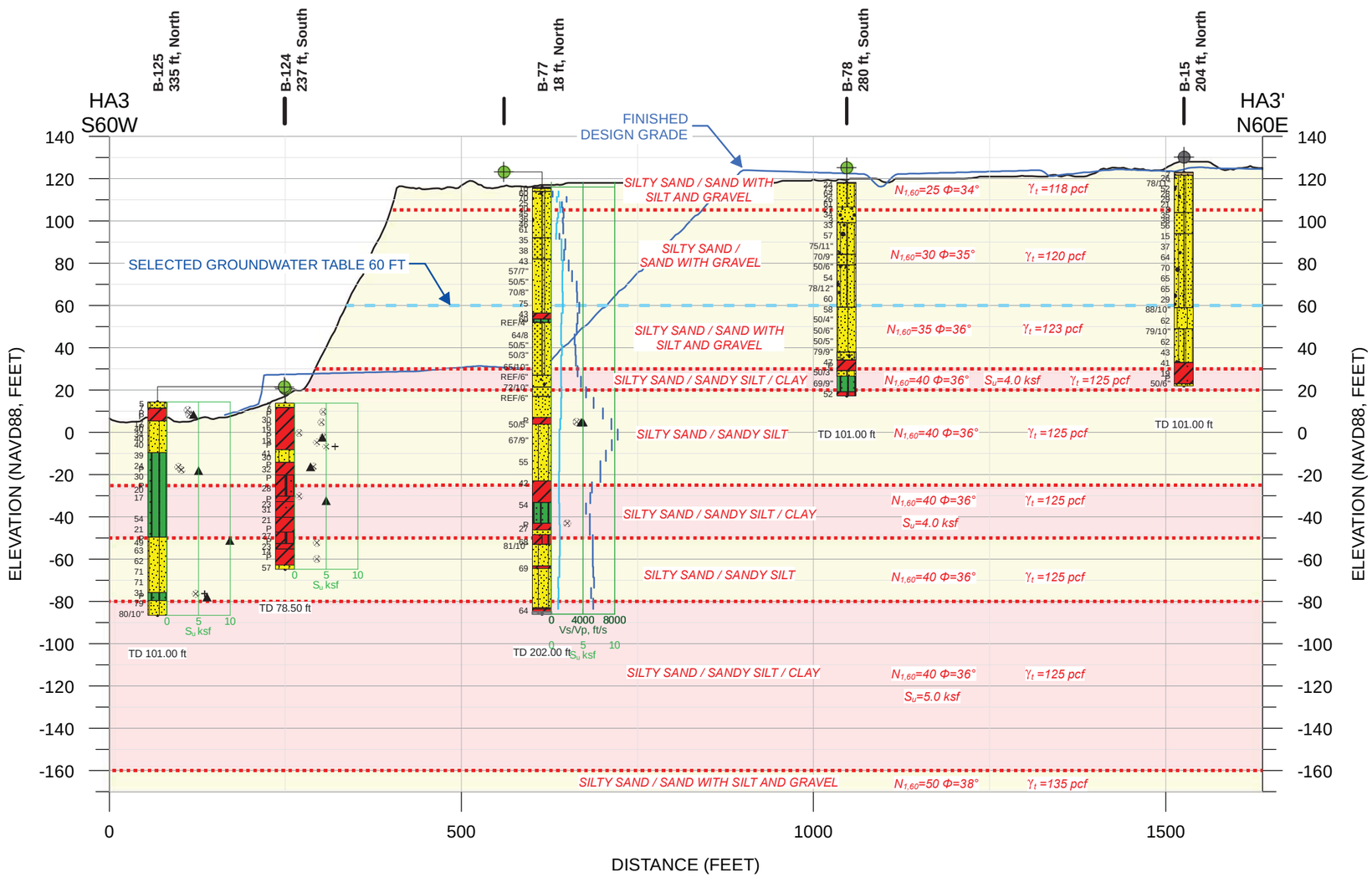
NOTES:

1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

60 ft
180 ft
Vertical Exaggeration = 3.0X

SELECTED DESIGN PROFILE - AREA 1
CROSS SECTION HA2 - HA2'
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21b_Xsect-HA2.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2015)
- Boring (2014)
- Topographic Elevation (KPB, 2008)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Sandy Lean CLAY (CL)
- Lean Clay with Sand (CL)
- Topsoil
- Silty Gravel with Sand (GM)
- Silt (ML)
- Sandy Silt (ML)
- Sandy Silt with Gravel (ML)
- Silty Sand (SM)
- Silty Sand with Gravel (SM)
- Sand (SP)
- Sand with Gravel (SP)
- Sand with Silt and Gravel (SP-SM)
- Sand (SW)
- Sand with Gravel (SW)
- Sand with Silt and Gravel (SW-SM)

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

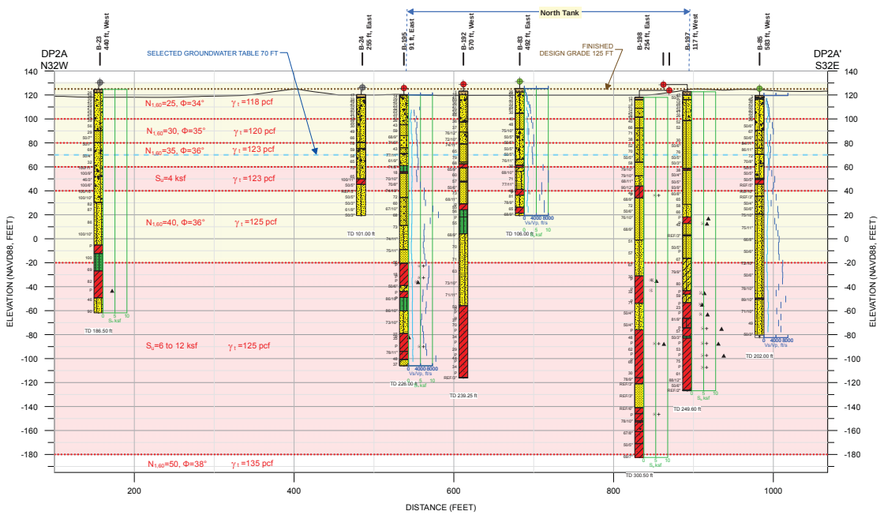
1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

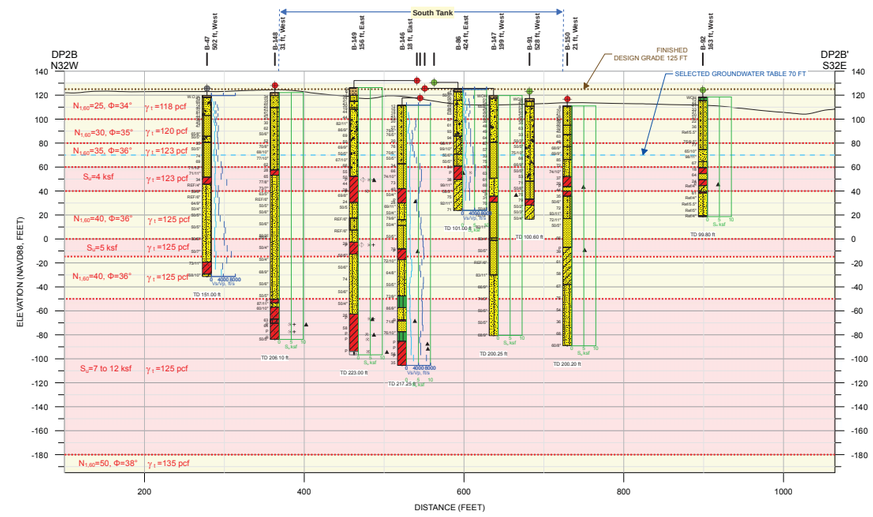
60 ft
180 ft
Vertical Exaggeration = 3.0X

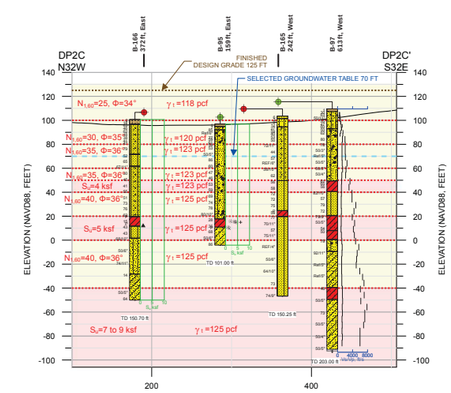
SELECTED DESIGN PROFILE - AREA 1
CROSS SECTION HA3 - HA3'
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21c_Xsect-HA3.mxd, 1/18/2017, kimk

Alaska LNG.







UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- ⊕ Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

1. Topographic source data is provided by McLane Consulting, Inc (2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

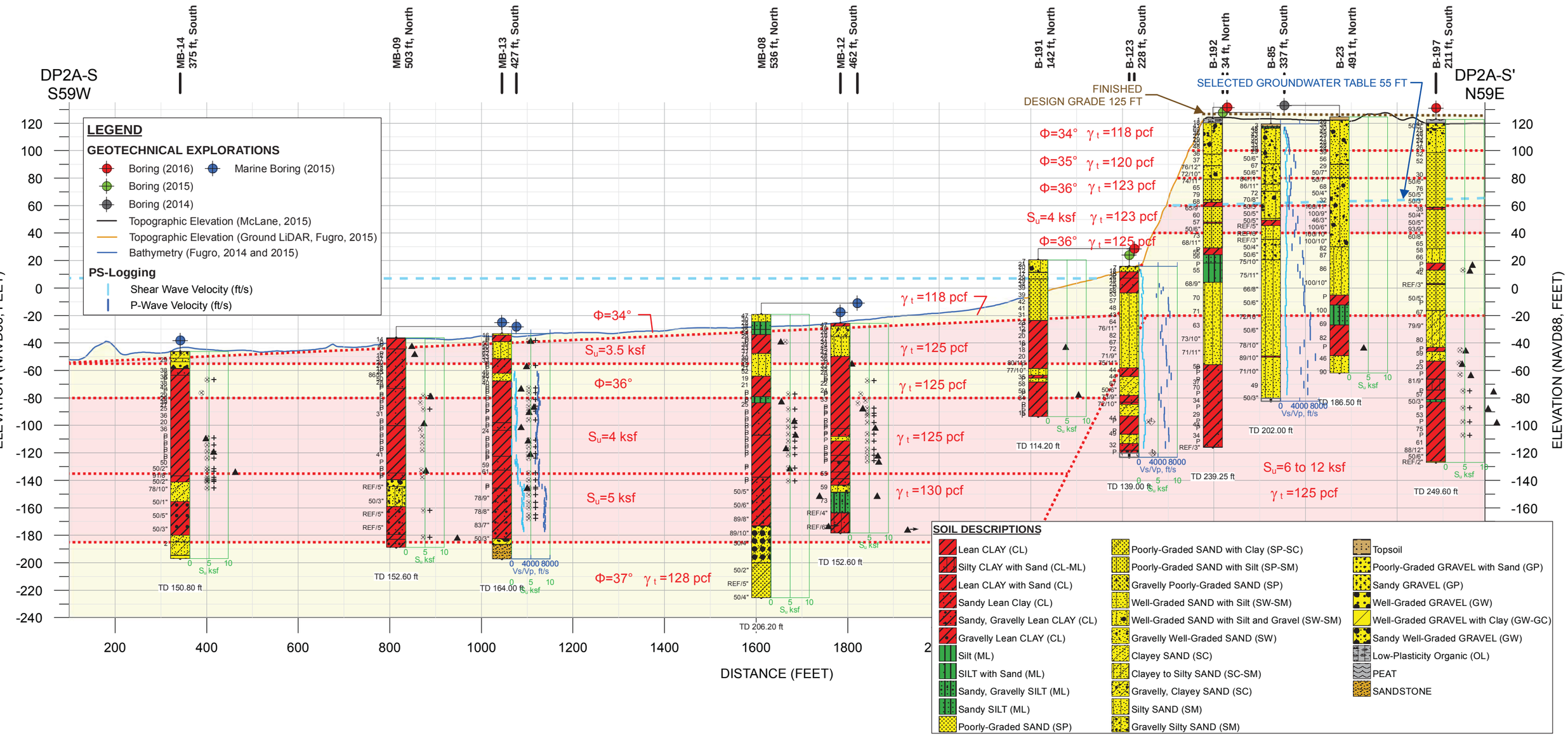
- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Silty CLAY with Sand (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Silt (ML)
- SILT with Sand (ML)
- Sandy, Gravelly SILT (ML)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Clay (SP-SC)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Gravelly, Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL with Silt (GP-GM)
- Sandy GRAVEL (GP)
- Sandy, Silty GRAVEL (GM)

50 ft
75 ft
Vertical Exaggeration = 1.5X

SELECTED DESIGN PROFILES - AREA 2
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21d

R:\04-10160\18 Project\001-GBR\04-10160001 - ASA\AVGR_GIR\Assets\1_REPORT\23016_11_04_Long-section\Report\02\Final\Plate-21b_Design\Profiles.mxd, 1/18/2017, sunk



SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY with Sand (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Sandy, Gravelly Lean CLAY (CL)
- Gravelly Lean CLAY (CL)
- Silt (ML)
- SILT with Sand (ML)
- Sandy, Gravelly SILT (ML)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Clay (SP-SC)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Gravelly, Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL (GW)
- Well-Graded GRAVEL with Clay (GW-GC)
- Sandy Well-Graded GRAVEL (GW)
- Low-Plasticity Organic (OL)
- PEAT
- SANDSTONE

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

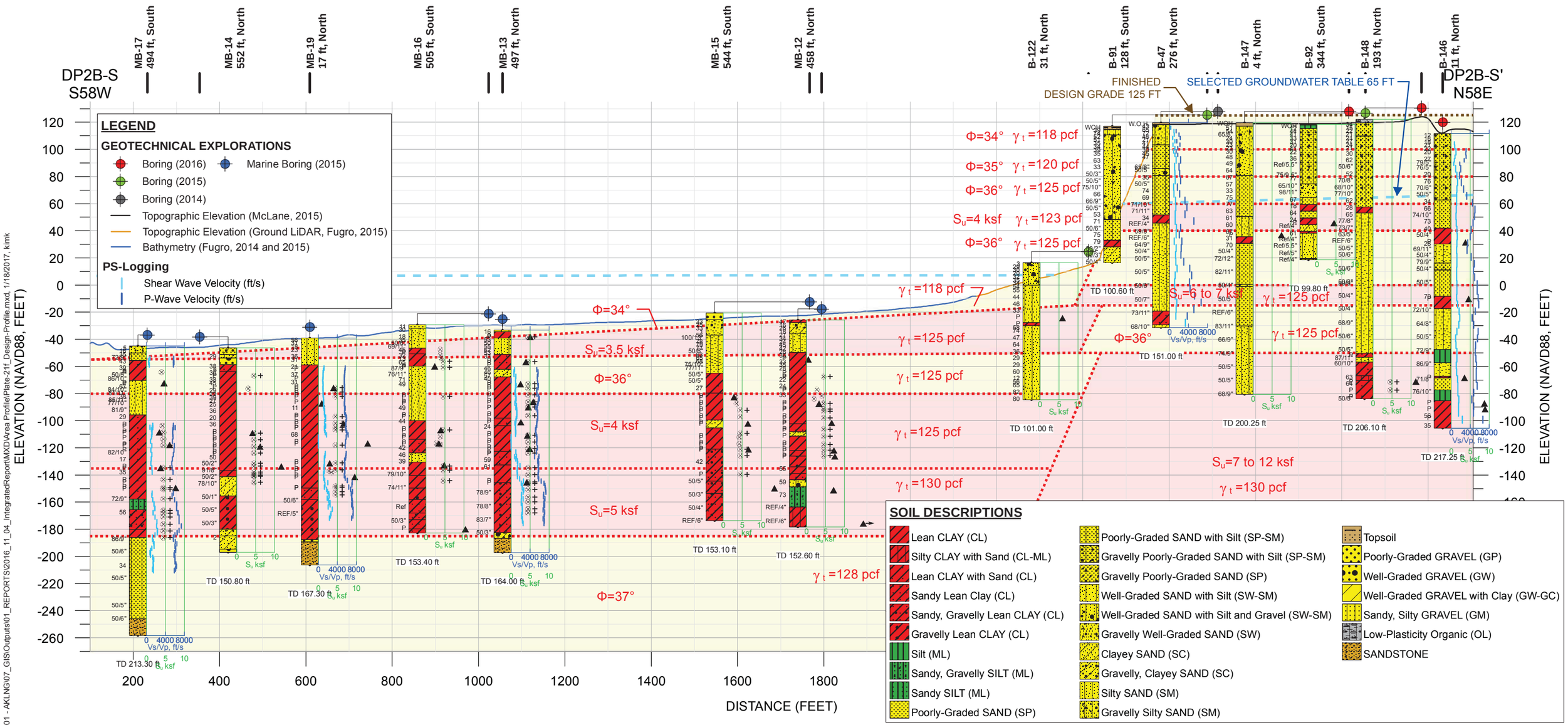
1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

70 ft
210 ft
Vertical Exaggeration = 3.0X

SELECTED DESIGN PROFILE - AREA 2 (DP2A-S - DP2A-S')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21e

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21e_Design_Profile.mxd, 2/1/2017, kimk



UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

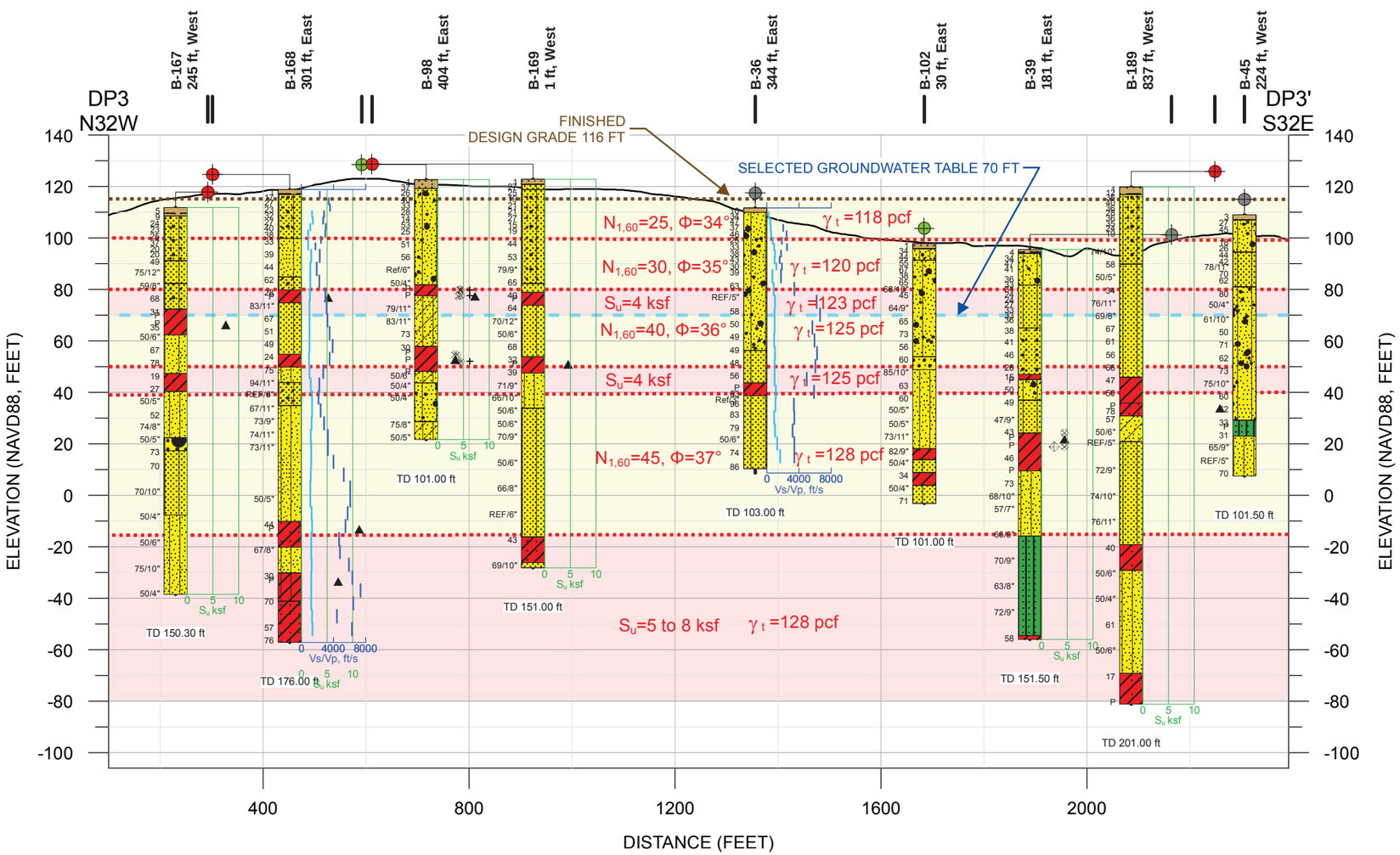
1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

70 ft
220 ft
Vertical Exaggeration = 3.1X

SELECTED DESIGN PROFILE - AREA 2 (DP2B-S - DP2B-S')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21f

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReportMXD\Area Profile\Plate-21f_Design-Profile.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Sandy SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Clayey SAND (SC)
- Clayey to Silty SAND (SC-SM)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL with Silt and Sand (GW-GM)

50 ft
250 ft
Vertical Exaggeration = 5.0X

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

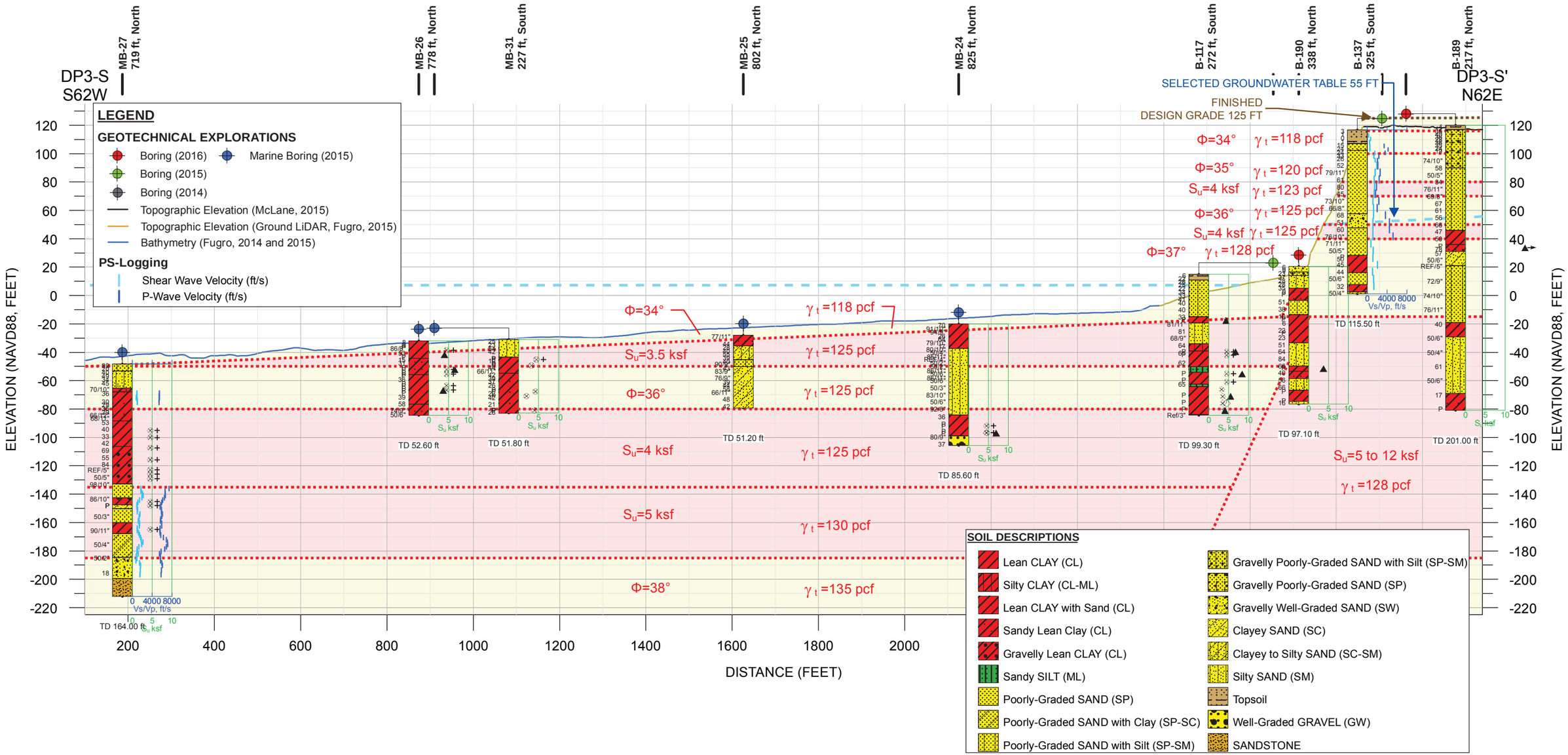
P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

SELECTED DESIGN PROFILE - AREA 3
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-20c_Design-Profile.mxd, 2/1/2017, kink



UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

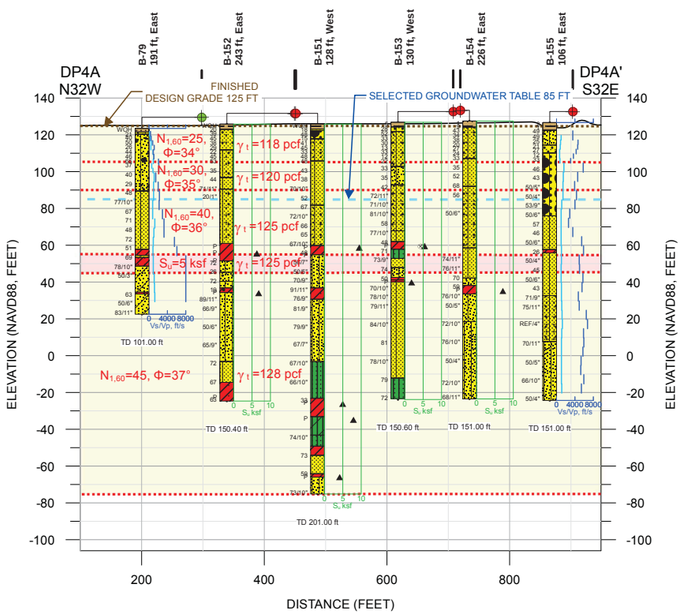
1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

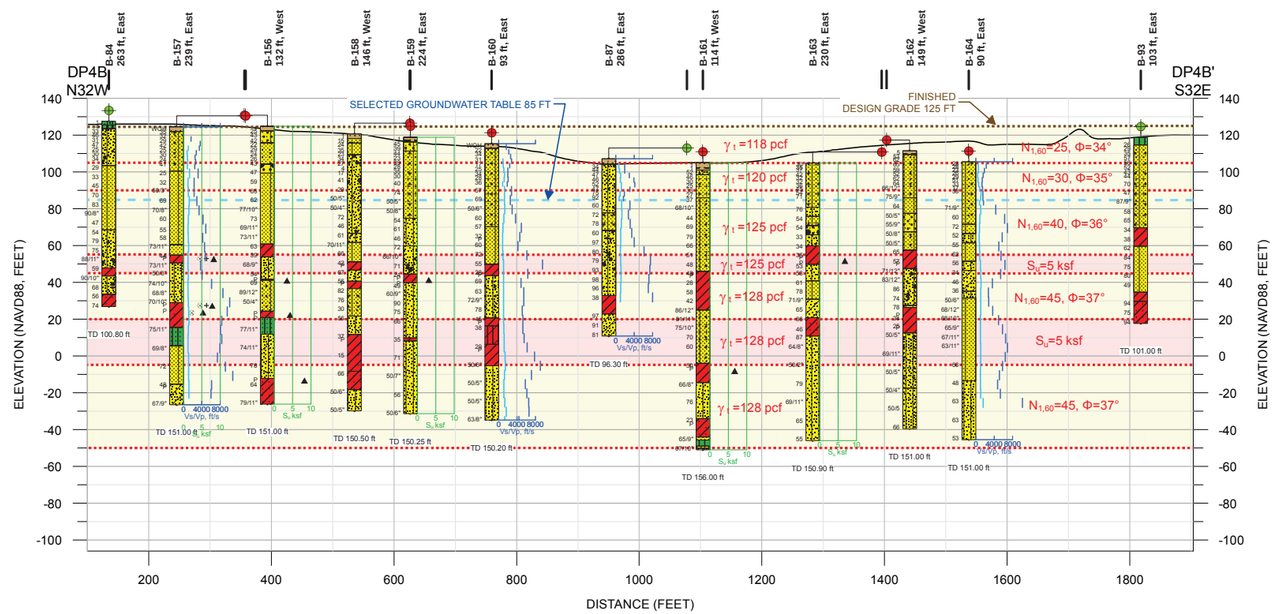
70 ft
230 ft
Vertical Exaggeration = 3.3X

SELECTED DESIGN PROFILE - AREA 3 (DP3-S - DP3-S')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21h

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21h_Design-Profile.mxd, 1/18/2017, kimk





LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- Gravelly Lean CLAY (CL)
- Silt (ML)
- Clayey SILT (ML)
- SILT with Sand (ML)
- Sandy, Gravelly SILT (ML)
- Sandy SILT (ML)
- Elastic Silt with Sand (MH)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Clay (SW)
- Well-Graded SAND with Silt (SW-SM)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Poorly-Graded GRAVEL with Sand (GP)
- Sandy GRAVEL with Silt (GP-GM)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL with Silt and Sand (GW-GM)
- Sandy Well-Graded GRAVEL (GW)

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

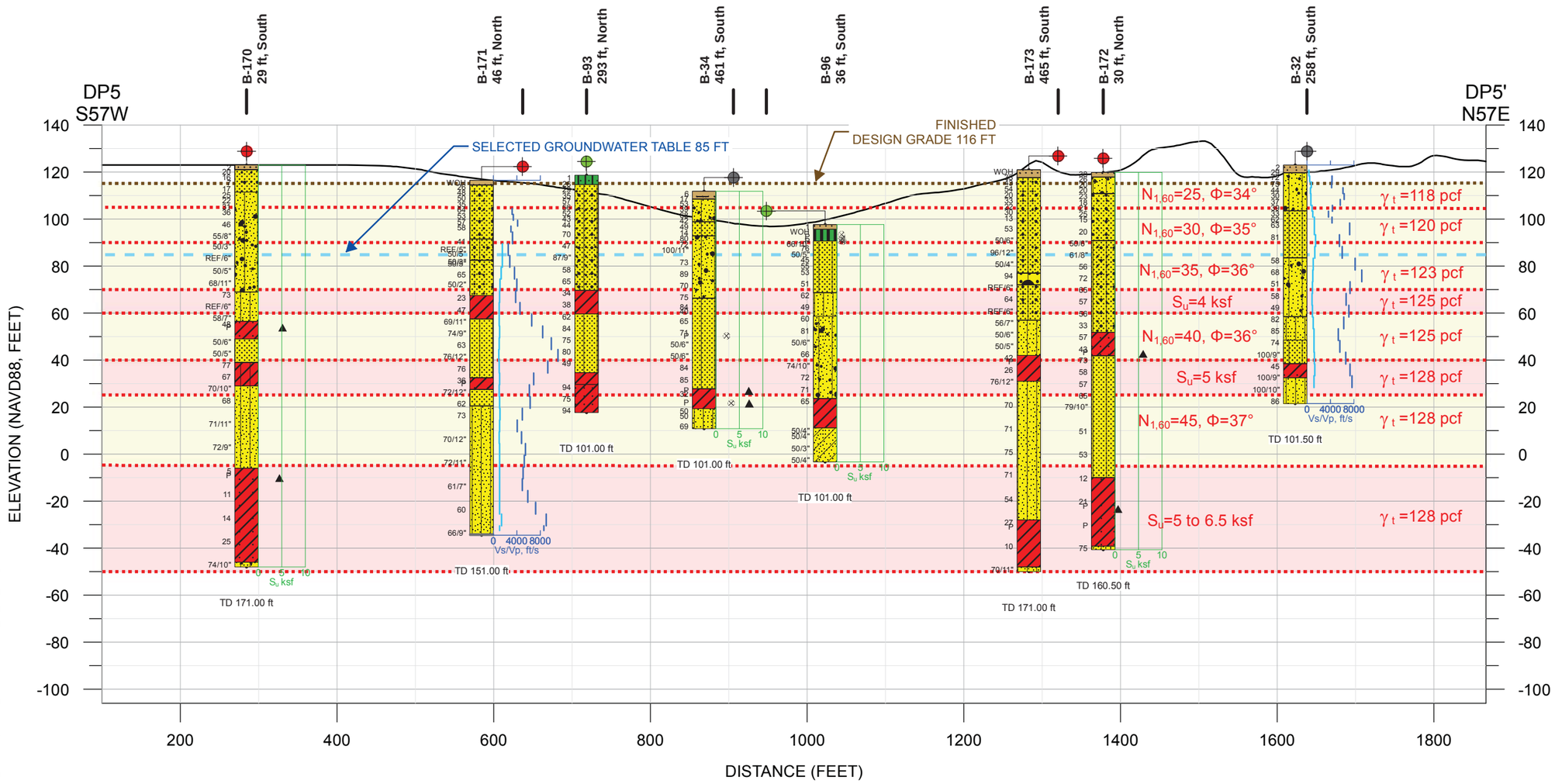
1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

50 ft
150 ft
Vertical Exaggeration = 3.0X

SELECTED DESIGN PROFILES - AREA 4
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21i

R:\04100\2016 Projects\0001-0099\04_10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-20d_Design-Profile.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Lean CLAY with Sand (CL)
- Sandy Lean Clay (CL)
- SILT with Sand (ML)
- Elastic Silt (MH)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Topsoil
- Well-Graded GRAVEL with Silt and Sand (GW-GM)

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

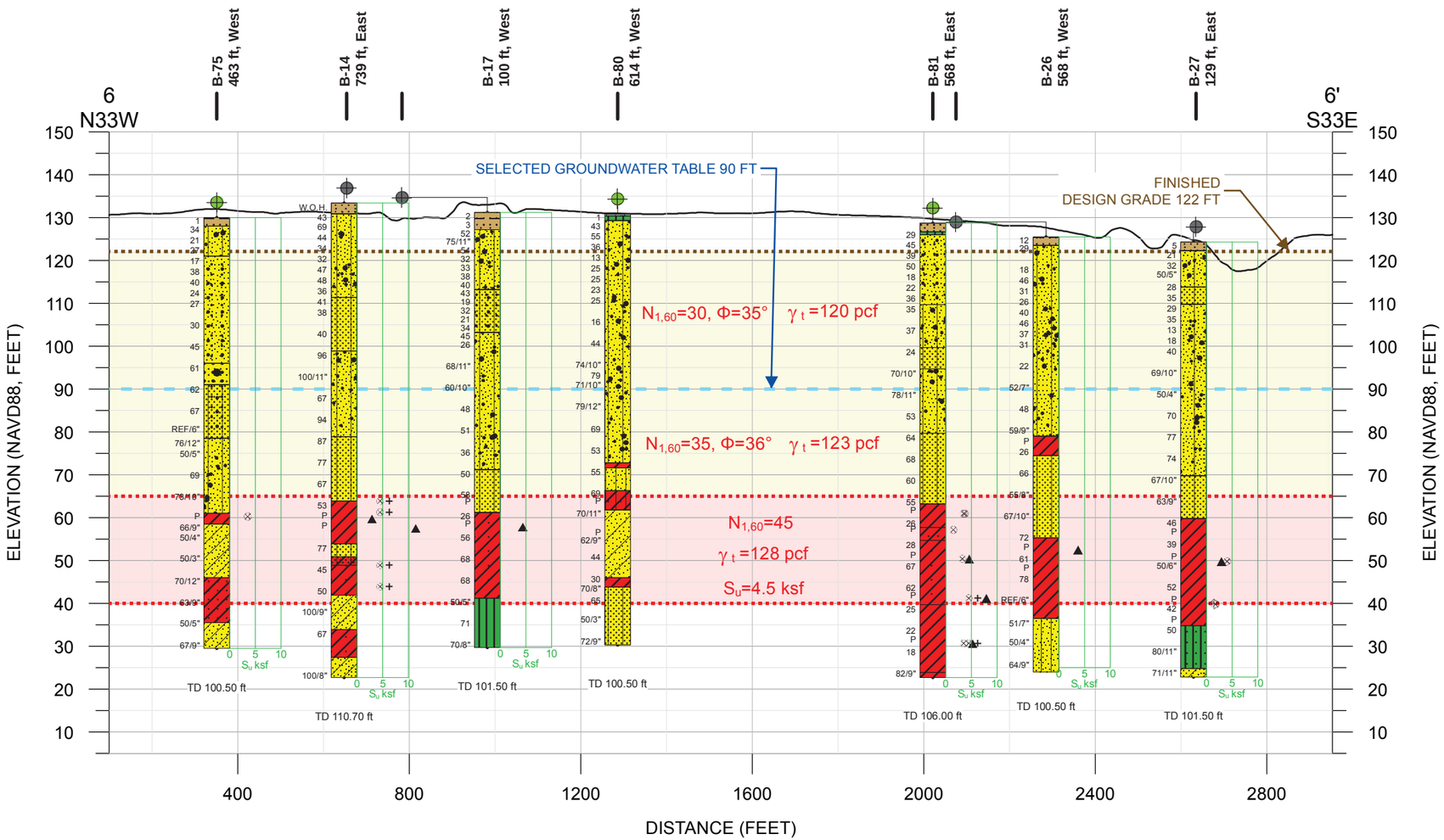
NOTES:

1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

SELECTED DESIGN PROFILE - AREA 5
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 21j

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-20e_Design-Profile.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2015)
- Boring (2014)
- Topographic Elevation (McLane, 2015)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean Clay with Gravel (CL)
- Sandy Lean CLAY (CL)
- Lean Clay with Sand (CL)
- Topsoil
- Gravel with Sand (GW)
- Silt (ML)
- Silt with Gravel (ML)
- Silt with Sand (ML)
- Peat
- Clayey Sand (SC)
- Silty Sand (SM)
- Sand (SP)
- Sand with Silt and Gravel (SW-SM)
- Sand with Silt and Gravel (SP-SM)
- Sand (SW)
- Sand with Gravel (SW)

30 ft

300 ft

Vertical Exaggeration = 10.0X

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

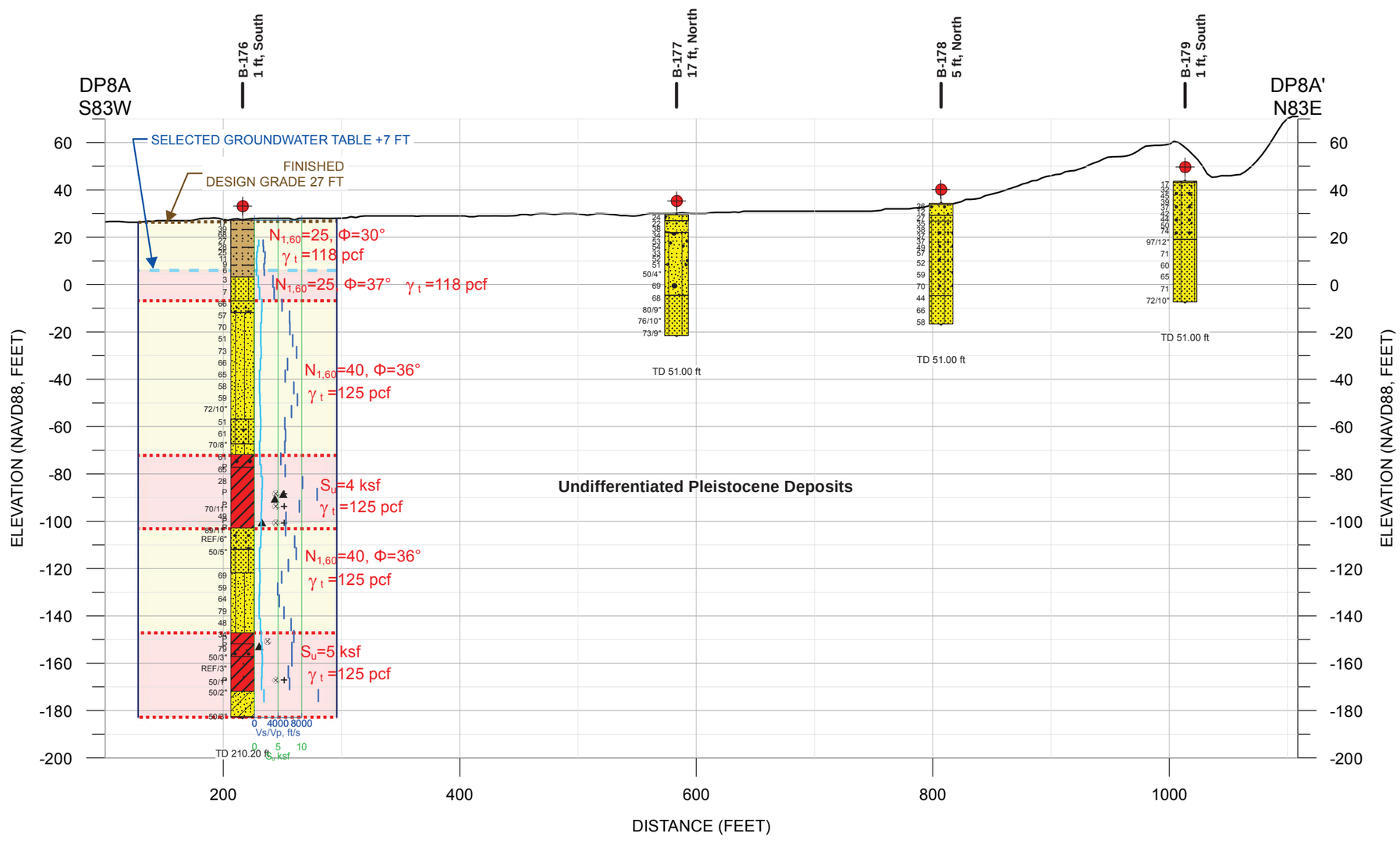
NOTES:

1. Topographic source data is provided by McLane Consulting, Inc.(2015) and Fugro (2015). Data is processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88.
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

SELECTED DESIGN PROFILE - AREA 6

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21f_Design-Profile.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (KPB, 2008)

PS-Logging

- Shear Wave Velocity (ft/s)
- P-Wave Velocity (ft/s)

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Lean CLAY with Sand (CL)
- Sandy, Gravelly Lean CLAY (CL)
- Gravelly Lean CLAY (CL)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Topsoil

50 ft

100 ft

Vertical Exaggeration = 2.0X

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

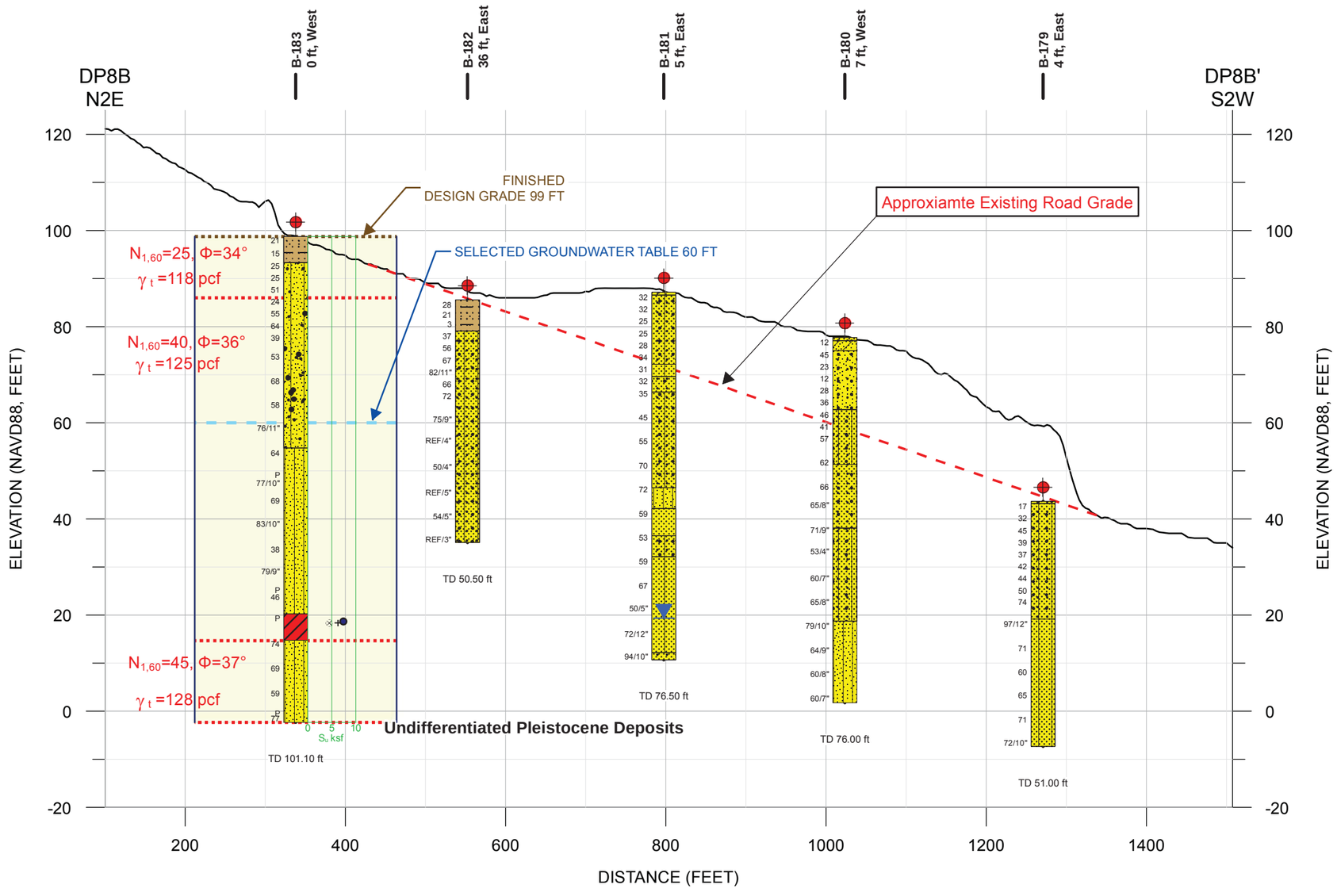
NOTES:

1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

SELECTED DESIGN PROFILE - AREA 8

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21I_DP_Area8_1.mxd, 1/18/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Boring (2015)
- Boring (2014)
- Topographic Elevation (KPB, 2008)
- Water Level Reading After Drilling

SOIL DESCRIPTIONS

- Lean CLAY (CL)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Silty SAND (SM)
- Topsoil

25 ft
150 ft
Vertical Exaggeration = 6.0X

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

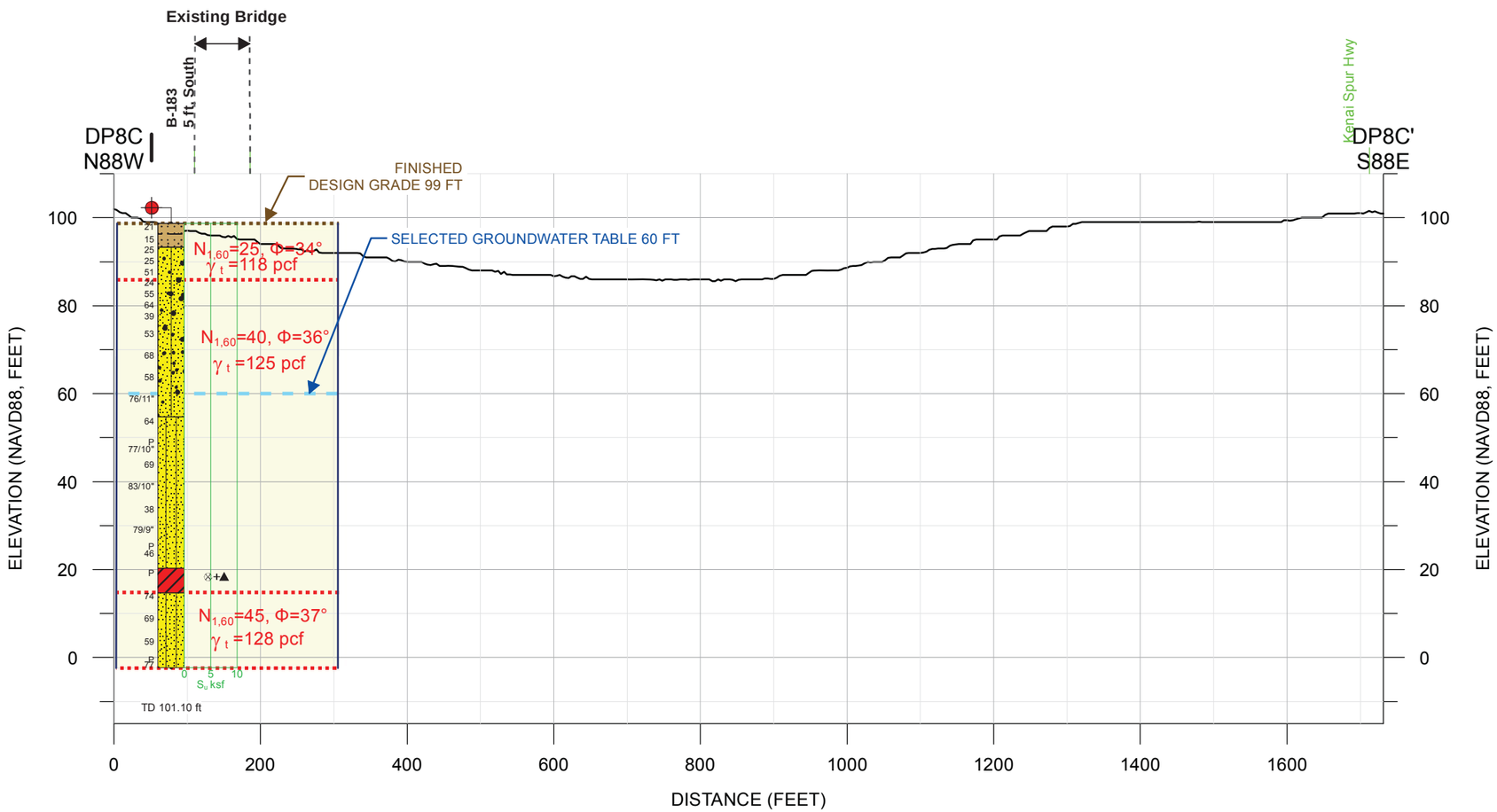
P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

SELECTED DESIGN PROFILE - AREA 8
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21m_Design-Profile.mxd, 2/1/2017, kimk



LEGEND

GEOTECHNICAL EXPLORATIONS

- Boring (2016)
- Topographic Elevation (KPB, 2008)

SOIL DESCRIPTIONS

- Topsoil
- Well-Graded SAND with Silt and Gravel (SW-SM)
- Silty SAND (SM)
- Lean CLAY (CL)

30 ft

180 ft

Vertical Exaggeration = 6.0X

UNDRAINED SHEAR STRENGTH (S_u)

- ▲ Unconsolidated Undrained Triaxial
- Torvane
- ⊗ Pocket Penetrometer
- + Soil Strength Exceeds Instrument's Maximum Measurable Strength.

TUBE AND SPT SAMPLES

P	Push thin-walled 3" tube.
20	Number of blows to produce 12" of penetration after the initial 6" of seating.
86/11"	Number of blows required to produce the indicated penetration after an initial 6" seating.
Ref/3"	50 blows produced the indicated penetration during the initial 6" interval.
W.O.H.	Weight of Hammer

NOTES:

1. Topographic source data is from Kenai Peninsula Borough LiDAR, collected in 2008 and processed using 4 feet bin interval. Topographic elevation is referenced to NAVD88. Data provided by EXP Geomatics (EXP).
2. As-built coordinates of exploration locations provided by JOA Surveying Services, Inc. (2014) and McLane Consulting, Inc.(2015, 2016).
3. Stratigraphic contacts are approximate, and interpreted from borings. Conditions vary both along and perpendicular to the section line. The lateral extent of the top soil is not known based on the limited borings.
4. Boring data are projected onto the cross section line, therefore, stratigraphic contacts may not correspond to the descriptions (lithology, shear strength, etc.) on the logs.
5. Material descriptions are generalized. Materials may vary within the stratigraphic unit and include layers of material that differ from the general description. Refer to boring logs for detailed descriptions of the materials encountered at the exploration location.
6. See Plate 20 for location of explorations and cross section lines.
7. Elevation (MLLW, ft) = Elevation (NAVD88, ft) + 7.32 ft.

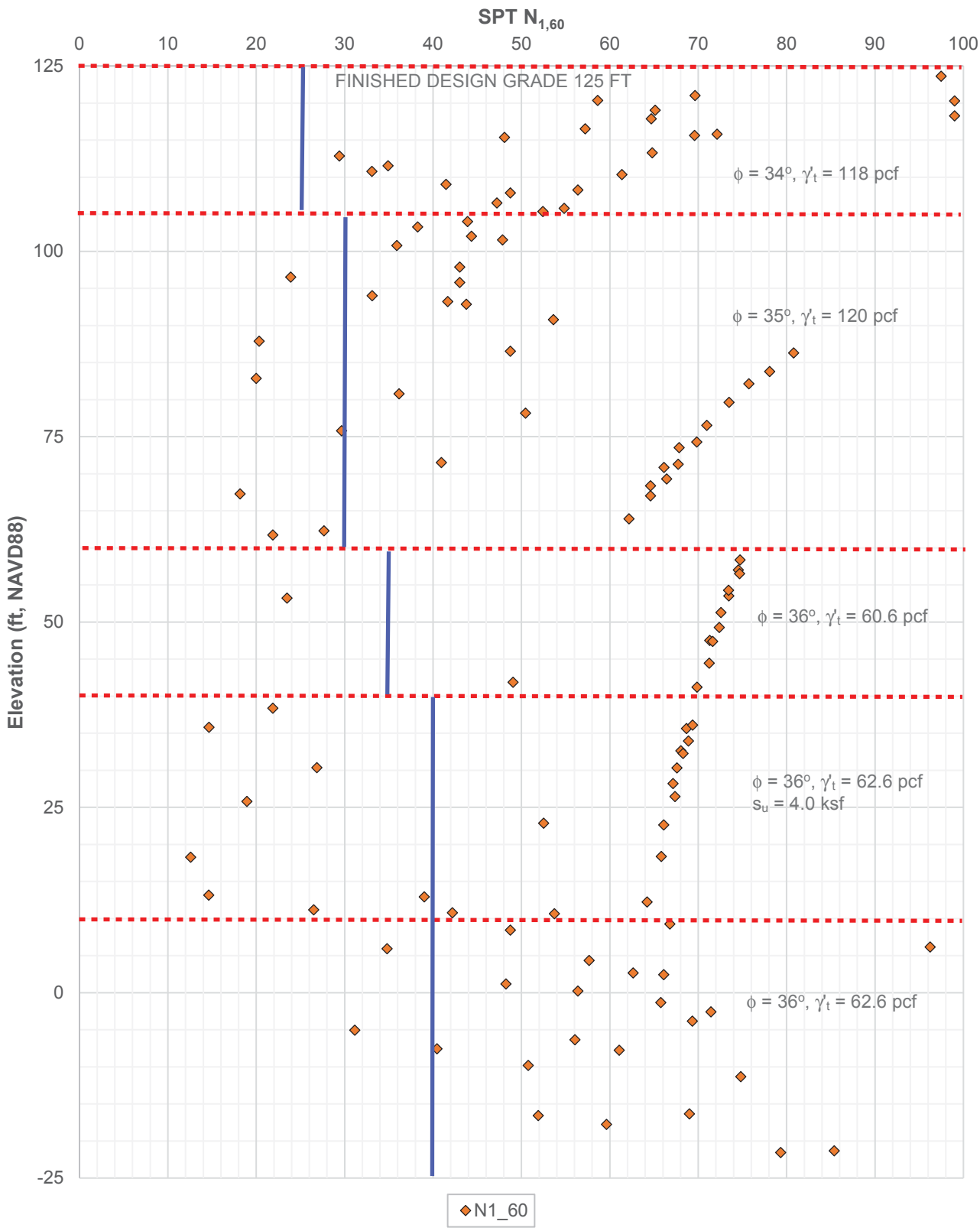
SELECTED DESIGN PROFILE - AREA 8
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

R:\04100\2016 Projects\0001-0099\04.10160001 - AKLNG\07_GIS\Outputs\01_REPORTS\2016_11_04_IntegratedReport\MXD\Area Profile\Plate-21n_Design-Profile.mxd, 2/1/2017, kimk

SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 1 (HA1-HA1')

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

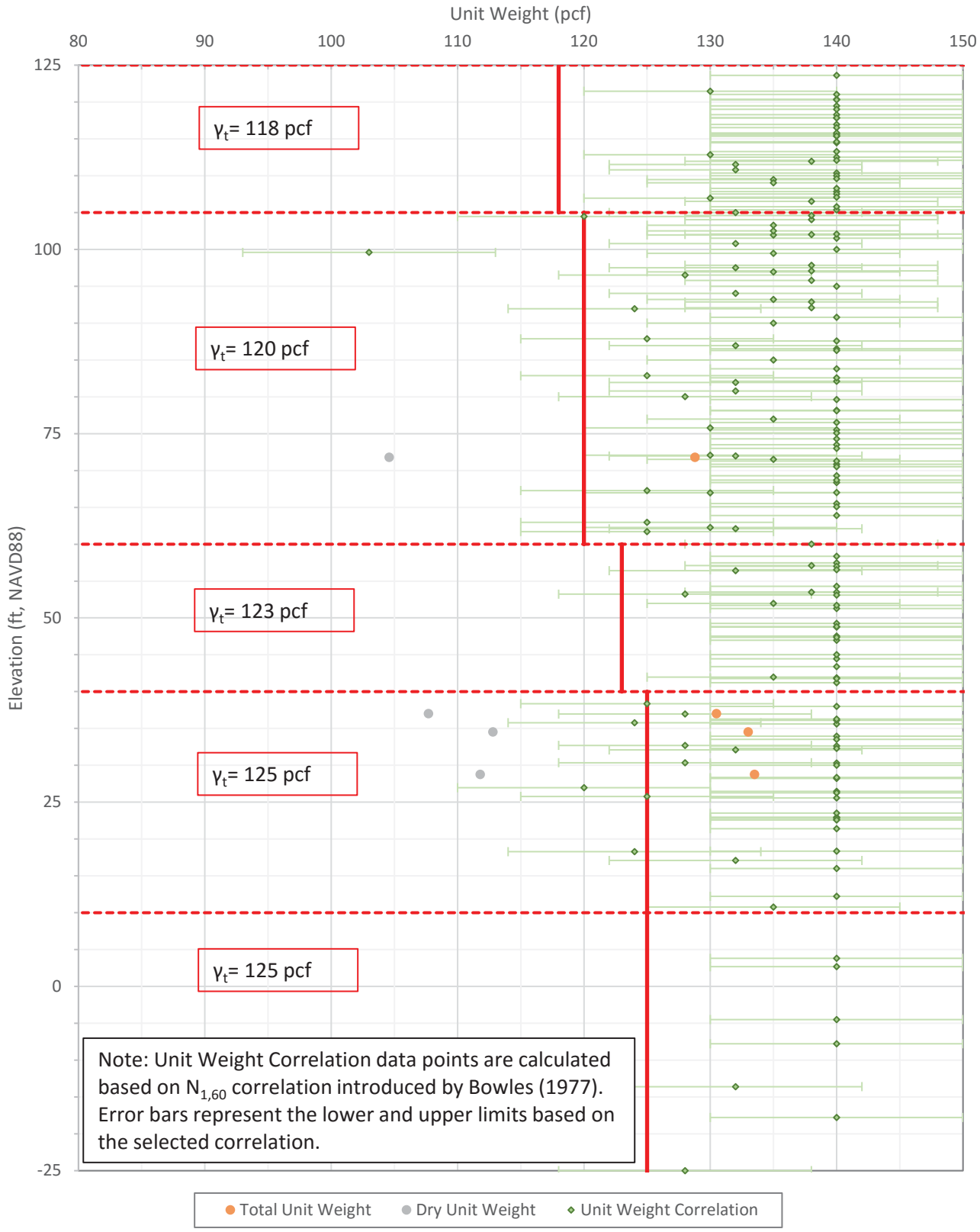
PLATE 22a



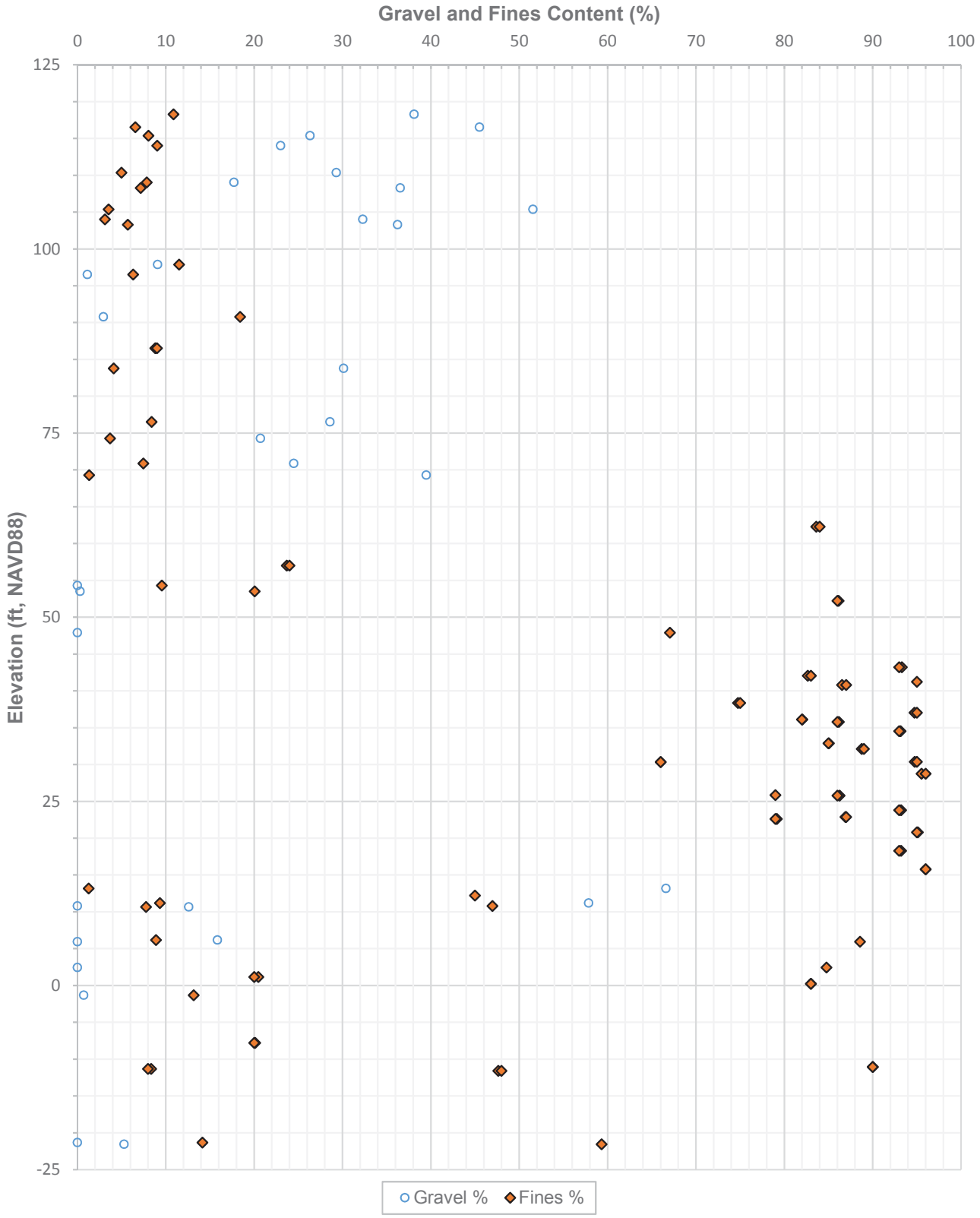
SPT $N_{1,60}$ WITH SELECTED PARAMETERS - AREA 1 (HA1-HA1')

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 22b

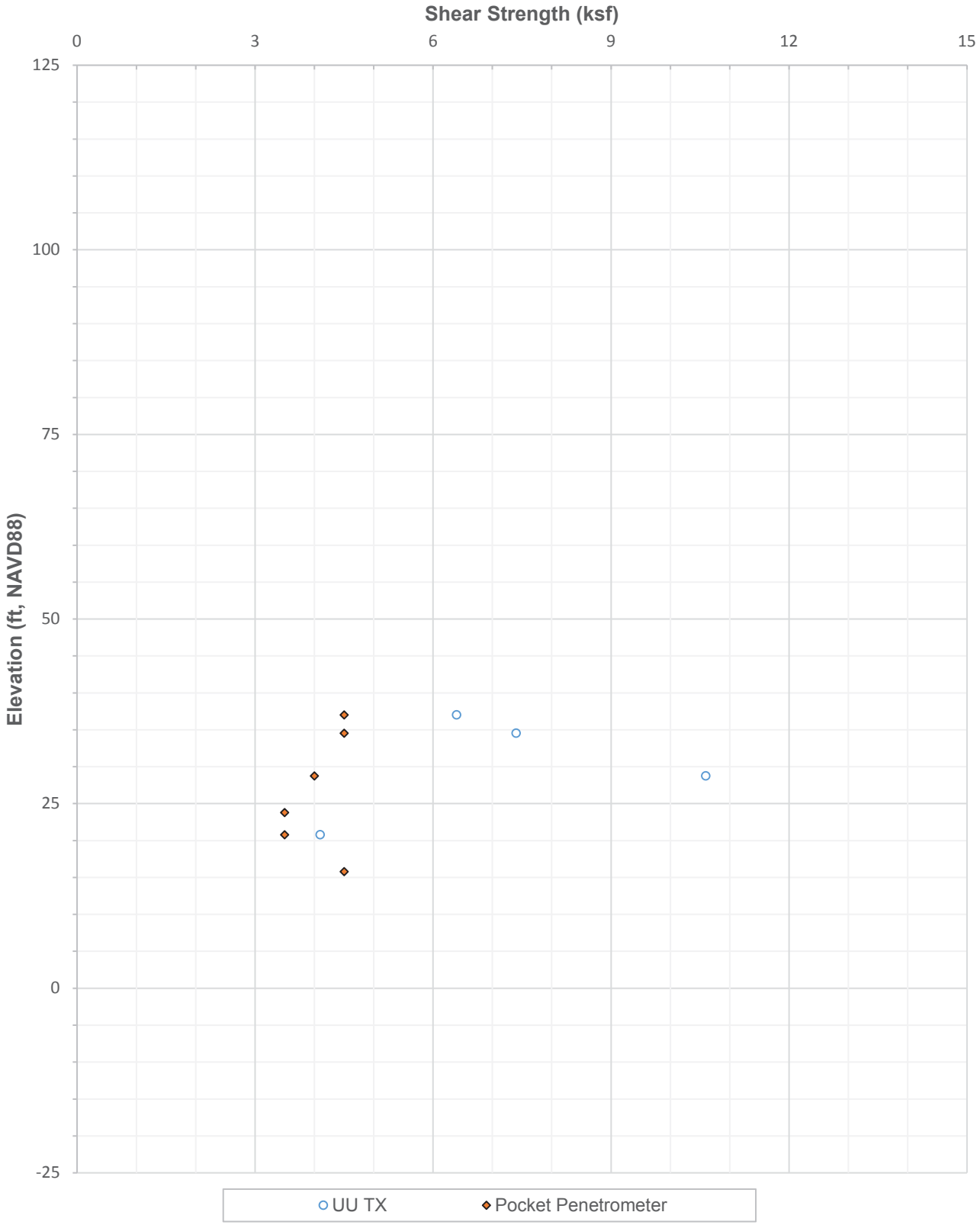


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 1 (HA1-HA1')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



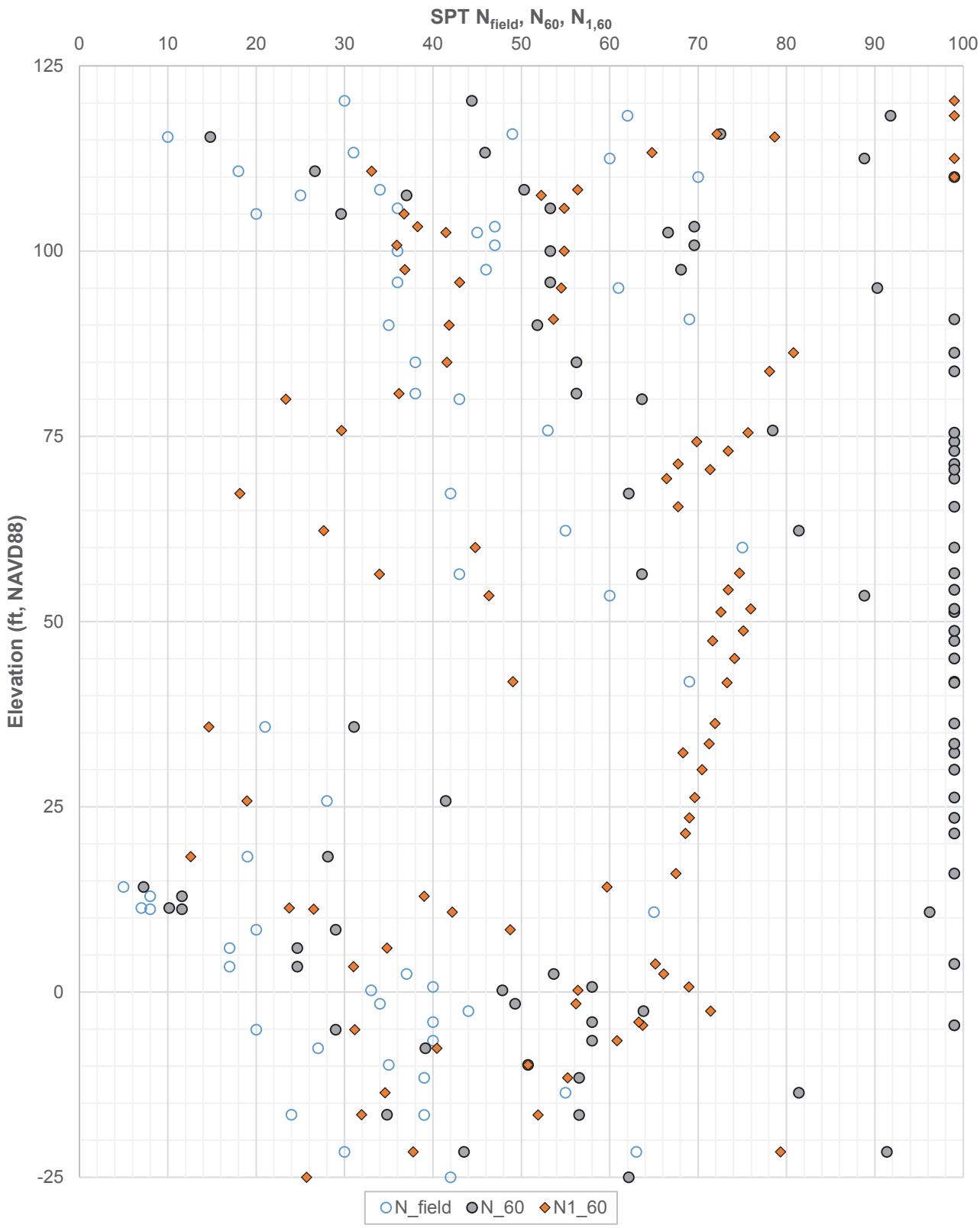
GRAVEL AND FINES CONTENT - AREA 1 (HA1-HA1')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 22d



UNDRAINED SHEAR STRENGTH - AREA 1 (HA1-HA1')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

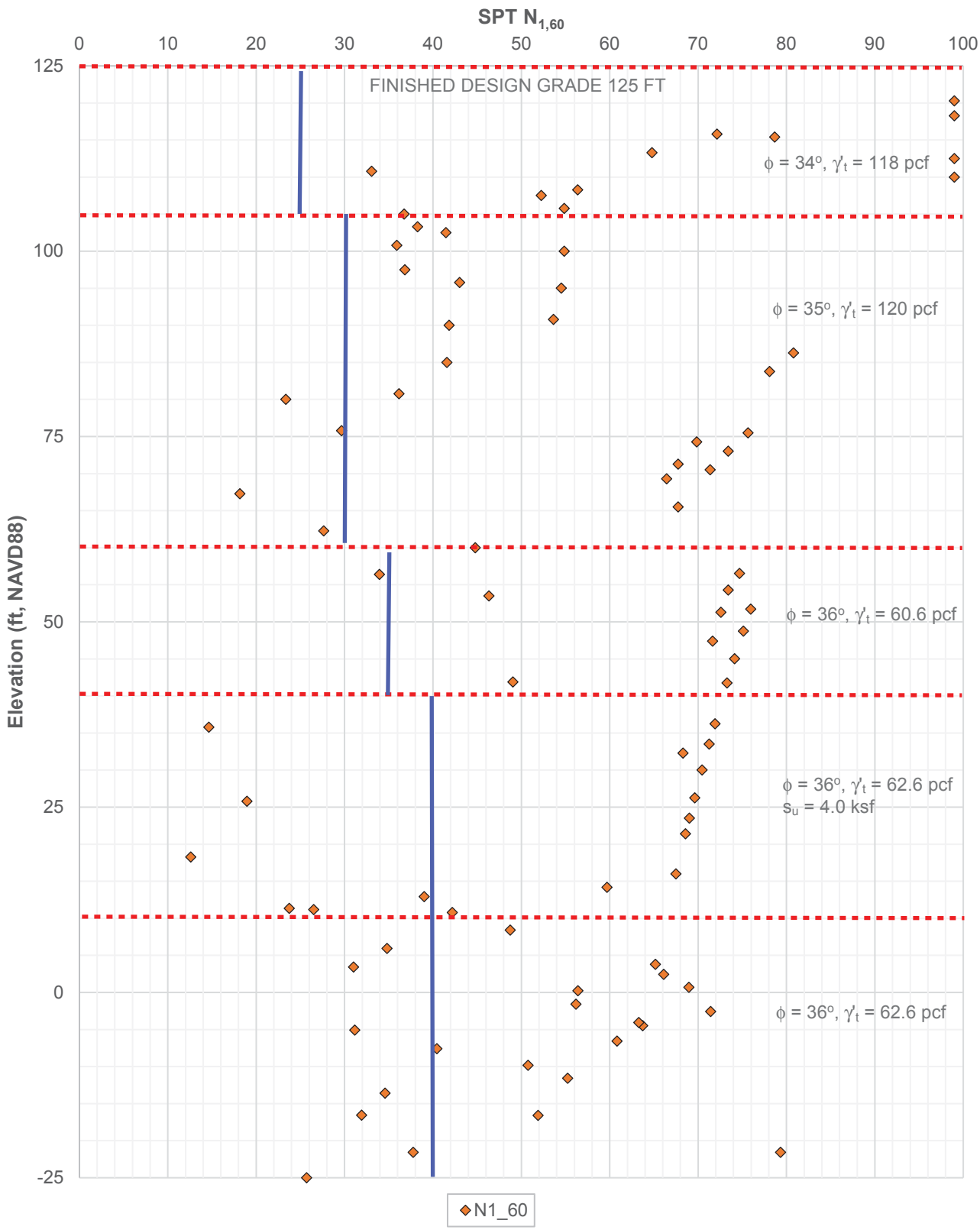
PLATE 22e



SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 1 (HA2-HA2')

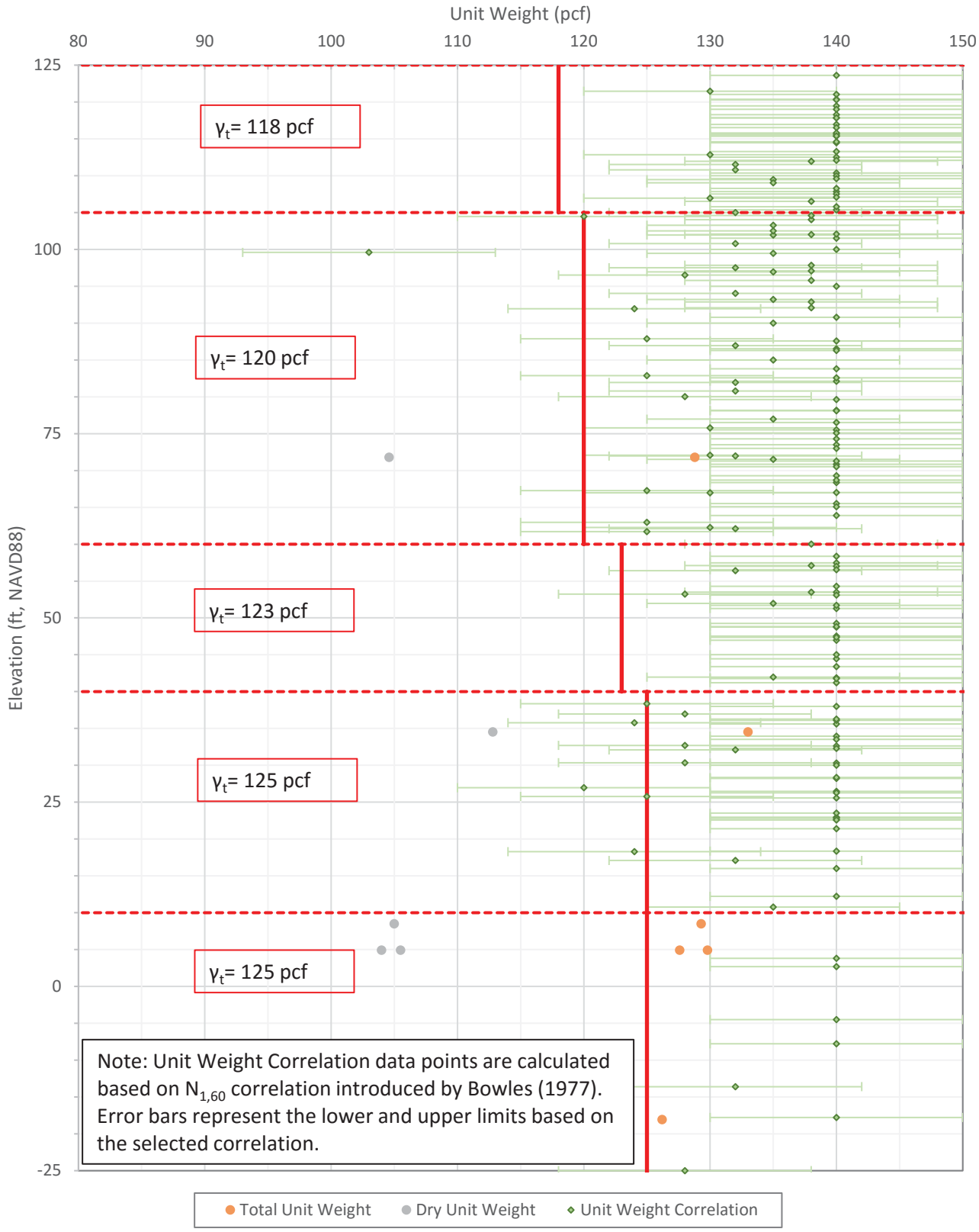
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 23a

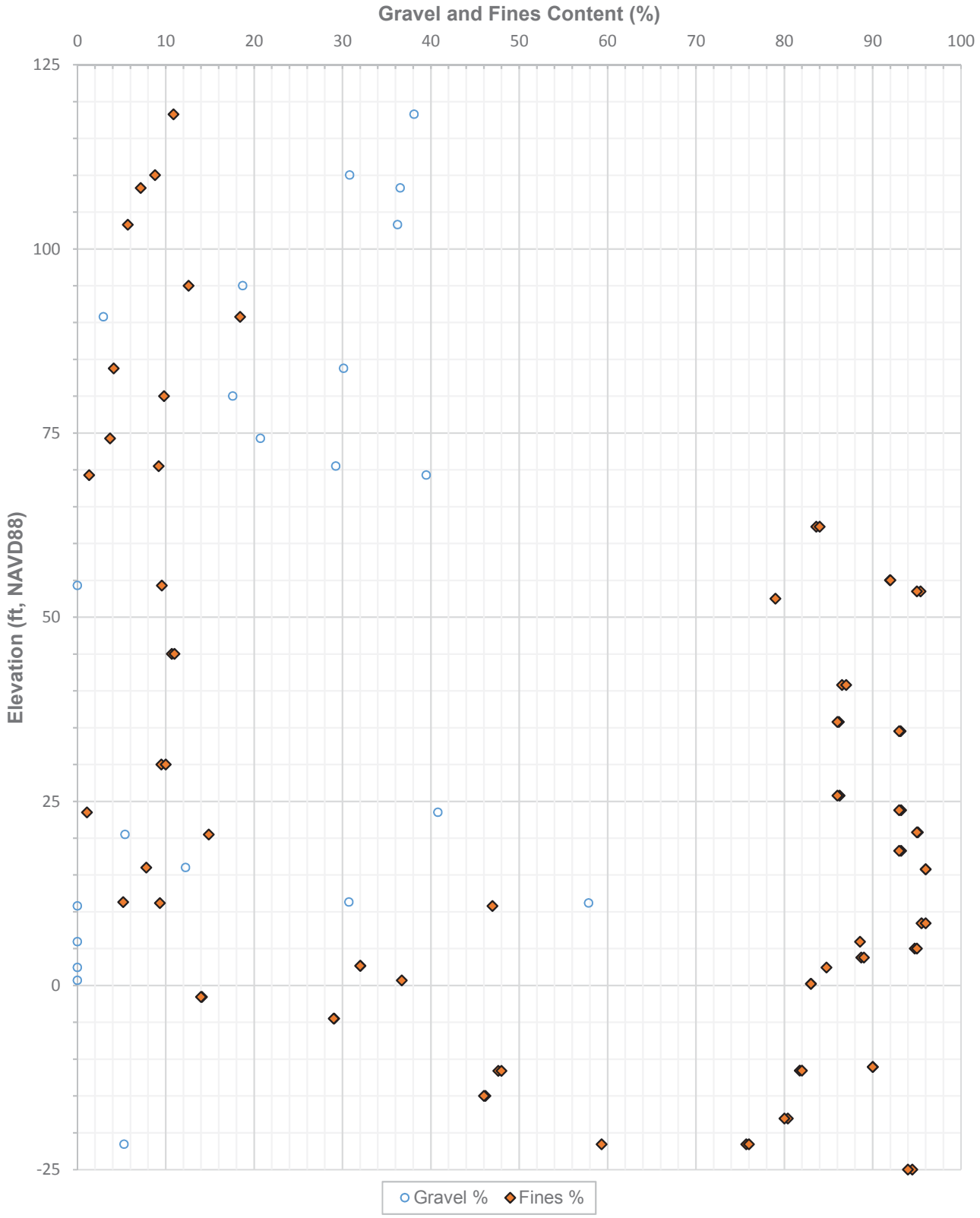


SPT $N_{1,60}$ WITH SELECTED PARAMETERS - AREA 1 (HA2-HA2')

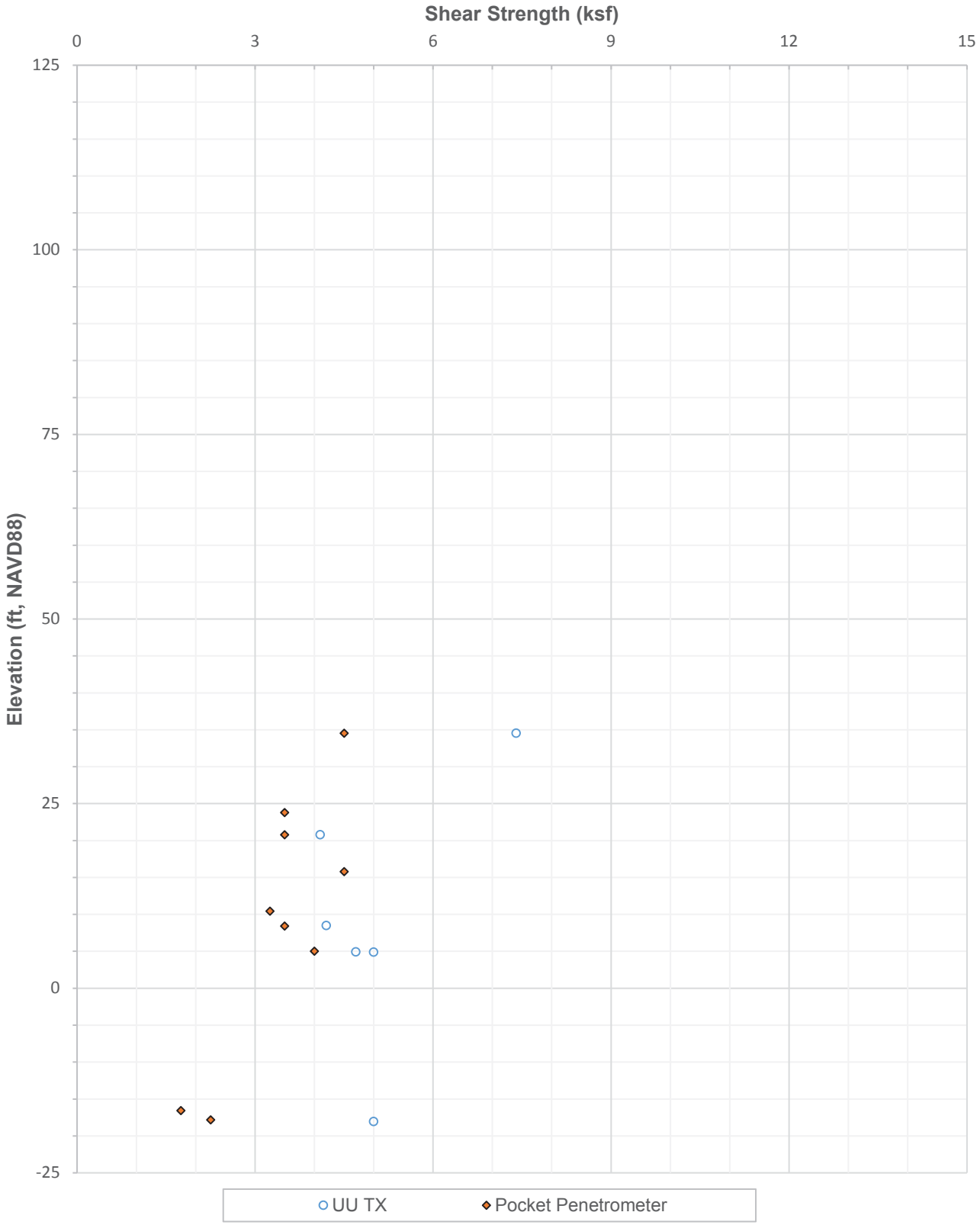
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



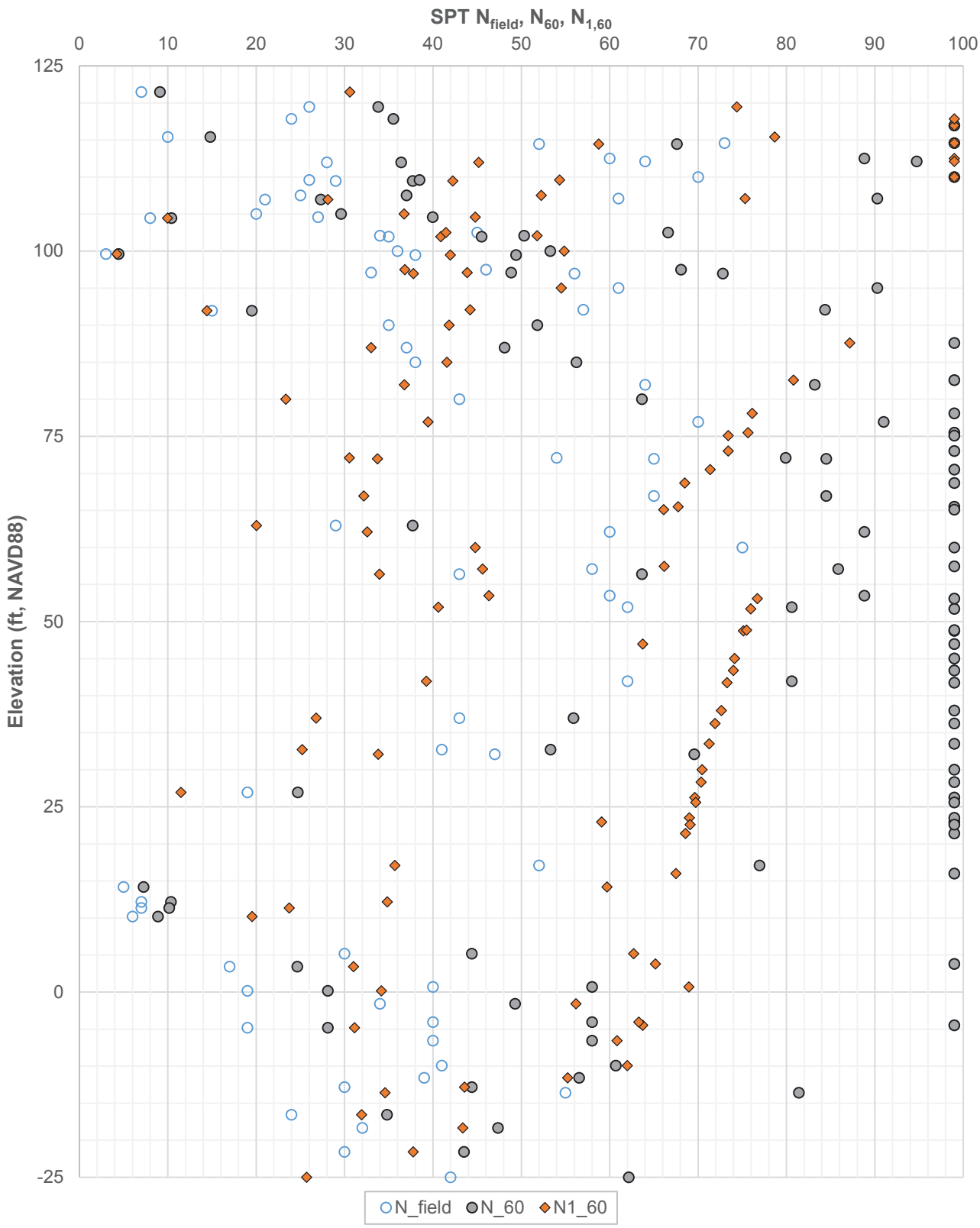
TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 1 (HA2-HA2')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



GRAVEL AND FINES CONTENT - AREA 1 (HA2-HA2')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



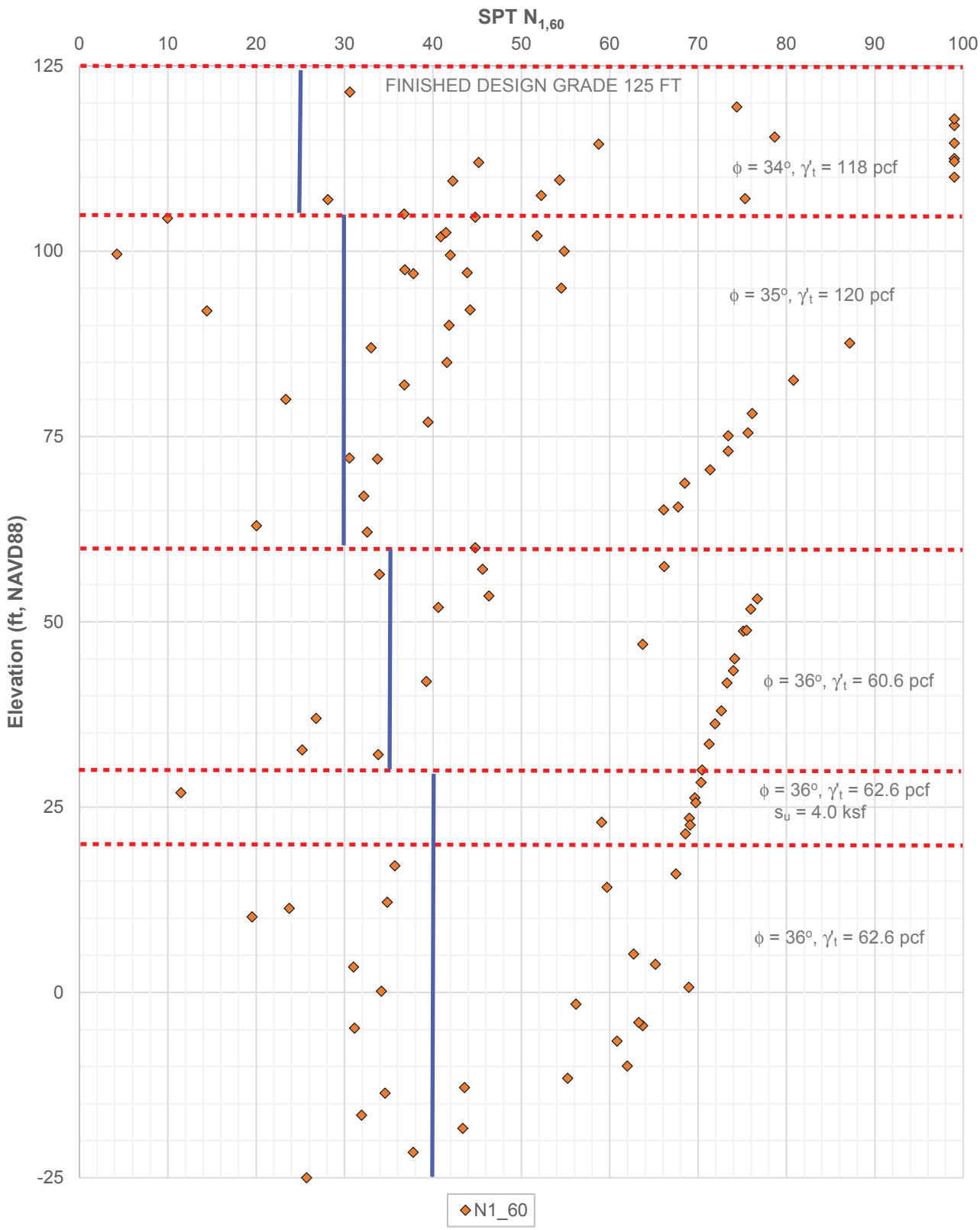
UNDRAINED SHEAR STRENGTH - AREA 1 (HA2-HA2')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 1 (HA3-HA3')

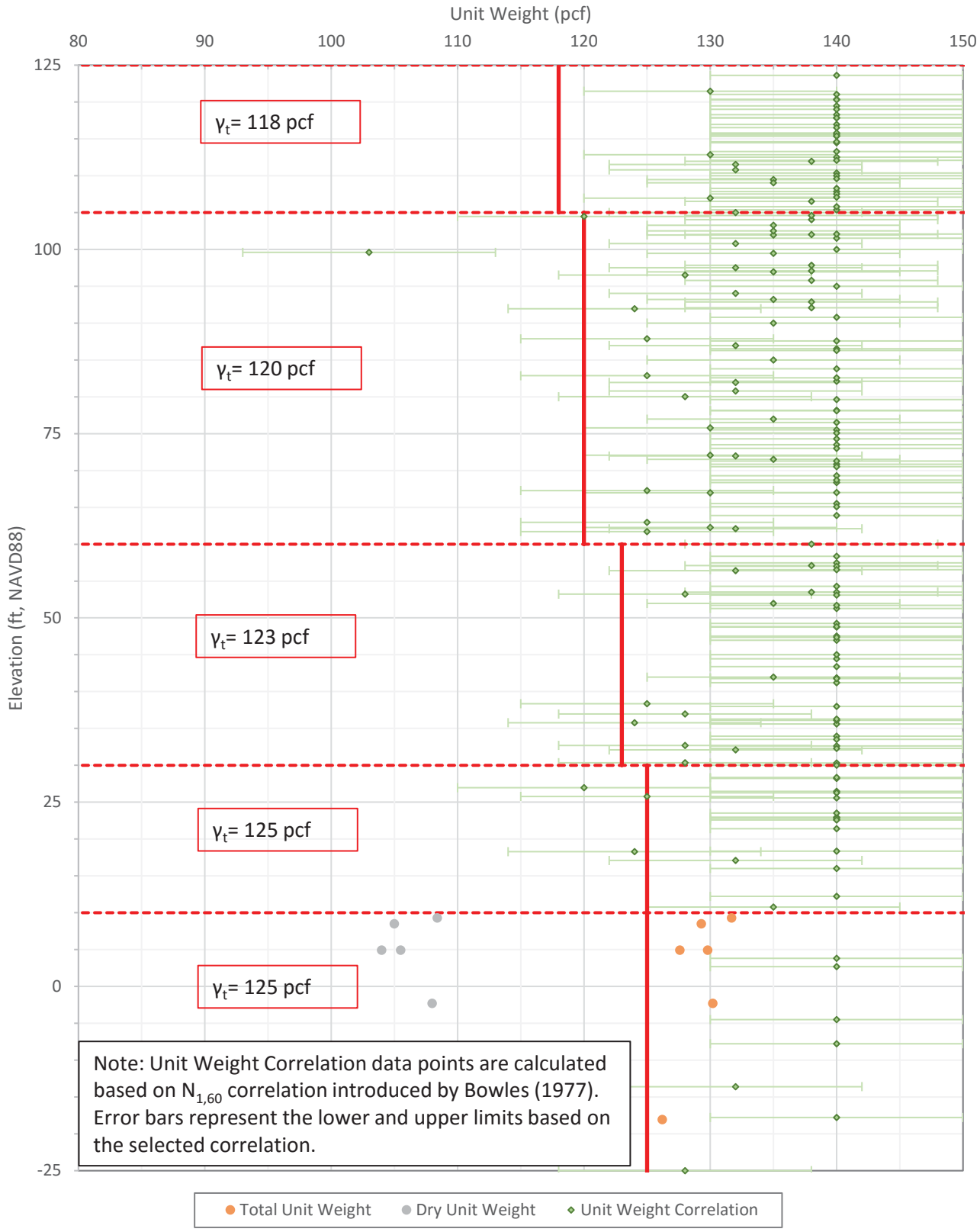
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 24a

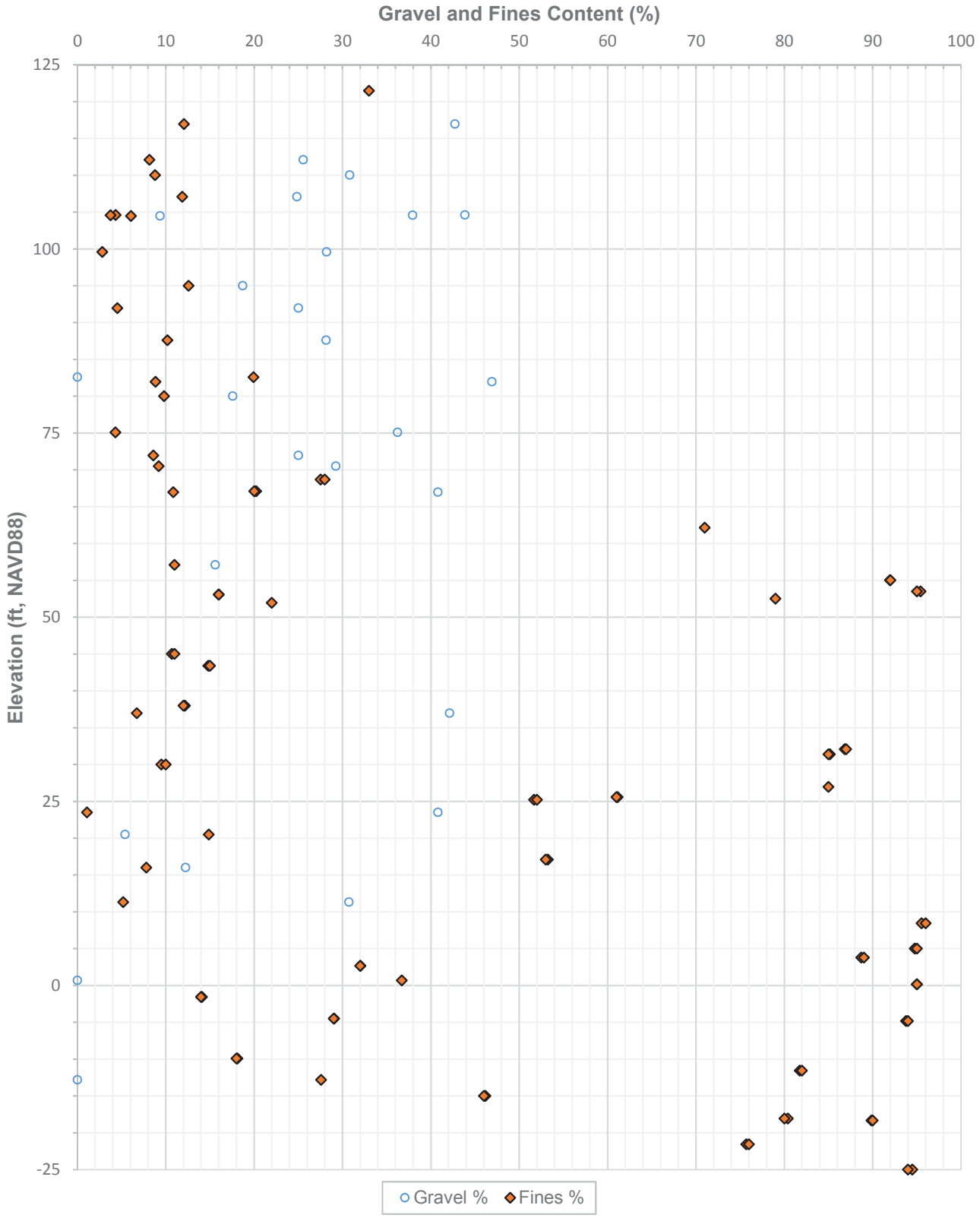


SPT $N_{1,60}$ WITH SELECTED PARAMETERS - AREA 1 (HA3-HA3')

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

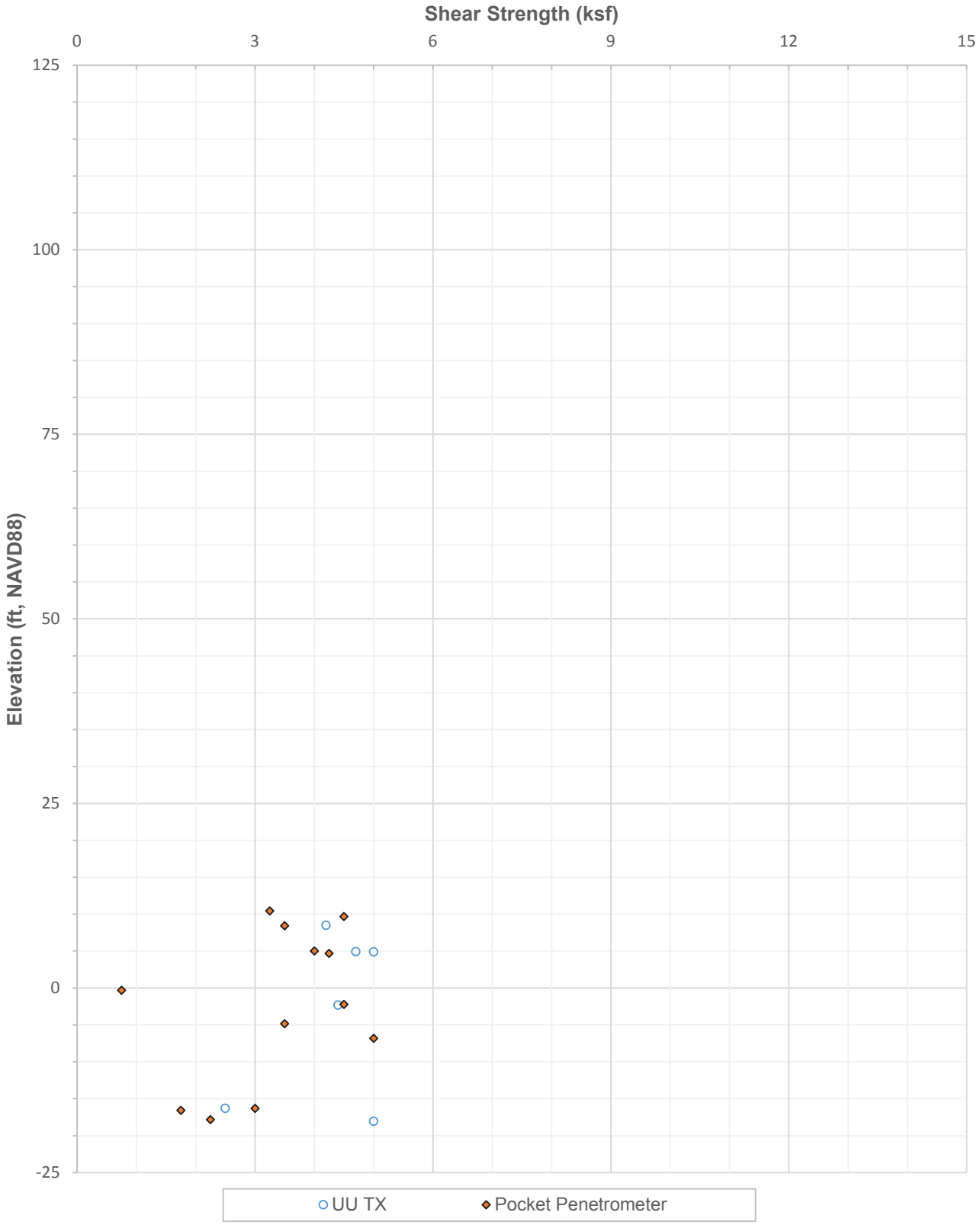


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 1 (HA3-HA3')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

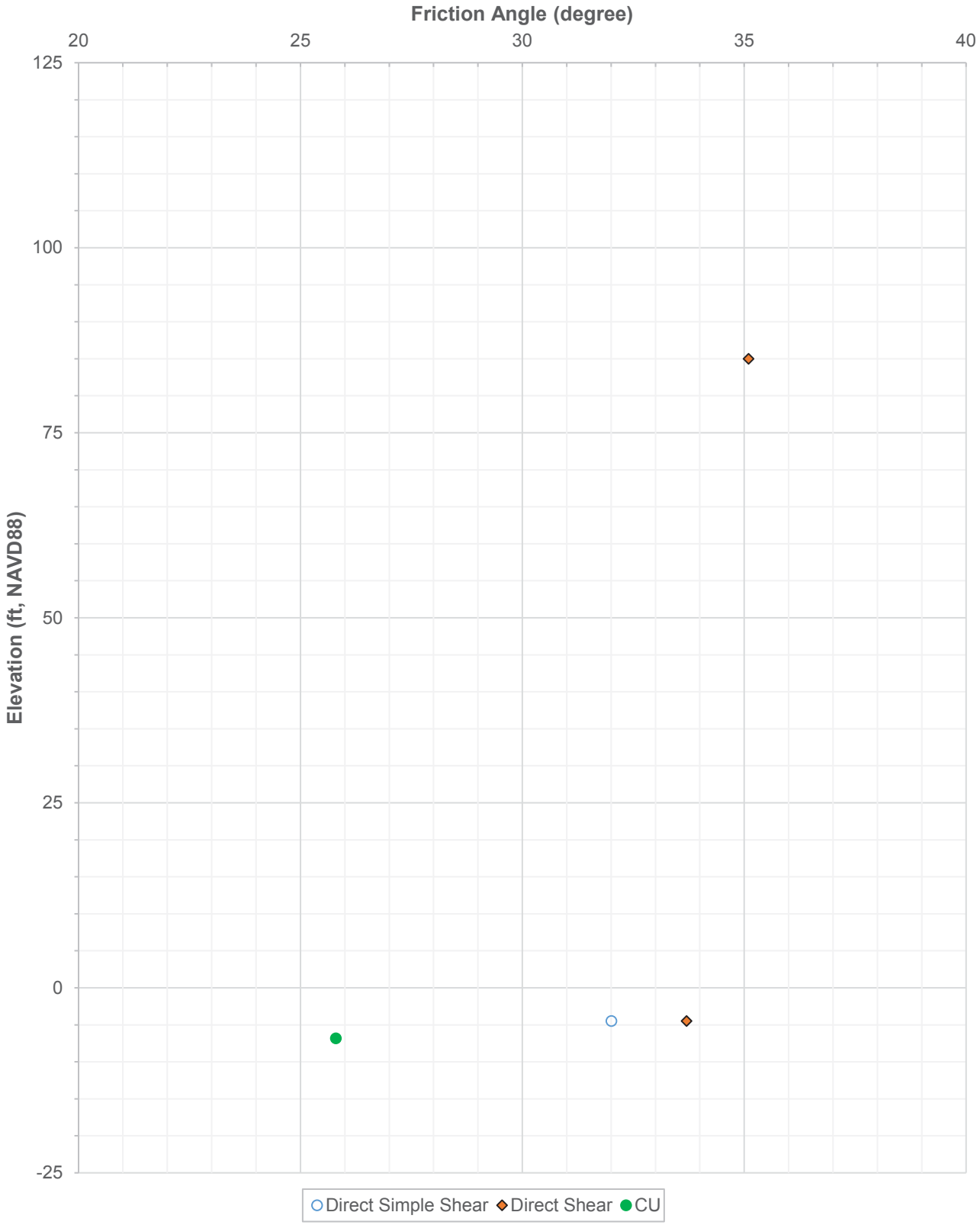


GRAVEL AND FINES CONTENT - AREA 1 (HA3-HA3')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 24d

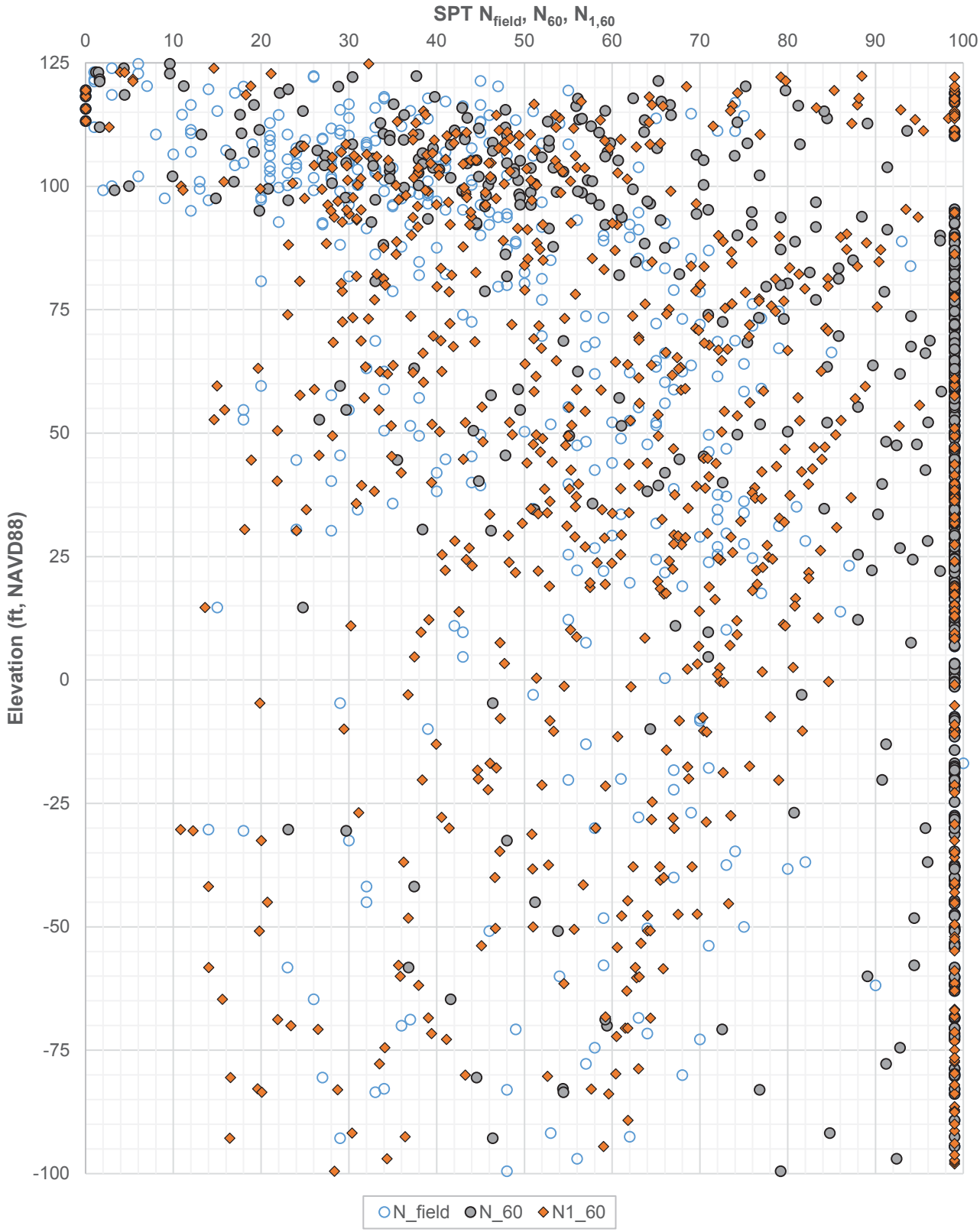


UNDRAINED SHEAR STRENGTH - AREA 1 (HA3-HA3')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

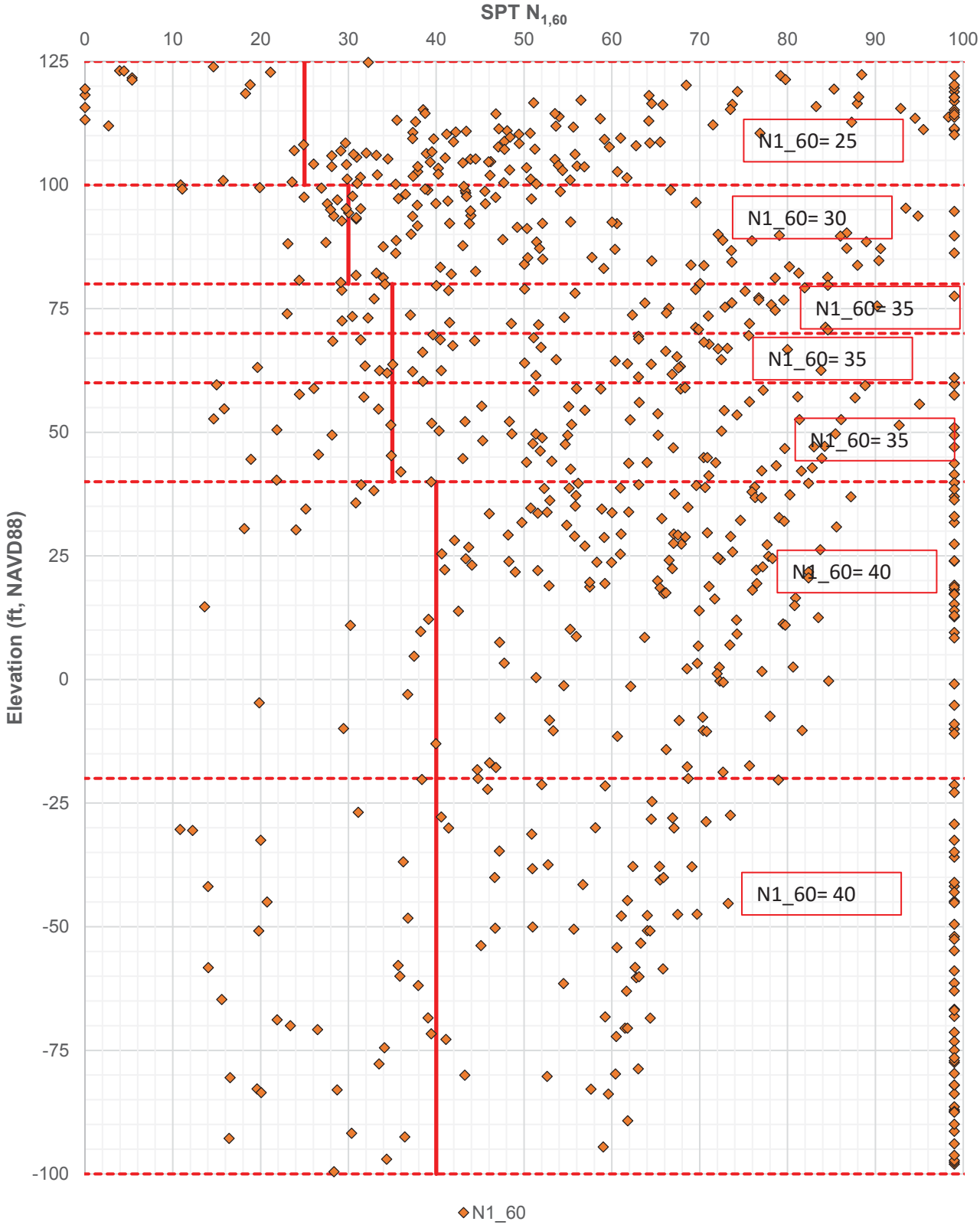


FRICTION ANGLES - AREA 1 (HA3-HA3')
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

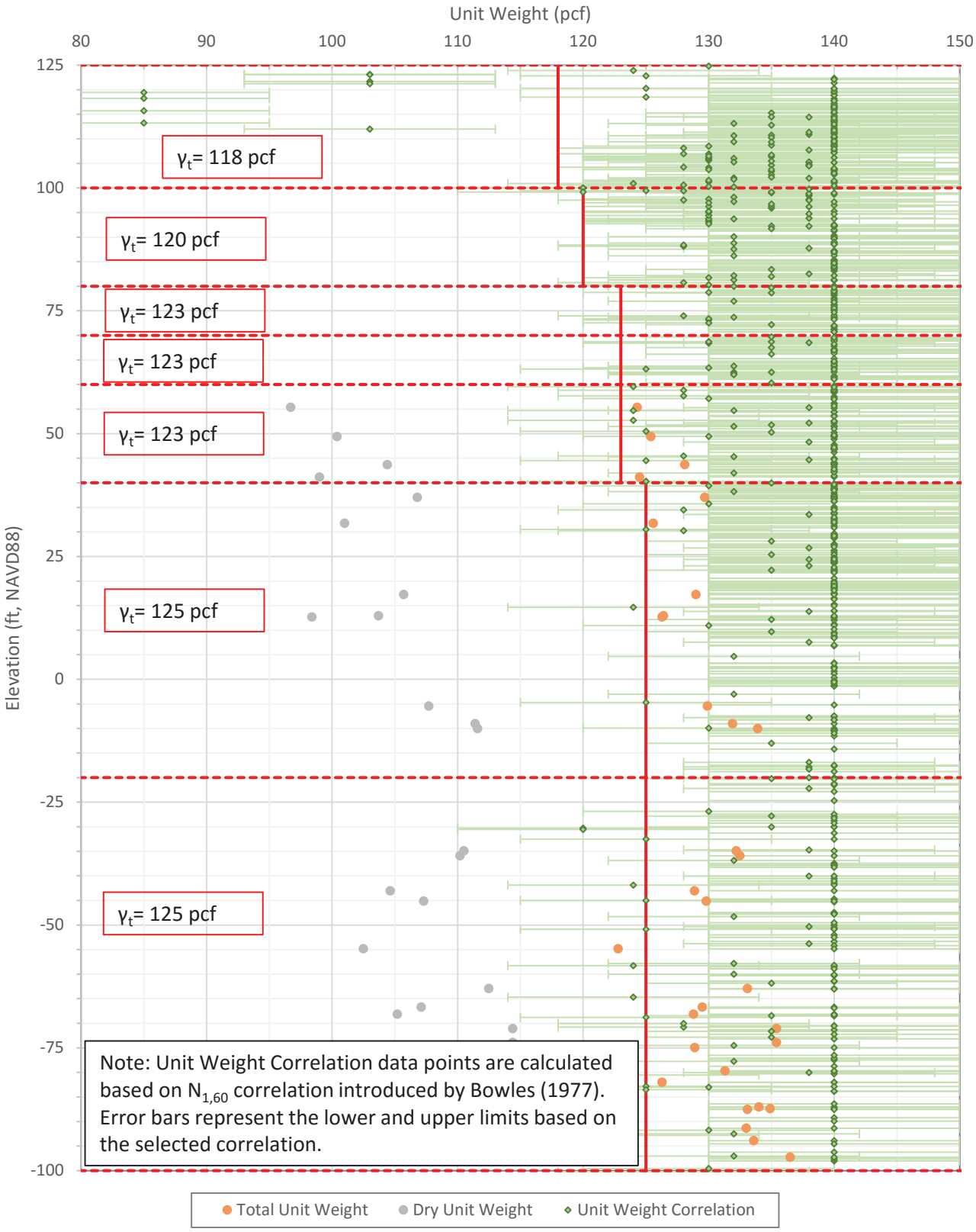
PLATE 24f



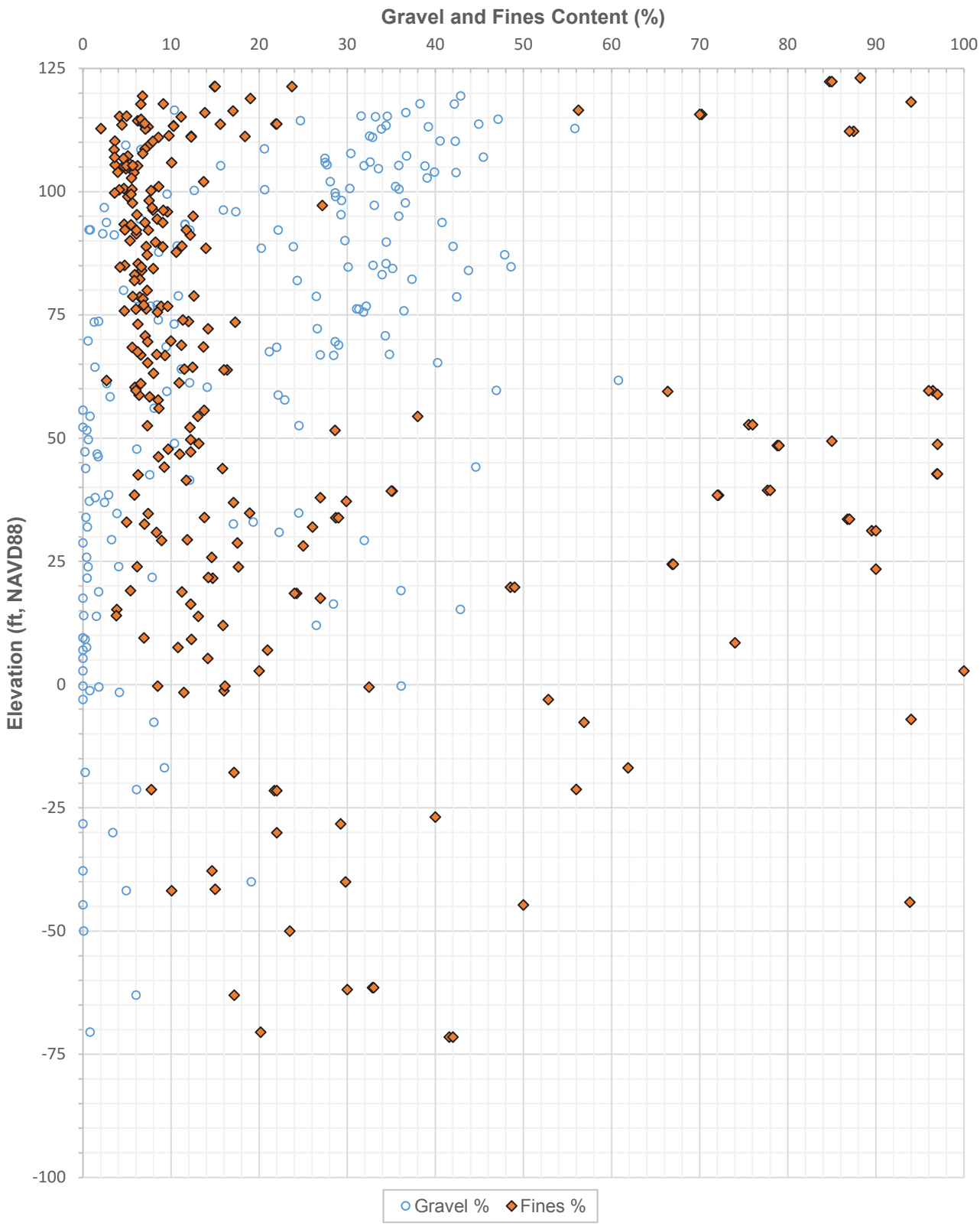
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT $N_{1,60}$ - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

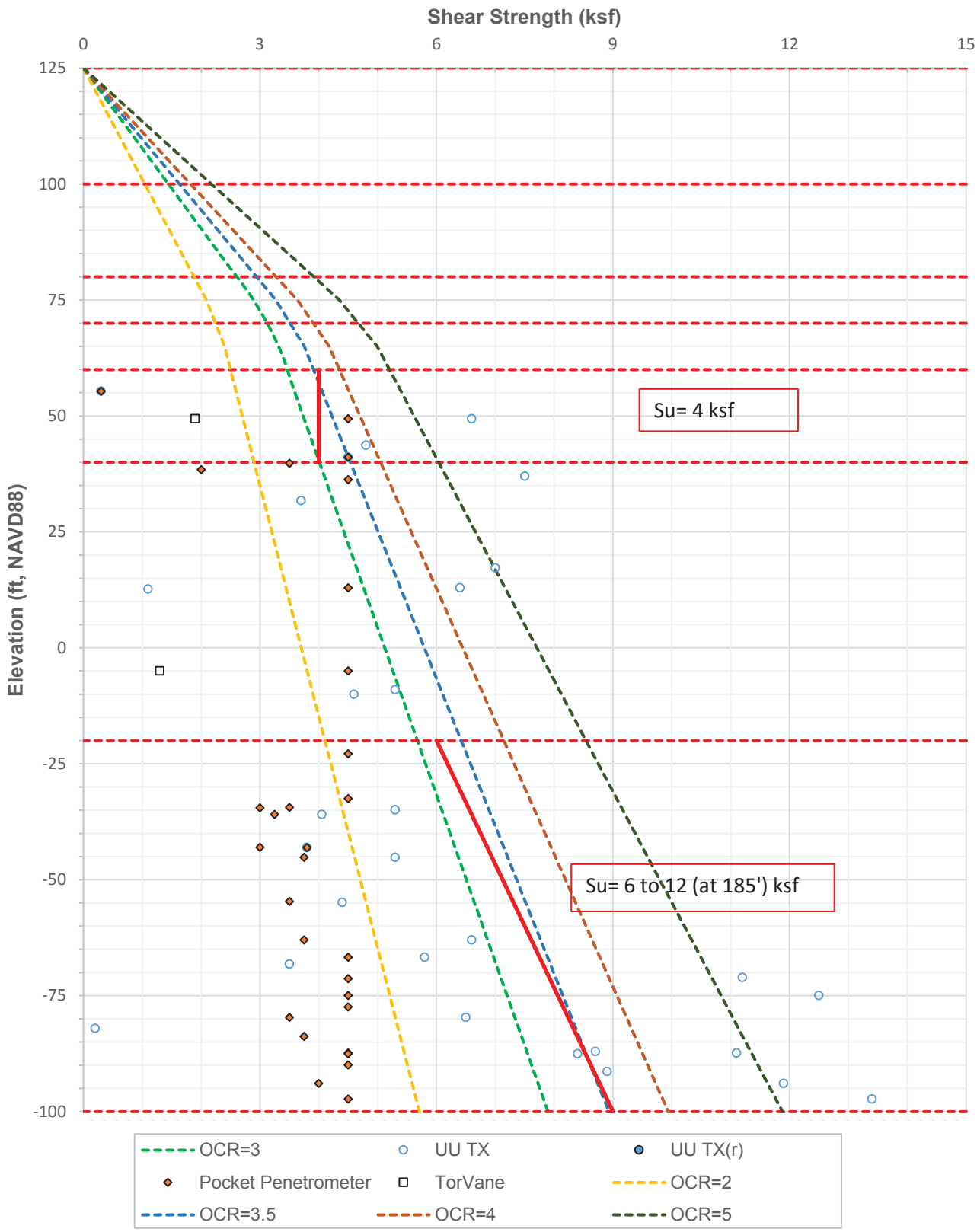


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

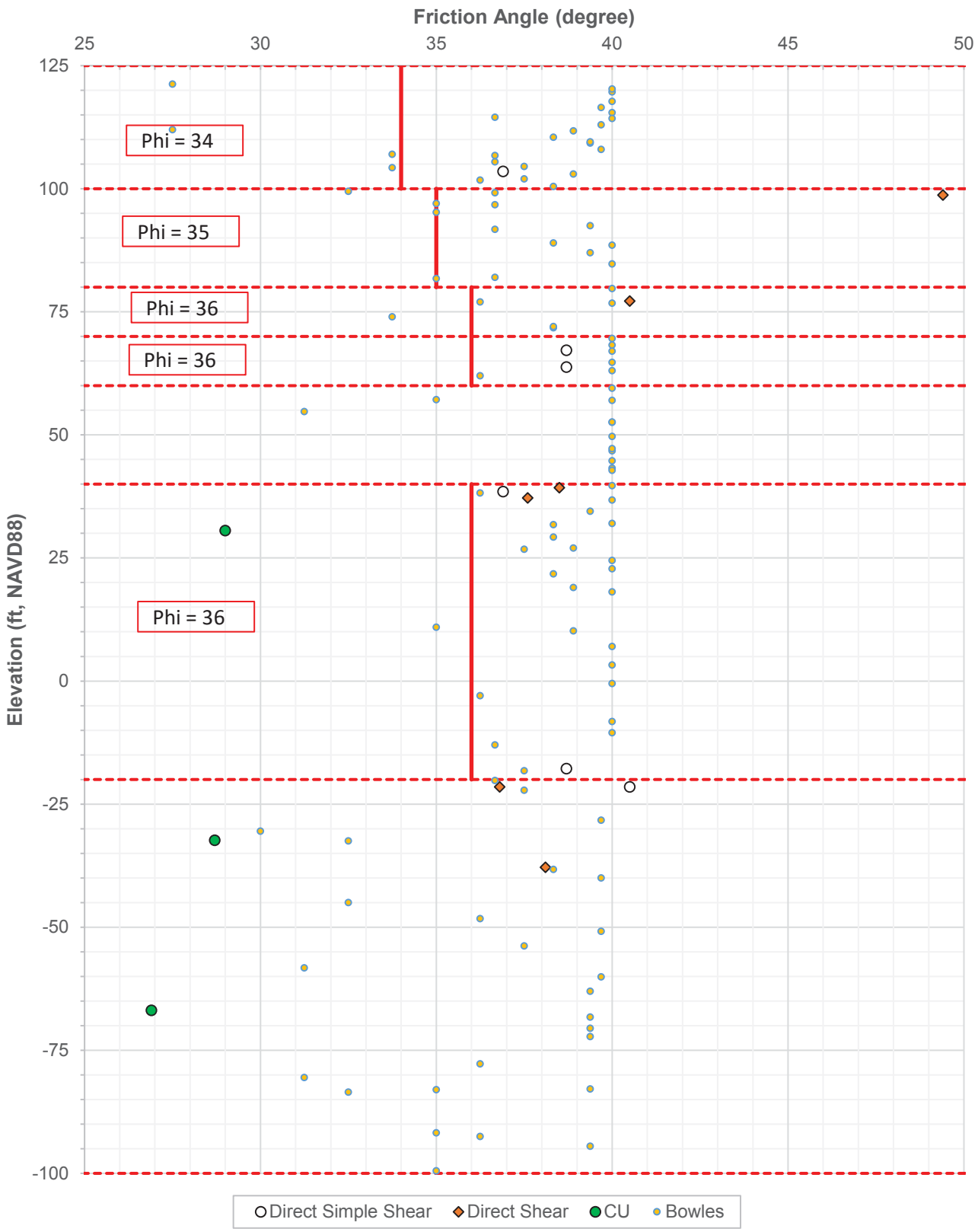


GRAVEL AND FINES CONTENT - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

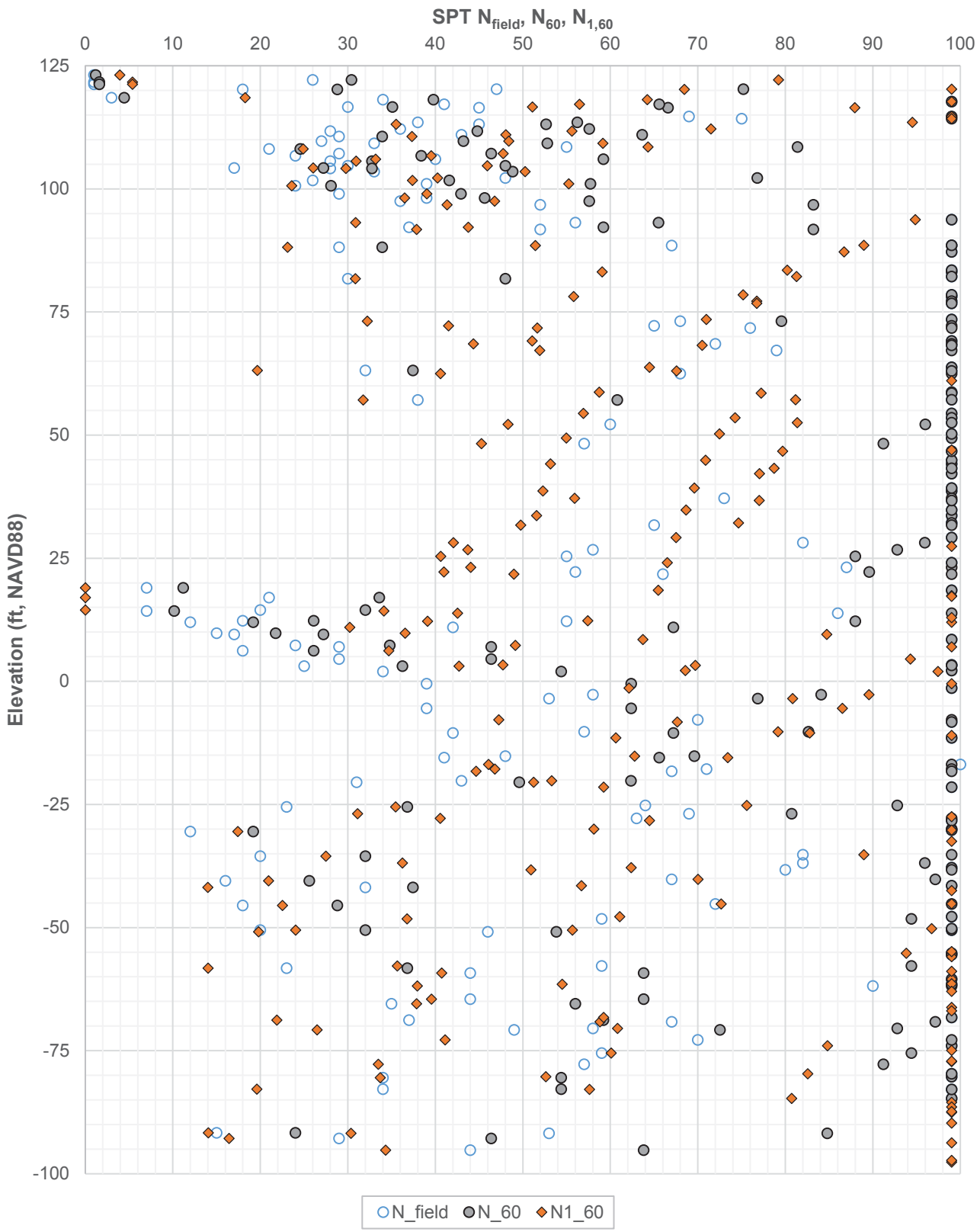
PLATE 25d



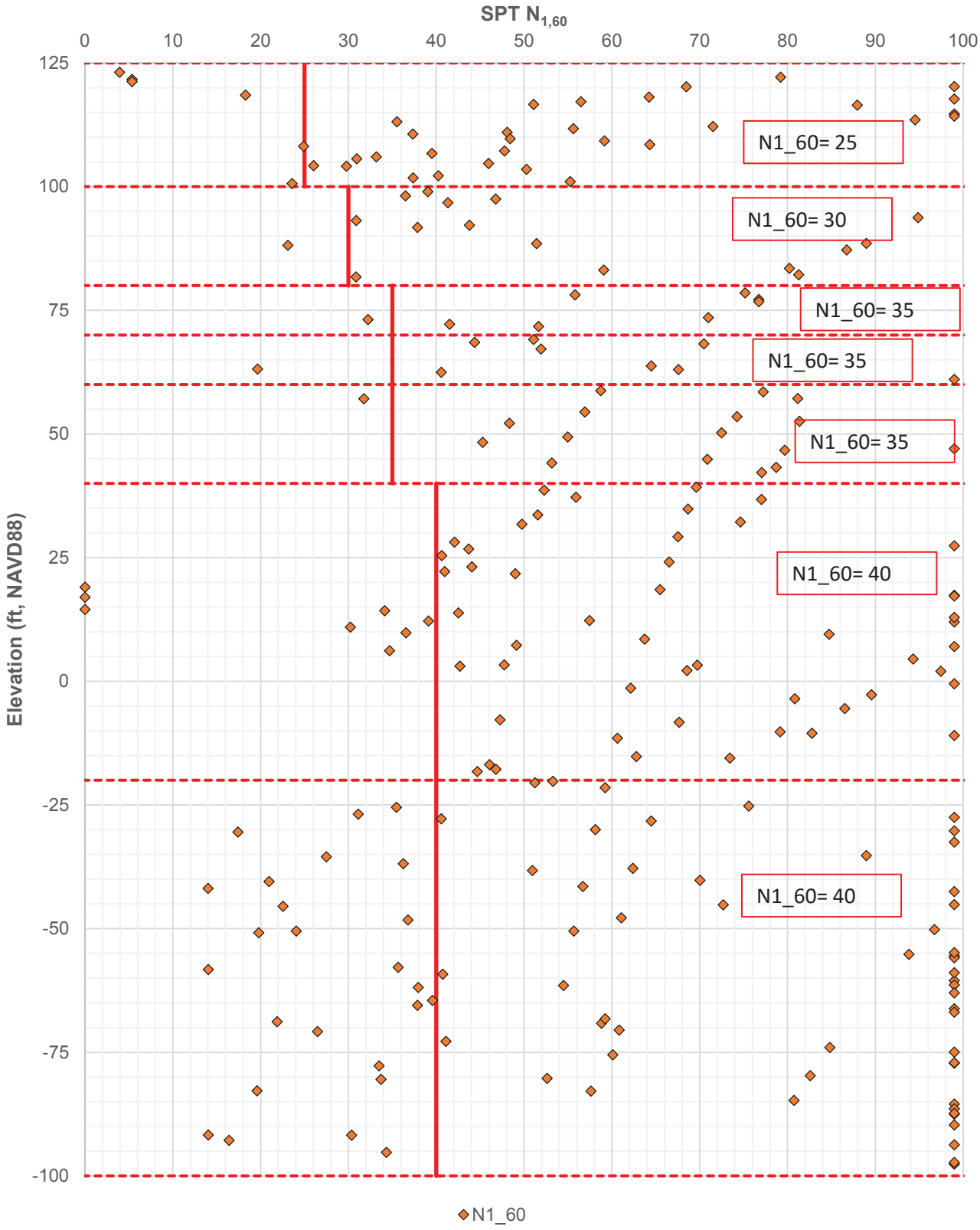
UNDRAINED SHEAR STRENGTH - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



FRICTION ANGLES - AREA 2A (DP2A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

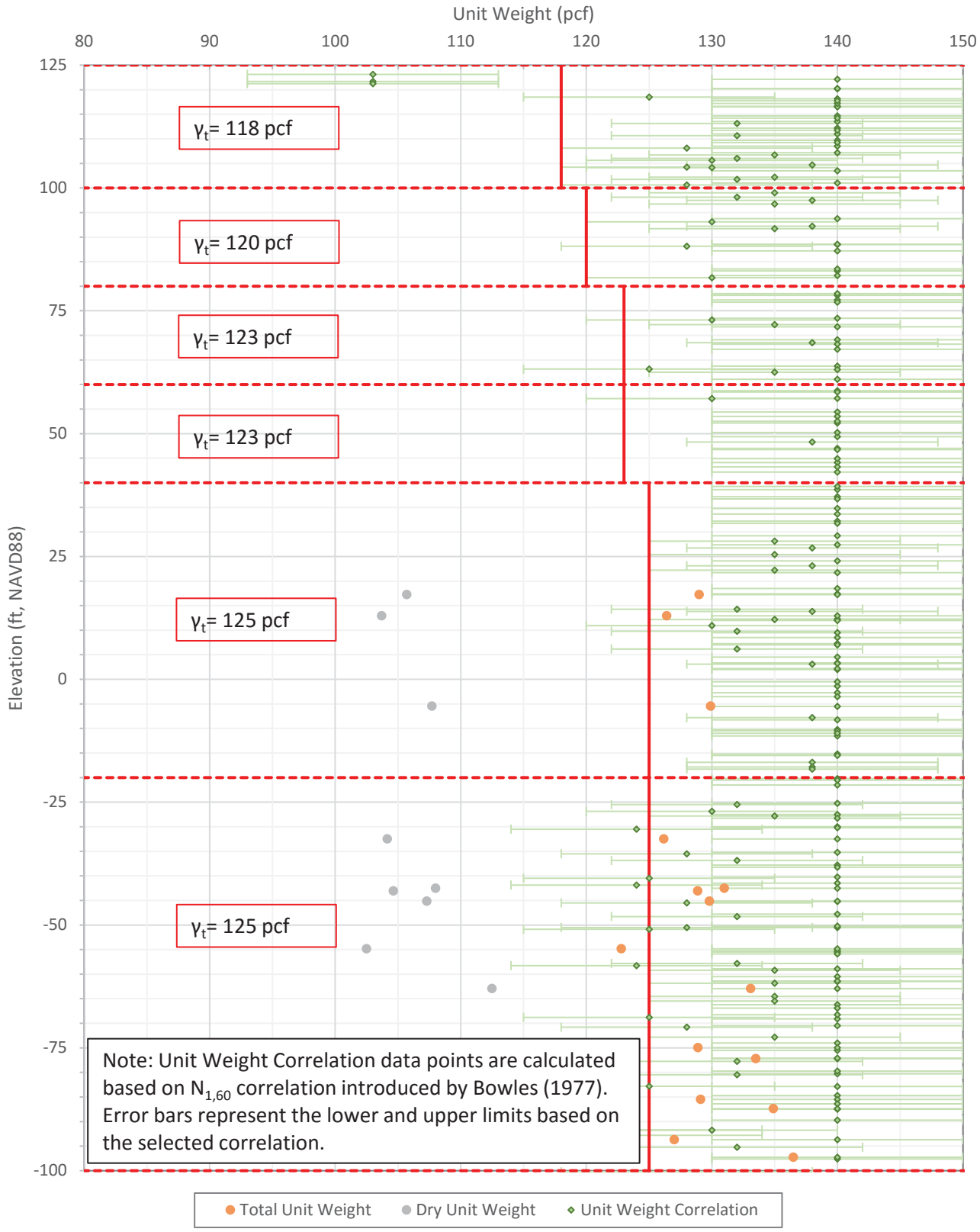


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

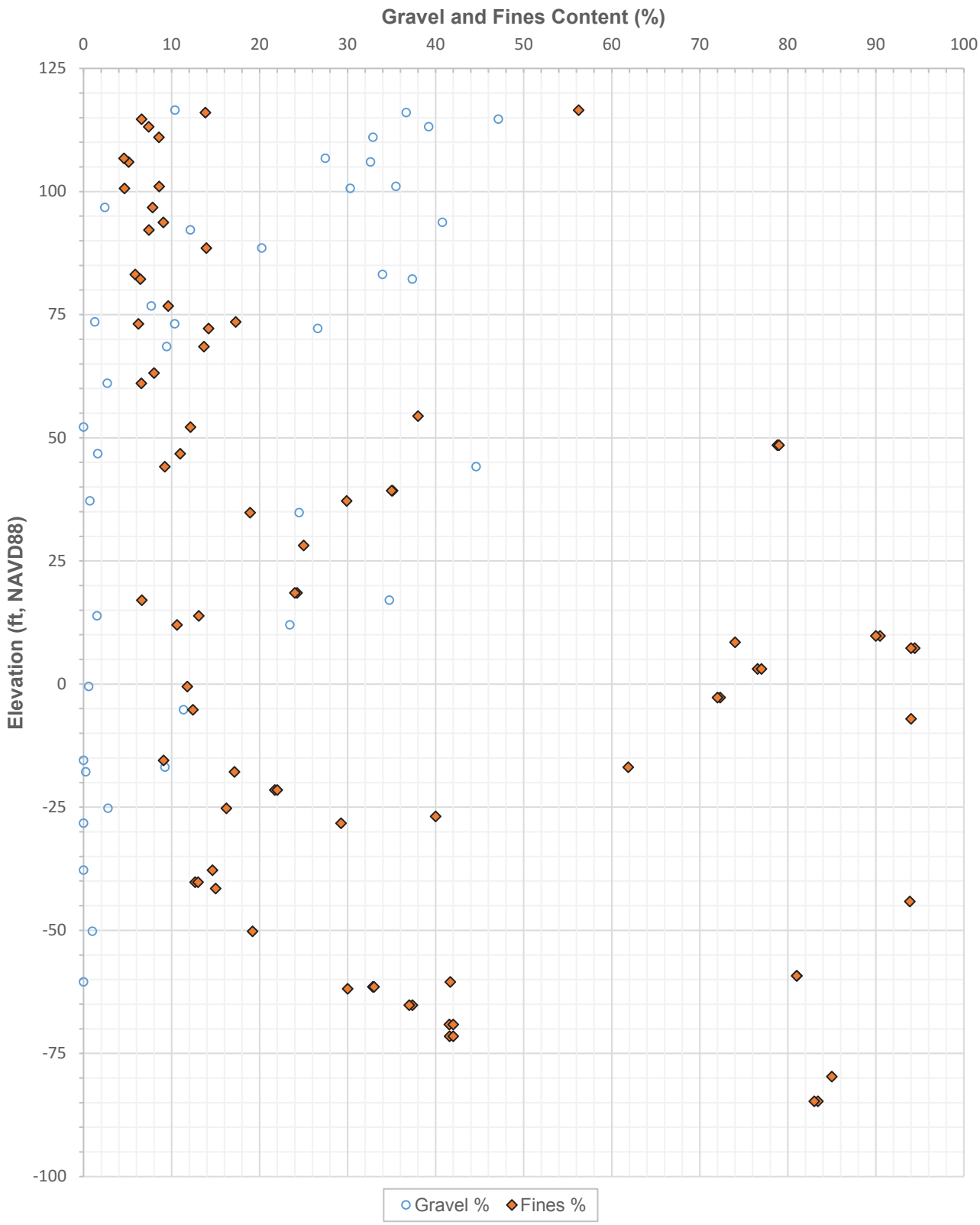


SPT $N_{1,60}$ - AREA 2A (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 26b

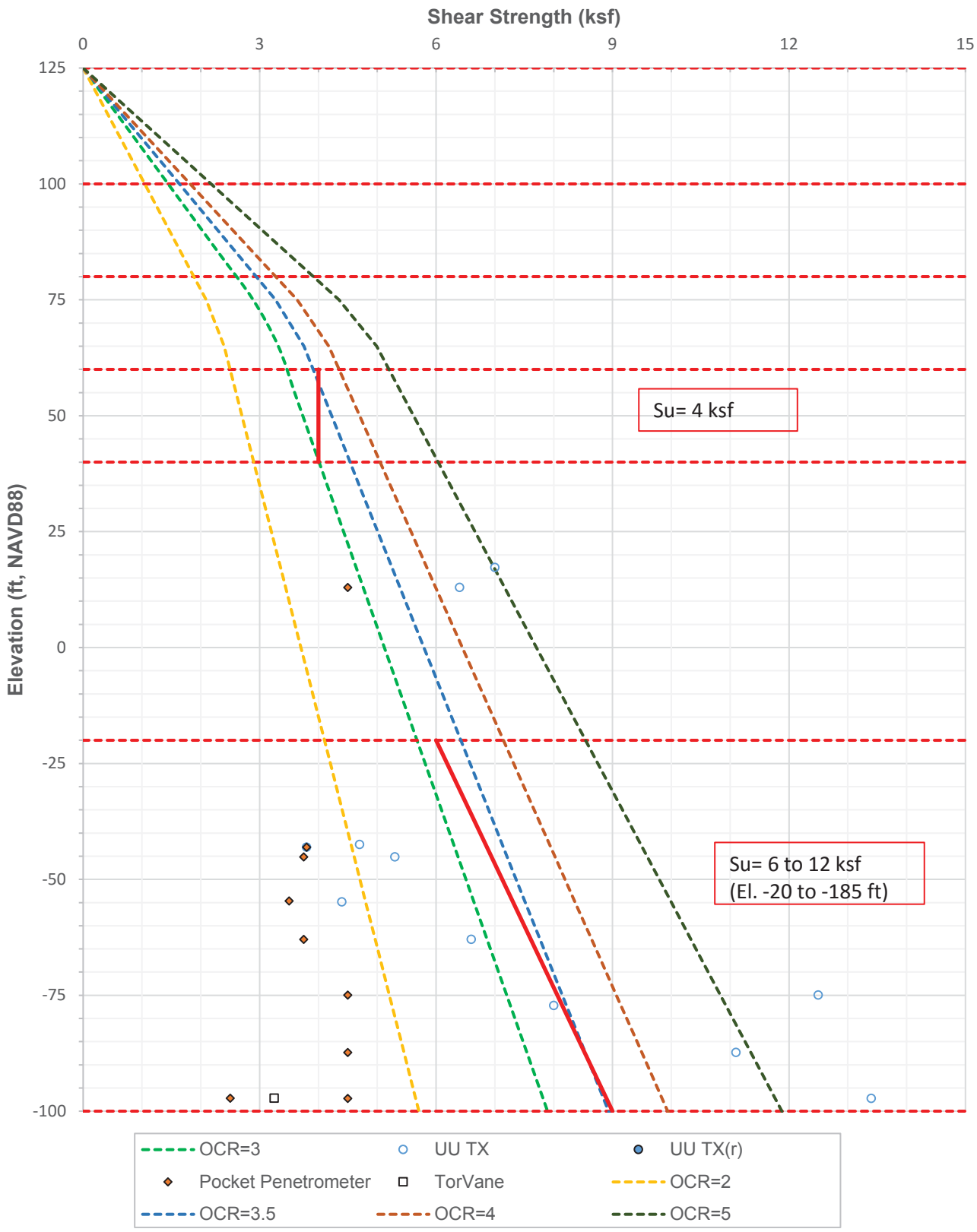


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

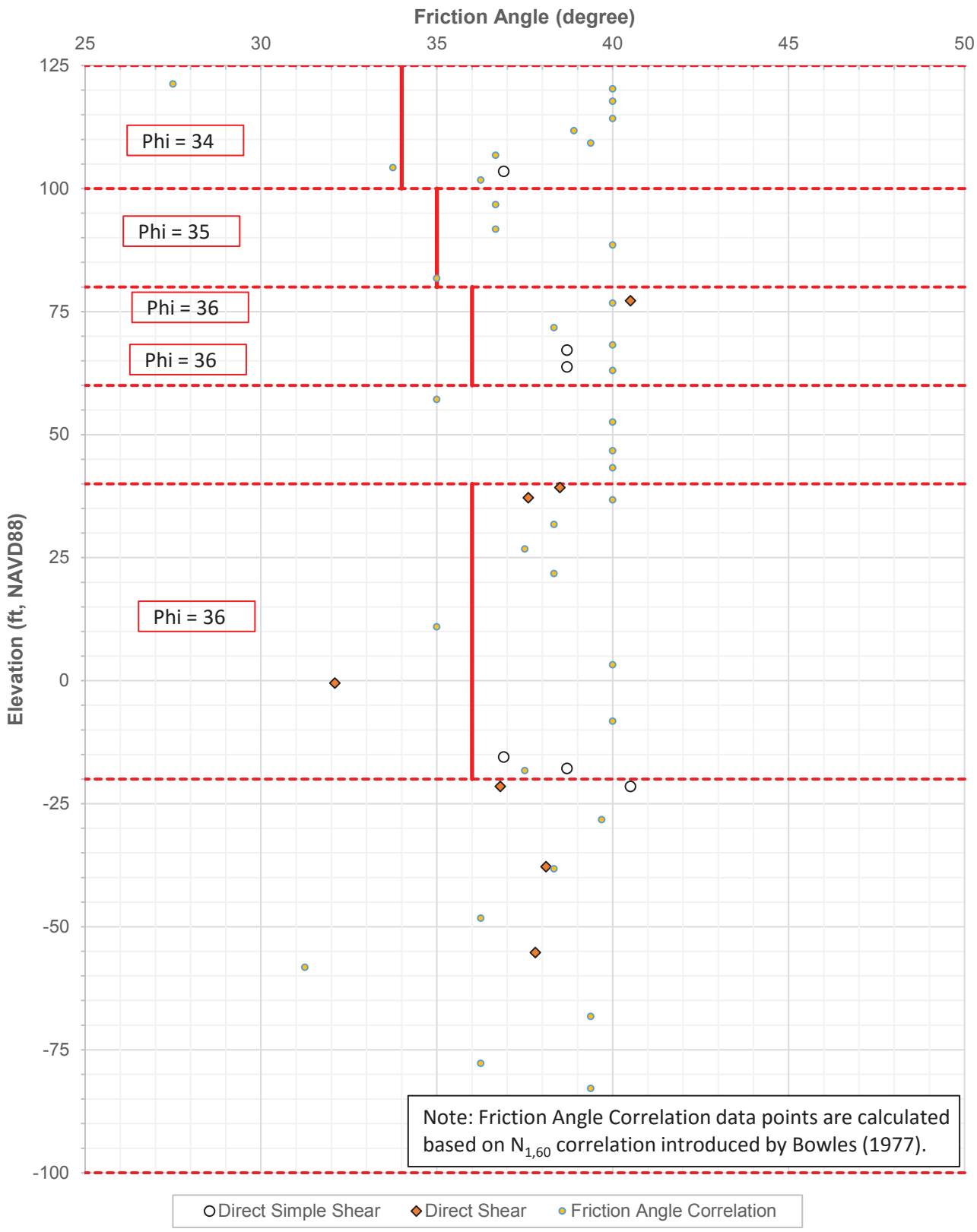


GRAVEL AND FINES CONTENT - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

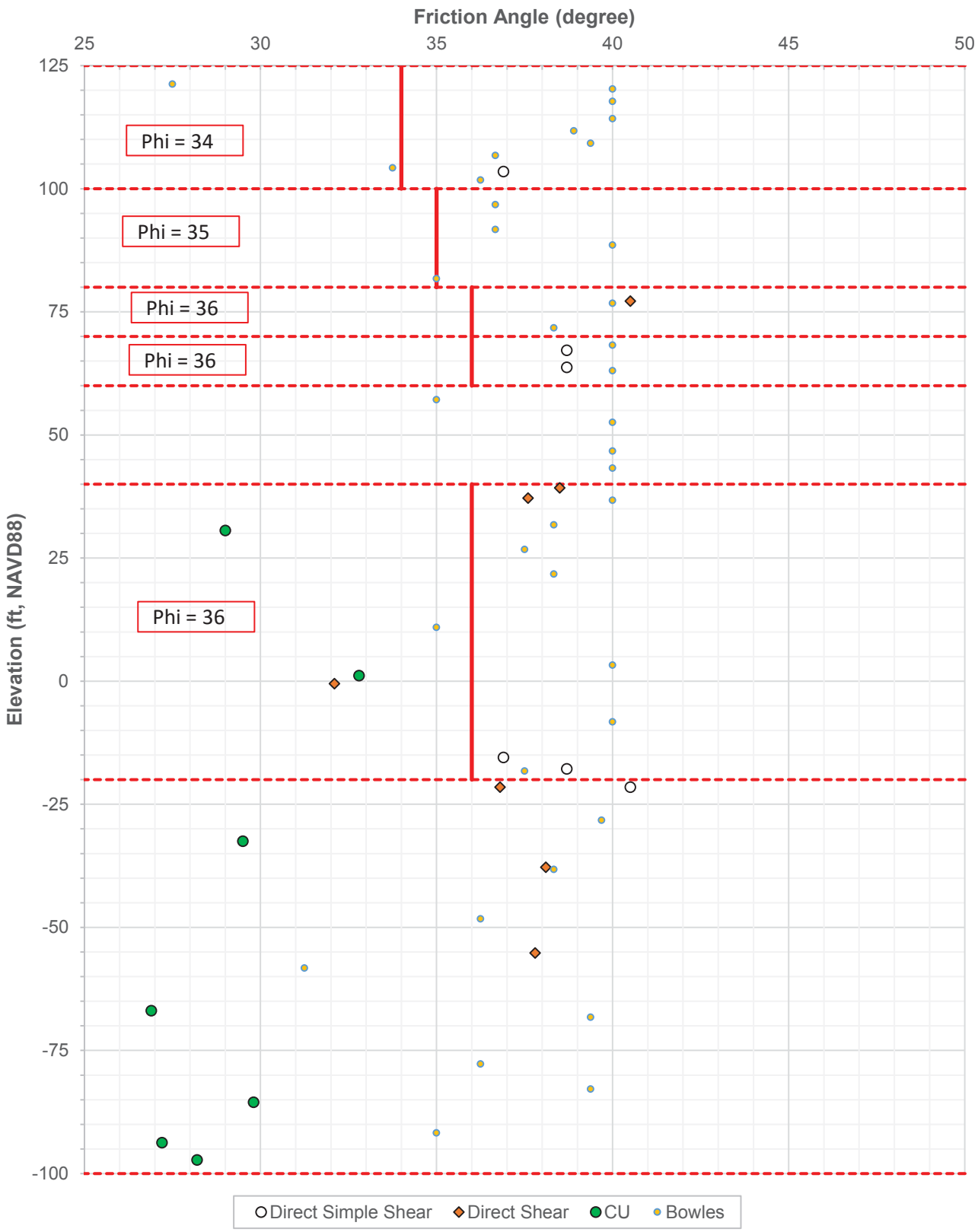
PLATE 26d



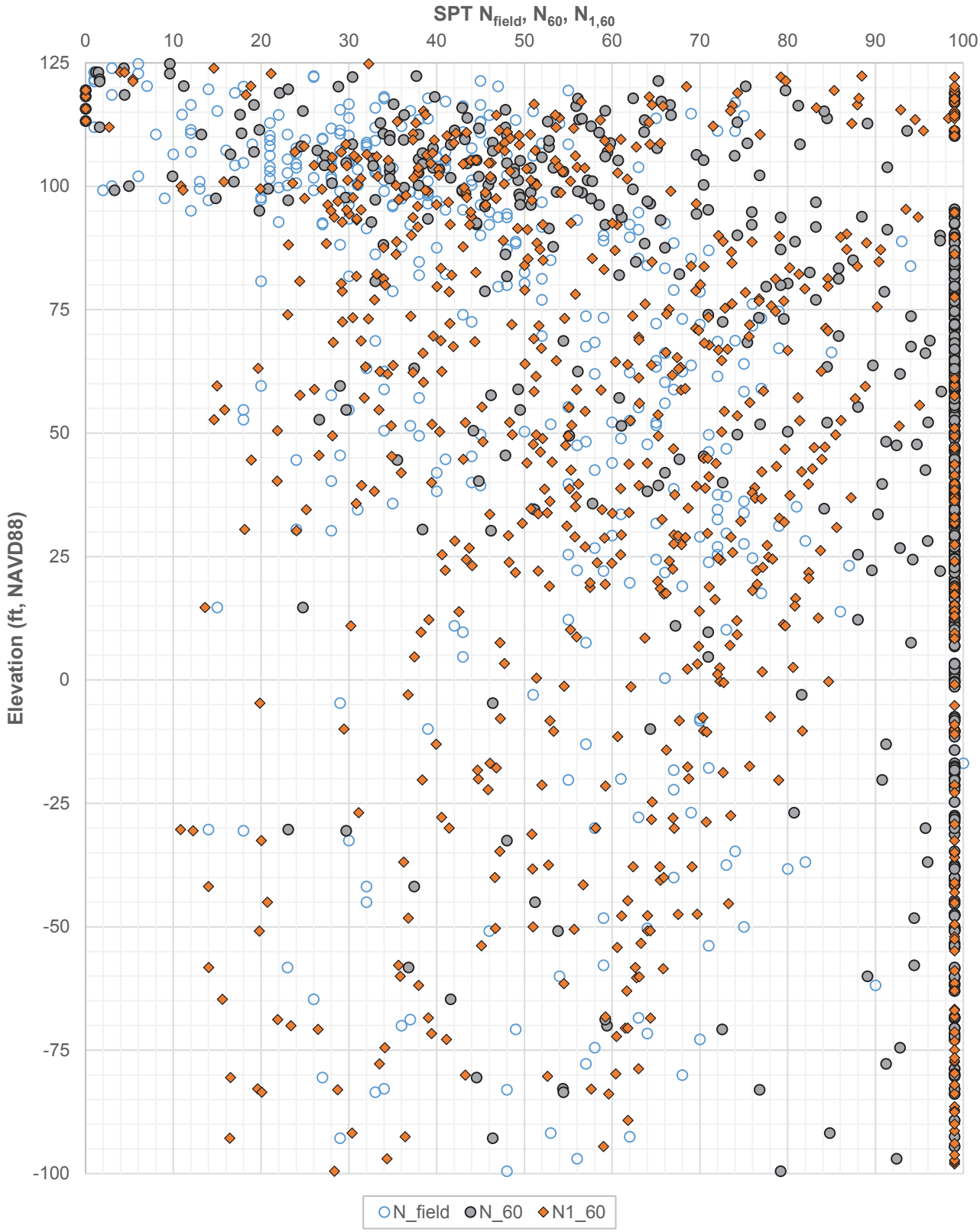
UNDRAINED SHEAR STRENGTH - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



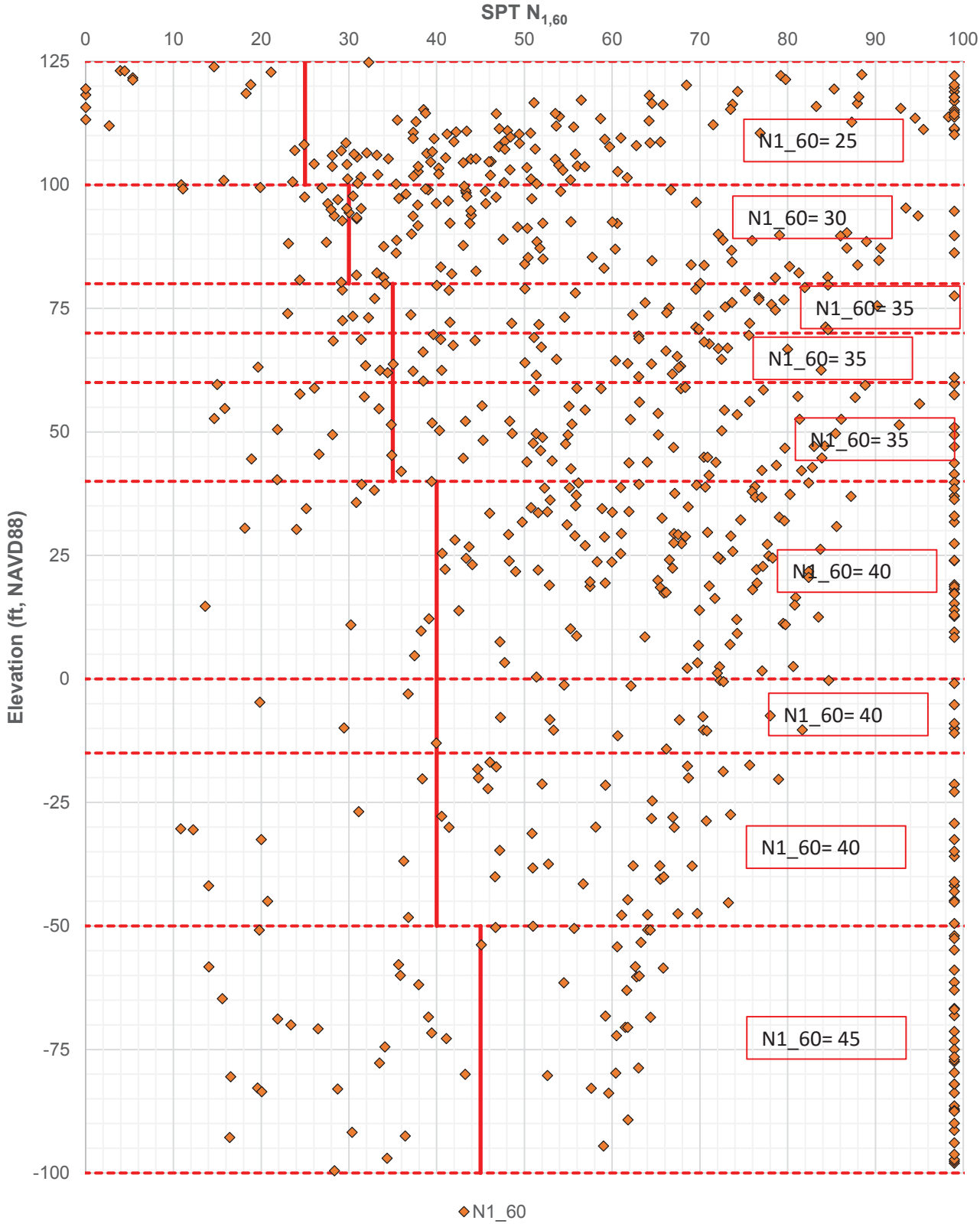
FRICTION ANGLES - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



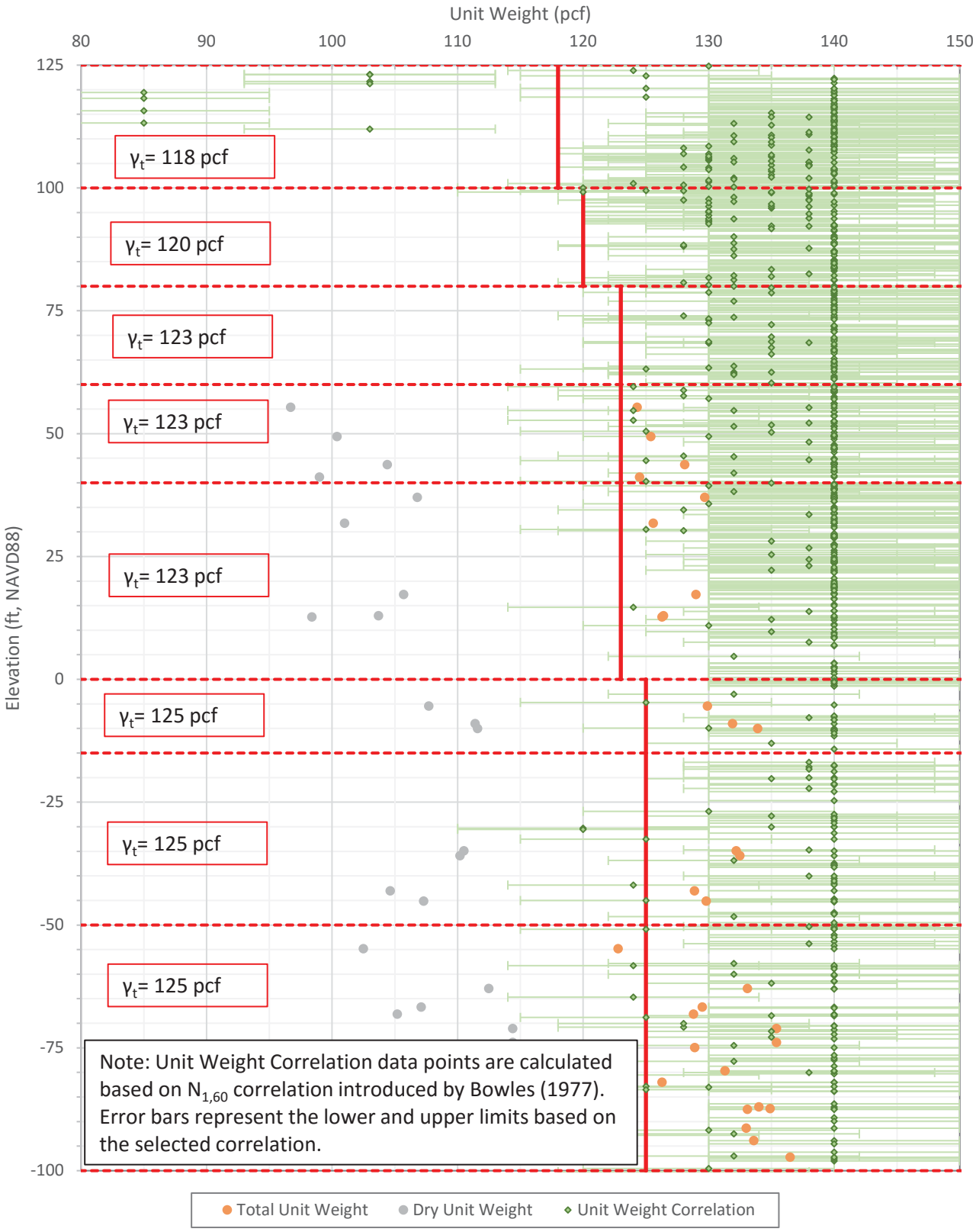
FRICTION ANGLES - AREA 2A-S (DP2A-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



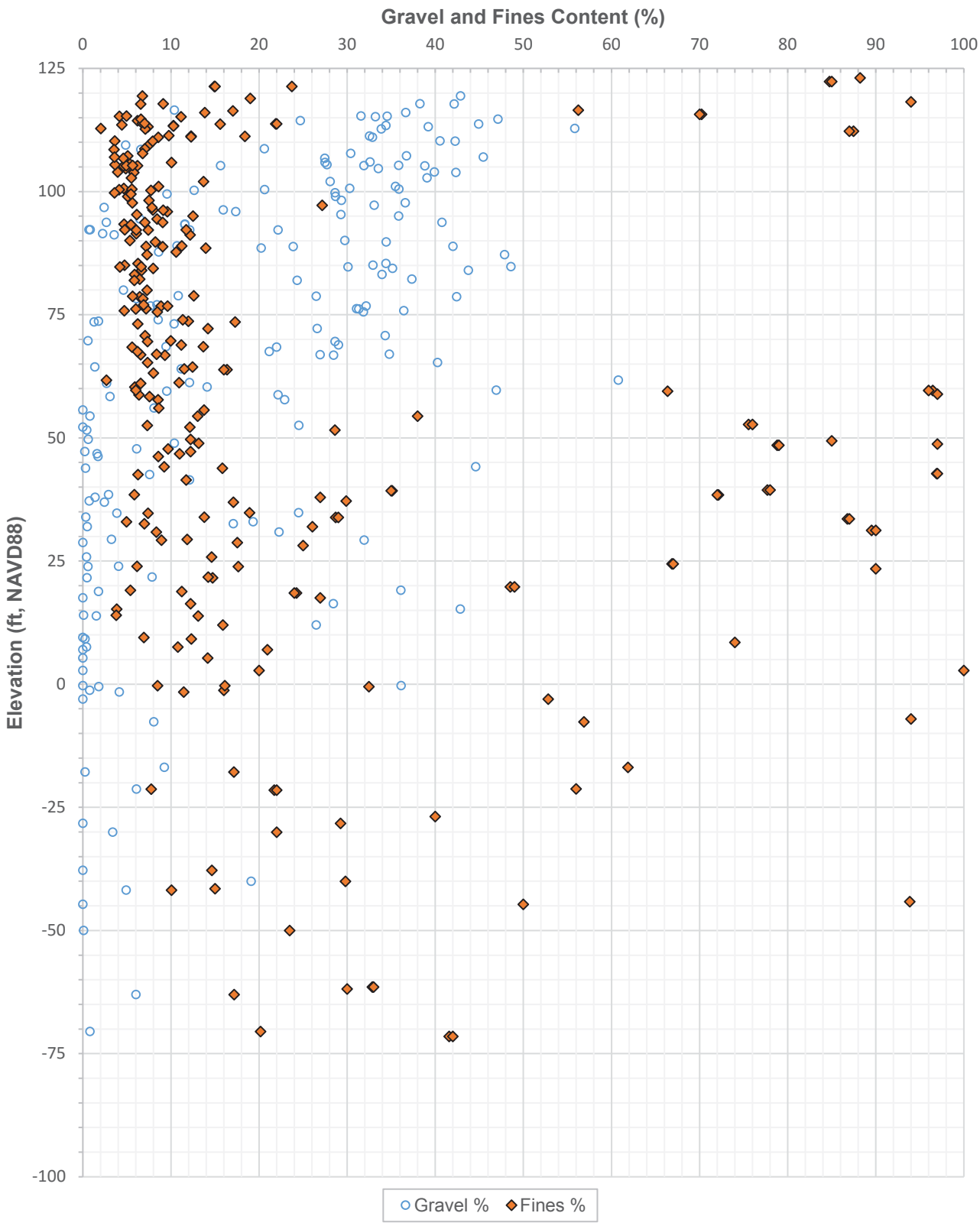
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT $N_{1,60}$ - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

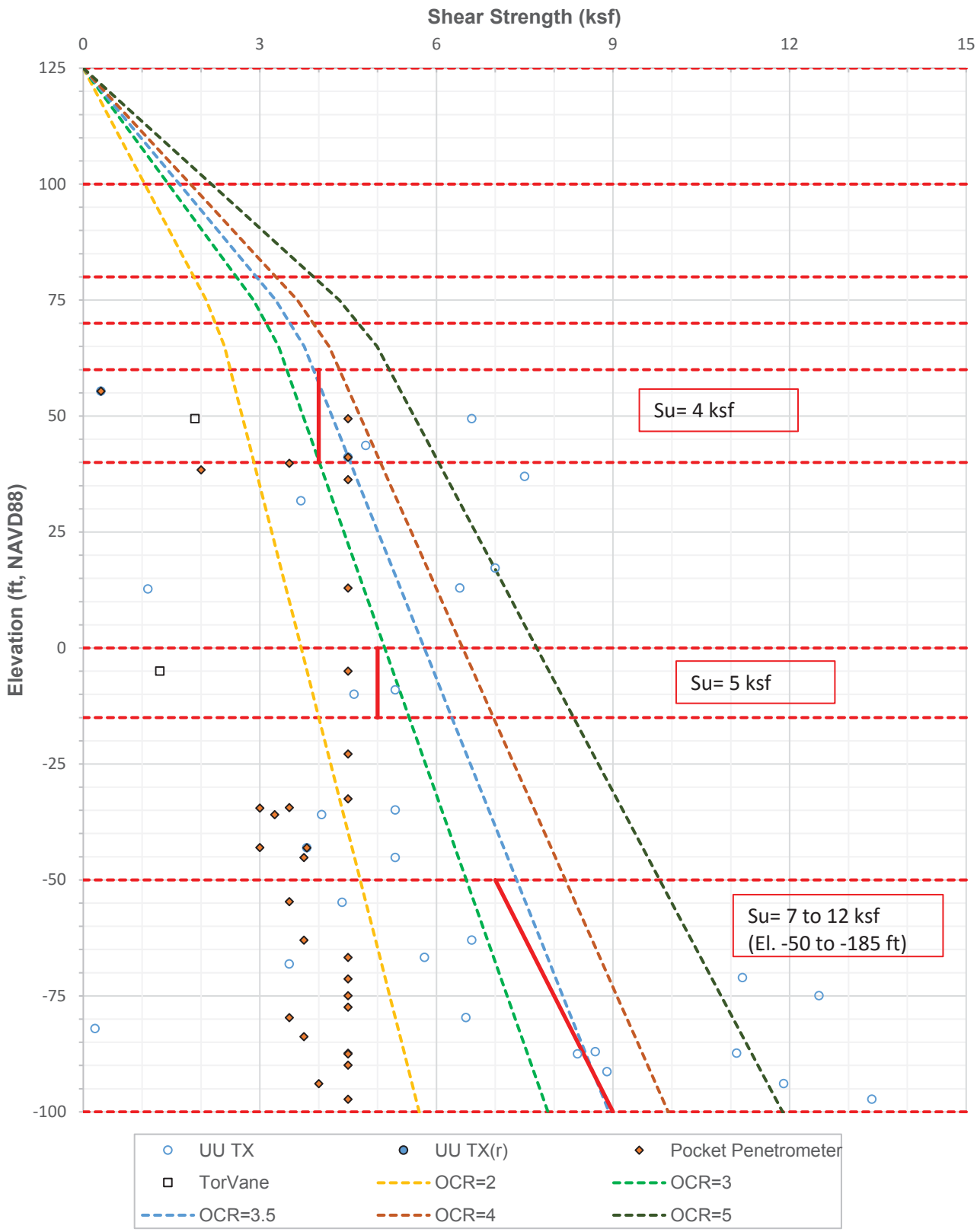


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

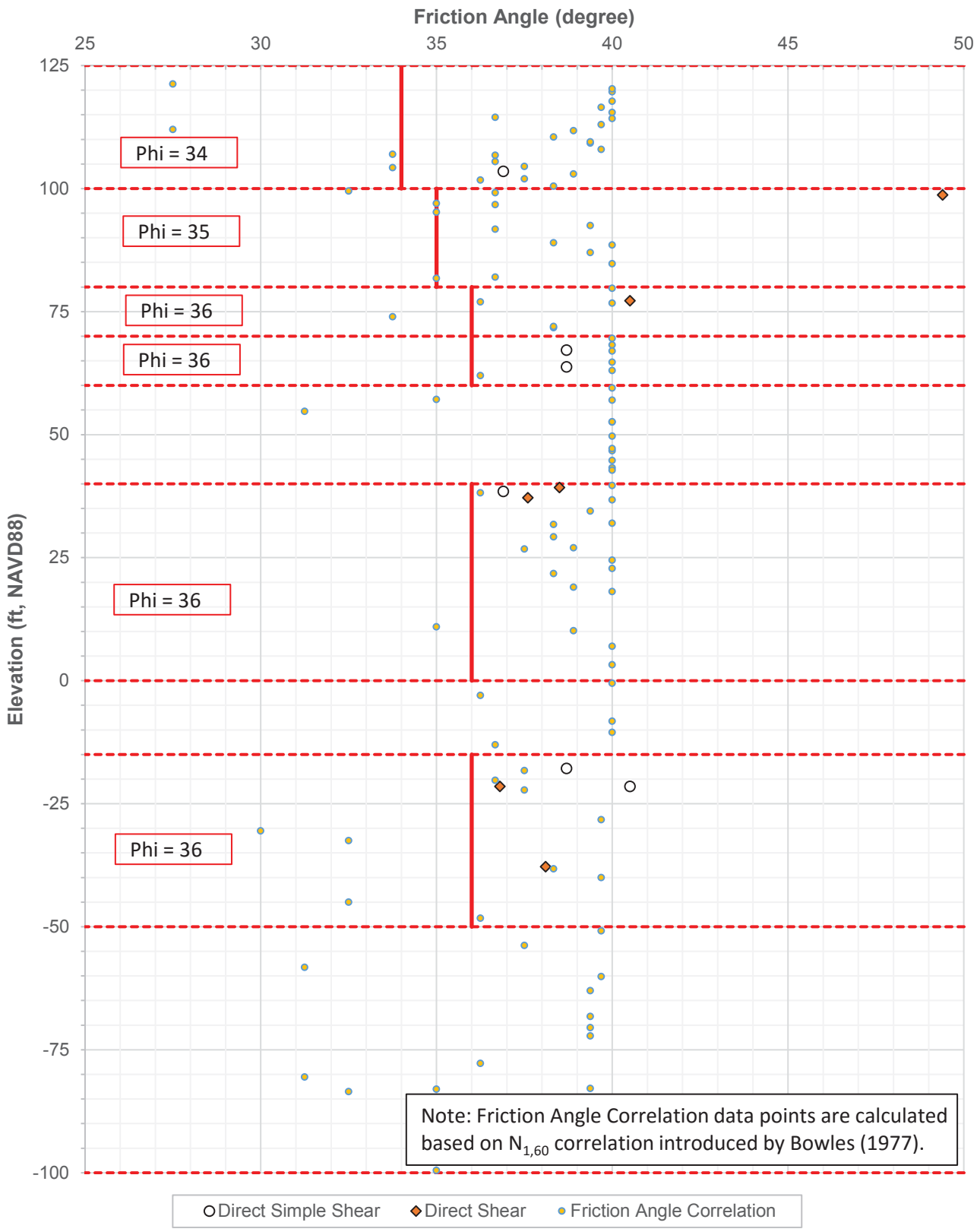


GRAVEL AND FINES CONTENT - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

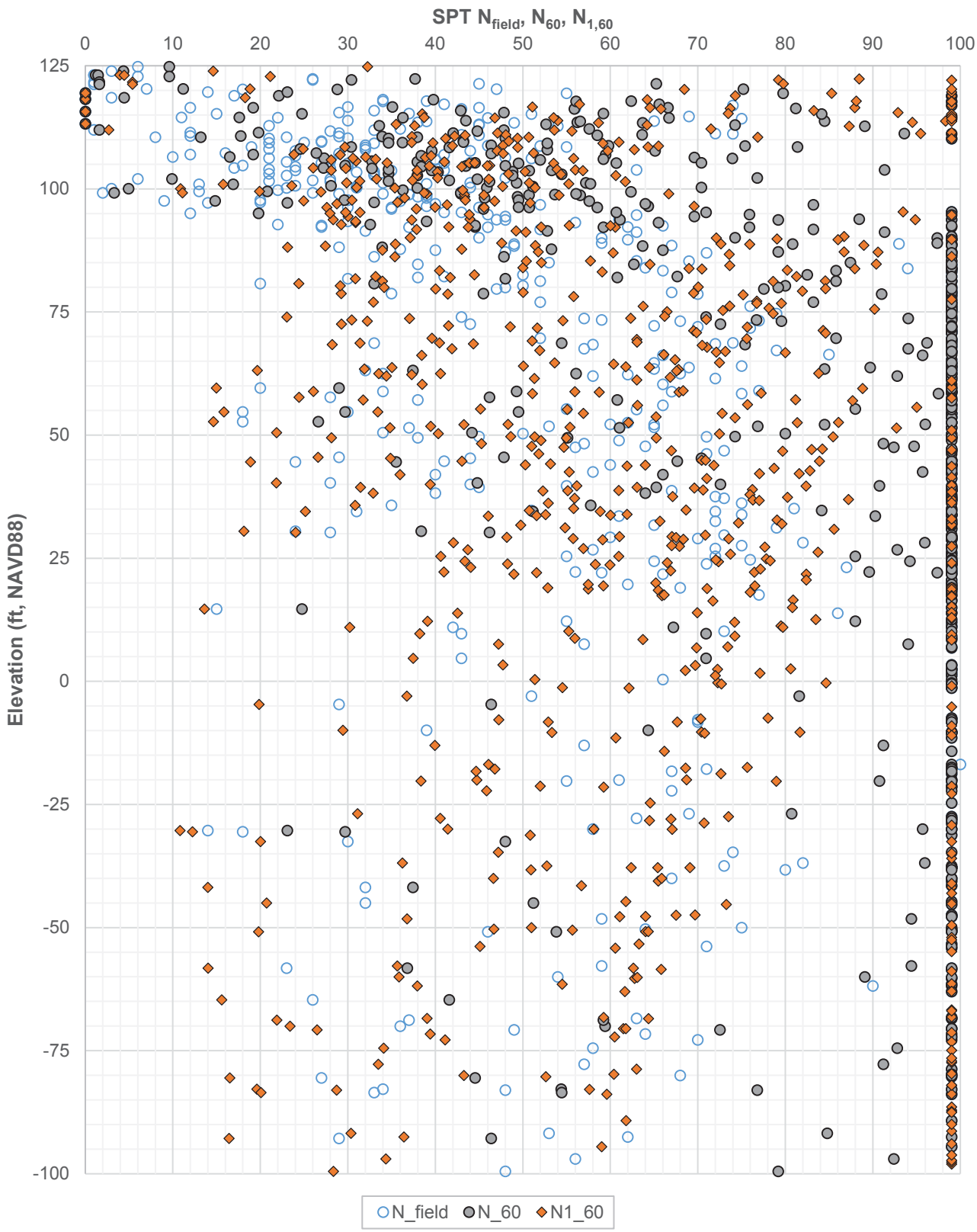
PLATE 27d



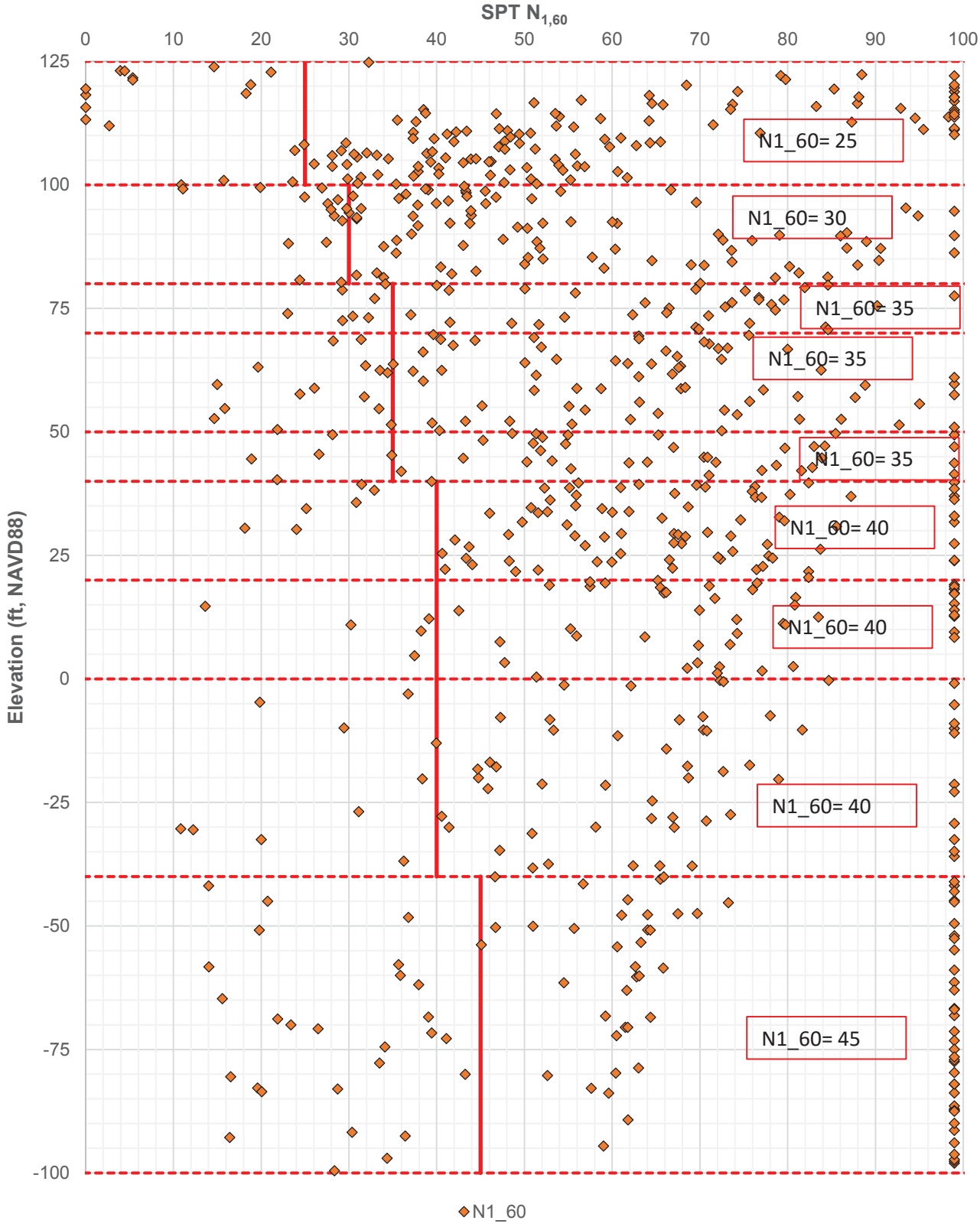
UNDRAINED SHEAR STRENGTH - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



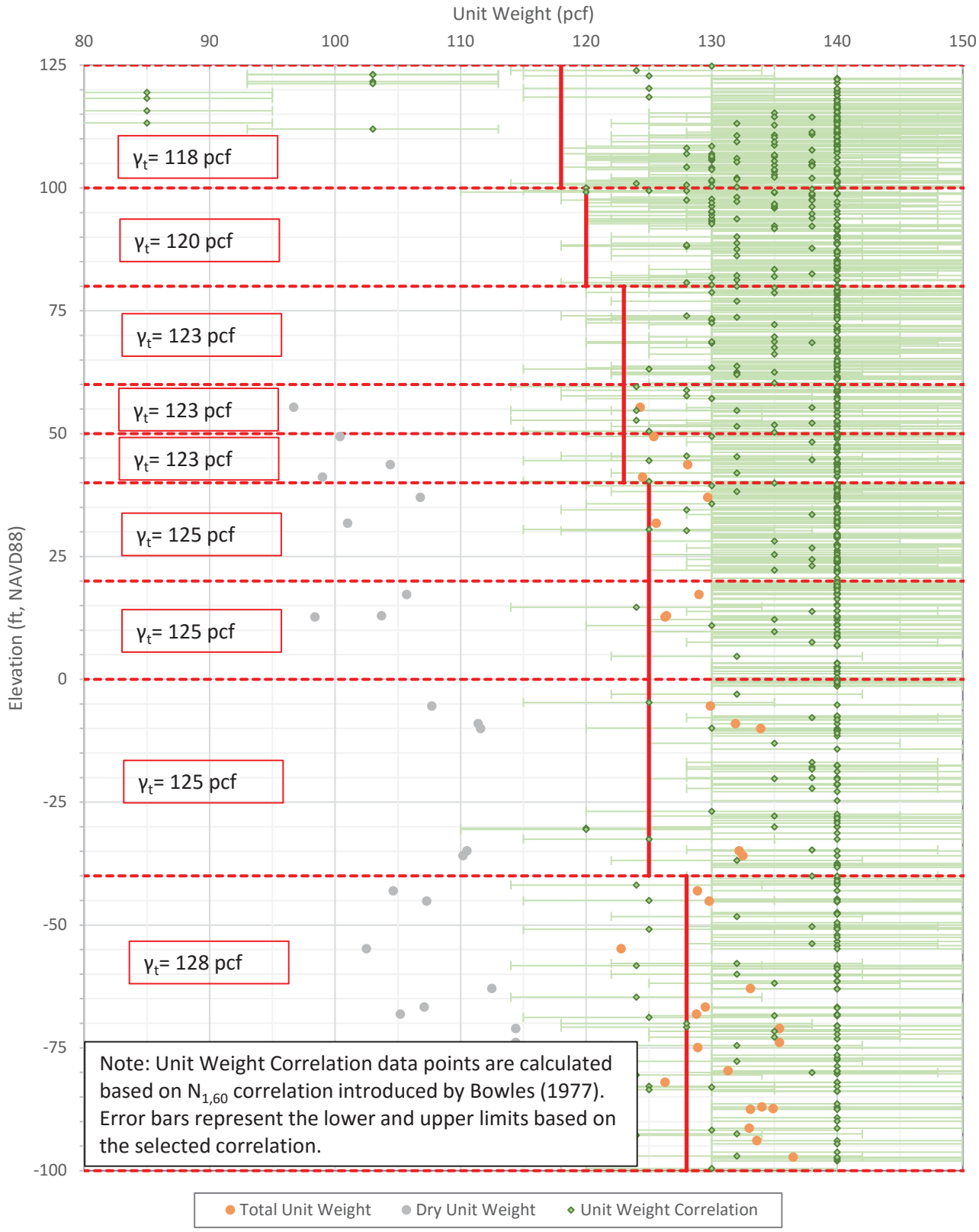
FRICTION ANGLES - AREA 2B (DP2B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



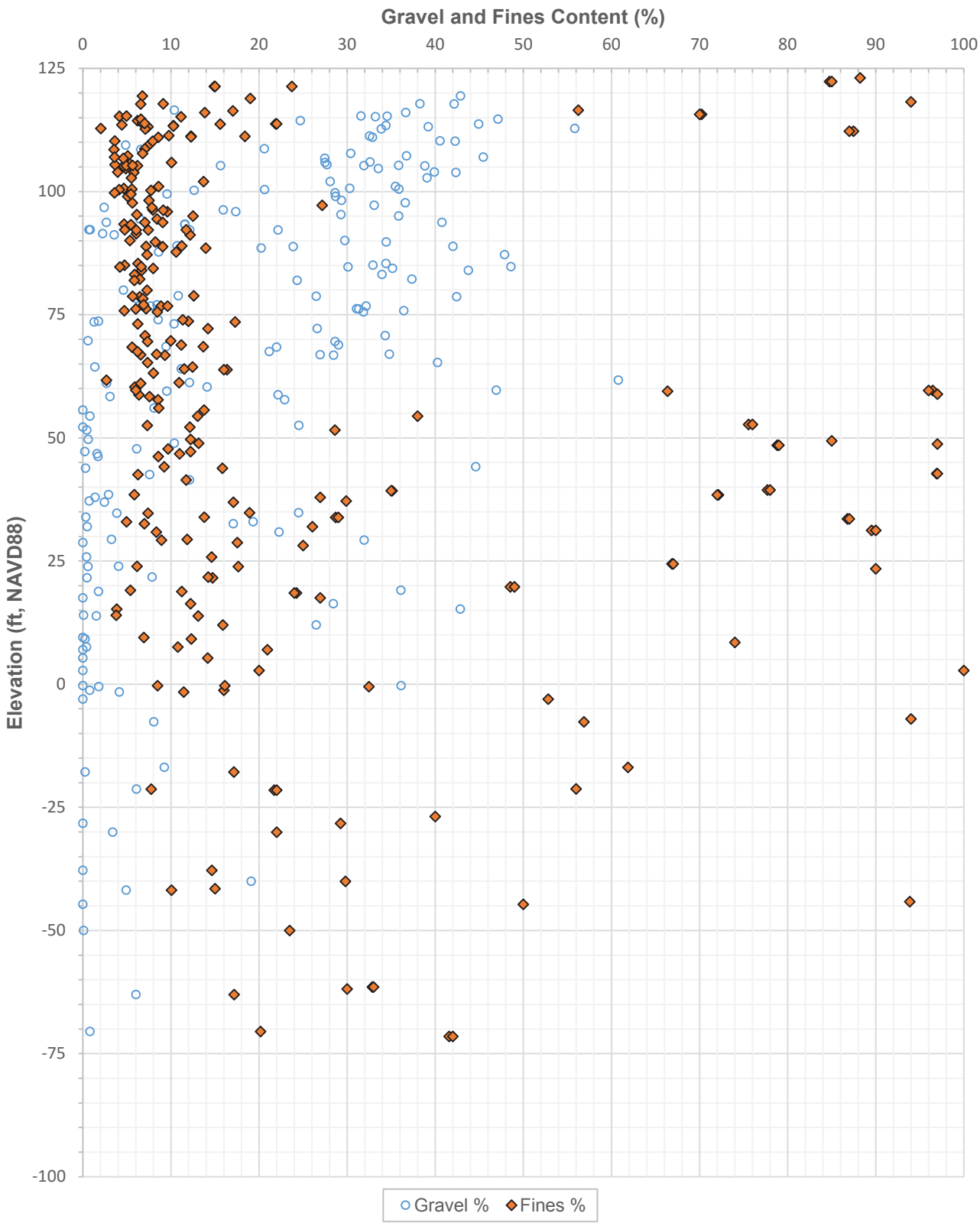
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT $N_{1,60}$ - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

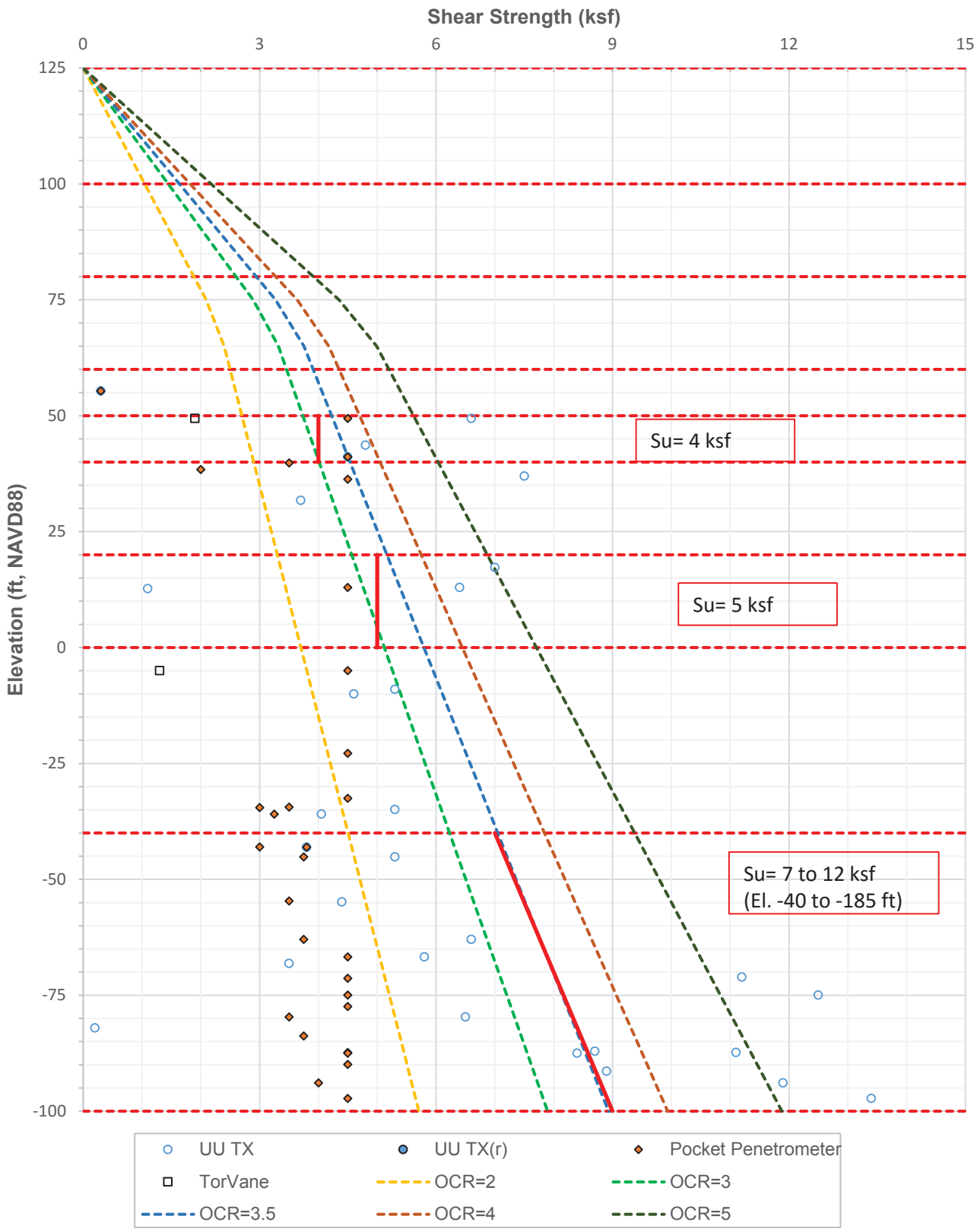


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

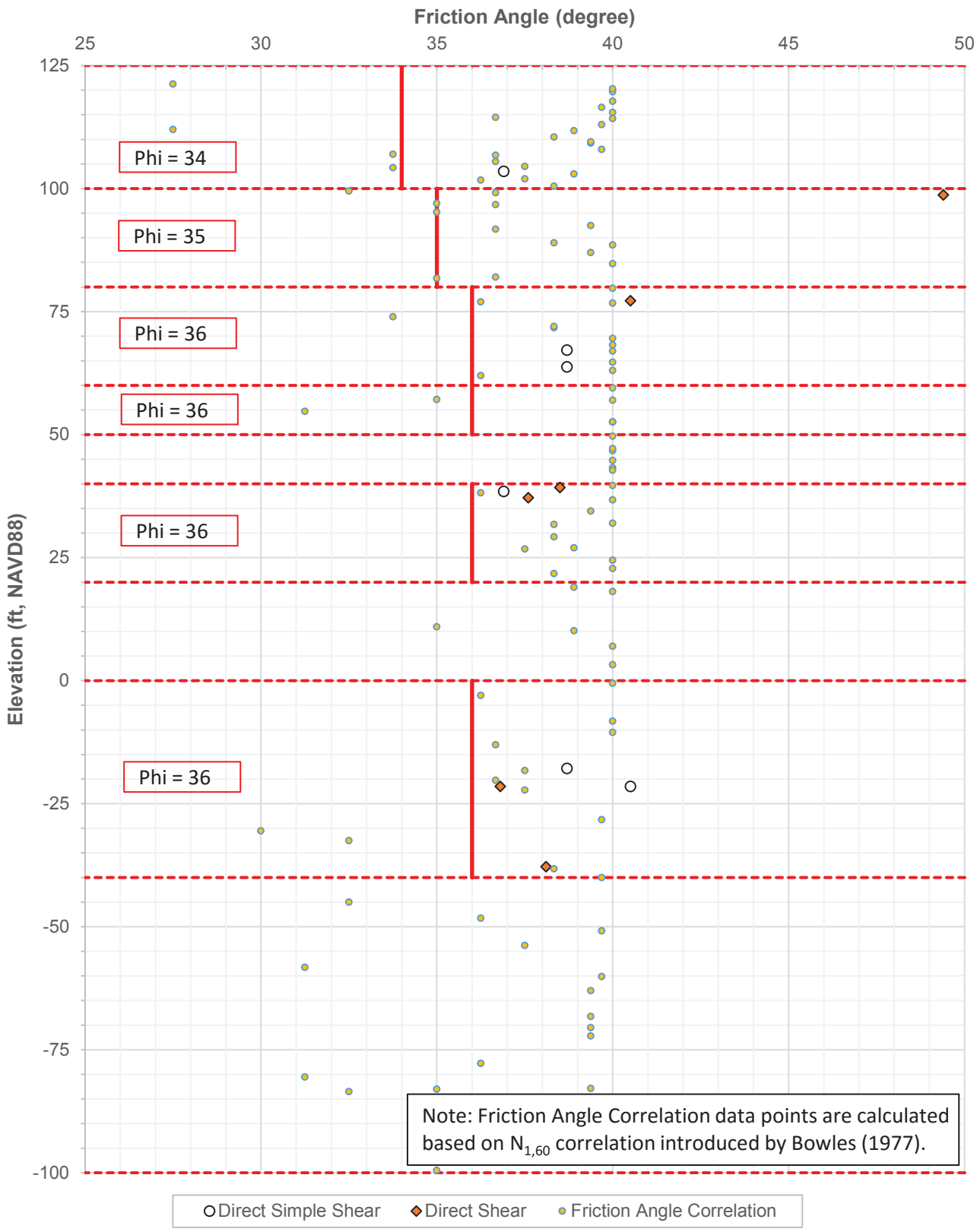


GRAVEL AND FINES CONTENT - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 28d

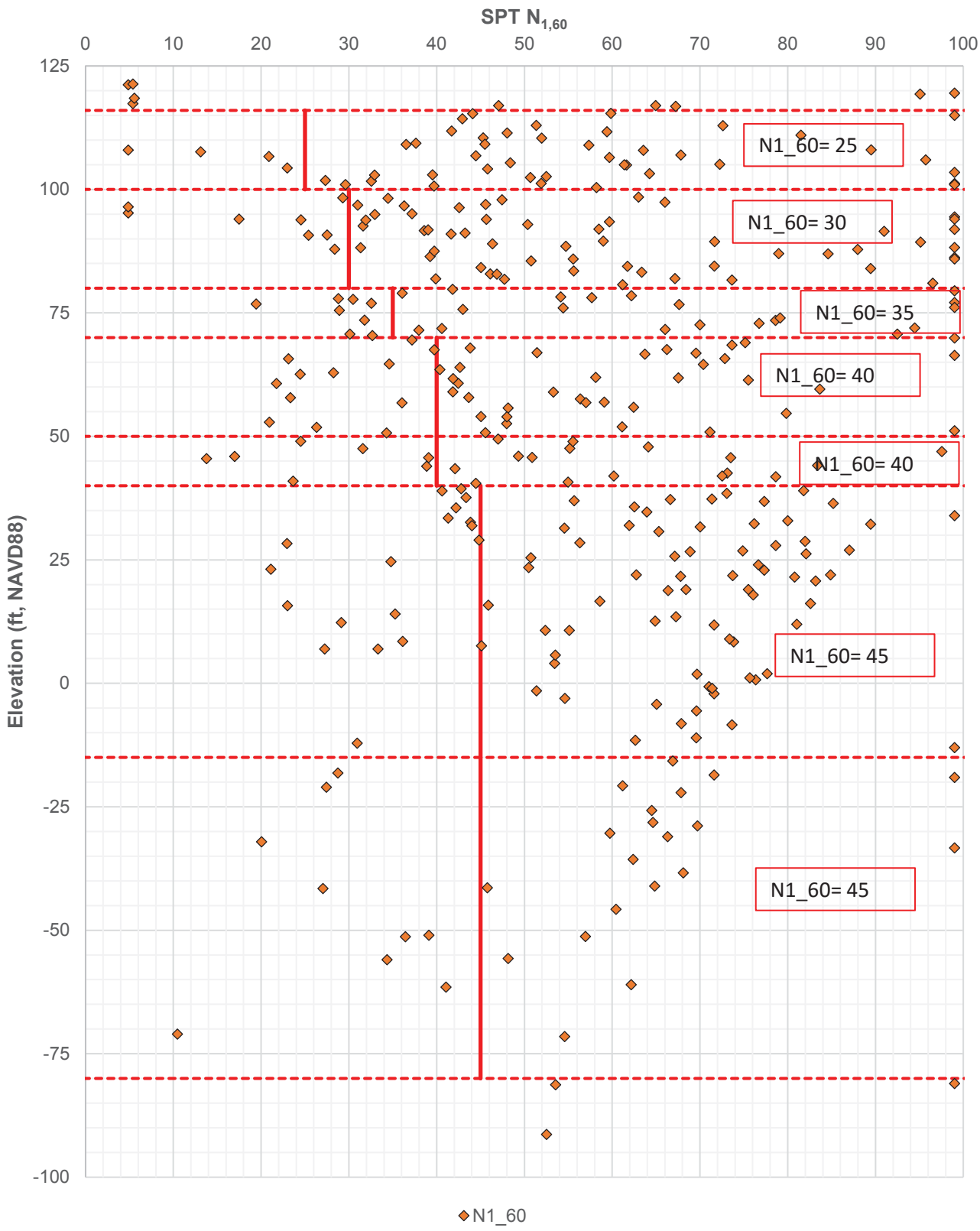


UNDRAINED SHEAR STRENGTH - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

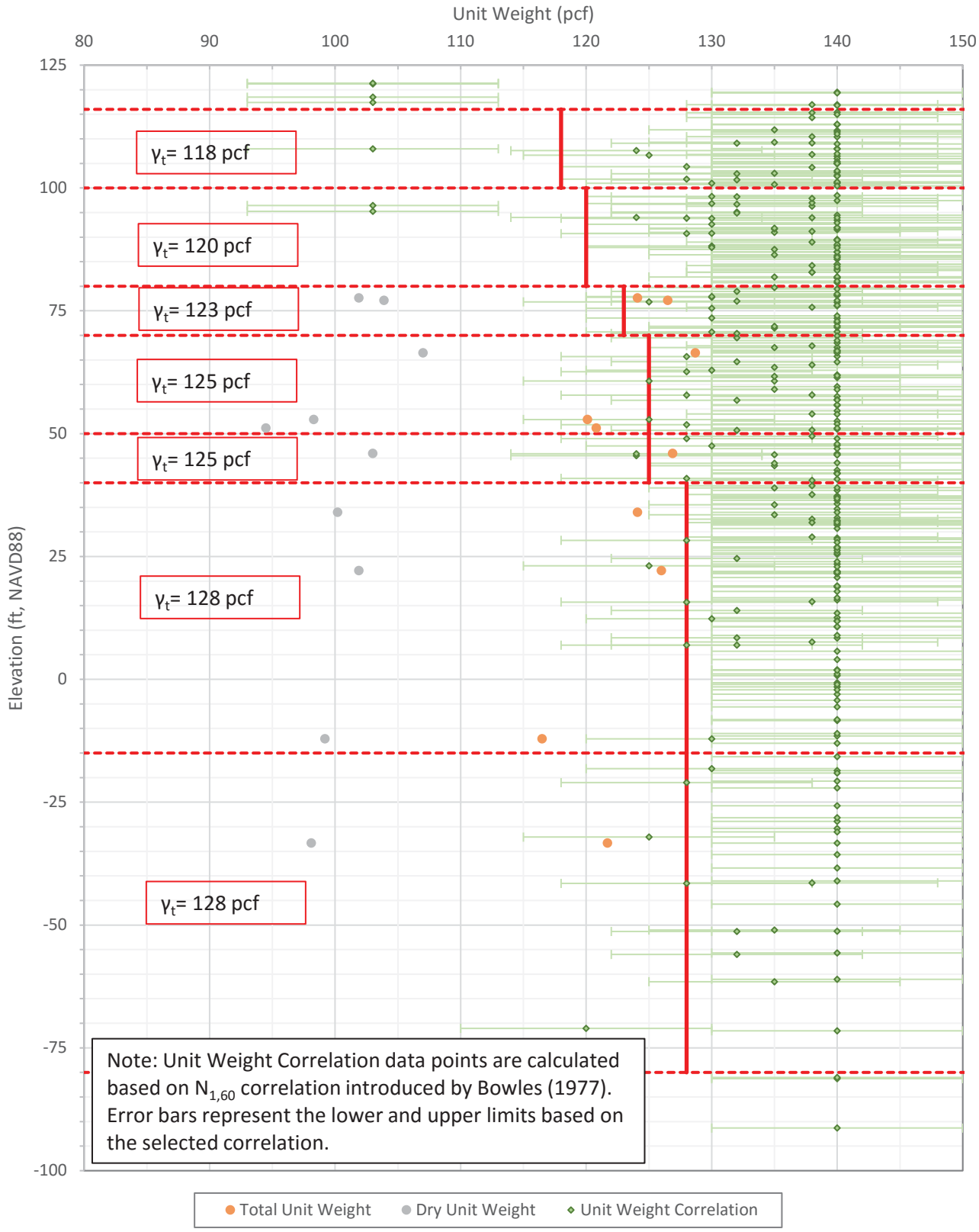


FRICTION ANGLES - AREA 2C (DP2C)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

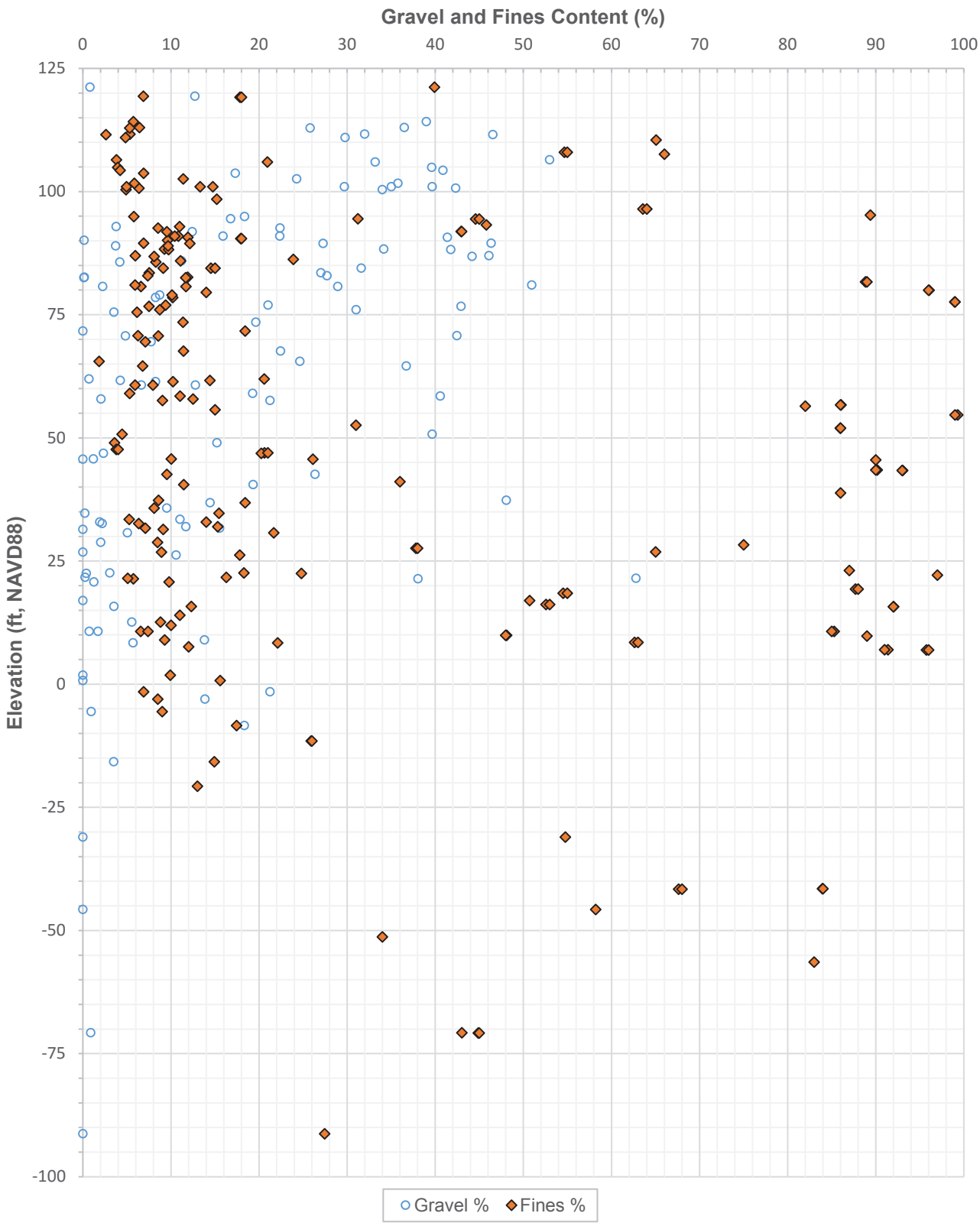
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT $N_{1,60}$ - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

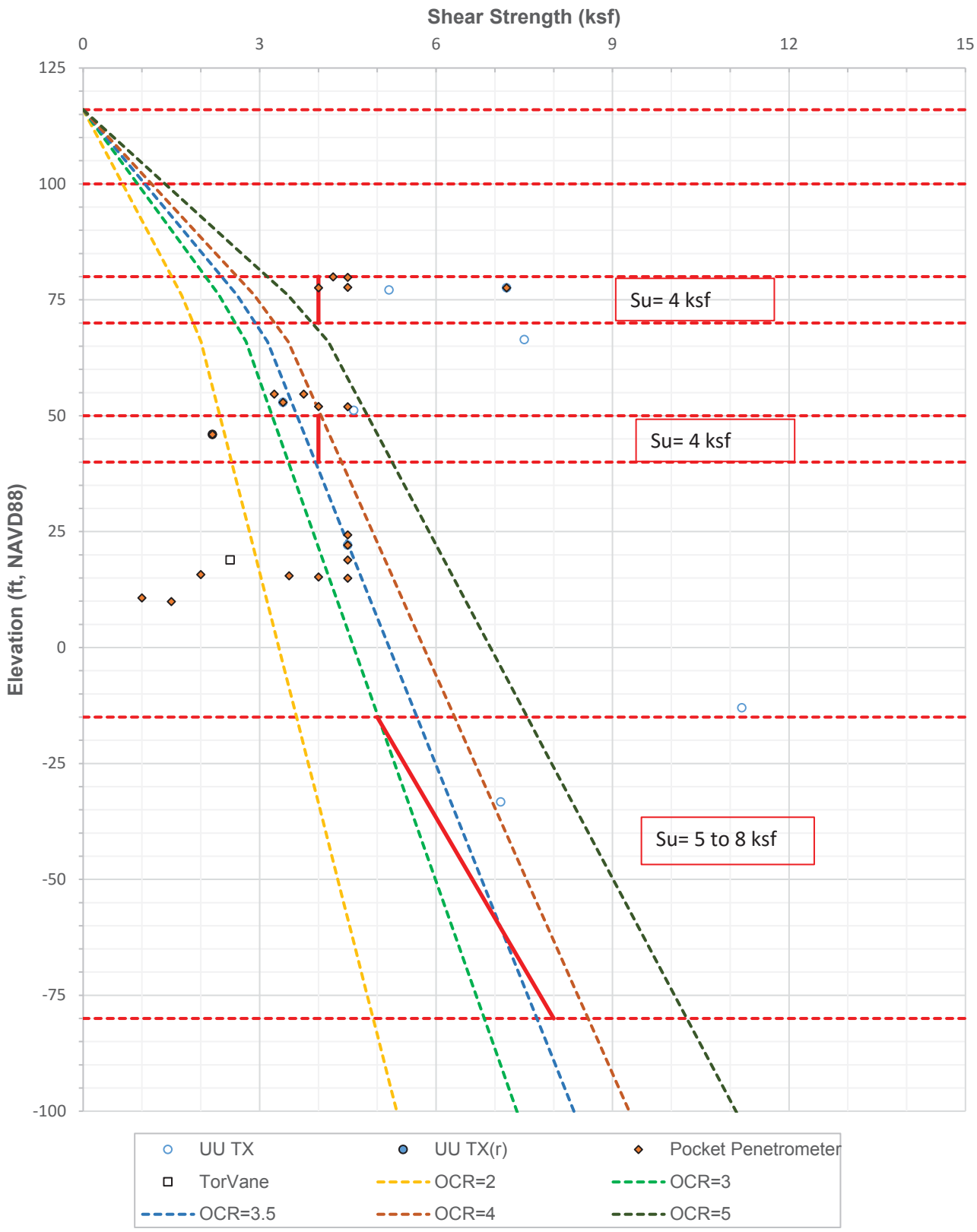


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

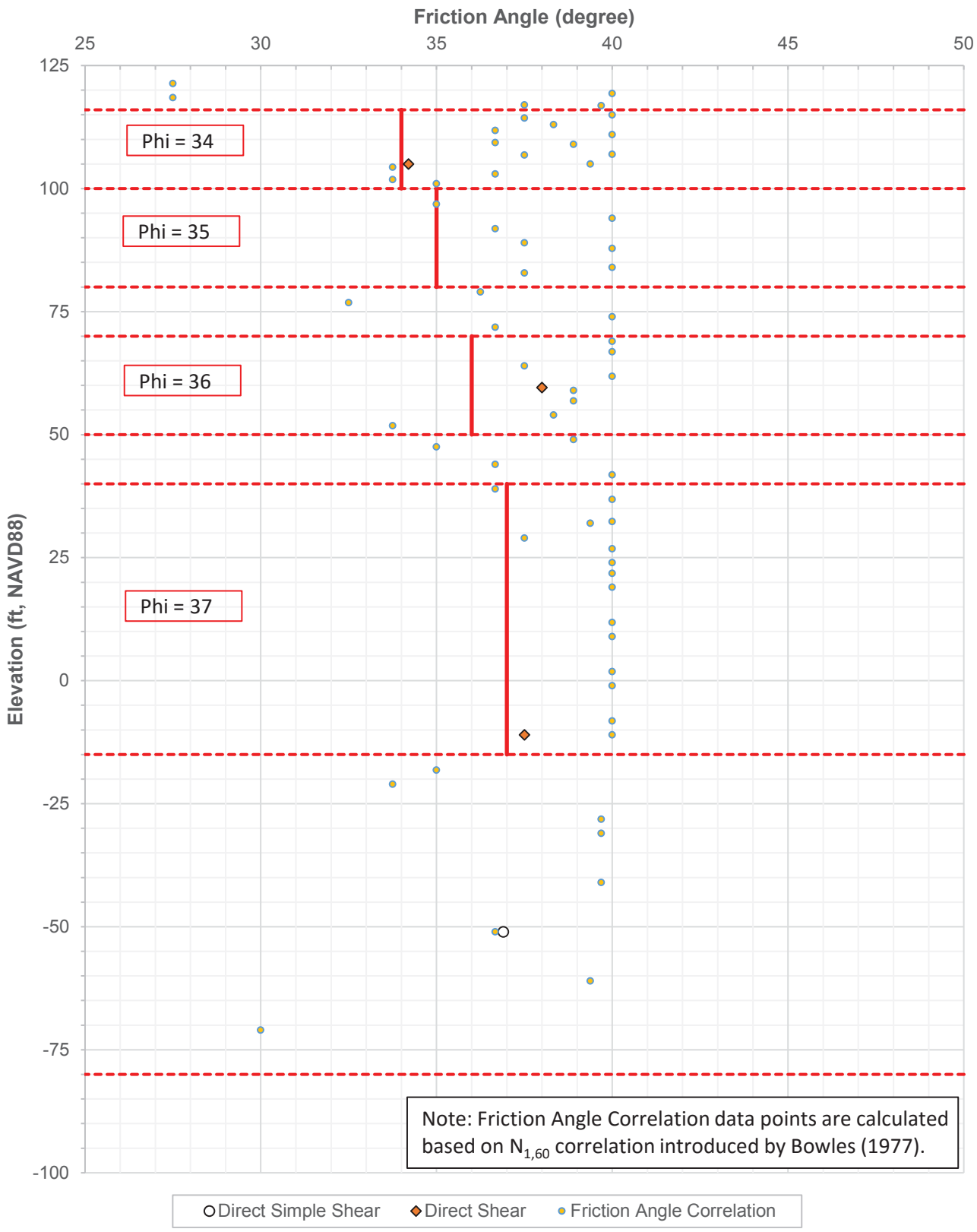


GRAVEL AND FINES CONTENT - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

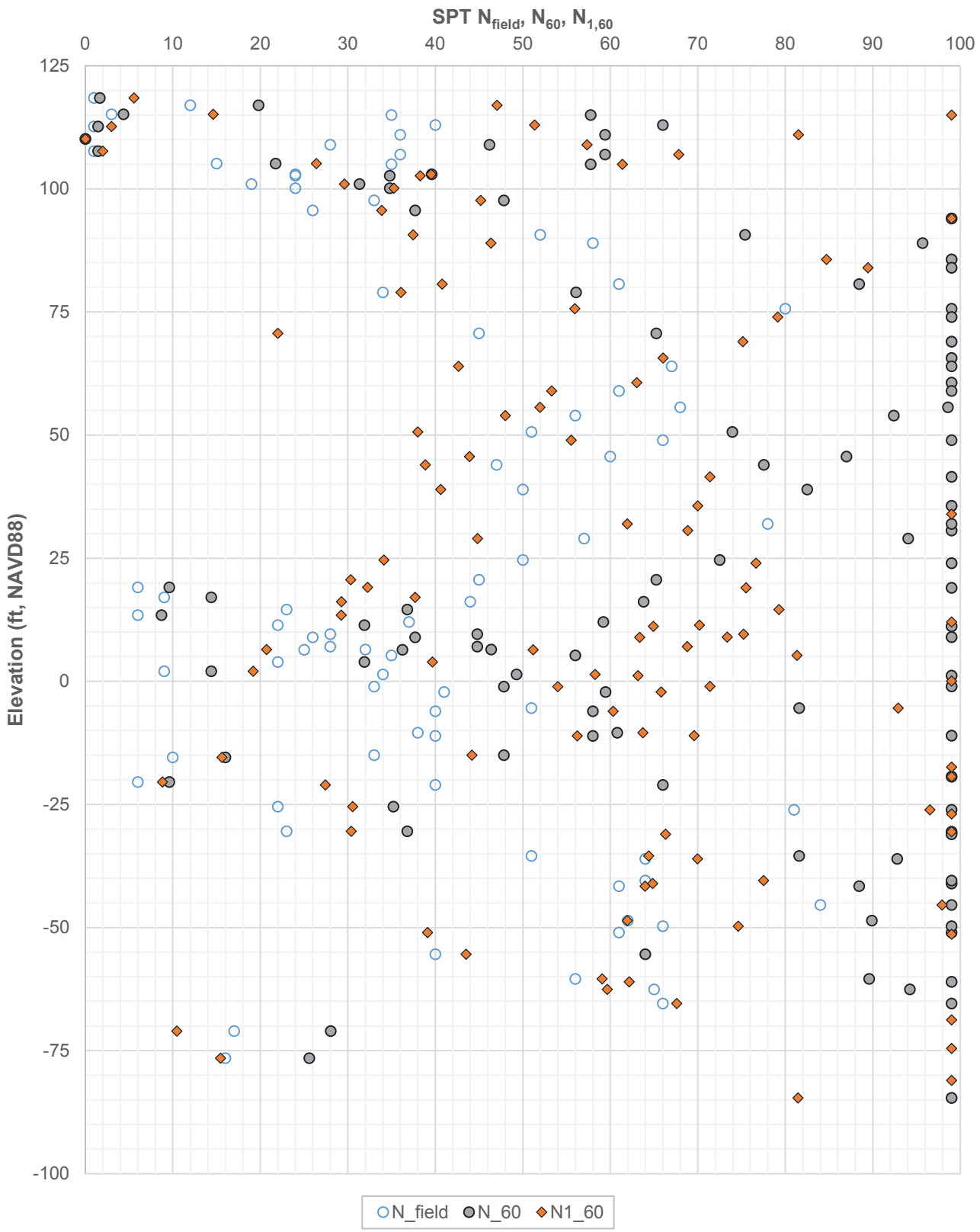
PLATE 29d



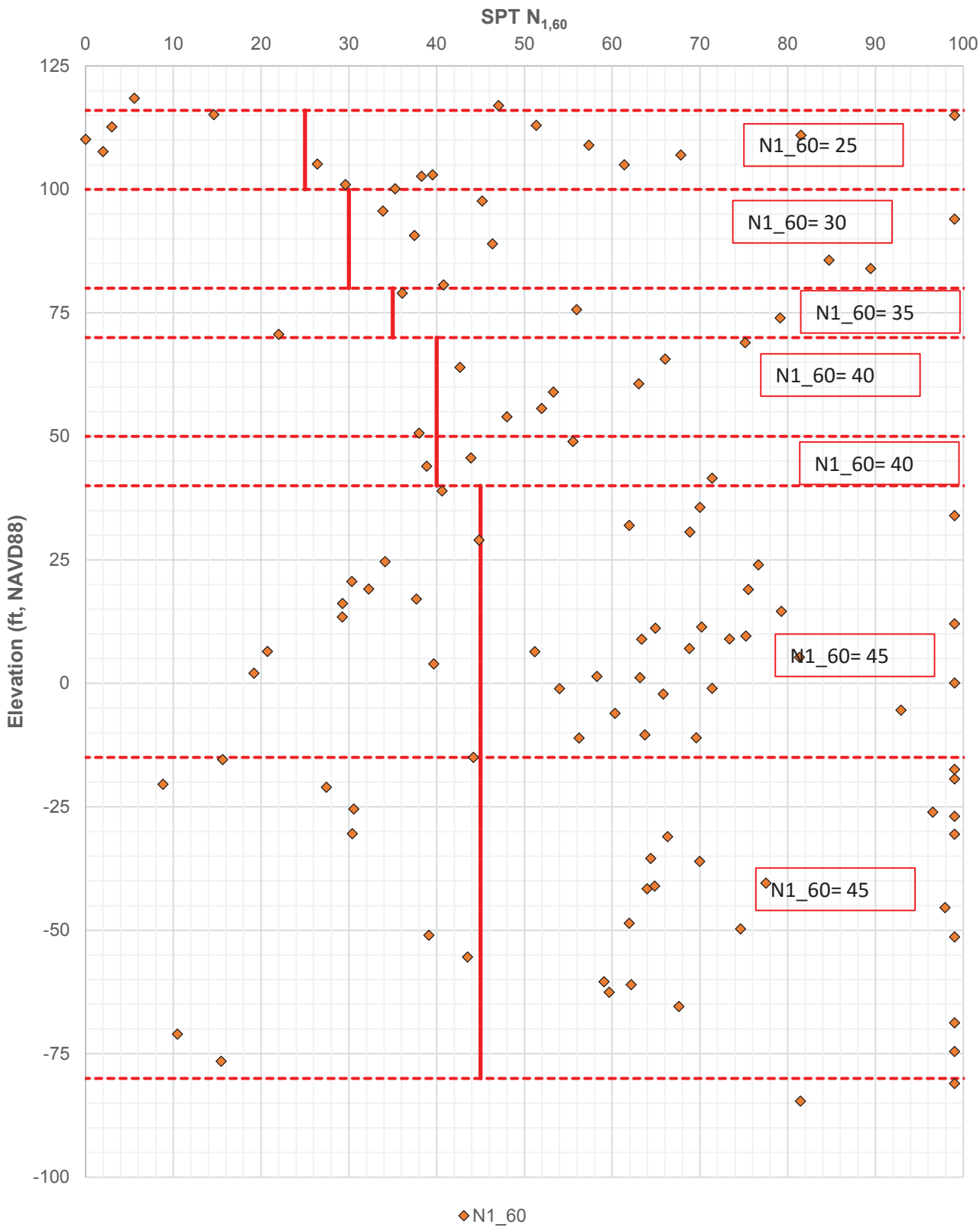
UNDRAINED SHEAR STRENGTH - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



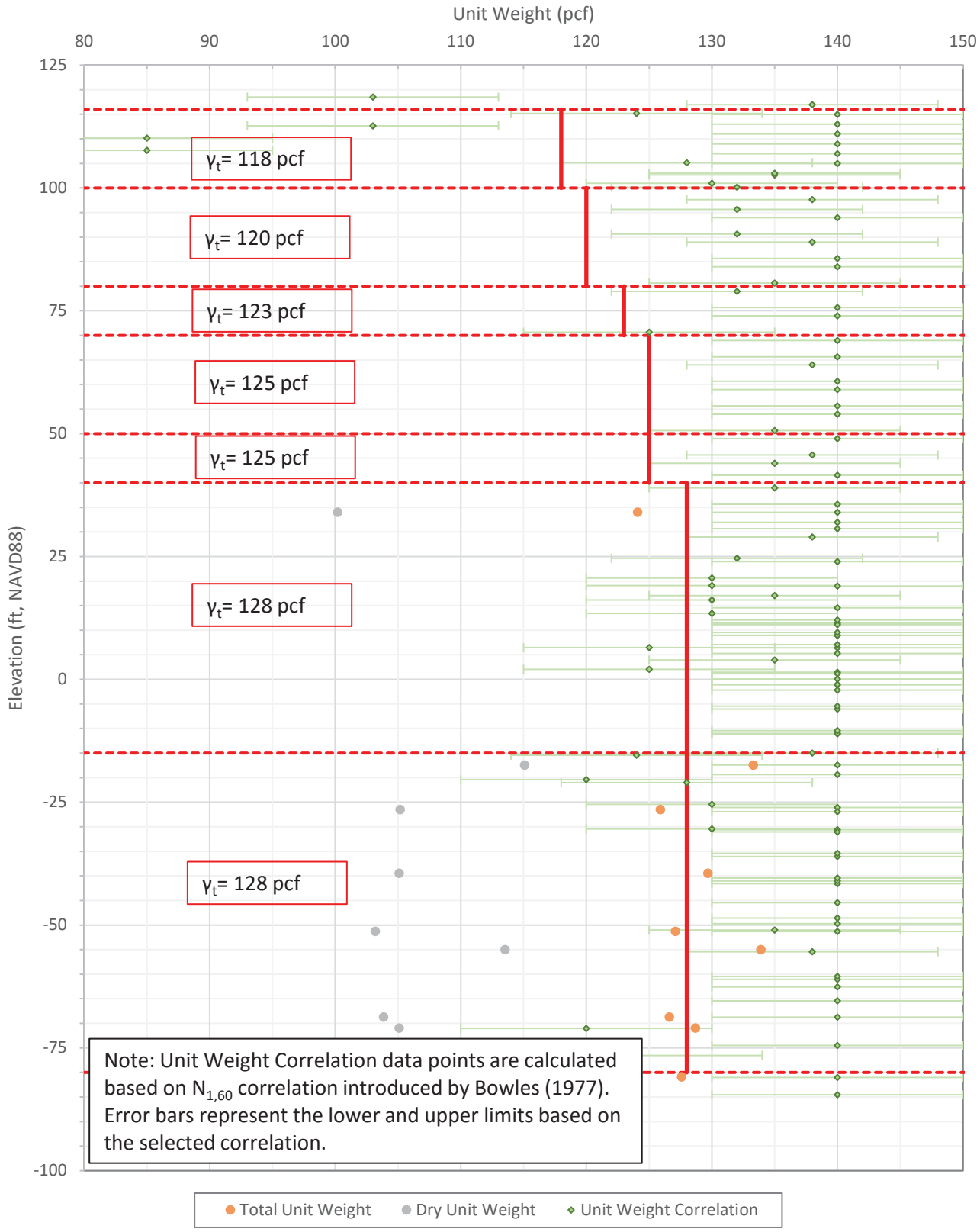
FRICTION ANGLES - AREA 3 (DP3)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



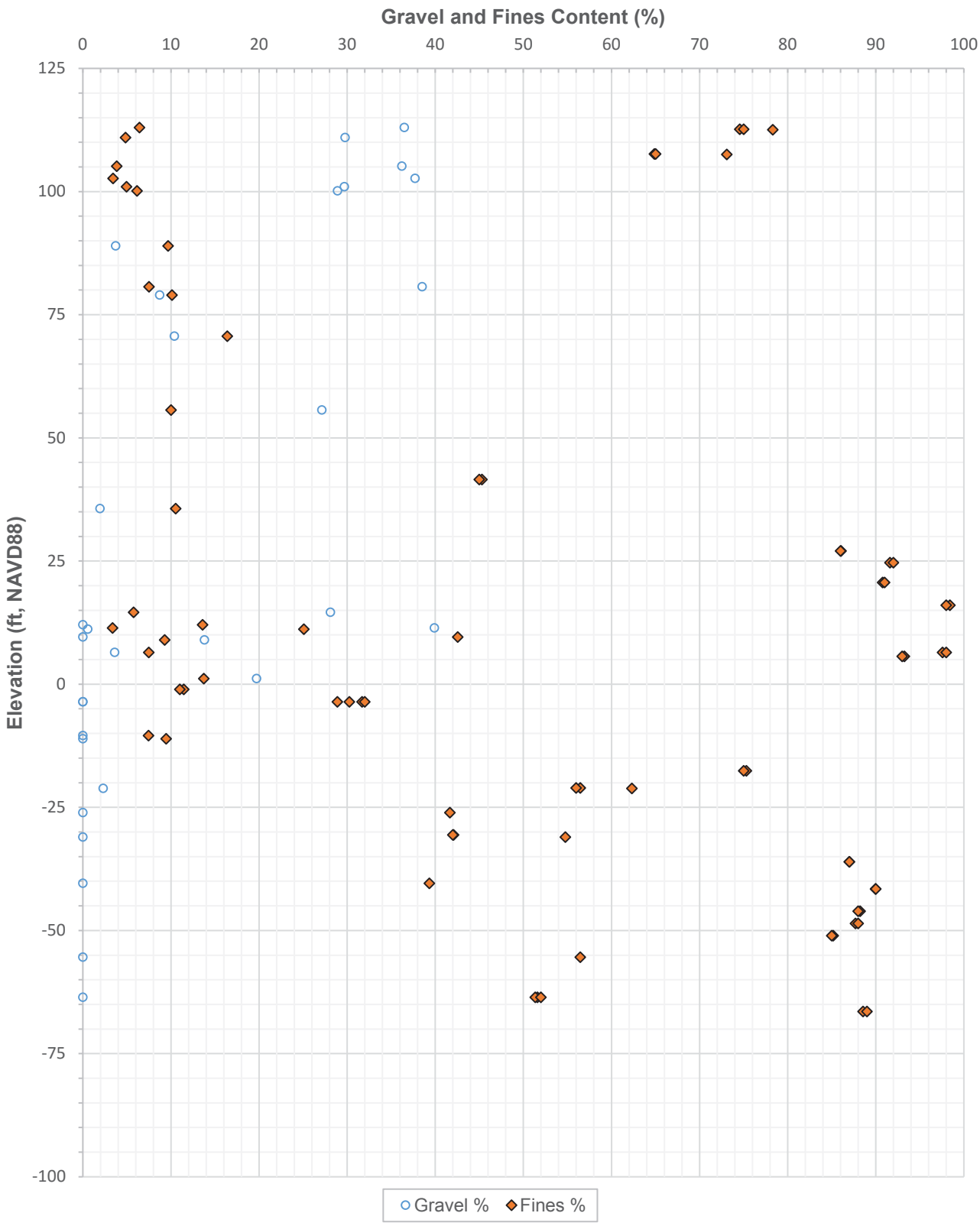
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



SPT $N_{1,60}$ - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

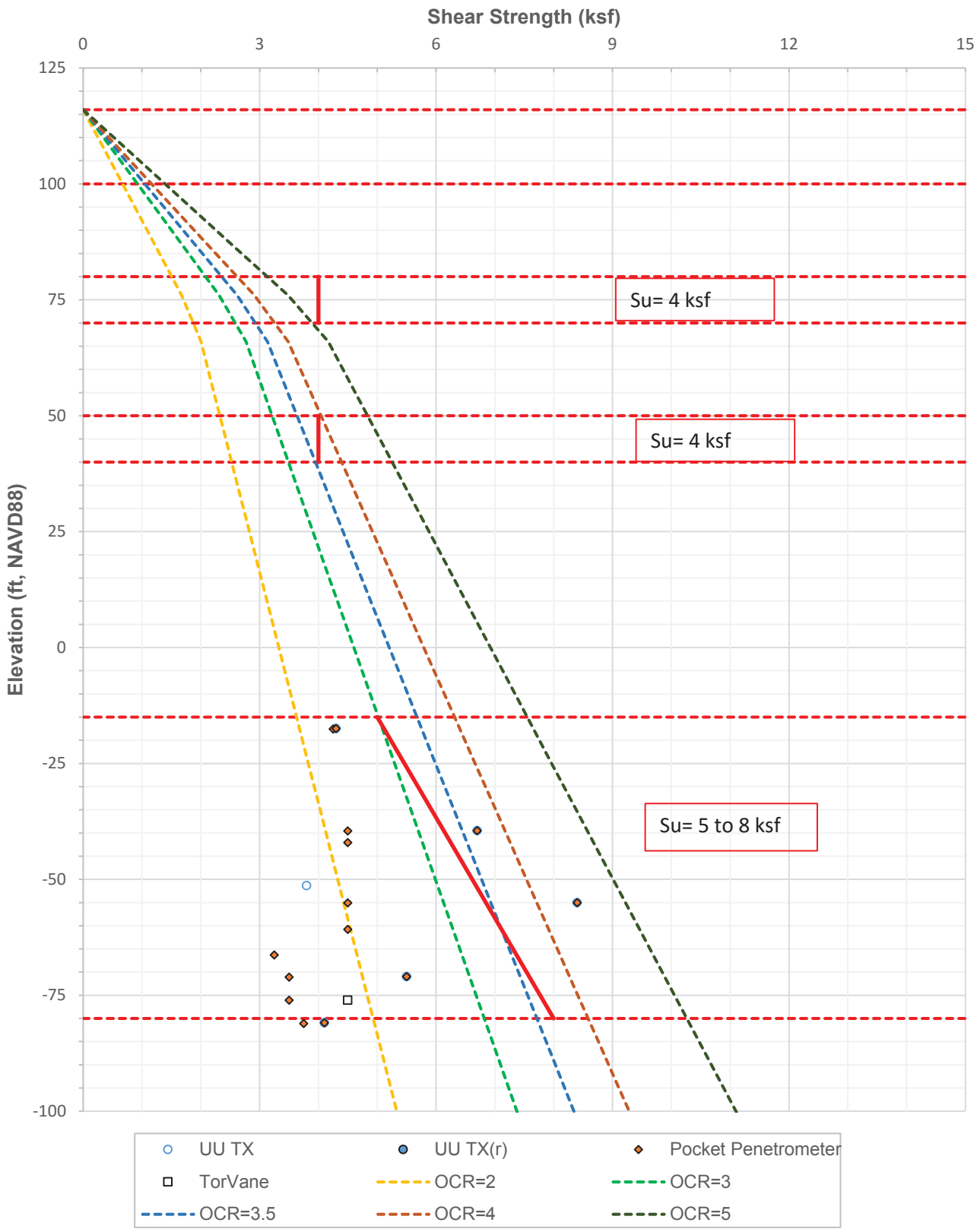


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

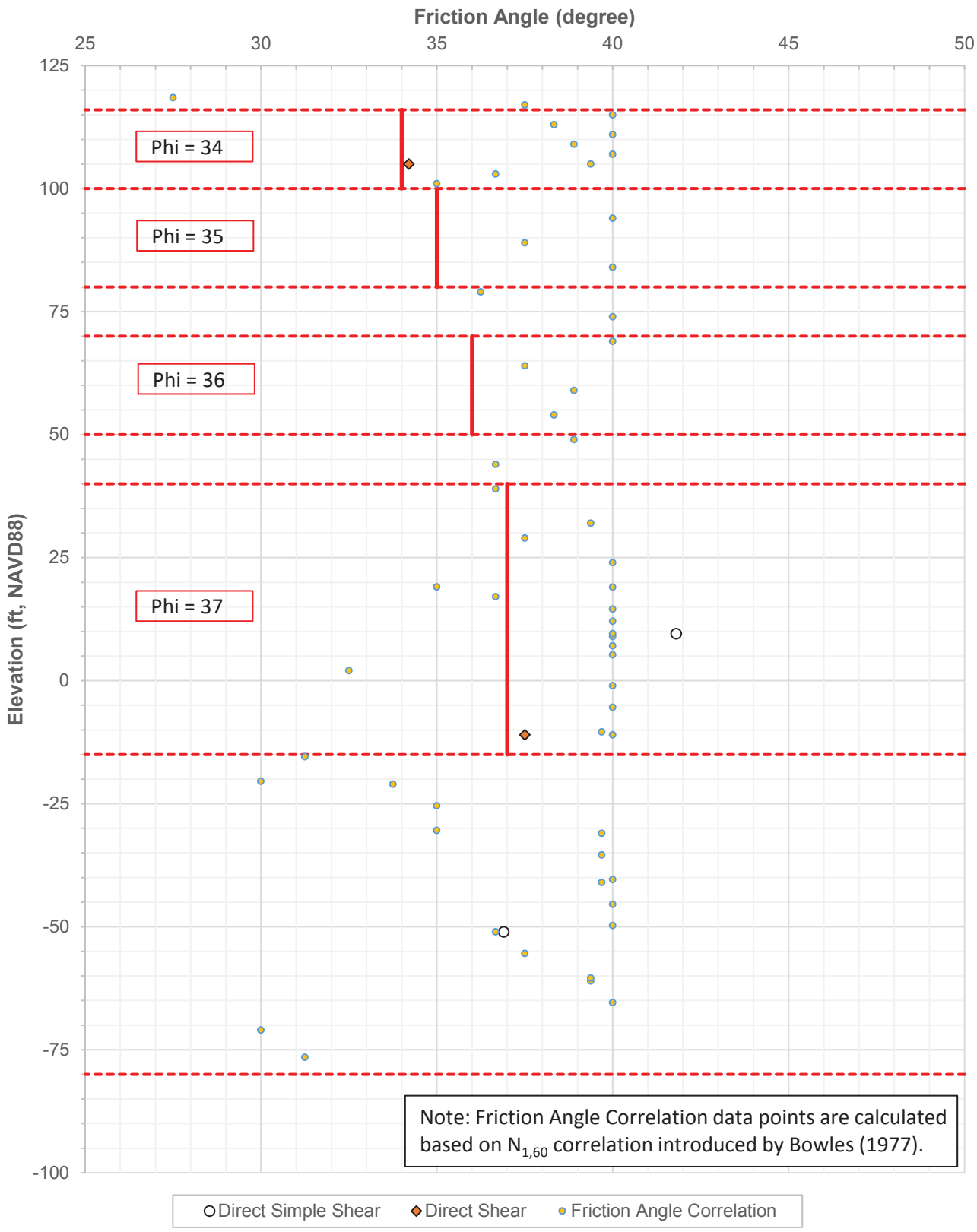


GRAVEL AND FINES CONTENT - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

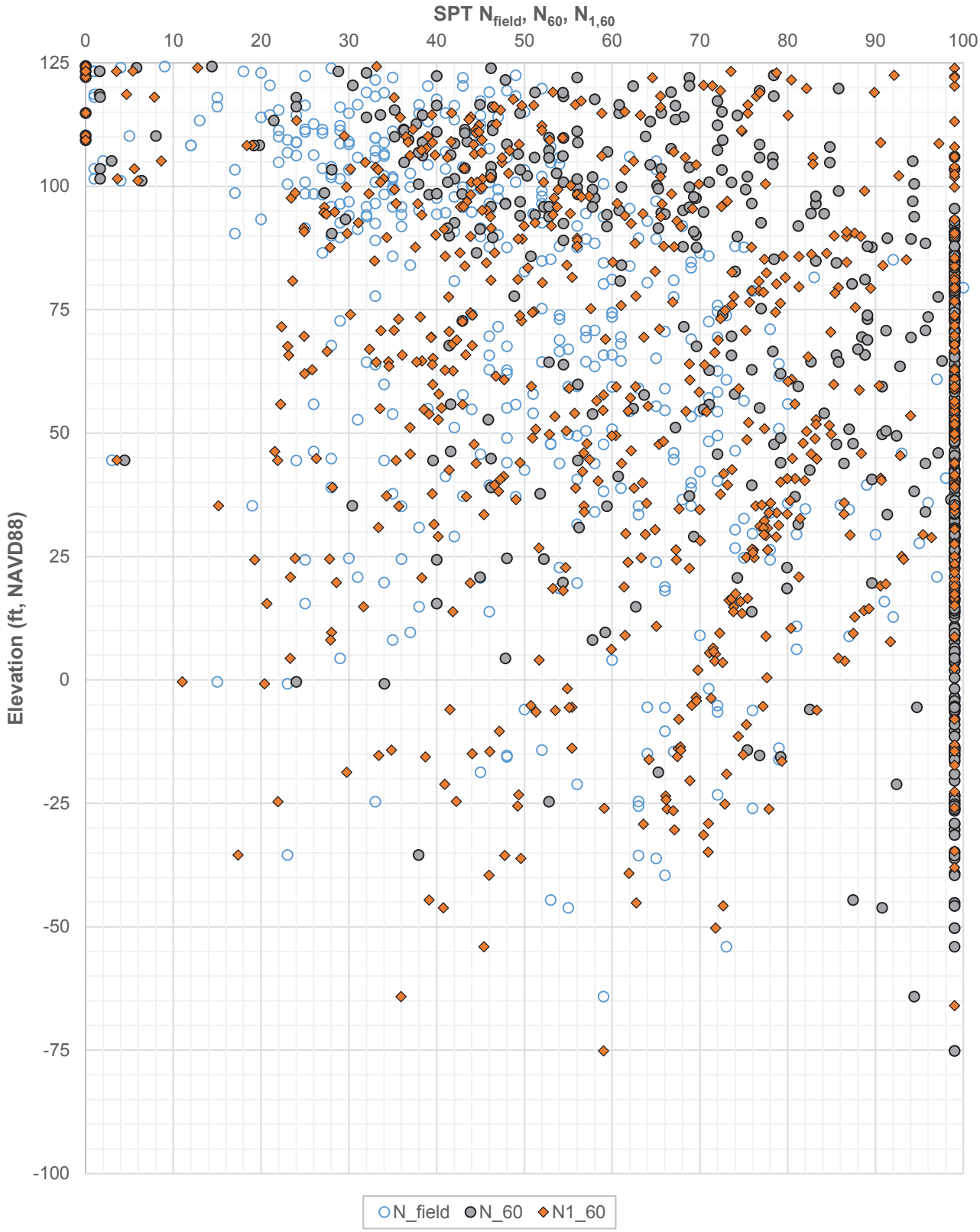
PLATE 30d



UNDRAINED SHEAR STRENGTH - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

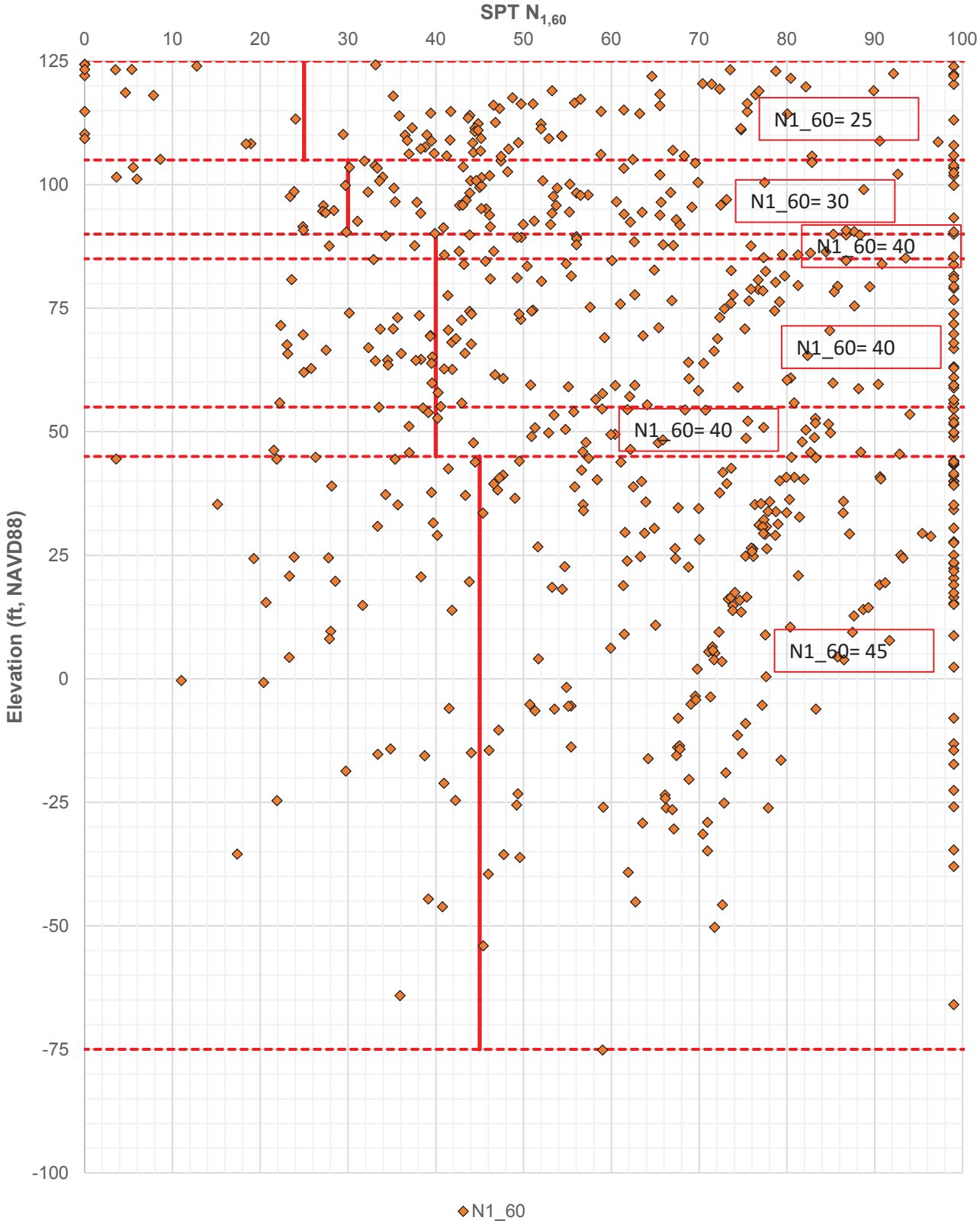


FRICTION ANGLES - AREA 3-S (DP3-S)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

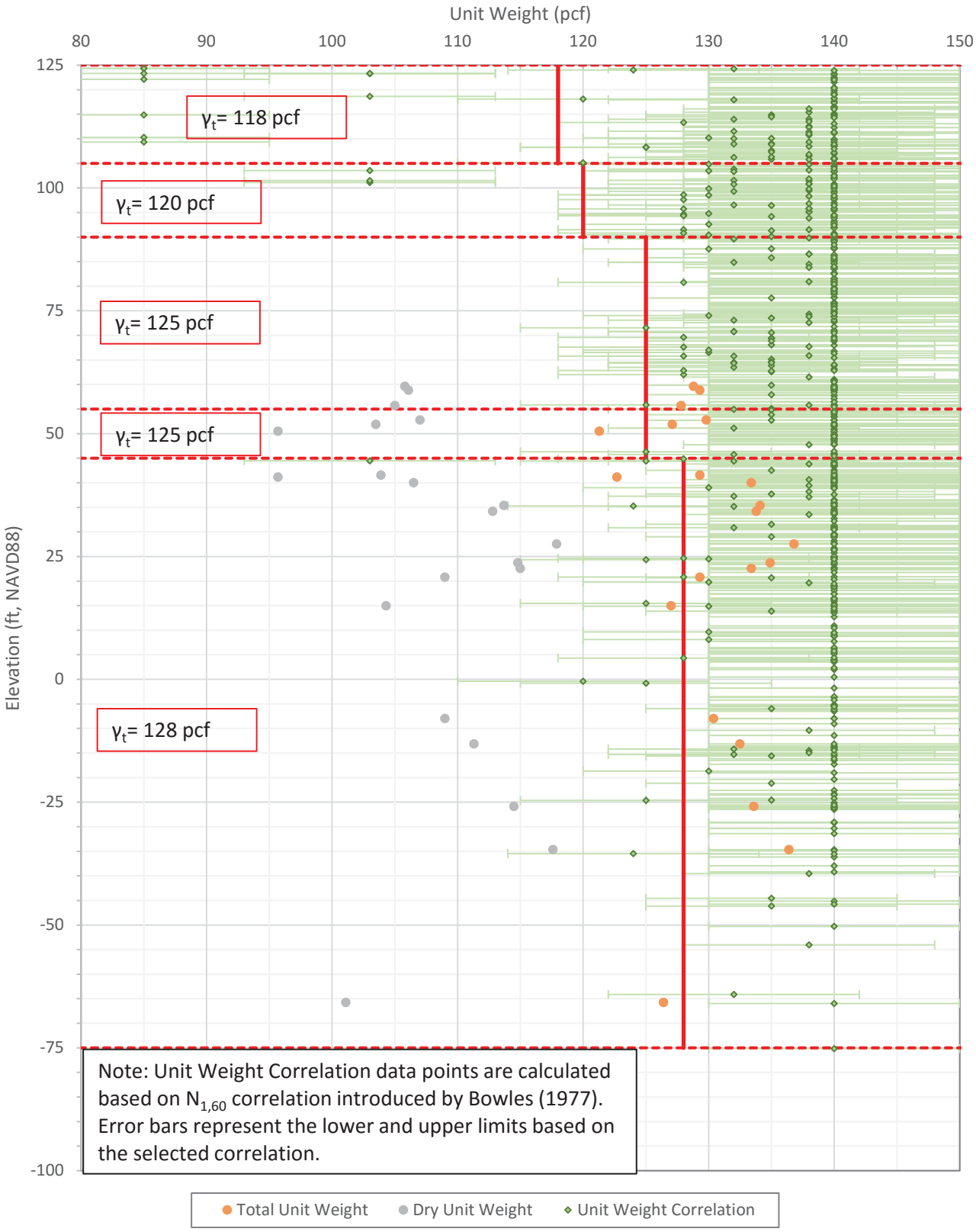


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 4A (DP4A)

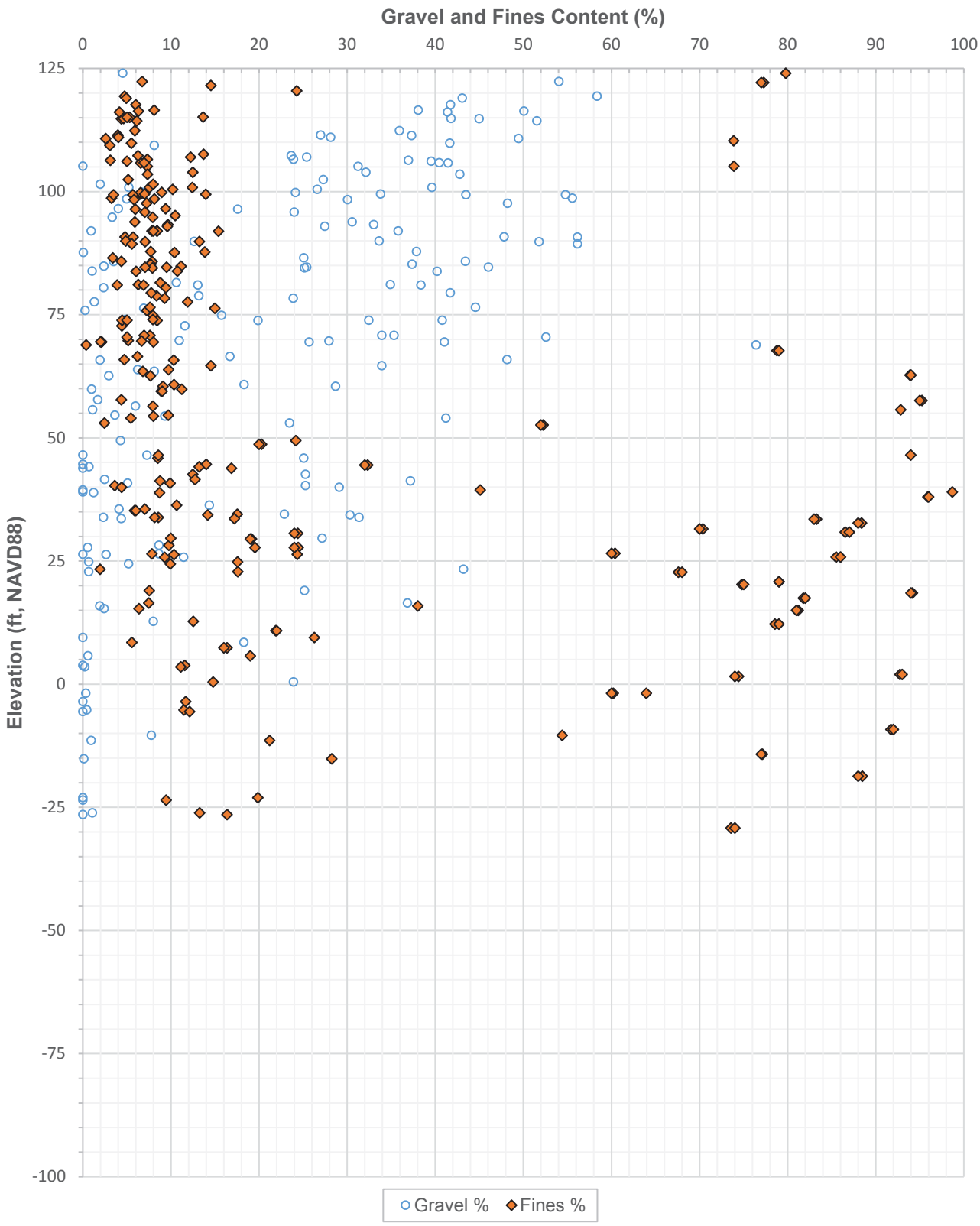
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



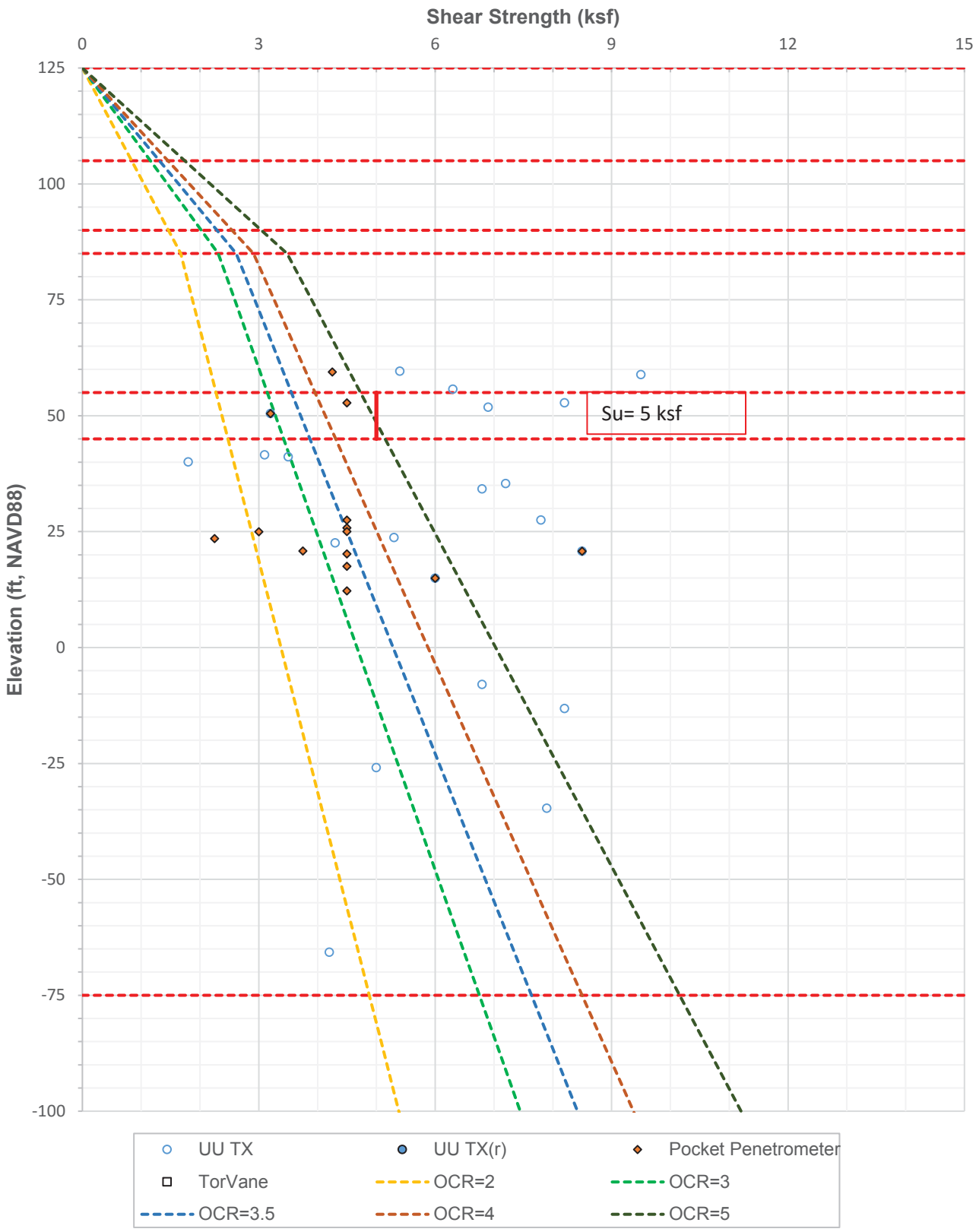
SPT $N_{1,60}$ - AREA 4A (DP4A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



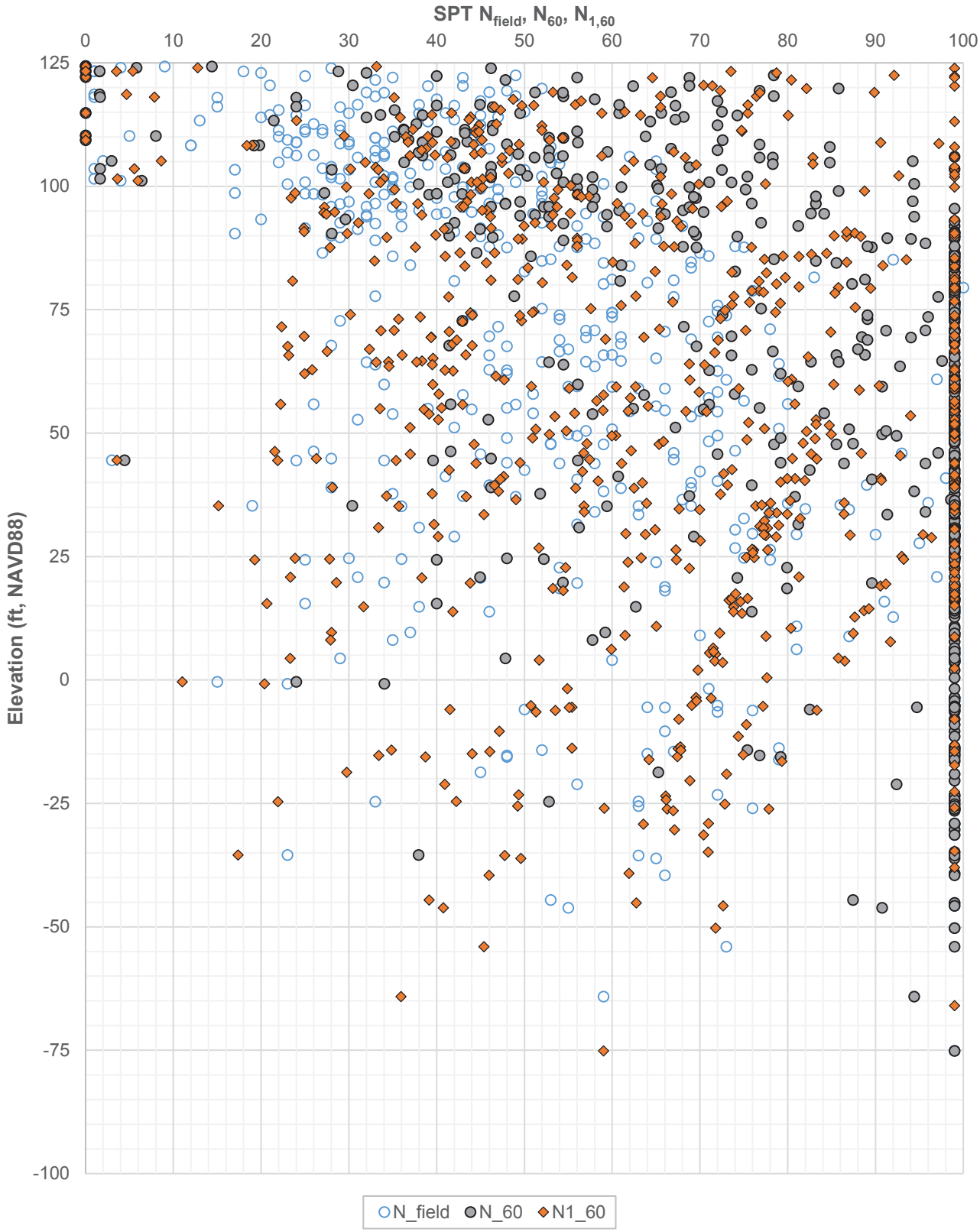
TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 4A (DP4A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



GRAVEL AND FINES CONTENT - AREA 4A (DP4A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

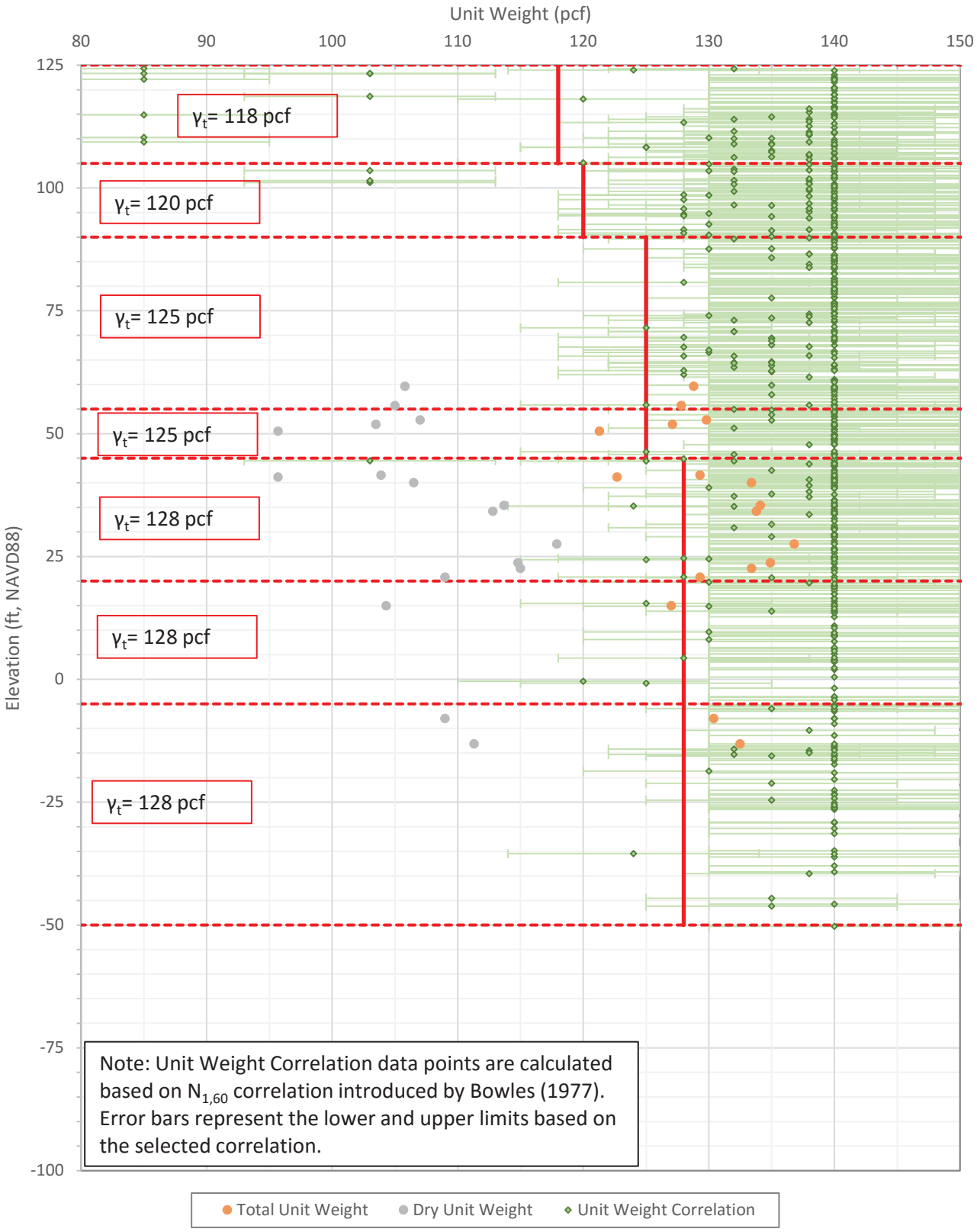


UNDRAINED SHEAR STRENGTH - AREA 4A (DP4A)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

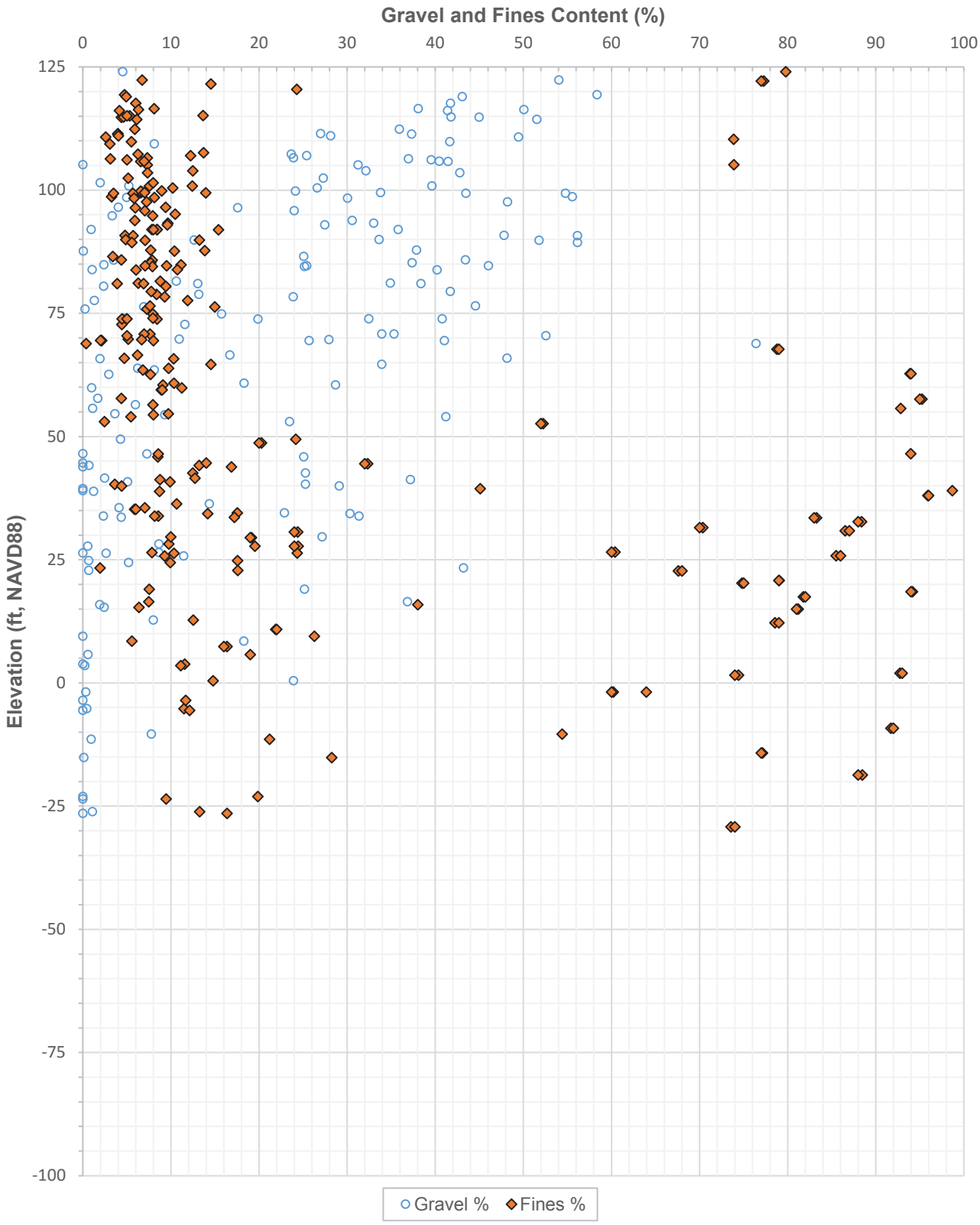


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 4B (DP4B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

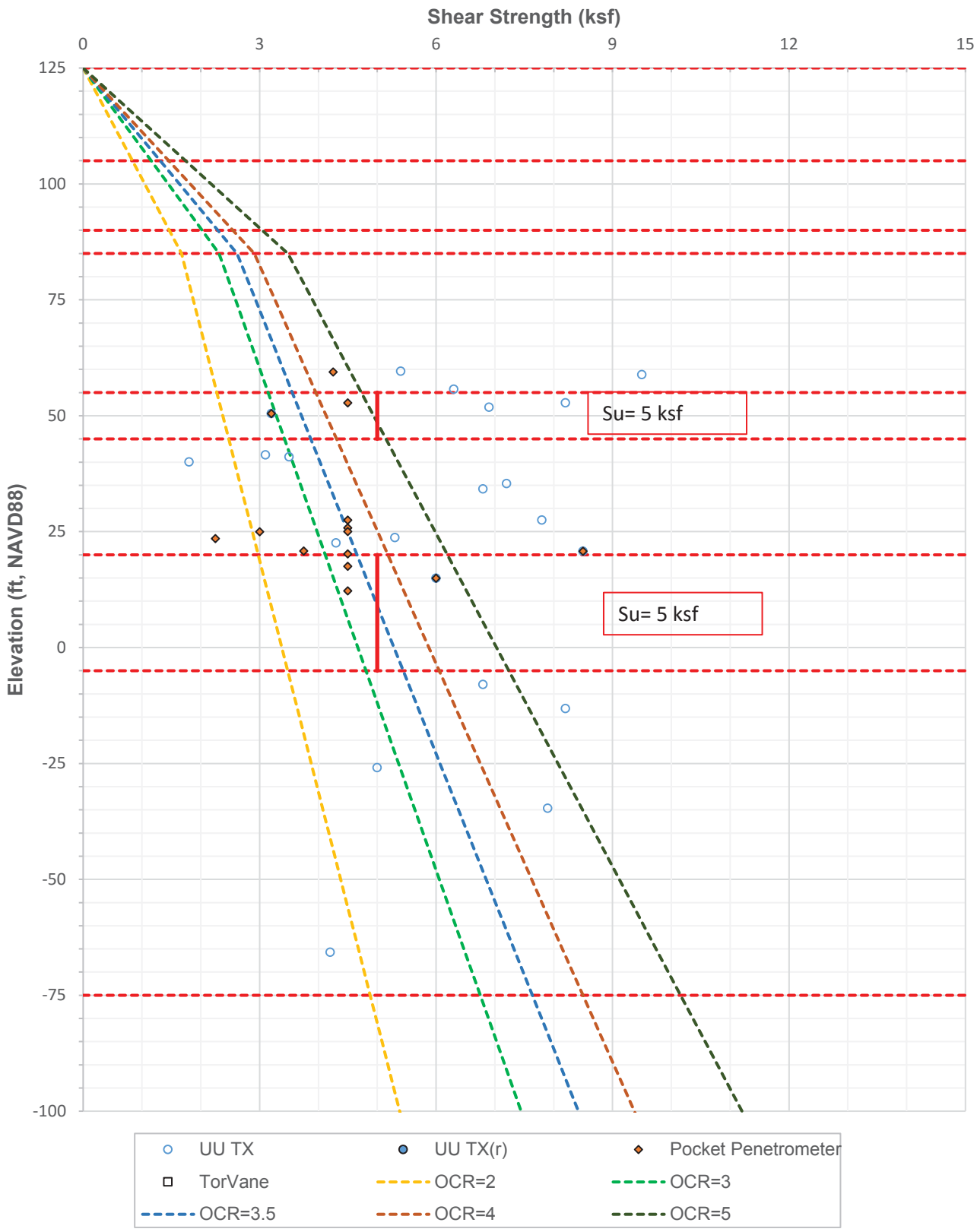
SPT $N_{1,60}$ - AREA 4B (DP4B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



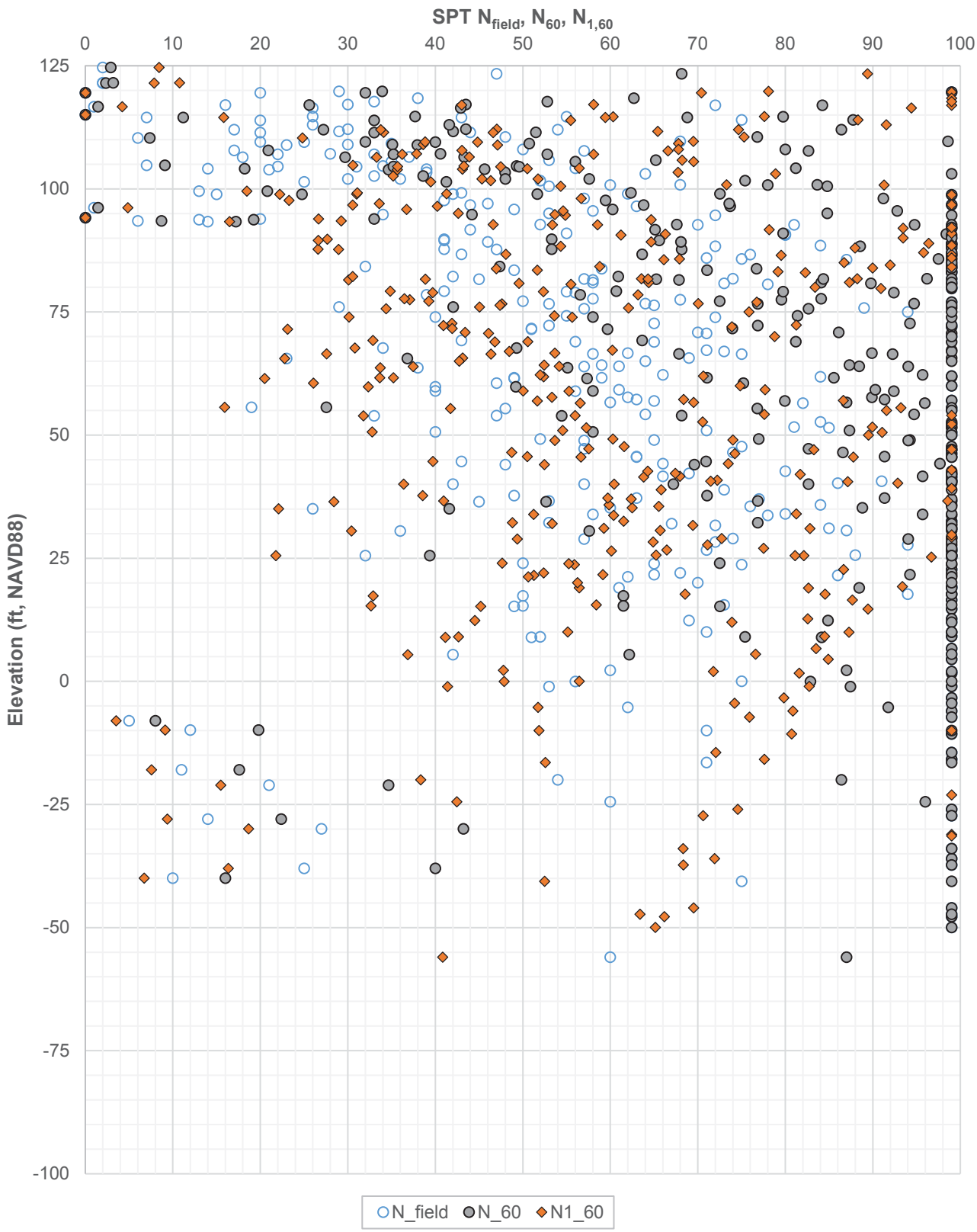
TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 4B (DP4B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



GRAVEL AND FINES CONTENT - AREA 4B (DP4B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

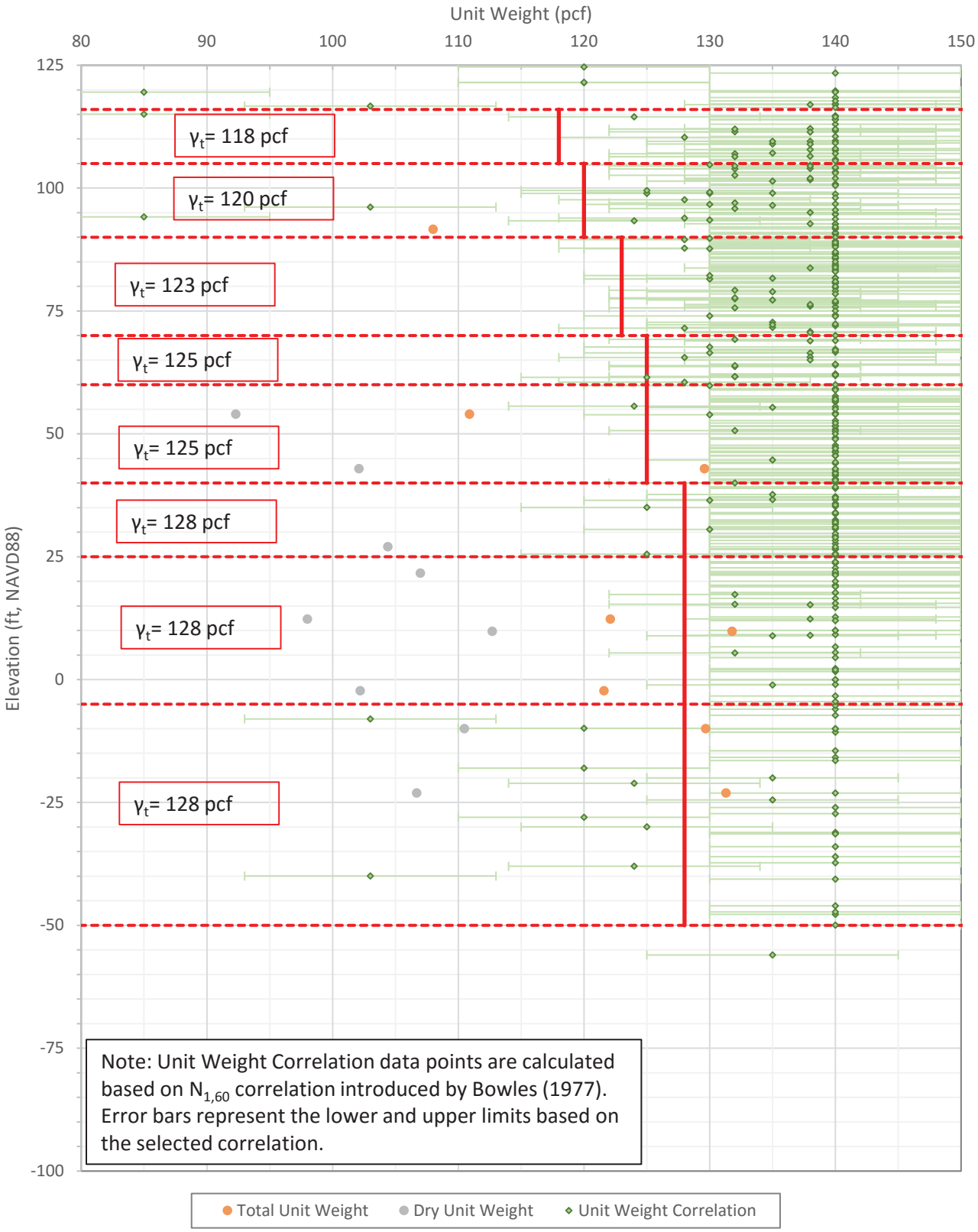


UNDRAINED SHEAR STRENGTH - AREA 4B (DP4B)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

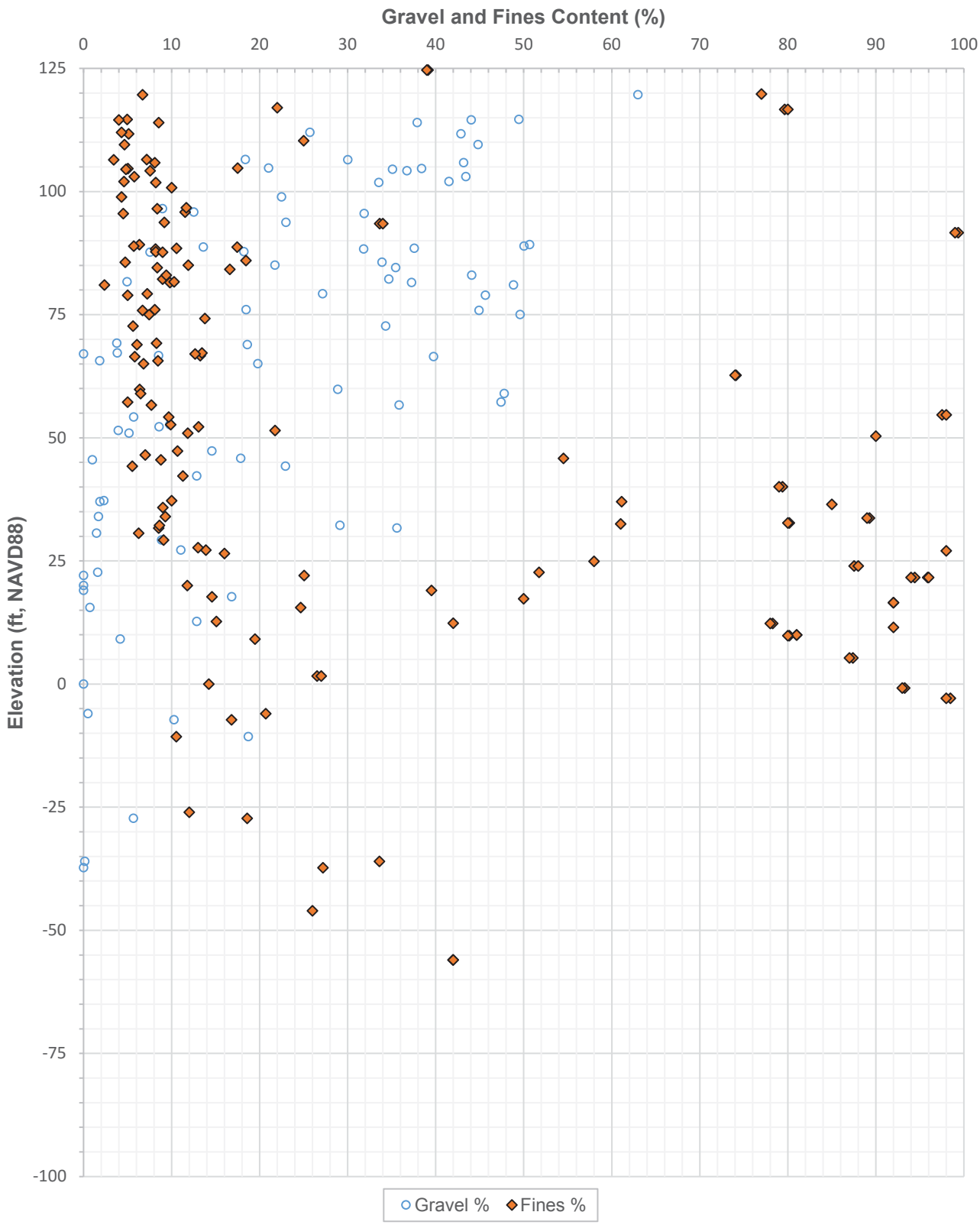


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 5 (DP5)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

SPT $N_{1,60}$ - AREA 5 (DP5)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

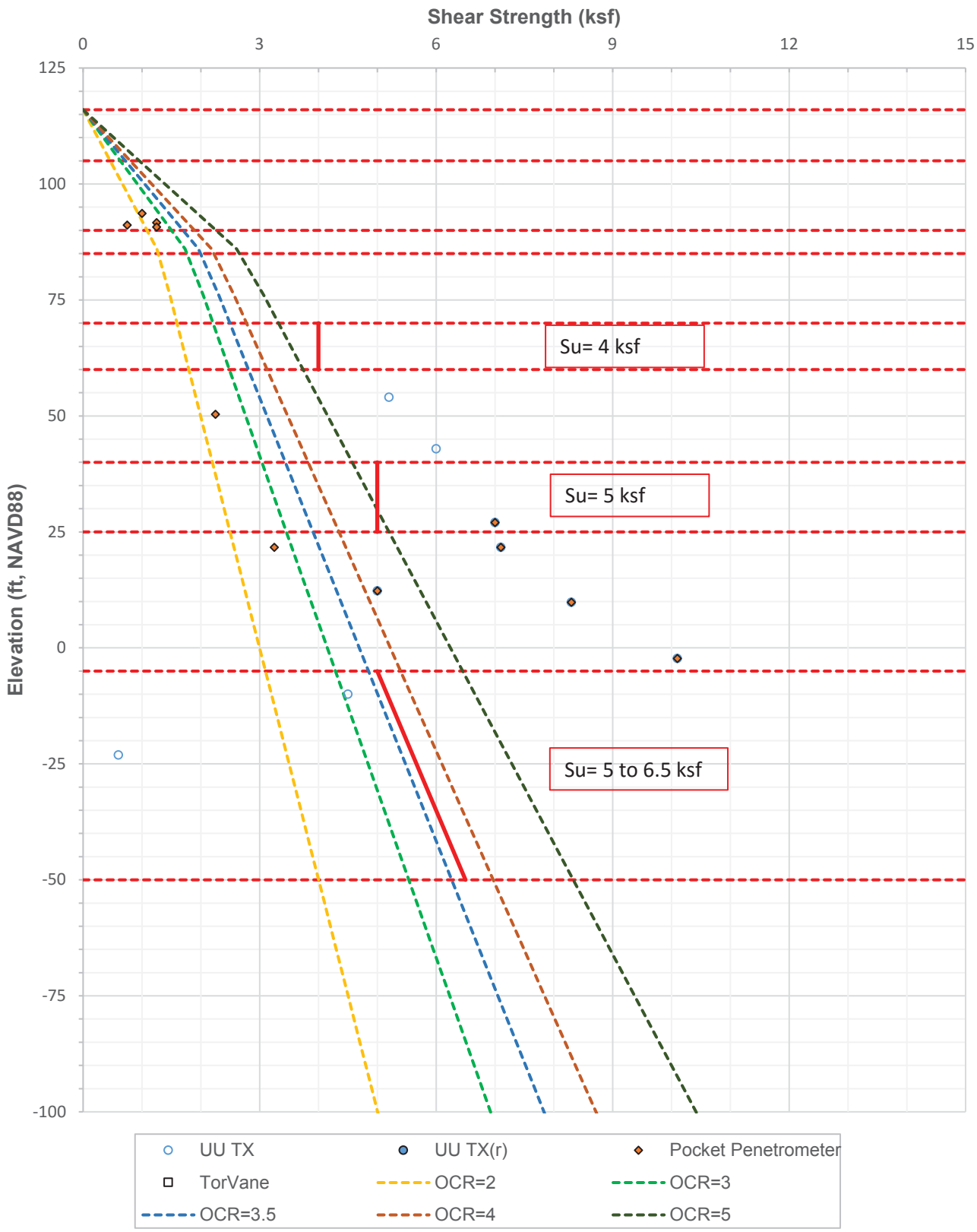


TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 5 (DP5)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

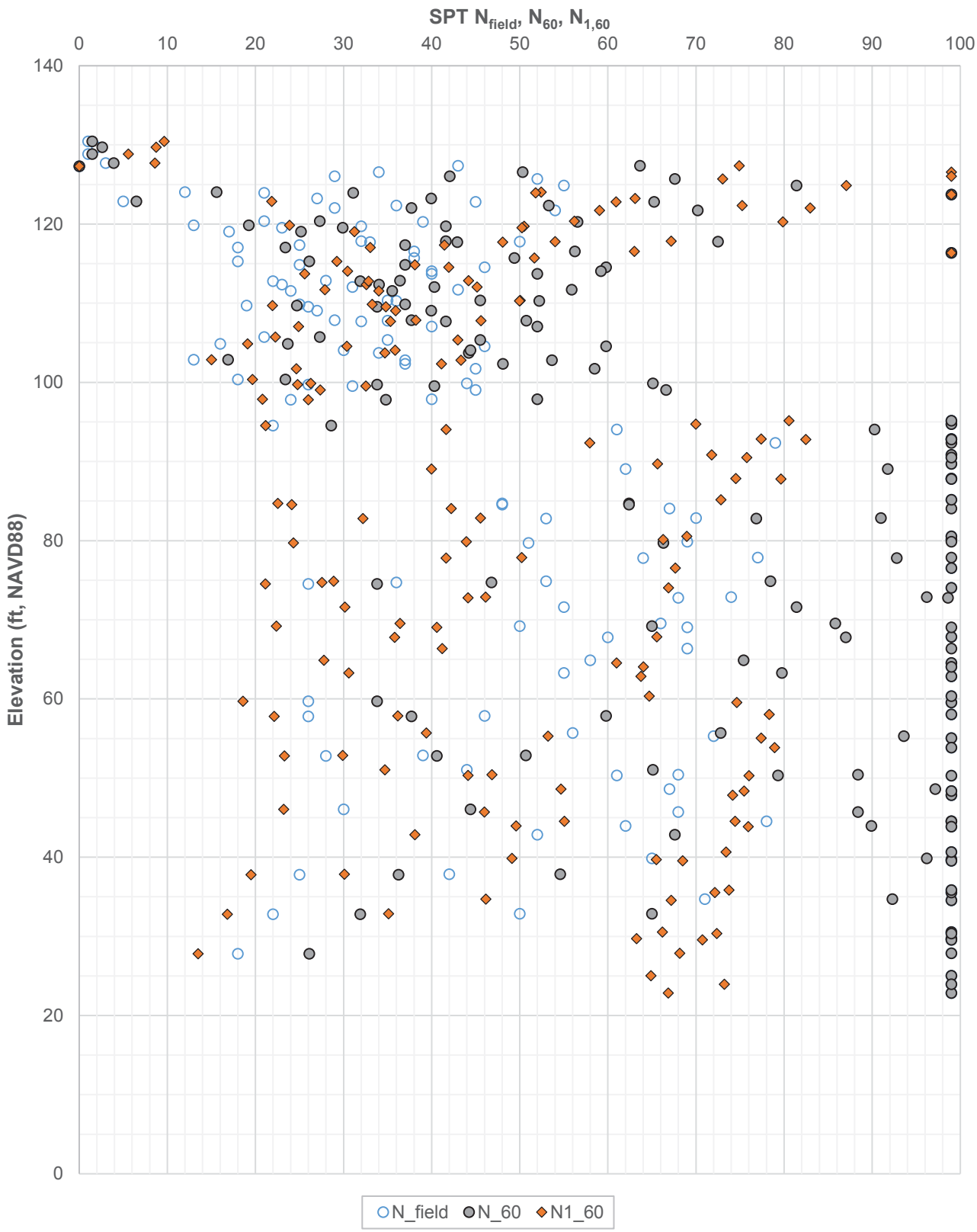


GRAVEL AND FINES CONTENT - AREA 5 (DP5)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

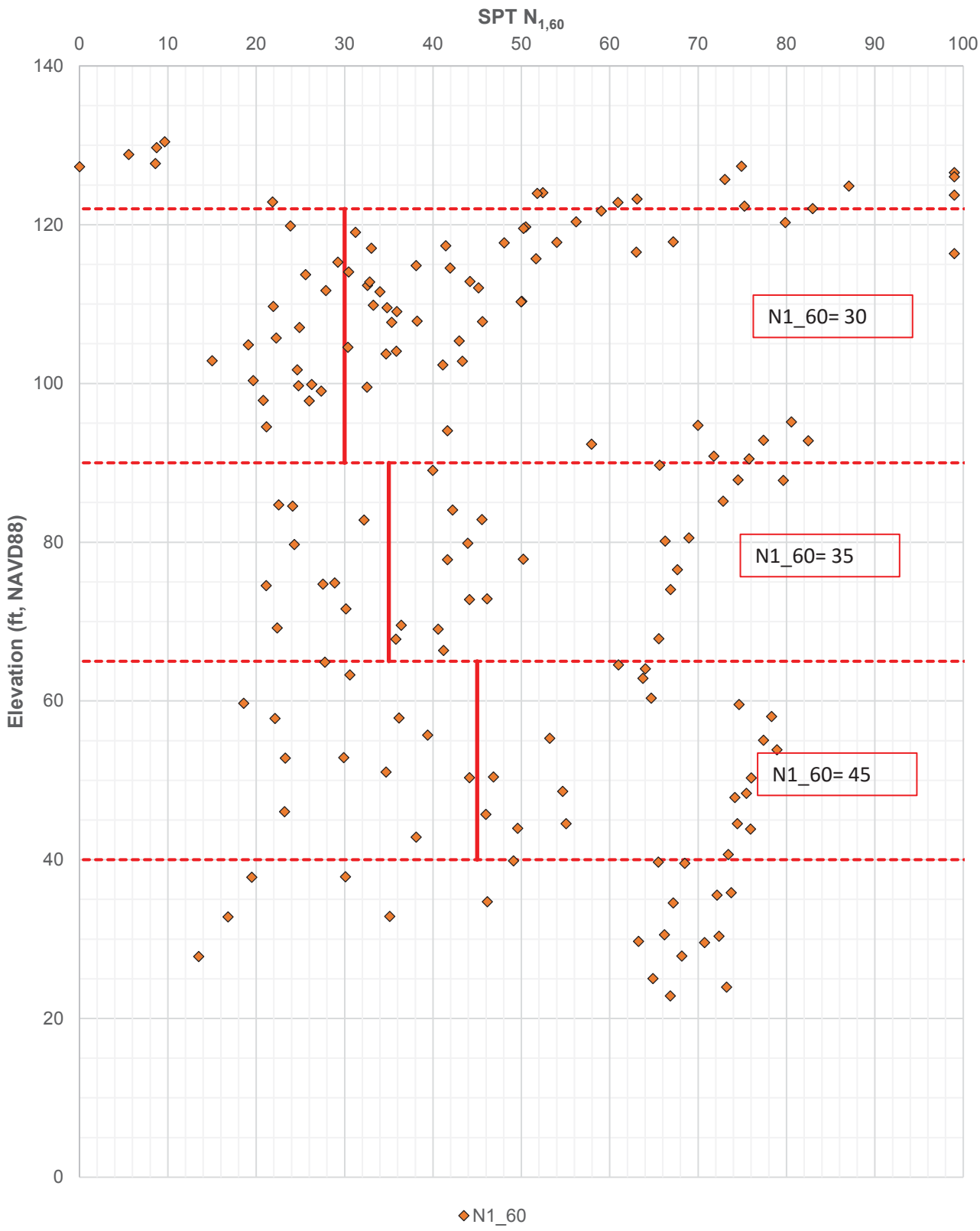
PLATE 33d



UNDRAINED SHEAR STRENGTH - AREA 5 (DP5)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

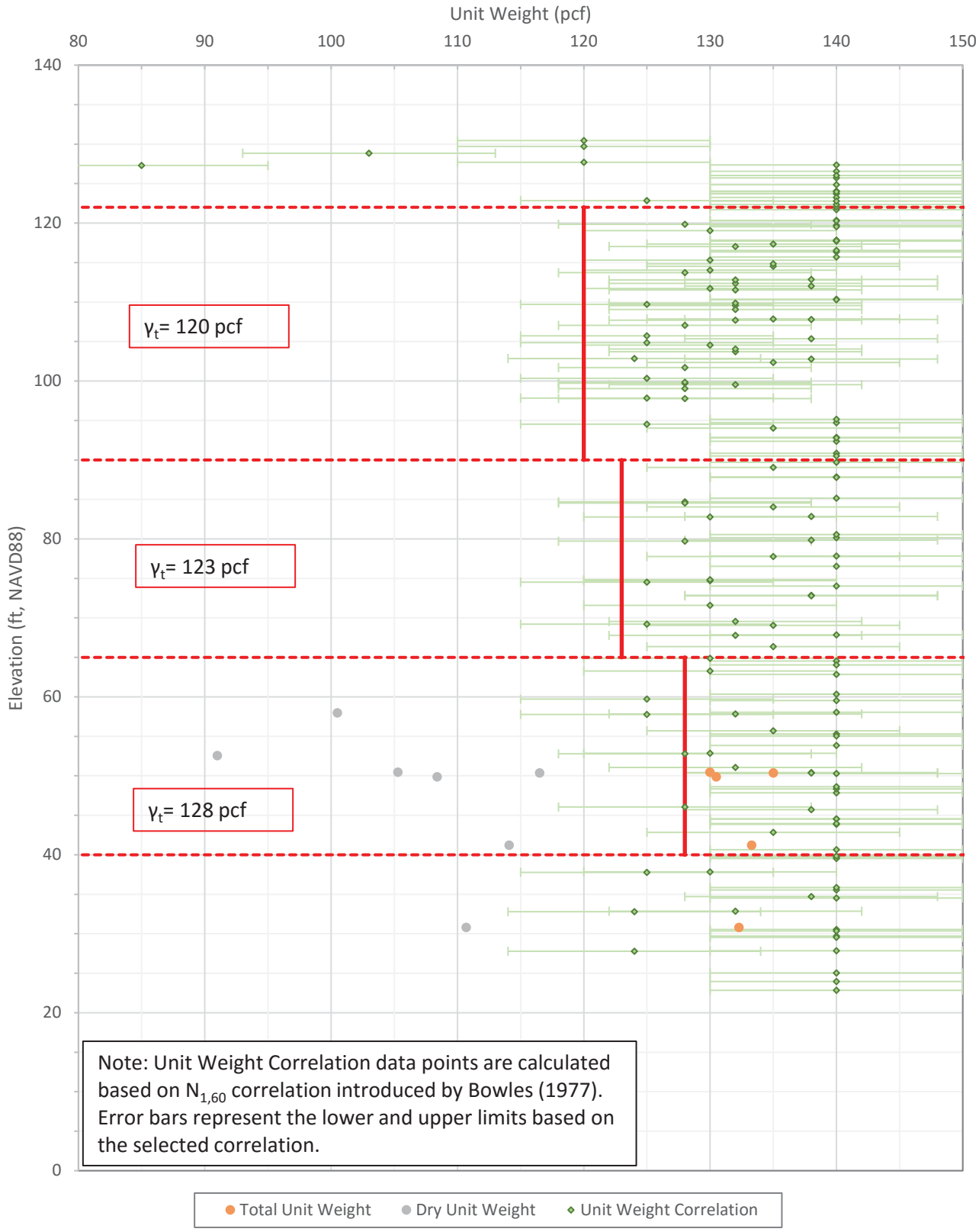


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 6 (DP6)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



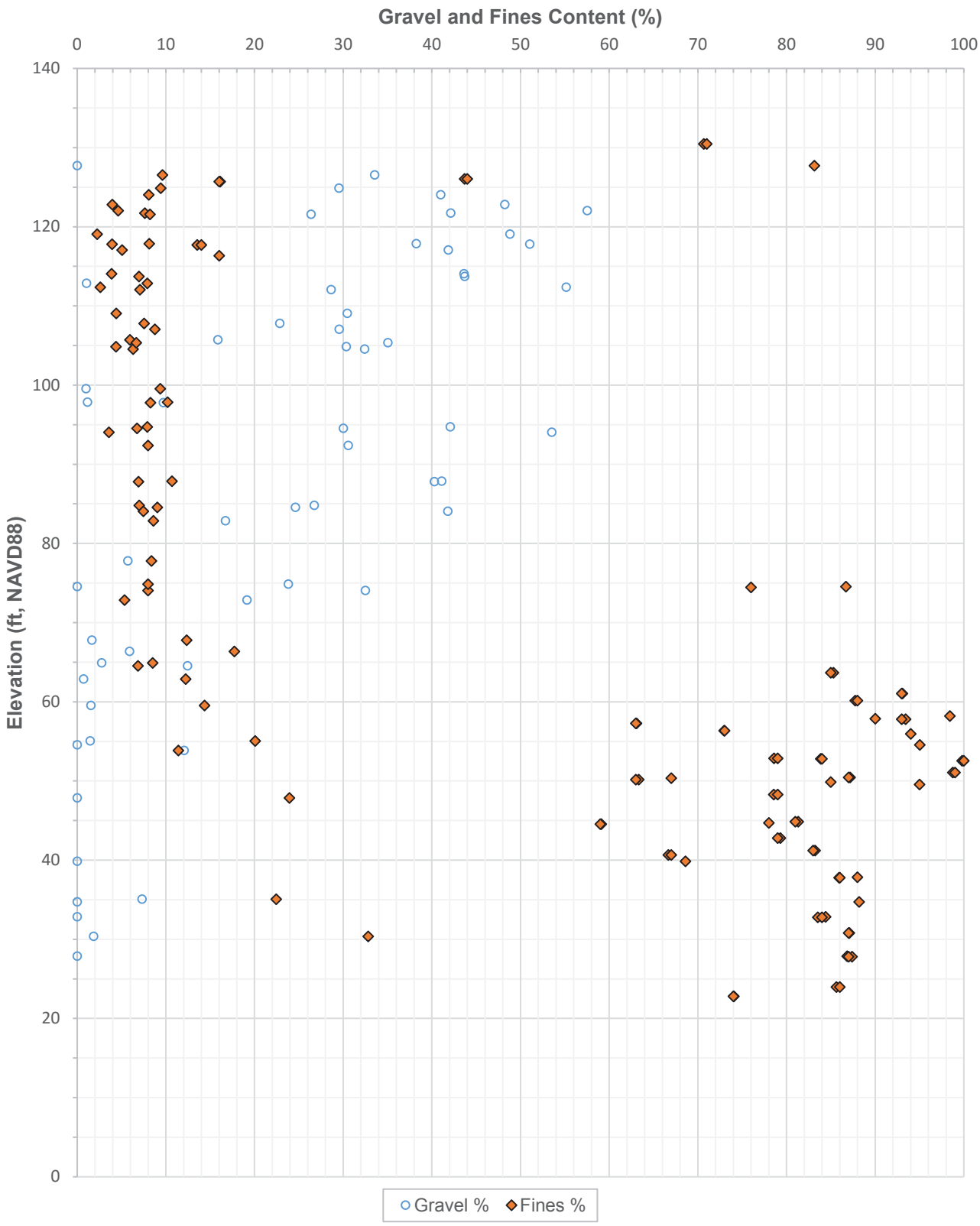
SPT $N_{1,60}$ - AREA 6 (DP6)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 34b



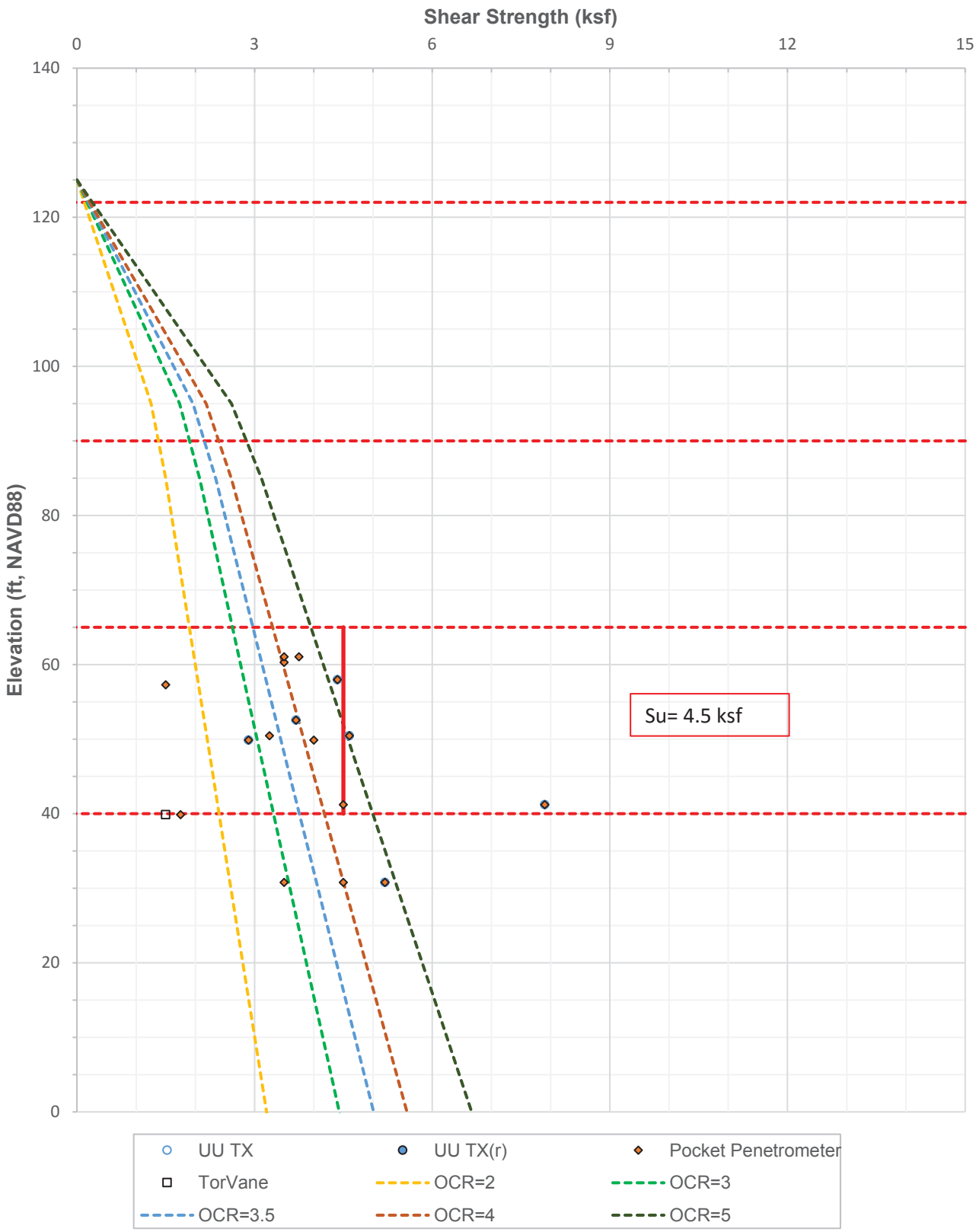
TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 6 (DP6)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 34c

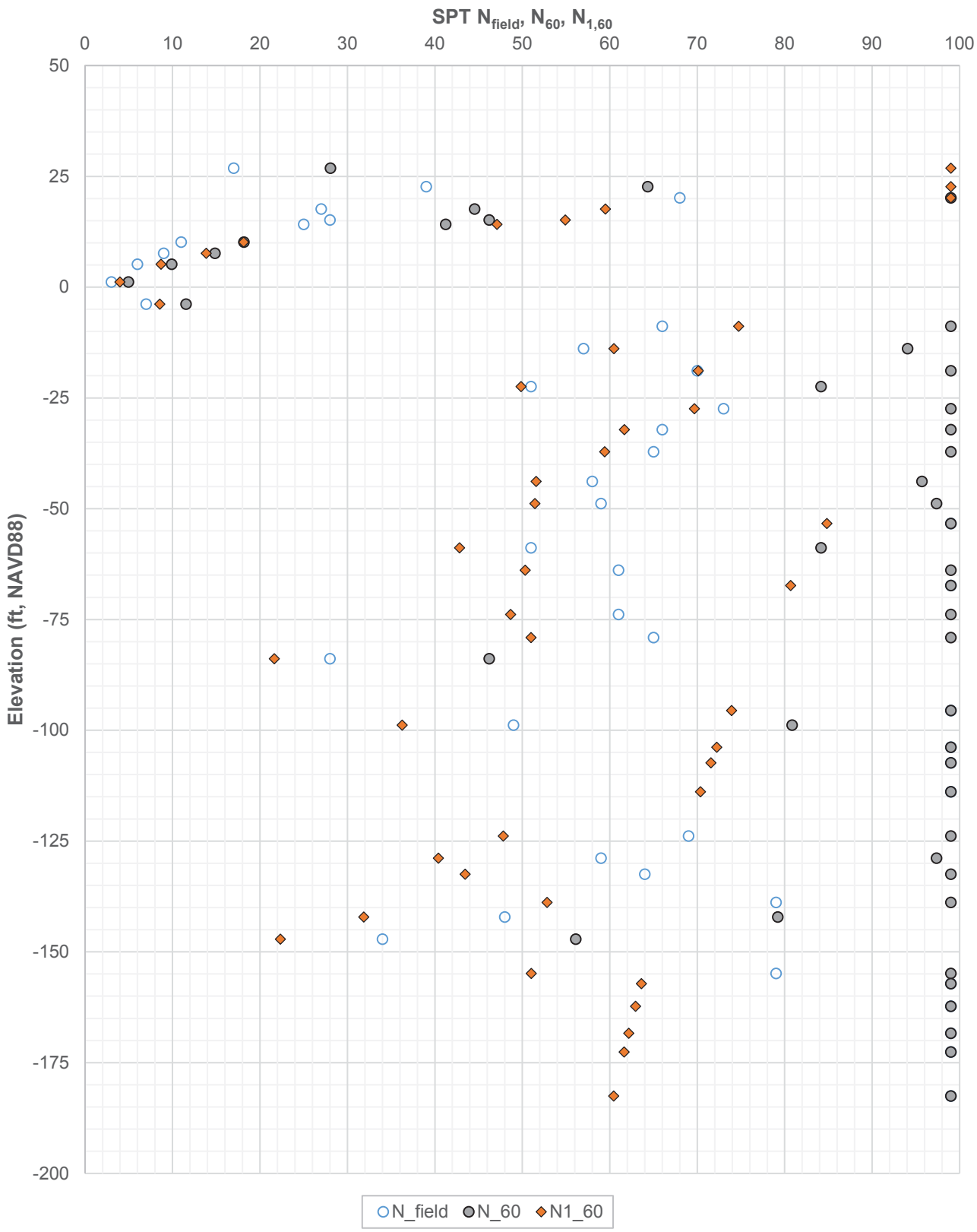


GRAVEL AND FINES CONTENT - AREA 6 (DP6)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

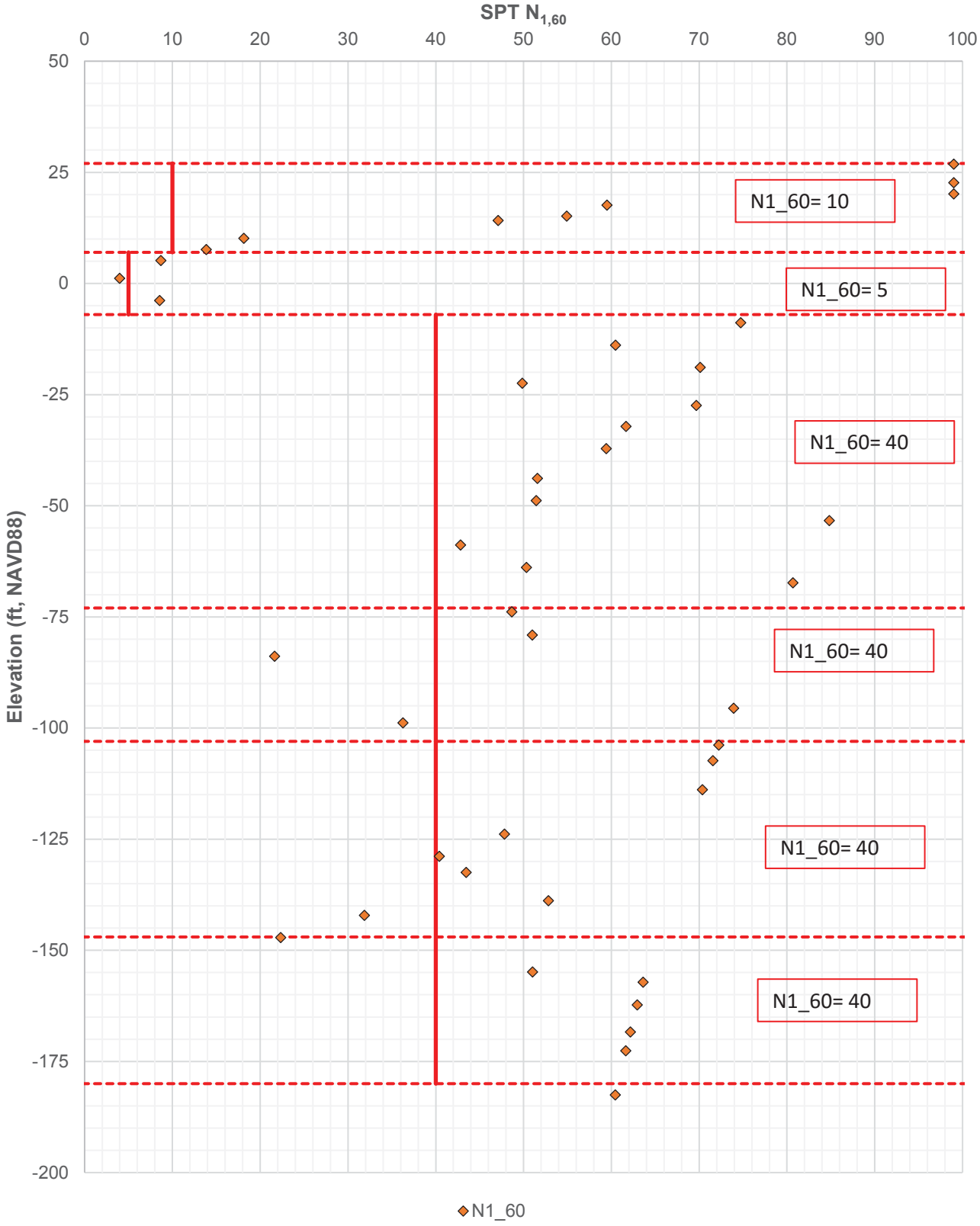
PLATE 34d



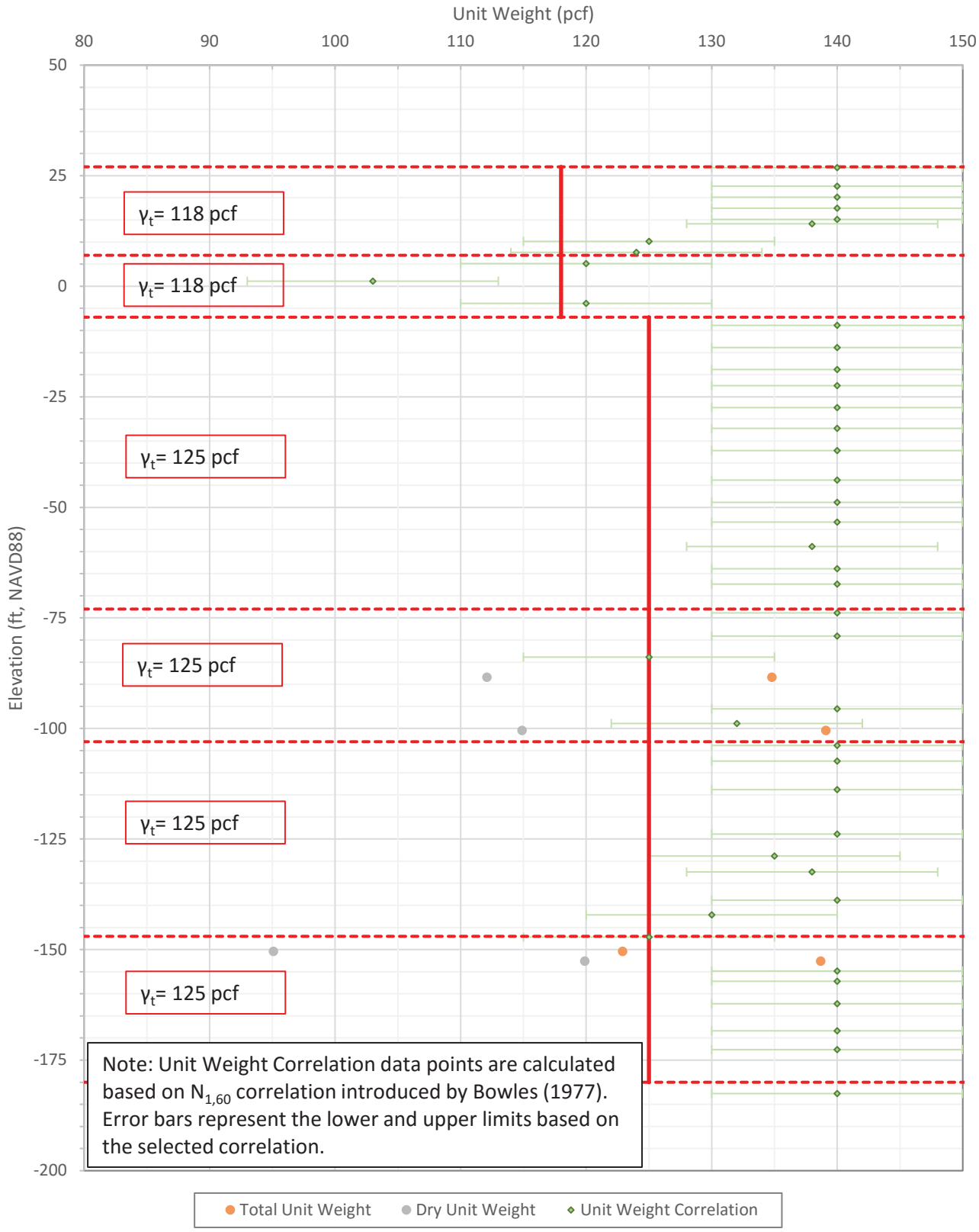
UNDRAINED SHEAR STRENGTH - AREA 6 (DP6)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



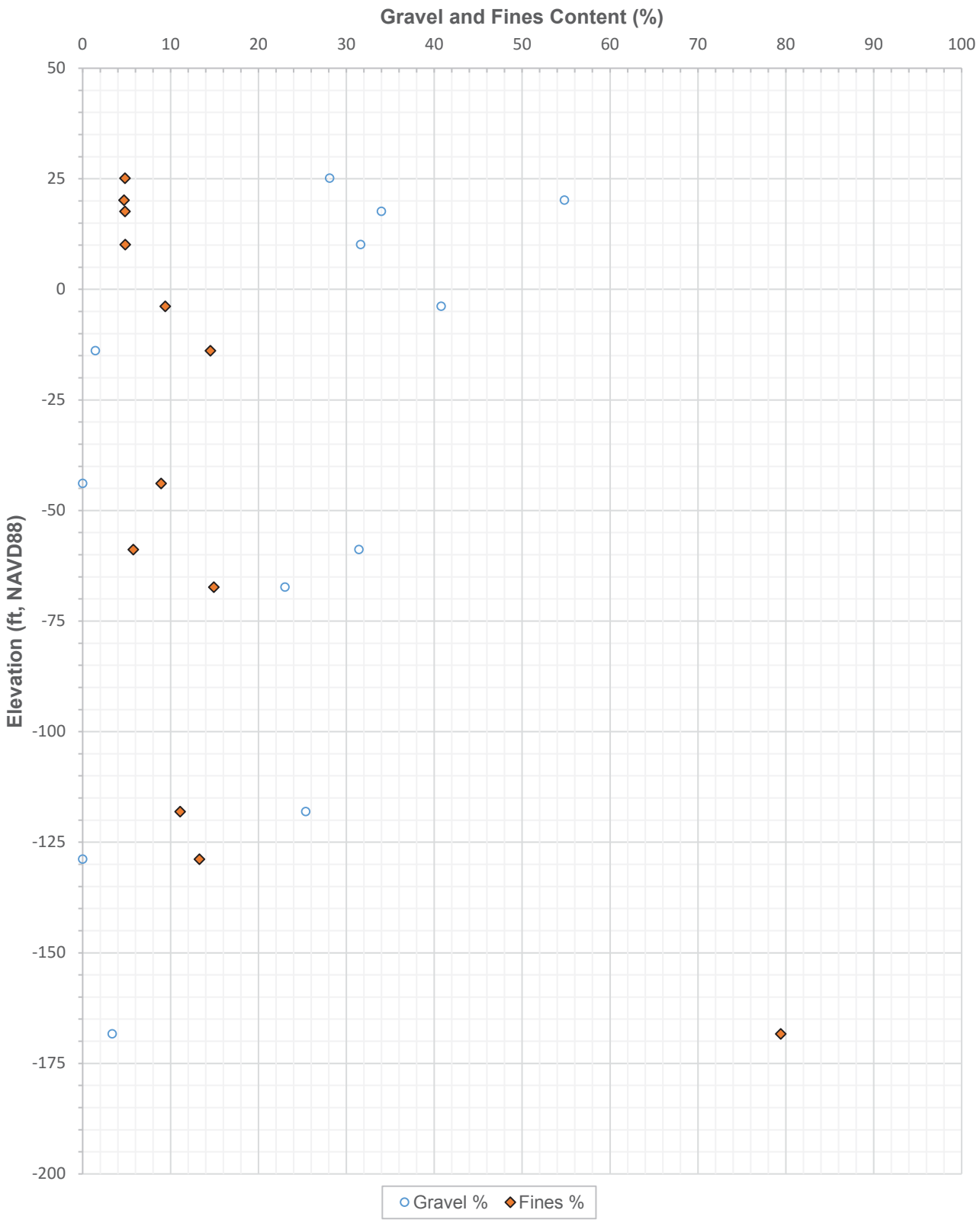
SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 8 (B-176)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



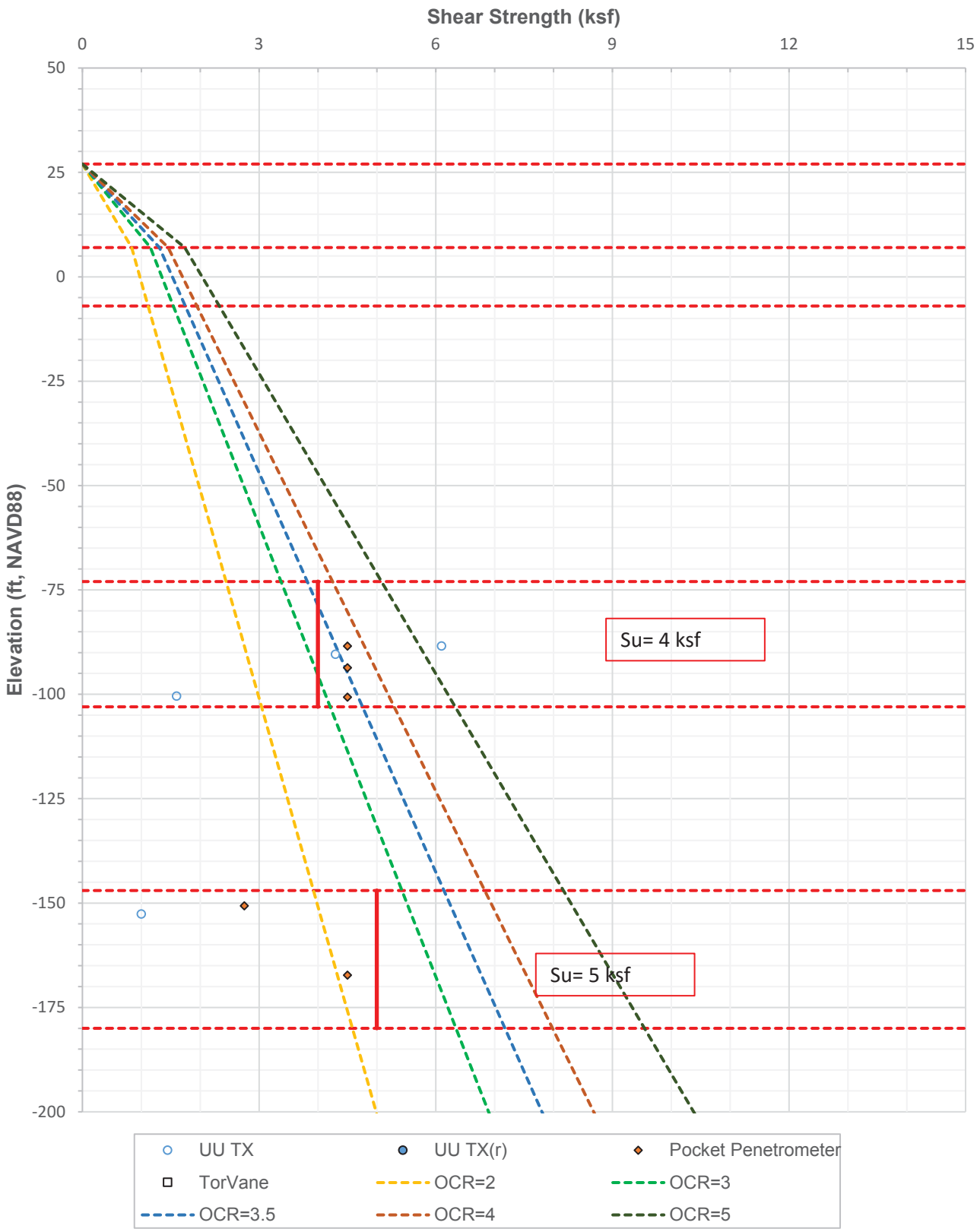
SPT $N_{1,60}$ - AREA 8 (B-176)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



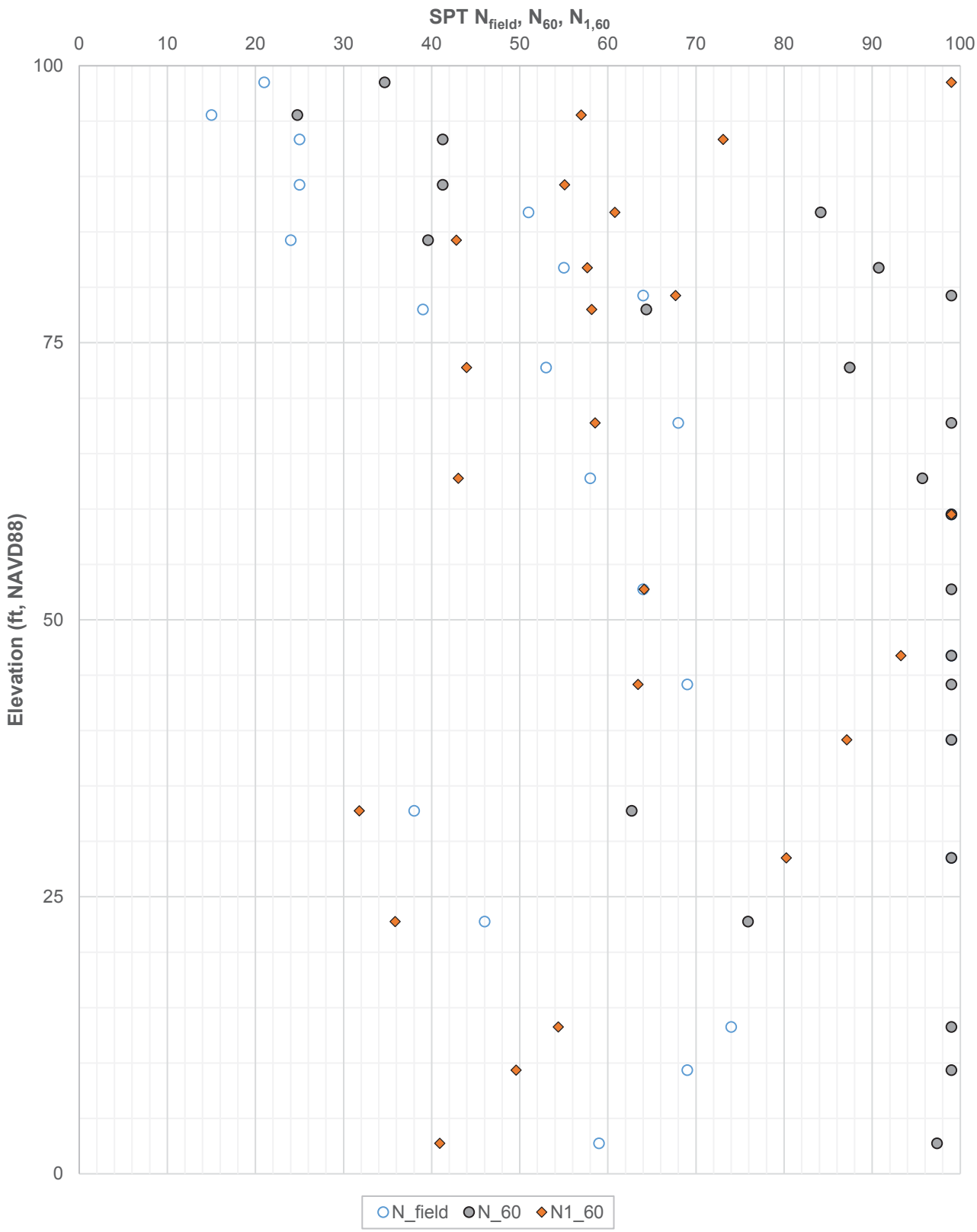
TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 8 (B-176)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



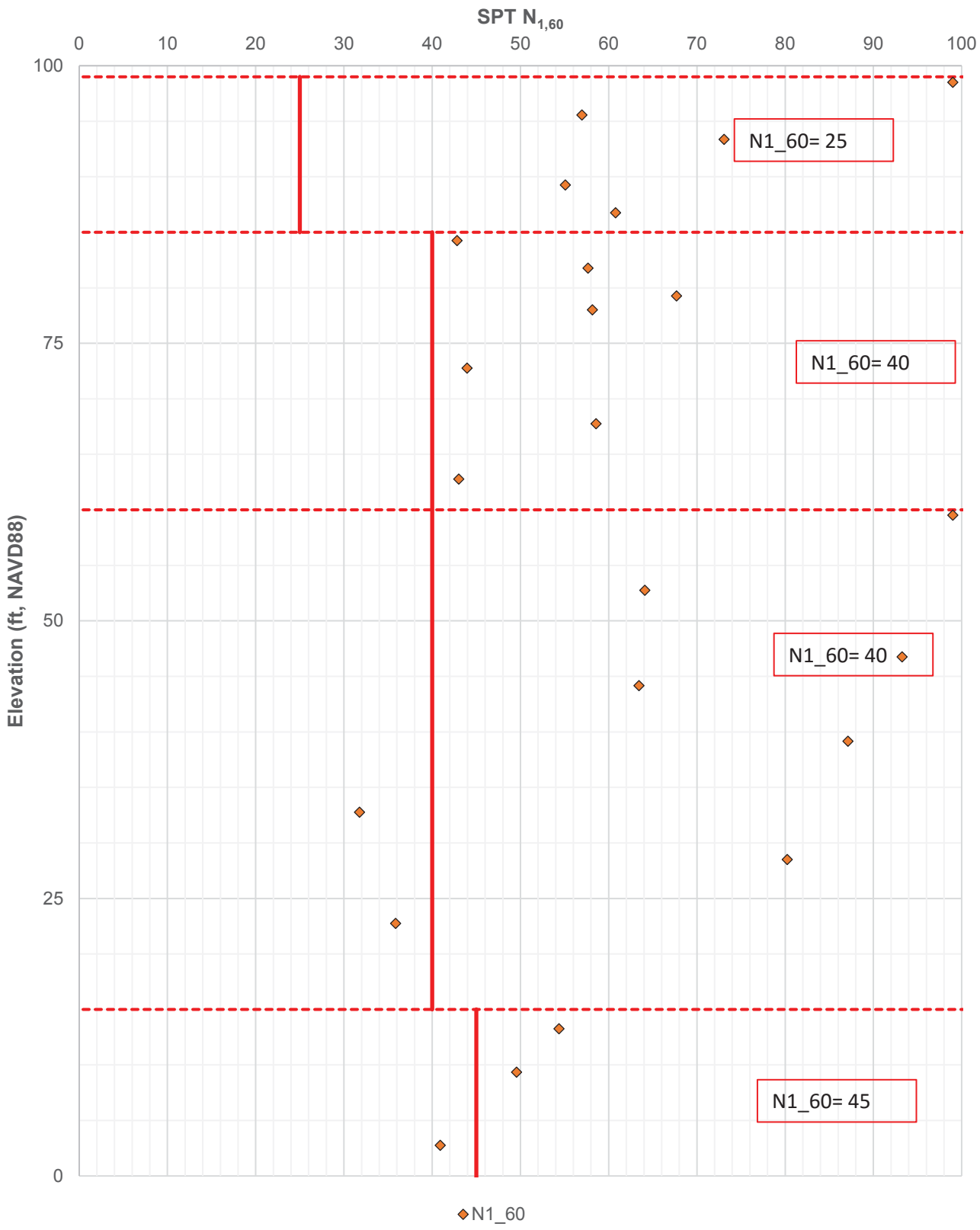
GRAVEL AND FINES CONTENT - AREA 8 (B-176)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



UNDRAINED SHEAR STRENGTH - AREA 8 (B-176)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

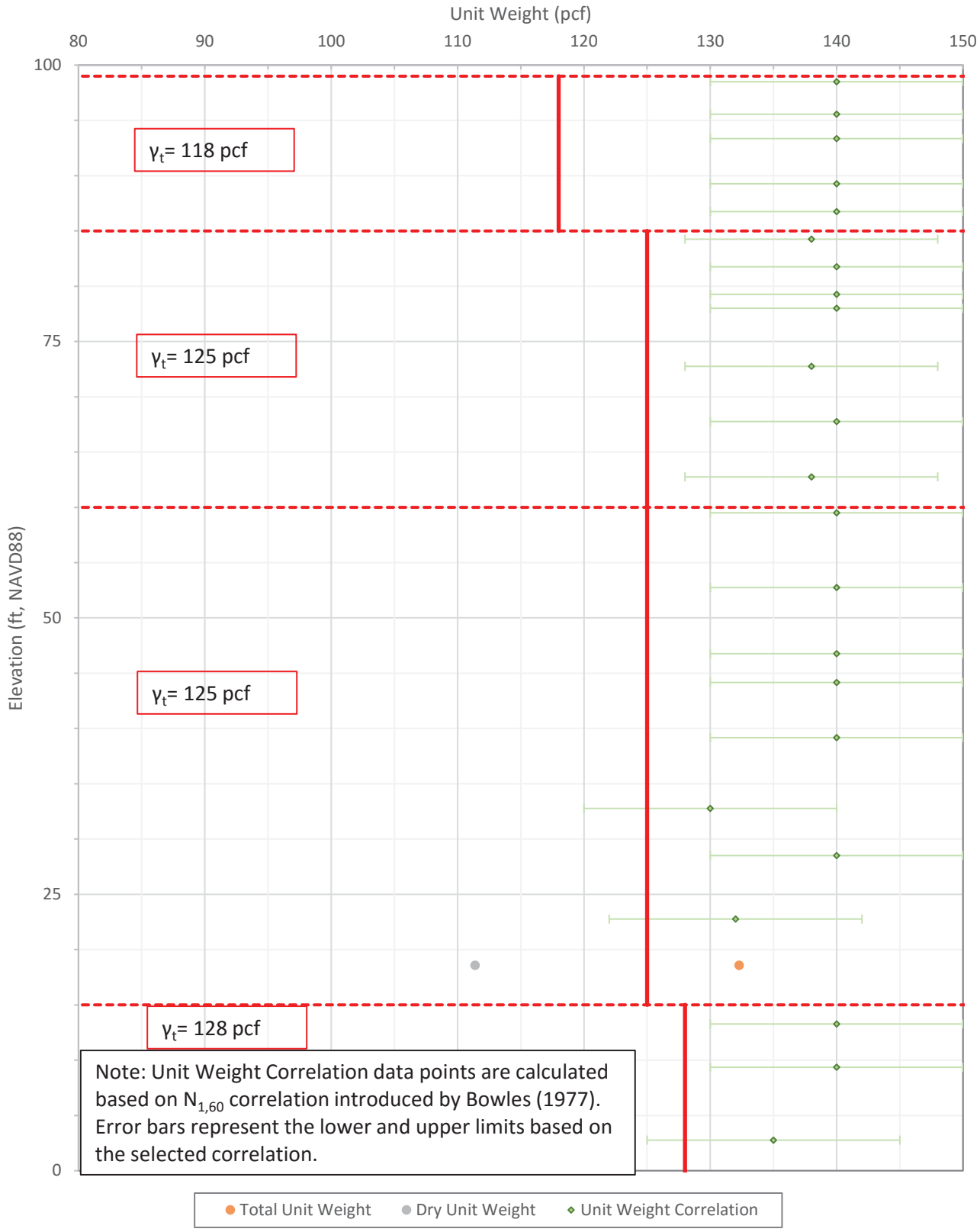


SPT N_{field}, N_{60}, $N_{1,60}$ - AREA 8 (B-183)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

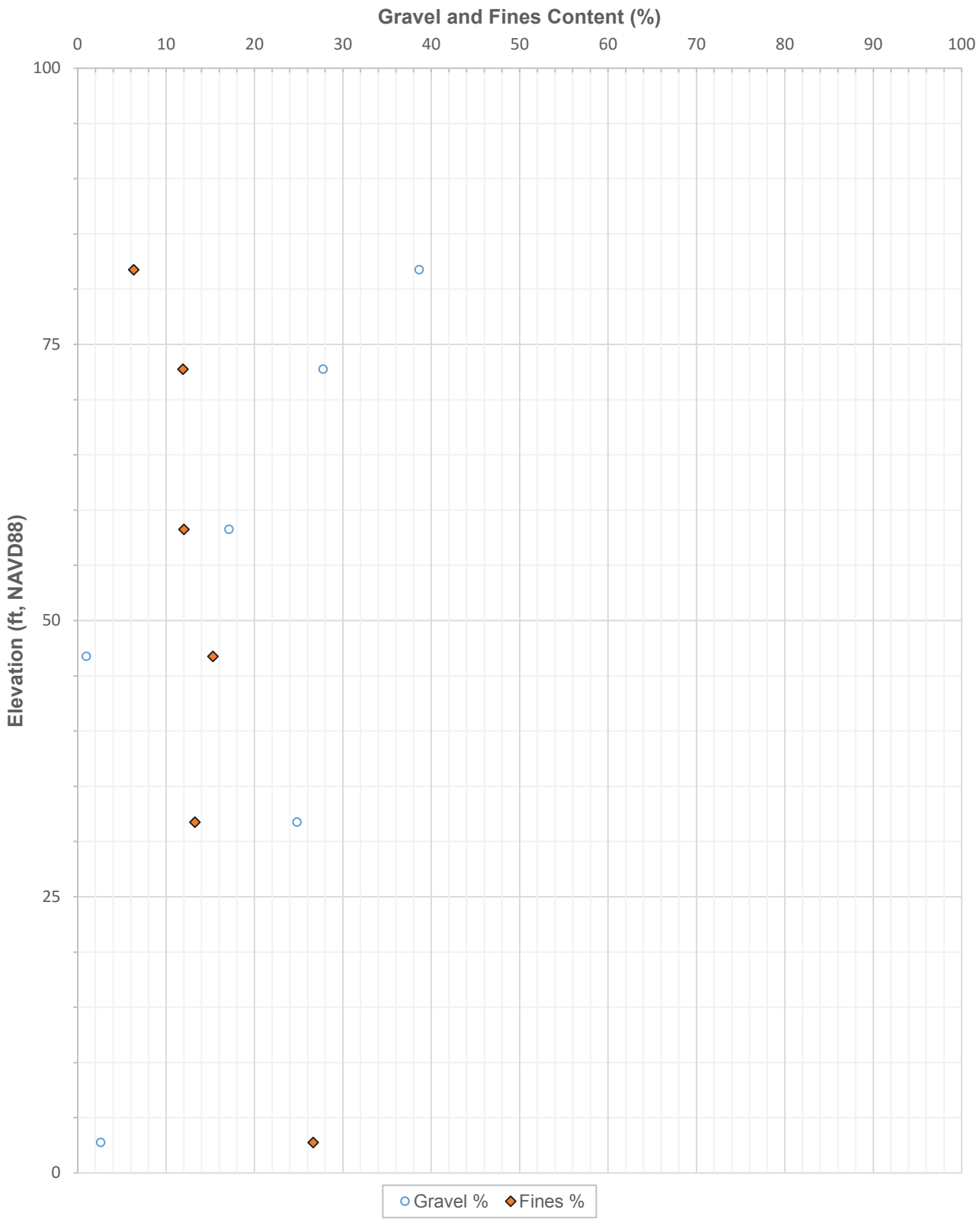


SPT $N_{1,60}$ - AREA 8 (B-183)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 36b



TOTAL AND DRY UNIT WEIGHTS (CLAY) - AREA 8 (B-183)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



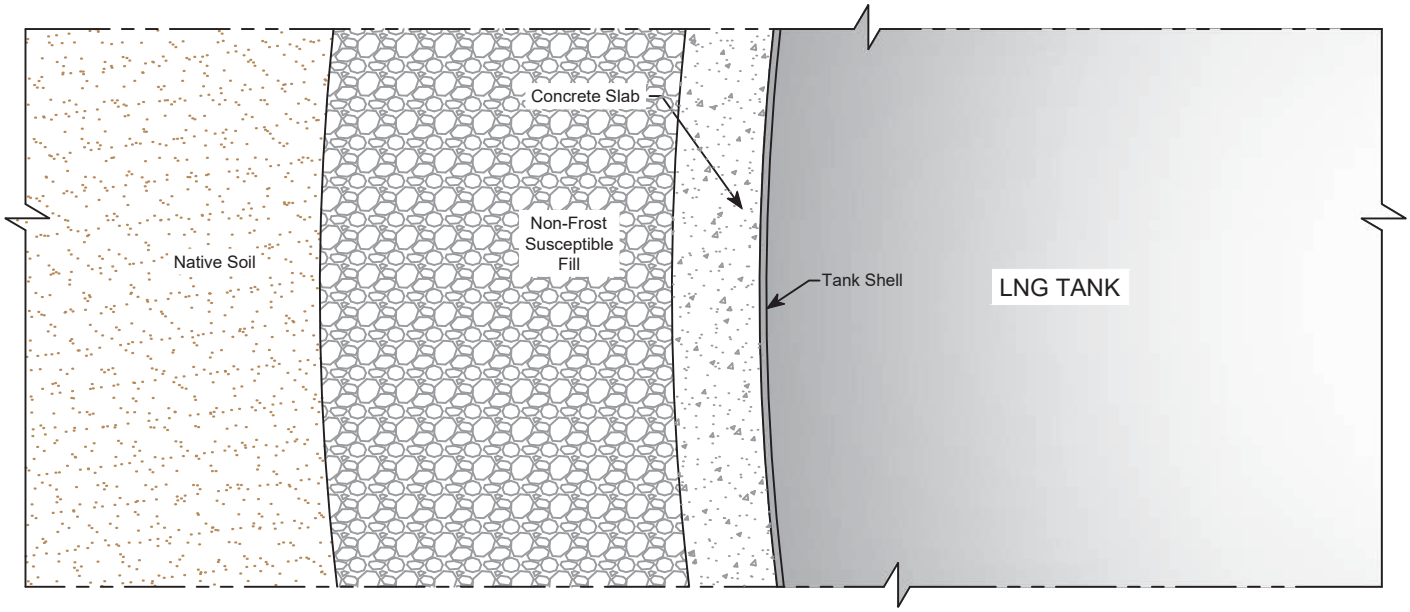
GRAVEL AND FINES CONTENT - AREA 8 (B-183)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

PLATE 36d

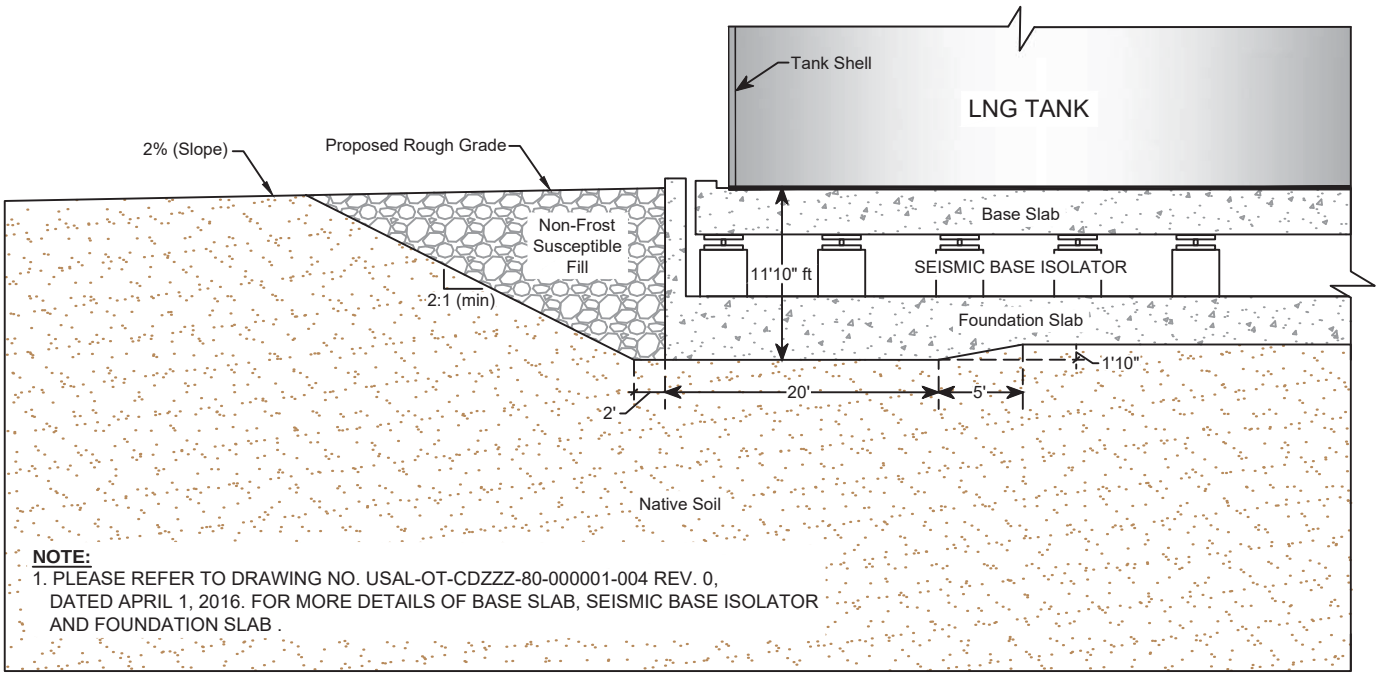
APPENDIX A

LNG Tank Shallow Foundation Recommendations

Schematic View of LNG Tank Foundations A-1
Tank Stability ..... A-2 thru A-9
Tank Settlement ..... A-10 thru A-15



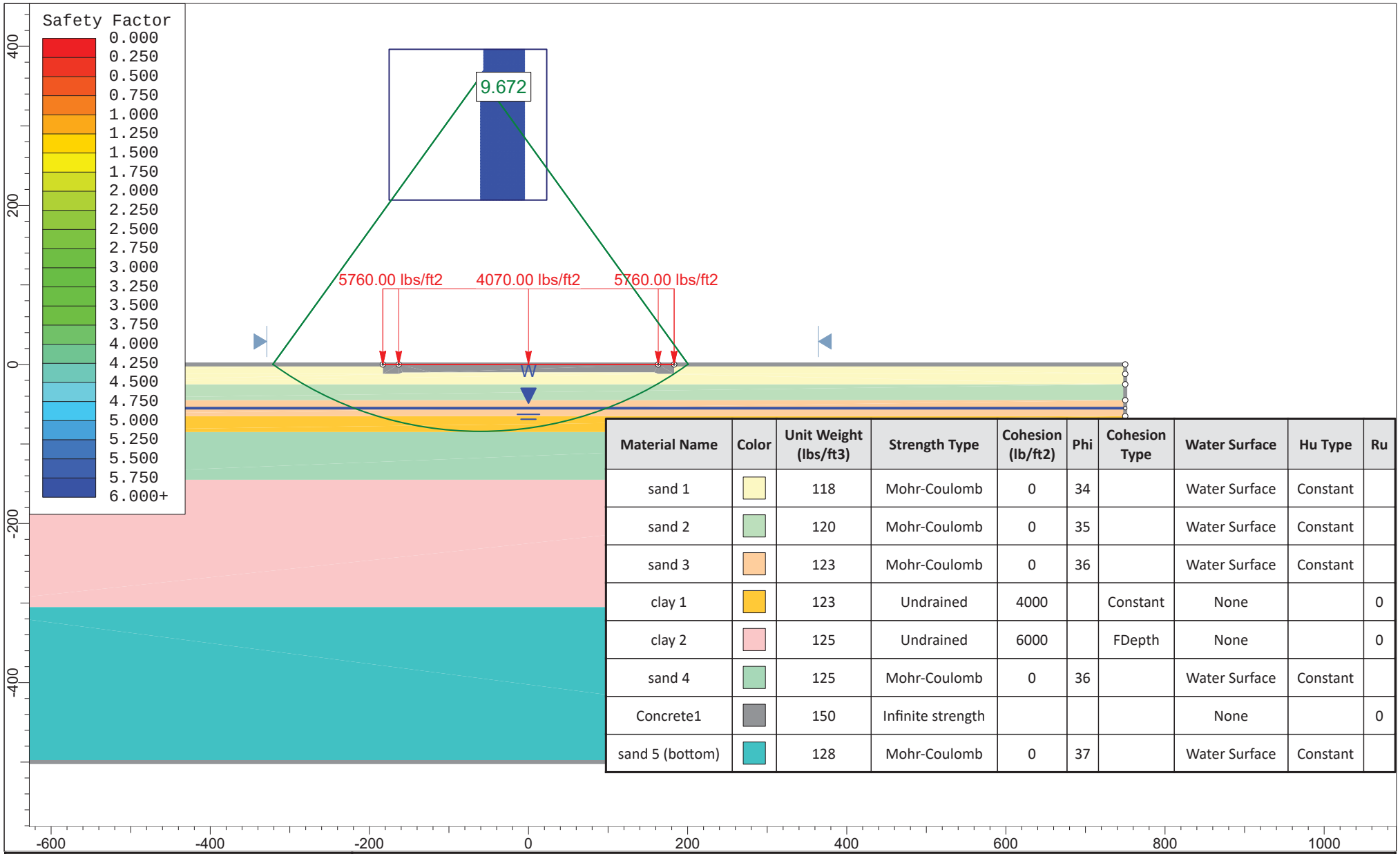
PLAN VIEW OF TANK FOUNDATION



CROSS-SECTION VIEW OF TANK FOUNDATION

SCHEMATIC VIEW OF LNG TANK FOUNDATIONS

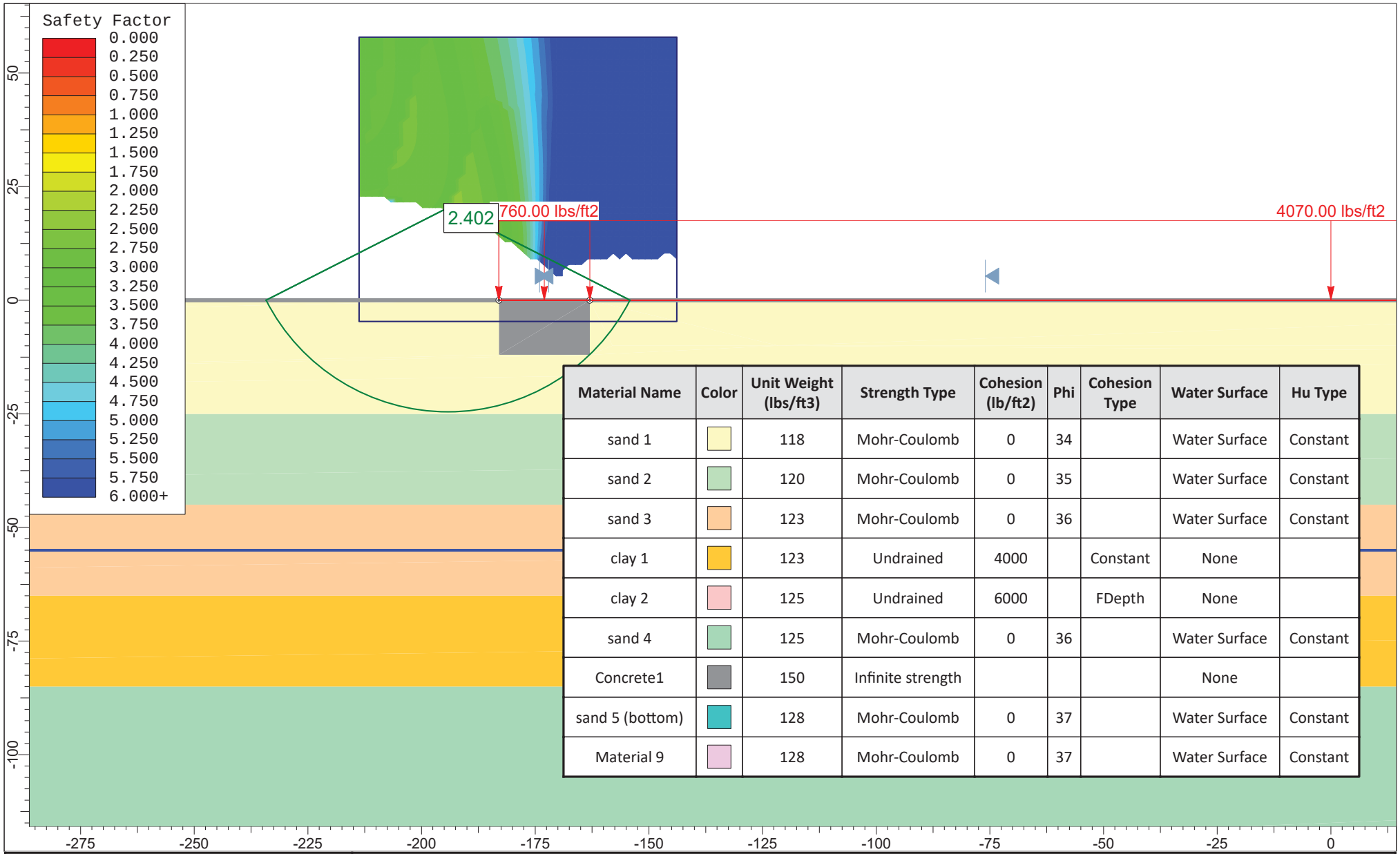
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
sand 1		118	Mohr-Coulomb	0	34		Water Surface	Constant	
sand 2		120	Mohr-Coulomb	0	35		Water Surface	Constant	
sand 3		123	Mohr-Coulomb	0	36		Water Surface	Constant	
clay 1		123	Undrained	4000		Constant	None		0
clay 2		125	Undrained	6000		FDepth	None		0
sand 4		125	Mohr-Coulomb	0	36		Water Surface	Constant	
Concrete1		150	Infinite strength				None		0
sand 5 (bottom)		128	Mohr-Coulomb	0	37		Water Surface	Constant	

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (North) Base Shear Analysis - Hydrotesting Load		
Drawn By	Sochan Jung	Scale 1:2000	Company Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		File Name A-2 Tank-N (366-ft) Base Shear - Hydrotestng (undrained).slim



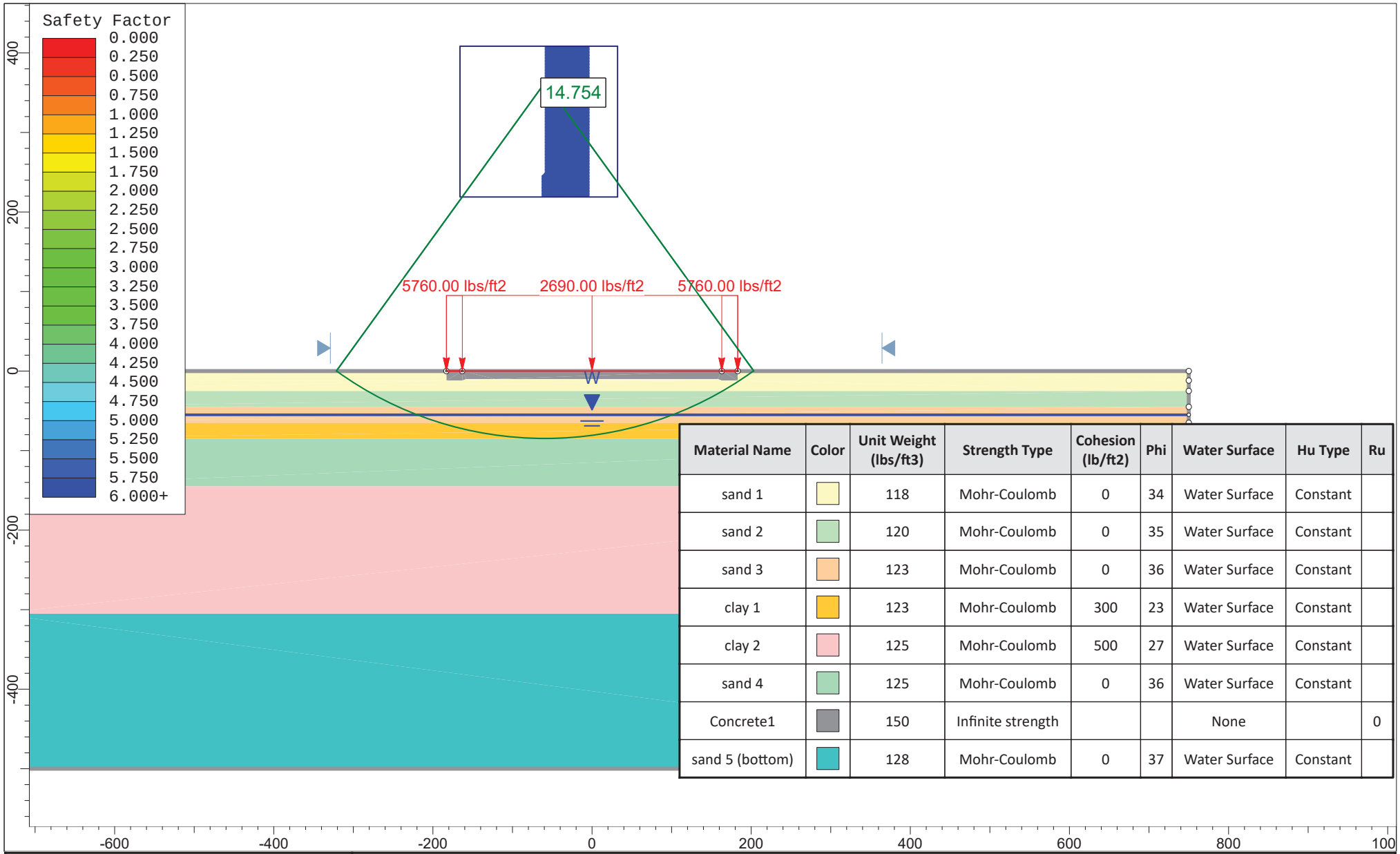
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type
sand 1		118	Mohr-Coulomb	0	34		Water Surface	Constant
sand 2		120	Mohr-Coulomb	0	35		Water Surface	Constant
sand 3		123	Mohr-Coulomb	0	36		Water Surface	Constant
clay 1		123	Undrained	4000		Constant	None	
clay 2		125	Undrained	6000		FDepth	None	
sand 4		125	Mohr-Coulomb	0	36		Water Surface	Constant
Concrete1		150	Infinite strength				None	
sand 5 (bottom)		128	Mohr-Coulomb	0	37		Water Surface	Constant
Material 9		128	Mohr-Coulomb	0	37		Water Surface	Constant

FUGRO

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (North) Edge Shear Analysis - Hydrotesting Load		
Drawn By Sochan Jung	*Scale* 1:350	*Company* Fugro Consultants, Inc.	
Date 11/18/2016, 8:00:57 AM		*File Name* A-3 Tank-N (366-ft) Edge Shear - Hydrotestng (undrained).slim	

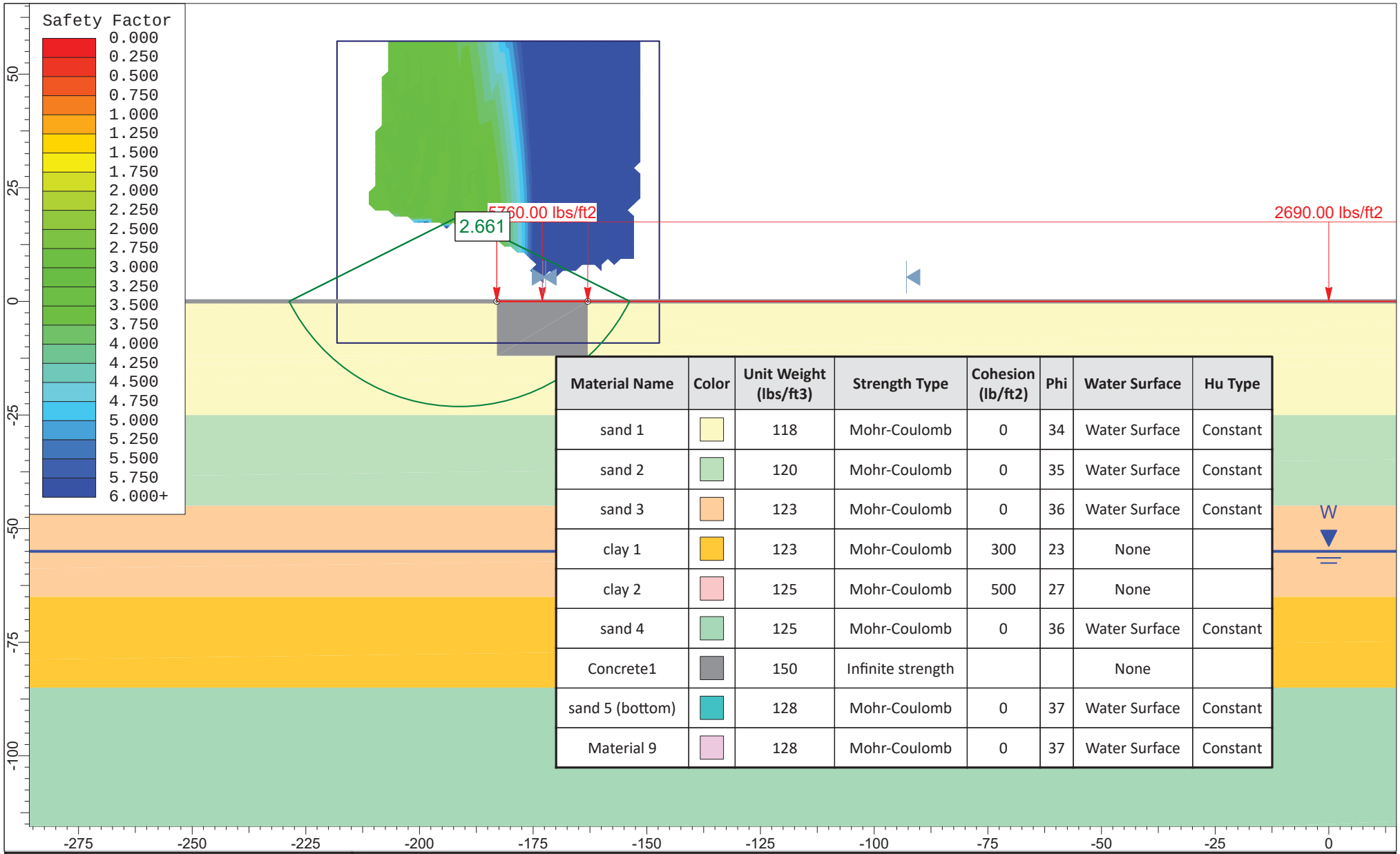
PLATE A-3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
sand 1		118	Mohr-Coulomb	0	34	Water Surface	Constant	
sand 2		120	Mohr-Coulomb	0	35	Water Surface	Constant	
sand 3		123	Mohr-Coulomb	0	36	Water Surface	Constant	
clay 1		123	Mohr-Coulomb	300	23	Water Surface	Constant	
clay 2		125	Mohr-Coulomb	500	27	Water Surface	Constant	
sand 4		125	Mohr-Coulomb	0	36	Water Surface	Constant	
Concrete1		150	Infinite strength			None		0
sand 5 (bottom)		128	Mohr-Coulomb	0	37	Water Surface	Constant	

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (North) Base Shear Analysis - Operating Load		
Drawn By	Sochan Jung	Scale 1:2000	Company Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		File Name A-4 Tank-N (366-ft) Base Shear - Operating (drained).slim

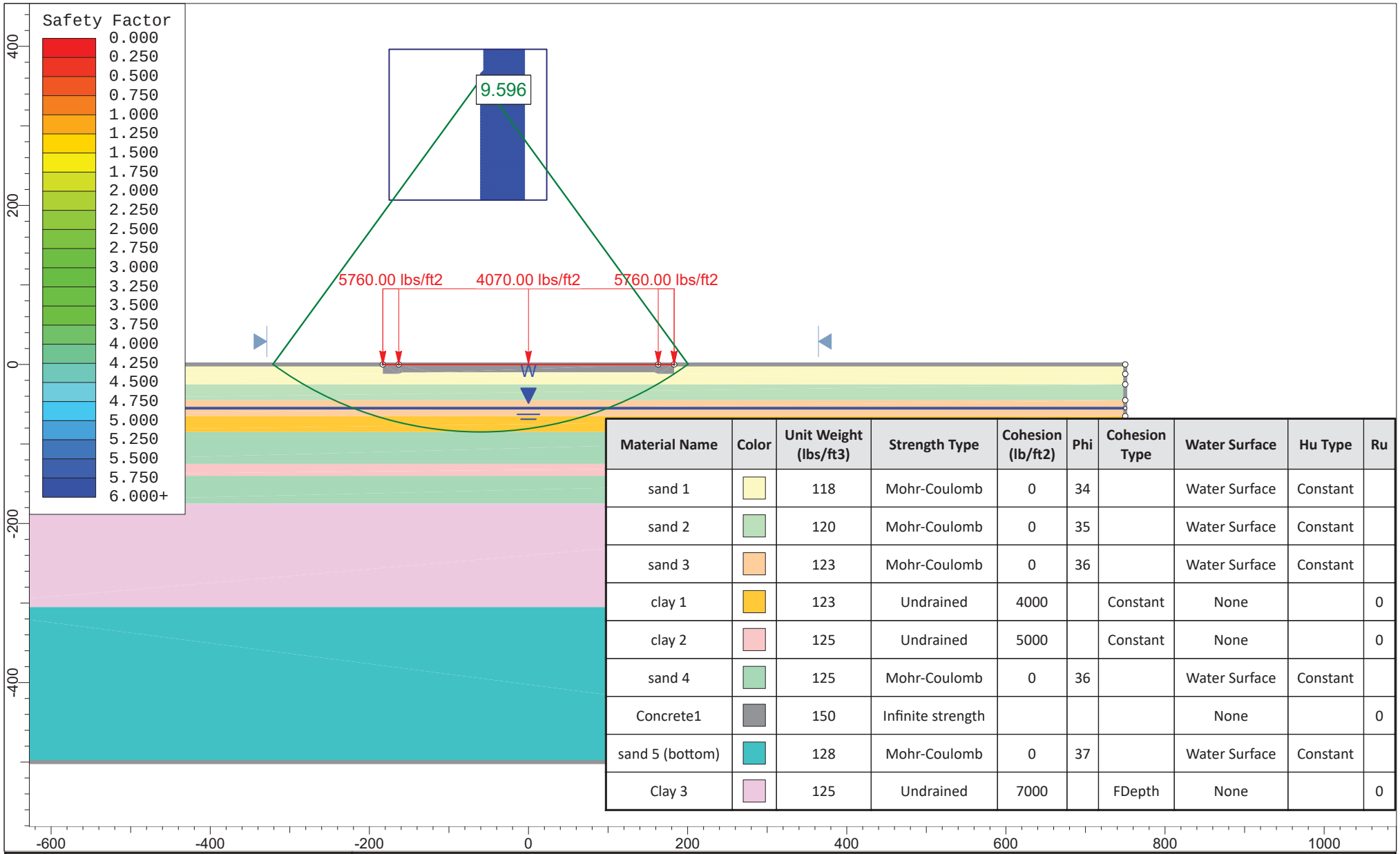


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
sand 1		118	Mohr-Coulomb	0	34	Water Surface	Constant
sand 2		120	Mohr-Coulomb	0	35	Water Surface	Constant
sand 3		123	Mohr-Coulomb	0	36	Water Surface	Constant
clay 1		123	Mohr-Coulomb	300	23	None	
clay 2		125	Mohr-Coulomb	500	27	None	
sand 4		125	Mohr-Coulomb	0	36	Water Surface	Constant
Concrete1		150	Infinite strength			None	
sand 5 (bottom)		128	Mohr-Coulomb	0	37	Water Surface	Constant
Material 9		128	Mohr-Coulomb	0	37	Water Surface	Constant

FUGRO

SLIDEINTERPRET 6.012

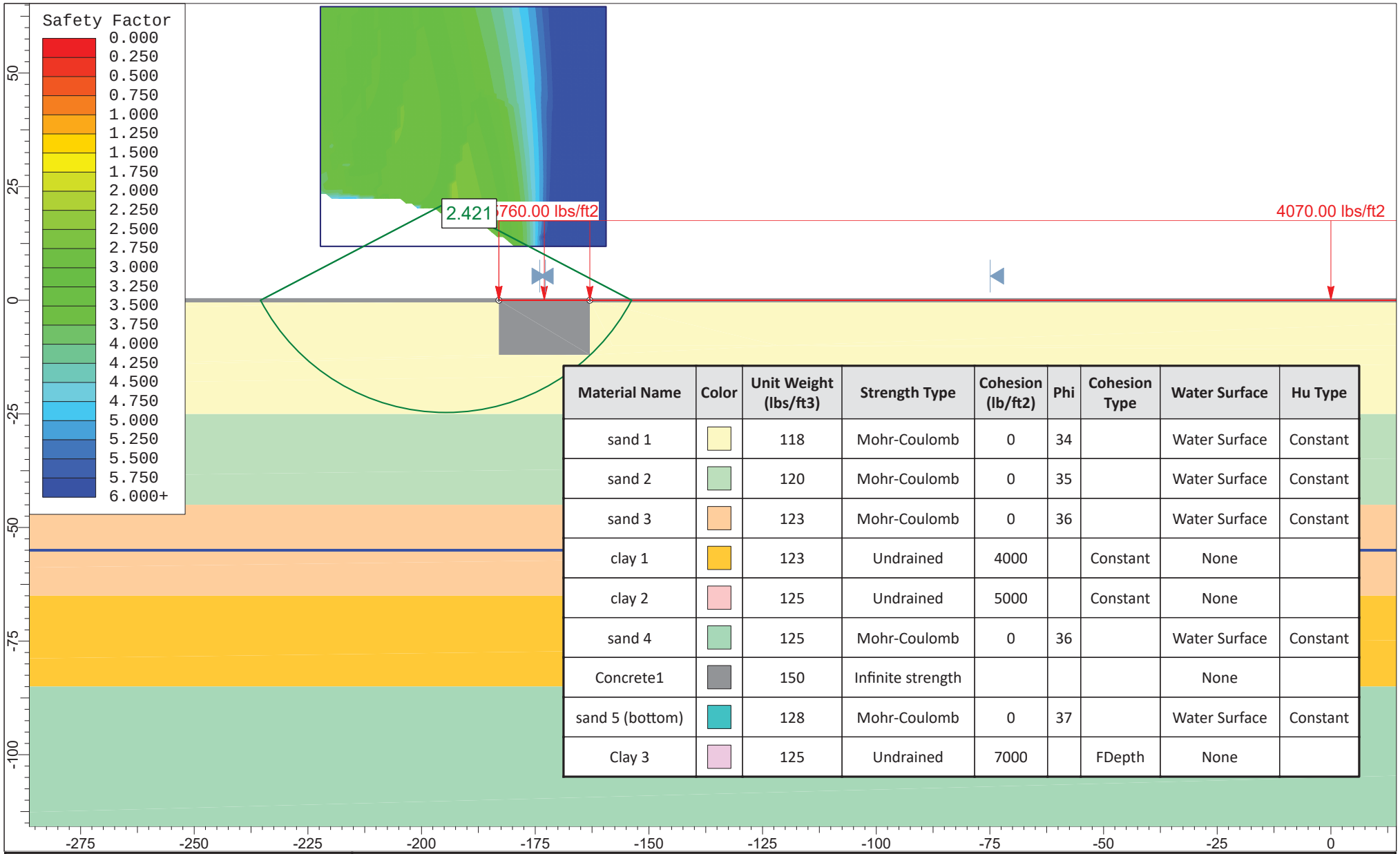
Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (North) Edge Shear Analysis - Operating Load		
Drawn By	Sochan Jung	*Scale* 1:350	*Company* Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		*File Name* A-5 Tank-N (366-ft) Edge Shear - Operating (drained).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
sand 1		118	Mohr-Coulomb	0	34		Water Surface	Constant	
sand 2		120	Mohr-Coulomb	0	35		Water Surface	Constant	
sand 3		123	Mohr-Coulomb	0	36		Water Surface	Constant	
clay 1		123	Undrained	4000		Constant	None		0
clay 2		125	Undrained	5000		Constant	None		0
sand 4		125	Mohr-Coulomb	0	36		Water Surface	Constant	
Concrete1		150	Infinite strength				None		0
sand 5 (bottom)		128	Mohr-Coulomb	0	37		Water Surface	Constant	
Clay 3		125	Undrained	7000		FDepth	None		0

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (South) Base Shear Analysis - Hydrotesting Load		
Drawn By	Sochan Jung	*Scale* 1:2000	*Company* Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		*File Name* A-6 Tank-S (366-ft) Base Shear - Hydrotestng (undrained).slim

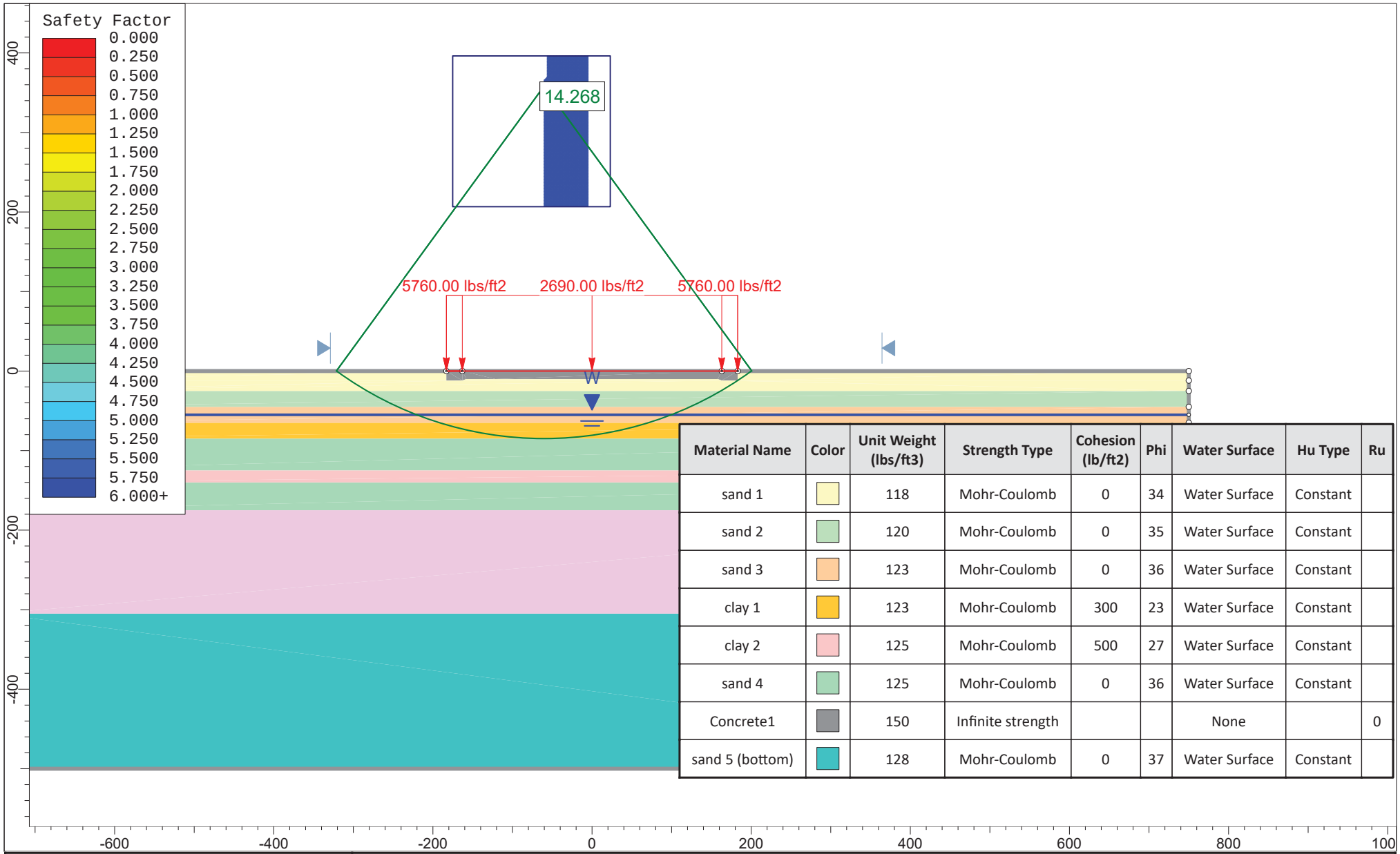


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type
sand 1		118	Mohr-Coulomb	0	34		Water Surface	Constant
sand 2		120	Mohr-Coulomb	0	35		Water Surface	Constant
sand 3		123	Mohr-Coulomb	0	36		Water Surface	Constant
clay 1		123	Undrained	4000		Constant	None	
clay 2		125	Undrained	5000		Constant	None	
sand 4		125	Mohr-Coulomb	0	36		Water Surface	Constant
Concrete1		150	Infinite strength				None	
sand 5 (bottom)		128	Mohr-Coulomb	0	37		Water Surface	Constant
Clay 3		125	Undrained	7000		FDepth	None	

FUGRO

SLIDEINTERPRET 6.012

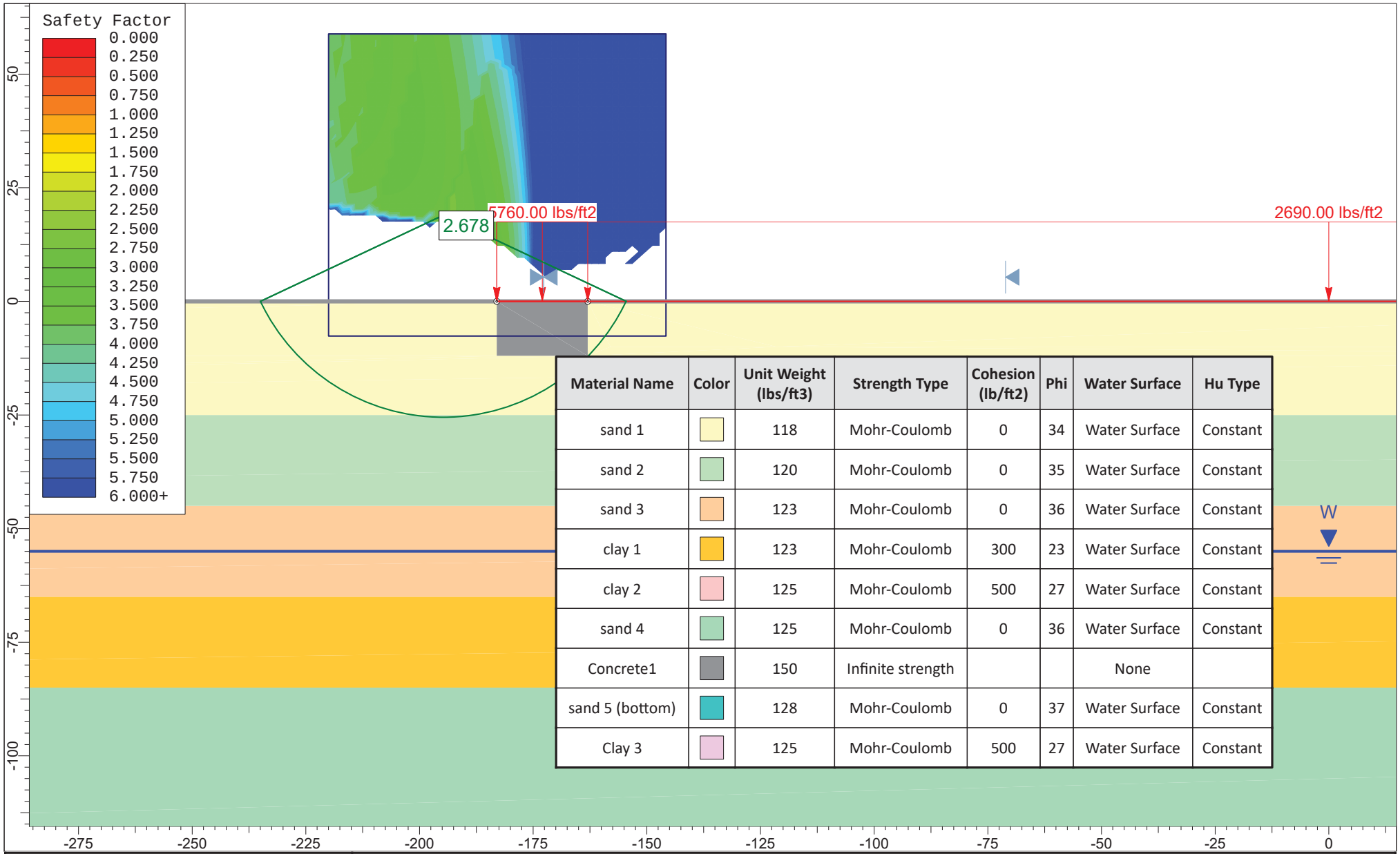
Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (South) Edge Shear Analysis - Hydrotesting Load		
Drawn By	Sochan Jung	Scale 1:350	Company Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		File Name A-7 Tank-S (366-ft) Edge Shear - Hydrotesting (undrained).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type	Ru
sand 1		118	Mohr-Coulomb	0	34	Water Surface	Constant	
sand 2		120	Mohr-Coulomb	0	35	Water Surface	Constant	
sand 3		123	Mohr-Coulomb	0	36	Water Surface	Constant	
clay 1		123	Mohr-Coulomb	300	23	Water Surface	Constant	
clay 2		125	Mohr-Coulomb	500	27	Water Surface	Constant	
sand 4		125	Mohr-Coulomb	0	36	Water Surface	Constant	
Concrete1		150	Infinite strength			None		0
sand 5 (bottom)		128	Mohr-Coulomb	0	37	Water Surface	Constant	

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (South) Base Shear Analysis - Operating Load		
Drawn By	Sochan Jung	Scale 1:2000	Company Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		File Name A-8 Tank-S (366-ft) Base Shear - Operating (drained).slim

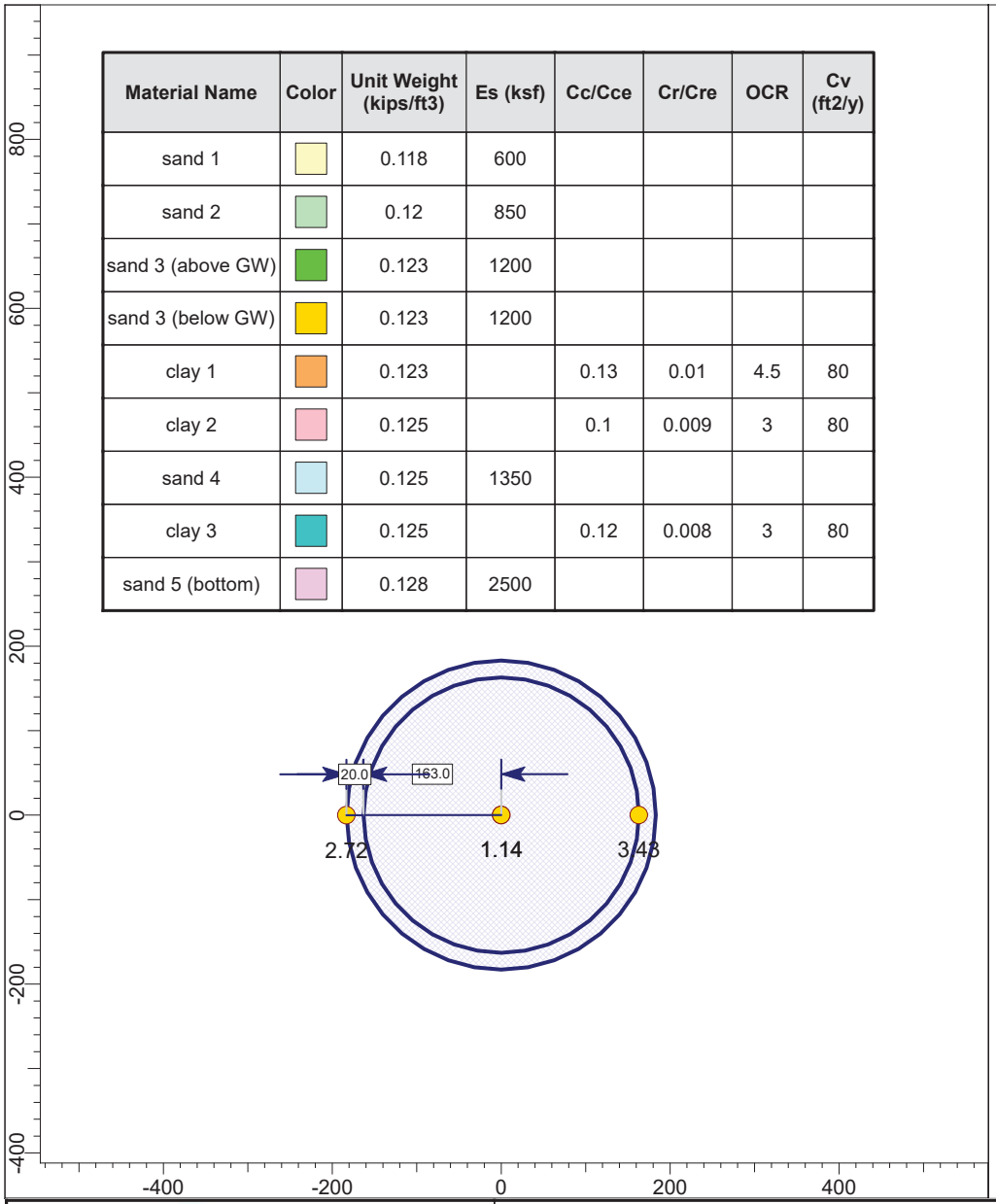


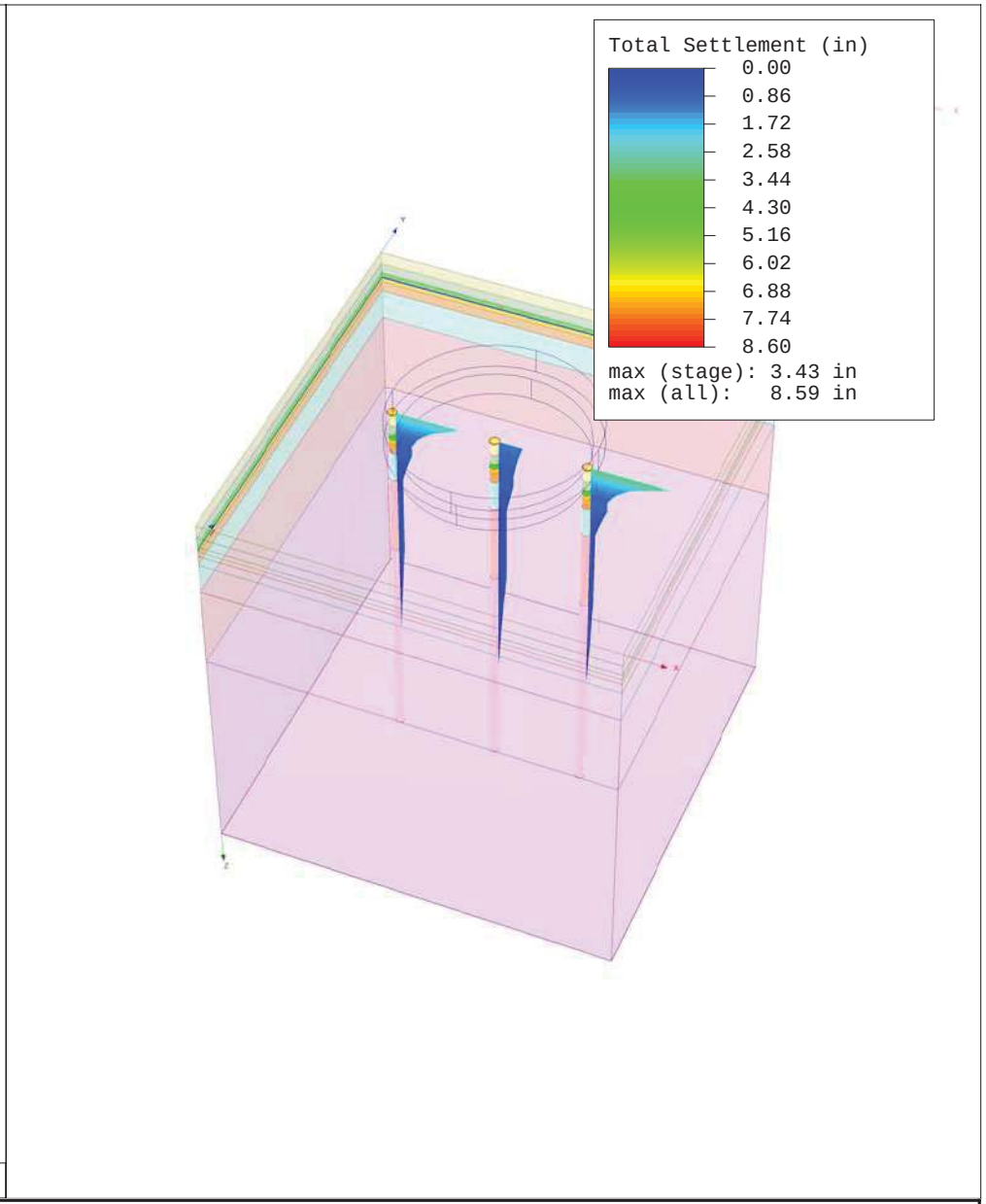
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Hu Type
sand 1		118	Mohr-Coulomb	0	34	Water Surface	Constant
sand 2		120	Mohr-Coulomb	0	35	Water Surface	Constant
sand 3		123	Mohr-Coulomb	0	36	Water Surface	Constant
clay 1		123	Mohr-Coulomb	300	23	Water Surface	Constant
clay 2		125	Mohr-Coulomb	500	27	Water Surface	Constant
sand 4		125	Mohr-Coulomb	0	36	Water Surface	Constant
Concrete1		150	Infinite strength			None	
sand 5 (bottom)		128	Mohr-Coulomb	0	37	Water Surface	Constant
Clay 3		125	Mohr-Coulomb	500	27	Water Surface	Constant

SLIDEINTERPRET 6.012

Project	Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
Analysis Description	Tank (South) Edge Shear Analysis - Operating Load		
Drawn By	Sochan Jung	*Scale* 1:350	*Company* Fugro Consultants, Inc.
Date	11/18/2016, 8:00:57 AM		*File Name* A-9 Tank-S (366-ft) Edge Shear - Operating (drained).slim

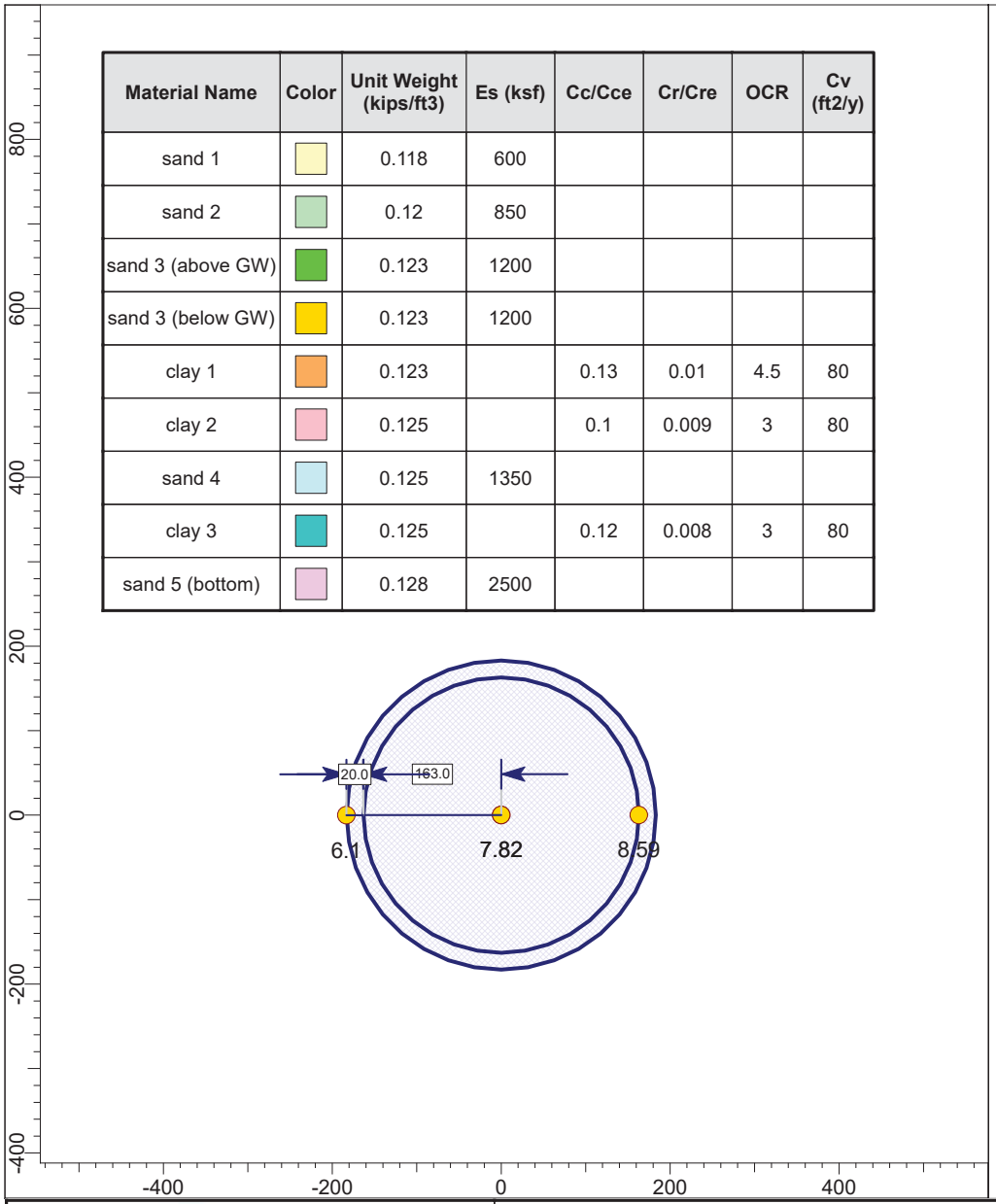
Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.12	0.008	3	80
sand 5 (bottom)		0.128	2500				

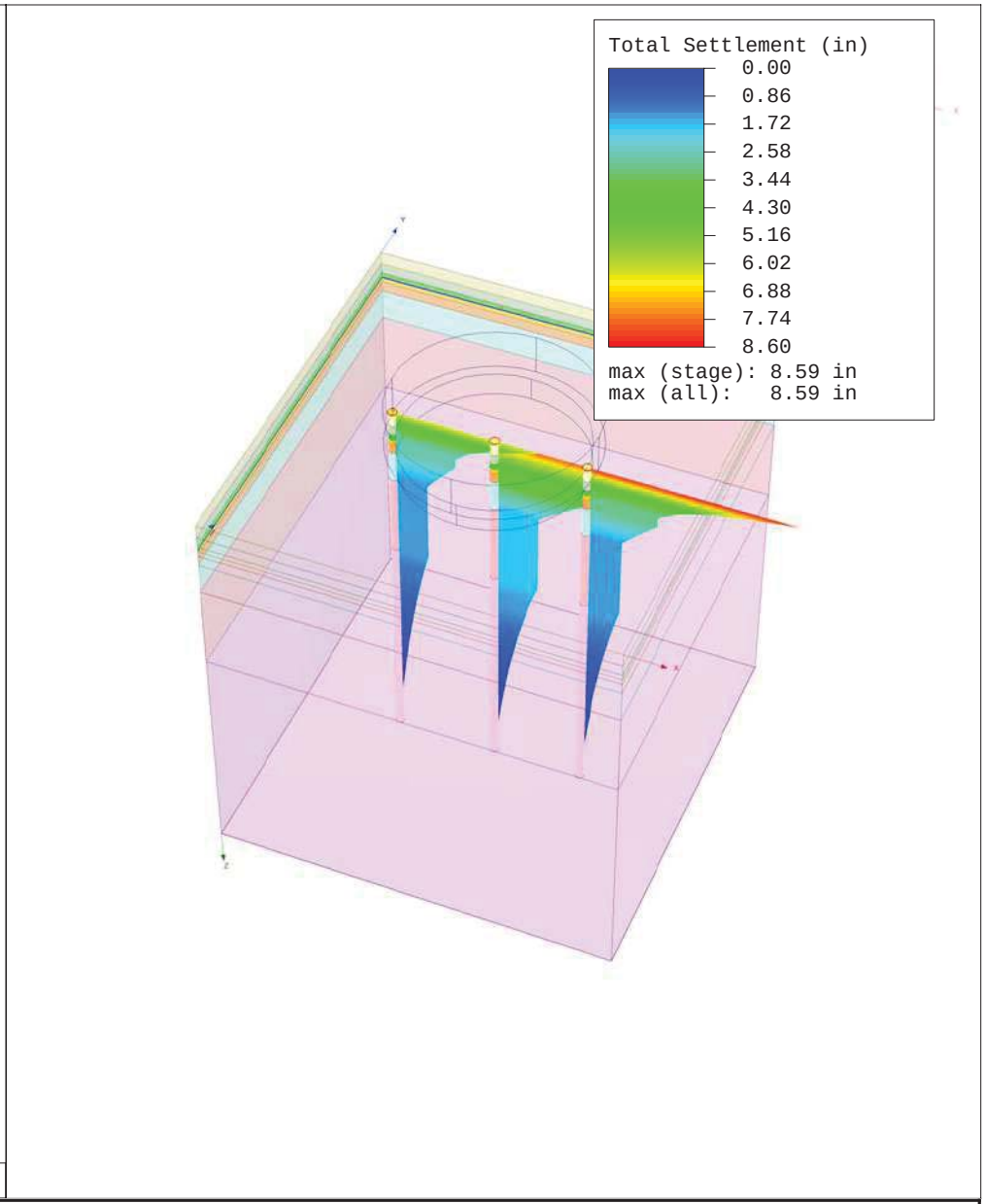




SETTLE3D 3.017	Project: Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
	Analysis Description: Tank (North) Settlement - End of Construction Load		
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
	Date: 11/21/2016	File Name: A-10 Tank (366-ft_North) 1st stage load.s3z	

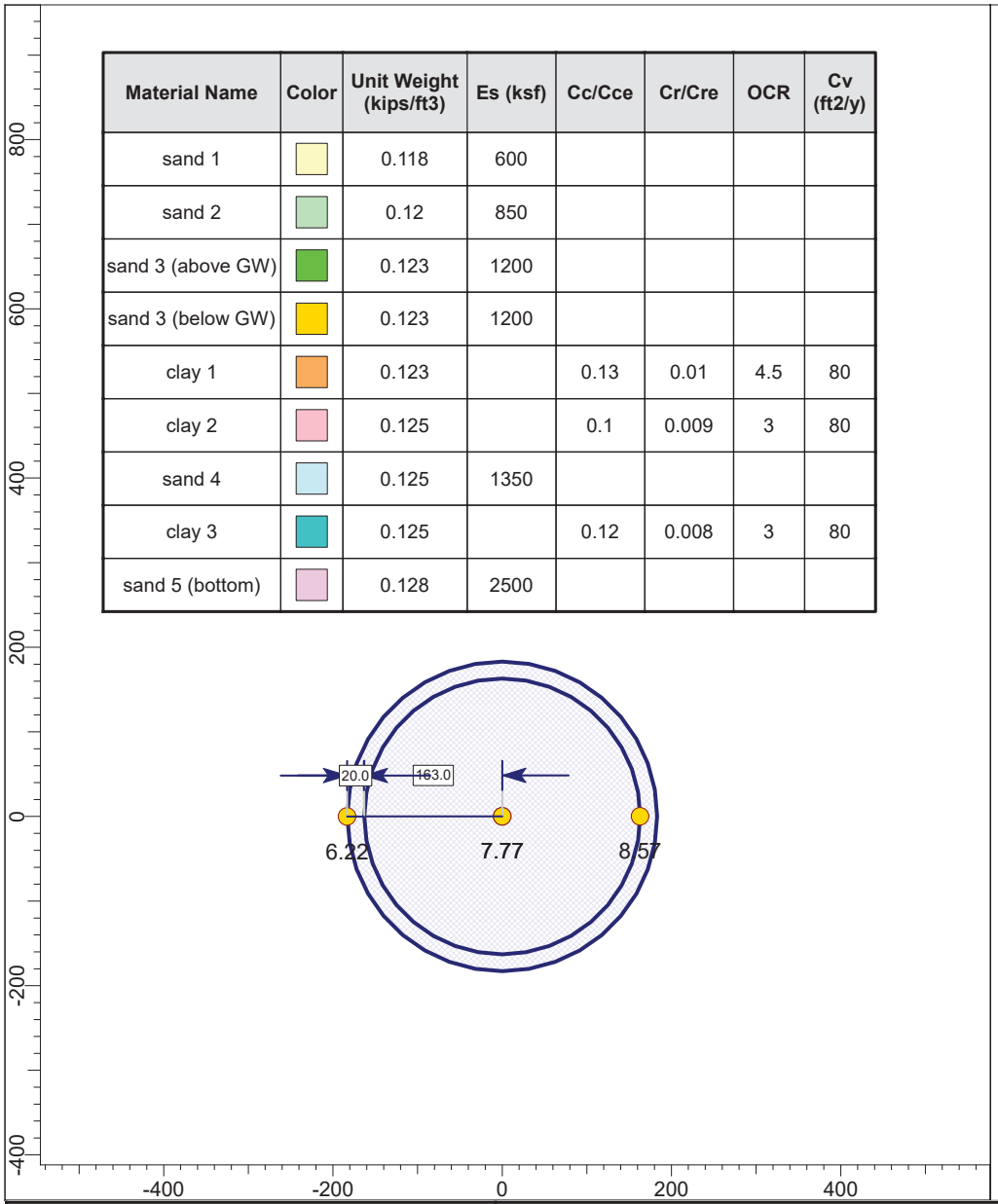
Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.12	0.008	3	80
sand 5 (bottom)		0.128	2500				

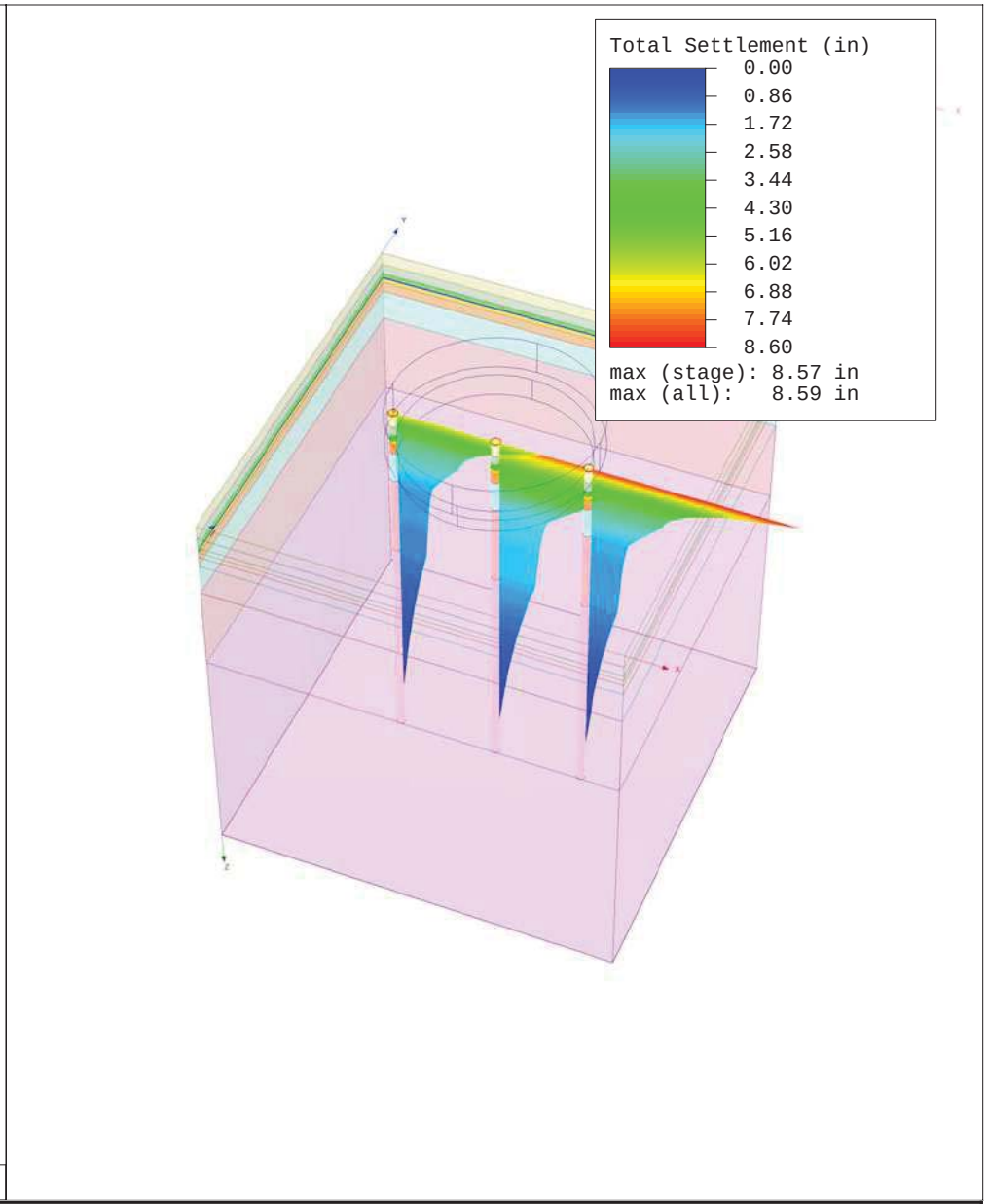




SETTLE3D 3.017	Project: Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
	Analysis Description: Tank (North) Settlement - Hydrotesting Load (15 Days)		
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
	Date: 11/21/2016	File Name: A-11 Tank (366-ft_North) 2nd stage load.s3z	

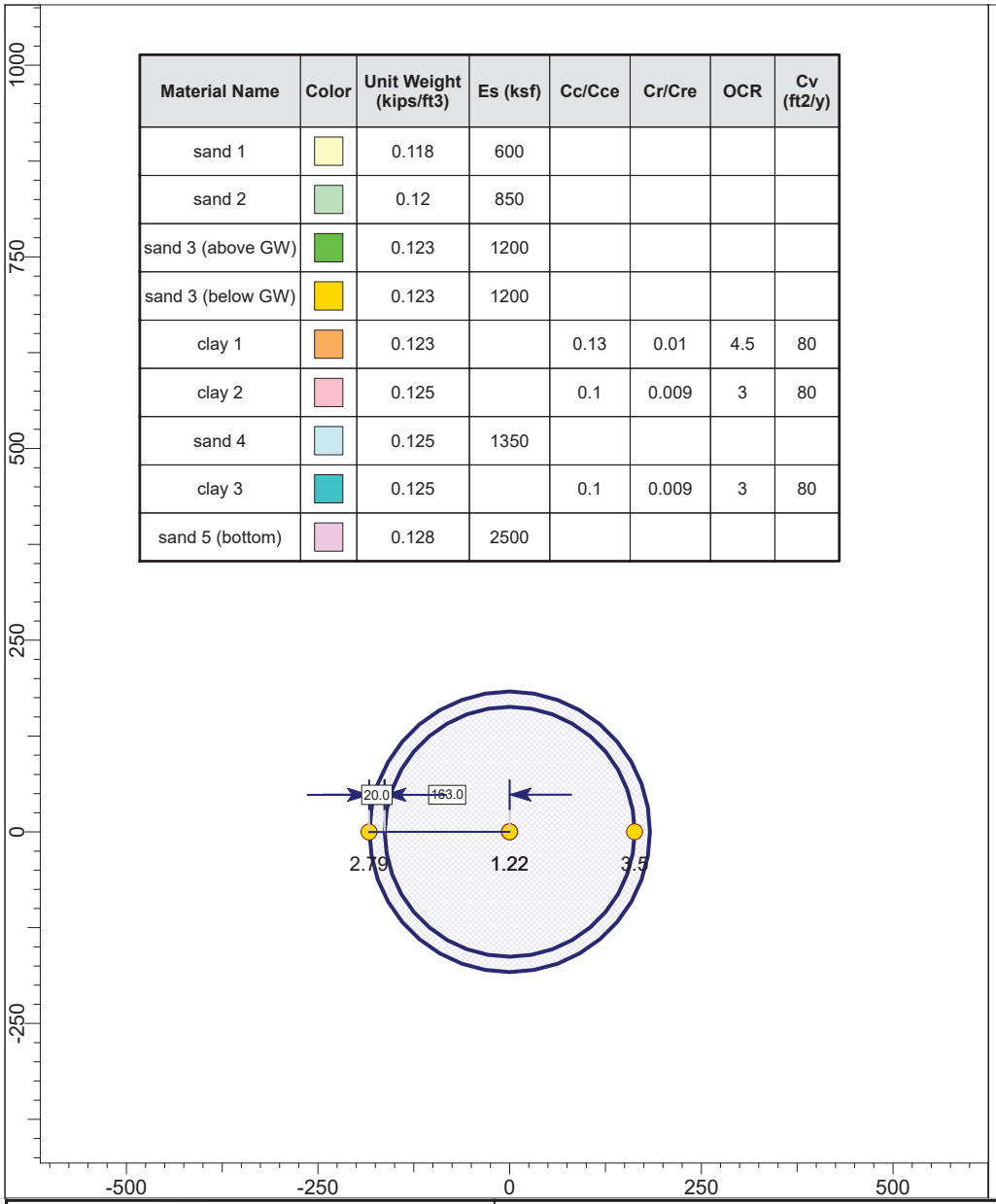
Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.12	0.008	3	80
sand 5 (bottom)		0.128	2500				

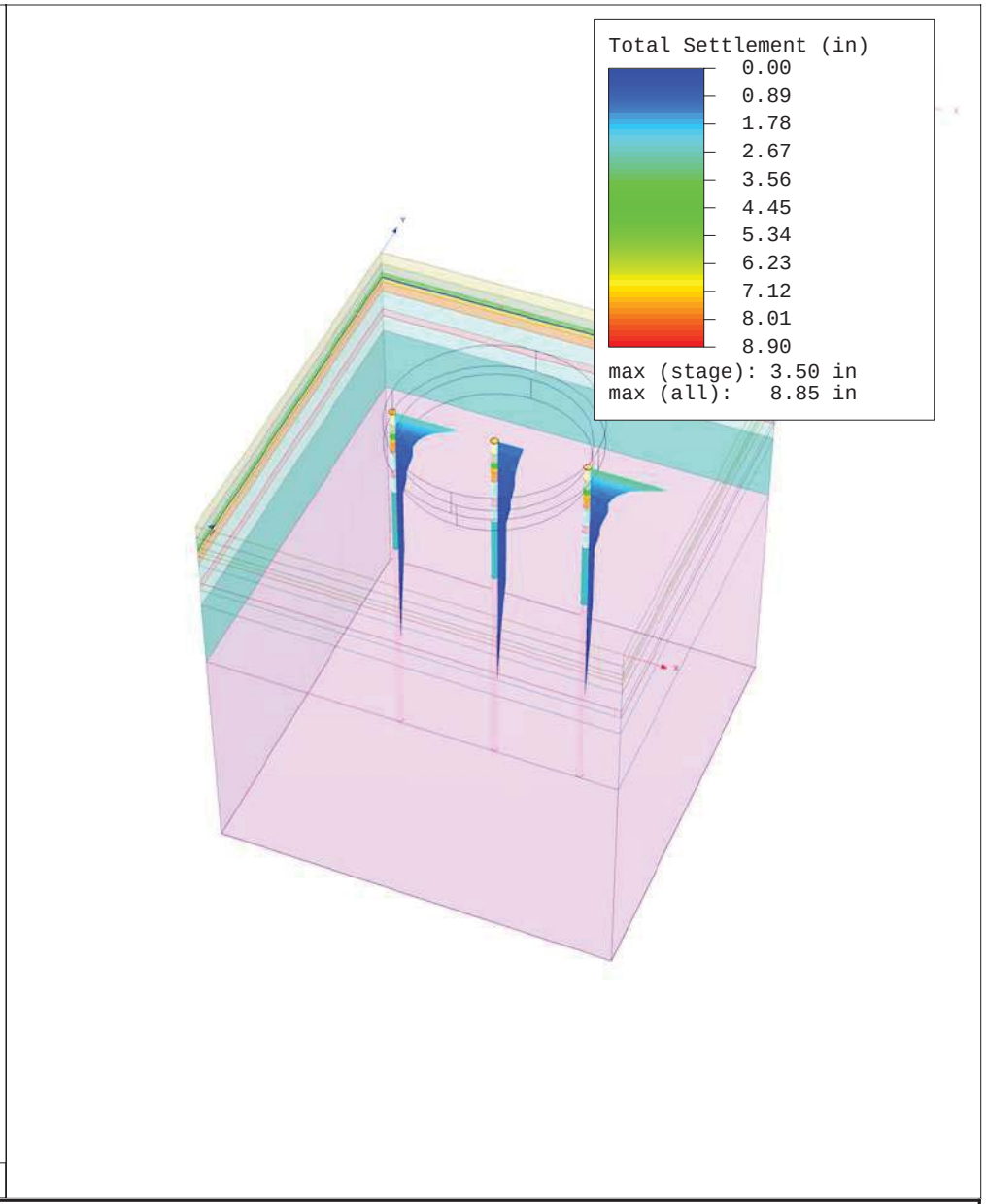




FUGRO	Project: Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
	Analysis Description: Tank (North) Settlement - Operating Load (30 Years)		
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
SETTLE3D 3.017	Date: 11/21/2016	File Name: A-12 Tank (366-ft_North) 3rd stage load.s3z	

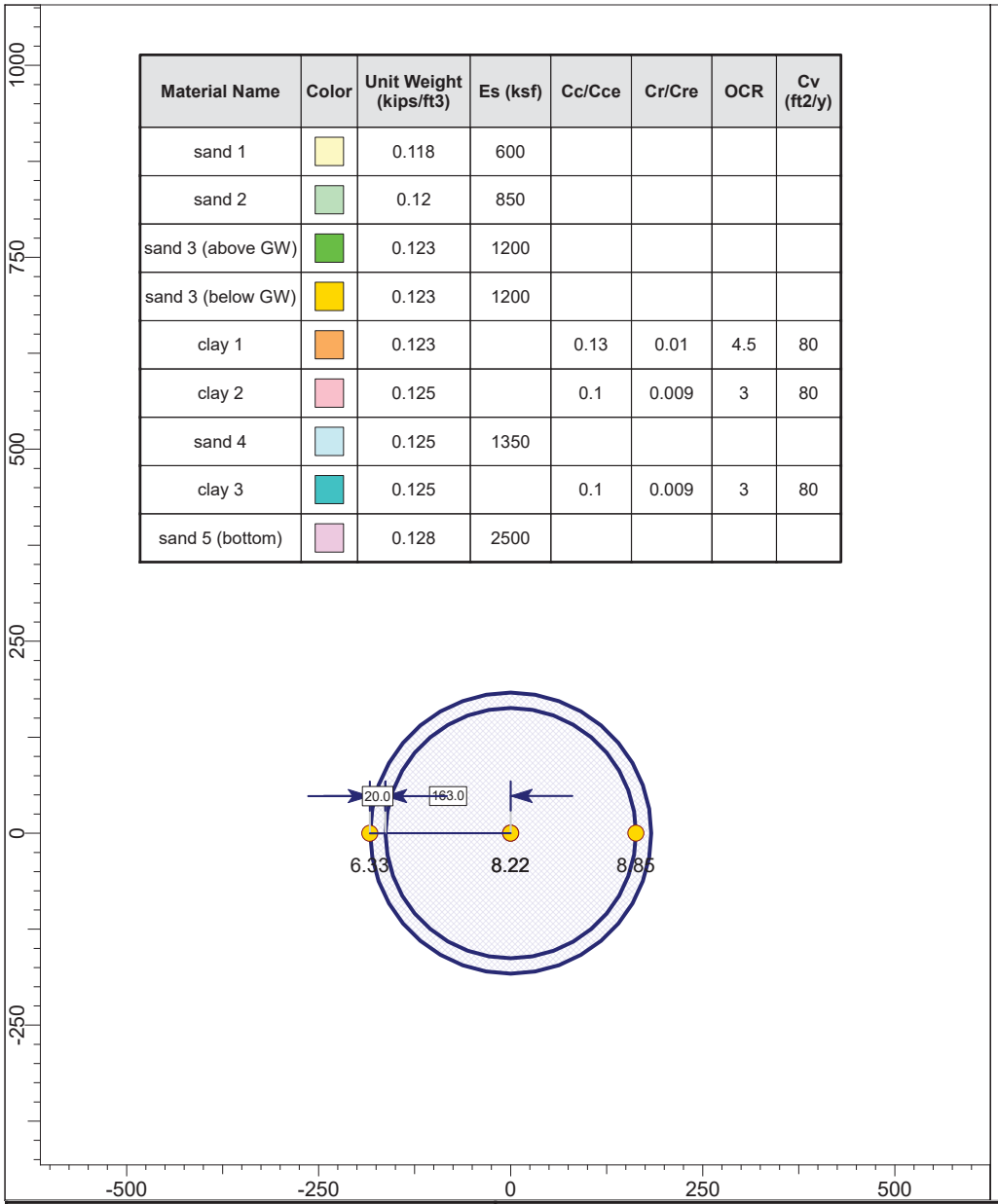
Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.1	0.009	3	80
sand 5 (bottom)		0.128	2500				

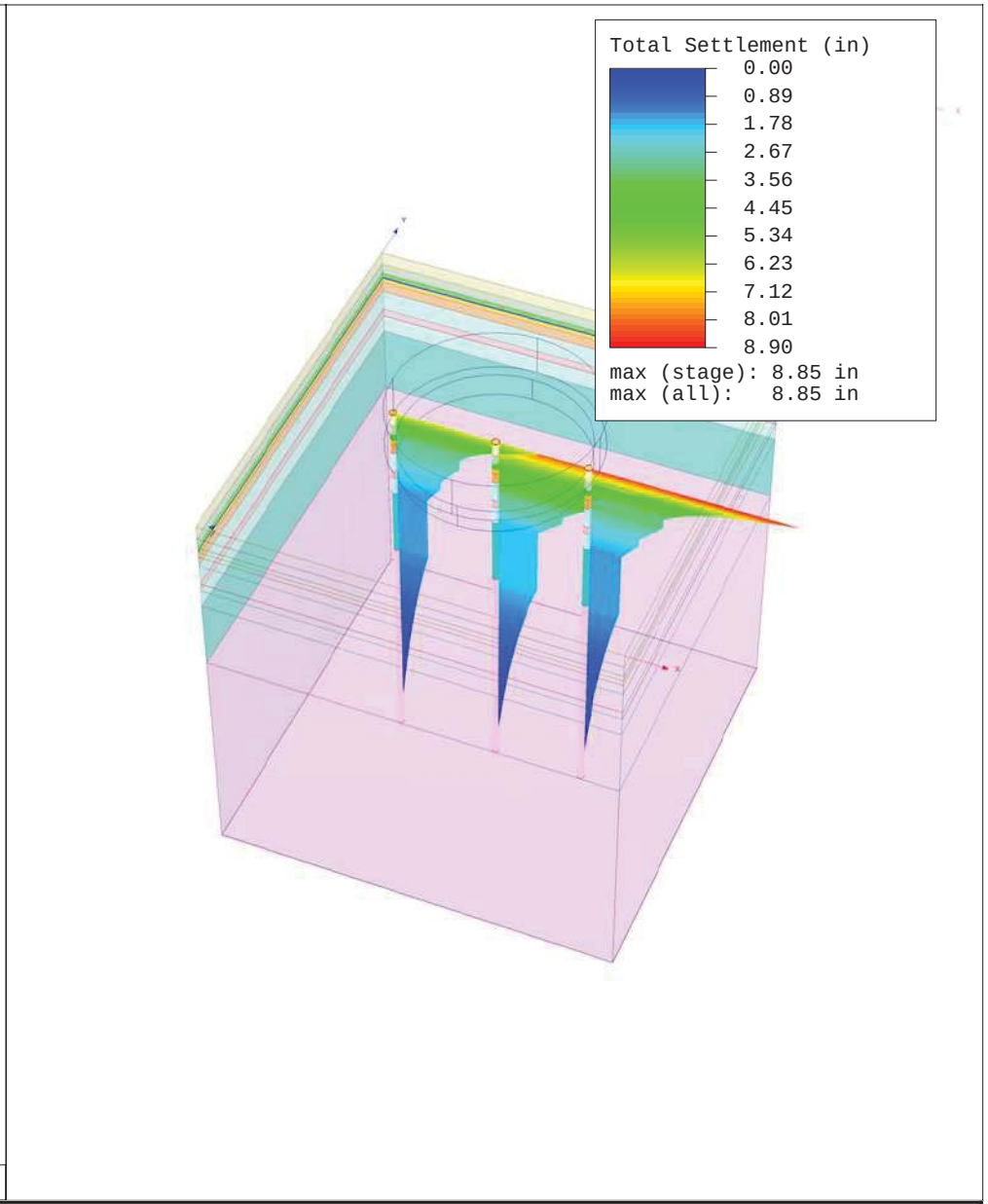




FUGRO	Project: Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
	Analysis Description: Tank (South) Settlement - End of Construction Load		
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
SETTLE3D 3.017	Date: 11/21/2016	FileName: A-13 Tank (366-ft_South) 1st stage load.s3z	

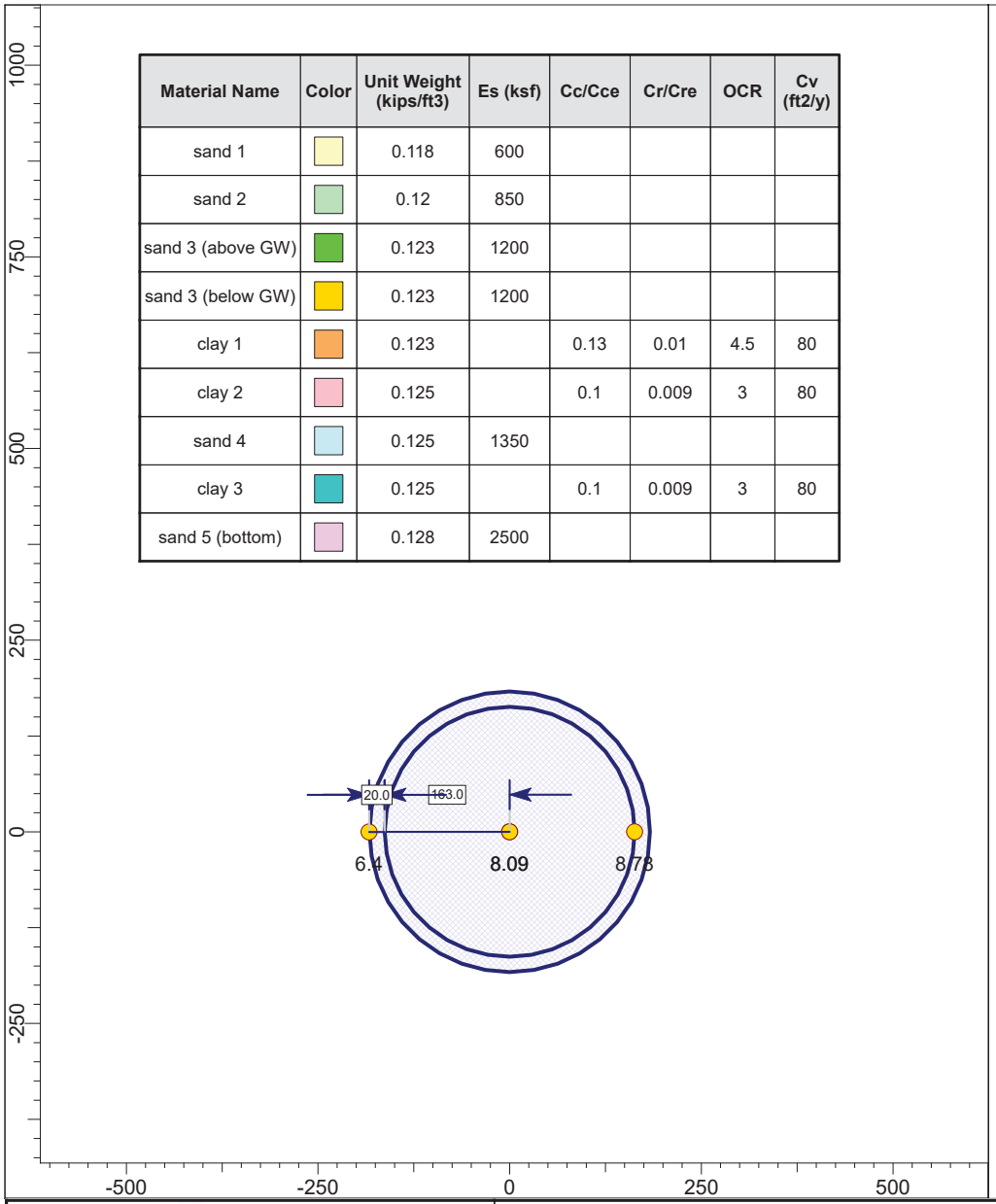
Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.1	0.009	3	80
sand 5 (bottom)		0.128	2500				

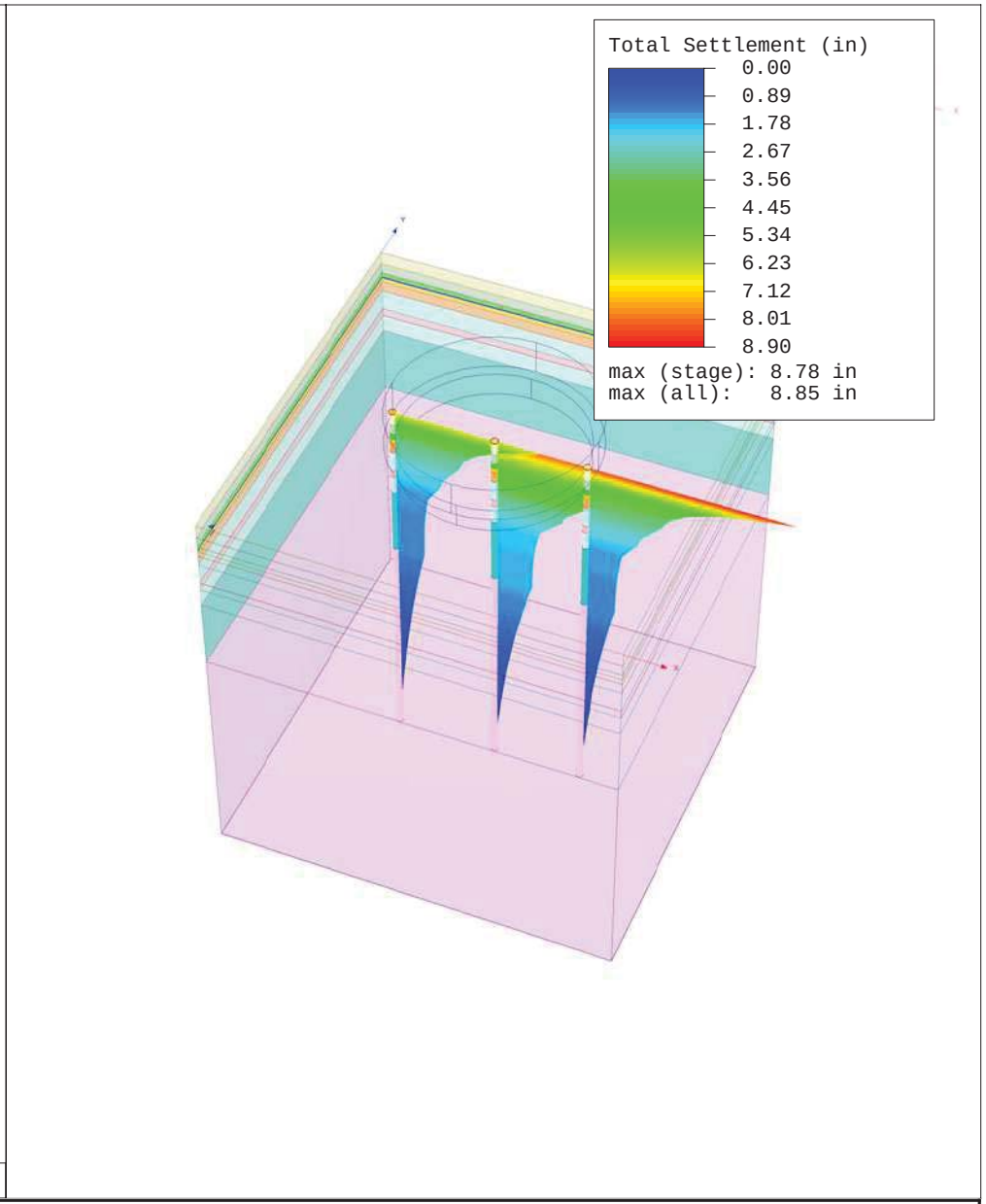




FUGRO	Project: Report No. 04.10160001-12 Alaska LNG Project - Onshore LNG Facilities		
	Analysis Description: Tank (South) Settlement - Hydrotesting Load (15 Days)		
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
SETTLE3D 3.017	Date: 11/21/2016	File Name: A-14 Tank (366-ft_South) 2nd stage load.s3z	

Material Name	Color	Unit Weight (kips/ft3)	Es (ksf)	Cc/Cce	Cr/Cre	OCR	Cv (ft2/y)
sand 1		0.118	600				
sand 2		0.12	850				
sand 3 (above GW)		0.123	1200				
sand 3 (below GW)		0.123	1200				
clay 1		0.123		0.13	0.01	4.5	80
clay 2		0.125		0.1	0.009	3	80
sand 4		0.125	1350				
clay 3		0.125		0.1	0.009	3	80
sand 5 (bottom)		0.128	2500				



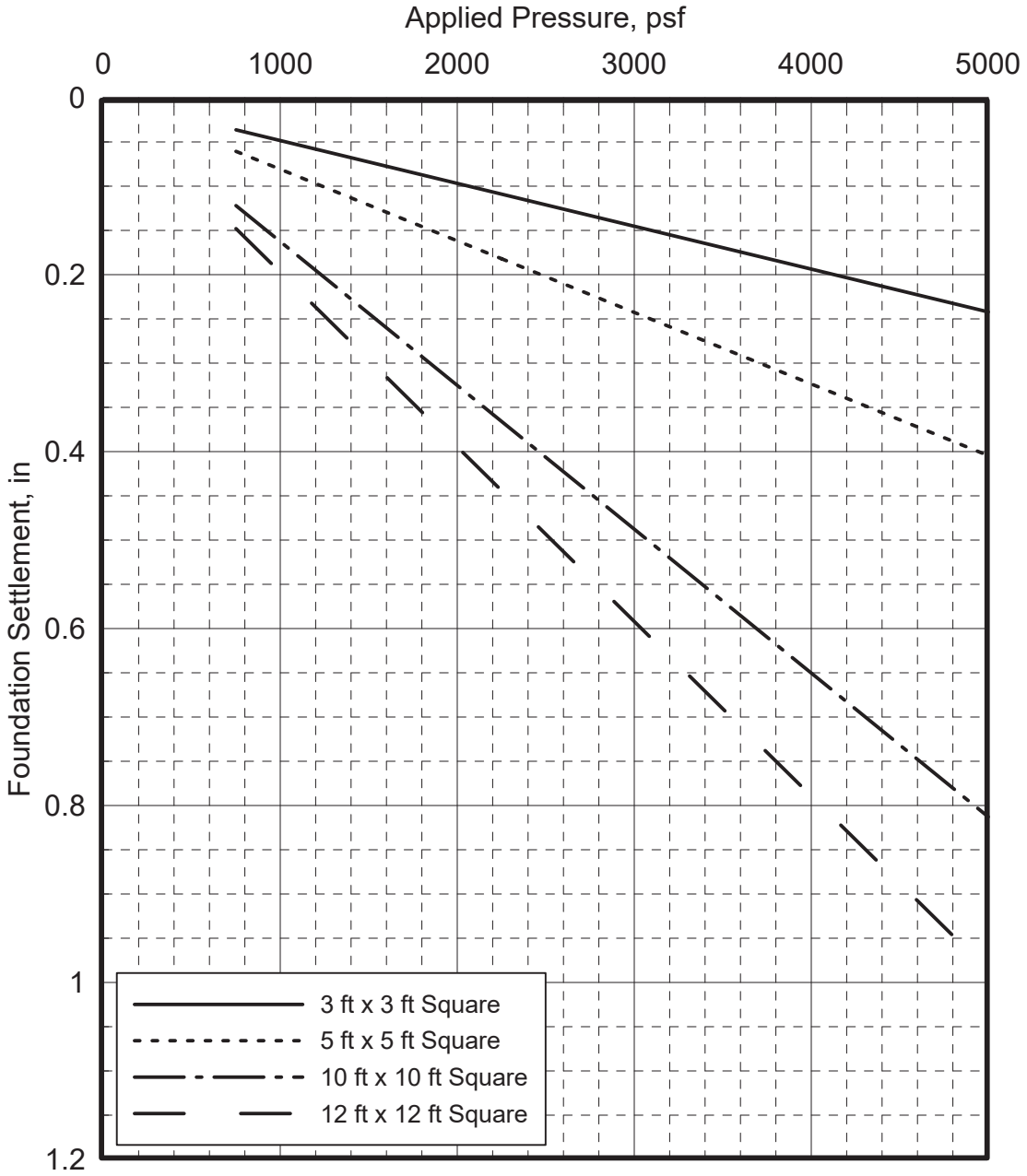


FUGRO	Project: Report No. 04.10160001-12	Alaska LNG Project - Onshore LNG Facilities	
	Analysis Description	Tank (South) Settlement - Operating Load (30 Years)	
	Drawn By: Sochan Jung	Company: Fugro Consultants, Inc.	
SETTLE3D 3.017	Date: 11/21/2016	File Name: A-15 Tank (366-ft_South) 3rd stage load.s3z	

APPENDIX B

Shallow Foundation Recommendations

Shallow Foundation Settlements ..... B-1 thru B-18
Computation of Bearing Pressures..... B-19
Shallow Foundation Insulation Recommendations..... B-20 thru B-23

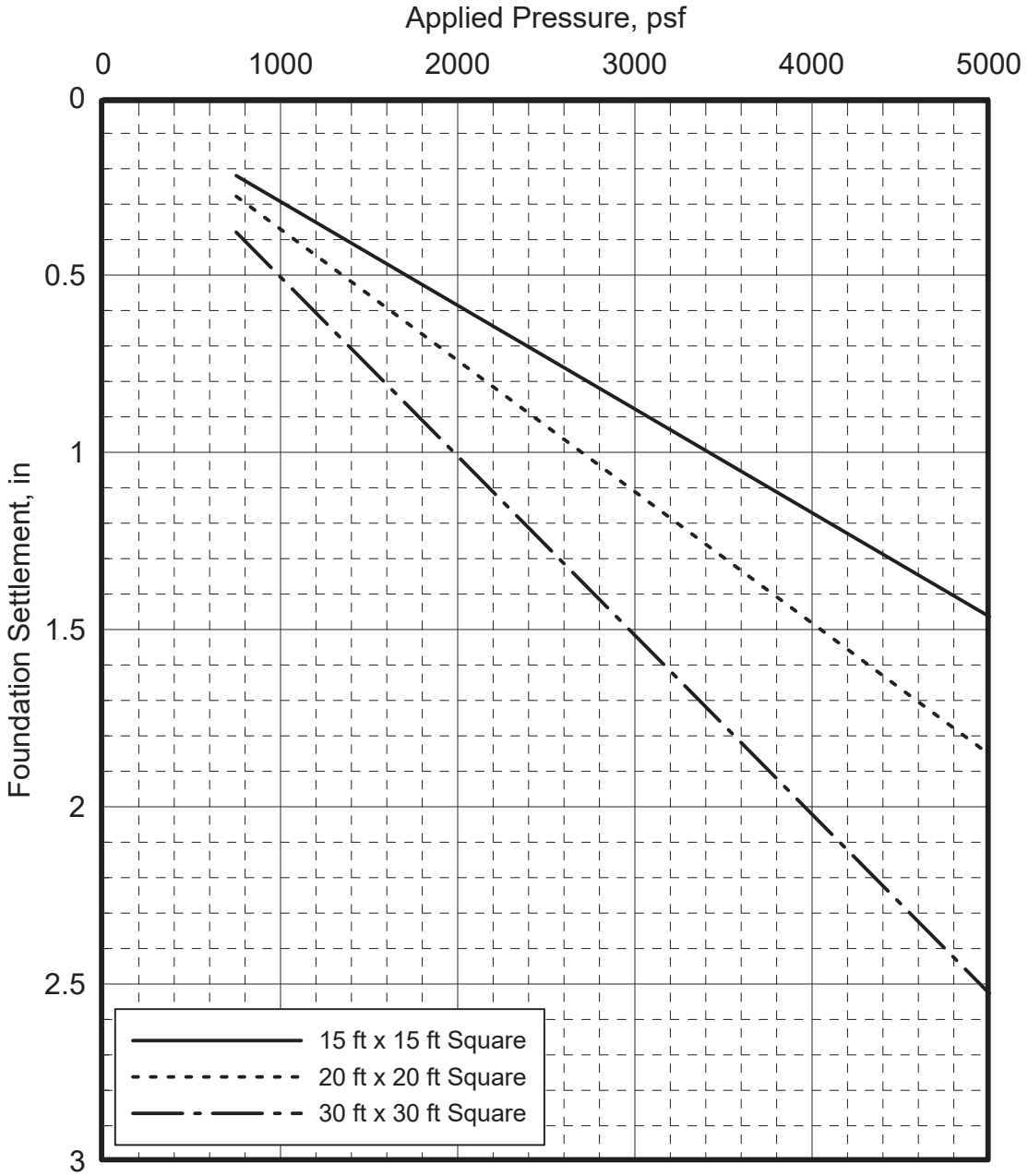


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

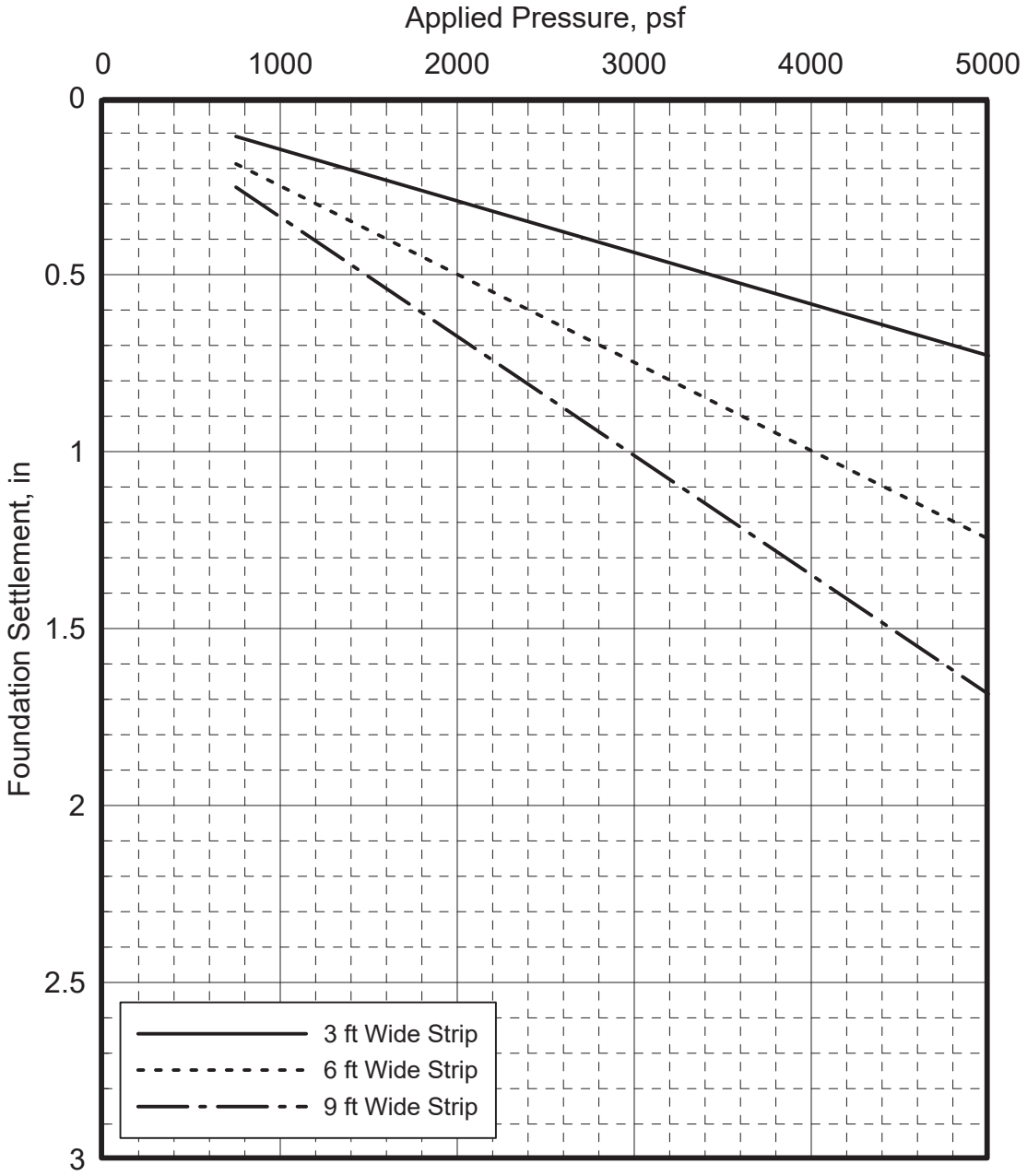


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

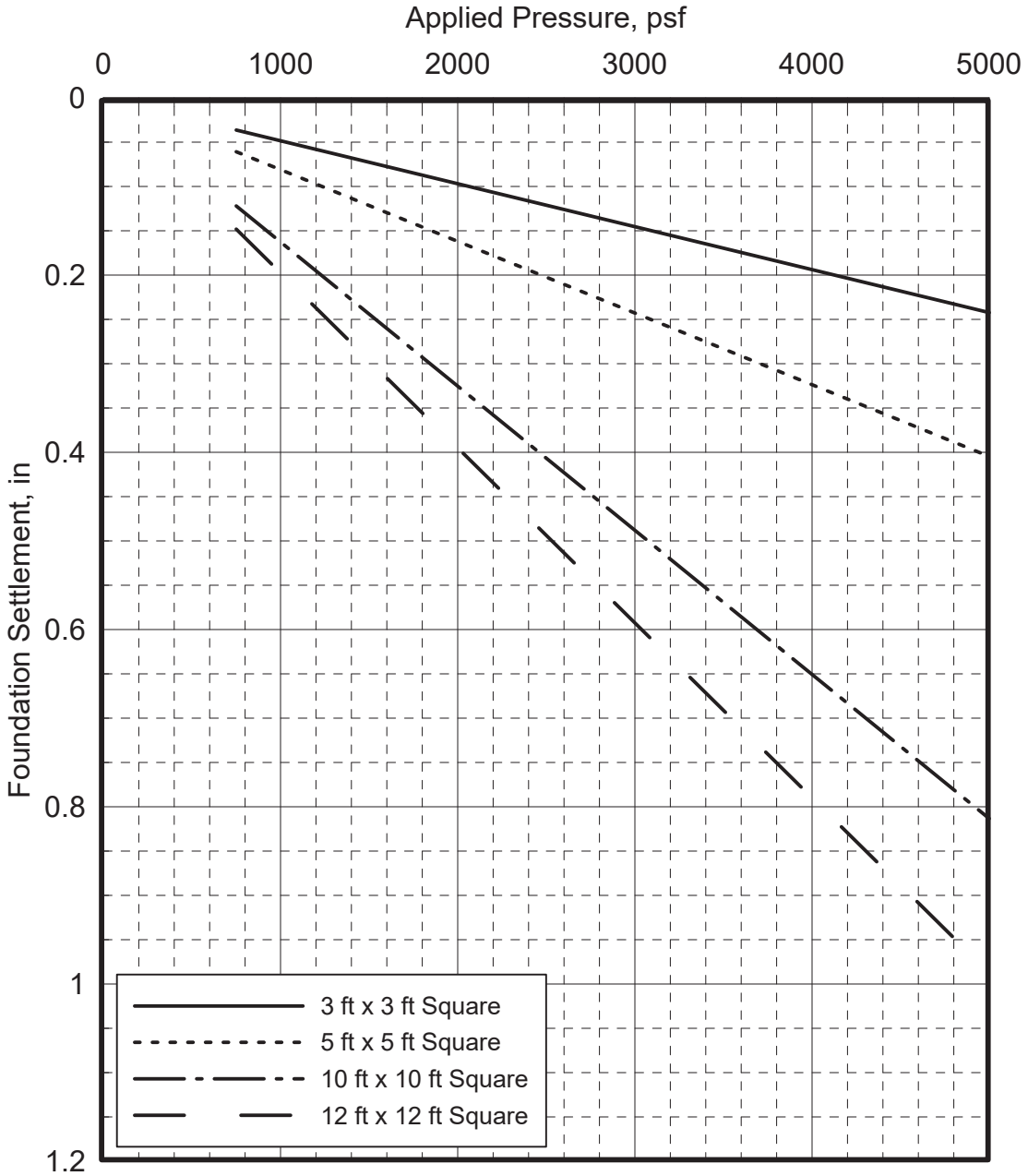


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

STRIP FOOTING SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

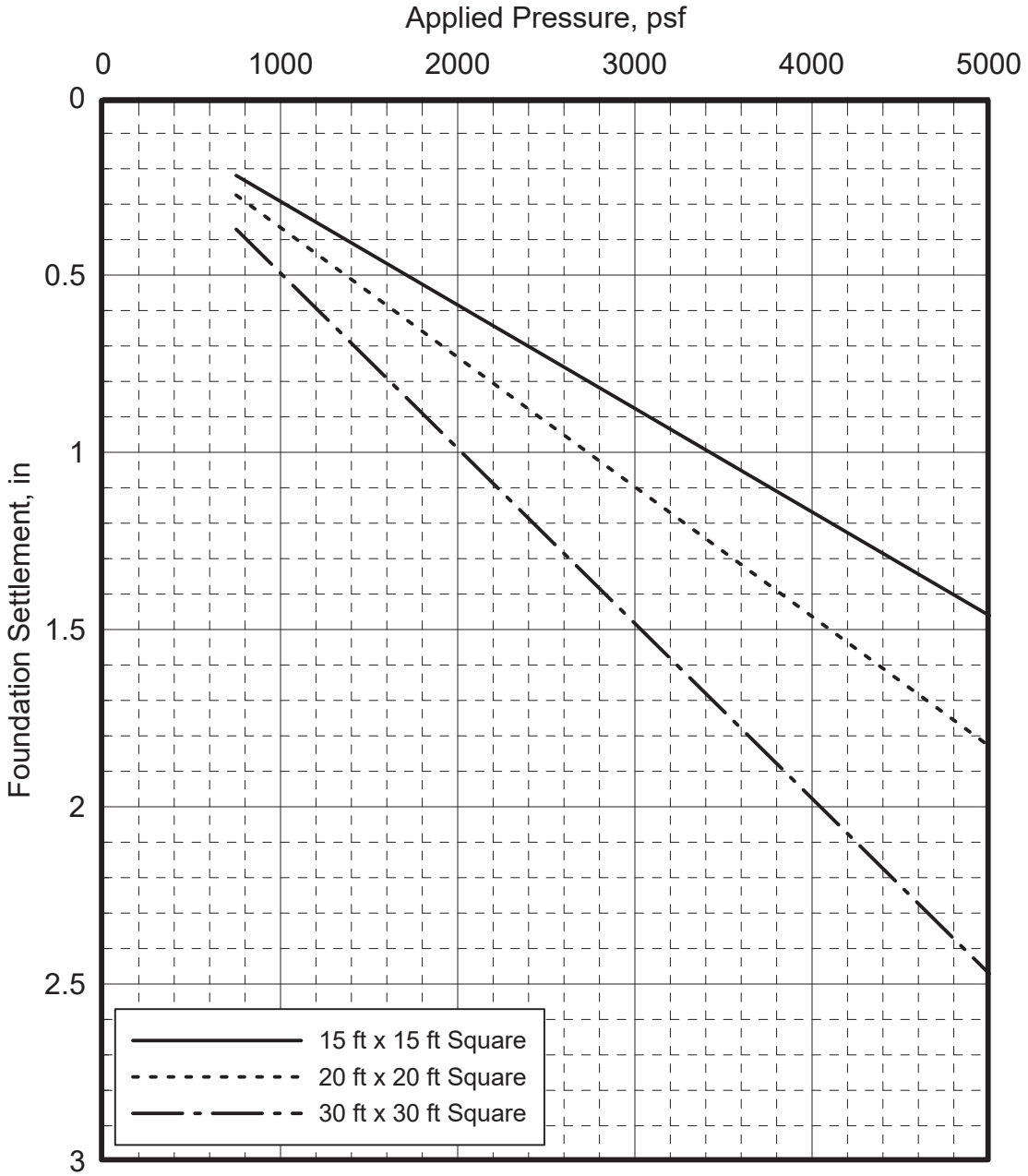


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

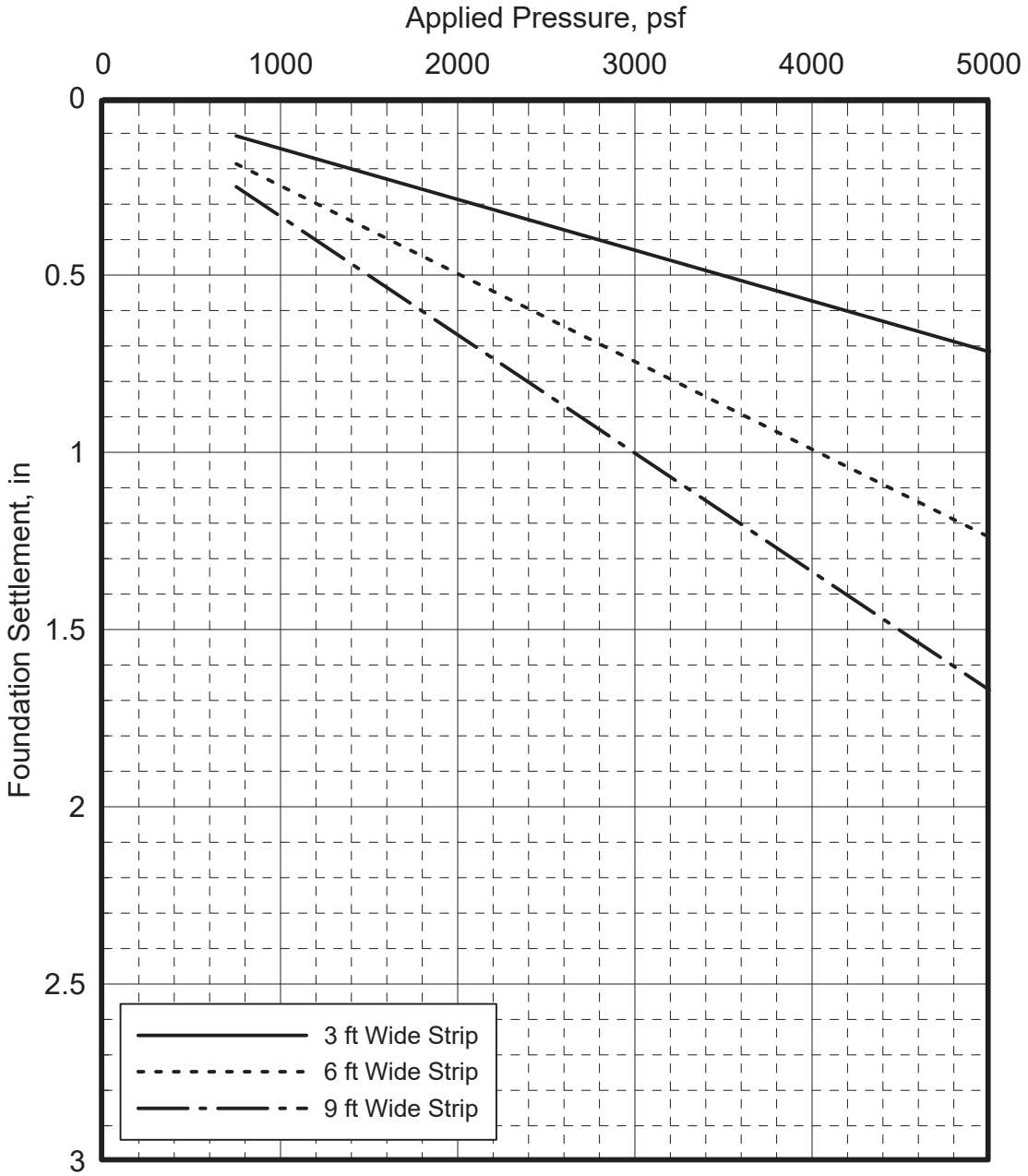


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

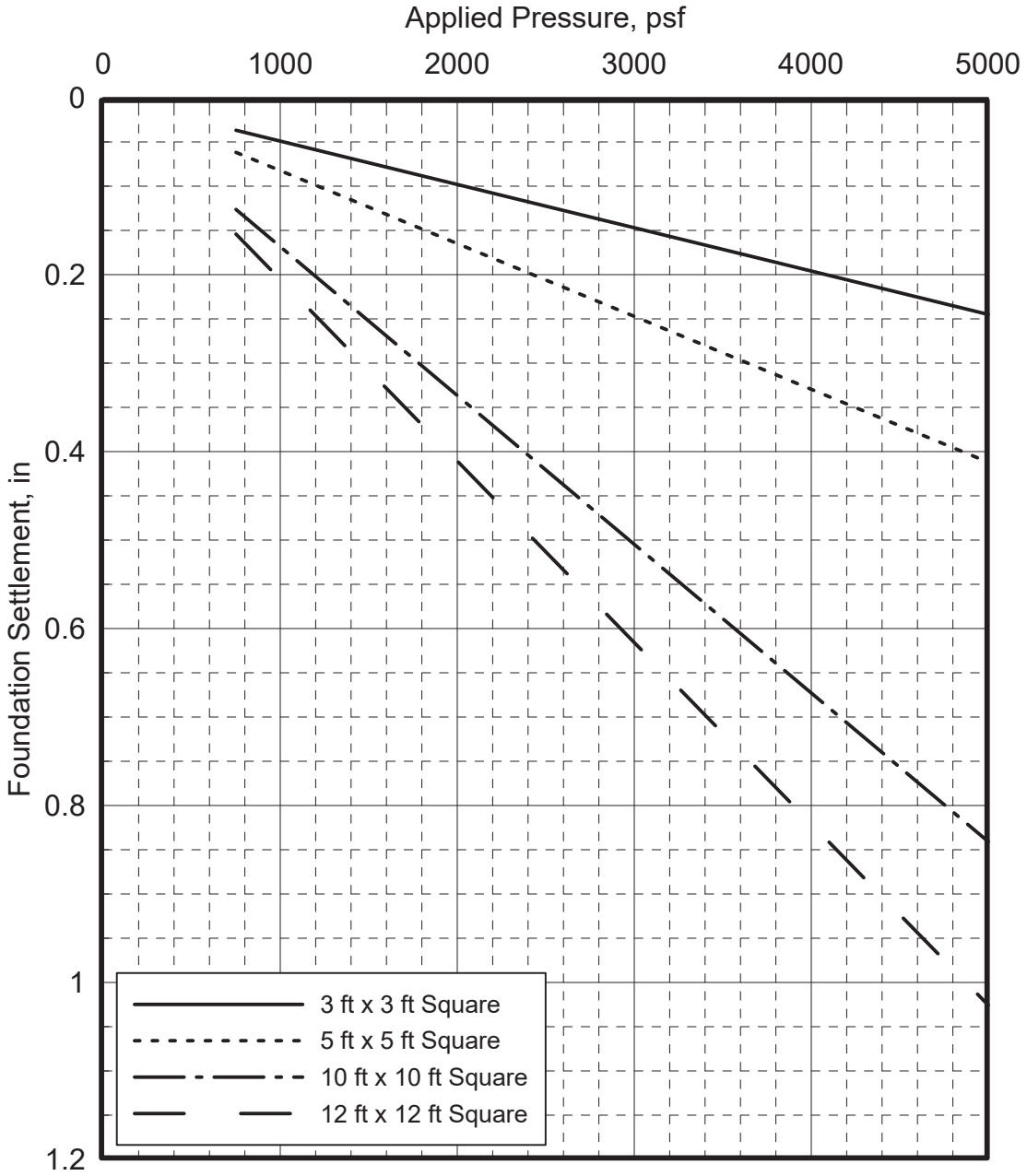


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 125 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

STRIP FOOTING SETTLEMENT - AREAS 2 AND 4
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

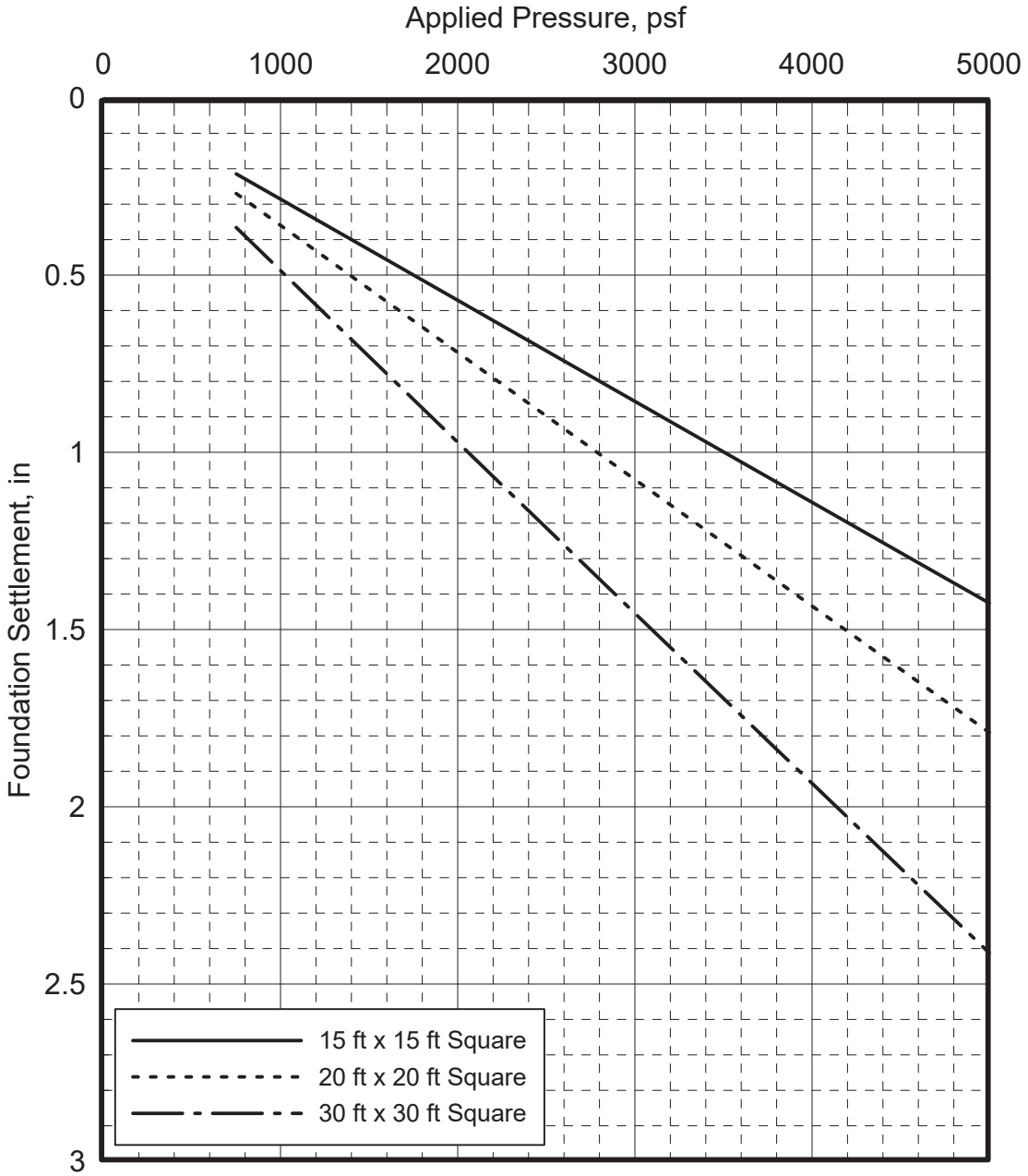


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

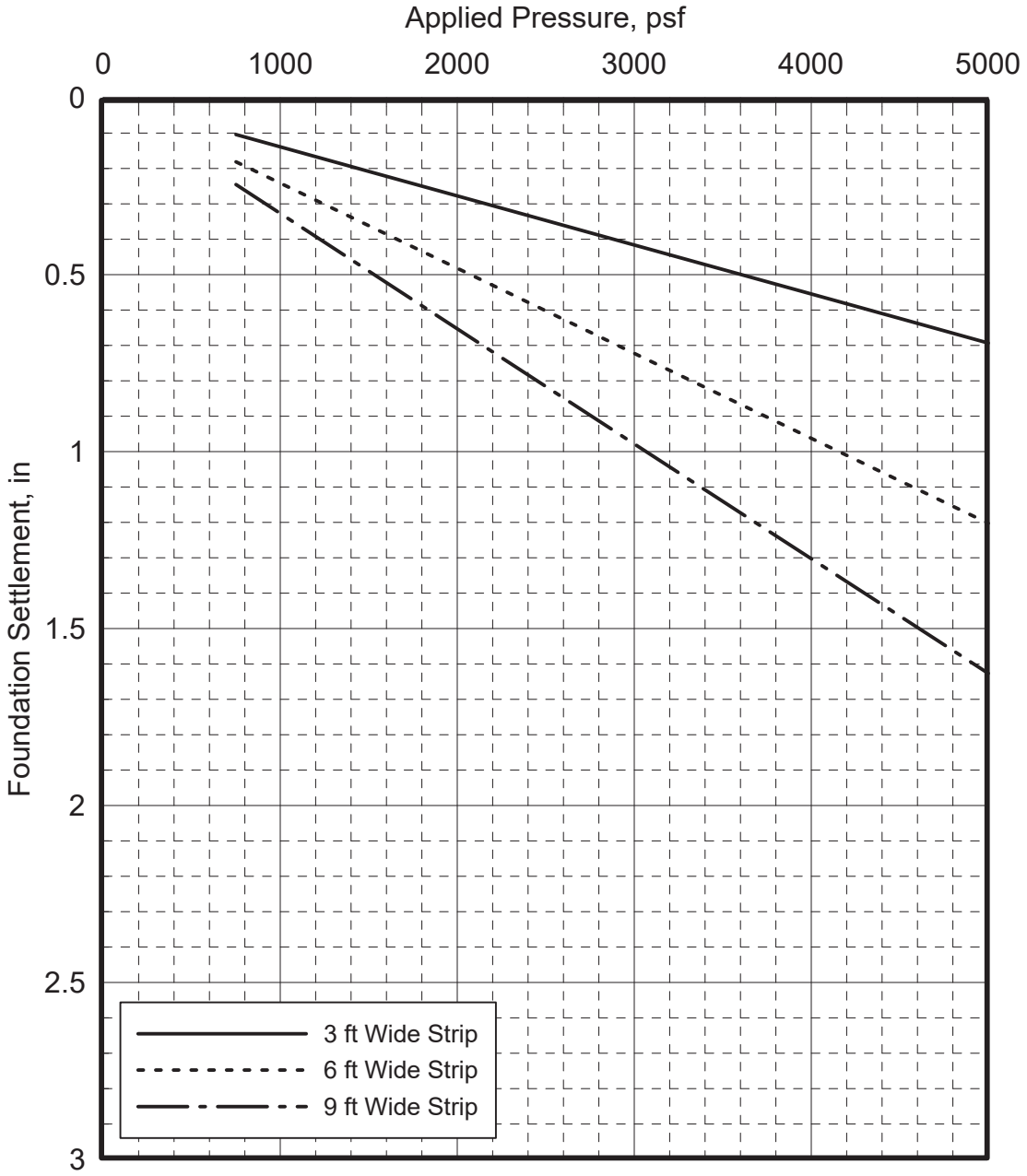


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

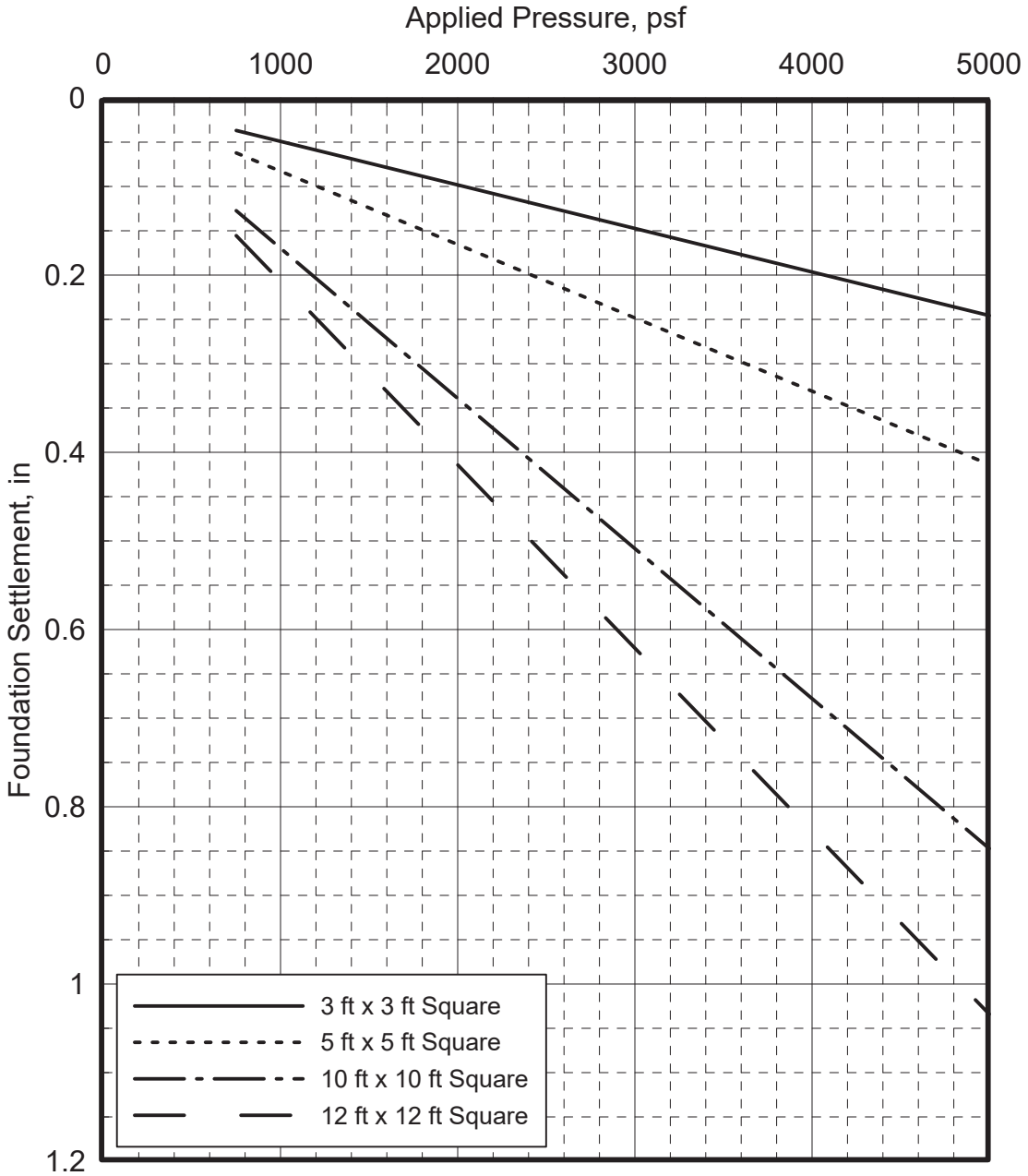


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

STRIP FOOTING SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

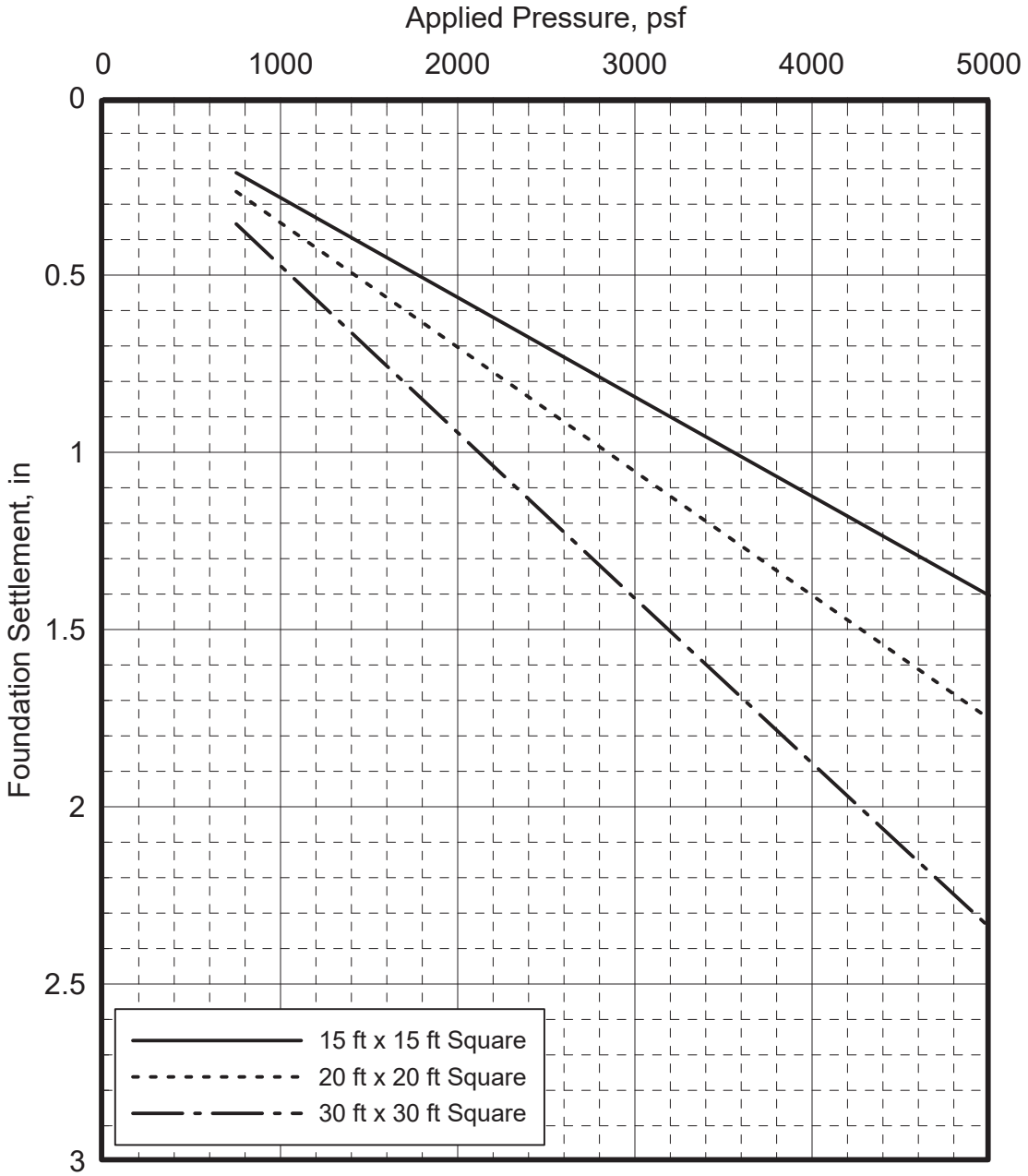


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

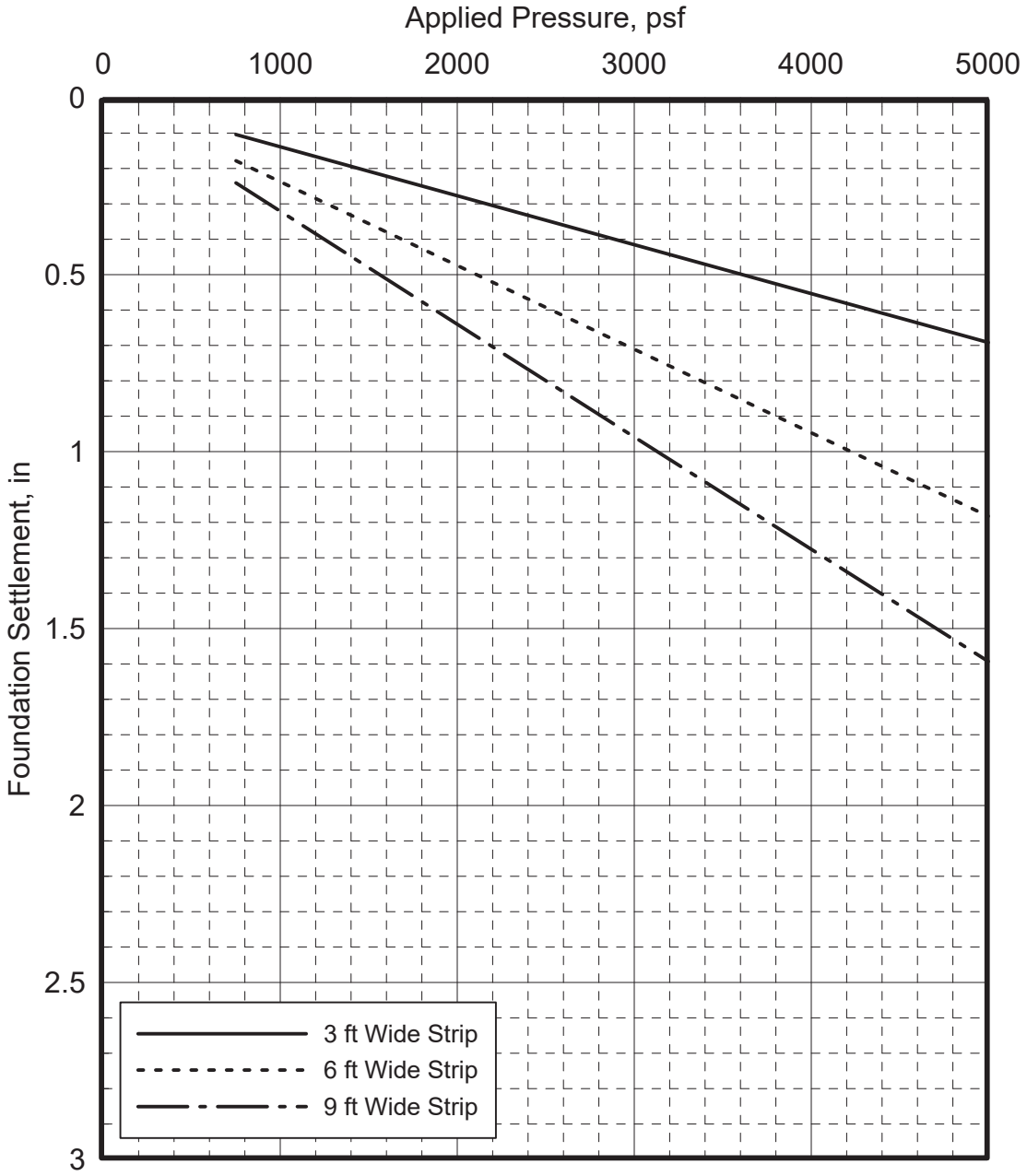


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

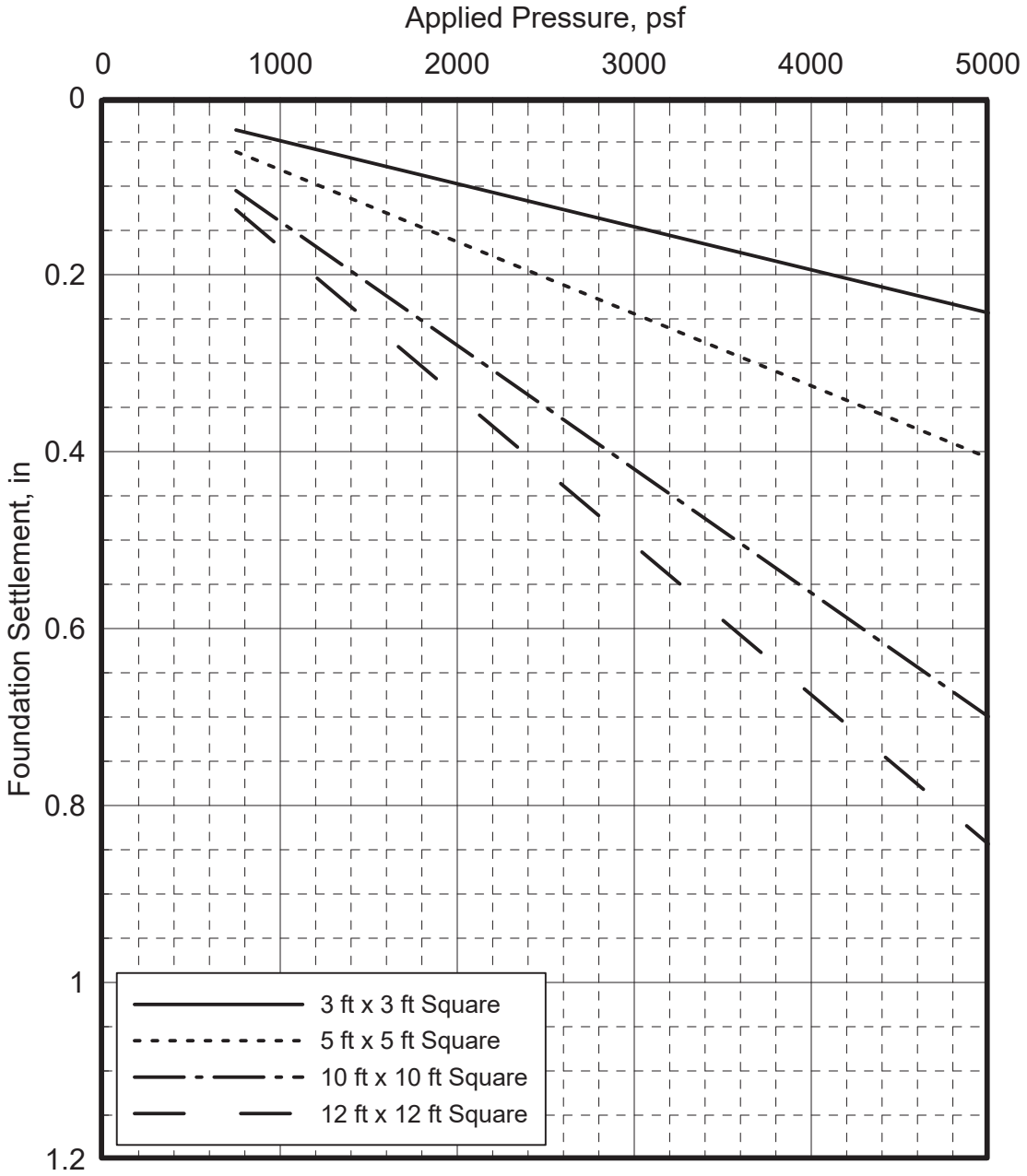


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 70 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

STRIP FOOTING SETTLEMENT - AREA 3
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

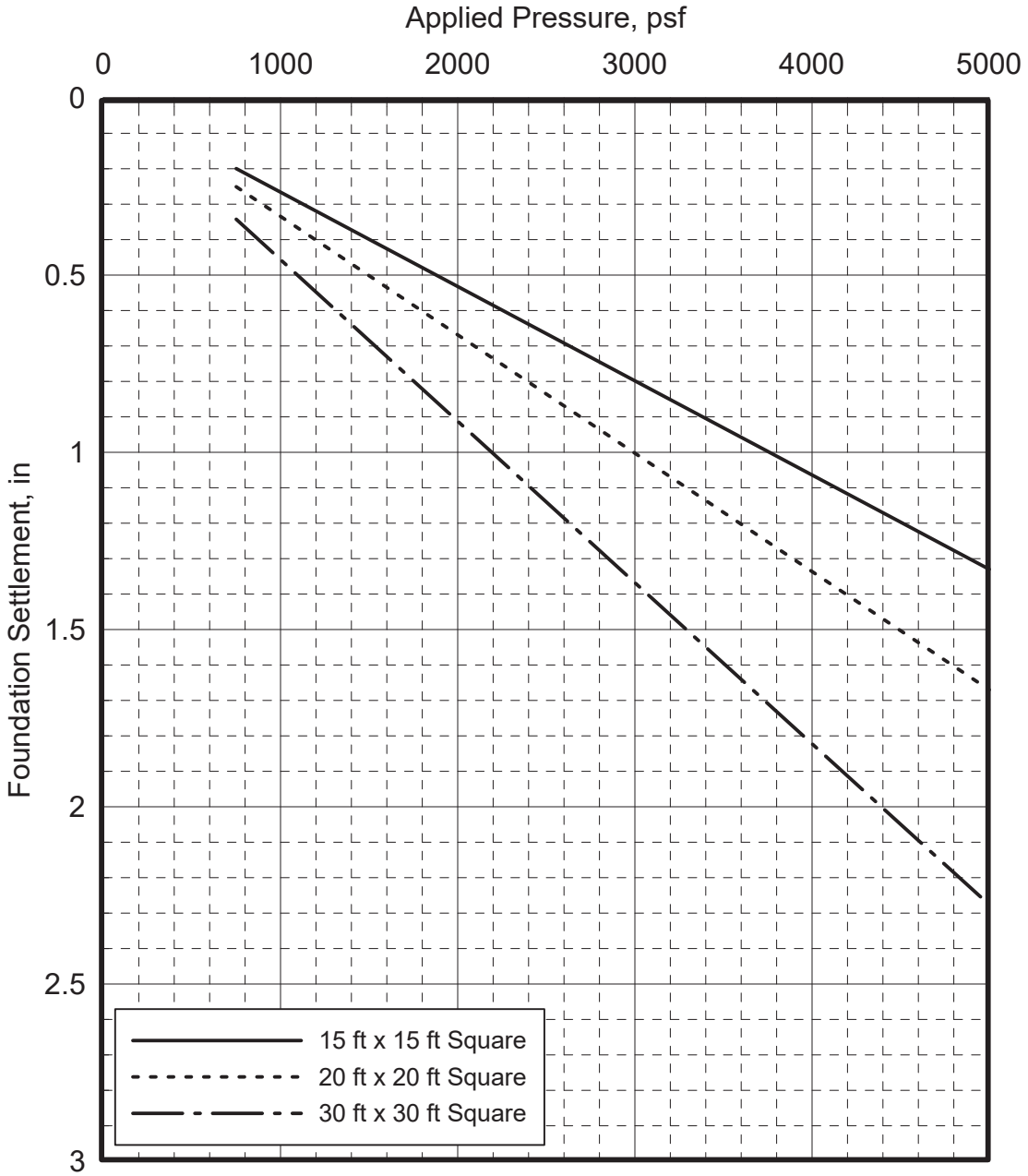


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

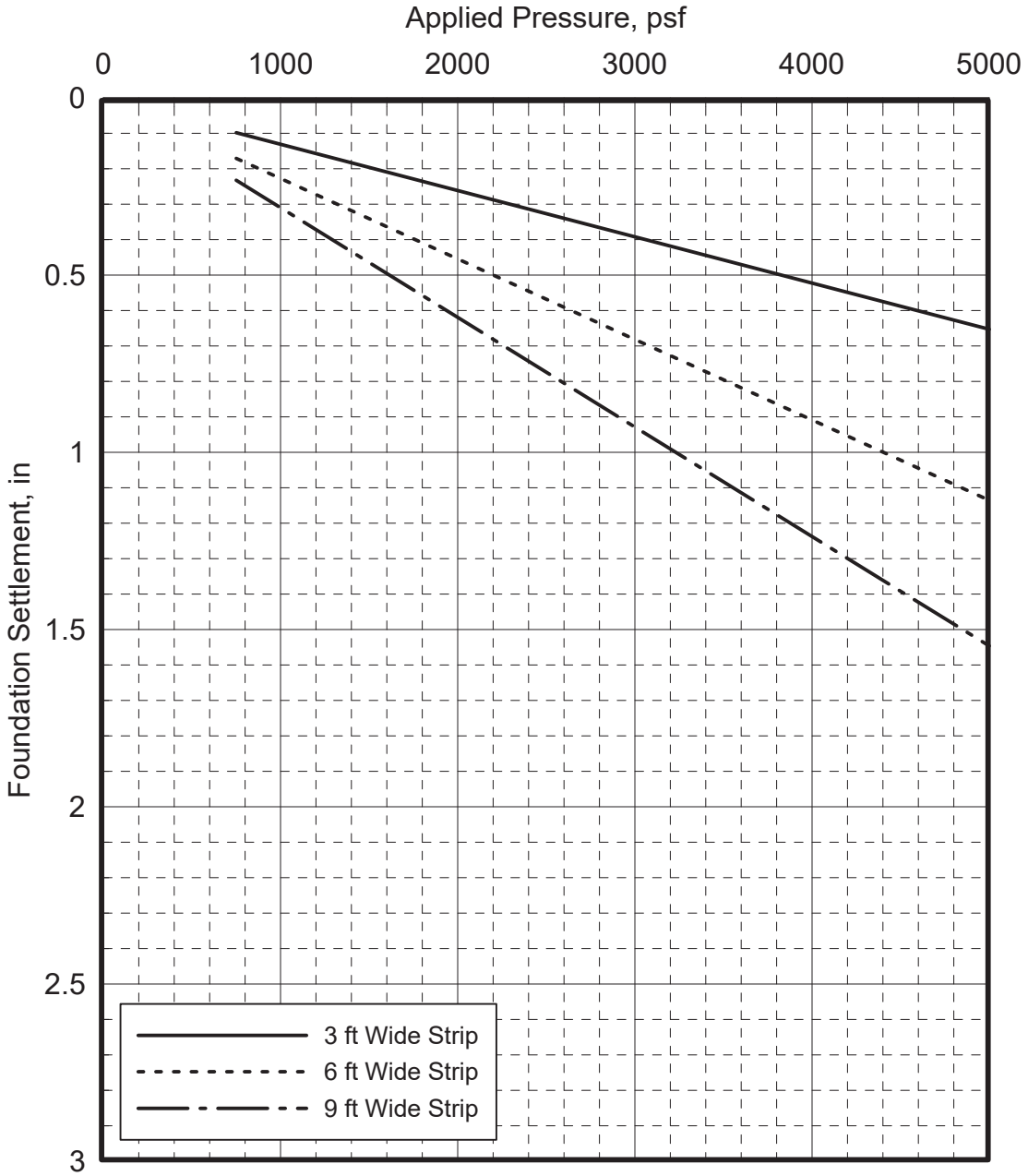


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

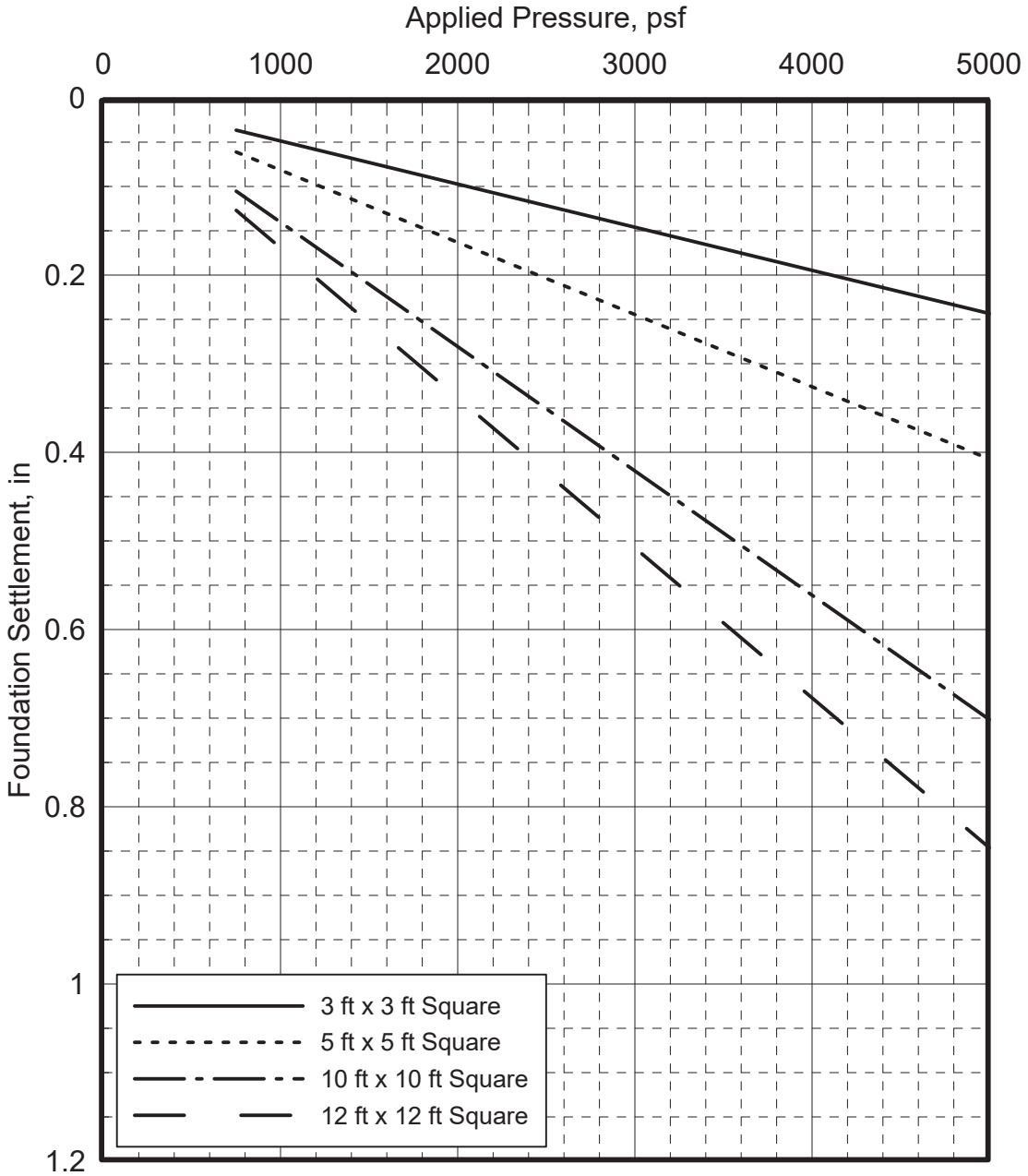


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

STRIP FOOTING SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 4 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

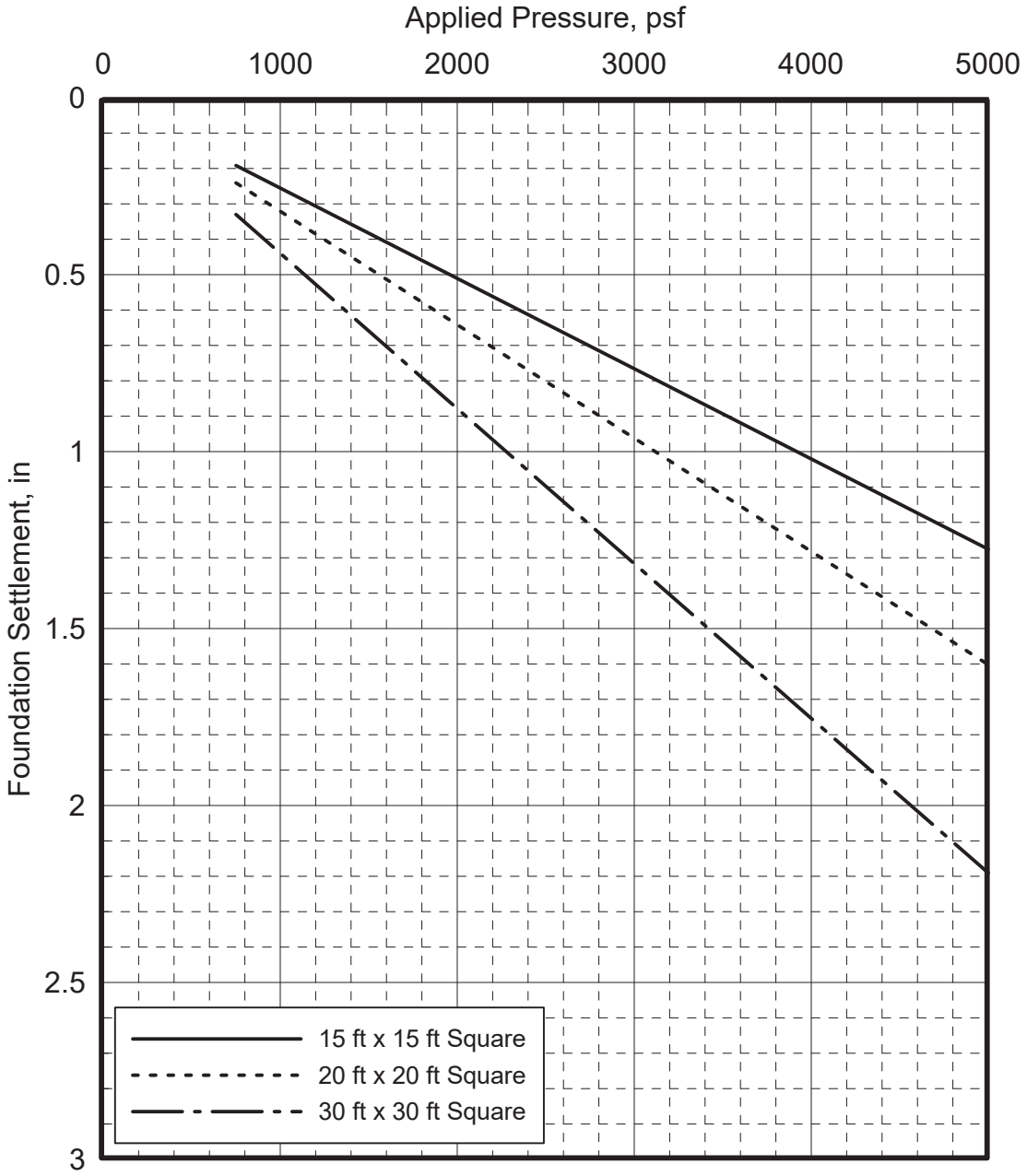


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE SPREAD FOOTING SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

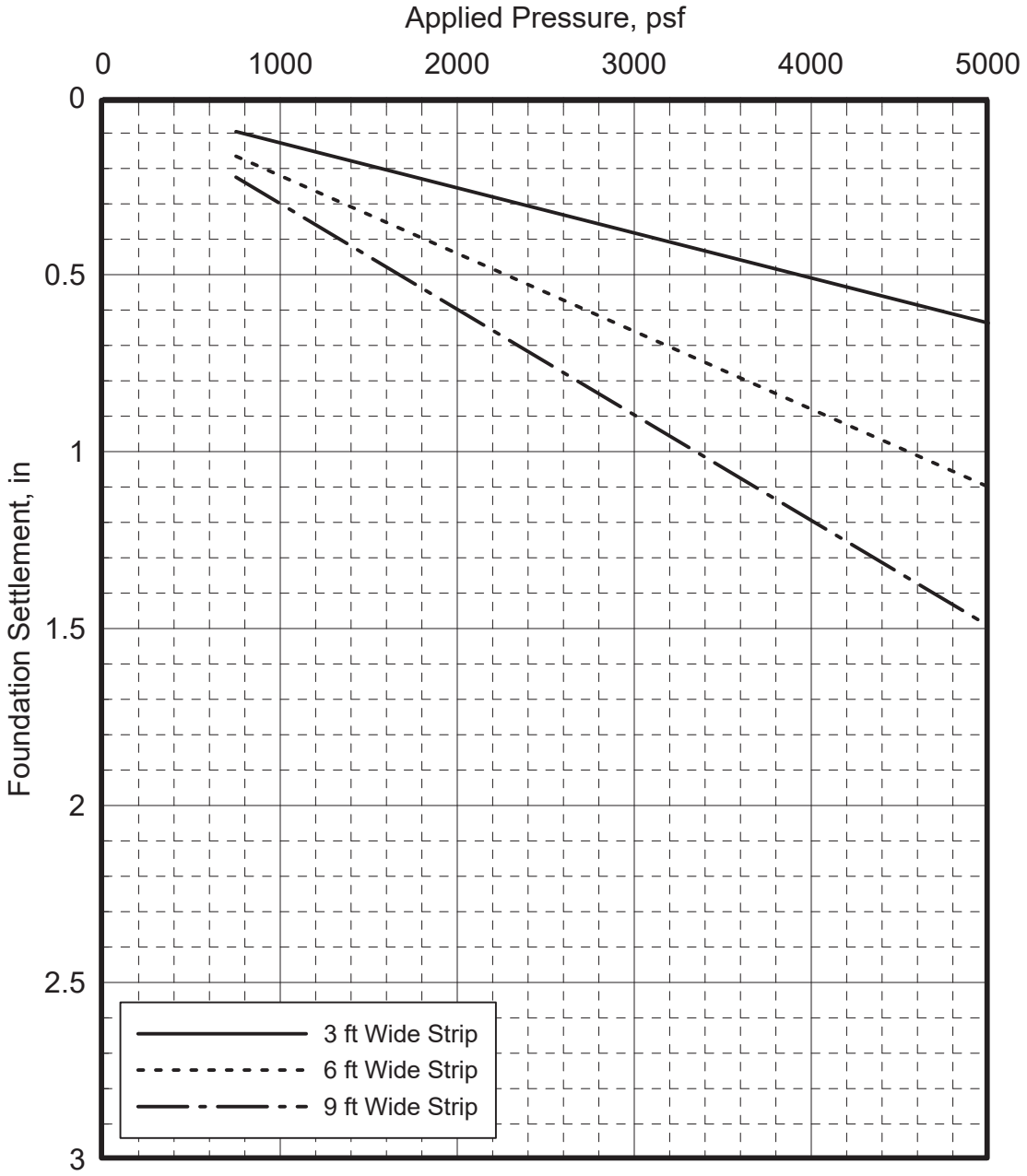


NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

SQUARE MAT FOUNDATION SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent the ***pressure applied to the ground by footings versus settlement of footings***.
2. Final grade was assumed at El. 116 ft and groundwater level was assumed at El. 85 ft.
3. Settlements provided herein do not include settlements within the fill and due to the proposed grade raise.

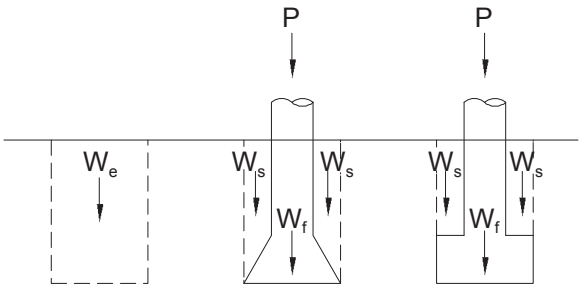
STRIP FOOTING SETTLEMENT - AREA 5
EMBEDMENT DEPTH = 7 FT

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

FOUNDATION DESIGN CRITERIA

A properly-sized foundation must satisfy the two following criteria with respect to the supporting soil.

1. For soil strength. The bearing pressure created on the base of the foundation by the maximum design load must be less than that which would cause shear failure in the soil. A factor of safety of 2 or more with respect to the soil shear strength is generally used.
2. For soil compressibility. The bearing pressure created on the base of the foundation by the sustained load must not produce sufficient consolidation in the underlying soil to result in foundation settlement that is detrimental to the safety or utility of the structure.



SYMBOLS

TERMS AND SYMBOLS

P = Column load (subscript can be used to denote character of load: P_s = sustained load, P_n = normal operating load, P_m = maximum design load).

W_e = Weight of soil located above base of foundation excavation and lowest adjacent grade.*

W_s = Weight of soil located above foundation.*

W_f = Weight of foundation.*

A = Area of base of foundation.

p = Average bearing pressure acting on soil (subsript can be used to correspond to column load: P_s, P_n, P_m).

* Position of groundwater level must be considered in determining weights. Effective, or buoyant, unit weights should be used below the highest expected groundwater level.

BEARING PRESSURES

Gross Bearing Pressure, p, for any column load is the total pressure acting on the base of the foundation.

$$p = 1/A\ (P + W_s + W_f)$$

Net Bearing Pressure, p', for any column load is the difference between the gross bearing pressure acting on the base of the foundation and the soil pressure existing at that elevation from the lowest overlying or adjacent soils.

$$p' = 1/A\ (P + W_s + W_f - W_e)$$

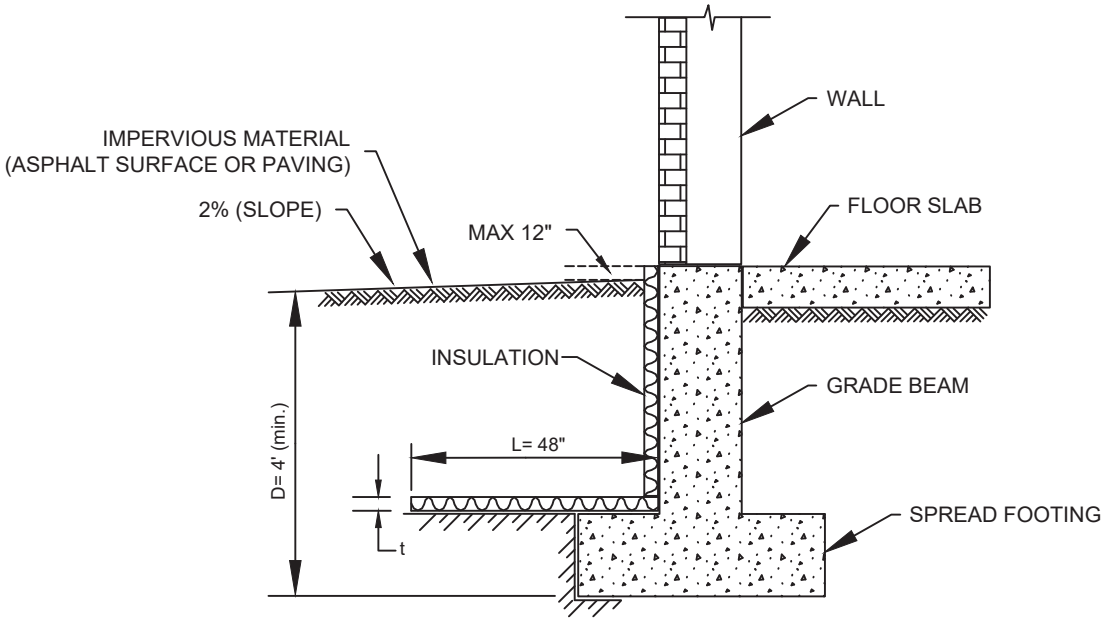
For analysis with regard to the first design criterion, soil strength, the column load in the above equations should usually be the maximum design load, P_m. Occasionally, the normal operating load, P_n, may also be used. If footing is loaded eccentrically, the increase in edge bearing pressure due to the eccentricity should be computed in the usual manner.

For analysis with regard to the second design criterion, soil compressibility, the column load in the above equations should be the sustained load, P_s. This load is the dead load plus the sustain live load.

For further references, see pp. 506 - 512, "Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck (2nd edition); and pp. 564 - 565, "Fundamentals of Soil Mechanics" by Donald W. Taylor.

COMPUTATION OF BEARING PRESSURES

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. THE PURPOSE OF THE IMPERVIOUS SURFACING IS TO REDUCE POTENTIAL SURFACE WATER FILTRATION INTO THE FOUNDATION BACKFILL THAT MAY ADVERSELY AFFECT FOUNDATION PERFORMANCE. THIS IS ALSO RECOMMENDED WHERE THERE IS A RISK OF HYDROCARBON SPILLAGE TO REDUCE POTENTIAL CONTACT WITH RIGID INSULATION.
2. REFERENCE HUD DESIGN GUIDE FOR FPSF (1994).

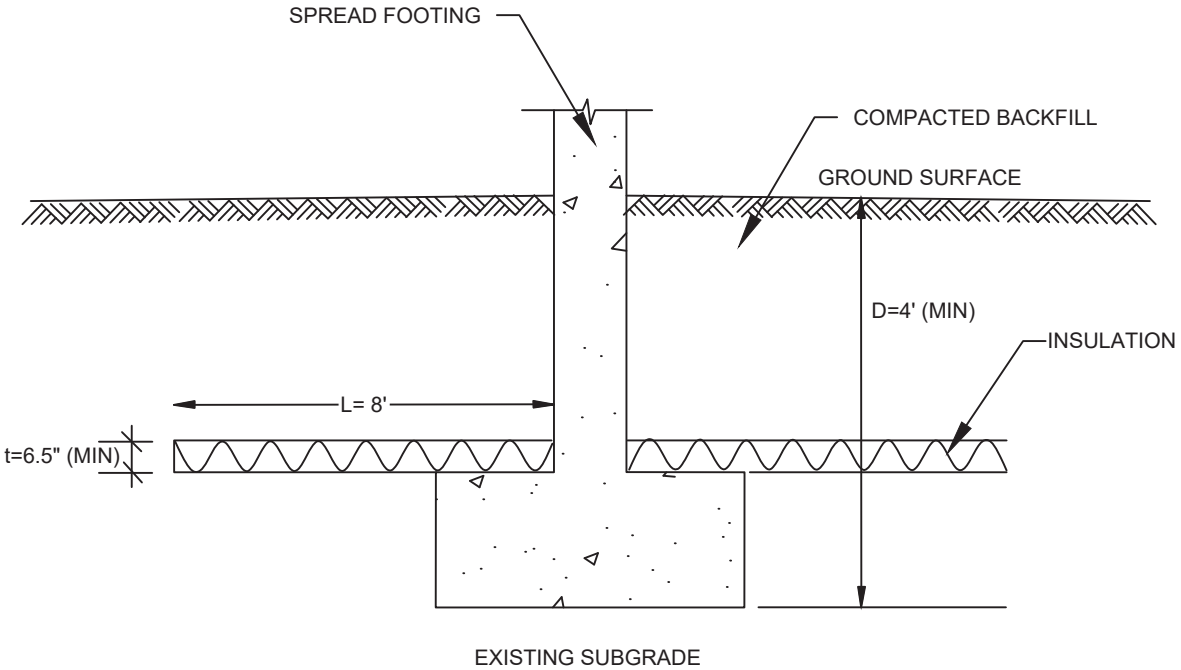
MINIMUM BUILDING TEMPERATURE	RECOMMENDED	
	MIN. RIGID INSULATION THICKNESS(t)	OUTWARD HORIZONTAL DISTANCE (L)
18°C (65°F)	2"	48"
7°C (45°F)	3"	48"
R Value = 9.0 ft^2. h. °F/Btu, Vertical, min along walls R Value = 8.0 ft^2. h. °F/Btu, Horizontal, min along walls R Value = 11.2 ft^2. h. °F/Btu, Horizontal, min along corners (ASCE 2001, ASTM C518)		

REFERENCES:

1) ASTM C 518. STANDARD TEST METHOD FOR STEADY-STATE THERMAL TRANSMISSION PROPERTIES BY MEANS OF THE HEAT FLOW METER APPARATUS.
2) ASCE DESIGN AND CONSTRUCTION OF FROST PROTECTED SHALLOW FOUNDATIONS. (2001). SEI/ASCE 32-01.
3) REVISED BUILDER'S GUIDE TO FROST PROTECTED SHALLOW FOUNDATIONS. (2004). NATIONAL ASSOCIATION OF HOME BUILDERS (NAHB) RESEARCH CENTER, INC.
4) DESIGN GUIDE FOR FROST-PROTECTED SHALLOW FOUNDATIONS. (1994). US DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT OFFICE OF POLICY DEVELOPMENT AND RESEARCH.

TYPICAL INSULATED SPREAD FOOTING
FOR HEATED BUILDING
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE B-20



R Value = 26.0 ft^2. h. °F/Btu, Horizontal, min

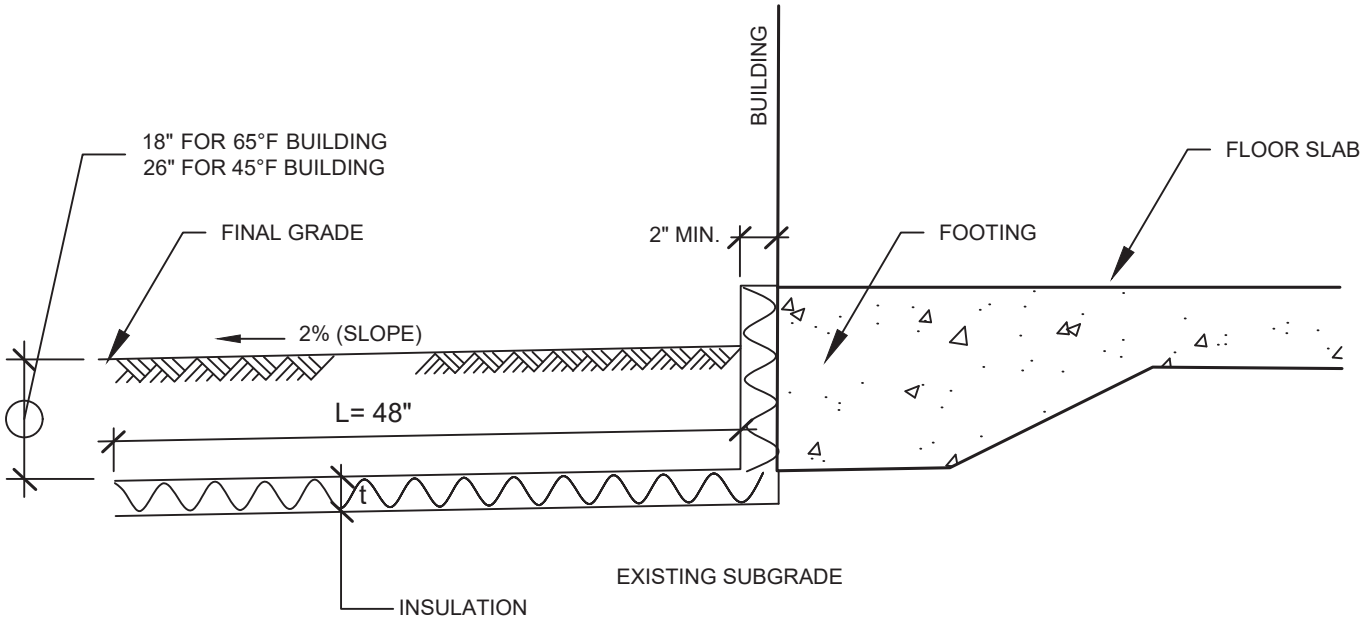
(ASCE 2001, ASTM C518)

REFERENCES:

1) ASTM C 518. STANDARD TEST METHOD FOR STEADY-STATE THERMAL TRANSMISSION PROPERTIES BY MEANS OF THE HEAT FLOW METER APPARATUS.
2) ASCE DESIGN AND CONSTRUCTION OF FROST PROTECTED SHALLOW FOUNDATIONS. (2001). SEI/ASCE 32-01.
3) REVISED BUILDER'S GUIDE TO FROST PROTECTED SHALLOW FOUNDATIONS. (2004). NATIONAL ASSOCIATION OF HOME BUILDERS (NAHB) RESEARCH CENTER, INC.
4) DESIGN GUIDE FOR FROST-PROTECTED SHALLOW FOUNDATIONS. (1994). US DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT OFFICE OF POLICY DEVELOPMENT AND RESEARCH.

TYPICAL INSULATED SPREAD FOOTING
FOR UNHEATED BUILDING
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE B-21



NOTE:

1. HEATED BUILDING SHOULD COVER THE ENTIRE CONCRETE FLOOR SLAB AREA TO PREVENT FROST FROM PENETRATING THROUGH THE CONCRETE FOOTING.

MINIMUM BUILDING TEMPERATURE	RECOMMENDED	
	RIGID INSULATION THICKNESS(t)	OUTWARD HORIZONTAL DISTANCE (L)
18°C (64°F)	2"	48"
7°C (45°F)	3"	48"
R Value = 9.0 ft^2. h. °F/Btu, Vertical, min R Value = 8.0 ft^2. h. °F/Btu, Horizontal, min (ASCE 2001, ASTM C518)		

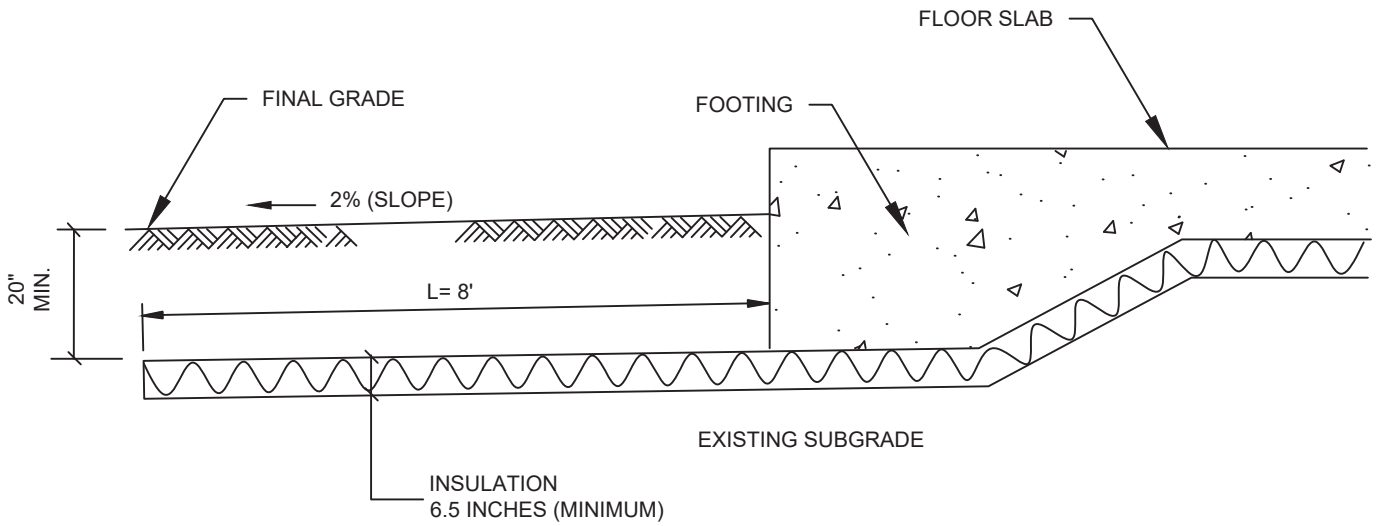
REFERENCES:

1) ASTM C 518. STANDARD TEST METHOD FOR STEADY-STATE THERMAL TRANSMISSION PROPERTIES BY MEANS OF THE HEAT FLOW METER APPARATUS.
2) ASCE DESIGN AND CONSTRUCTION OF FROST PROTECTED SHALLOW FOUNDATIONS. (2001). SEI/ASCE 32-01.
3) REVISED BUILDER'S GUIDE TO FROST PROTECTED SHALLOW FOUNDATIONS. (2004). NATIONAL ASSOCIATION OF HOME BUILDERS (NAHB) RESEARCH CENTER, INC.
4) DESIGN GUIDE FOR FROST-PROTECTED SHALLOW FOUNDATIONS. (1994). US DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT OFFICE OF POLICY DEVELOPMENT AND RESEARCH.

TYPICAL INSULATED MAT FOUNDATION / SLAB-ON-GRADE FOR HEATED BUILDING

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE B-22



R Value = 26.0 ft^2. h. °F/Btu, Horizontal, min

(ASCE 2001, ASTM C518)

REFERENCES:

1) ASTM C 518. STANDARD TEST METHOD FOR STEADY-STATE THERMAL TRANSMISSION PROPERTIES BY MEANS OF THE HEAT FLOW METER APPARATUS.
2) ASCE DESIGN AND CONSTRUCTION OF FROST PROTECTED SHALLOW FOUNDATIONS. (2001). SEI/ASCE 32-01.
3) REVISED BUILDER'S GUIDE TO FROST PROTECTED SHALLOW FOUNDATIONS. (2004). NATIONAL ASSOCIATION OF HOME BUILDERS (NAHB) RESEARCH CENTER, INC.
4) DESIGN GUIDE FOR FROST-PROTECTED SHALLOW FOUNDATIONS. (1994). US DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT OFFICE OF POLICY DEVELOPMENT AND RESEARCH.

TYPICAL INSULATED MAT FOUNDATION / SLAB-ON-GRADE FOR UNHEATED BUILDING

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE B-23

APPENDIX C

Deep Foundation Recommendations

Deep Foundation Design Parameters ..... C-1 and C-9
Ultimate Axial Capacity – Areas 2, 3, 4, 5 ..... C-10 thru C-33
Axial Load vs Pile Top Settlement – Areas 2, 3, 4, 5 ..... C-34 and C-41
Lateral Capacity – Areas 2 and 4 ..... C-42 thru C-63
Pile Cap Insulation Recommendations..... C-64

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 2A

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
125(2) to 123	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
123 to 120	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
120 to 100	5 to 25	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
100 to 80	25 to 45	Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
80 to 70	45 to 55	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	123	-	36	2.1	42	210	API Sand	-	36	1.0	2.50×10^5	-
70 to 60	55 to 65	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
60 to 40	65 to 85	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
40 to -20	85 to 145	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
-20 to -100	145 to 225	Stiff to Very Stiff Clay, Sandy Clay, or Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	6.0 to 9.0	-	-	-	-	Stiff Clay	6.0 to 9.0	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 125 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 70 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 2A
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 2B

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
125(2) to 123	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
123 to 120	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
120 to 100	5 to 25	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
100 to 80	25 to 45	Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
80 to 70	45 to 55	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	123	-	36	2.1	42	210	API Sand	-	36	1.0	2.50×10^5	-
70 to 60	55 to 65	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
60 to 40	65 to 85	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
40 to 0	85 to 125	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
0 to -15	125 to 140	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
-15 to -50	140 to 175	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
-50 to -100	175 to 225	Stiff to Very Stiff Clay, Sandy Clay, or Very Dense Silty and Gravelly Sand, Clayey Sand	65.6	7.0 to 9.0	-	-	-	-	Stiff Clay	7.0 to 9.0	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 125 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 70 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 2B
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 2C

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
125(2) to 123	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
123 to 120	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
120 to 100	5 to 25	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
100 to 80	25 to 45	Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
80 to 70	45 to 55	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	123	-	36	2.1	42	210	API Sand	-	36	1.0	2.50×10^5	-
70 to 50	55 to 75	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
50 to 40	75 to 85	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	60.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
40 to 20	85 to 105	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
20 to 0	105 to 125	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
0 to -40	125 to 165	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
-40 to -100	165 to 225	Stiff to Very Stiff Clay, Sandy Clay, or Very Dense Silty and Gravelly Sand, Clayey Sand	65.6	7.0 to 9.0	-	-	-	-	Stiff Clay	7.0 to 9.0	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 125 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 70 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 2C
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 3

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
116(2) to 114	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
114 to 111	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
111 to 100	5 to 20	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
100 to 80	20 to 40	New Fill or Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
80 to 70	40 to 55	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	123	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
70 to 50	55 to 75	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
50 to 40	75 to 85	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
40 to -15	85 to 105	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, Sandy Silt or Stiff to Very Stiff Clay	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-
-15 to -80	105 to 125	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	65.6	5.0 to 8.0	-	-	-	-	Stiff Clay	5.0 to 8.0	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 116 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 70 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 3
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 4A

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
125(2) to 123	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
123 to 120	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
120 to 105	5 to 20	Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
105 to 90	20 to 35	Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
90 to 85	35 to 40	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	125	-	36	2.1	42	210	API Sand	-	36	1.0	2.50×10^5	-
85 to 55	40 to 70	Dense to Very Dense Silty and Gravelly Sand	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
55 to 45	70 to 80	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
45 to -75	80 to 200	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, or Very Stiff Clay	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 125 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 85 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 4A
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 4B

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
125(2) to 123	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
123 to 120	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
120 to 105	5 to 20	Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
105 to 90	20 to 35	Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
90 to 85	35 to 40	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	125	-	36	2.1	42	210	API Sand	-	36	1.0	2.50×10^5	-
85 to 55	40 to 70	Dense to Very Dense Silty and Gravelly Sand	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
55 to 45	70 to 80	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
45 to 20	80 to 105	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, or Very Stiff Clay	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-
20 to -5	105 to 130	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	65.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
-5 to -75	130 to 200	Dense to Very Dense Silty and Gravelly Sand, Clayey Sand, or Very Stiff Clay	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 125 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 85 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 4B
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – AREA 5

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
116(2) to 114	0 to 2	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
114 to 111	2 to 5	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
111 to 105	5 to 11	New Fill or Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
105 to 90	11 to 26	New Fill or Medium Dense to Dense Silty and Gravelly Sand	120	-	35	2.0	40	200	API Sand	-	35	1.0	2.15×10^5	-
90 to 85	26 to 31	Dense to Very Dense Silty and Gravelly Sand	123	-	36	2.1	42	210	API Sand	-	36	1.0	2.5×10^5	-
85 to 70	31 to 46	Dense to Very Dense Silty and Gravelly Sand	60.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
70 to 60	46 to 56	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand, Clayey Sand	62.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
60 to 40	56 to 76	Dense to Very Dense Silty and Gravelly Sand or Stiff to Very Stiff Clay	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
40 to 25	76 to 91	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand	65.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004
25 to -5	91 to 121	Dense to Very Dense Silty and Gravelly Sand or Stiff to Very Stiff Clay	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-
-5 to -50	121 to 166	Stiff to Very Stiff Clay or Dense to Very Dense Silty and Gravelly Sand	65.6	5.0 to 6.5	-	-	-	-	Stiff Clay	5.0 to 6.5	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 116 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 85 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
AREA 5
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

TABLE: PILE CAPACITY DESIGN PARAMETERS – ASRC B-176

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,(4) pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	Limiting Skin Friction,(3) ksf	N_q(3)	Limiting End Bearing,(3)(8) ksf	Soil Type	Undrained Shear Strength,(3) ksf	Friction Angle (Φ),(3) degrees	*P*-Multiplier (10)	Subgrade Modulus,(7) pcf	ε_{50} (11)
27(2) to 25	0 to 2	Existing Fill: Sand with Silt and Gravel	118	Neglected to account for disturbance and freeze-thaw conditions(5)					Neglected to account for disturbance and freeze-thaw conditions(6)					
25 to 22	2 to 5	Existing Fill: Sand with Silt and Gravel	118						API Sand	-	25(6)	1.0	0.35×10^5 (6)	-
22 to 7	5 to 20	Existing Fill: Sand with Silt and Gravel	118	-	30	1.7	20	100	API Sand	-	30	1.0	0.35×10^5	-
7 to -7	20 to 34	Existing Fill or Natural Very Loose to Loose Sand with Silt and Gravel	55.6	(0.2)(10)	27	1.5	15	76	API Sand	-	27	0.1	0.35×10^5	-
-7 to -73	34 to 100	Dense to Very Dense Sand with Silt and Gravel	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
-73 to -103	100 to 130	Stiff to very Stiff Clay and Gravelly Clay	62.6	4.0	-	-	-	-	Stiff Clay	4.0	-	1.0	-	0.005
-103 to -147	130 to 174	Dense to Very Dense Silty and Gravelly Sand	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
-147 to -180	174 to 207	Stiff to very Stiff Clay, Sandy Clay, and Gravelly Clay	62.6	5.0	-	-	-	-	Stiff Clay	5.0	-	1.0	-	0.004

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 27 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015) and correlations with API (2000) recommended design parameters.
4. Groundwater table was assumed at El. 7 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit end-bearing value for open-ended steel pipe piles is based on API (2000) recommendations.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. The p-multiplier provided takes into consideration of liquefaction effects. The residual shear strength of the liquefiable layer is estimated to be 0.2 ksf based on Idriss and Boulanger (2008).
11. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
ASRC B-176
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

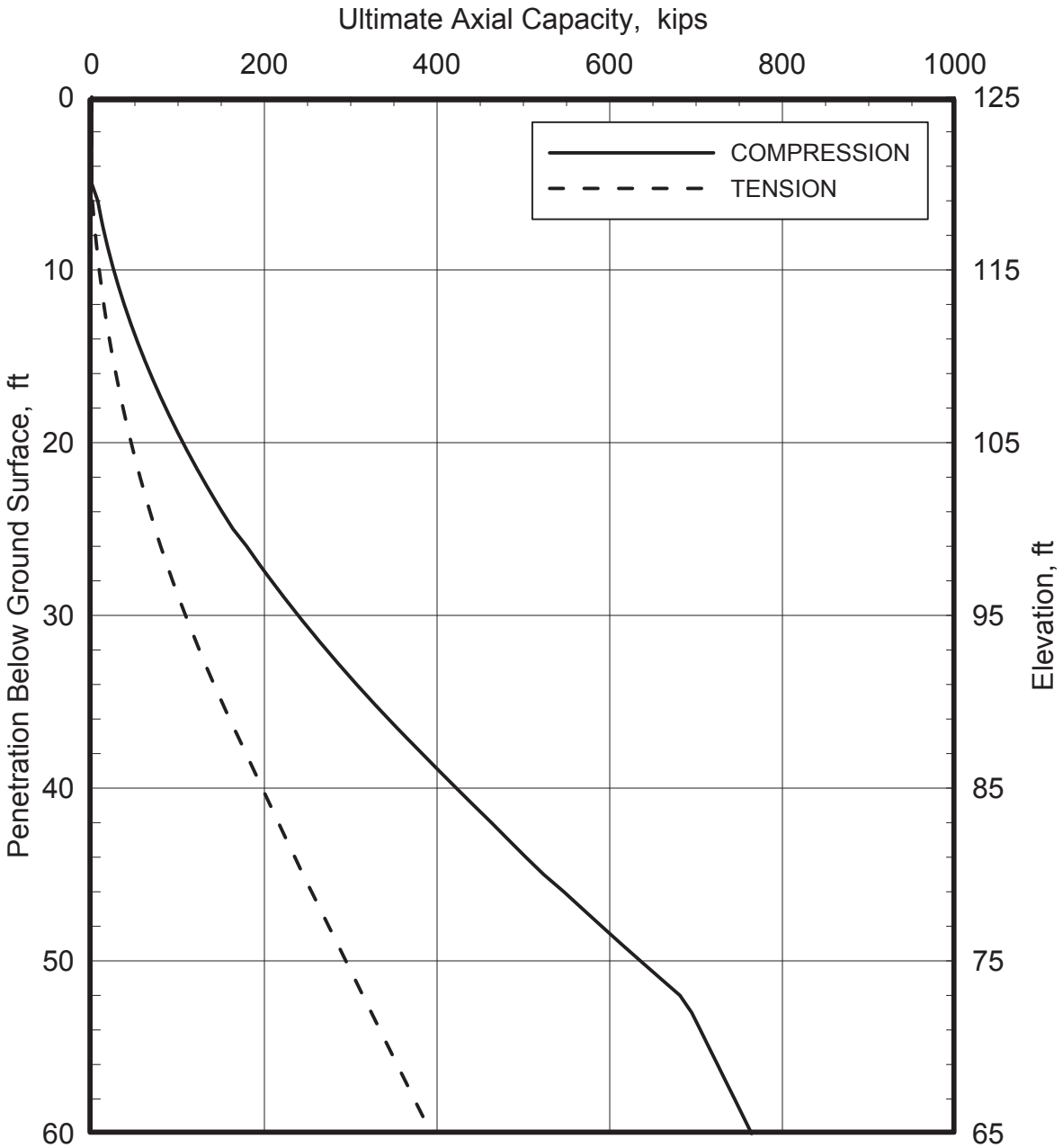
TABLE: PILE CAPACITY DESIGN PARAMETERS – ASRC B-183

Elevation, ft	Depth Below Finished Grade, ft	Soil Description	Effective Unit Weight,[4] pcf	Axial Capacity					Lateral Capacity					
				Undrained Shear Strength,[3] ksf	Friction Angle (Φ),[3] degrees	API (1986)[8] Limiting Skin Friction,[3][8] ksf	API (1986)[8] N_q[3]	API (1986)[8] Limiting End Bearing,[3][8] ksf	Soil Type	Undrained Shear Strength,[3] ksf	Friction Angle (Φ),[3] degrees	*P*-Multiplier [11]	Subgrade Modulus,[7] pcf	ε_{50} [12]
99[2] to 97	0 to 2	Existing Fill or Medium Dense Sand with Gravel	118	Neglected to account for disturbance and freeze-thaw conditions[5]					Neglected to account for disturbance and freeze-thaw conditions[6]					
97 to 94	2 to 5	Existing Fil or Medium Dense Sand with Gravel	118						API Sand	-	25[6]	1.0	0.35×10^5 [6]	-
94 to 85	5 to 14	Medium Dense to Dense Silty and Gravelly Sand	118	-	34	1.9	36	180	API Sand	-	34	1.0	1.68×10^5	-
85 to 60	14 to 39	Dense Silty and Gravelly Sand and Sand	125	-	36	2.1	42	210	API Sand	-	36	1.0	2.5×10^5	-
60 to 15	39 to 84	Dense Silty and Gravelly Sand and Sand	62.6	-	36	2.1	42	210	API Sand	-	36	1.0	1.58×10^5	-
15 to -1	84 to 100	Dense to Very Dense Silty Sand	65.6	-	37	2.2	44	220	API Sand	-	37	1.0	1.78×10^5	-

NOTES:

1. This table presents design parameters used for ultimate axial and lateral capacity.
2. Final grade was assumed at about El. 99 ft.
3. Selection of friction angles and other engineering parameters are based on SPT (N) values of pertinent boreholes performed by Fugro (2014, 2015, 2016). Recommended design parameters correlated with API (1986) and FHWA (1988) are used for open-ended steel pipe piles and cast-in-place concrete piles, respectively.
4. Groundwater table was assumed at El. 60 ft.
5. For axial capacity in compression and tension, we neglected soil strength in the upper 5 feet below final grade to account for construction disturbance and freeze-thaw conditions.
6. For lateral capacity, we neglected soil strength only in the upper 2 feet below final grade and reduced soil strength from 2 feet to 5 feet below final grade as the pile is engaged by soil with small lateral movement.
7. Subgrade modulus values for lateral capacity analyses are based on API recommendations, and are correlated with the assumed friction angles.
8. The limiting unit skin friction and the limiting unit end bearing values are based on API (1986) and FHWA (1988). Note that this table presents the API (1986) limiting design parameters. FHWA (1988) limiting unit skin friction is equal to 4.2 ksf. The FHWA (1988) limiting unit end bearing is limited to the capacity that is achieved once the downward movement of the pile tip reaches 5% of the pile base diameter.
9. For steel pipe piles, API recommends using coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.8.
10. For cast-in-place concrete piles, coefficients of lateral earth pressure for compression (kc) and tension (kt) equal to 0.5 were used based on Fugro's experience.
11. The p-multiplier was not reduced as potential for liquefaction evaluated to be not significant.
12. Strains at 50% of maximum stress for lateral capacity analyses are based on the recommendation of L-Pile 6 (2010), and are correlated with the estimated undrained shear strength.

PILE CAPACITY DESIGN PARAMETERS
ASRC B-183
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



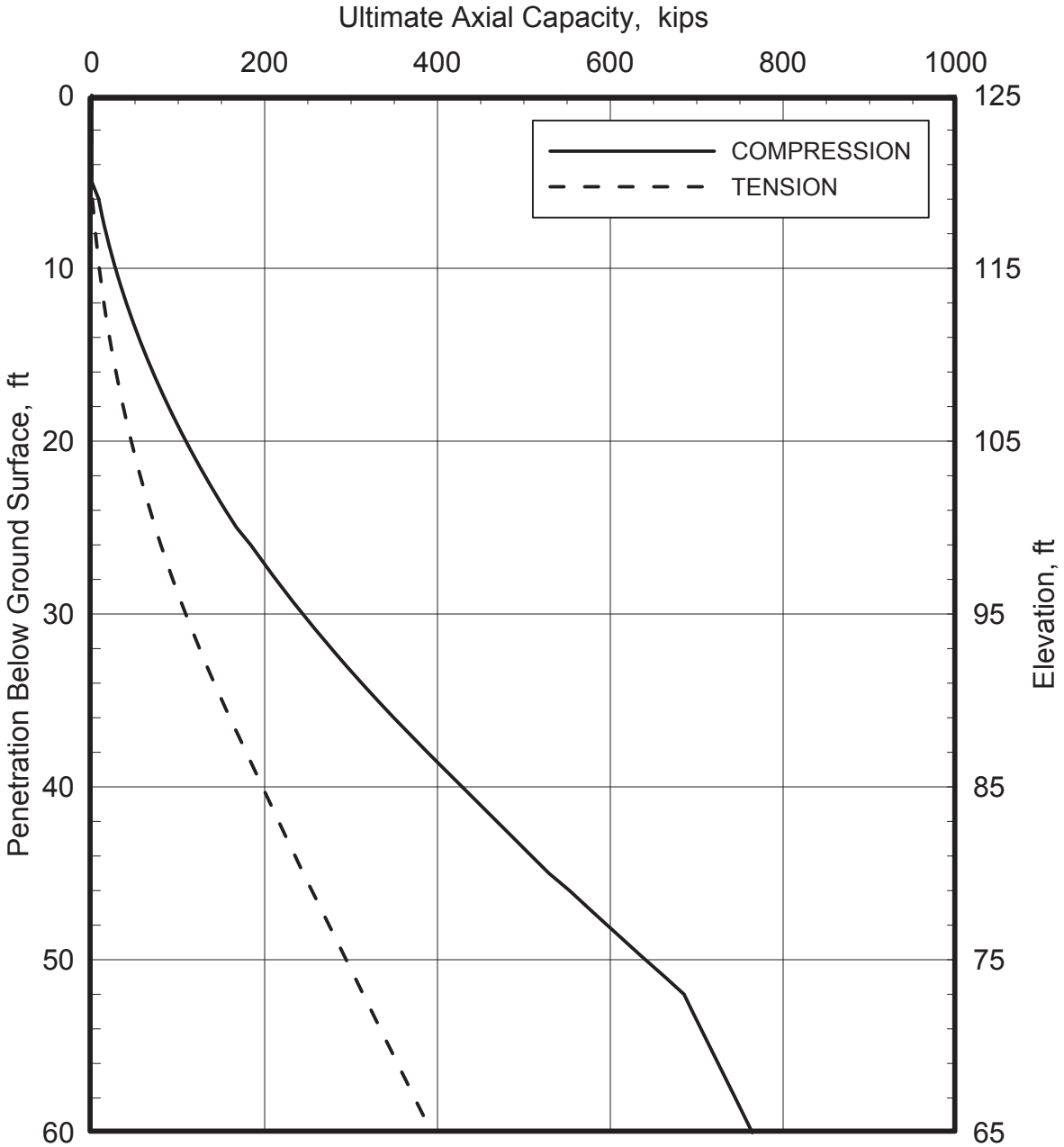
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 2

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



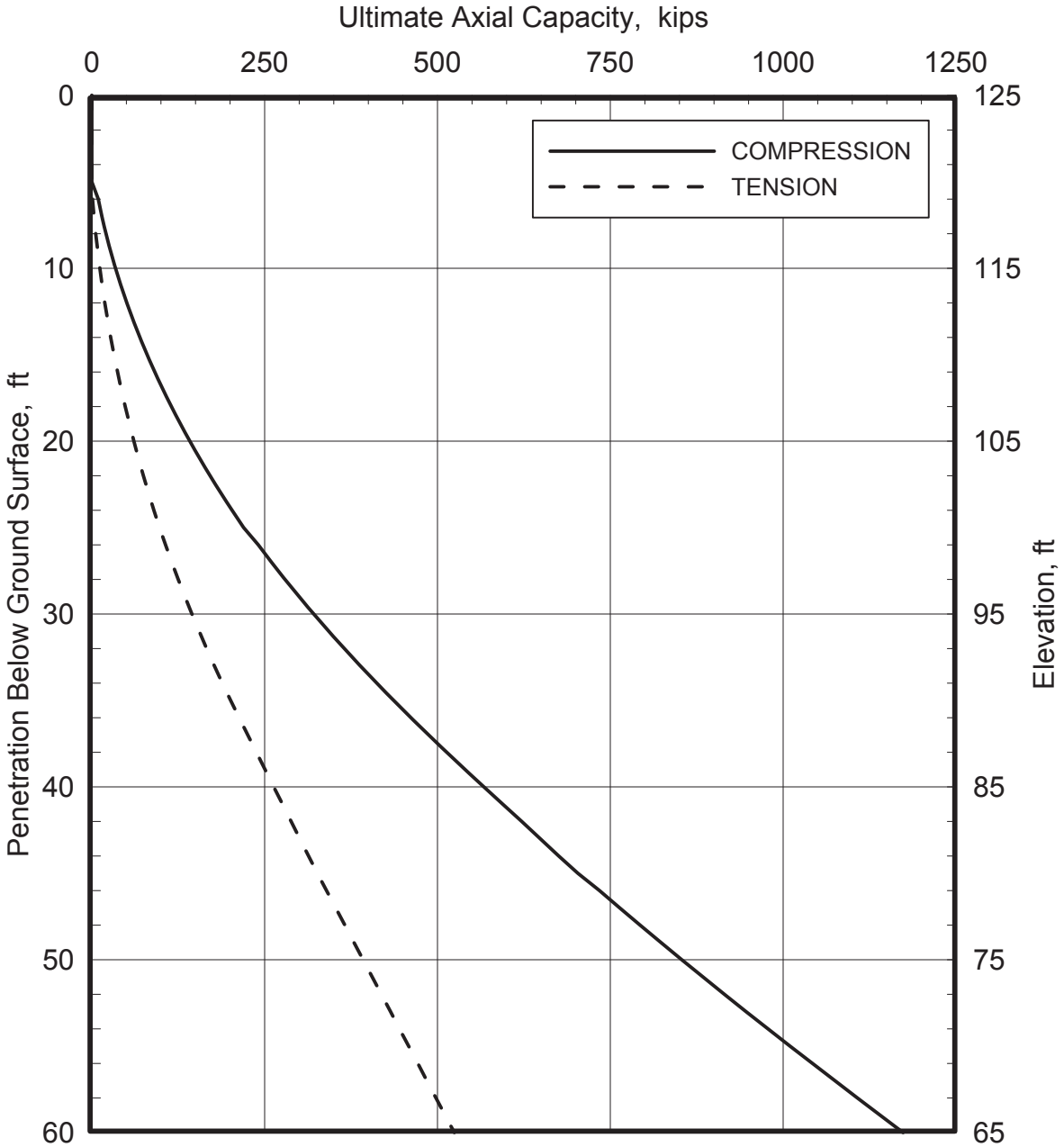
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 2

18-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



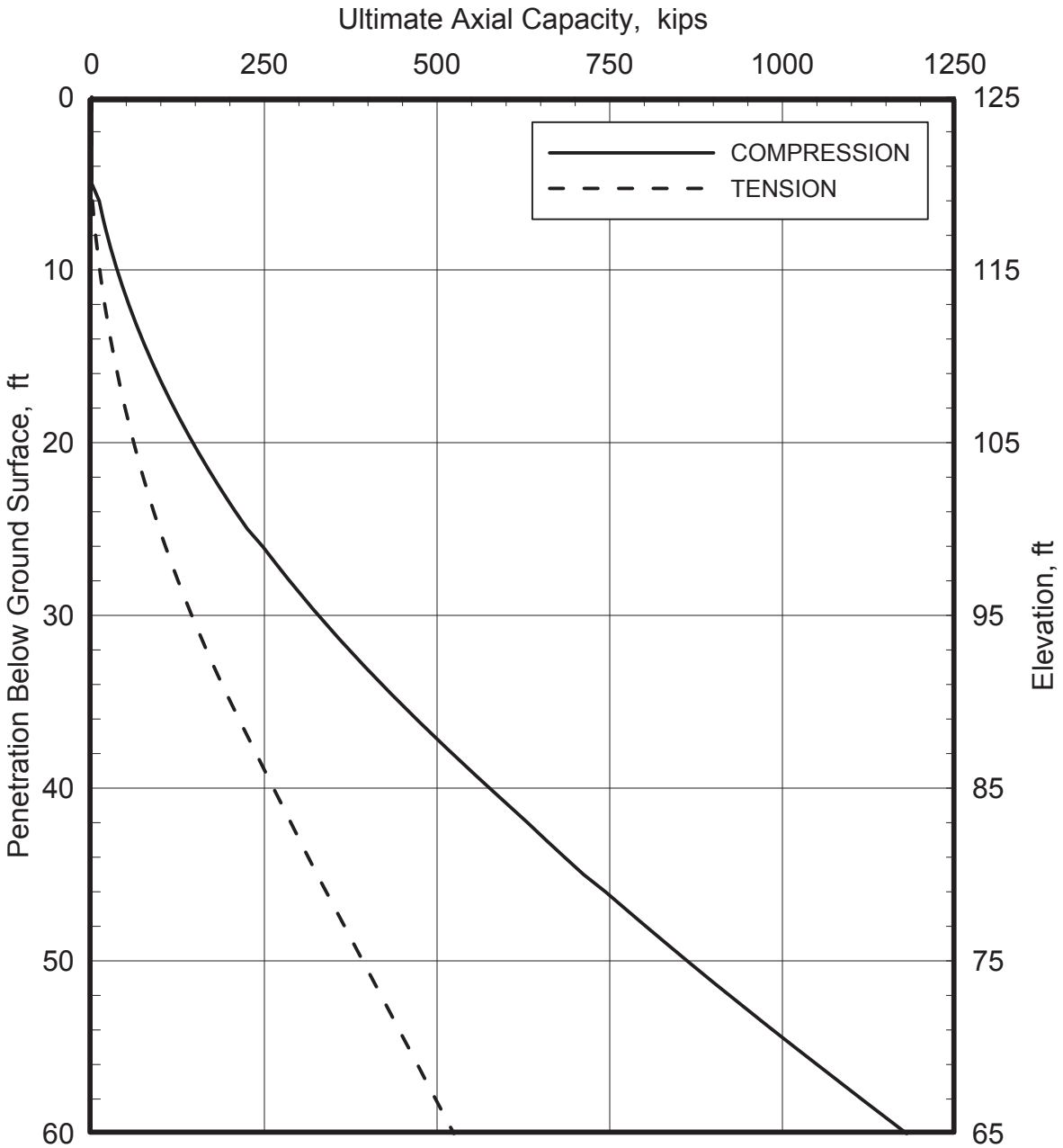
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 2

24-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



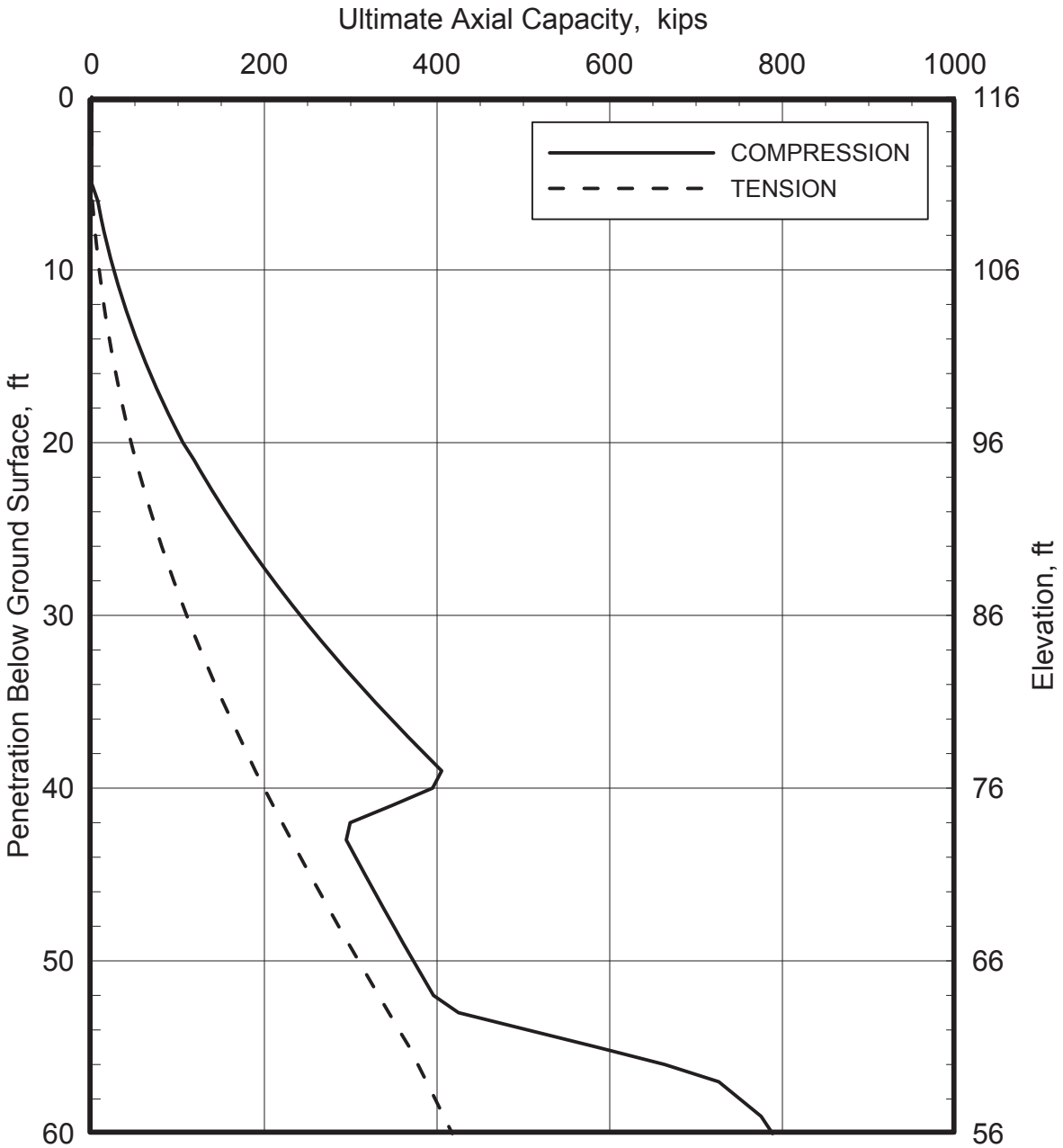
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 2

24-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



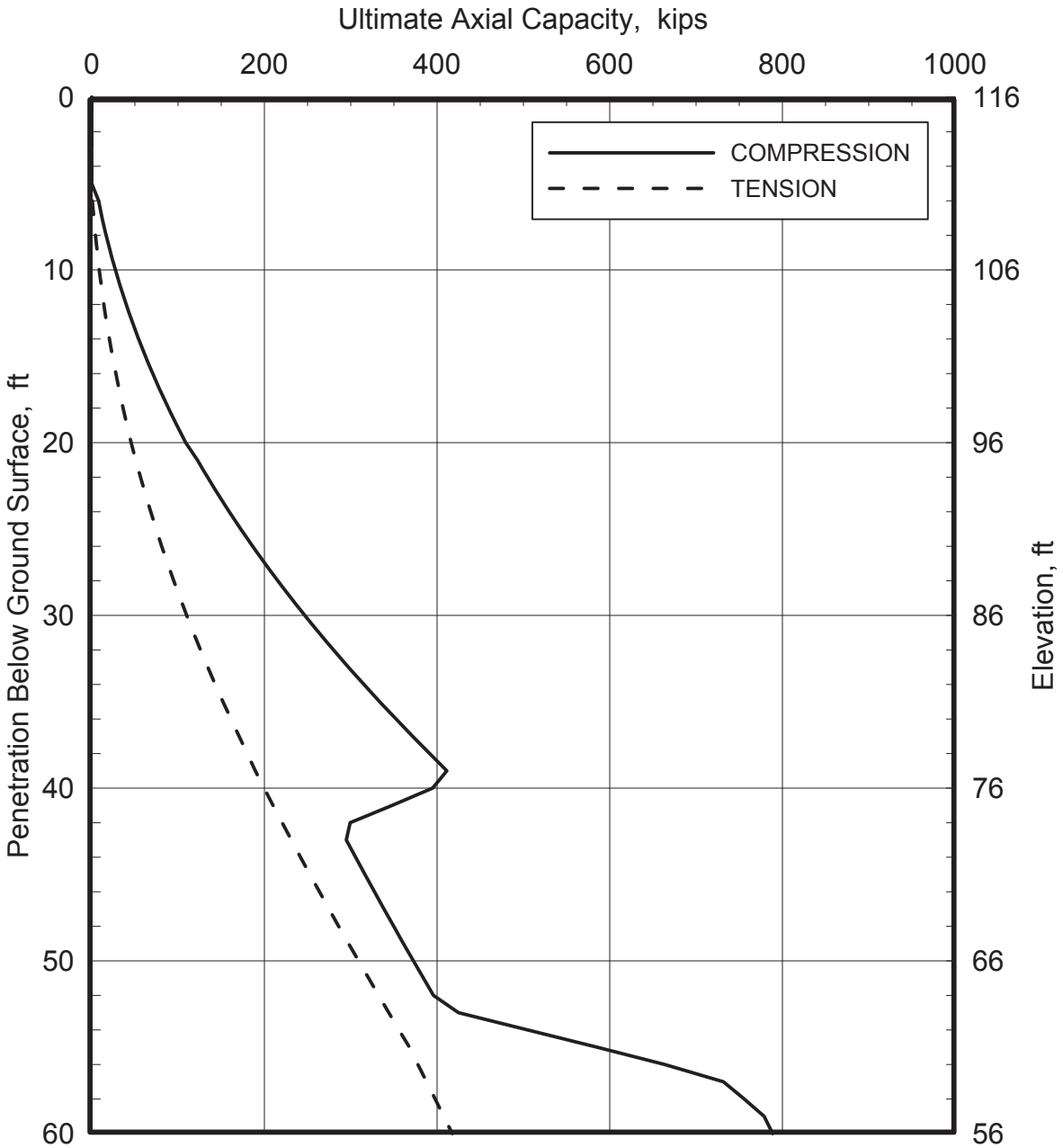
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 3

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



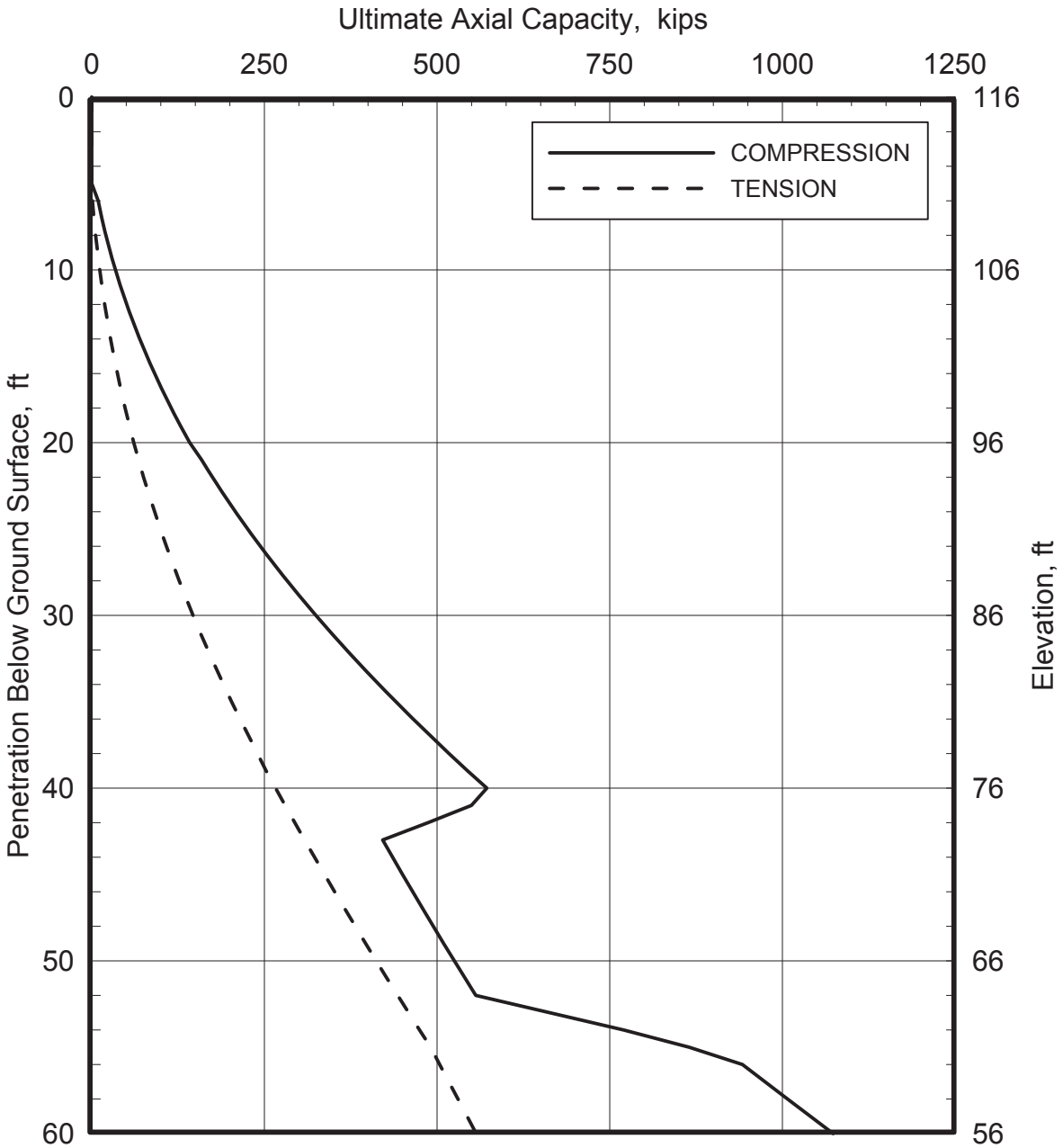
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 3

18-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



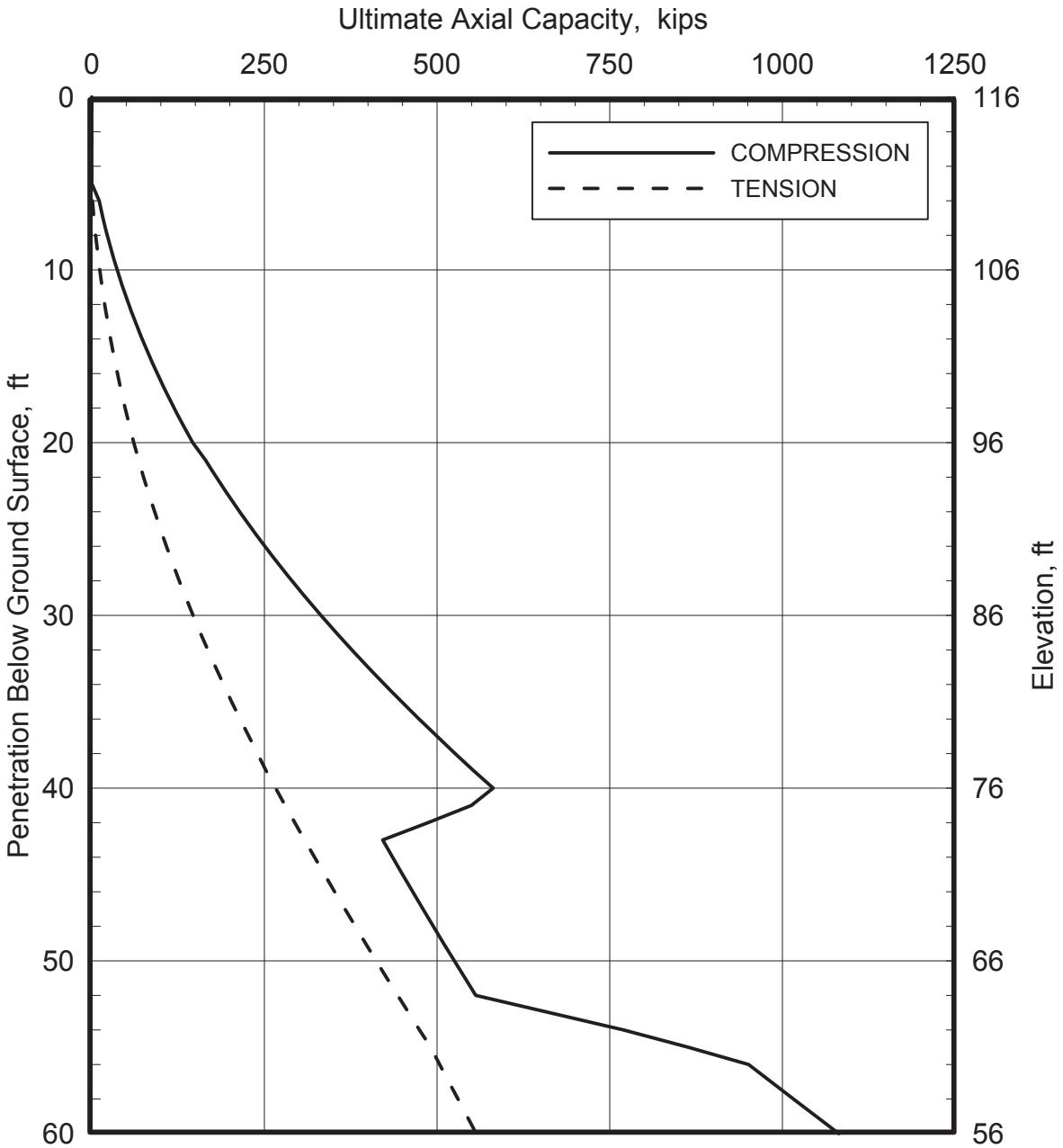
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 3

24-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



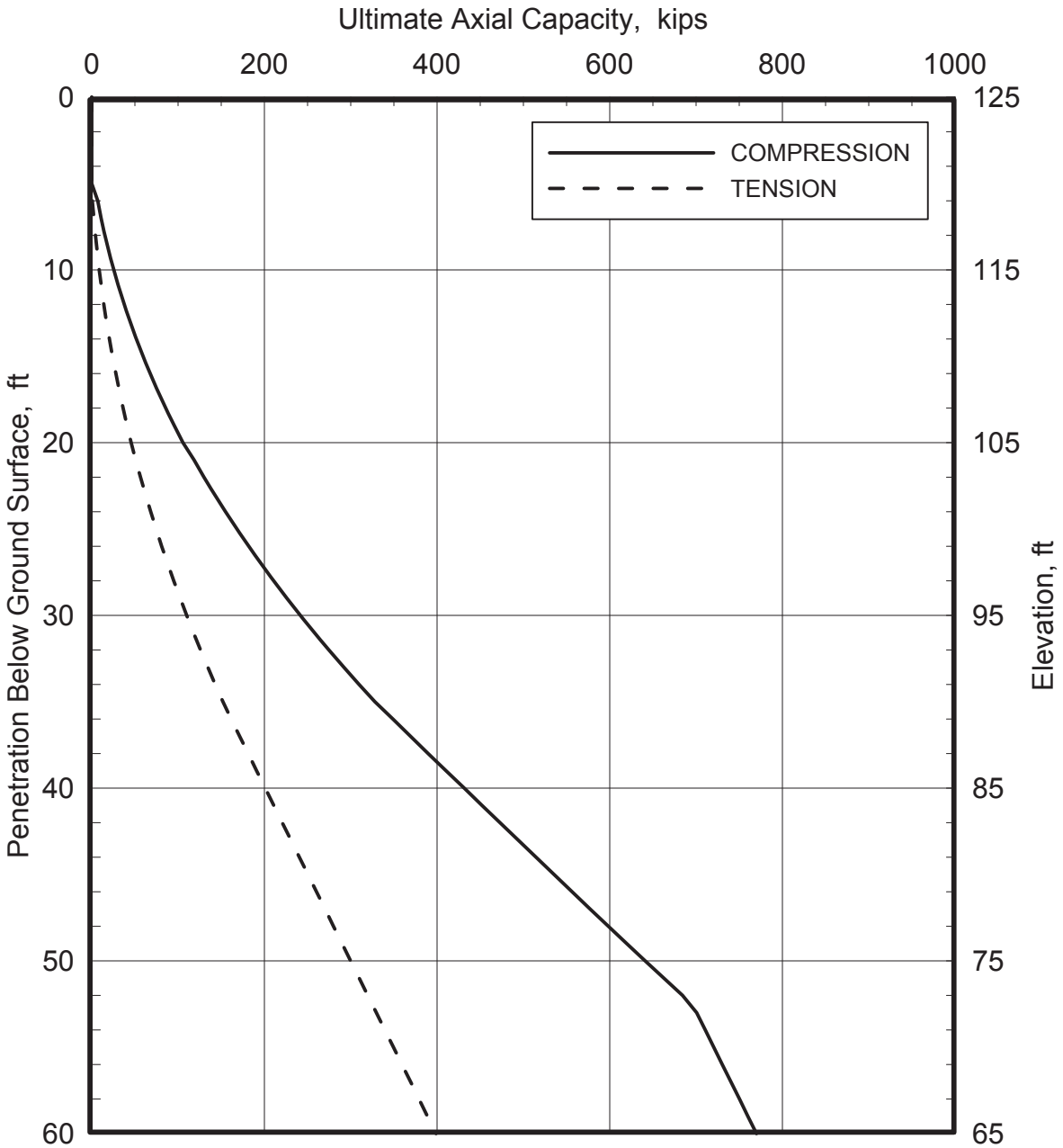
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 3

24-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



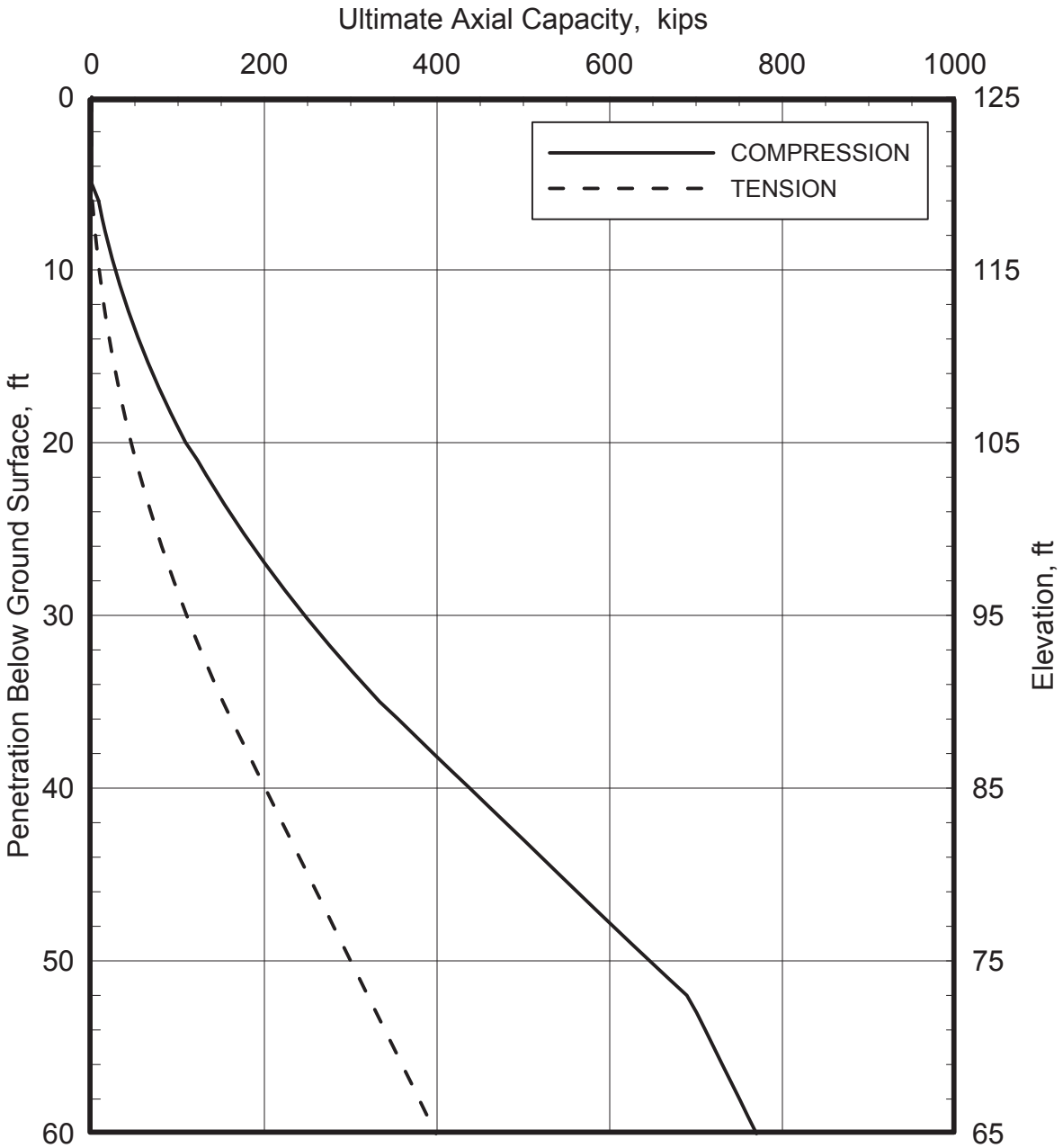
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 4

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



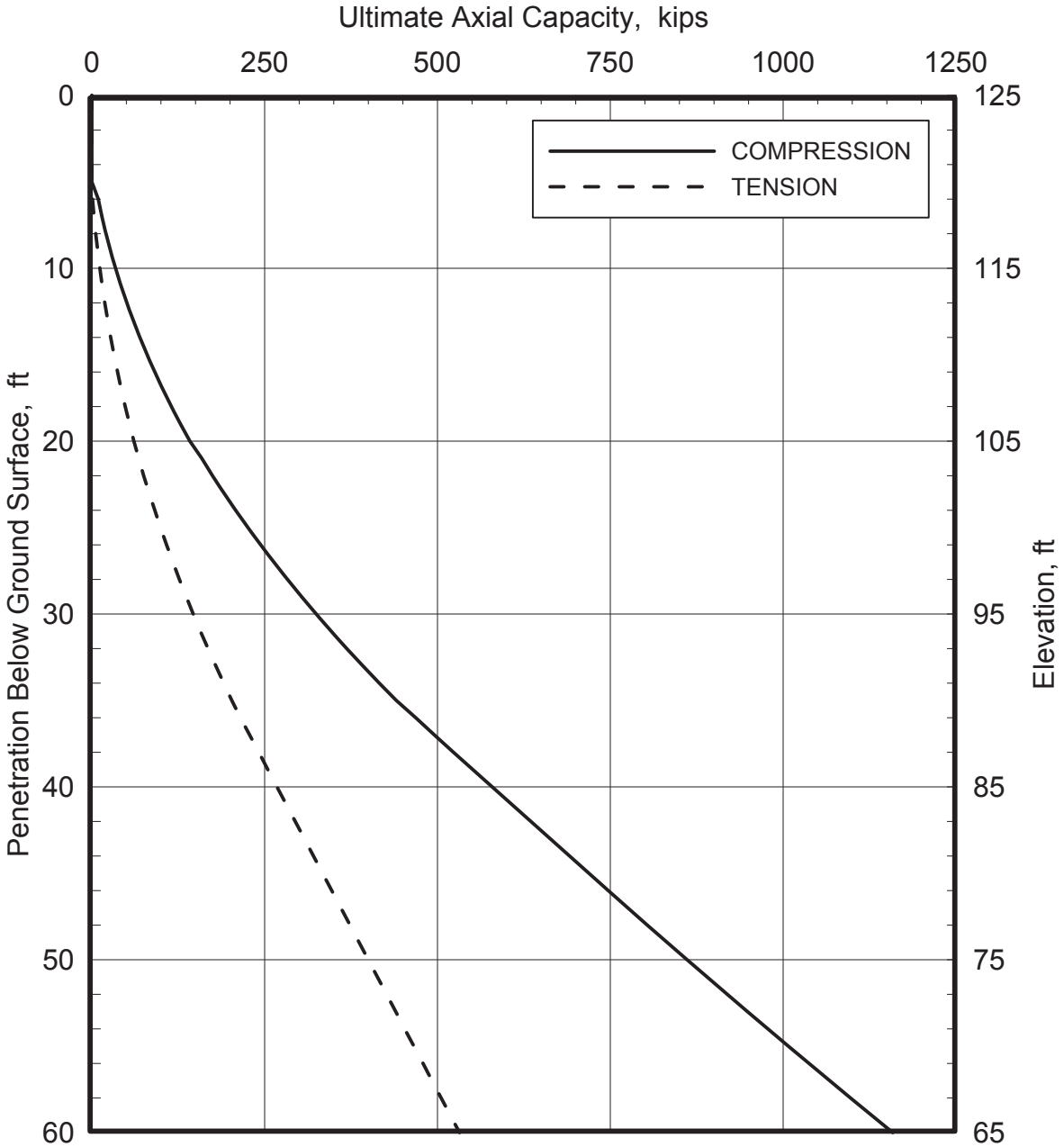
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 4

18-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



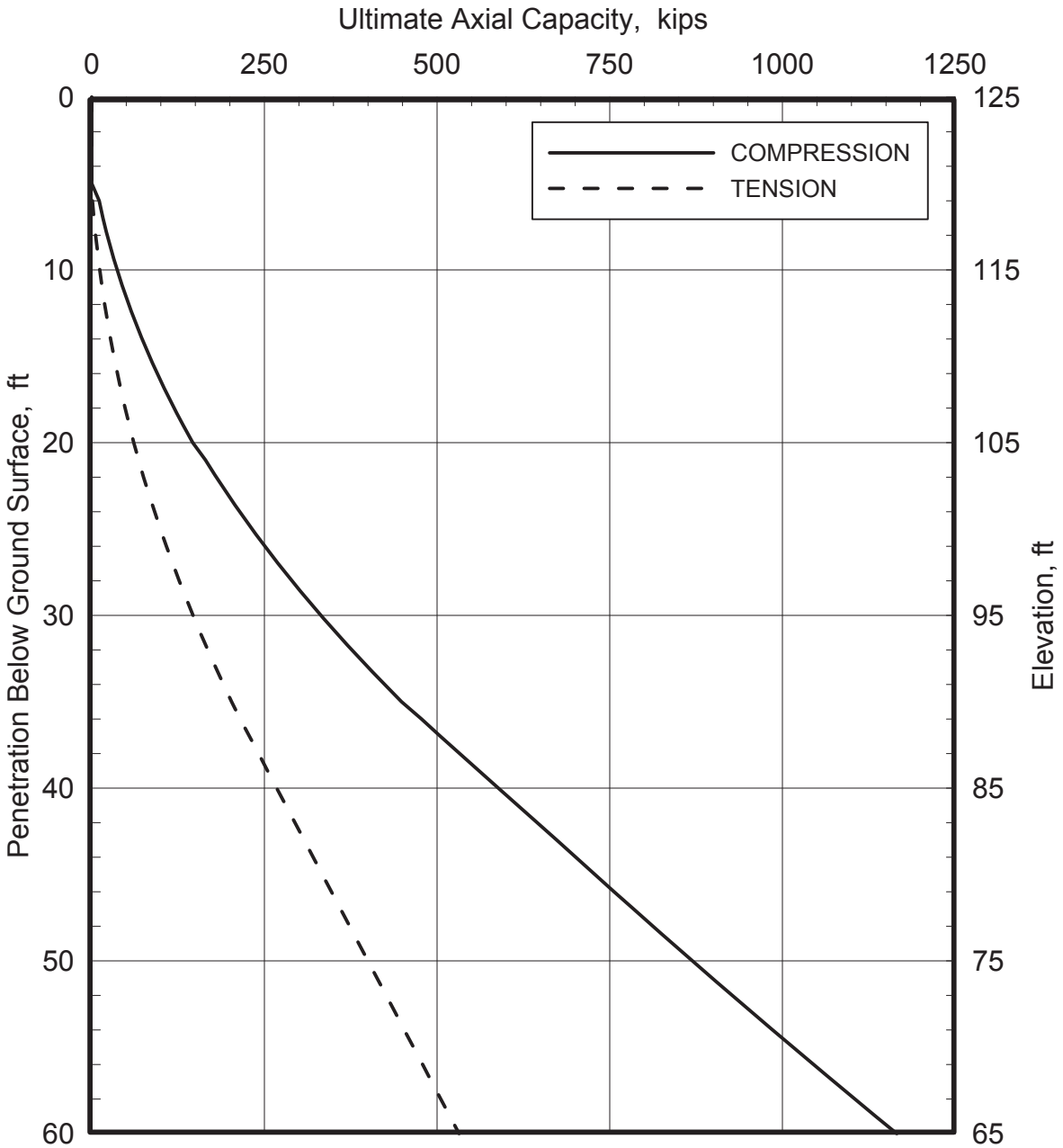
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 4

24-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



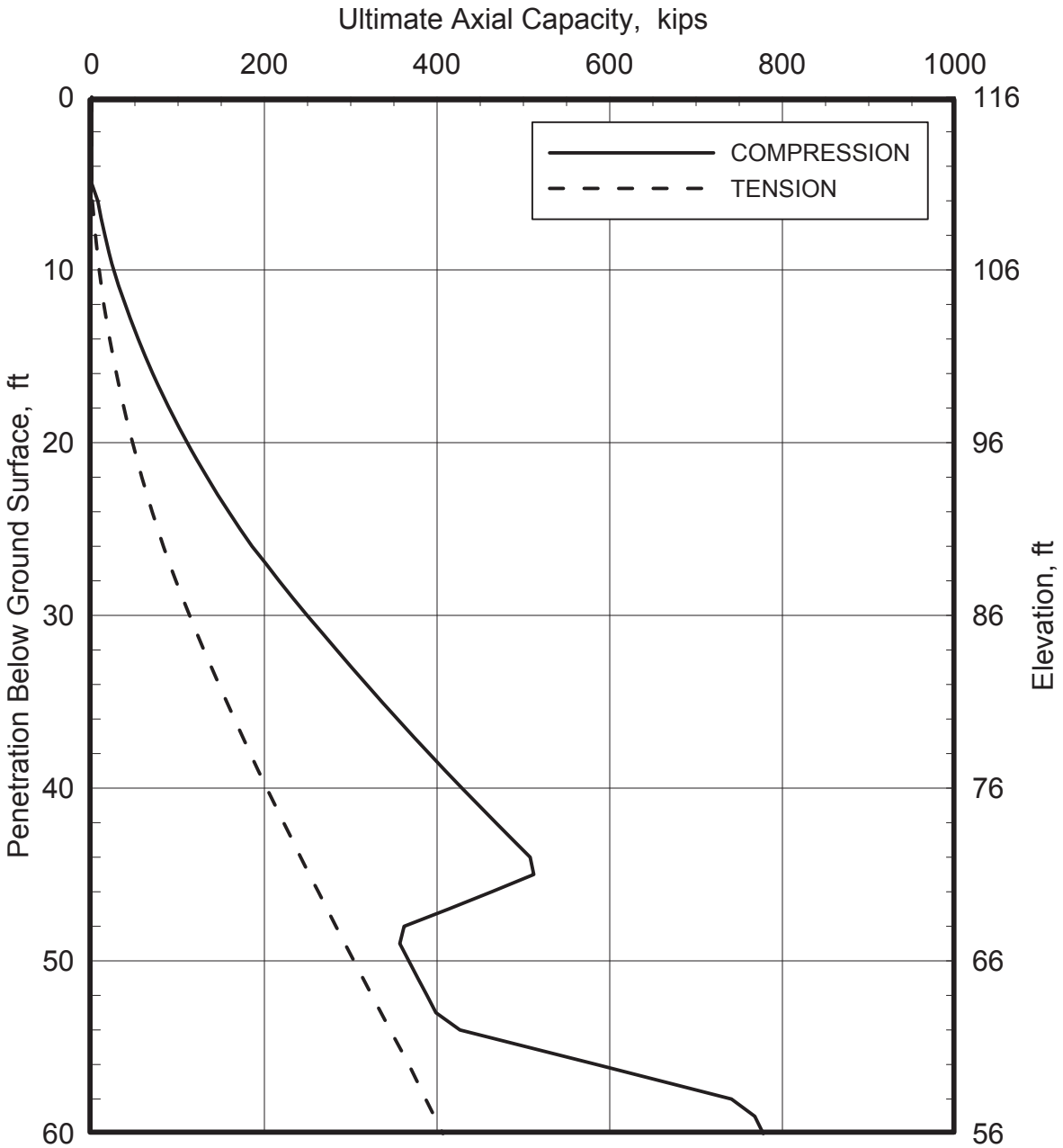
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 4

24-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



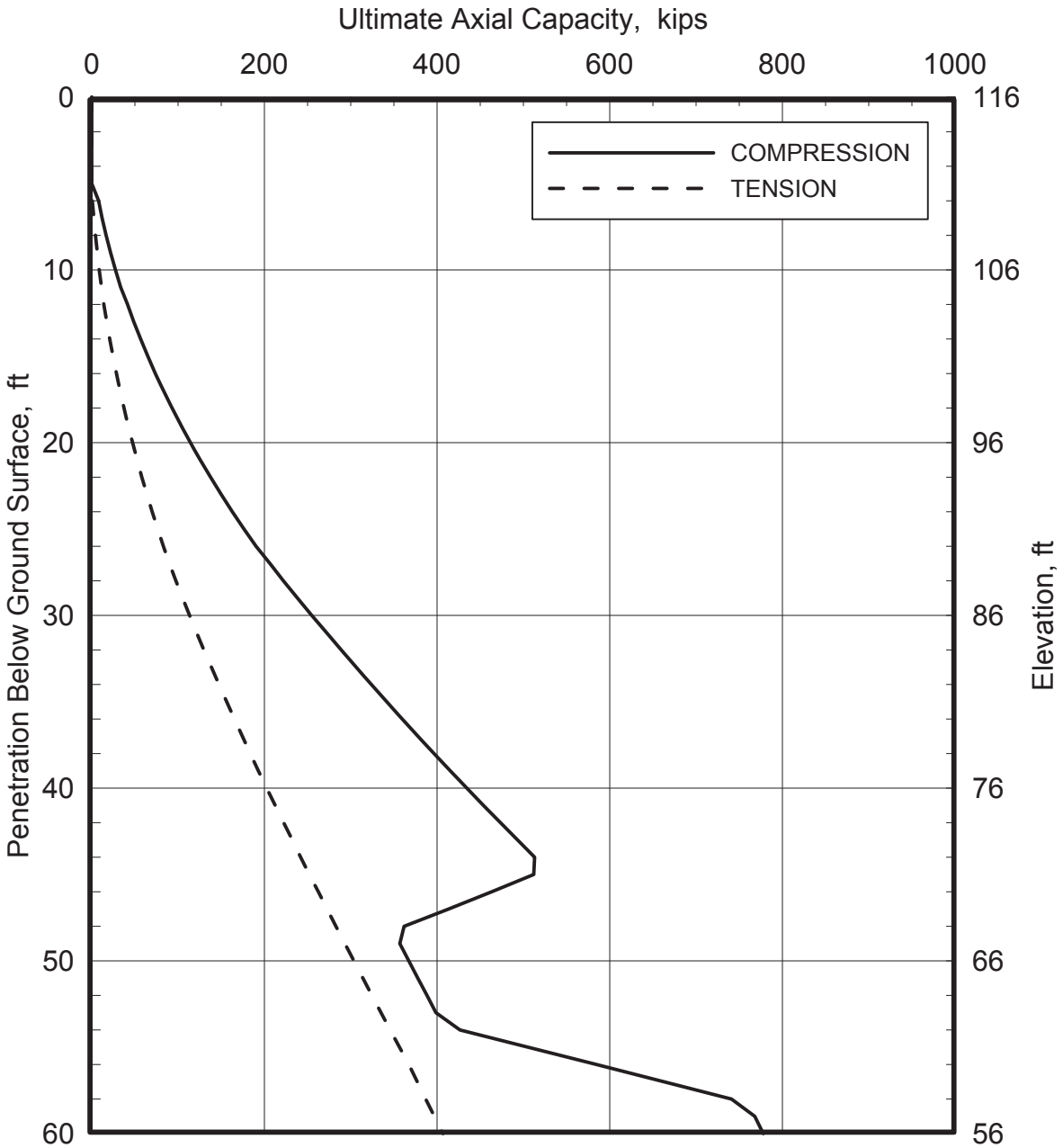
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 5

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



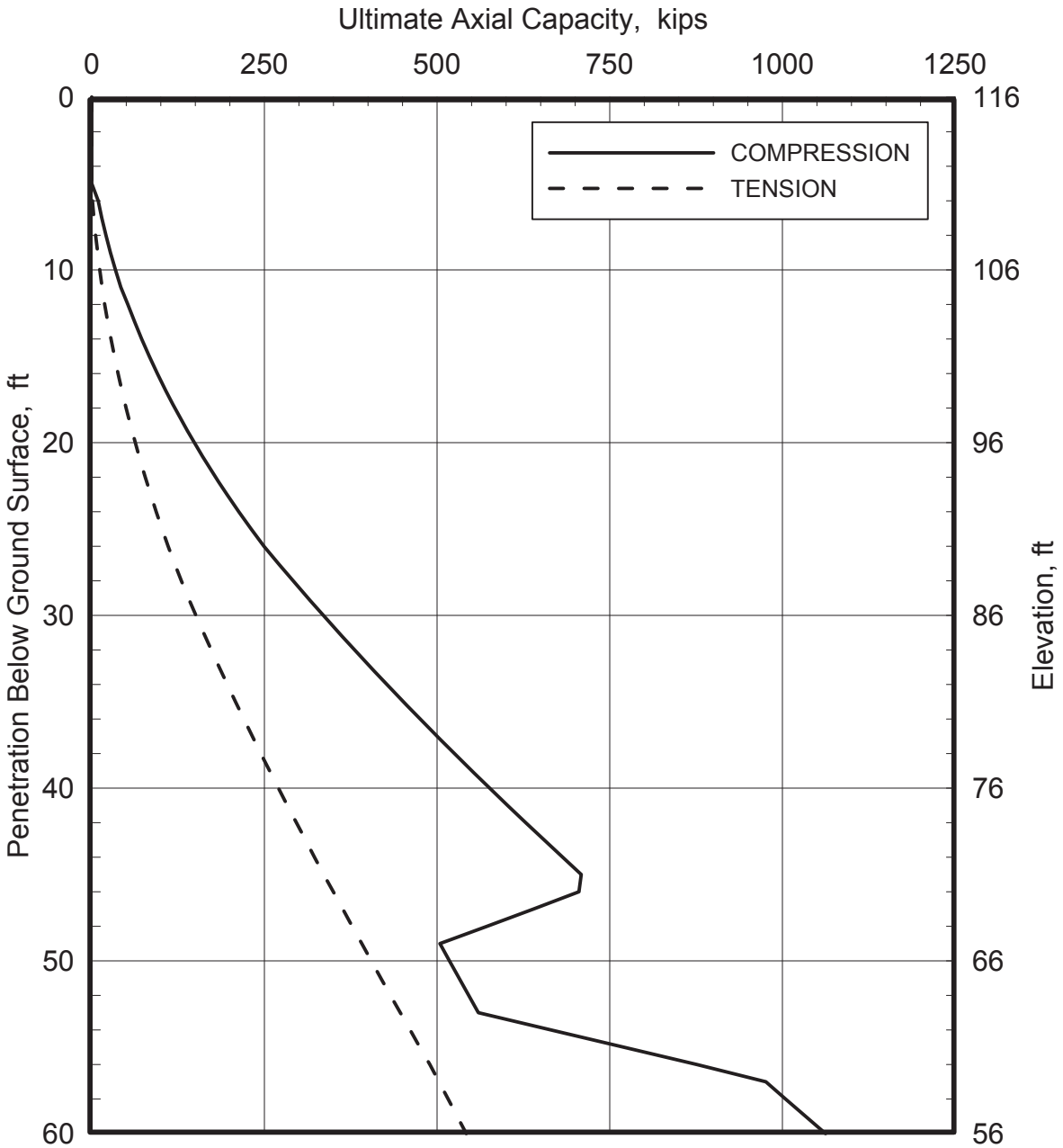
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 5

18-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



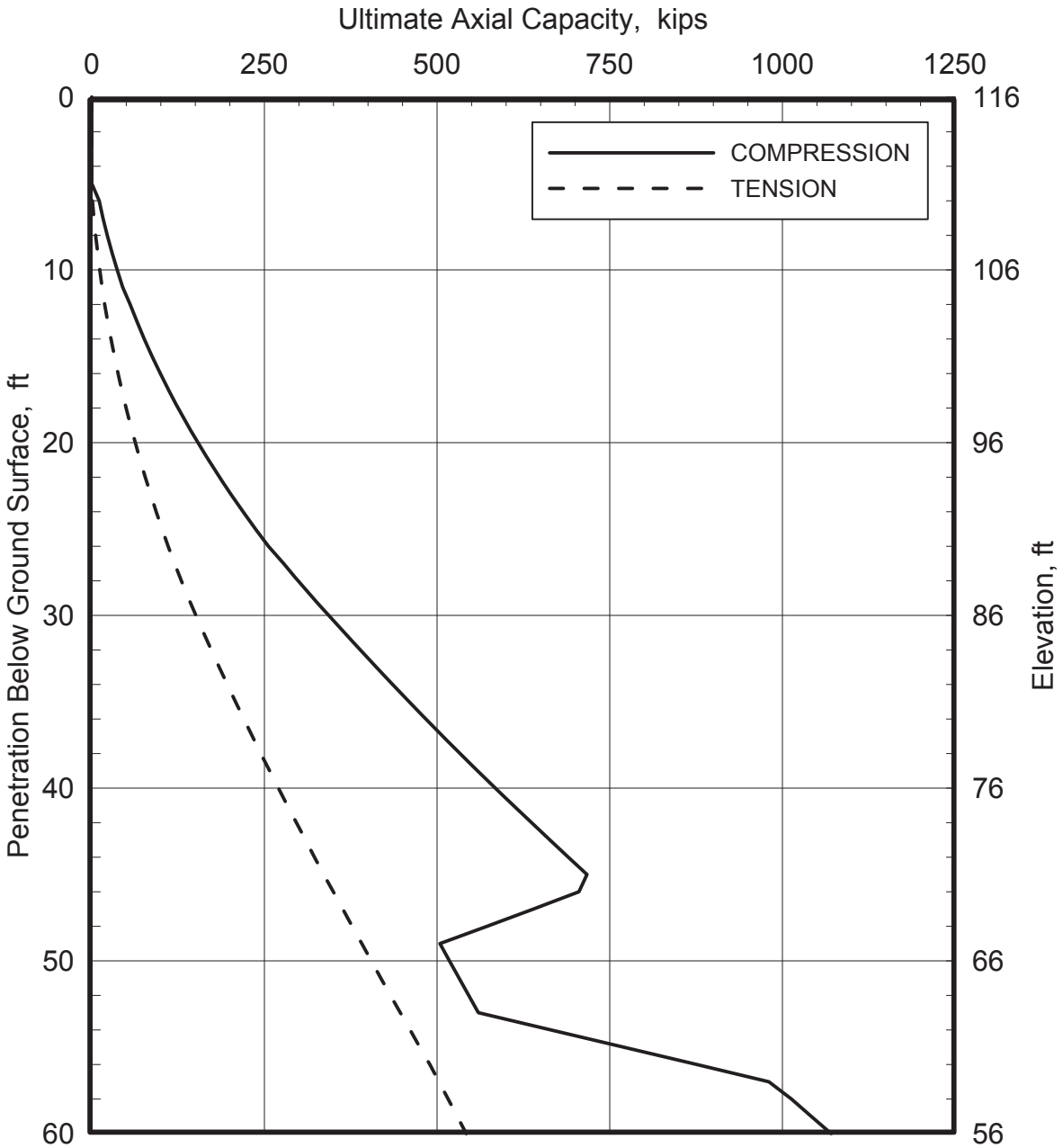
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 5

24-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



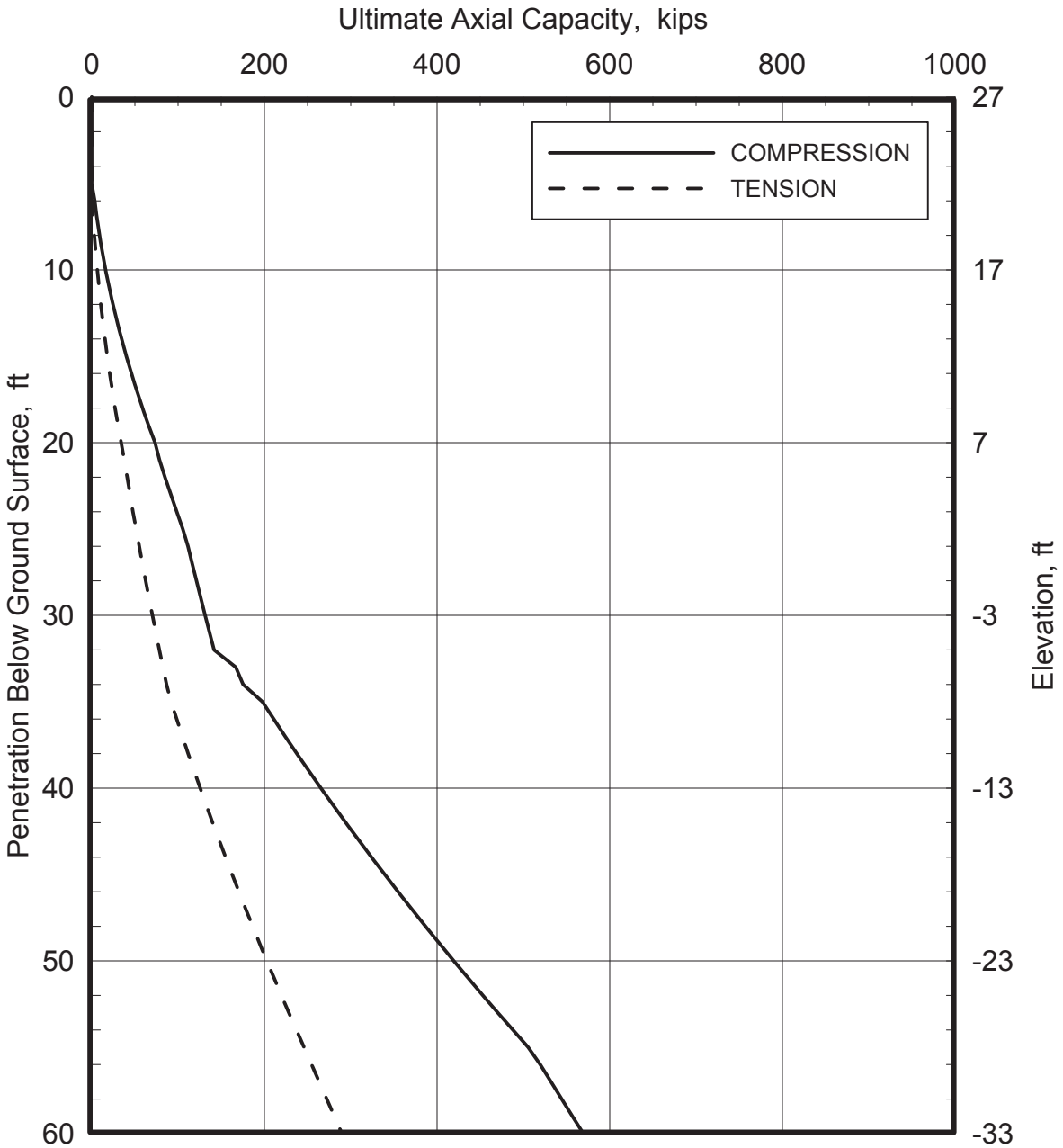
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 116 ft. Groundwater level was assumed at El. 85 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 5

24-IN DIAMETER (0.625-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



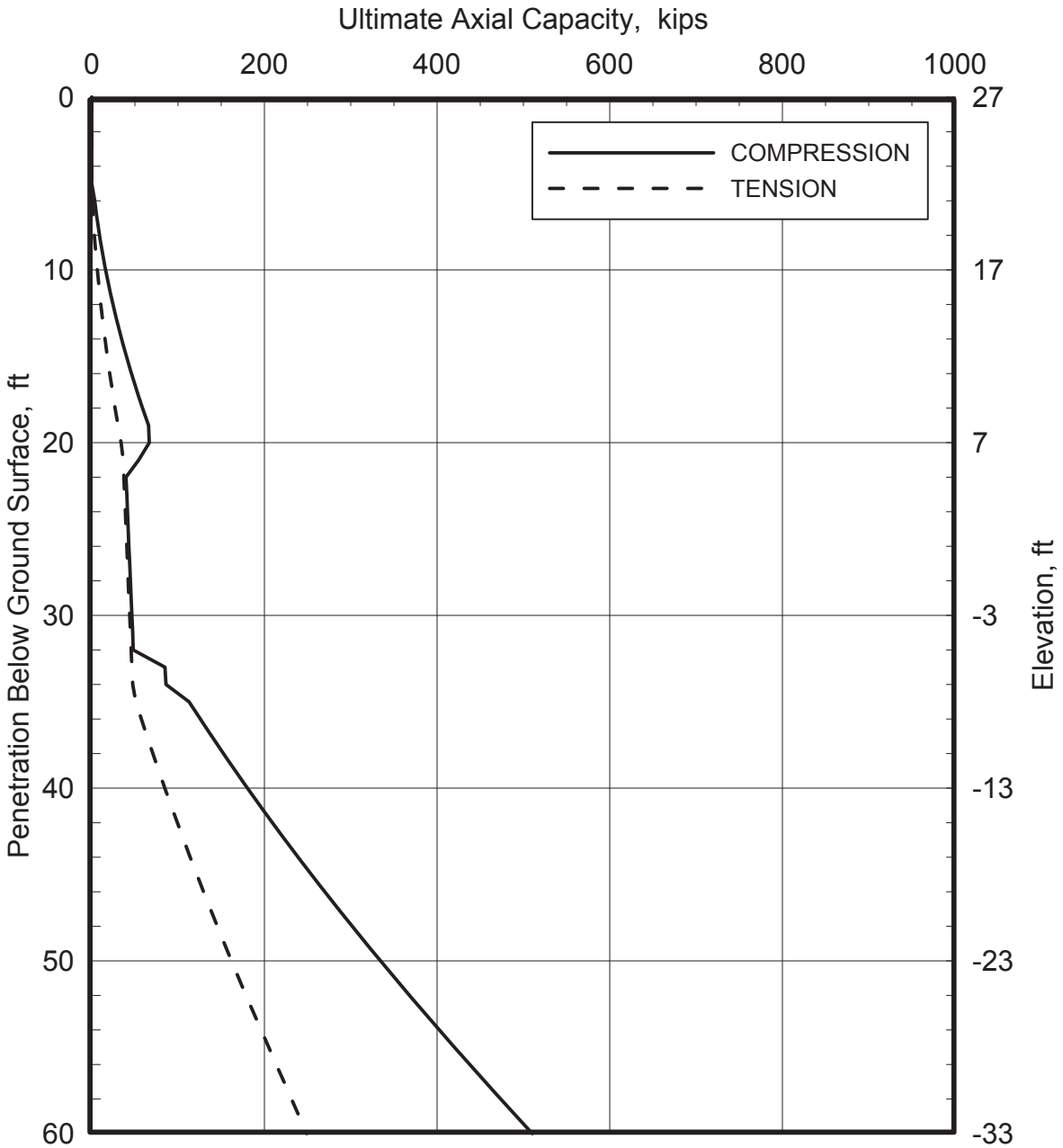
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 27 ft. Groundwater level was assumed at El. 20 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-176)

16-IN DIAMETER (0.375-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



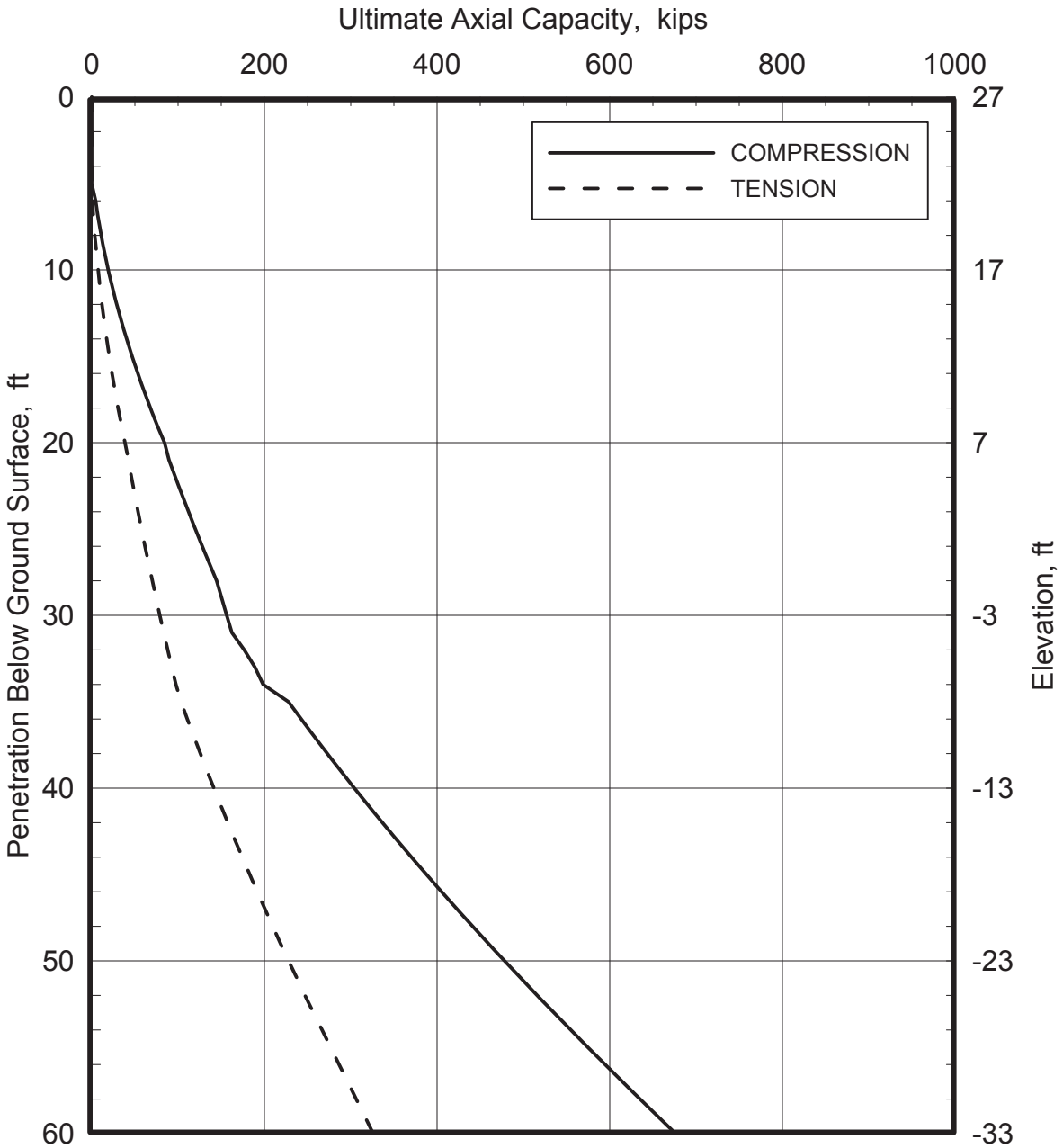
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-176) - SEISMIC CONDITIONS

16-IN DIAMETER (0.375-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



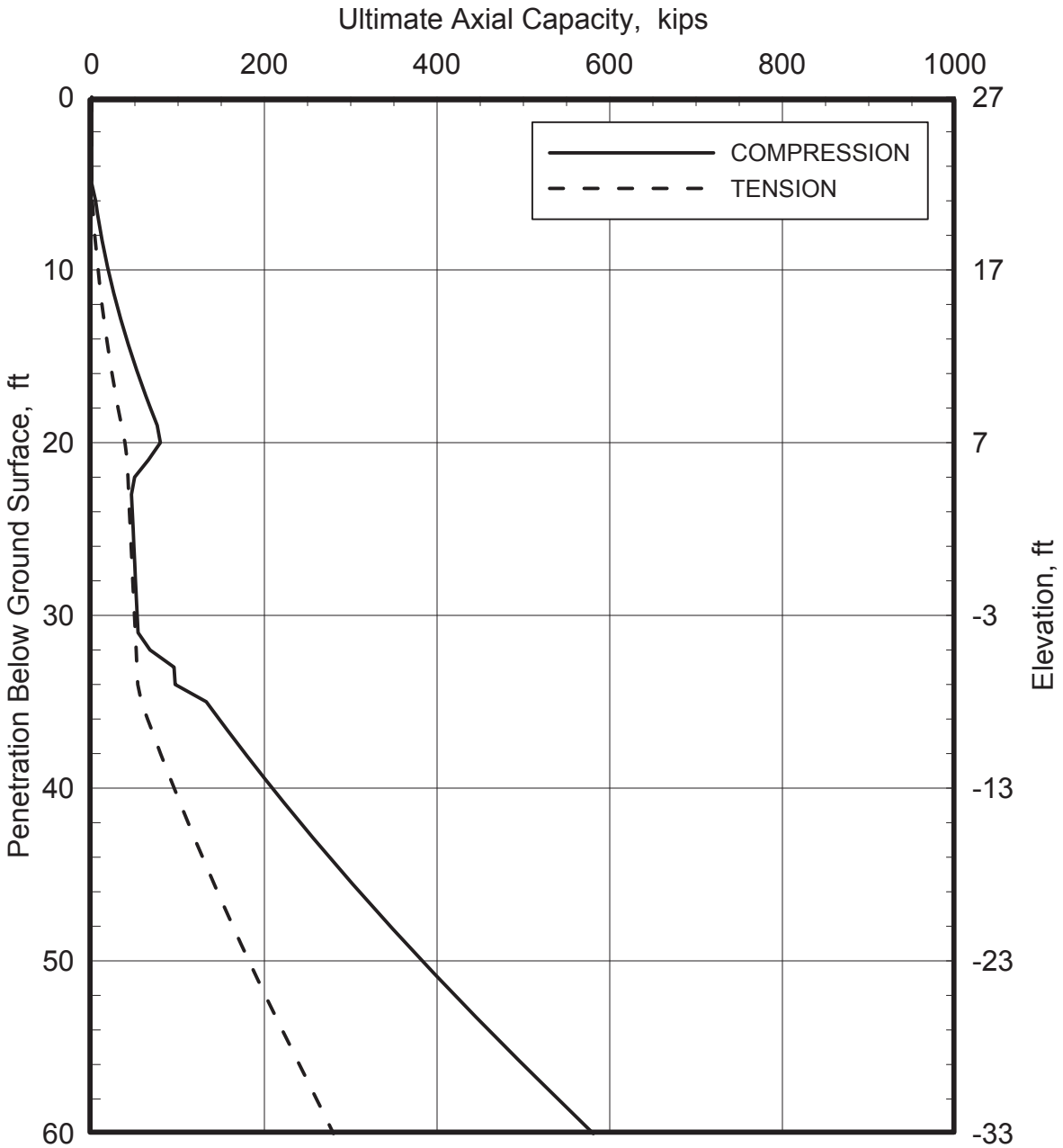
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 27 ft. Groundwater level was assumed at El. 20 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-176)

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



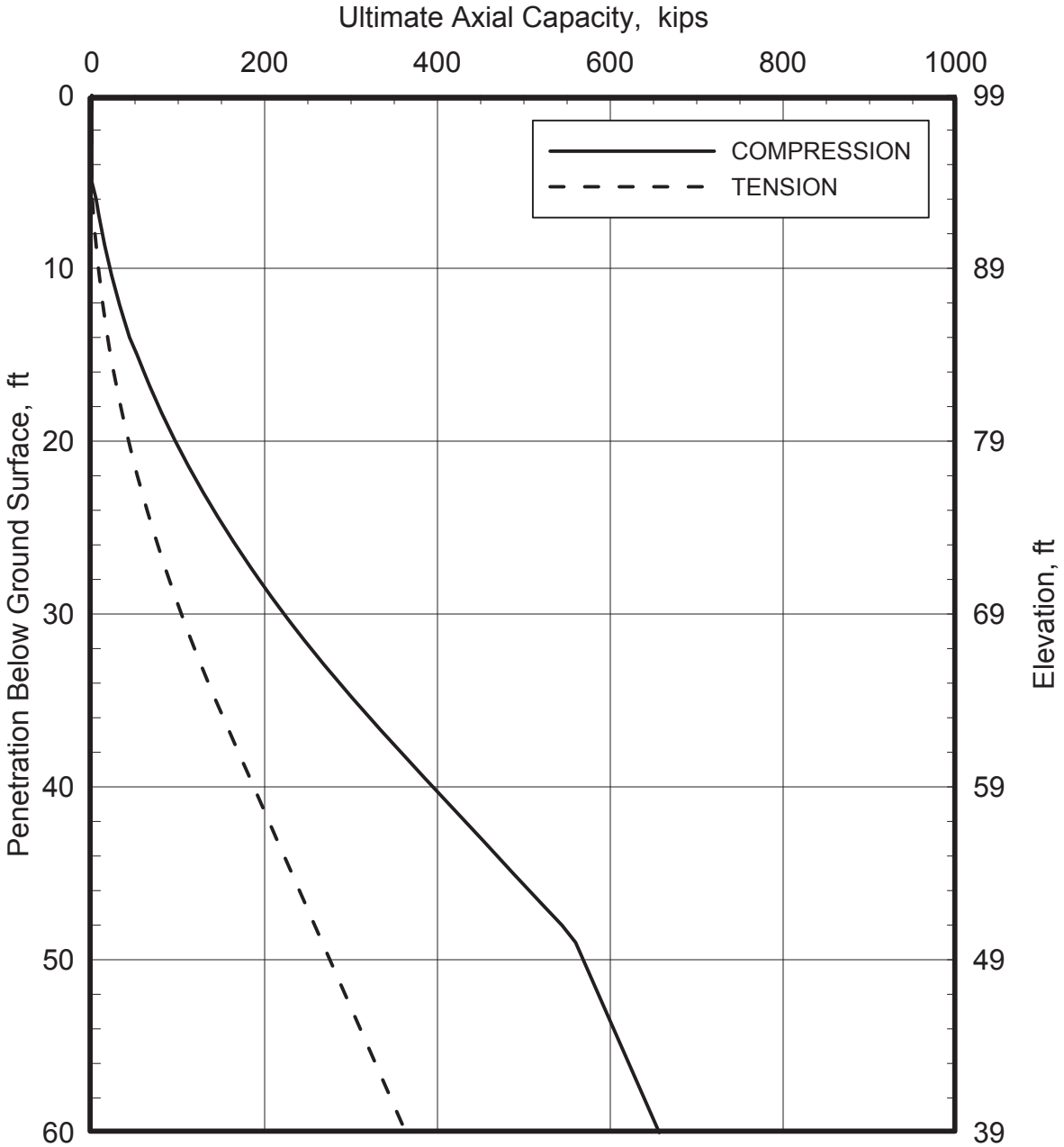
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-176) - SEISMIC CONDITIONS

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



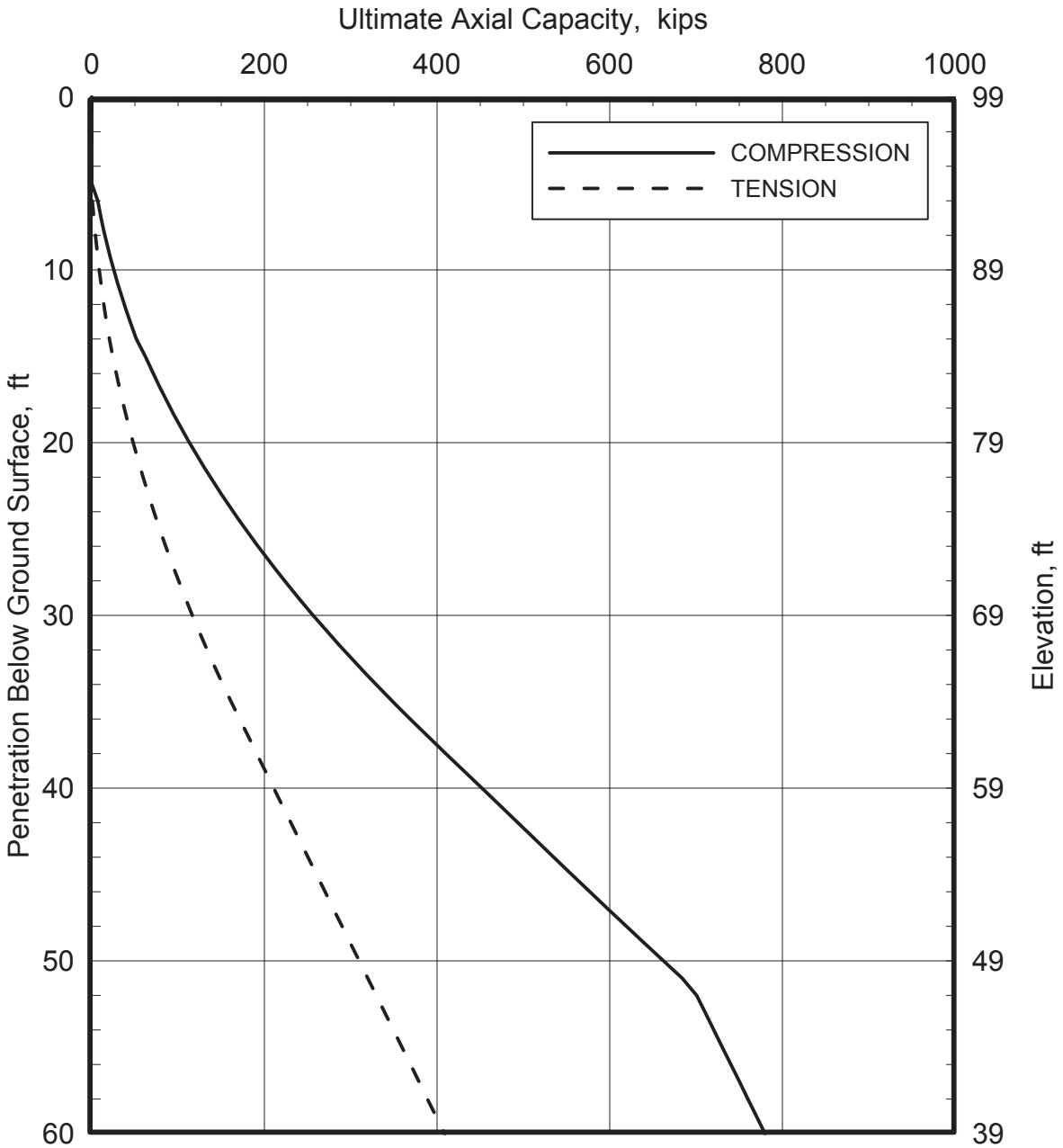
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 99 ft. Groundwater level was assumed at El. 39 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-183)

16-IN DIAMETER (0.375-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



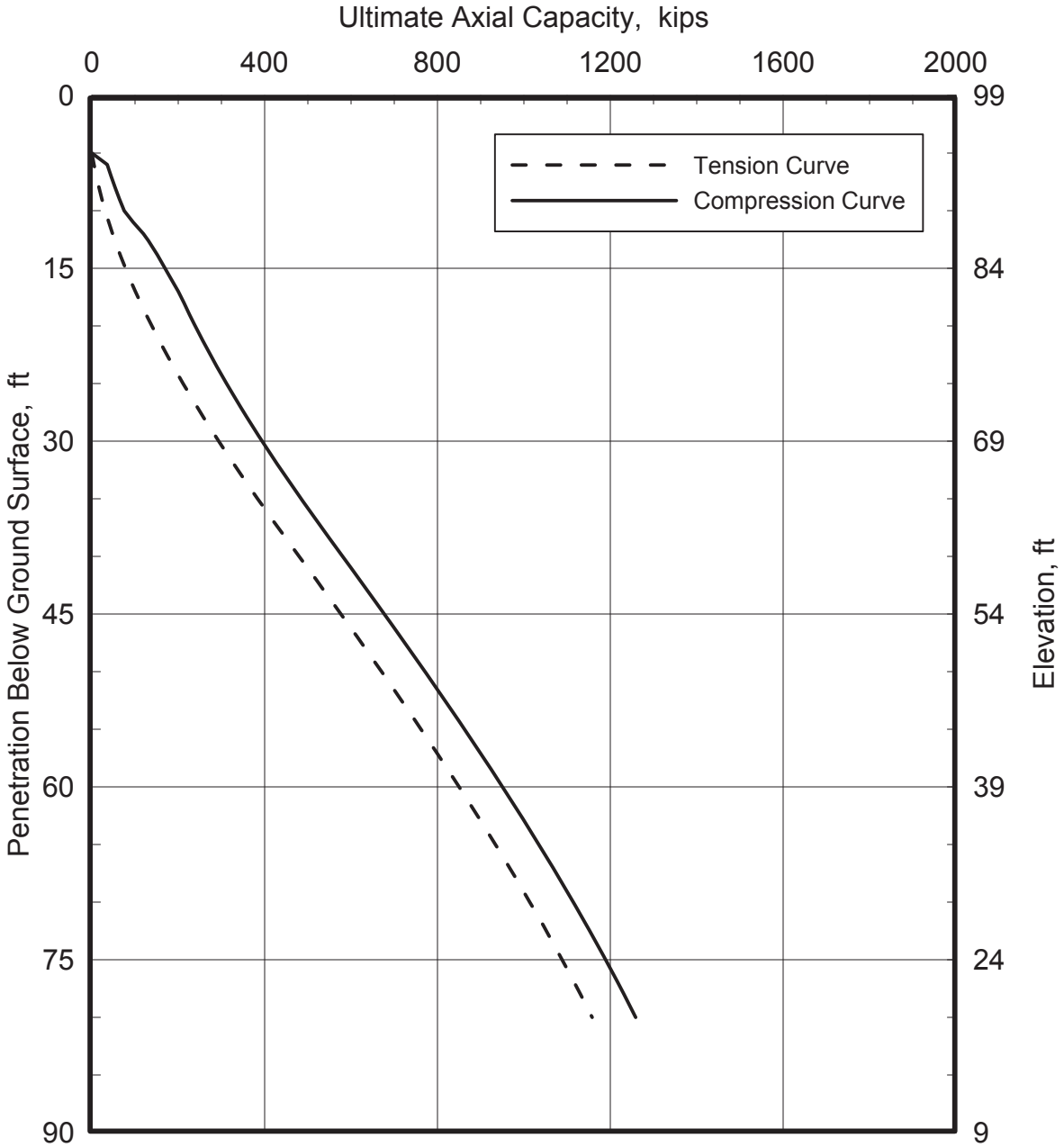
NOTES:

1. These curves represent ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weights of the pile and soil plug are not included in the computations.

ULTIMATE AXIAL CAPACITY - AREA 8 (B-183)

18-IN DIAMETER (0.5-IN-THICK WALL) OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



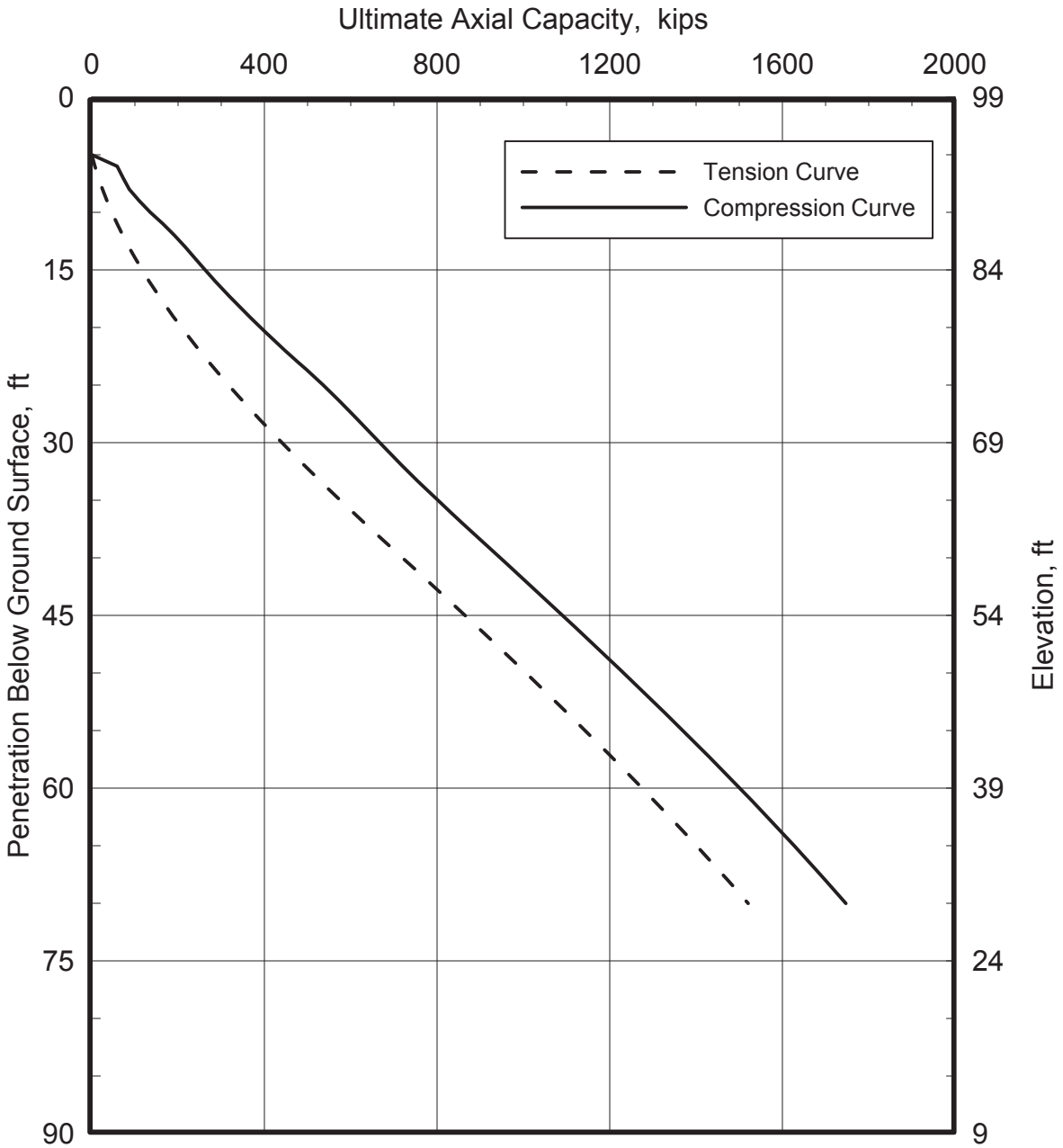
NOTES:

1. This curve represents ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. This curve is for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weight of the pile was not included in the computations.
6. Pile penetration was assumed to be 10 diameters above the termination depth of the boring used to establish the design soil profile (B-183).

ULTIMATE AXIAL CAPACITY - AREA 8 (B-183)

24-IN DIAMETER CAST-IN-PLACE CONCRETE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



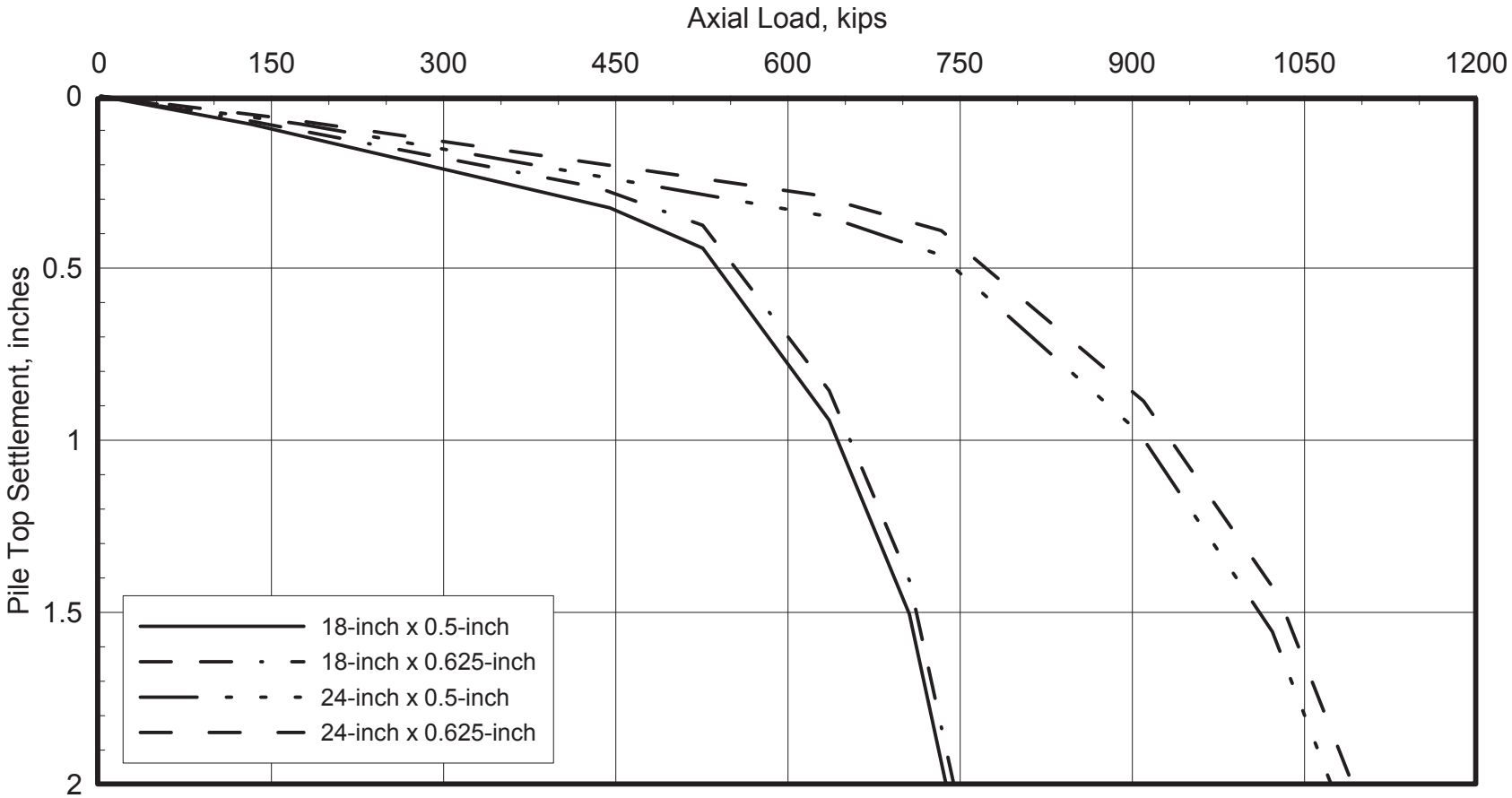
NOTES:

1. This curve represents ultimate values. A safety factor of 2.0 should be applied for sustained and transient compression loads or transient uplift loads. A safety factor of 3.0 should be applied for sustained uplift loads and 1.0 for uplift loads due to adfreeze stresses.
2. This curve is for a single isolated pile. Group effects are discussed in the report text.
3. Proposed rough grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.
4. Minimum pile penetration should be determined based on adfreeze stresses, lateral, and axial capacity analysis, whichever is the most critical.
5. Weight of the pile was not included in the computations.
6. Pile penetration was assumed to be 10 diameters above the termination depth of the boring used to establish the design soil profile (B-183).

ULTIMATE AXIAL CAPACITY - AREA 8 (B-183)

36-IN DIAMETER CAST-IN-PLACE CONCRETE PILE

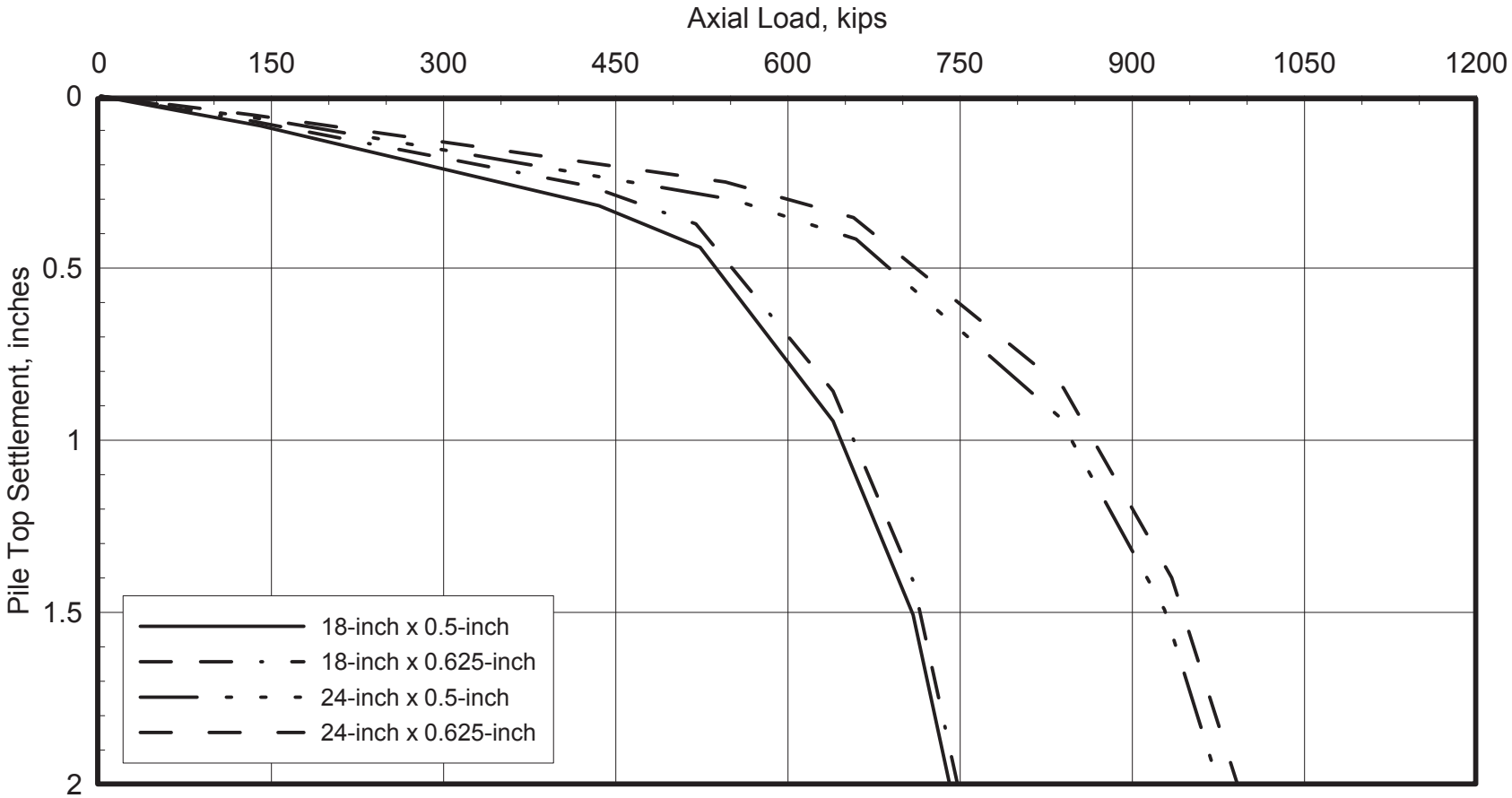
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 125 ft. Groundwater level was assumed at 70 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

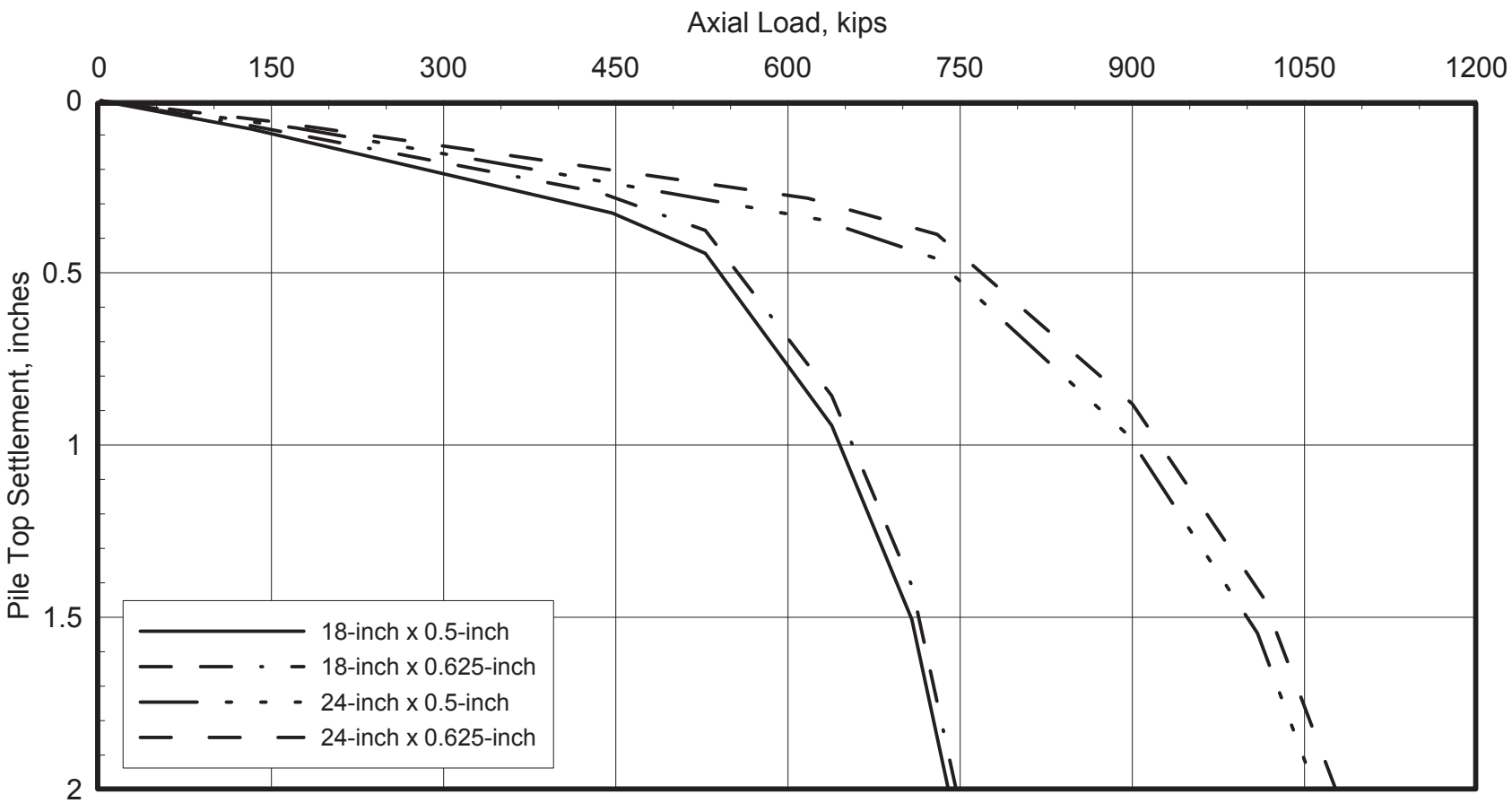
AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 2
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 116 ft. Groundwater level was assumed at 70 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

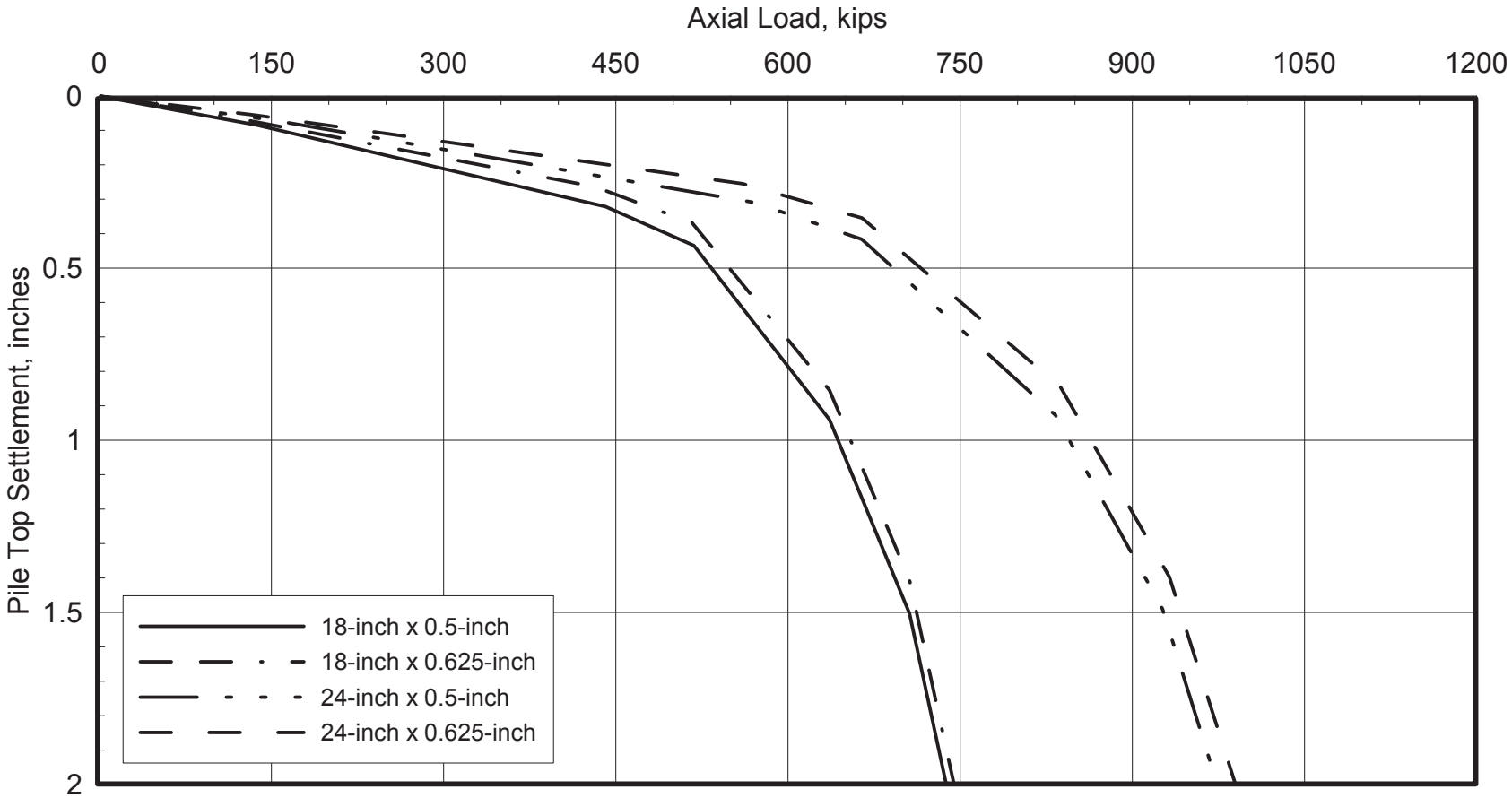
AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 3
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 125 ft. Groundwater level was assumed at 85 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

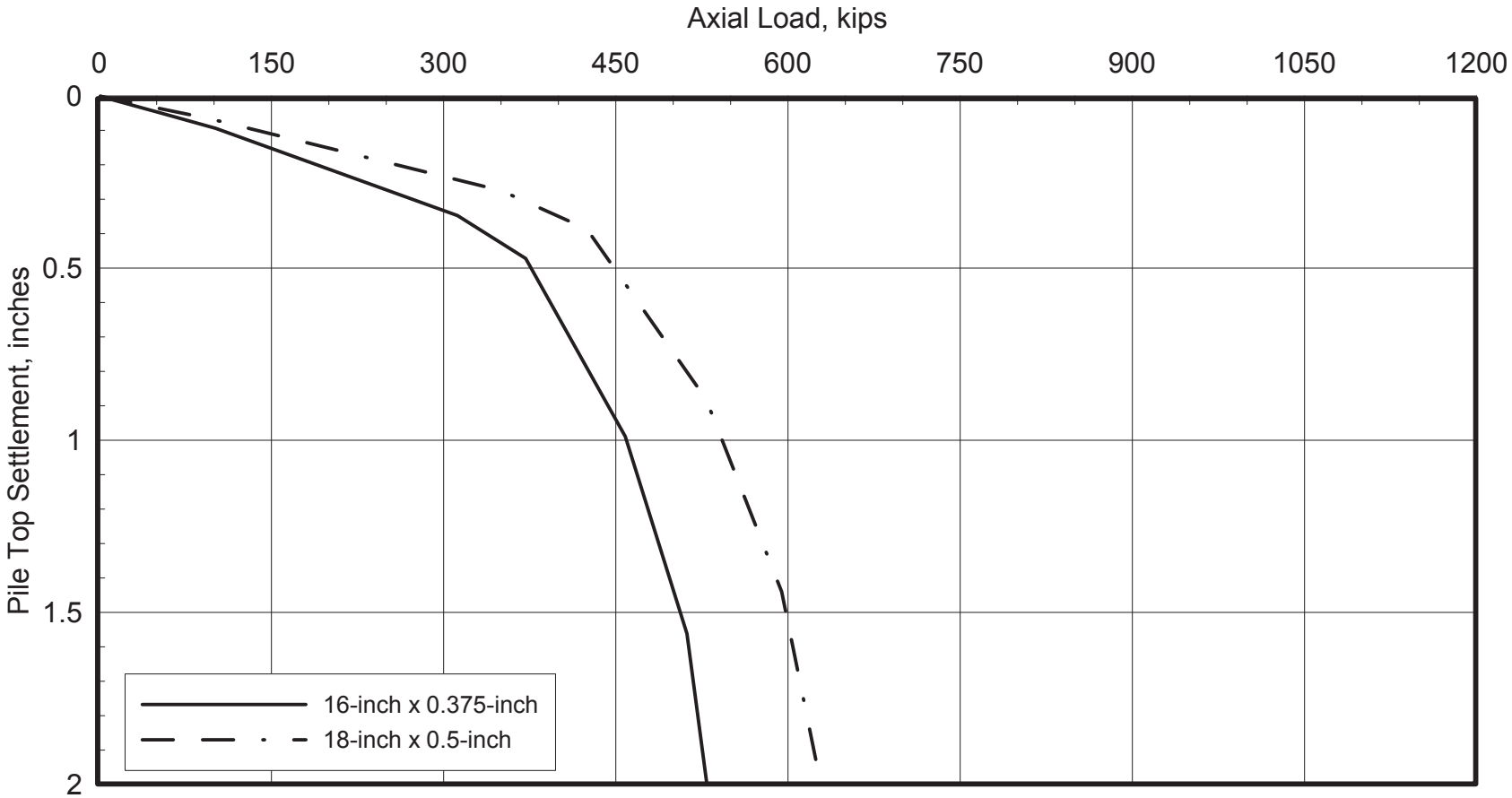
AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 4
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 116 ft. Groundwater level was assumed at 85 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

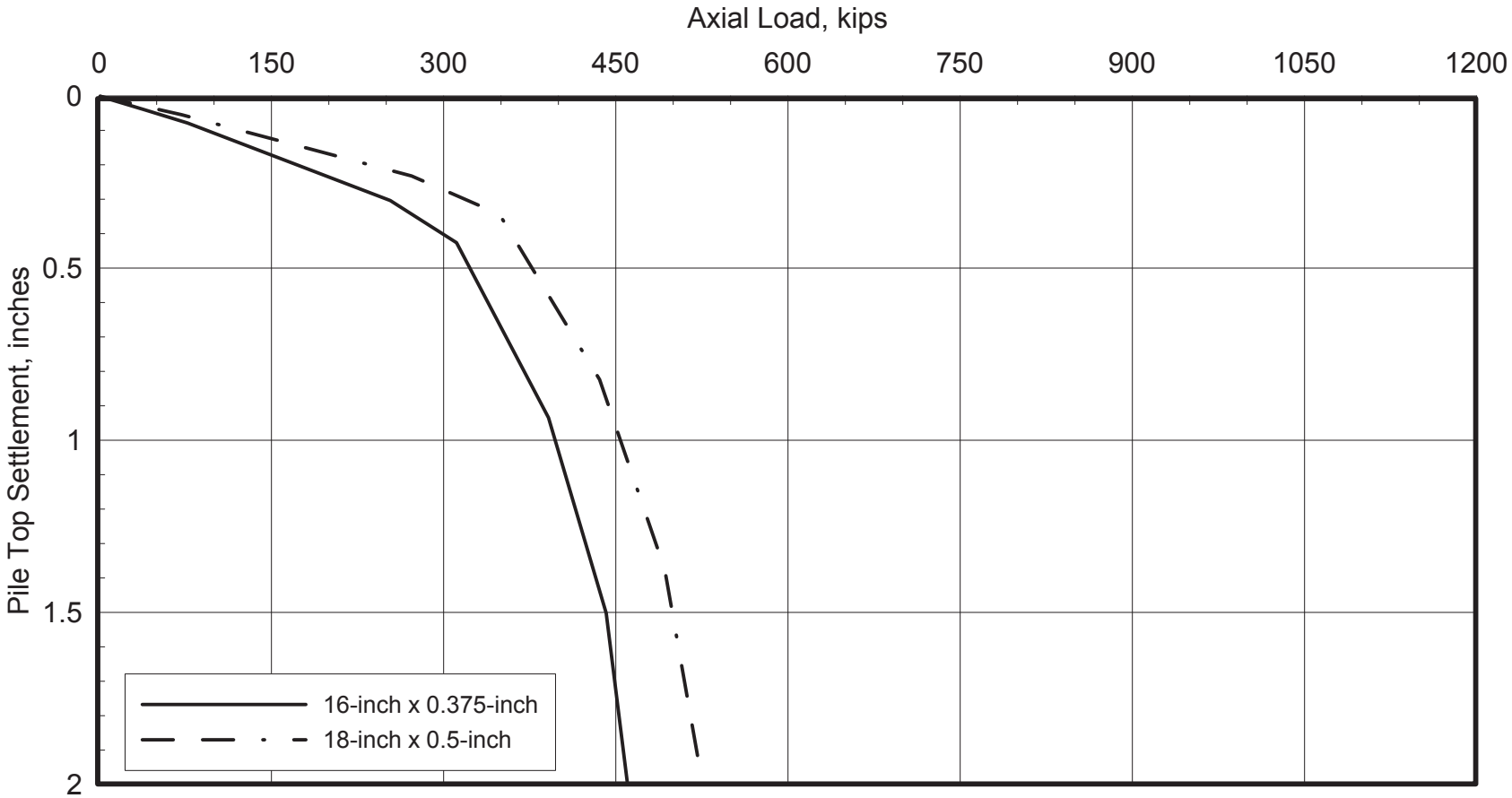
AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 5
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 27 ft. Groundwater level was assumed at 7 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 8 (B-176)
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

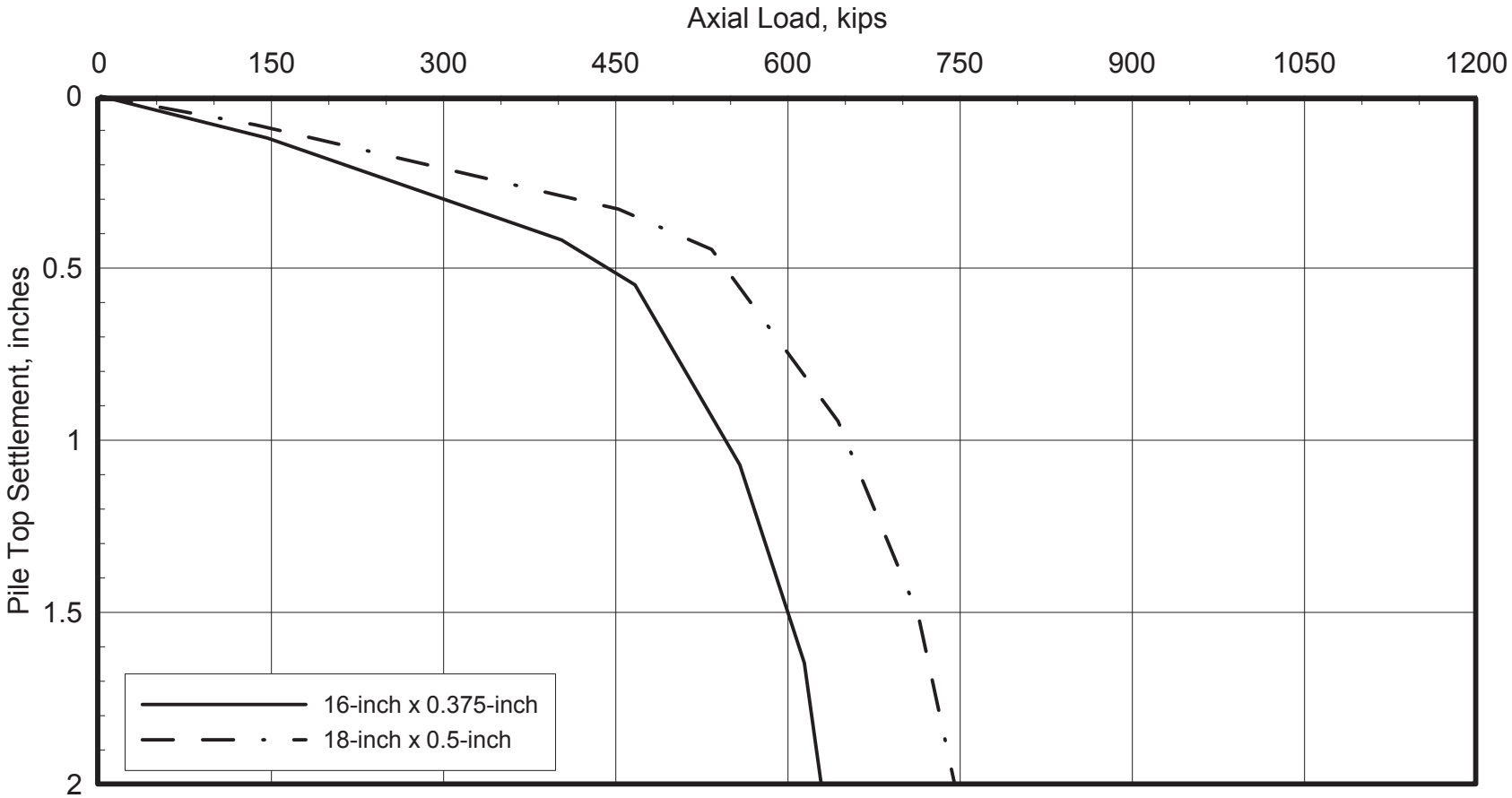


NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 27 ft. Groundwater level was assumed at 7 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 8 (B-176) - SEISMIC CONDITIONS

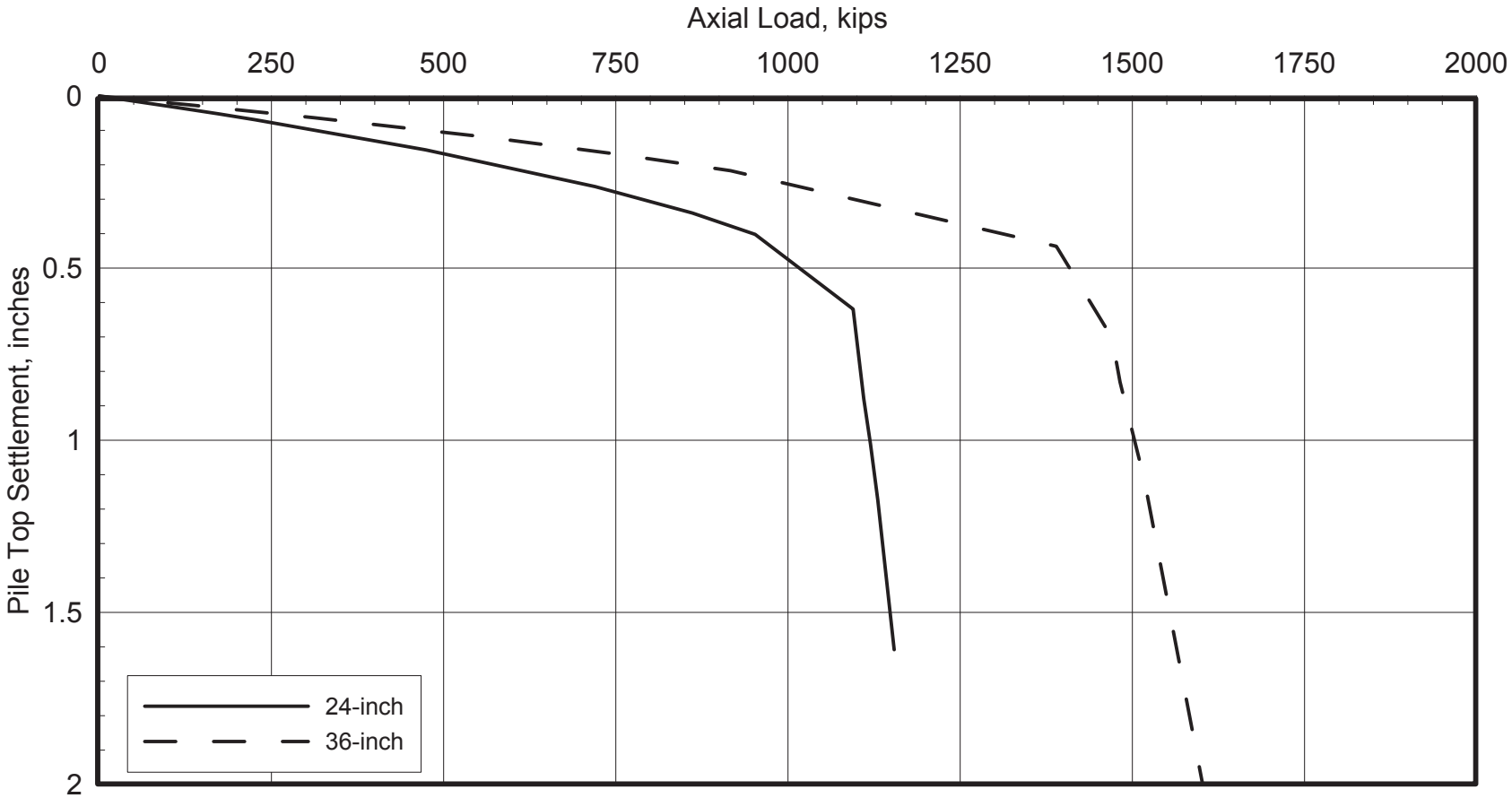
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at 60 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of piles is included in these curves.

AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 8 (B-183)
60-FT LONG OPEN-ENDED STEEL PIPE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

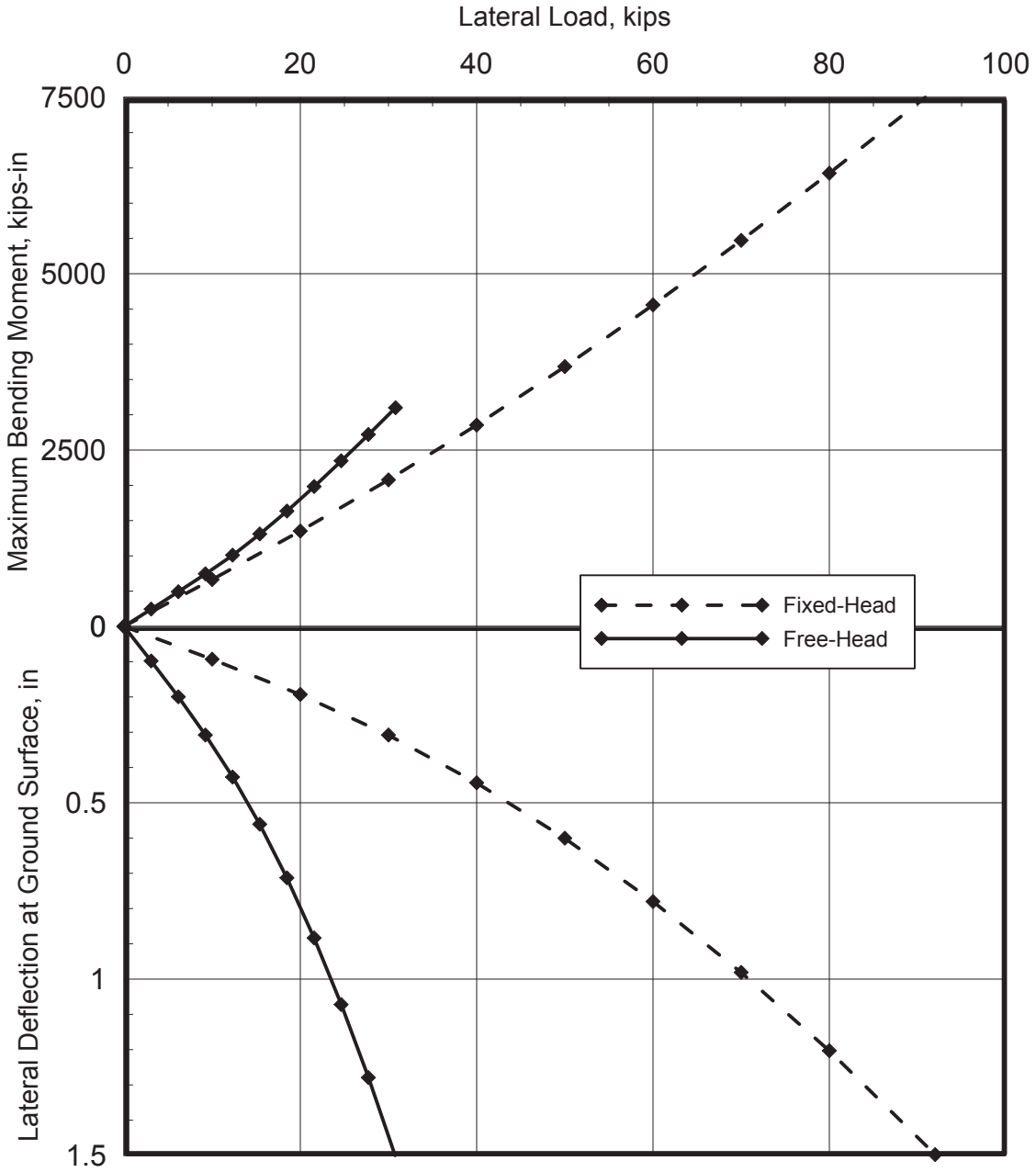


NOTES:

1. These curves represent ***axial load versus pile top settlement.***
2. These curves are for a single isolated pile. Group effects are discussed in the report text.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at 60 ft below final grade.
4. Pile top was assumed at final grade. Elastic shortening of shafts is included in these curves.
5. The pile embedment was assumed to be 80 ft for the 24-inch pile and 70 for the 36-inch pile.

AXIAL LOAD VERSUS PILE TOP SETTLEMENT - AREA 8 (B-183)

CAST-IN-PLACE CONCRETE PILE
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



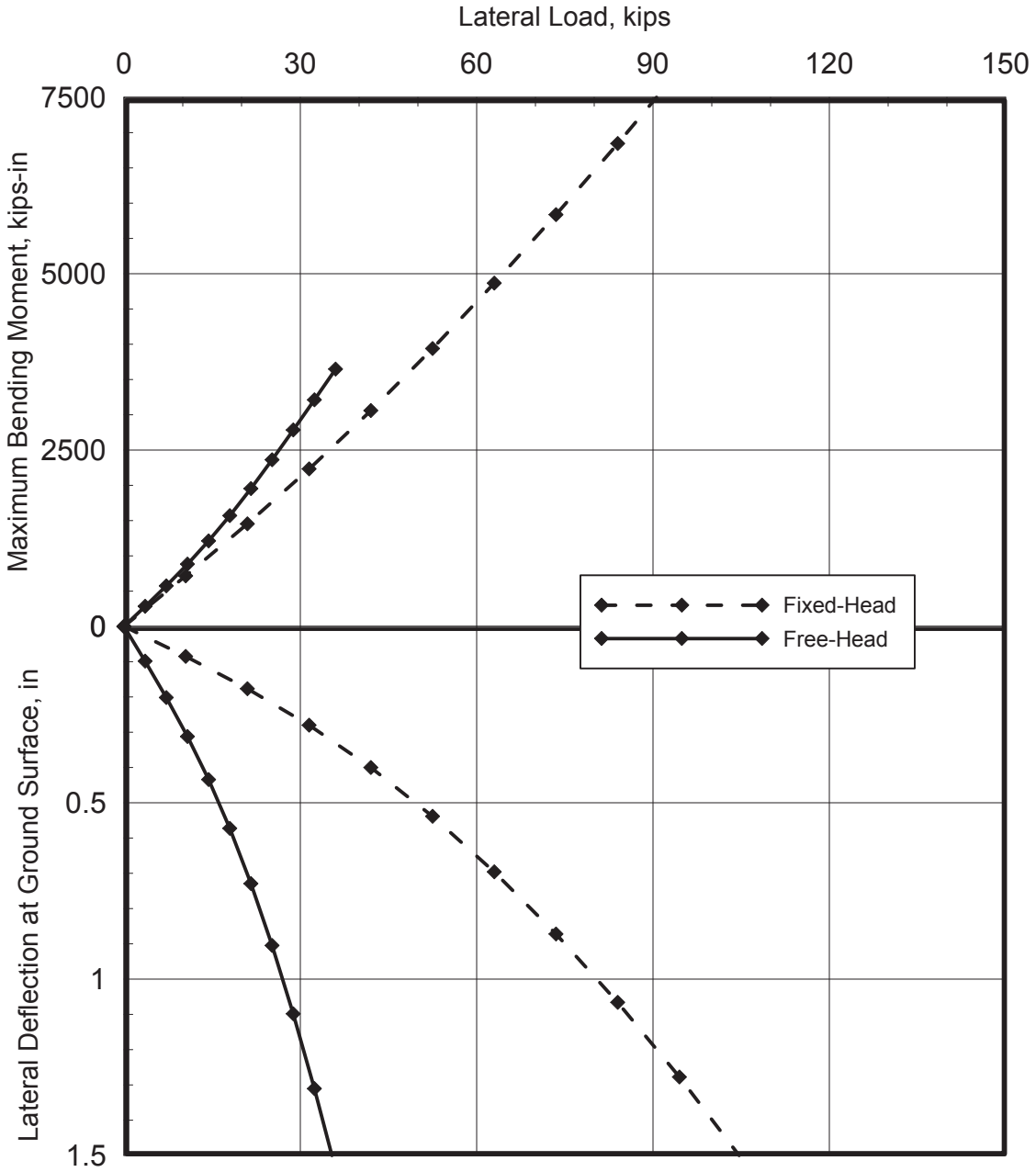
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 29 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.

LATERAL CAPACITY - AREA 2

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



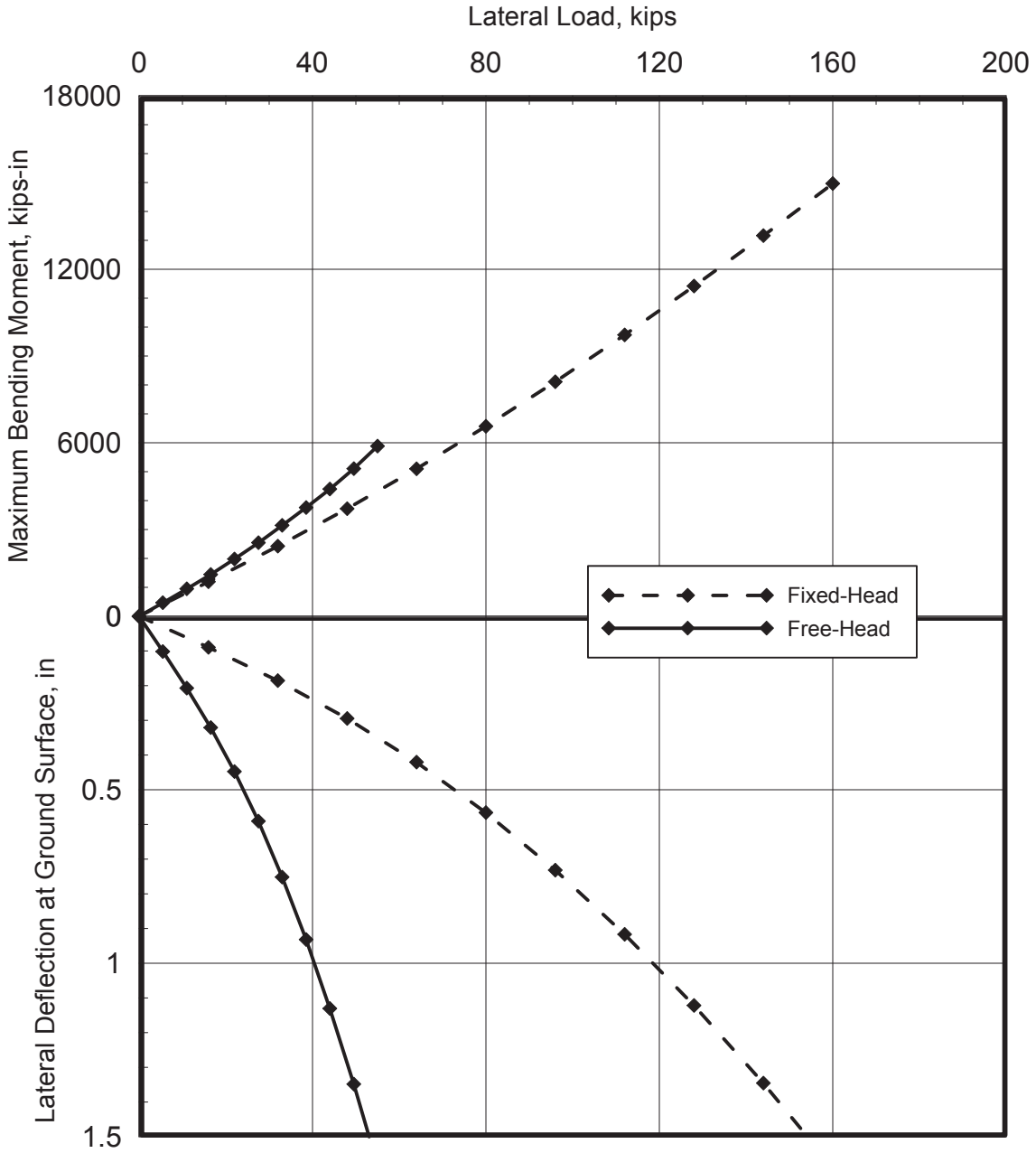
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 31 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.

LATERAL CAPACITY - AREA 2

18-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



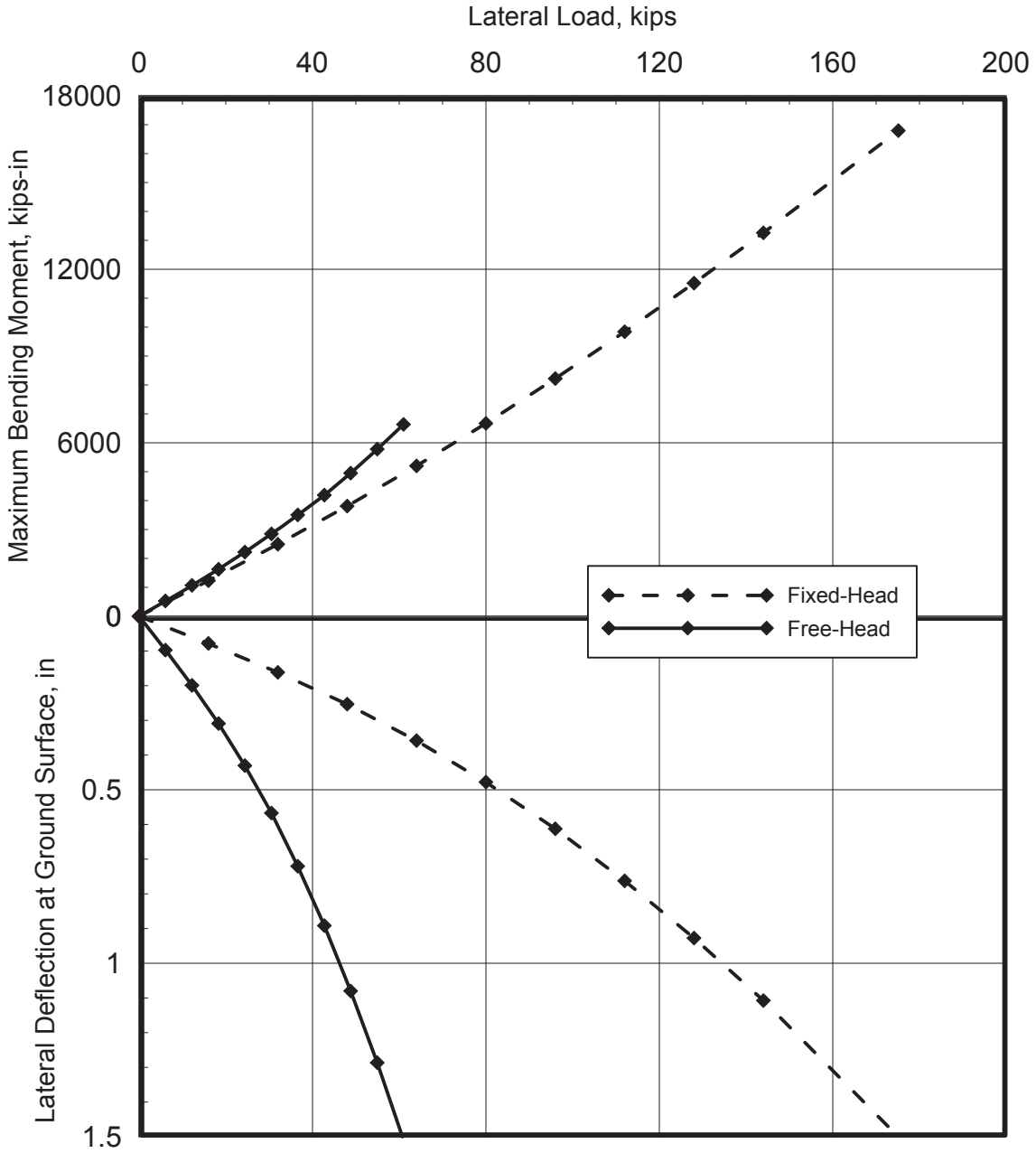
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 33 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.

LATERAL CAPACITY - AREA 2

24-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



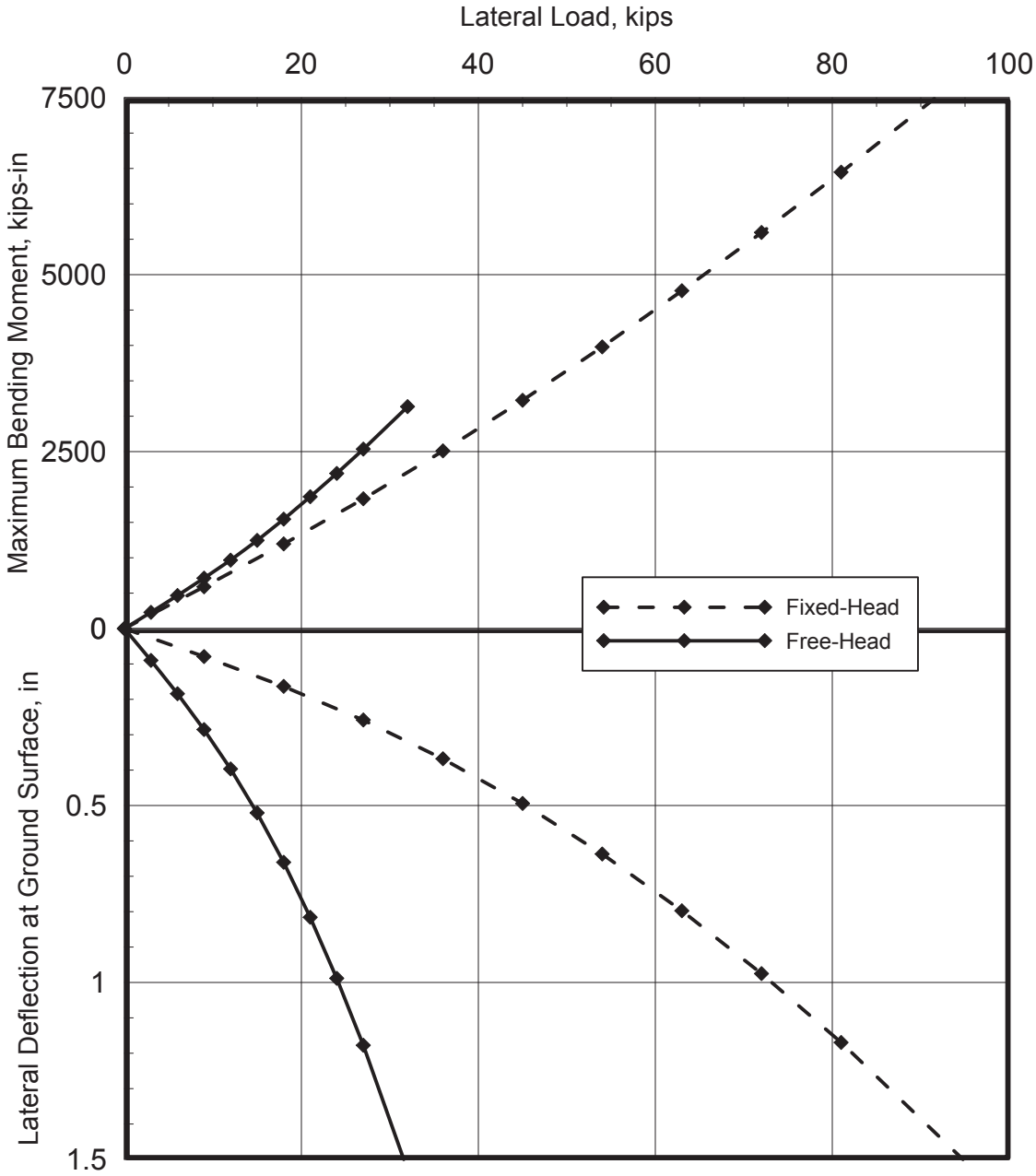
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 36 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 70 ft.

LATERAL CAPACITY - AREA 2

24-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



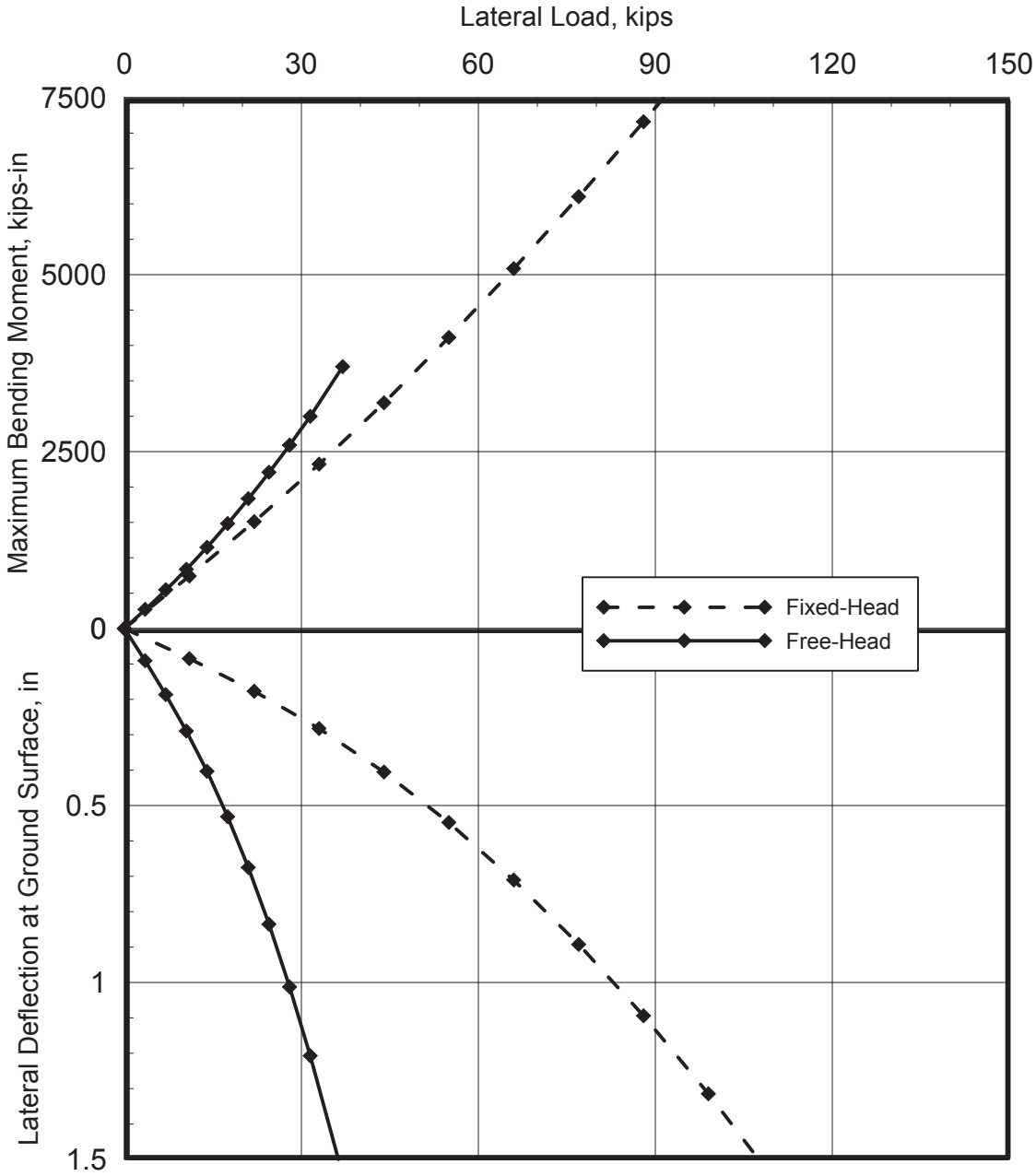
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 29 ft.
4. Final grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft in Area 3 and El. 85 ft in Area 5.

LATERAL CAPACITY - AREAS 3 AND 5

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



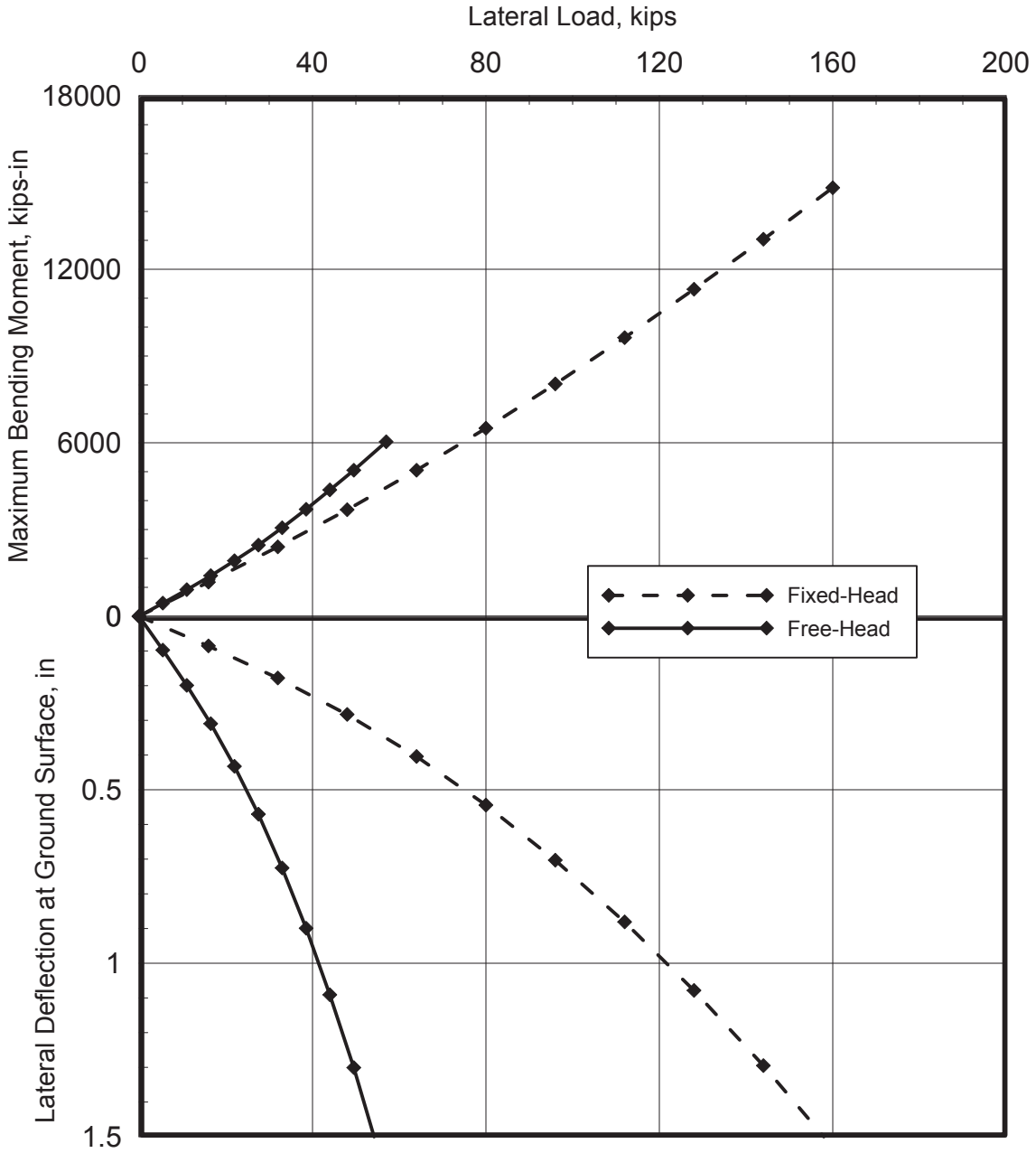
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 30 ft.
4. Final grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft in Area 3 and El. 85 ft in Area 5.

LATERAL CAPACITY - AREAS 3 AND 5

18-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



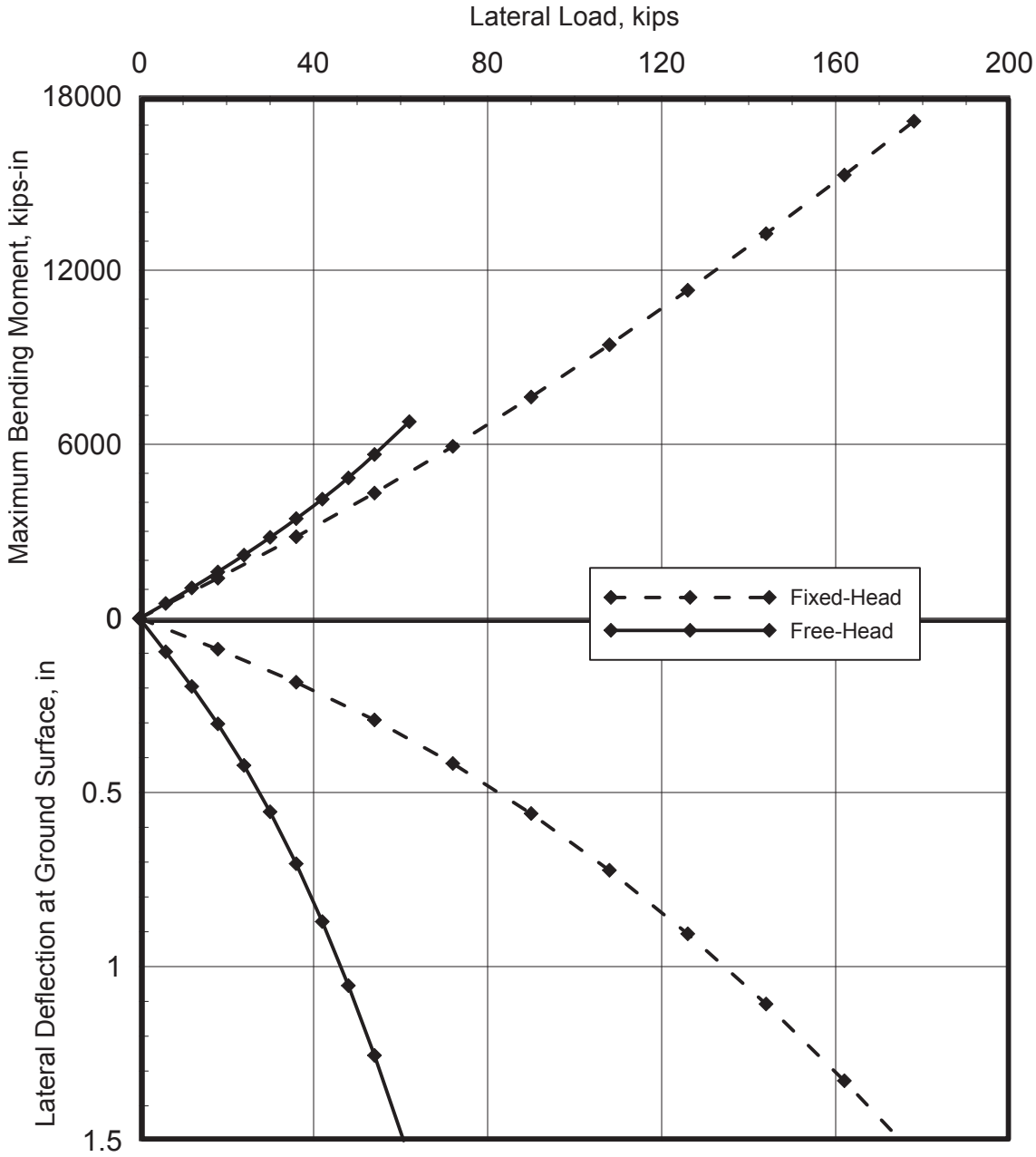
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 34 ft.
4. Final grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft in Area 3 and El. 85 ft in Area 5.

LATERAL CAPACITY - AREAS 3 AND 5

24-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



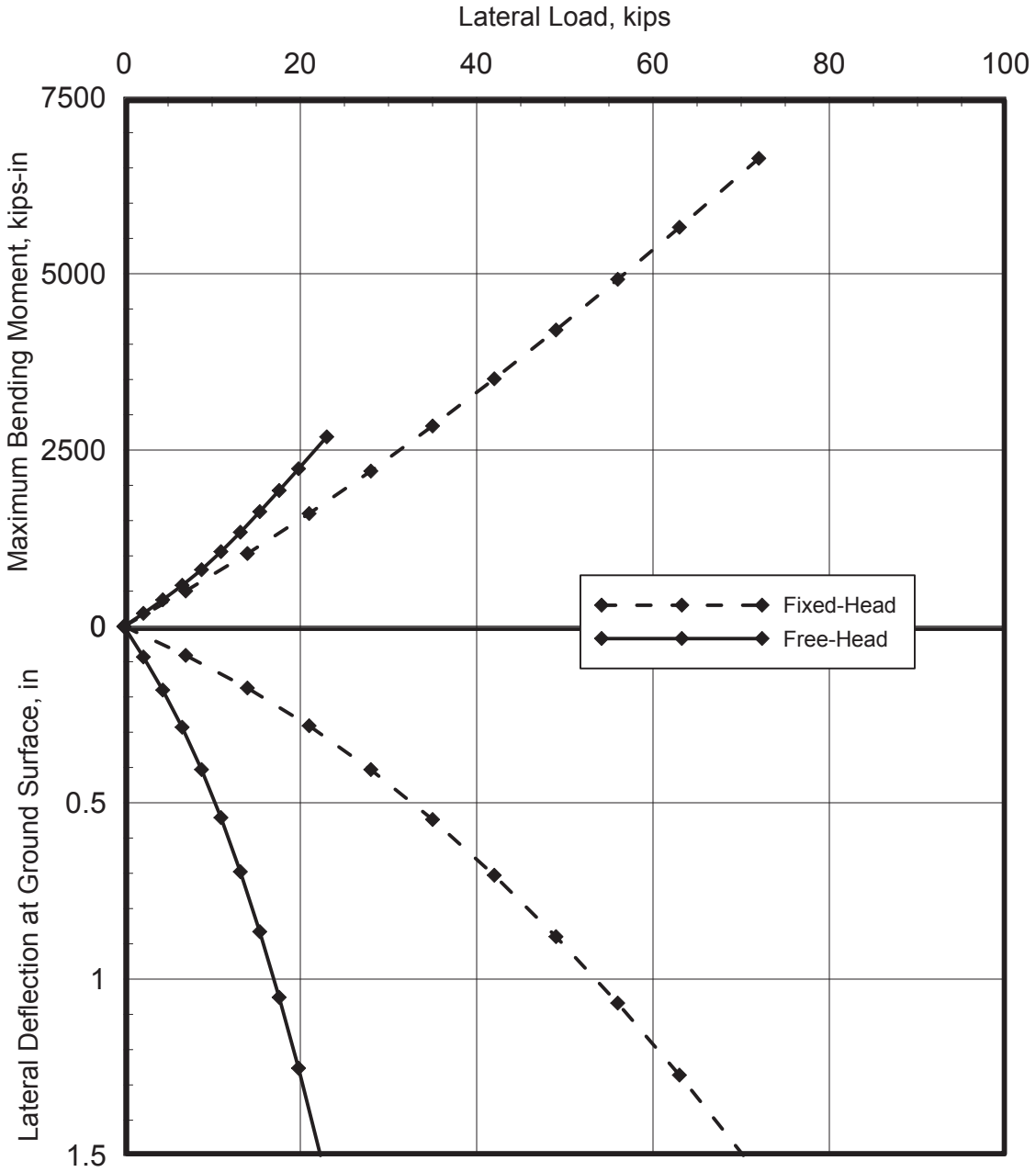
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 35 ft.
4. Final grade was assumed at El. 116 ft. Groundwater level was assumed at El. 70 ft in Area 3 and El. 85 ft in Area 5.

LATERAL CAPACITY - AREAS 3 AND 5

24-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



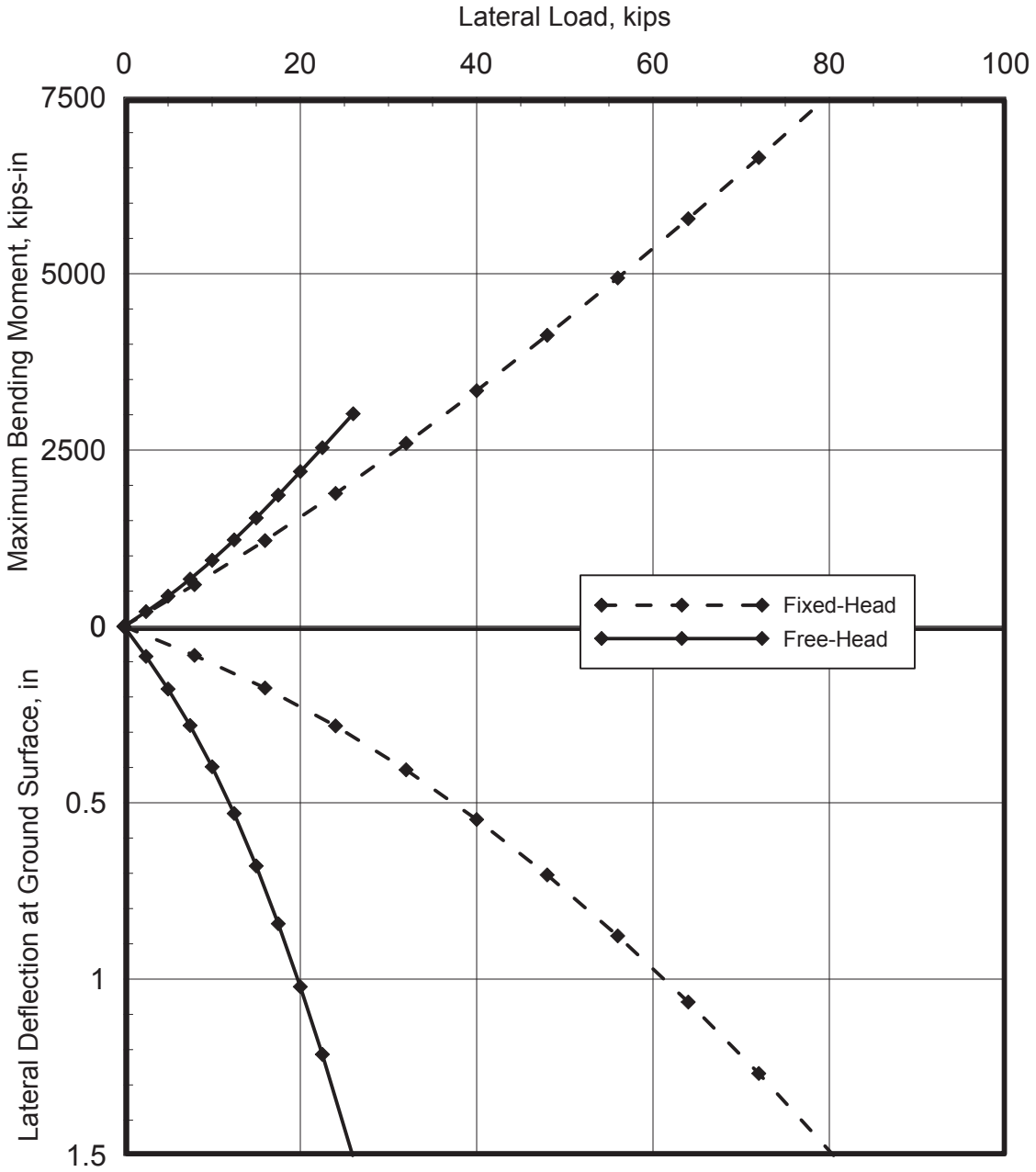
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 30 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.

LATERAL CAPACITY - AREA 4

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



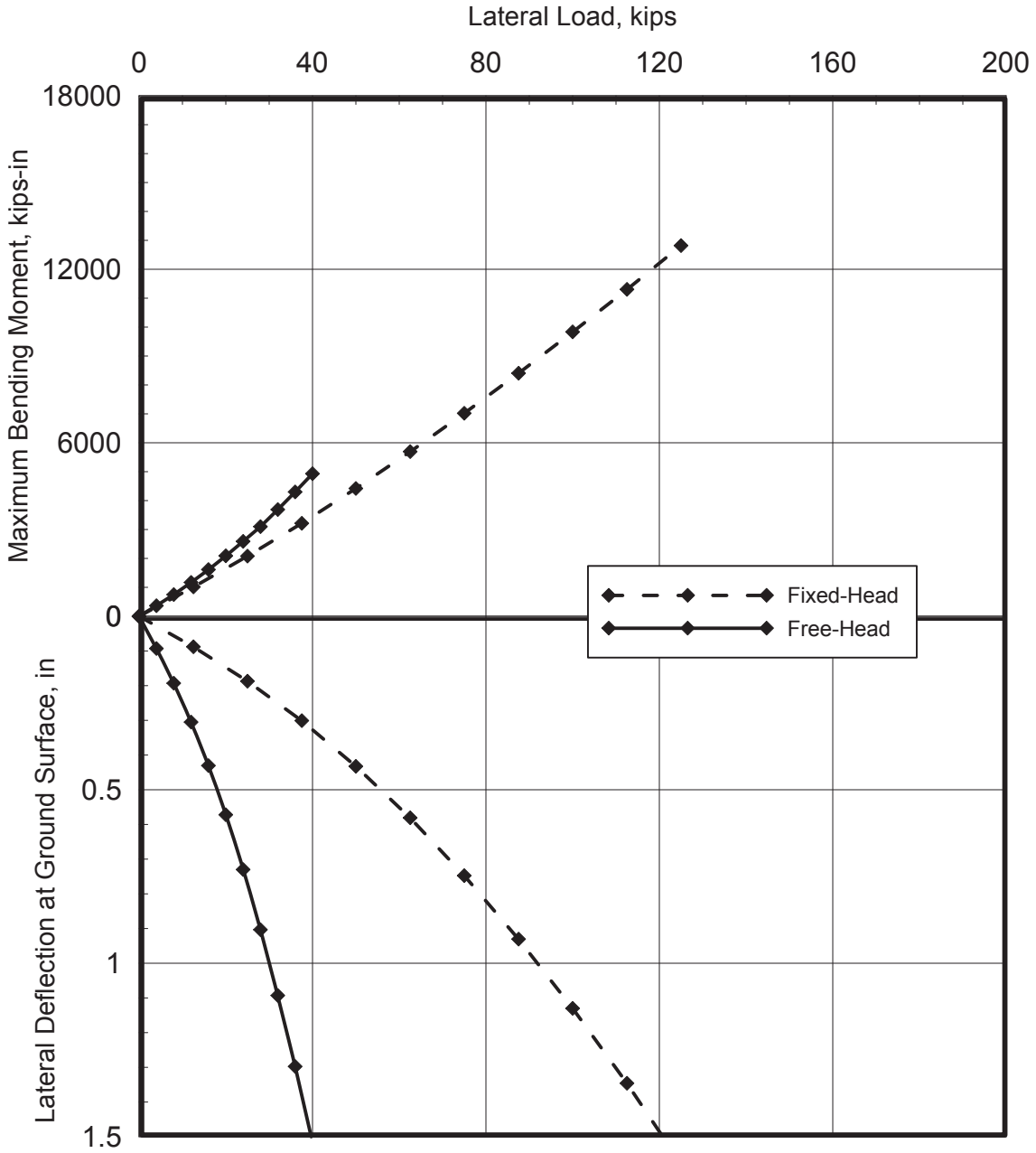
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 32 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.

LATERAL CAPACITY - AREA 4

18-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



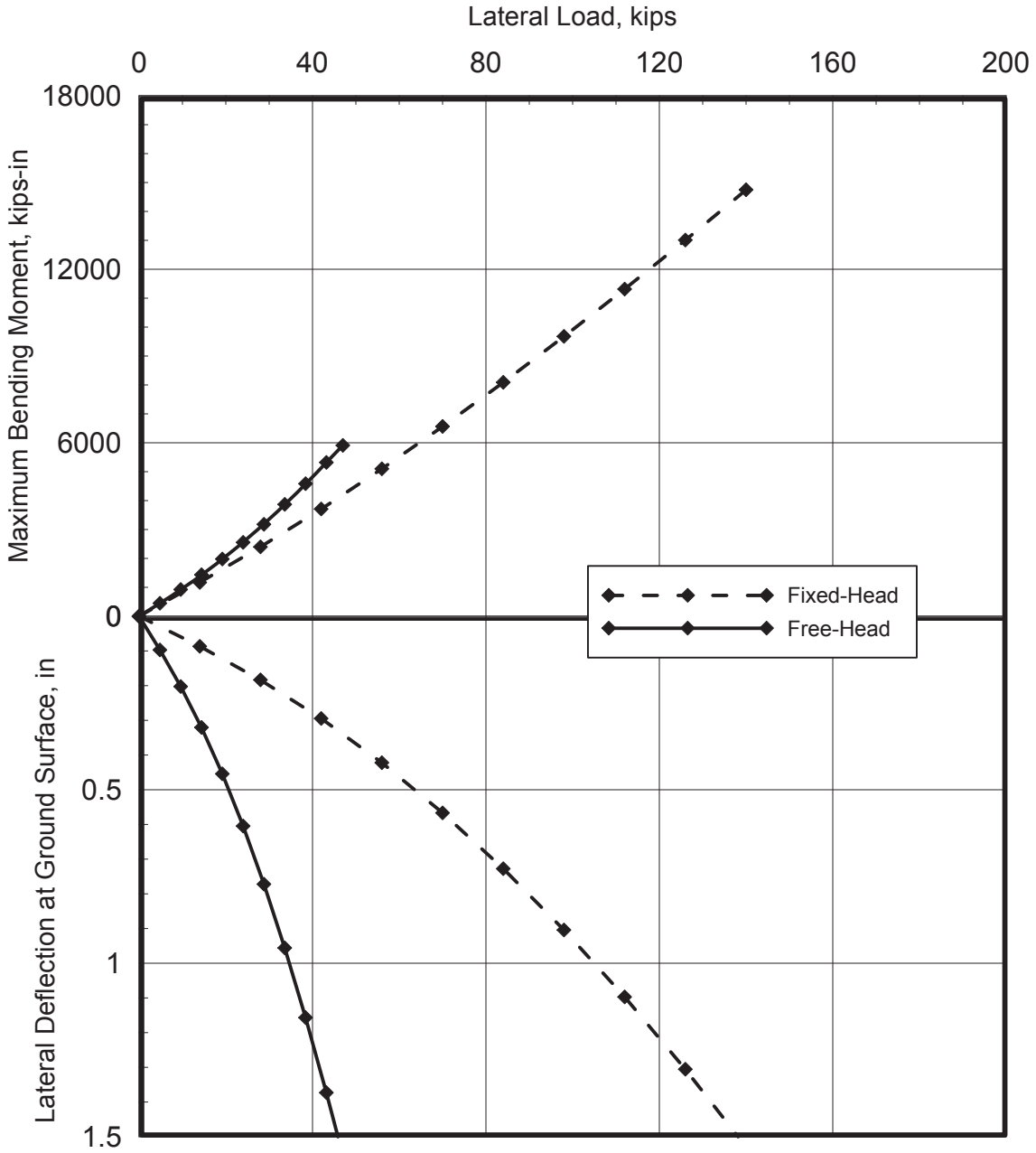
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 36 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.

LATERAL CAPACITY - AREA 4

24-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



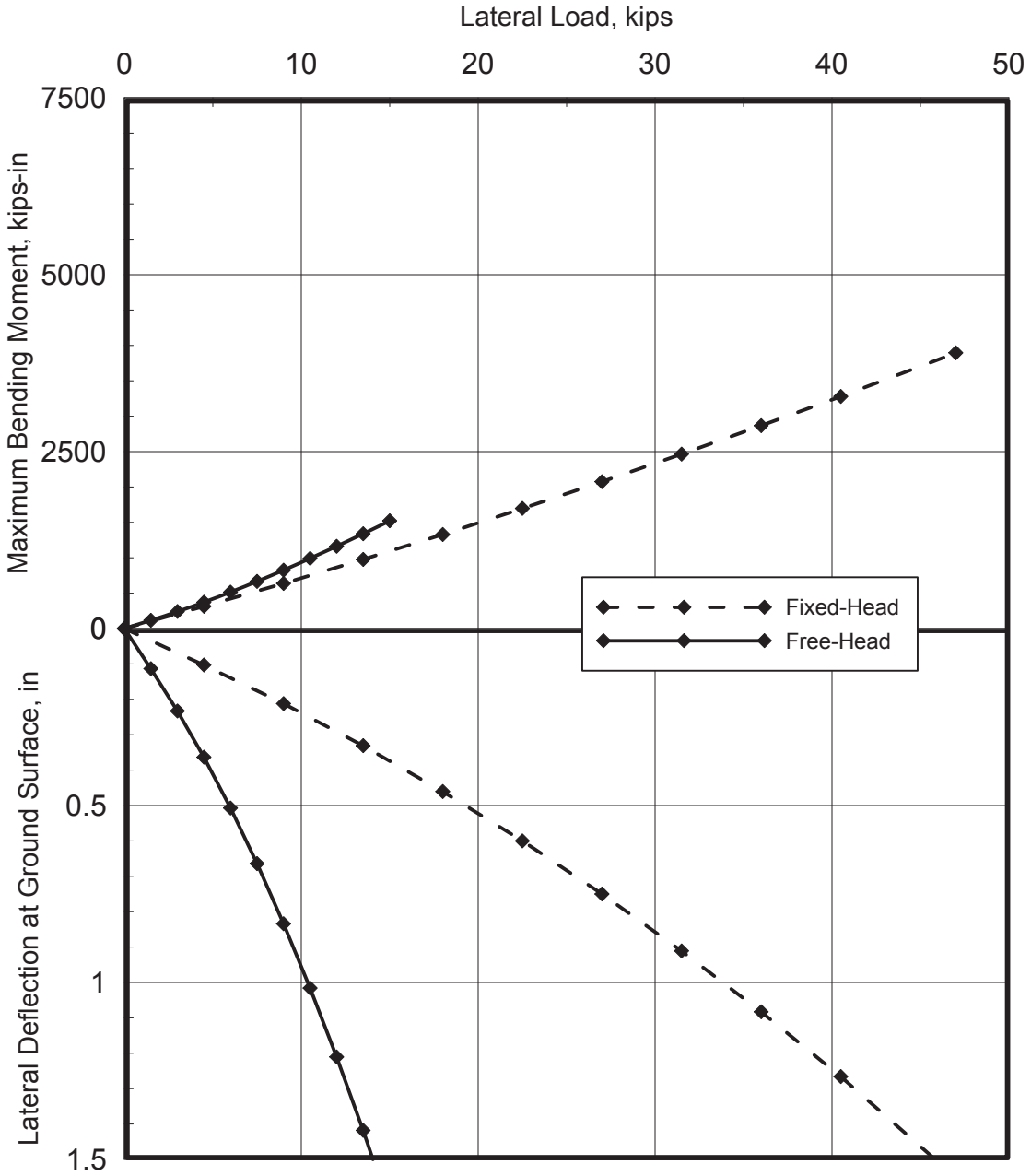
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 37 ft.
4. Final grade was assumed at El. 125 ft. Groundwater level was assumed at El. 85 ft.

LATERAL CAPACITY - AREA 4

24-IN (0.625-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



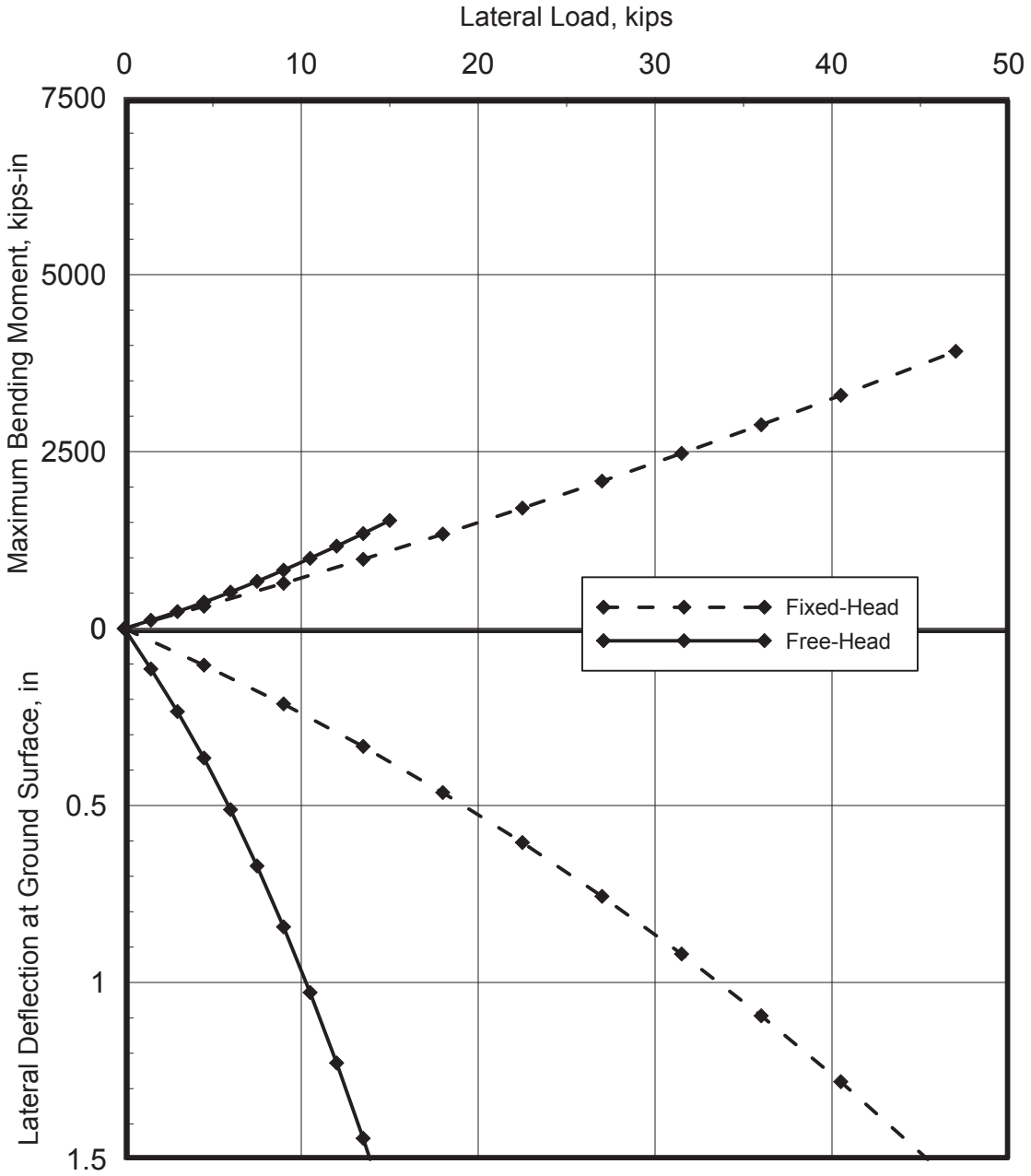
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 34 ft.
4. Final grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.

LATERAL CAPACITY - AREA 8 (B-176)

16-IN (0.375-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



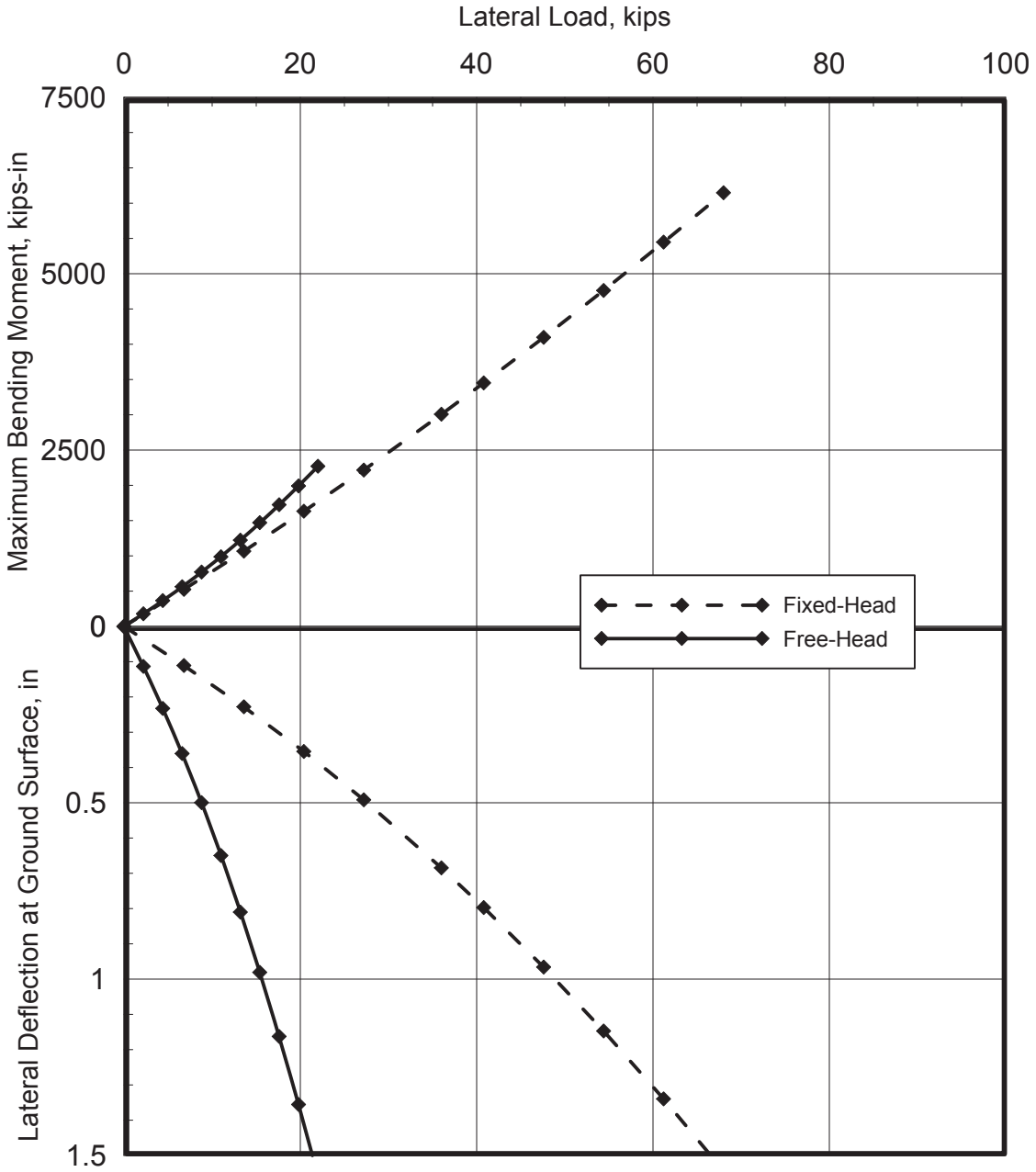
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 34 ft.
4. Final grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.

LATERAL CAPACITY - AREA 8 (B-176) - SEISMIC CONDITIONS

16-IN (0.375-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



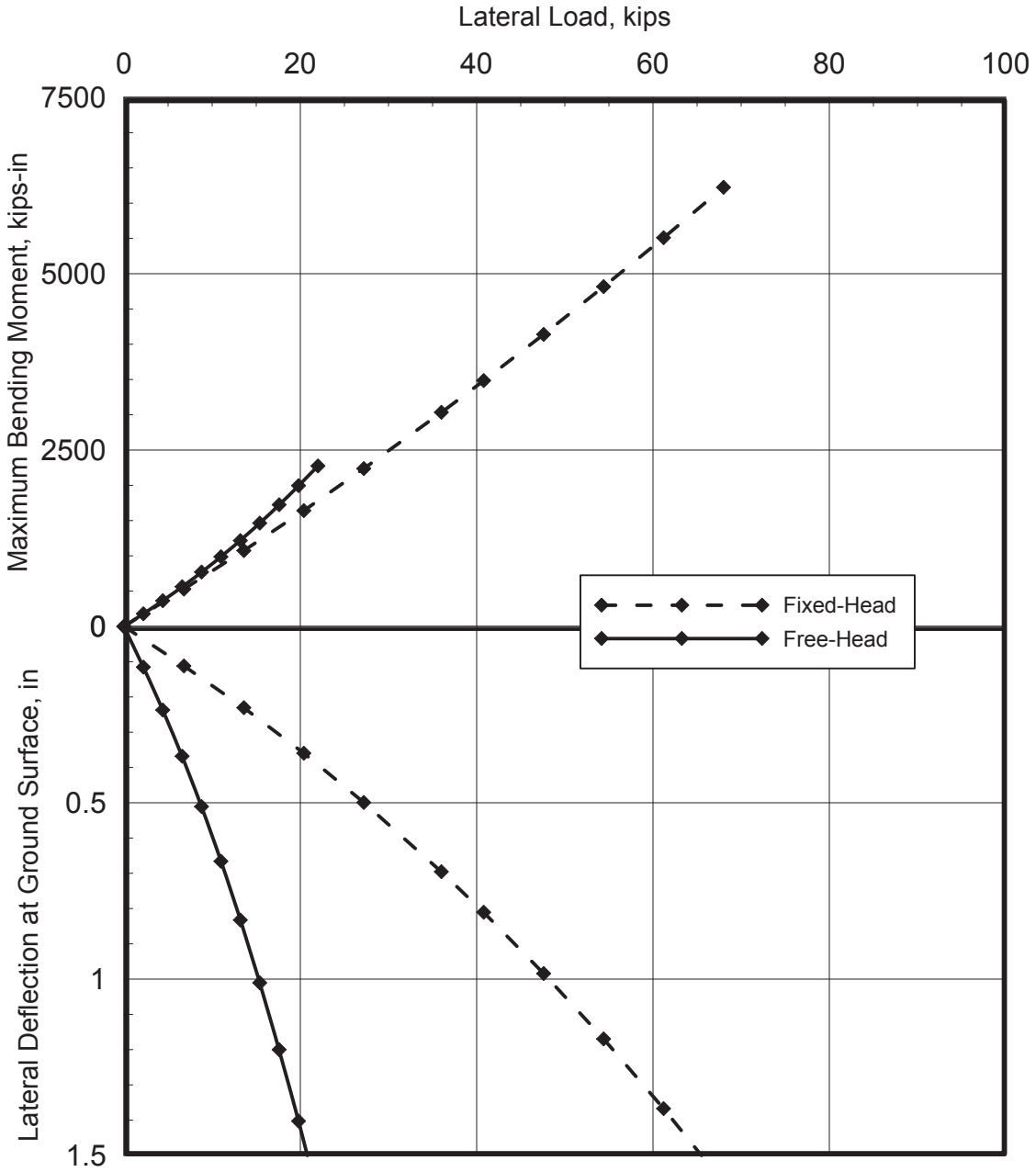
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 36 ft.
4. Final grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.

LATERAL CAPACITY - AREA 8 (B-176)

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



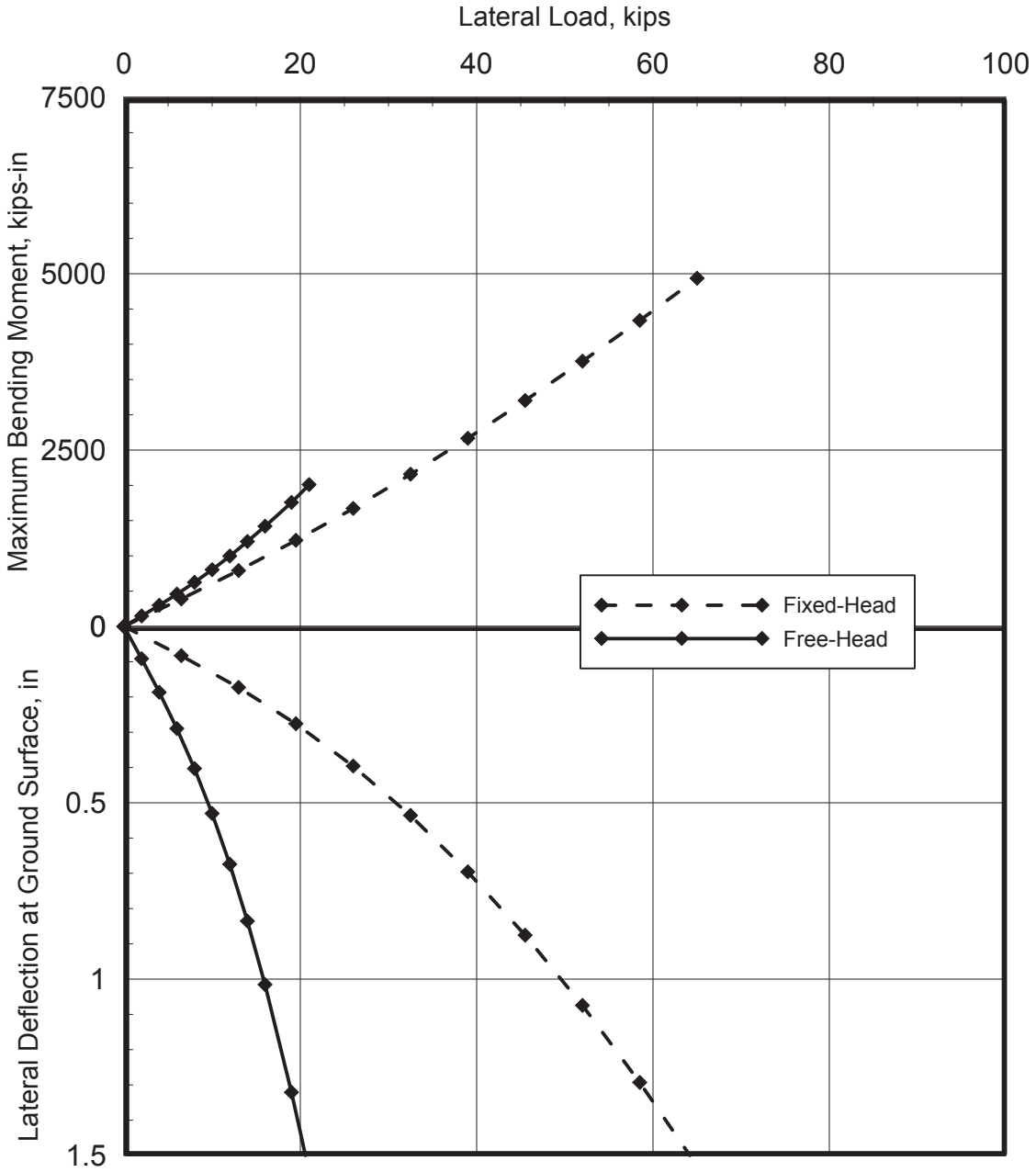
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 36 ft.
4. Final grade was assumed at El. 27 ft. Groundwater level was assumed at El. 7 ft.

LATERAL CAPACITY - AREA 8 (B-176) - SEISMIC CONDITIONS

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



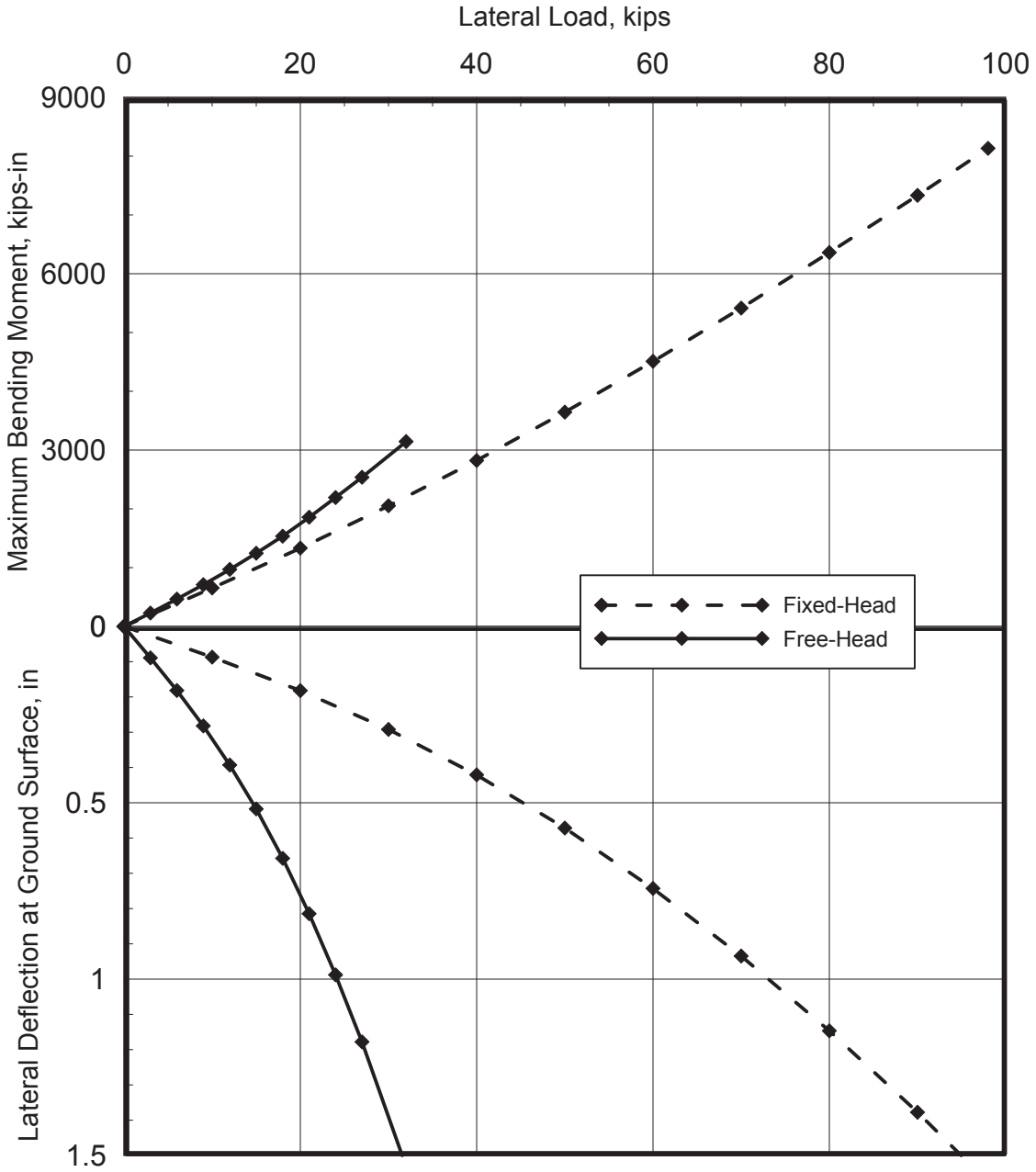
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 25 ft.
4. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)

16-IN (0.375-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



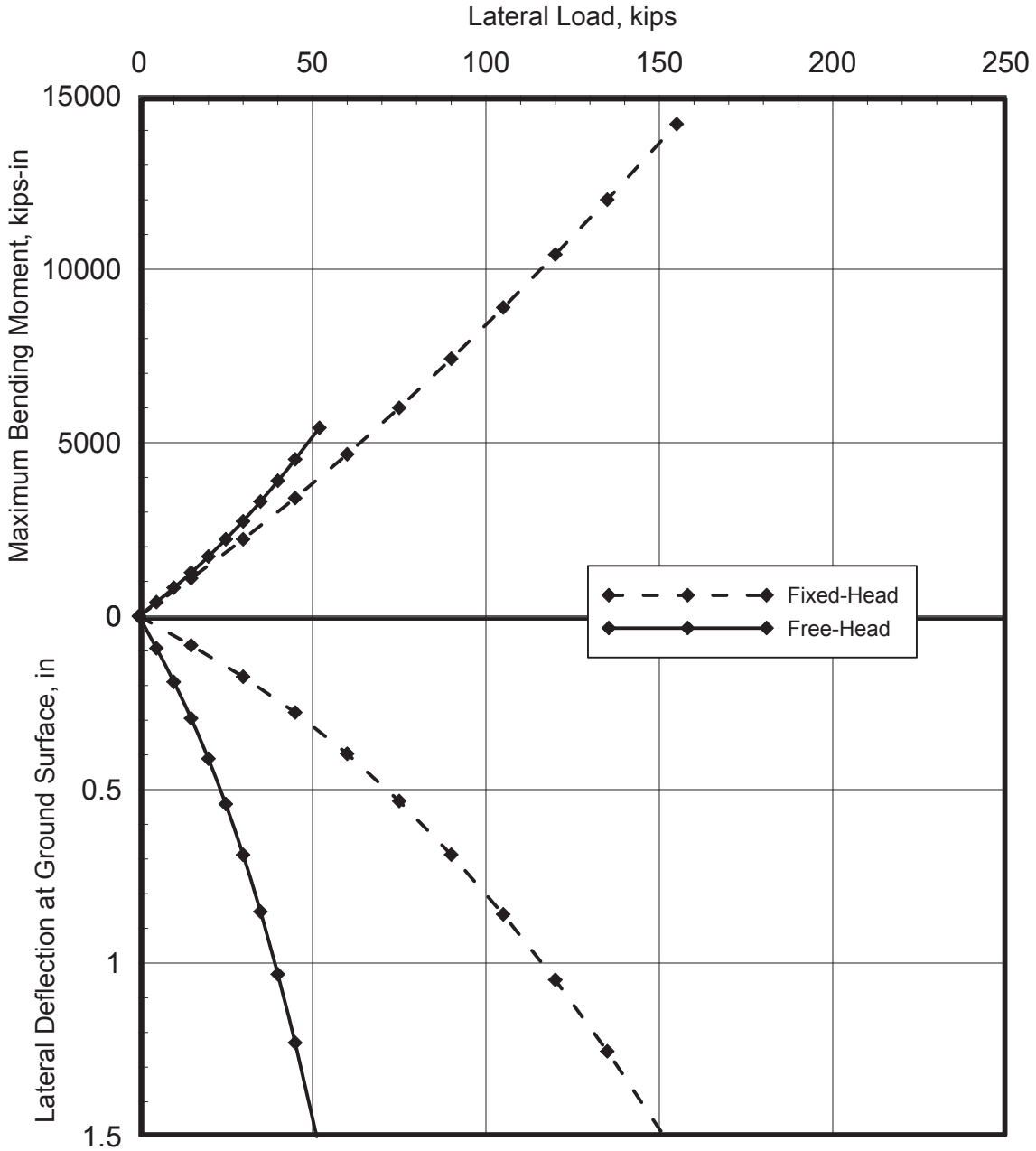
NOTES:

1. These curves are for a single, isolated open-ended steel pipe pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. The wall thickness was reduced by 0.06-in to account for corrosion.
3. These curves can be used for pile lengths of greater than 28 ft.
4. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)

18-IN (0.5-IN-THICK WALL) DIAMETER OPEN-ENDED STEEL PIPE PILE

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

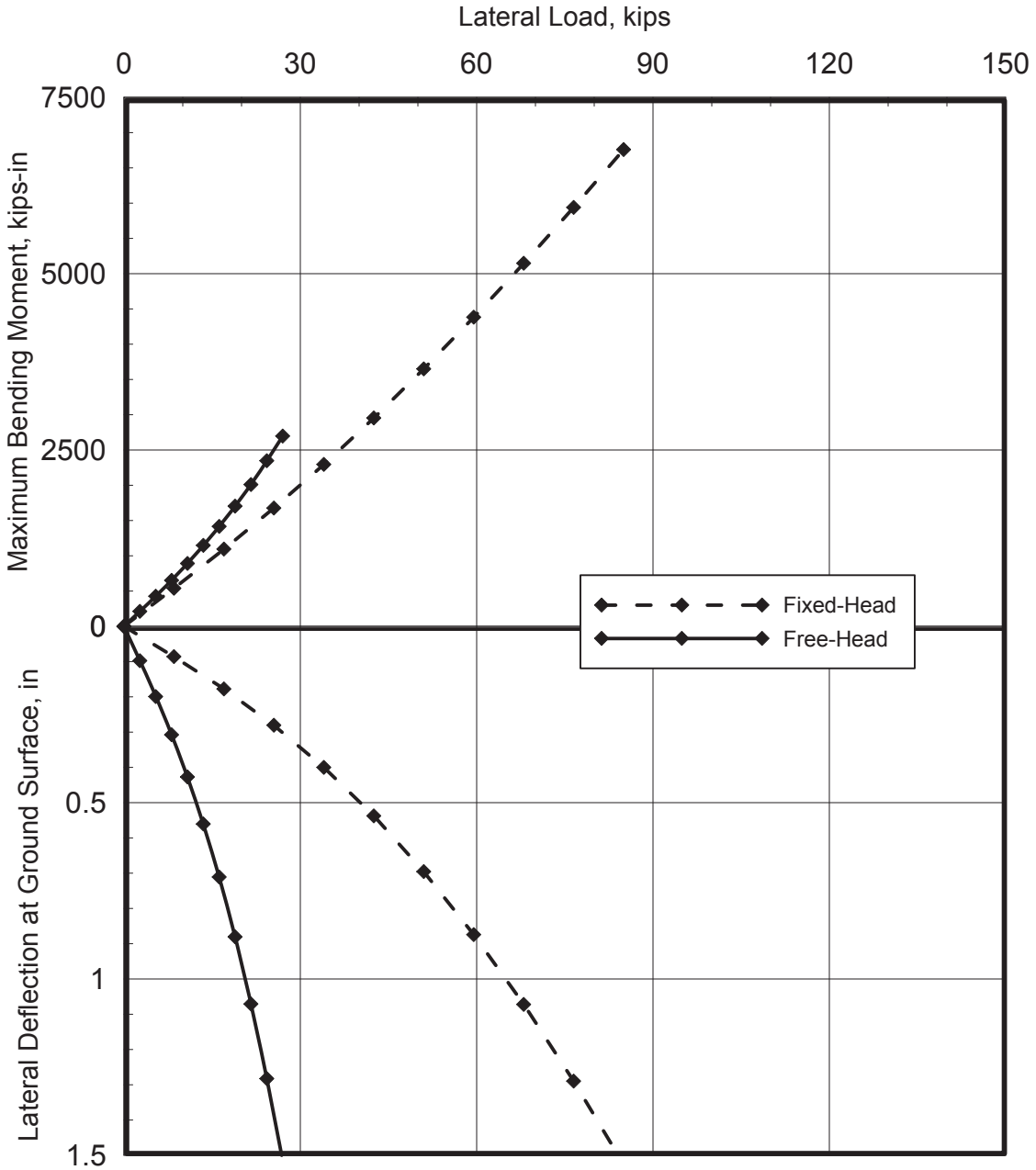


NOTES:
1. These curves are for a single, isolated cast-in-place concrete pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. These curves can be used for shaft lengths of greater than 32 ft.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)

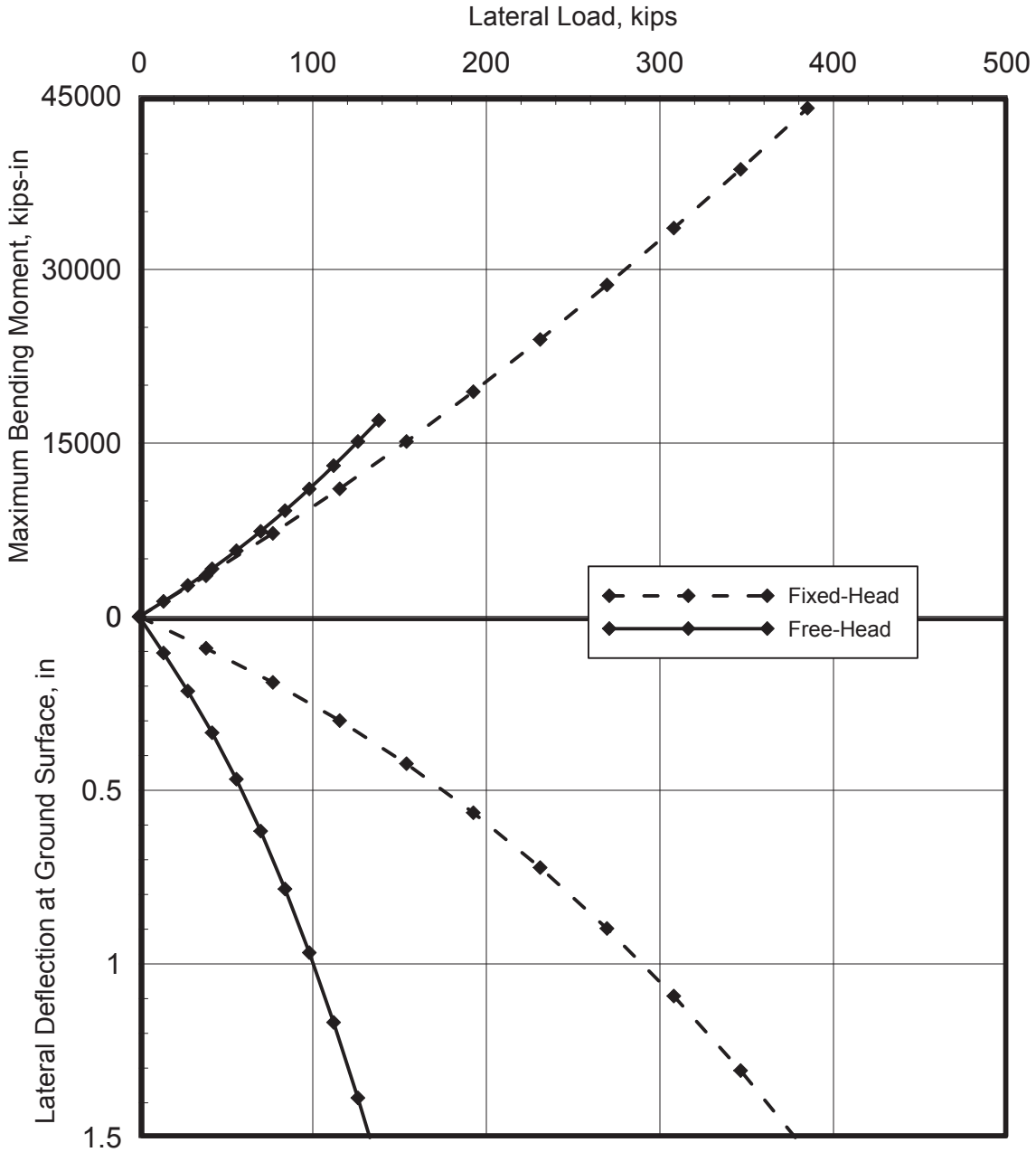
24-IN DIAMETER CAST-IN-PLACE CONCRETE PILE (UNCRACKED CONDITIONS)

ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



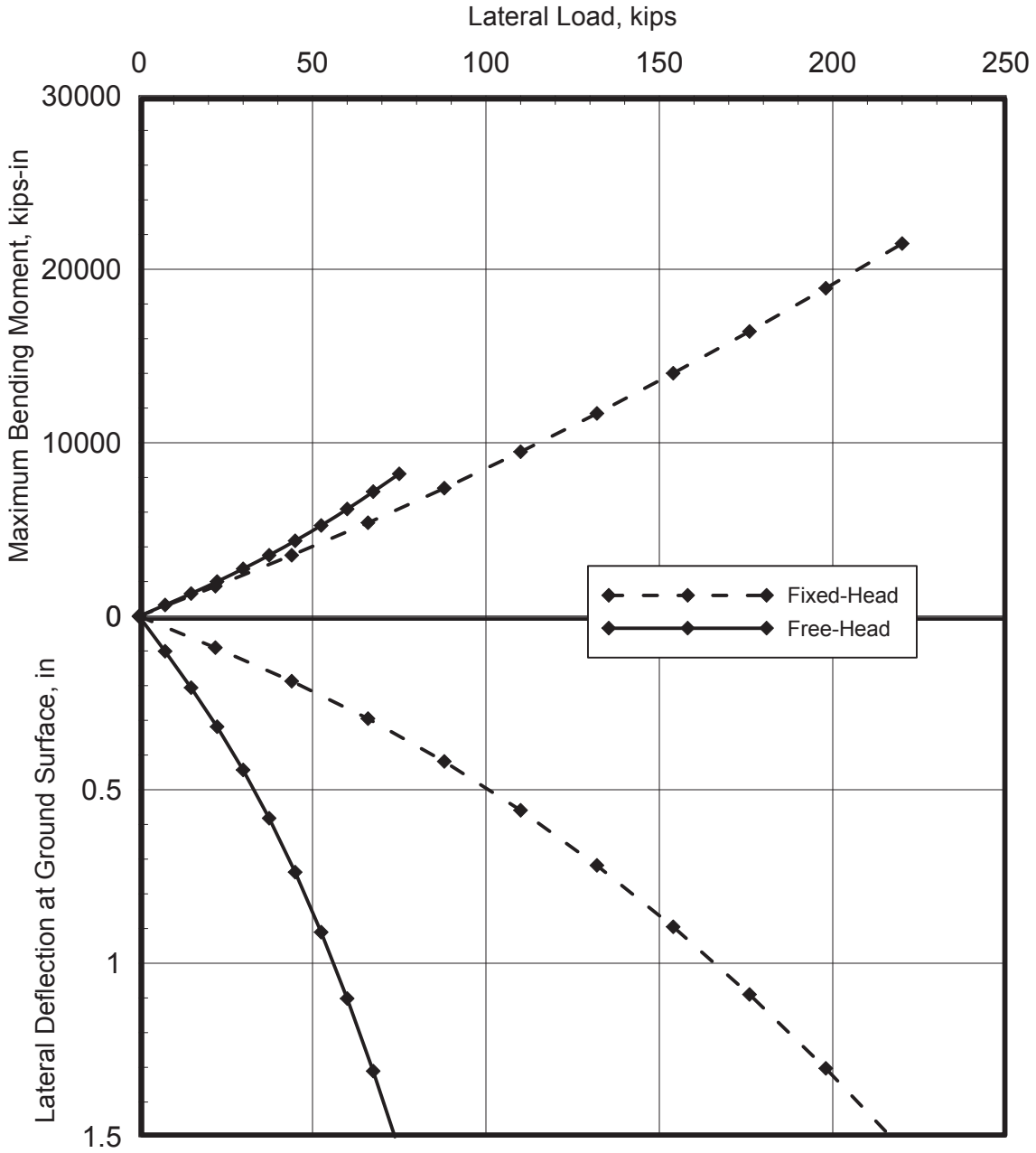
NOTES:
1. These curves are for a single, isolated cast-in-place concrete pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. These curves can be used for shaft lengths of greater than 27 ft.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)
24-IN DIAMETER CAST-IN-PLACE CONCRETE PILE (CRACKED CONDITIONS)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



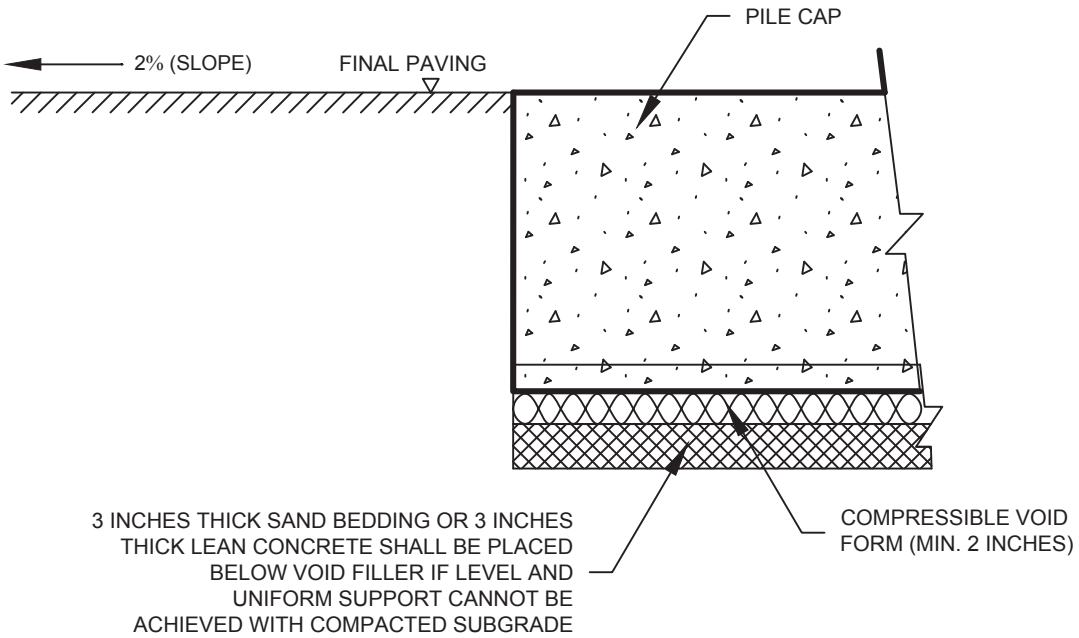
NOTES:
1. These curves are for a single, isolated cast-in-place concrete pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. These curves can be used for shaft lengths of greater than 42 ft.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)
36-IN DIAMETER CAST-IN-PLACE CONCRETE PILE (UNCRACKED CONDITIONS)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA



NOTES:
1. These curves are for a single, isolated cast-in-place concrete pile with free- / fixed-head conditions. Lateral load is applied at the ground surface, which is also at the pile top. Group effects are addressed in the report text.
2. These curves can be used for shaft lengths of greater than 36 ft.
3. Final grade was assumed at El. 99 ft. Groundwater level was assumed at El. 60 ft.

LATERAL CAPACITY - AREA 8 (B-183)
36-IN DIAMETER CAST-IN-PLACE CONCRETE PILE (CRACKED CONDITIONS)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA

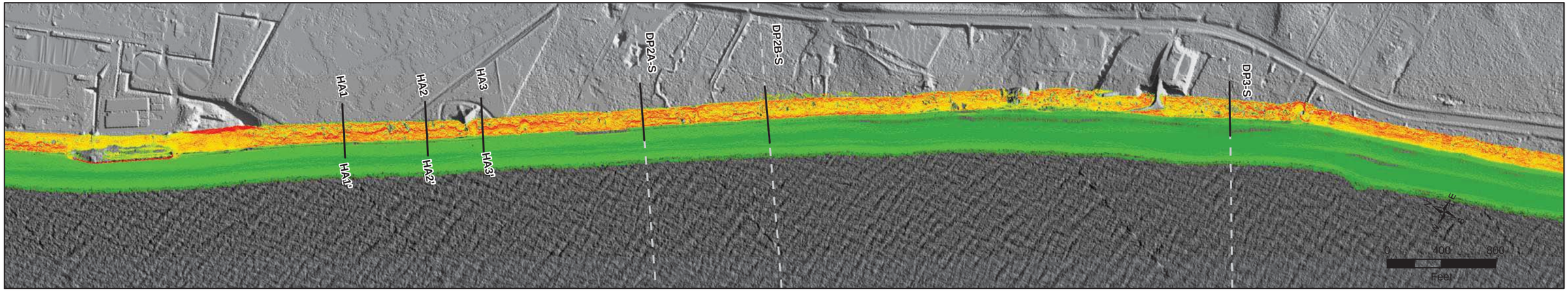


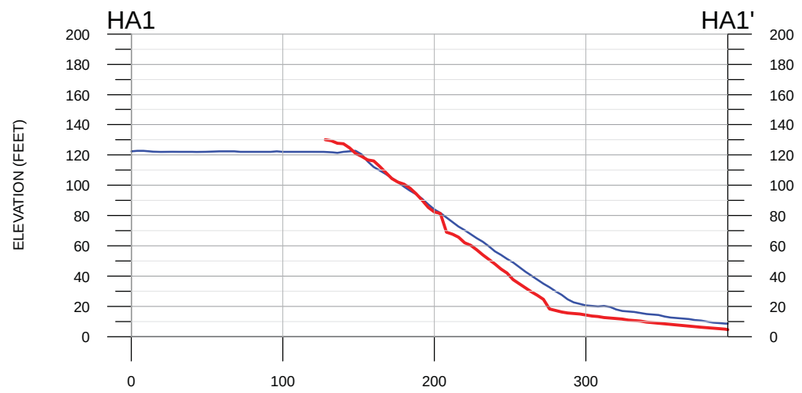
TYPICAL SECTION VIEW FOR PILE CAP
IN UN-HEATED AREAS WITH VOID FORM
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

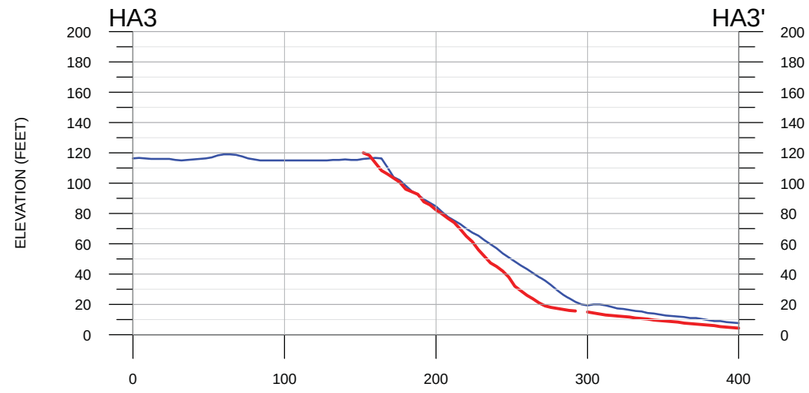
APPENDIX D

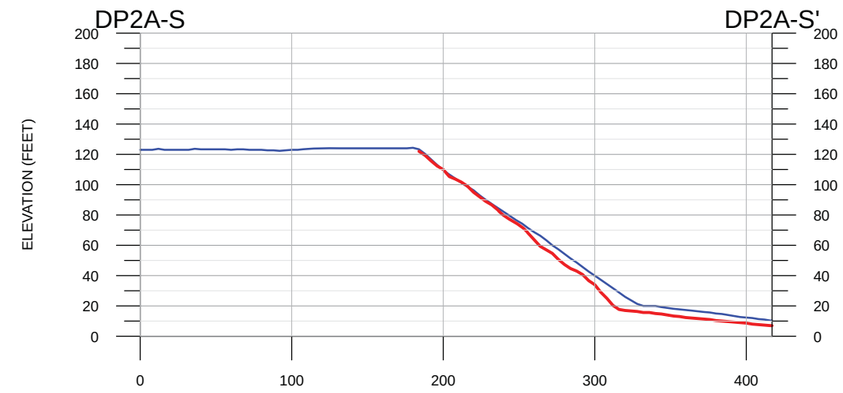
Bluff Slope Stability Recommendations

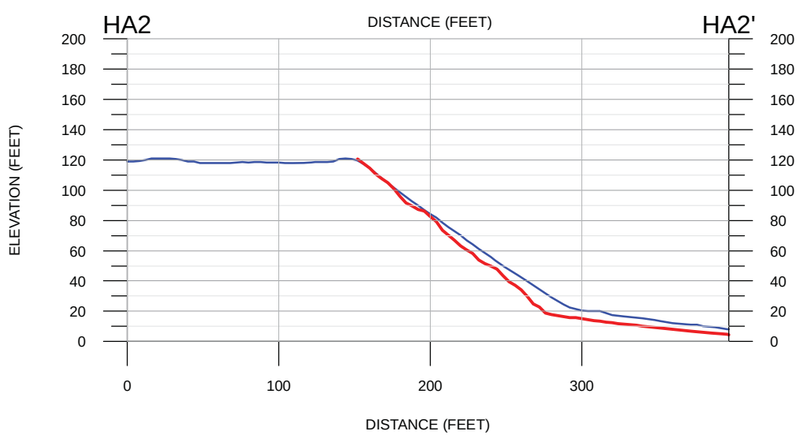
Overall Bluff Cross Sections	D-1
Overall Bluff Slope Angles	D-2a thru D-2f
Slope Configurations & Loading Conditions	D-3a thru D-3d
Area 1 - Static Slope Stability Results	D-4 thru D-13
Area 1 - Pseudo-Static Slope Stability Results	D-14 thru D-18
Area 2 - Static Slope Stability Results	D-19 thru D-20
Area 2 - Pseudo-Static Slope Stability Results	D-21 and D-22
Area 3 - Static Slope Stability Results	D-23 thru D-28
Area 3 - Pseudo-Static Slope Stability Results	D-29 thru D-31

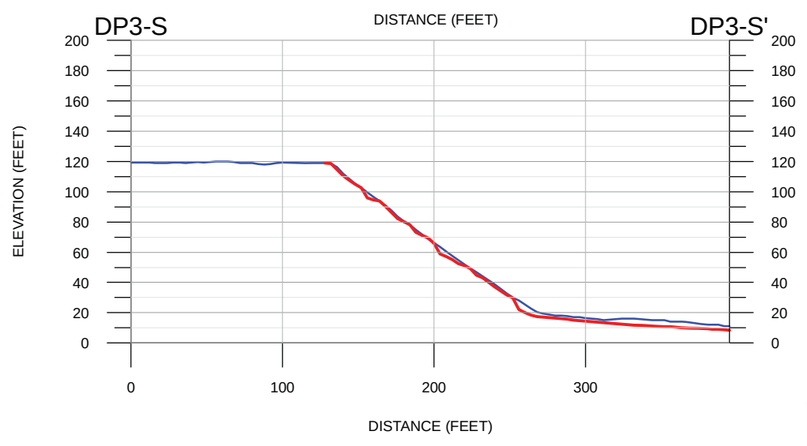


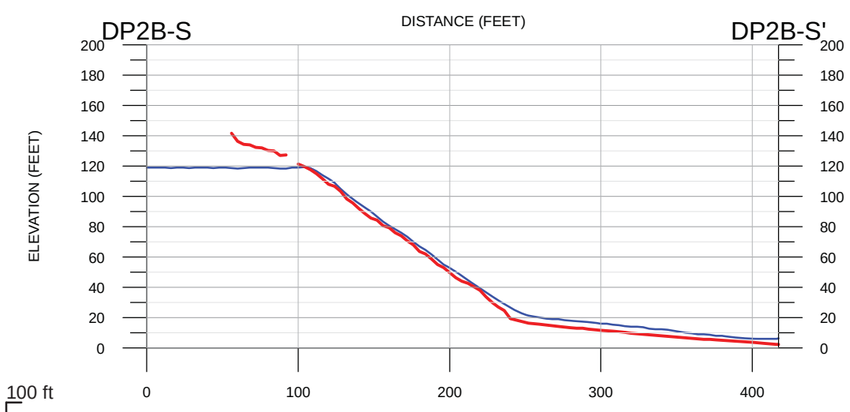












LEGEND

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

2015 Elevation in Feet (NAVD88 Datum)
Source: Fugro 2015 LiDAR data

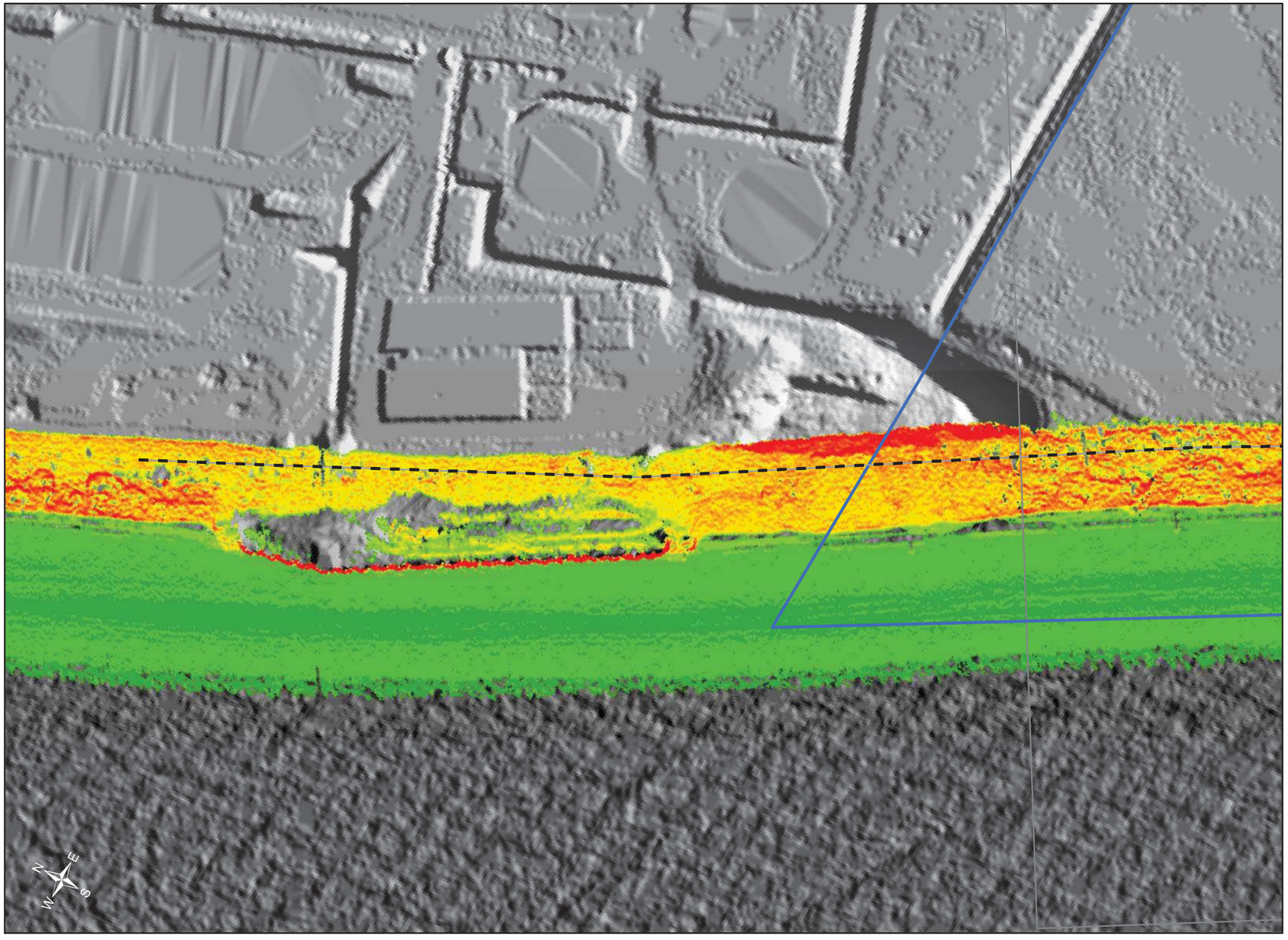
2012 Elevation in Feet (NAVD88 Datum)
Source: Kenai Peninsula Borough 2012 Digital Elevation Model

100 ft
100 ft
Vertical Exaggeration = 1.0X

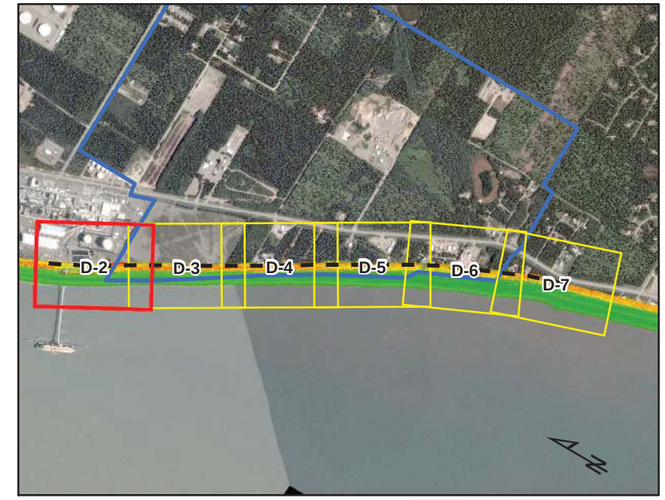
OVERALL BLUFF CROSS SECTIONS
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

PLATE D-1

D-2



0 200 400 Feet



0 2,000 4,000 Feet

LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

NOTES:

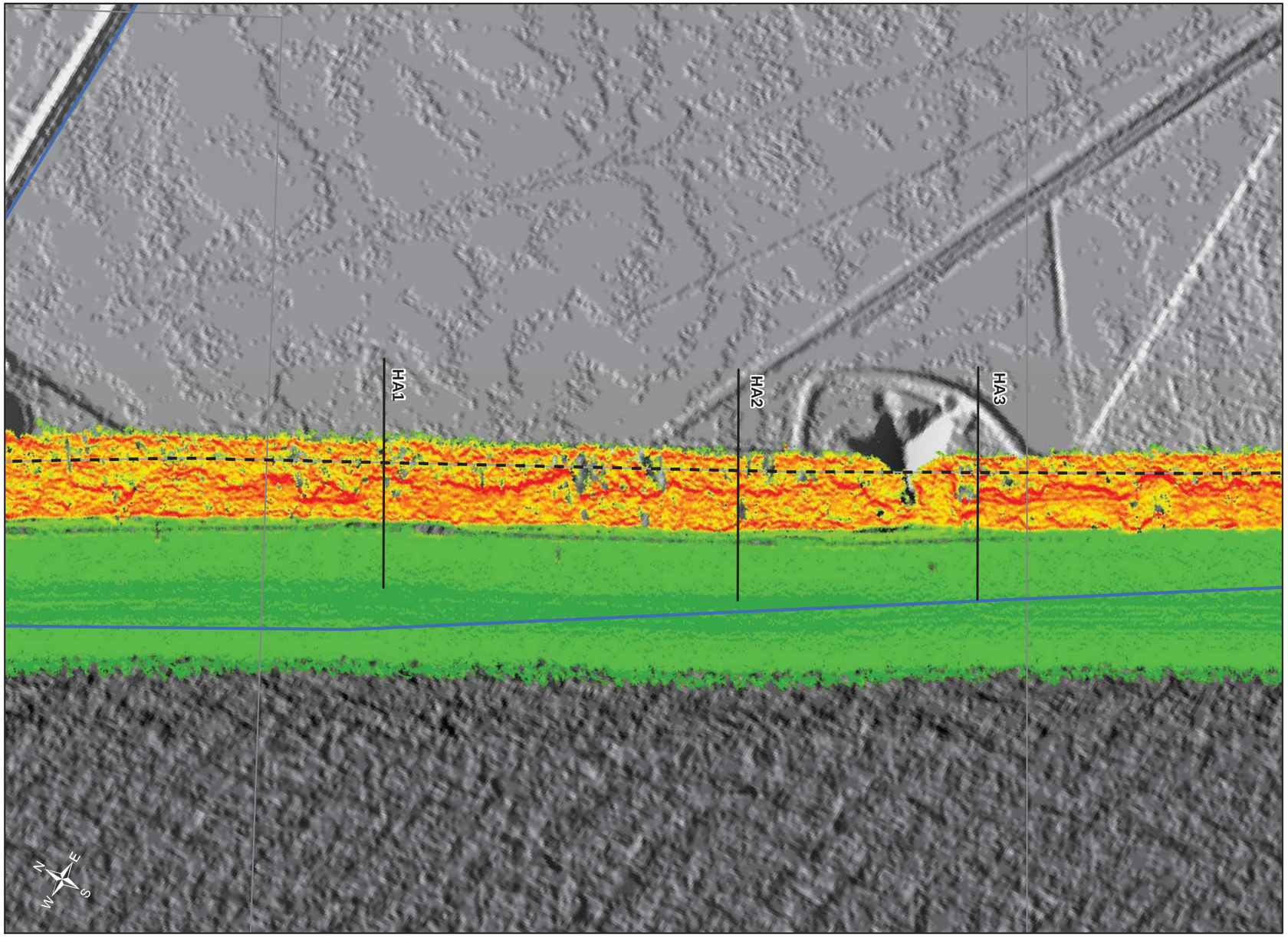
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

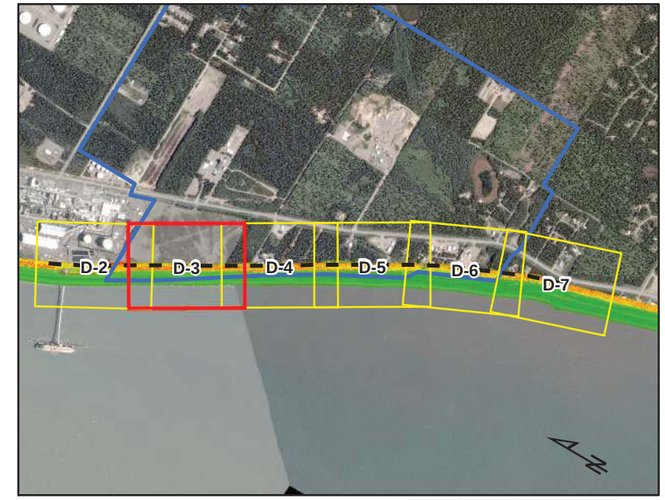
PLATE D-2a

R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk

D-3



0 200 400 Feet



0 2,000 4,000 Feet

LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

NOTES:

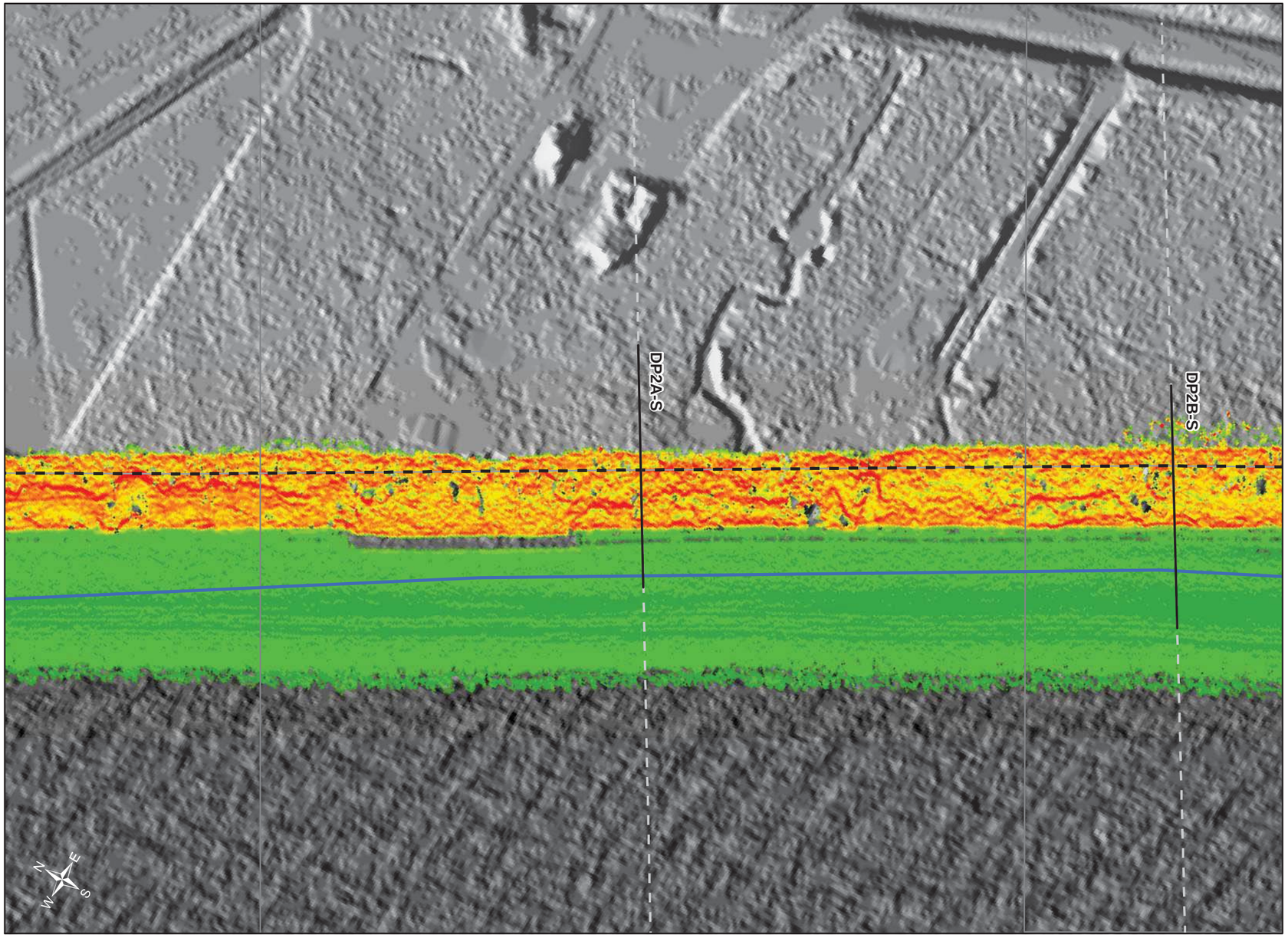
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

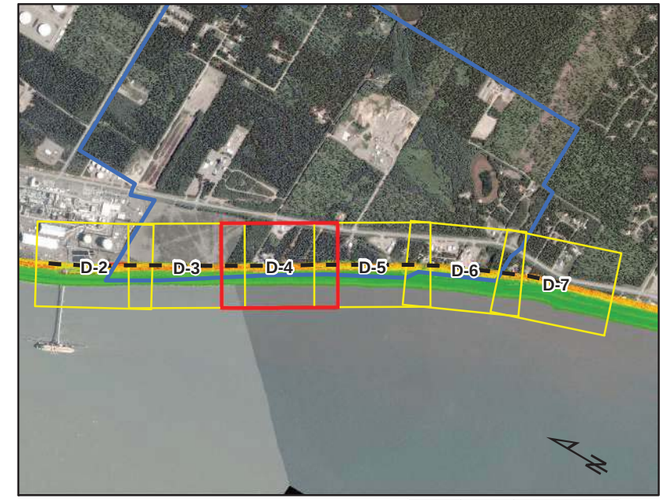
PLATE D-2b

R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk

D-4



0 200 400 Feet



0 2,000 4,000 Feet

LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

NOTES:

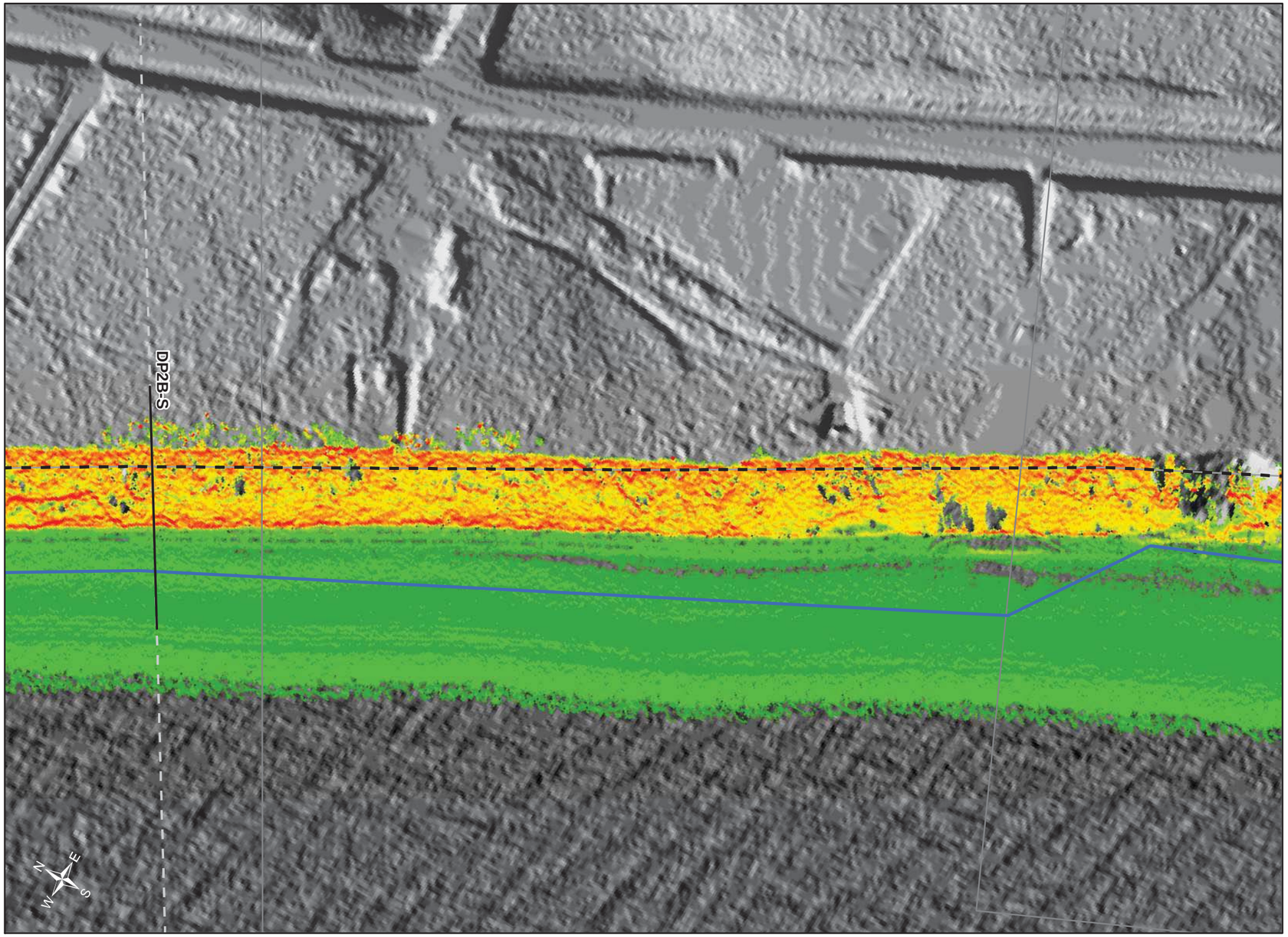
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

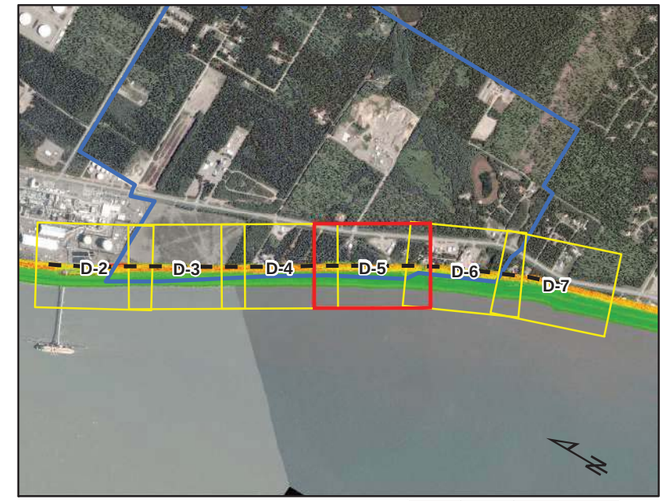
PLATE D-2c

R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk

D-5



0 200 400 Feet



0 2,000 4,000 Feet

LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

NOTES:

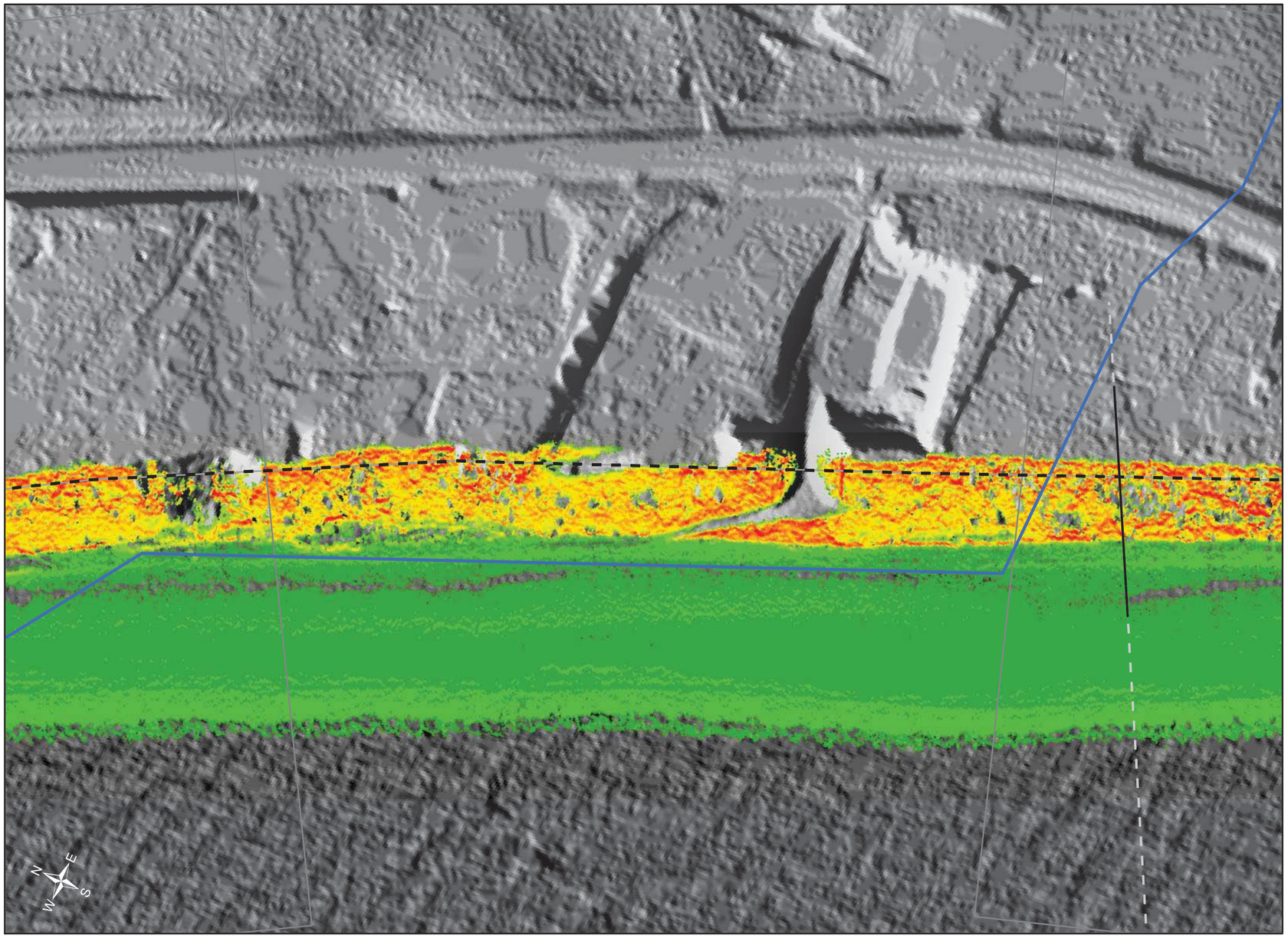
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

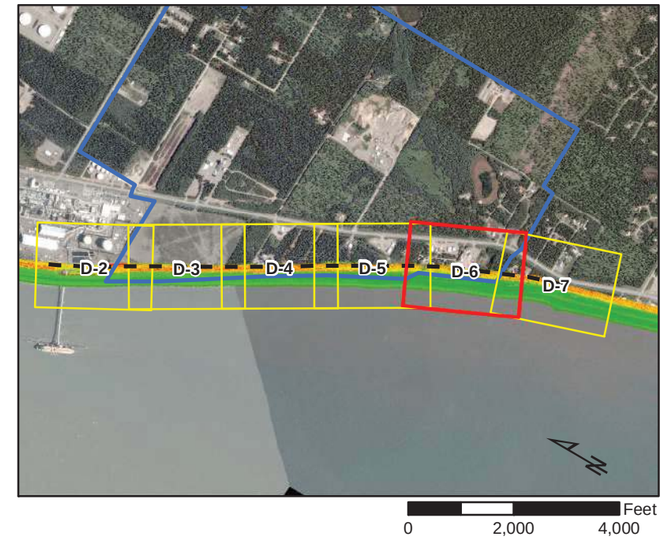
OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

PLATE D-2d

R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk

D-6





LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

NOTES:

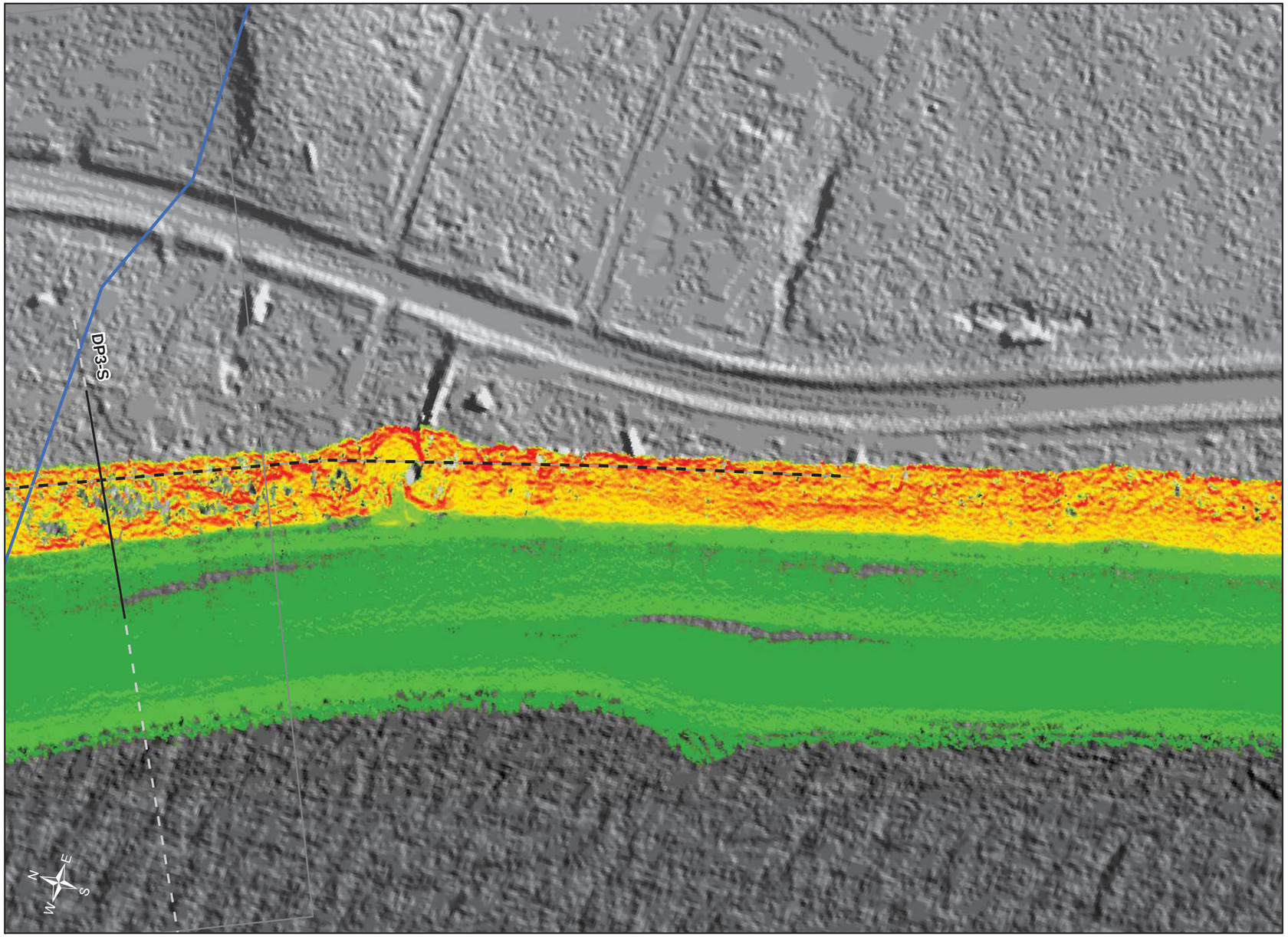
1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

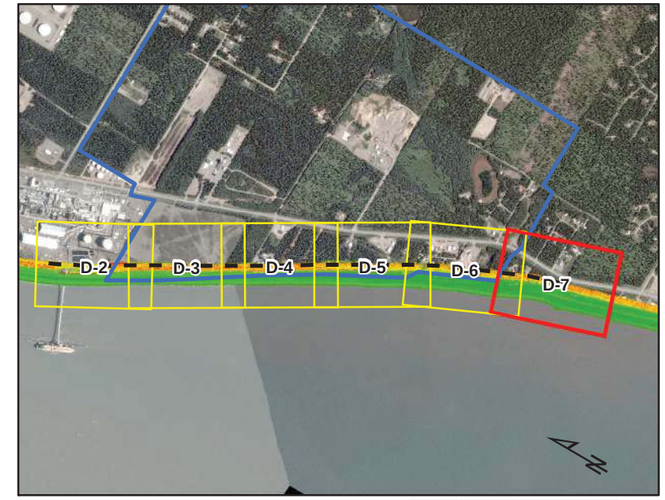
PLATE D-2e

R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk

D-7



0 200 400 Feet



0 2,000 4,000 Feet

LEGEND

Onshore LNG Facilities Study Area

SLOPE (Degrees)

- 0 - 5
- 5 - 10
- 10 - 20
- 20 - 25
- 25 - 30
- 30- 35
- 35 - 40
- 40 - 45
- 45 - 50
- 50 - 80

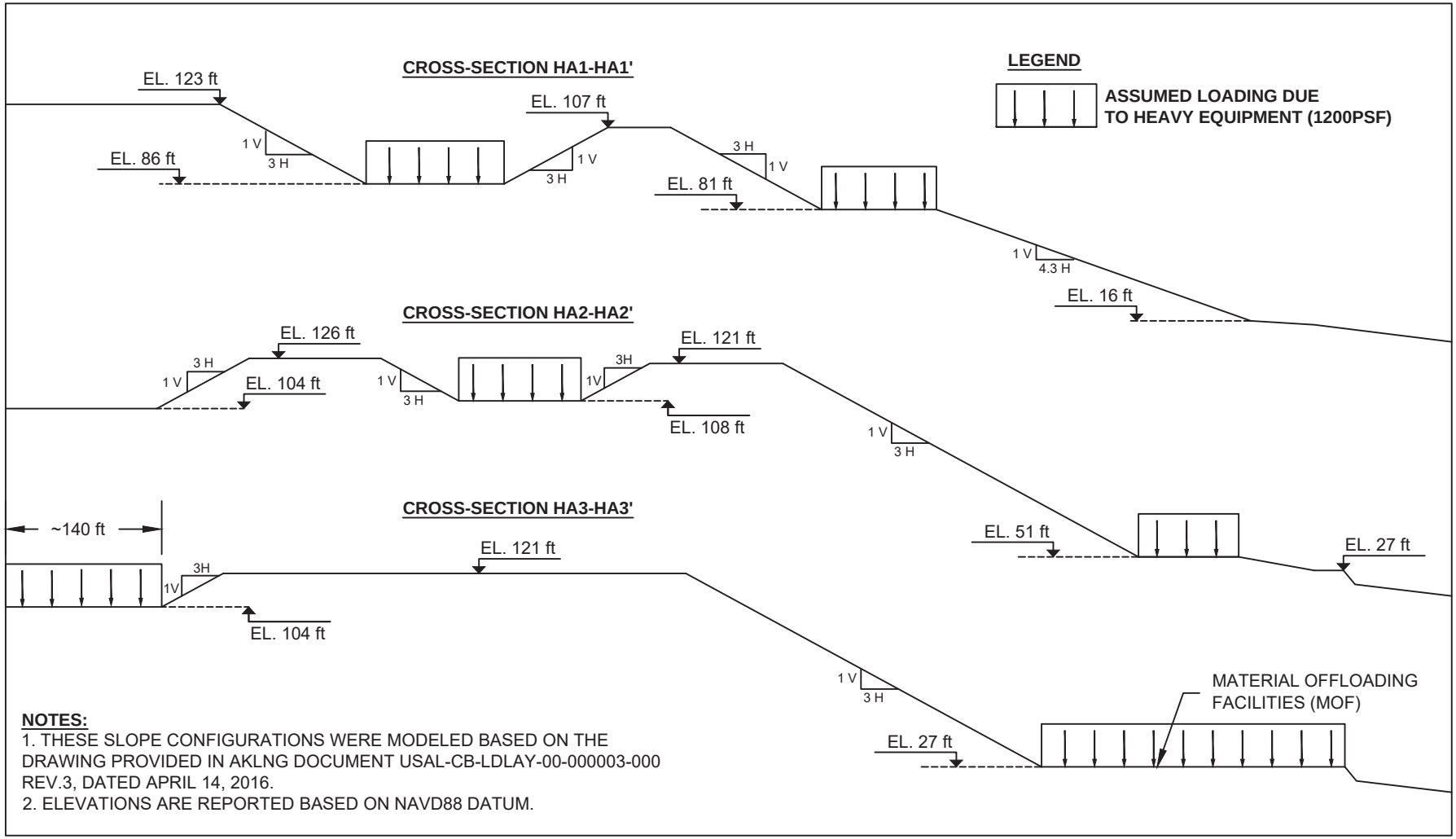
NOTES:

1. Onshore LNG Facilities Study Area provided by AKLNG.
2. Slope is calculated from ground LiDAR (Fugro, 2015).
3. Hillshade map is created from LiDAR Digital Elevation Model (DEM), (KPB, 2008).

OVERALL BLUFF SLOPE ANGLES
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISI, ALASKA

PLATE D-2f

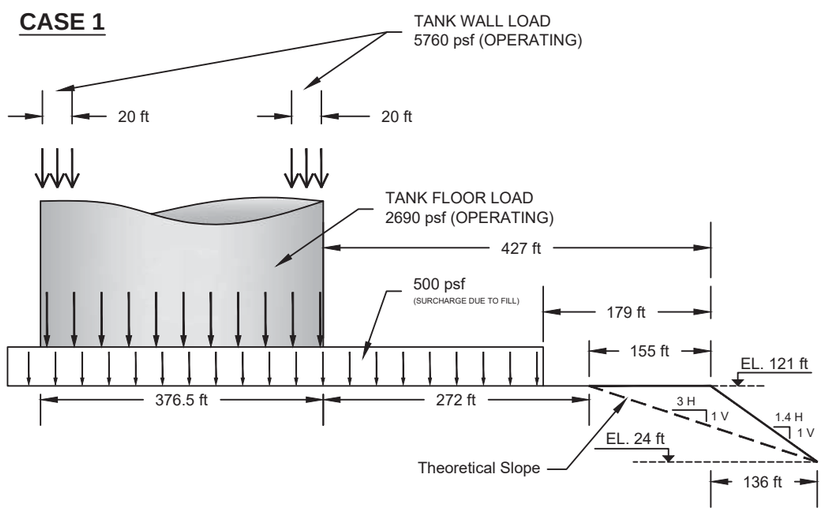
R:\04100\2014 Projects\04.10140094 - AKLNG\05_GIS\Outputs\2015_01_29_Slope_Analysis\MXD\SlopeComparison_MapBook.mxd, 12/16/2016, kimk



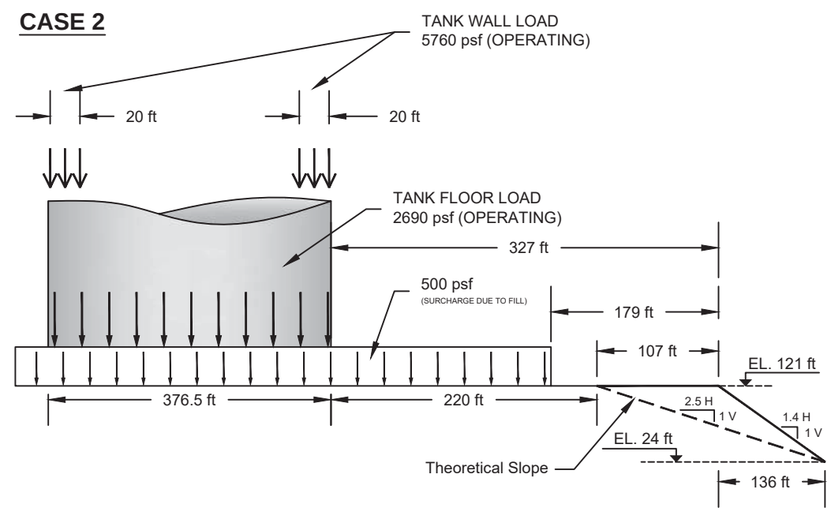
NOTES:
1. THESE SLOPE CONFIGURATIONS WERE MODELED BASED ON THE DRAWING PROVIDED IN AKLNG DOCUMENT USAL-CB-LDLAY-00-000003-000 REV.3, DATED APRIL 14, 2016.
2. ELEVATIONS ARE REPORTED BASED ON NAVD88 DATUM.

SLOPE AND LOADING CONFIGURATIONS FOR STATIC AND PSEUDO-STATIC ANALYSES
BLUFF SLOPE STABILITY - AREA 1 (HEAVY HAUL ROAD)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE D-3a

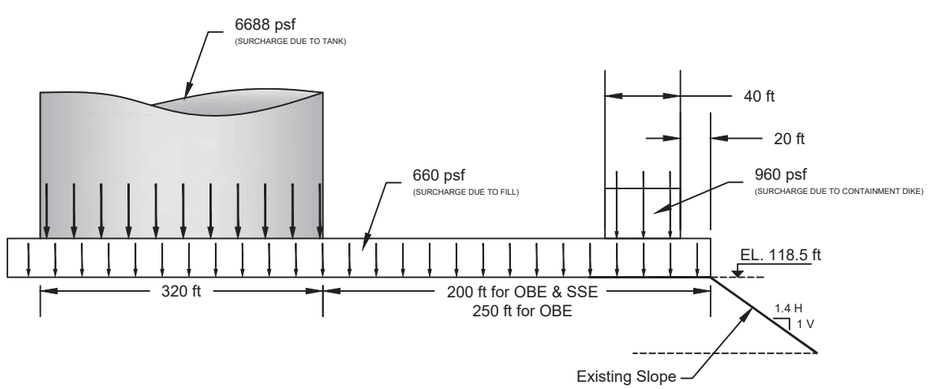


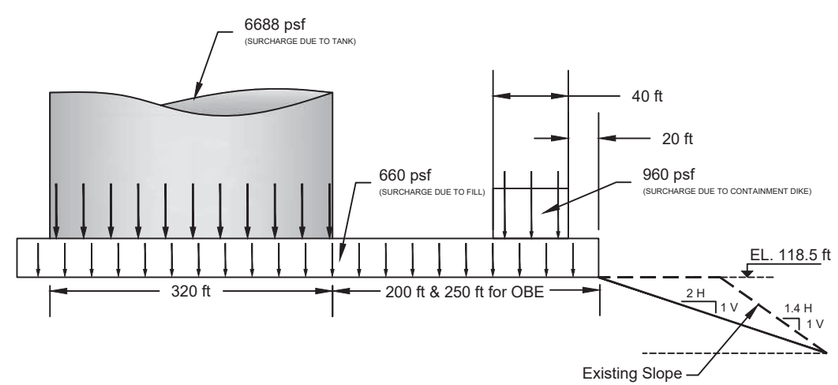
NOTE:
CASE 1 SLOPE CONFIGURATION WAS MODELED BASED ON THE DRAWINGS PROVIDED IN AKLNG DOCUMENT NO. USAL-CB-LDLAY-00-000003-000 REV.3, DATED APRIL 14, 2016.



NOTE:
CASE 2 SLOPE CONFIGURATION WAS MODELED BASED ON THE DRAWINGS PROVIDED IN AKLNG DOCUMENT NO. USAL-CB-LDLAY-00-000003-000 REV.3, DATED APRIL 14, 2016.

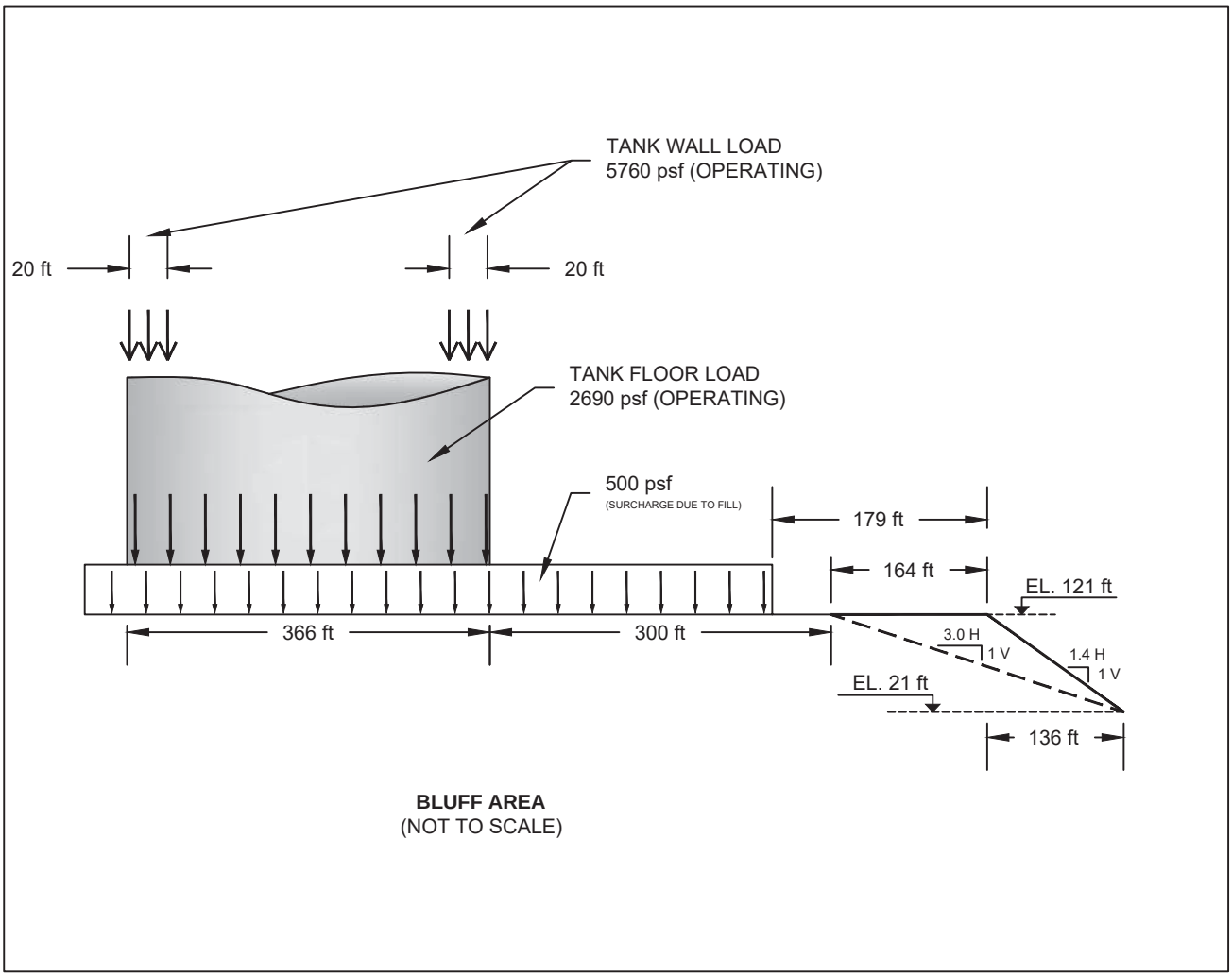
PRELIMINARY CASES





NOTES:
1. THESE PRELIMINARY CASES CORRESPOND TO THE PRELIMINARY LOAD CONFIGURATIONS AND SLOPE INCLINATIONS PRESENTED IN THE APPENDIX A OF THE AKLNG DOC. NO USAL-FG-GRZZZ-00-002015-018 (APPENDIX E OF THIS REPORT).
2. PRELIMINARY CASES WERE ANALYZED FOR 1.4H:1V FOR OBE & SSE SEISMIC LOADING AND 2H:1V FOR OBE SEISMIC LOADING CONDITIONS.
3. PRELIMINARY CASES WERE MODELED BASED ON PLOT PLAN LAYOUT PROVIDED IN PACKAGE USAL-PL-CARFI-00-000002-00 (APPENDIX B).
4. ELEVATIONS ARE REPORTED BASED ON NAVD88 DATUM.

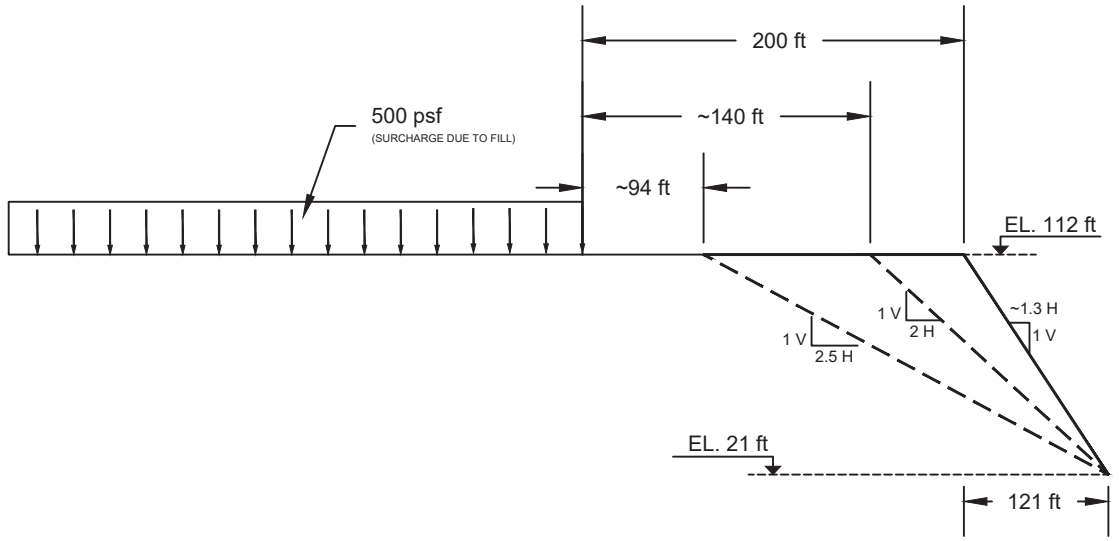
SLOPE AND LOADING CONFIGURATIONS FOR FLAC ANALYSES
BLUFF SLOPE STABILITY
AREA 2 (LNG TANKS)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)



NOTES:
1. THIS SLOPE CONFIGURATION WAS MODELED BASED ON THE DRAWINGS PROVIDED IN AKLNG DOCUMENT USAL-CB-LDLAY-00-000003-000 REV.6, DATED JUNE 30, 2016.
2. ELEVATIONS ARE REPORTED BASED ON NAVD88 DATUM.

SLOPE AND LOADING CONFIGURATIONS FOR STATIC AND PSEUDO-STATIC ANALYSES
BLUFF SLOPE STABILITY - AREA 2 (LNG TANKS)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE D-3c



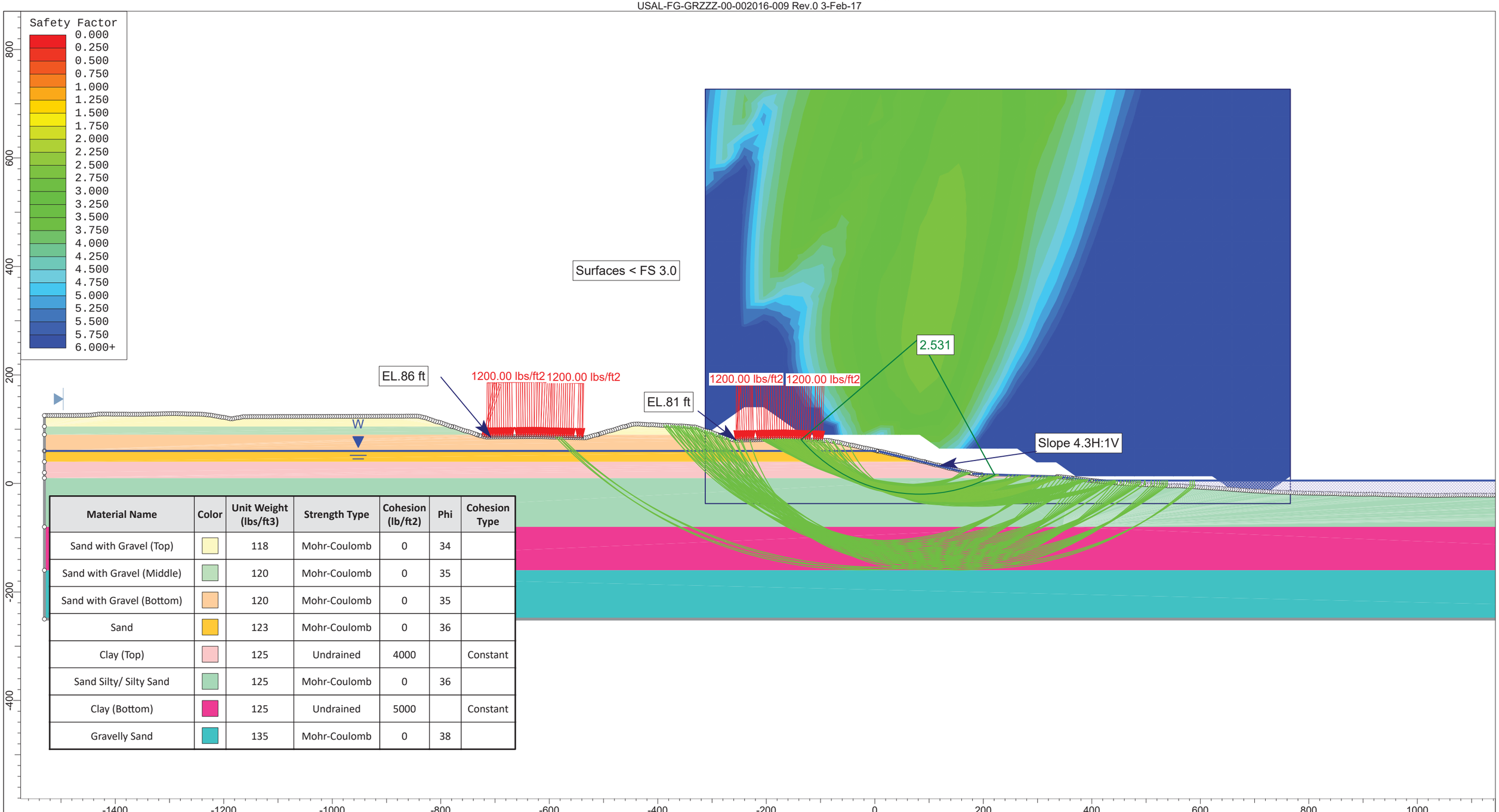
BLUFF AREA
(NOT TO SCALE)

NOTES:
1. THIS SLOPE CONFIGURATION WAS MODELED BASED ON THE DRAWINGS PROVIDED IN AKLNG DOCUMENT USAL-CB-LDLAY-00-000003-000 REV.6, DATED JUNE 30, 2016.
2. ELEVATIONS ARE REPORTED BASED ON NAVD88 DATUM.

SLOPE AND LOADING CONFIGURATIONS FOR STATIC AND PSEUDO-STATIC ANALYSES
BLUFF SLOPE STABILITY - AREA 3 (OUTFALL)
ONSHORE LNG FACILITIES
ALASKA LNG PROJECT
NIKISKI, ALASKA
(NOT TO SCALE)

PLATE D-3d

AREA 1 (HEAVY HAUL ROAD) – SLOPE STABILITY RESULTS

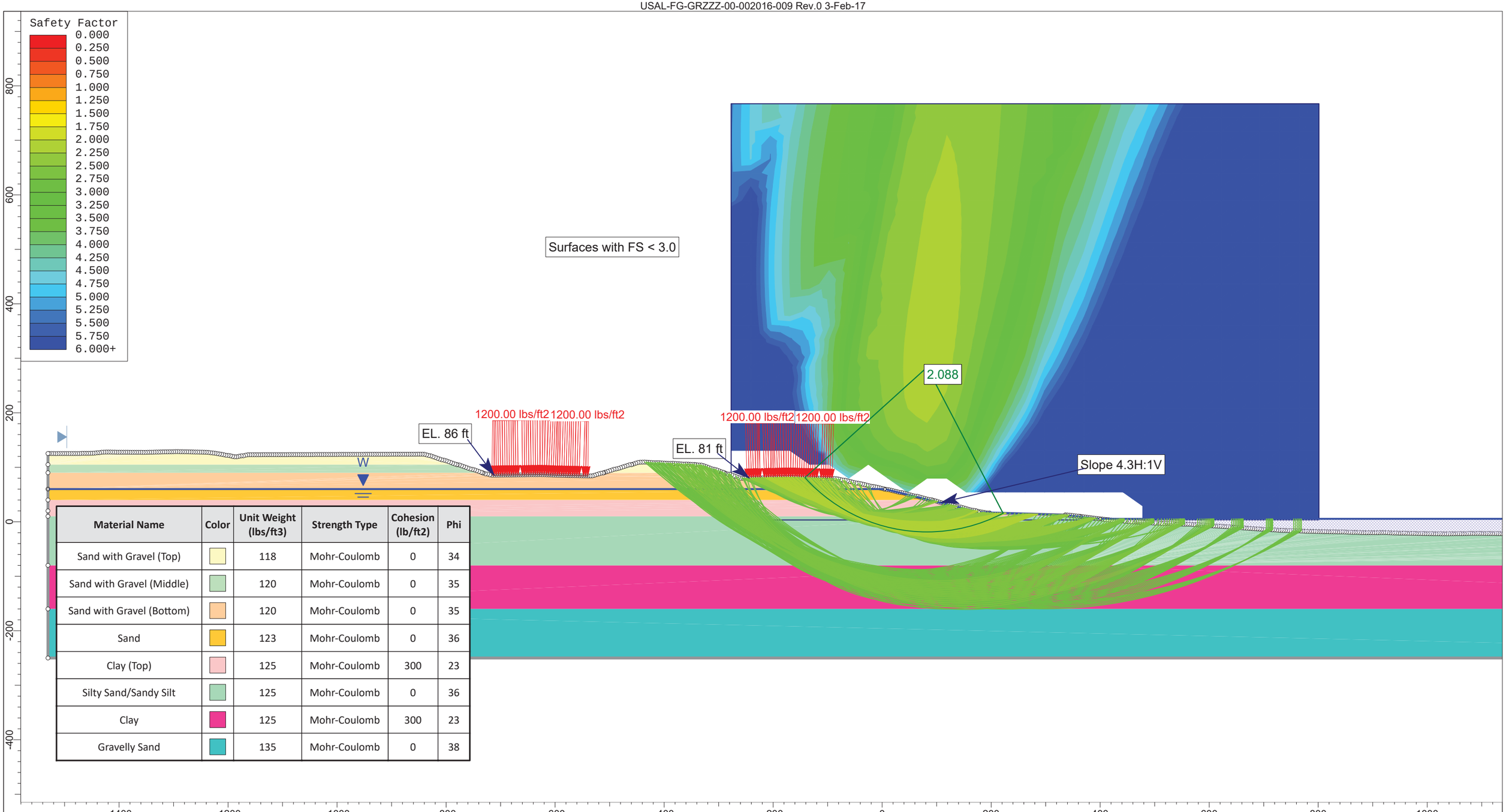


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sand Silty/ Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	

FUGRO

SLIDEINTERPRET 6.012

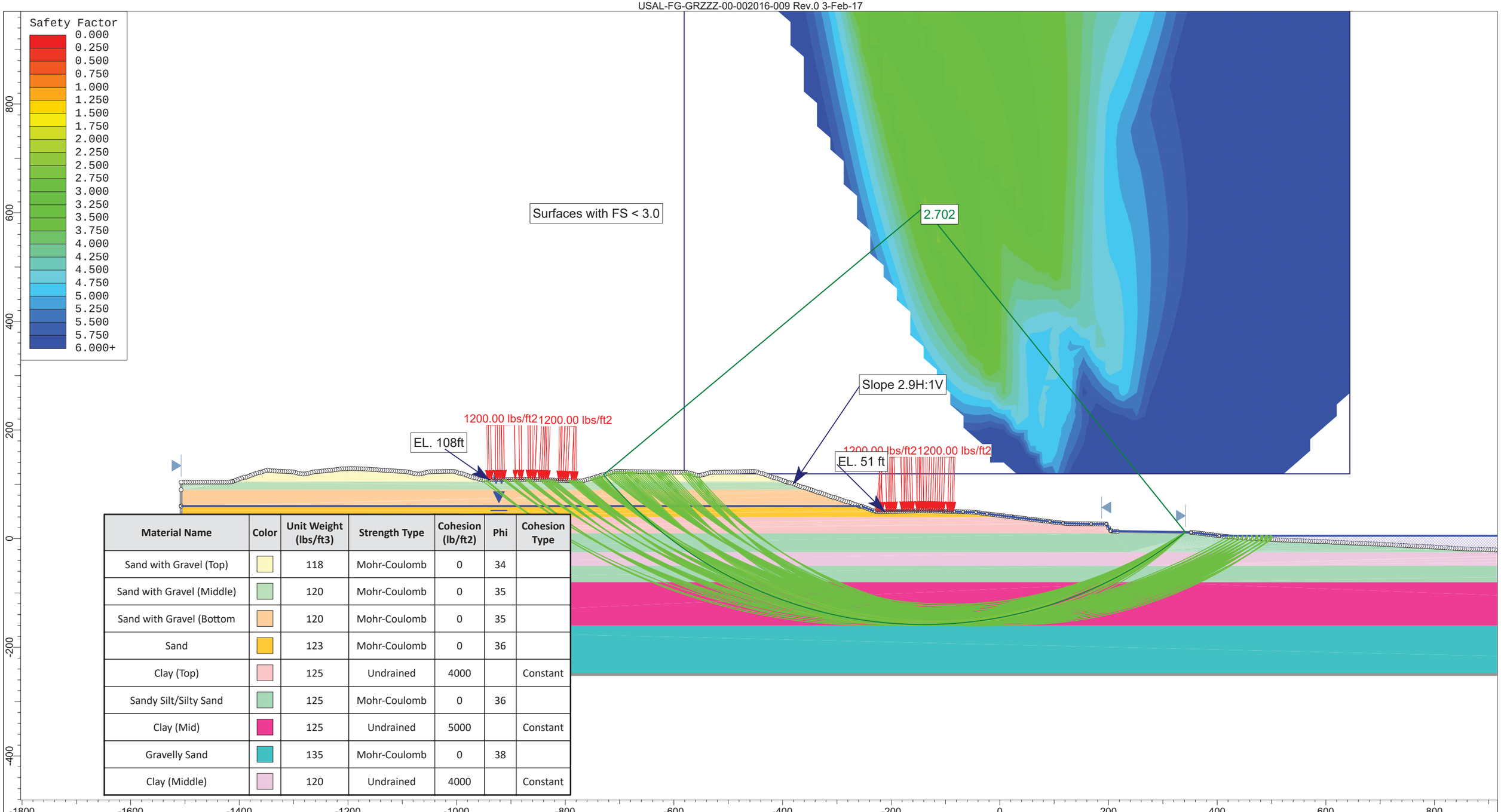
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA1-HA1'), static (undrained) condition		
Drawn By	Aditya Zutshi	*Scale* 1:2000	*Company* Fugro Consultants
Date	4/14/2016, 8:54:57 AM		*File Name* Plate D-4 (HA1_static-undrained).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35
Sand		123	Mohr-Coulomb	0	36
Clay (Top)		125	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Clay		125	Mohr-Coulomb	300	23
Gravelly Sand		135	Mohr-Coulomb	0	38

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA1-HA1'), static (drained) condition		
Drawn By	Aditya Zutshi	*Scale* 1:2000	*Company* Fugro Consultants
Date	4/14/2016, 8:54:57 AM		*File Name* Plate D-5 (HA1_static-drained).slim

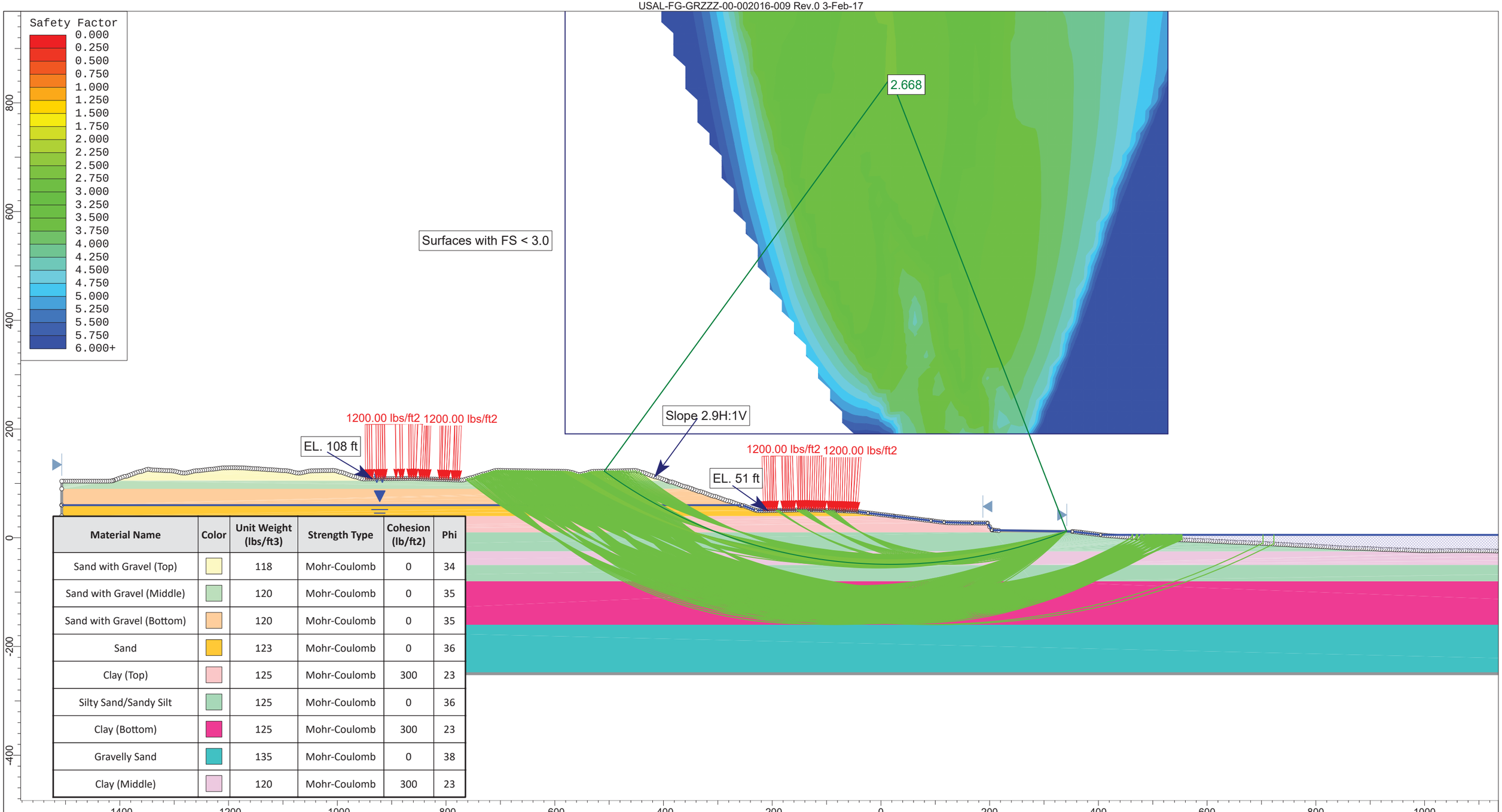


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sandy Silt/Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Mid)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), static (undrained) condition		
Drawn By	Aditya Zutshi	*Scale* 1:2000	*Company* Fugro Consultants Inc.
Date	4/13/2016, 2:17:17 PM		*File Name* Plate D-6 (HA2_static-undrained).slim

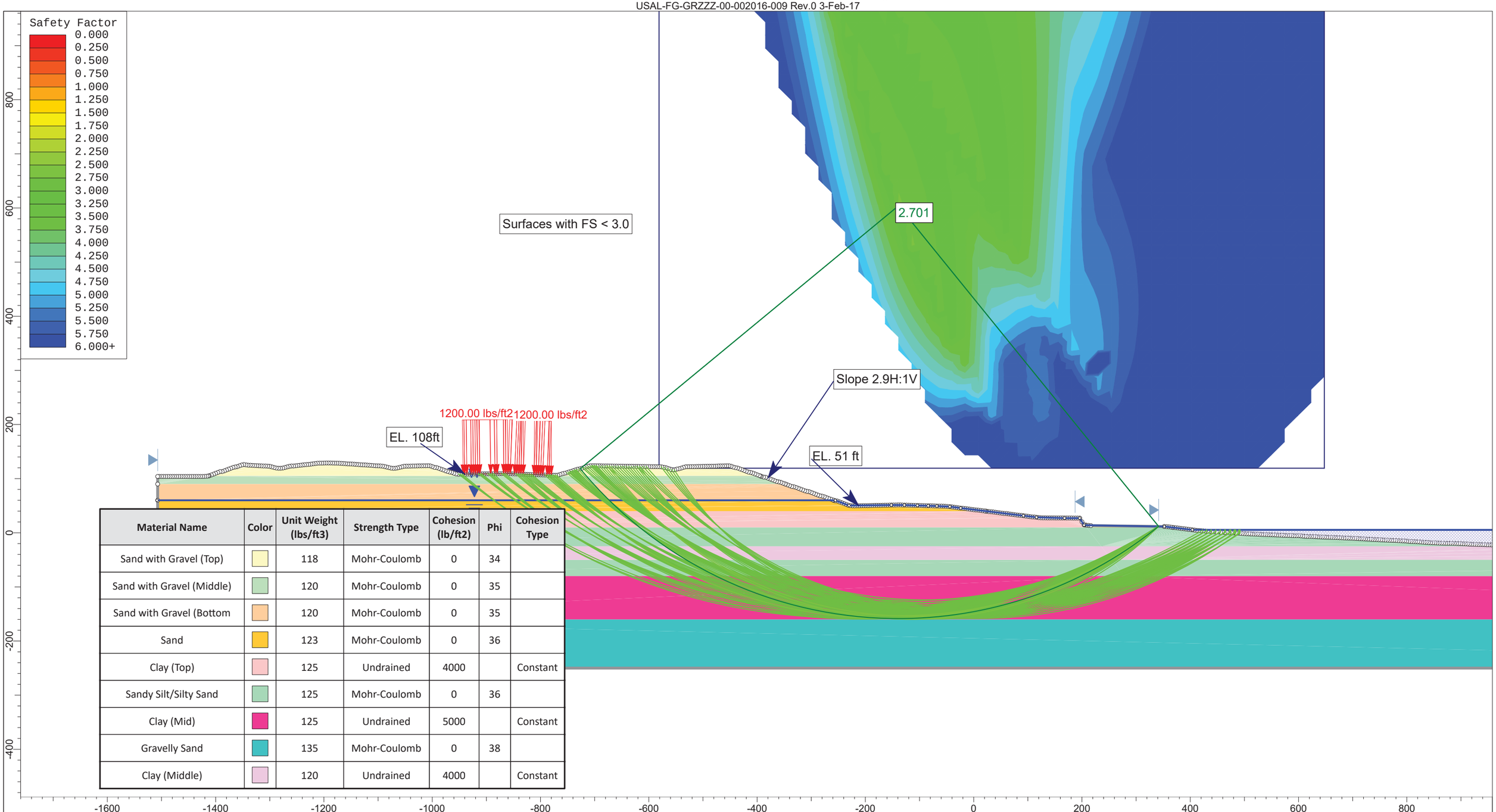


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35
Sand		123	Mohr-Coulomb	0	36
Clay (Top)		125	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Clay (Bottom)		125	Mohr-Coulomb	300	23
Gravelly Sand		135	Mohr-Coulomb	0	38
Clay (Middle)		120	Mohr-Coulomb	300	23

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), static (drained) condition		
Drawn By	Aditya Zutshi	Scale 1:2000	Company Fugro Consultants Inc.
Date	4/13/2016, 2:17:17 PM		File Name Plate D-7 (HA2_static-drained).slim

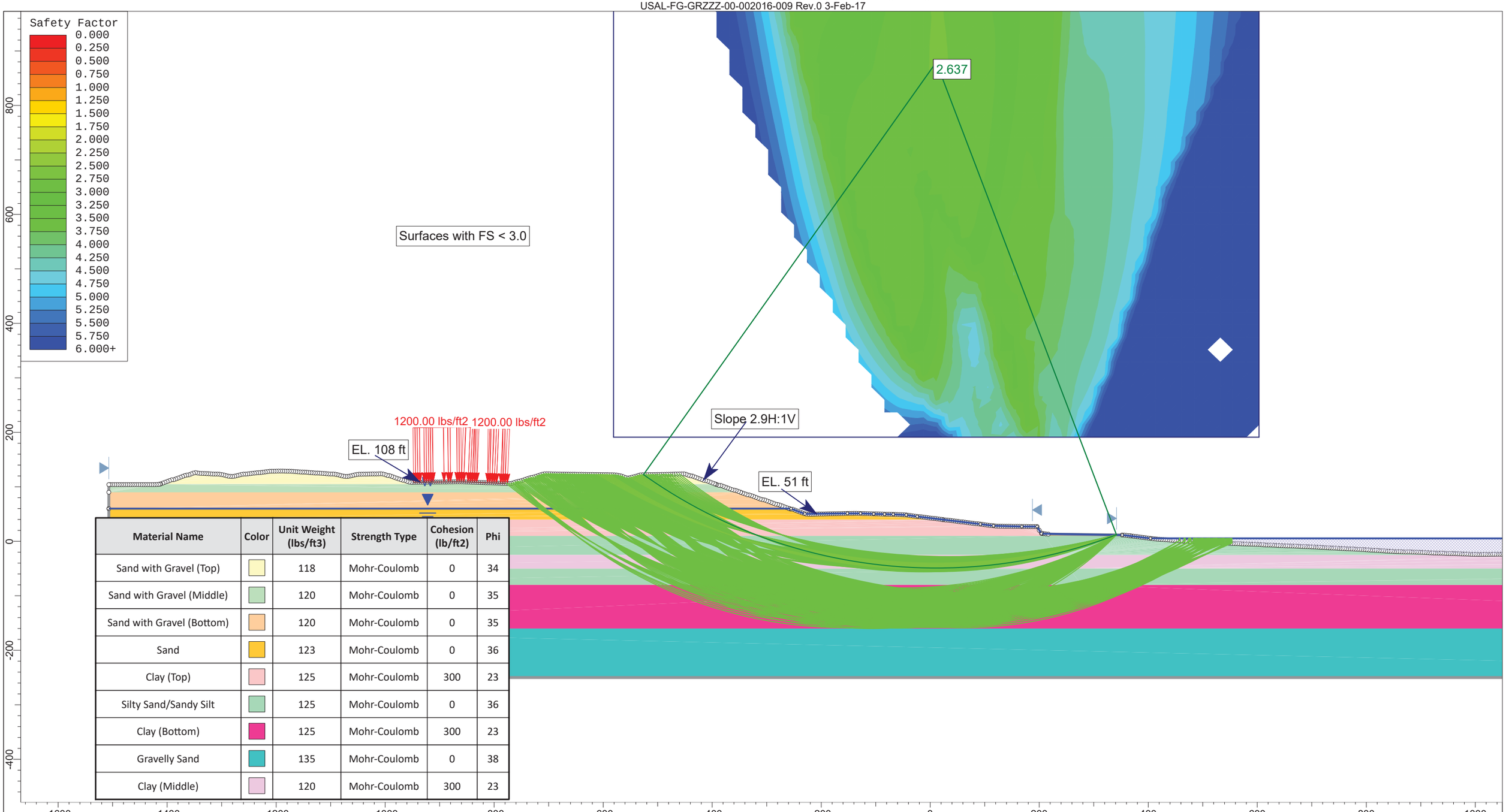


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sandy Silt/Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Mid)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

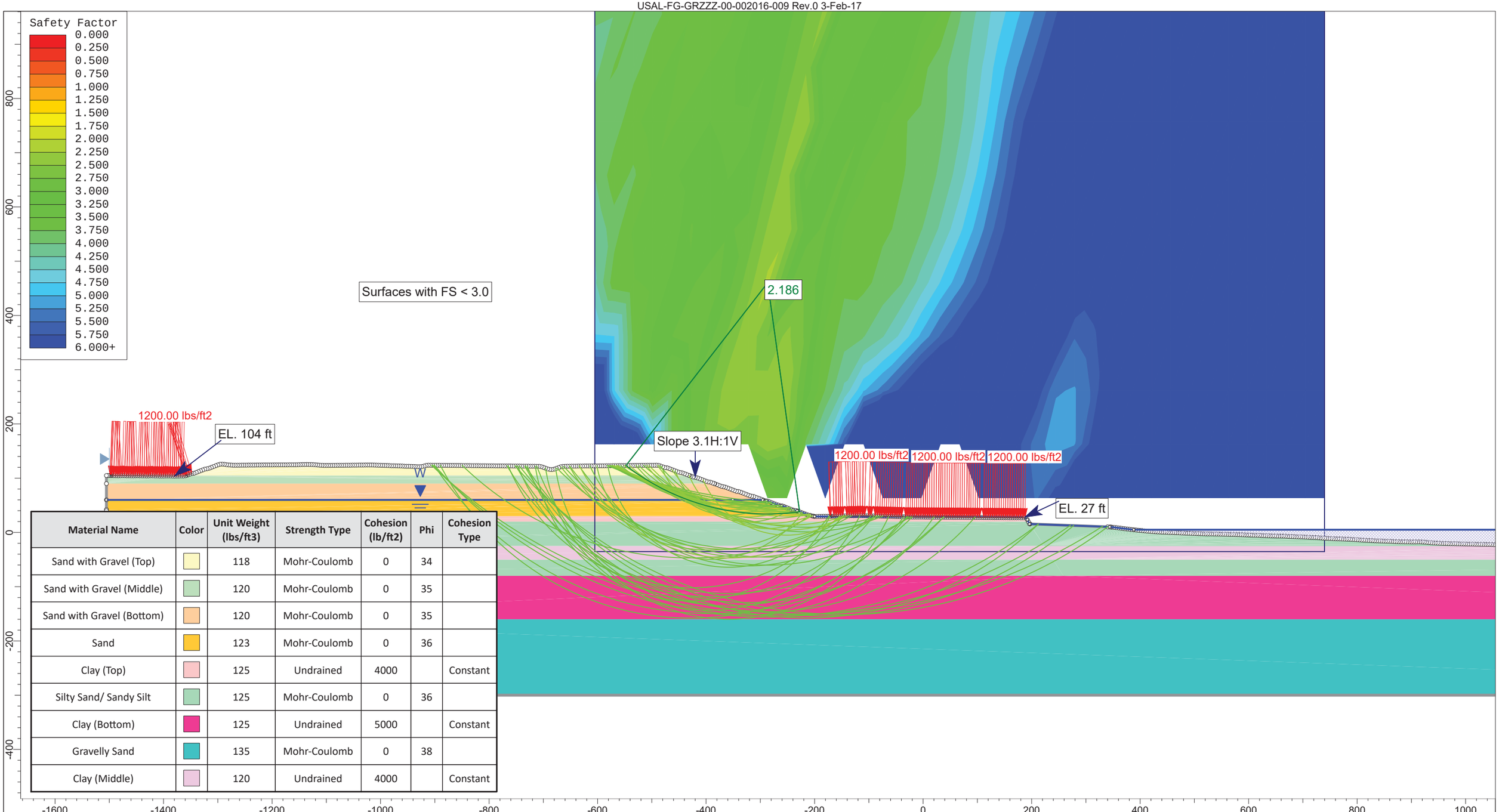
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), static (undrained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	*Scale* 1:2000	*Company* Fugro Consultants Inc.
Date	4/13/2016, 2:17:17 PM		*File Name* Plate D-8 (HA2_static-undrained) no bot load.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35
Sand		123	Mohr-Coulomb	0	36
Clay (Top)		125	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Clay (Bottom)		125	Mohr-Coulomb	300	23
Gravelly Sand		135	Mohr-Coulomb	0	38
Clay (Middle)		120	Mohr-Coulomb	300	23

SLIDEINTERPRET 6.012

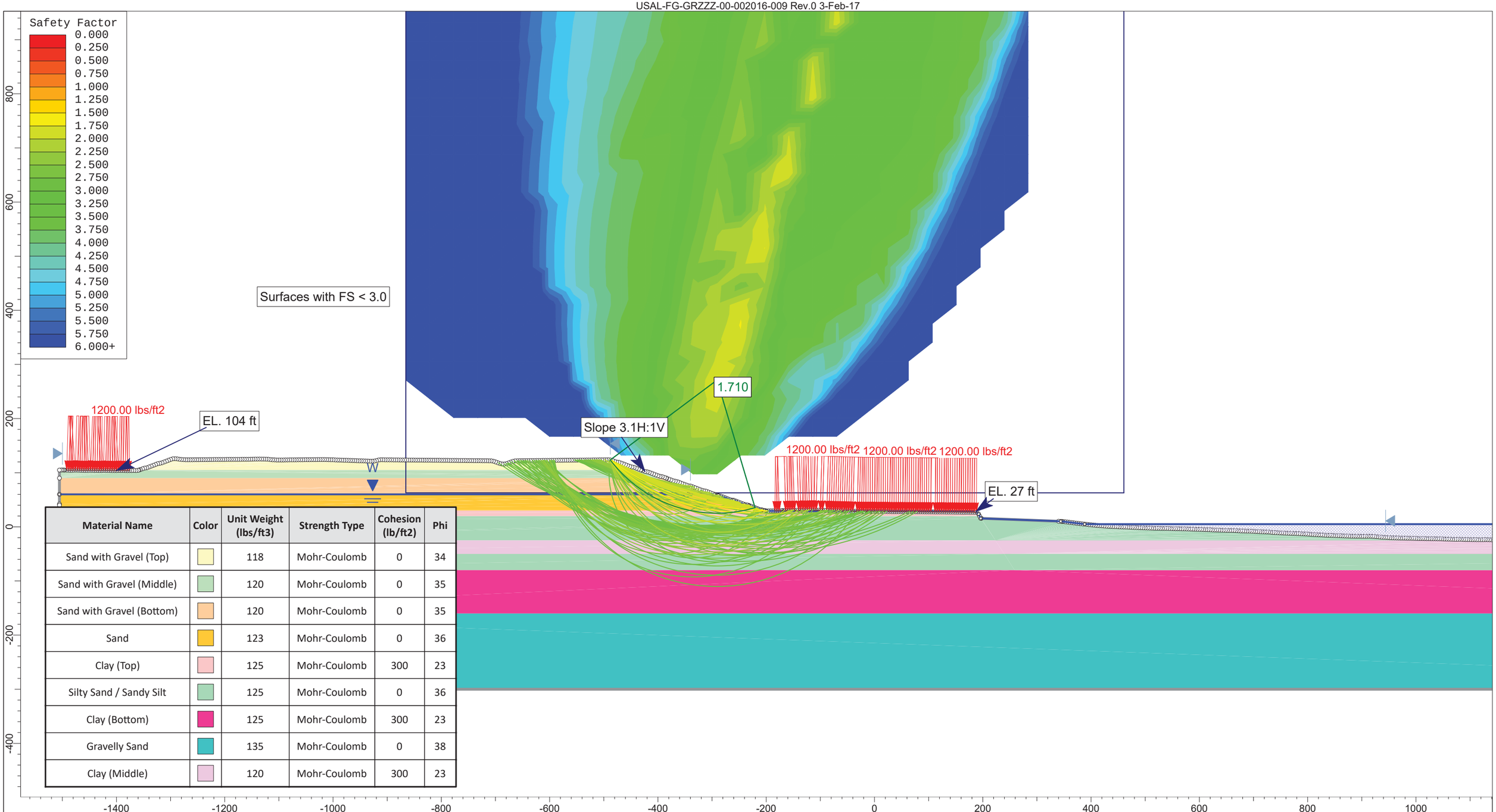
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), static (drained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	Scale	1:2000
Company	Fugro Consultants Inc.		
Date	4/13/2016, 2:17:17 PM	File Name	Plate D-9 (HA2_static-drained) no bot load.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Silty Sand/ Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

SLIDEINTERPRET 6.012

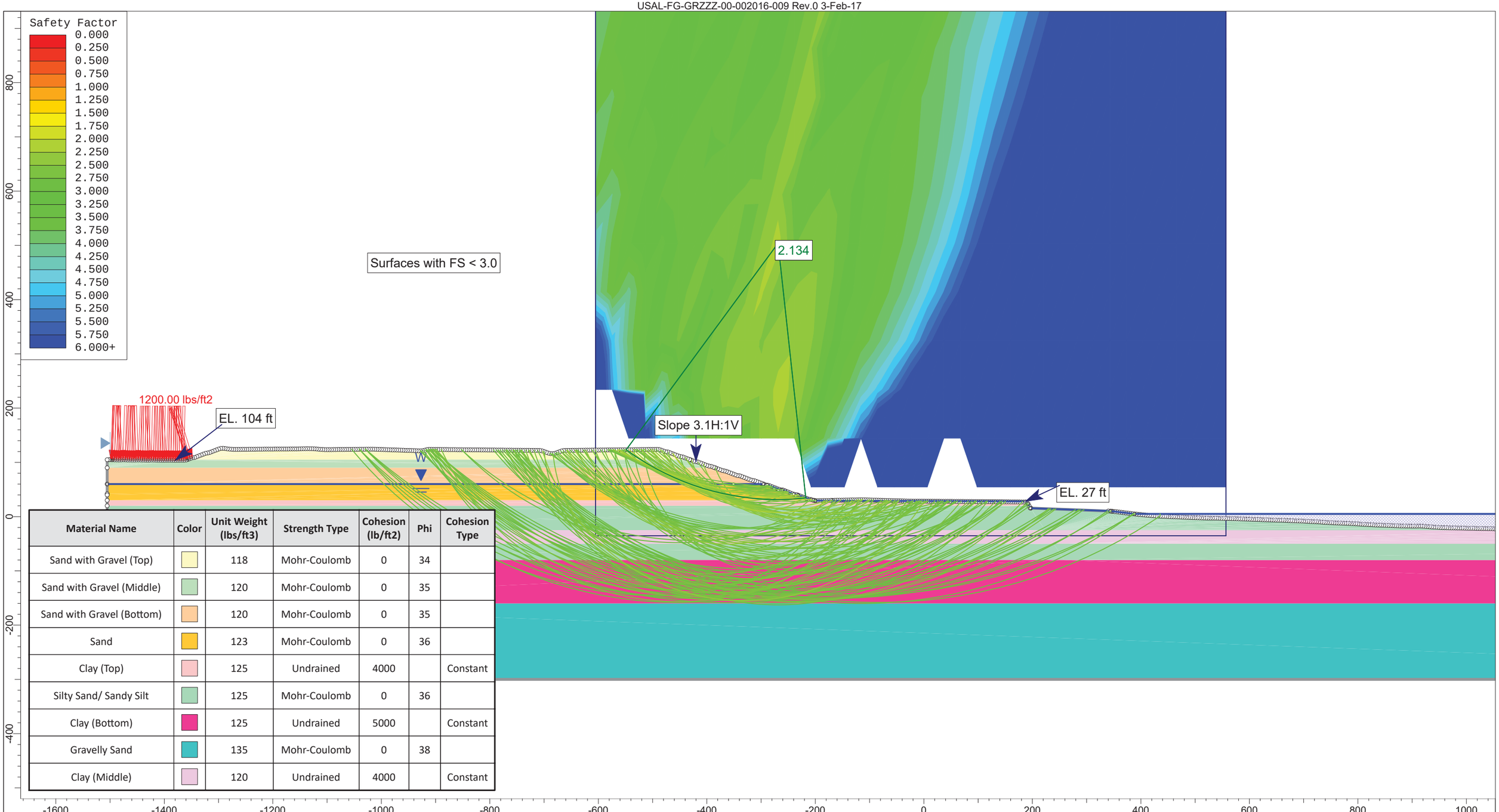
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), static (undrained) condition		
Drawn By	Aditya Zutshi	Scale 1:2000	Company Fugro Consultants
Date	4/14/2016, 8:47:22 AM		File Name Plate D-10 (HA3_static-undrained).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35
Sand		123	Mohr-Coulomb	0	36
Clay (Top)		125	Mohr-Coulomb	300	23
Silty Sand / Sandy Silt		125	Mohr-Coulomb	0	36
Clay (Bottom)		125	Mohr-Coulomb	300	23
Gravelly Sand		135	Mohr-Coulomb	0	38
Clay (Middle)		120	Mohr-Coulomb	300	23

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), static (drained) condition		
Drawn By	Aditya Zutshi	Scale	1:2000
Company	Fugro Consultants		
Date	4/14/2016, 8:47:22 AM	File Name	Plate D-11 (HA3_static-drained).slim

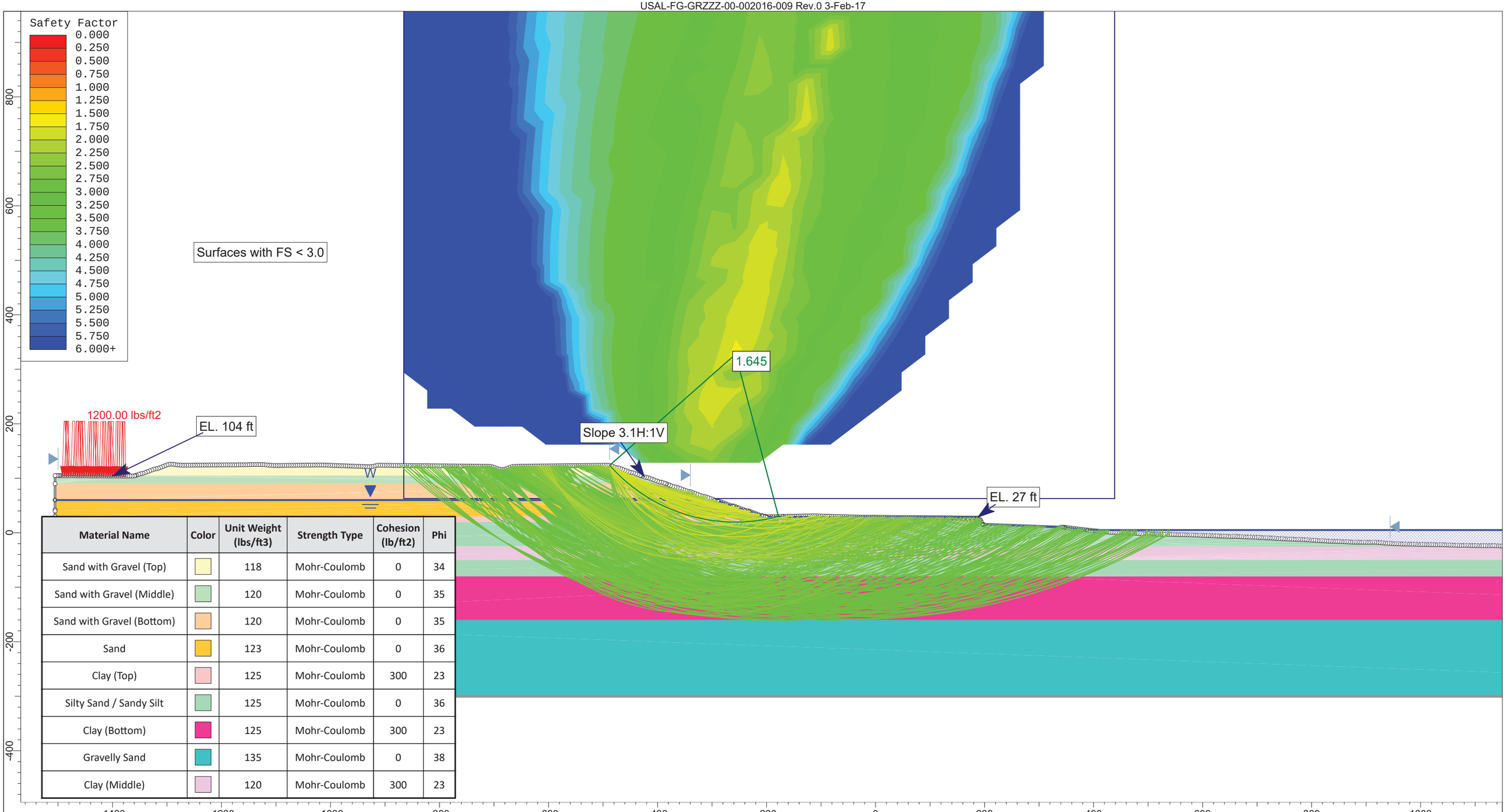


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Silty Sand/ Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

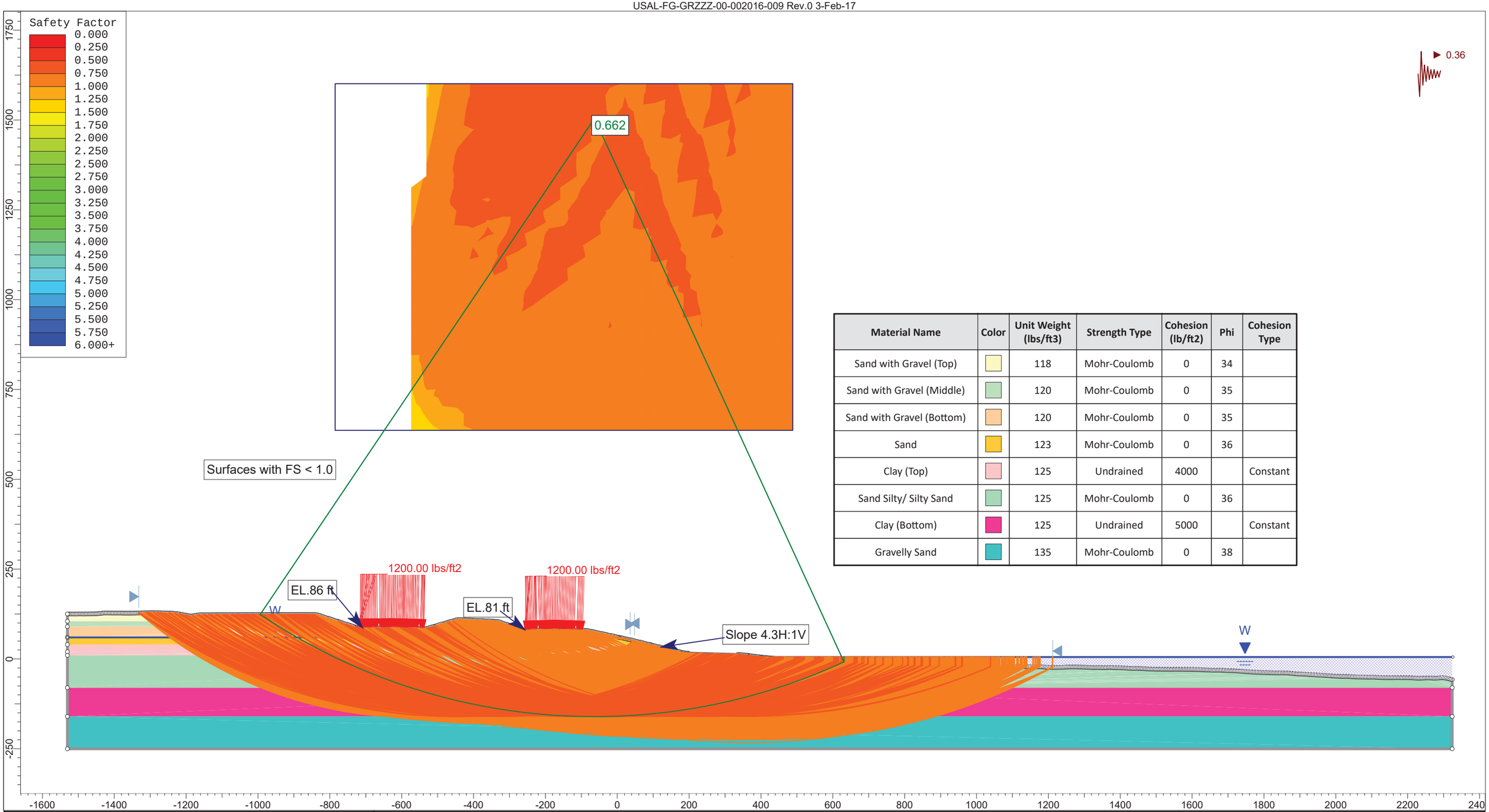
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), static (undrained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	Scale 1:2000	Company Fugro Consultants
Date	4/14/2016, 8:47:22 AM		File Name Plate D-12 (HA3_static-undrained) no bot load.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35
Sand		123	Mohr-Coulomb	0	36
Clay (Top)		125	Mohr-Coulomb	300	23
Silty Sand / Sandy Silt		125	Mohr-Coulomb	0	36
Clay (Bottom)		125	Mohr-Coulomb	300	23
Gravelly Sand		135	Mohr-Coulomb	0	38
Clay (Middle)		120	Mohr-Coulomb	300	23

SLIDEINTERPRET 6.012

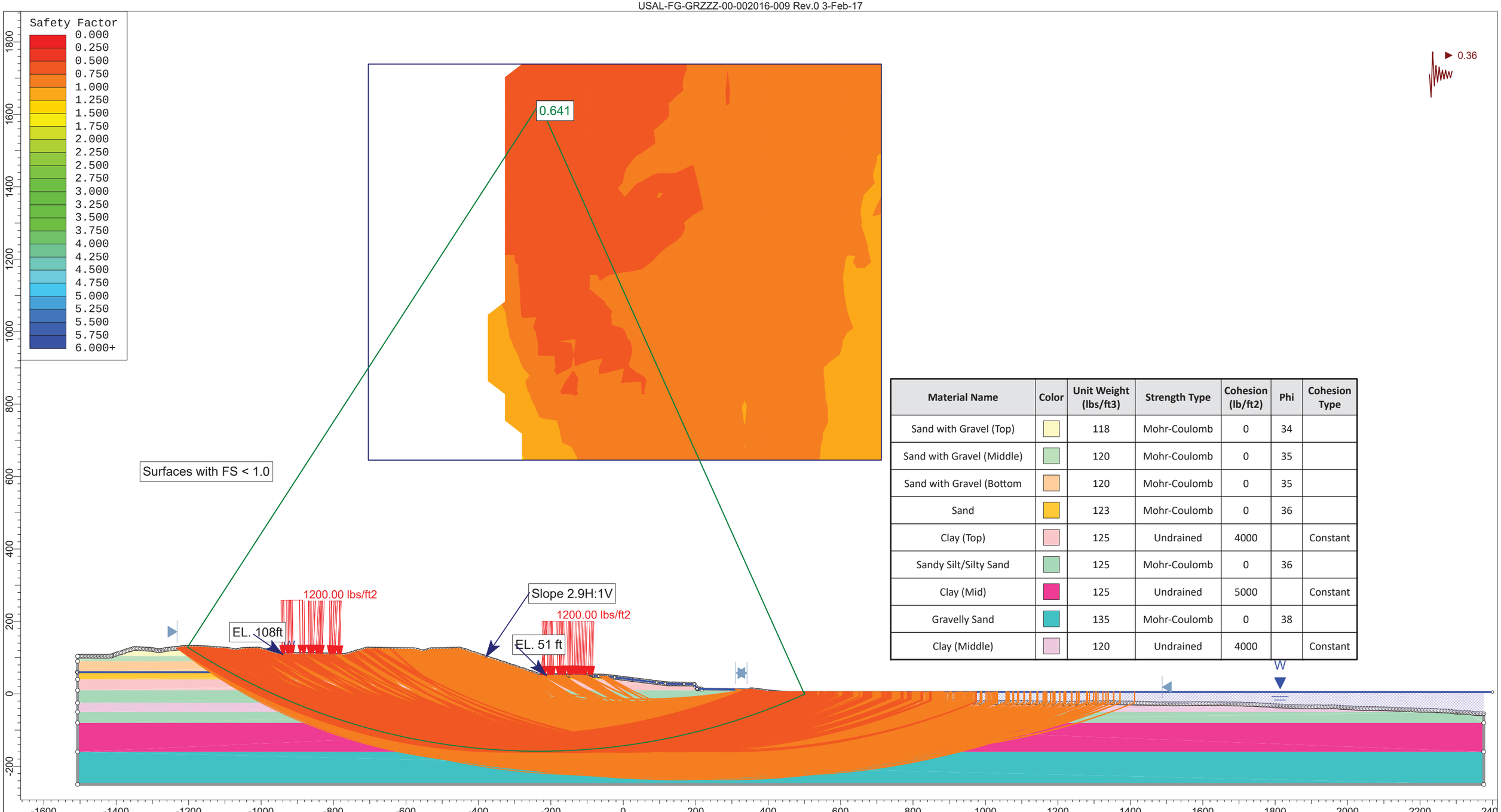
Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), static (drained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	*Scale* 1:2000	*Company* Fugro Consultants
Date	4/14/2016, 8:47:22 AM		*File Name* Plate D-13 (HA3_static-drained) no bot load.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sand Silty/ Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA1-HA1'), seismic (undrained) condition		
Drawn By	Aditya Zutshi	Scale	1:3000
Company	Fugro Consultants		
Date	4/14/2016, 8:54:57 AM	File Name	Plate D-14 (HA1_seismic).slim

SLIDEINTERPRET 6.012

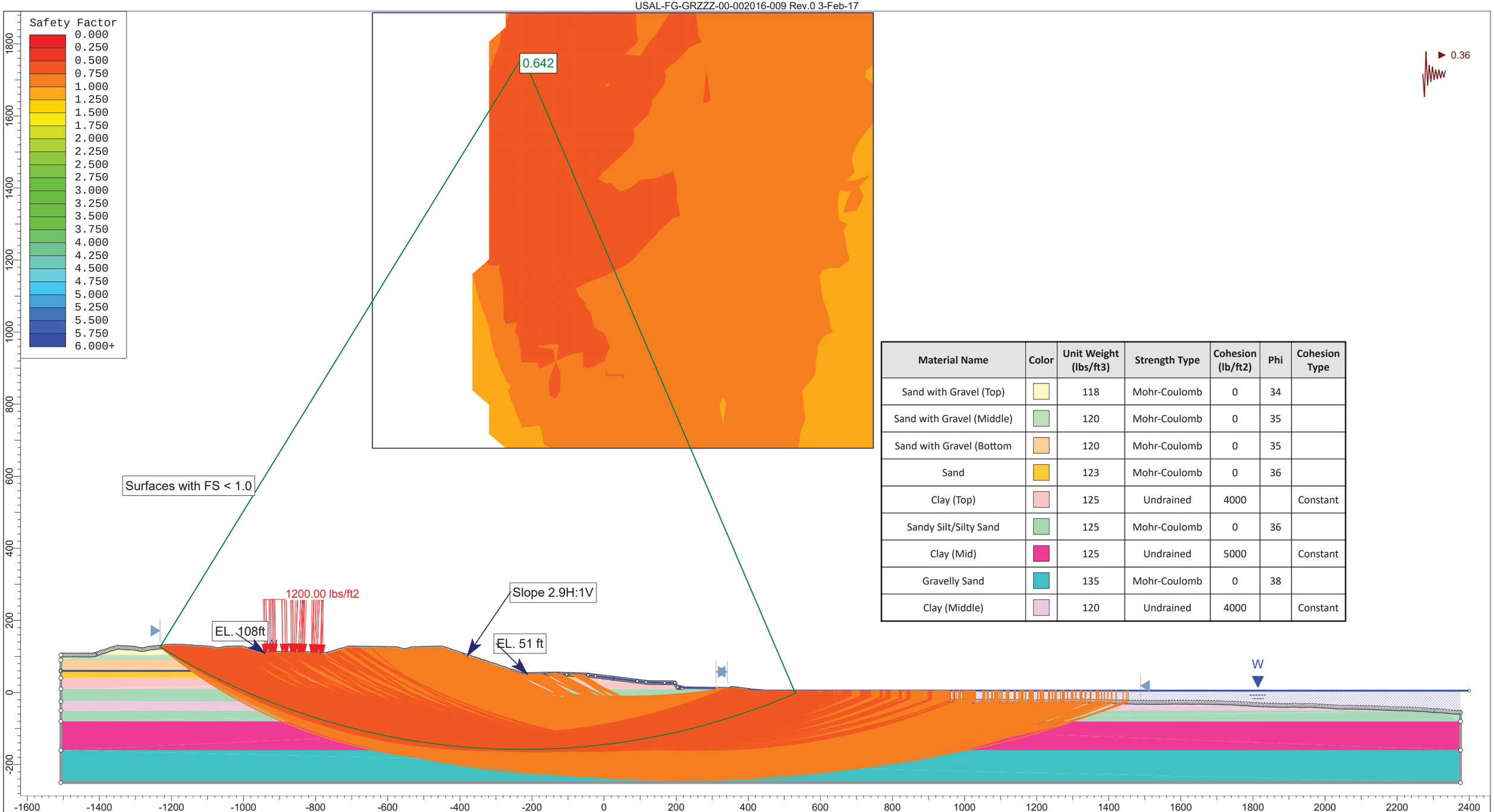


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sandy Silt/Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Mid)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), seismic (undrained) condition		
Drawn By	Aditya Zutshi	*Scale* 1:3000	*Company* Fugro Consultants Inc.
Date	4/13/2016, 2:17:17 PM		*File Name* Plate D-15 (HA2_seismic).slim

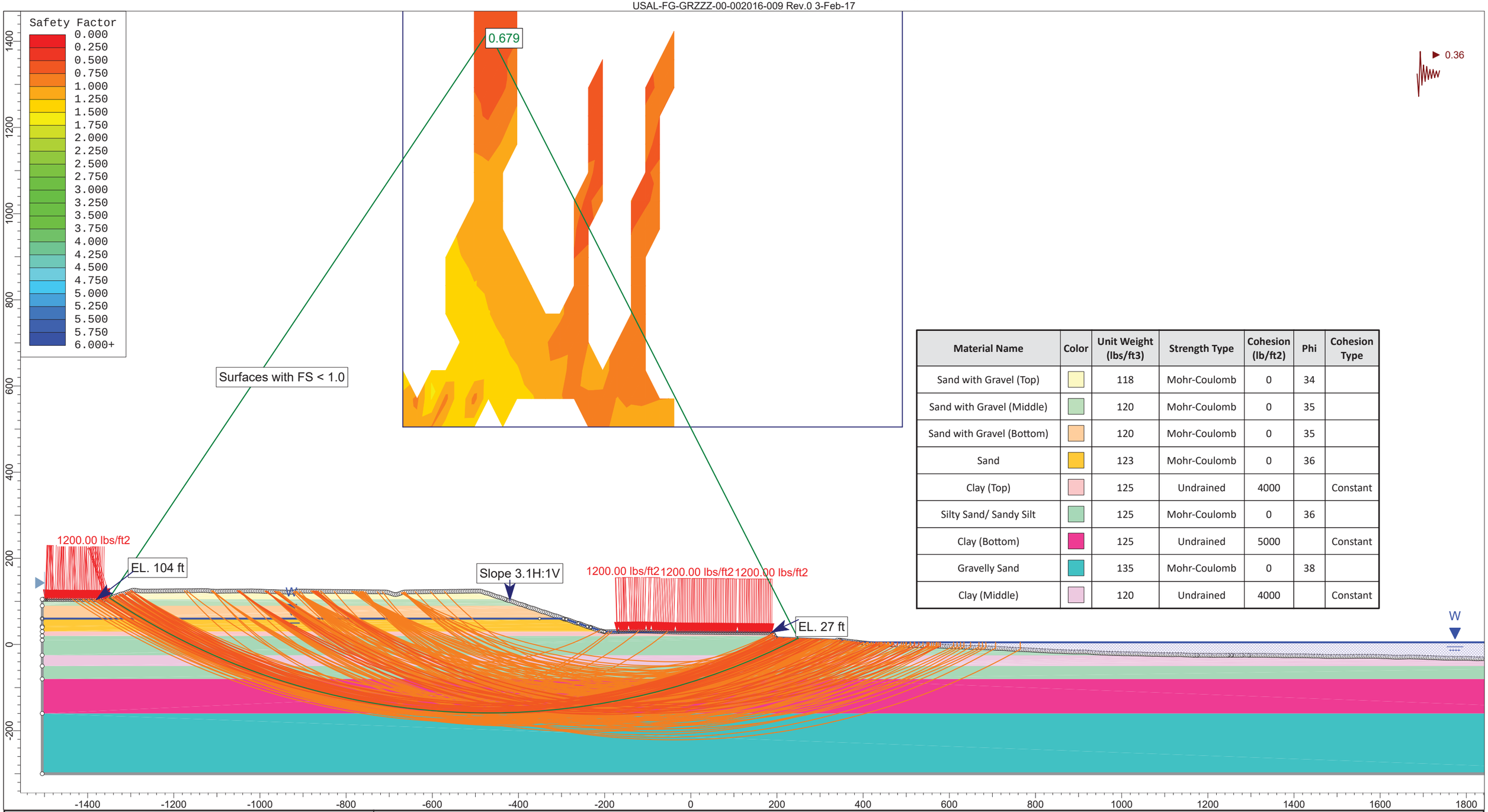


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Sandy Silt/Silty Sand		125	Mohr-Coulomb	0	36	
Clay (Mid)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA2-HA2'), seismic (undrained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	Scale	1:3000
Company	Fugro Consultants Inc.		
Date	4/13/2016, 2:17:17 PM	File Name	Plate D-16 (HA2_seismic) no bot load.slim

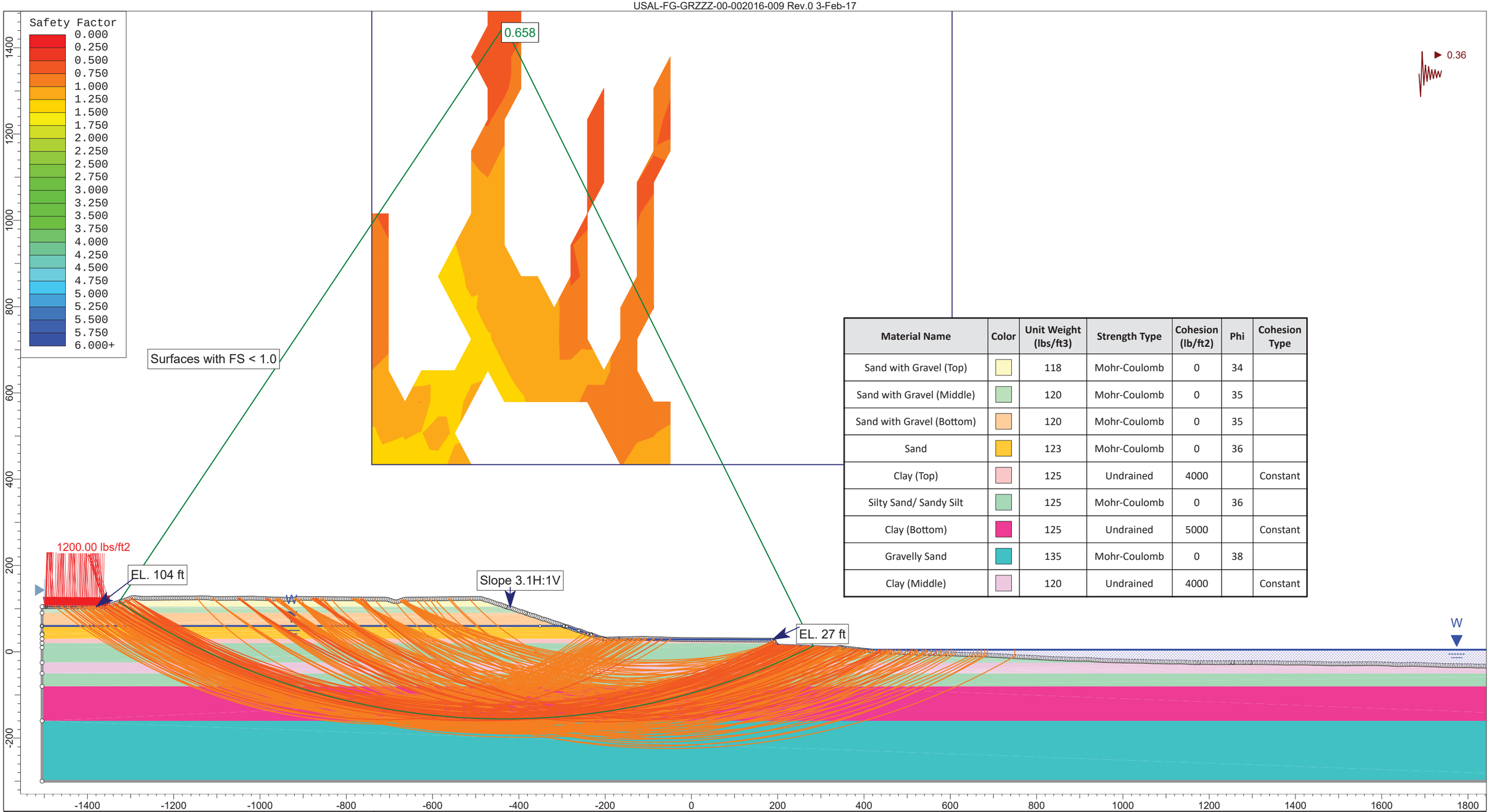


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Silty Sand/ Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), seismic (undrained) condition		
Drawn By	Aditya Zutshi	*Scale* 1:2500	*Company* Fugro Consultants
Date	4/14/2016, 8:47:22 AM		*File Name* Plate D-17 (HA3_seismic).slim



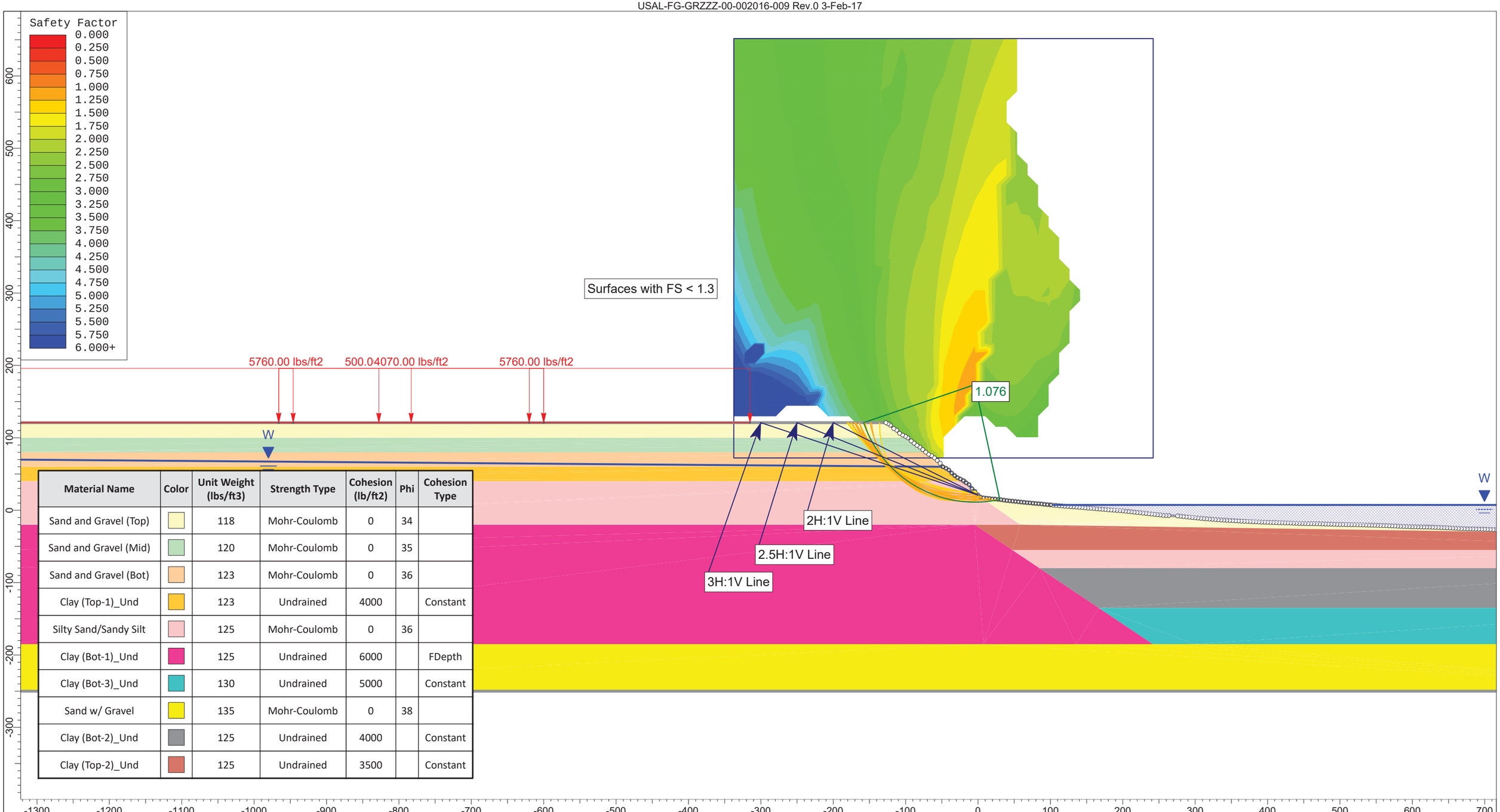
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand with Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand with Gravel (Middle)		120	Mohr-Coulomb	0	35	
Sand with Gravel (Bottom)		120	Mohr-Coulomb	0	35	
Sand		123	Mohr-Coulomb	0	36	
Clay (Top)		125	Undrained	4000		Constant
Silty Sand/ Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bottom)		125	Undrained	5000		Constant
Gravelly Sand		135	Mohr-Coulomb	0	38	
Clay (Middle)		120	Undrained	4000		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Heavy Haul Road (Area 1, HA3-HA3'), seismic (undrained) condition, no bottom traffic load		
Drawn By	Aditya Zutshi	*Scale* 1:2500	*Company* Fugro Consultants
Date	4/14/2016, 8:47:22 AM		*File Name* Plate D-18 (HA3_seismic) no bot load.slim

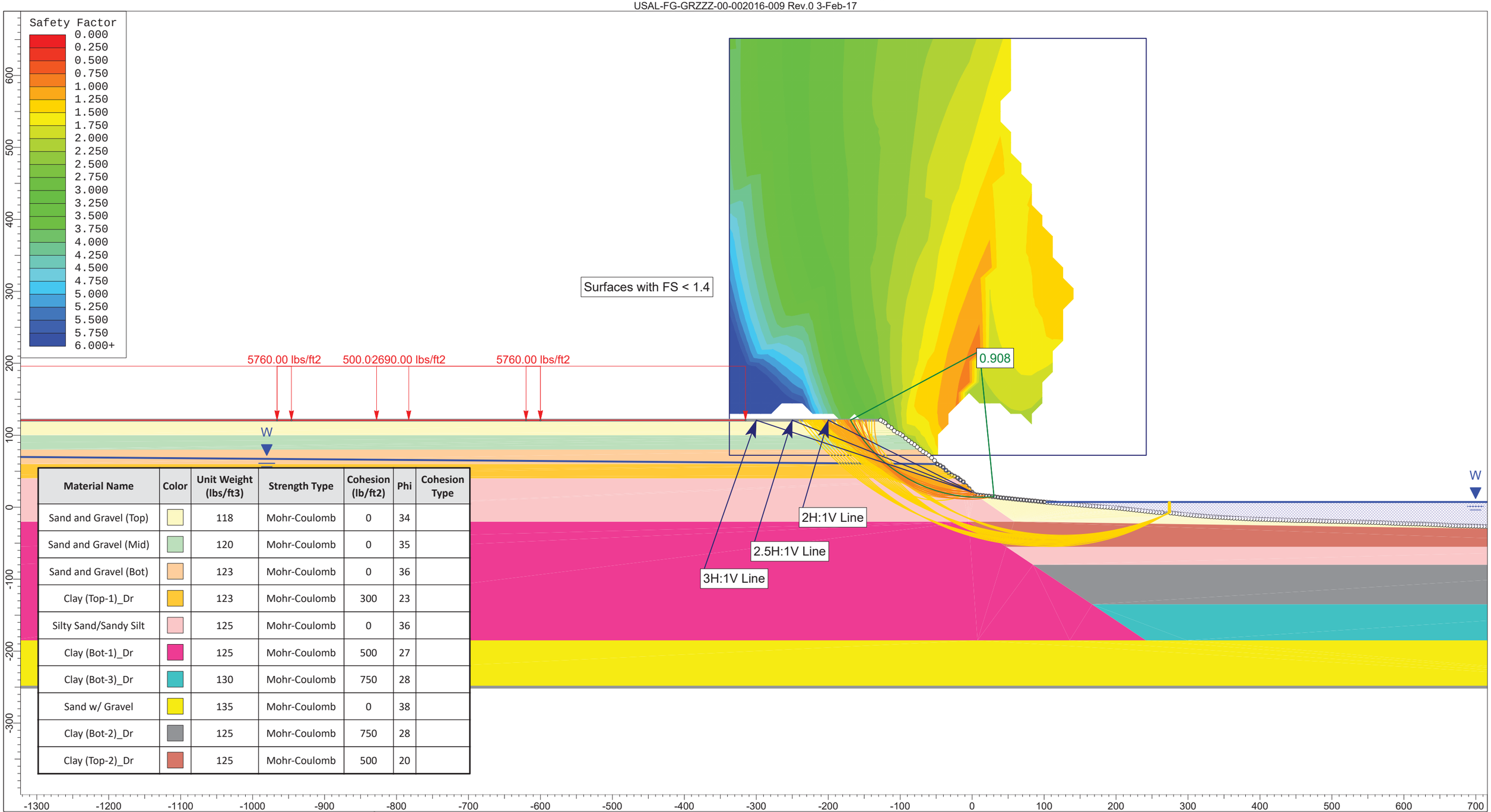
AREA 2 (LNG TANKS) – SLOPE STABILITY RESULTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		123	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bot-1)_Und		125	Undrained	6000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Existing slope (1.4H:1V), reference (3H:1V) line, 300 ft from Tank (North-Area 2) with hydrotesting load, static (undrained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-19 (DP2A-S Tank North)_hydrotesting (static).slim

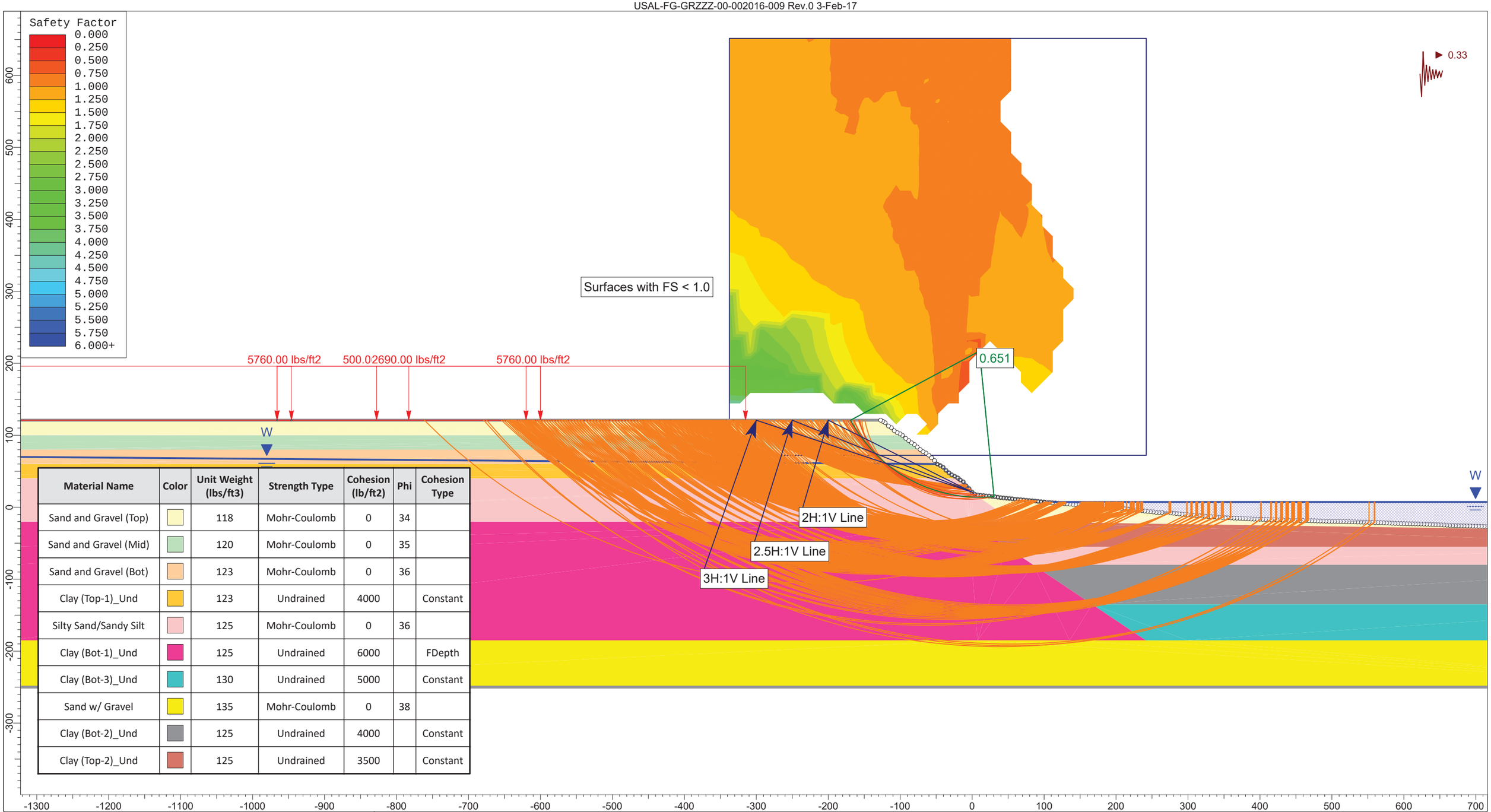


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		123	Mohr-Coulomb	0	36	
Clay (Top-1)_Dr		123	Mohr-Coulomb	300	23	
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bot-1)_Dr		125	Mohr-Coulomb	500	27	
Clay (Bot-3)_Dr		130	Mohr-Coulomb	750	28	
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Dr		125	Mohr-Coulomb	750	28	
Clay (Top-2)_Dr		125	Mohr-Coulomb	500	20	

FUGRO

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Existing slope (1.4H:1V), reference (3H:1V) line, 300 ft from Tank (North-Area 2) with operating load, static (drained) condition		
Drawn By	Sochan Jung	Scale	1:1500
Company	Fugro Consultants		
Date	12/1/2016, 5:46:56 PM	File Name	Plate D-20 (DP2A-S Tank North)_operating (static).slim

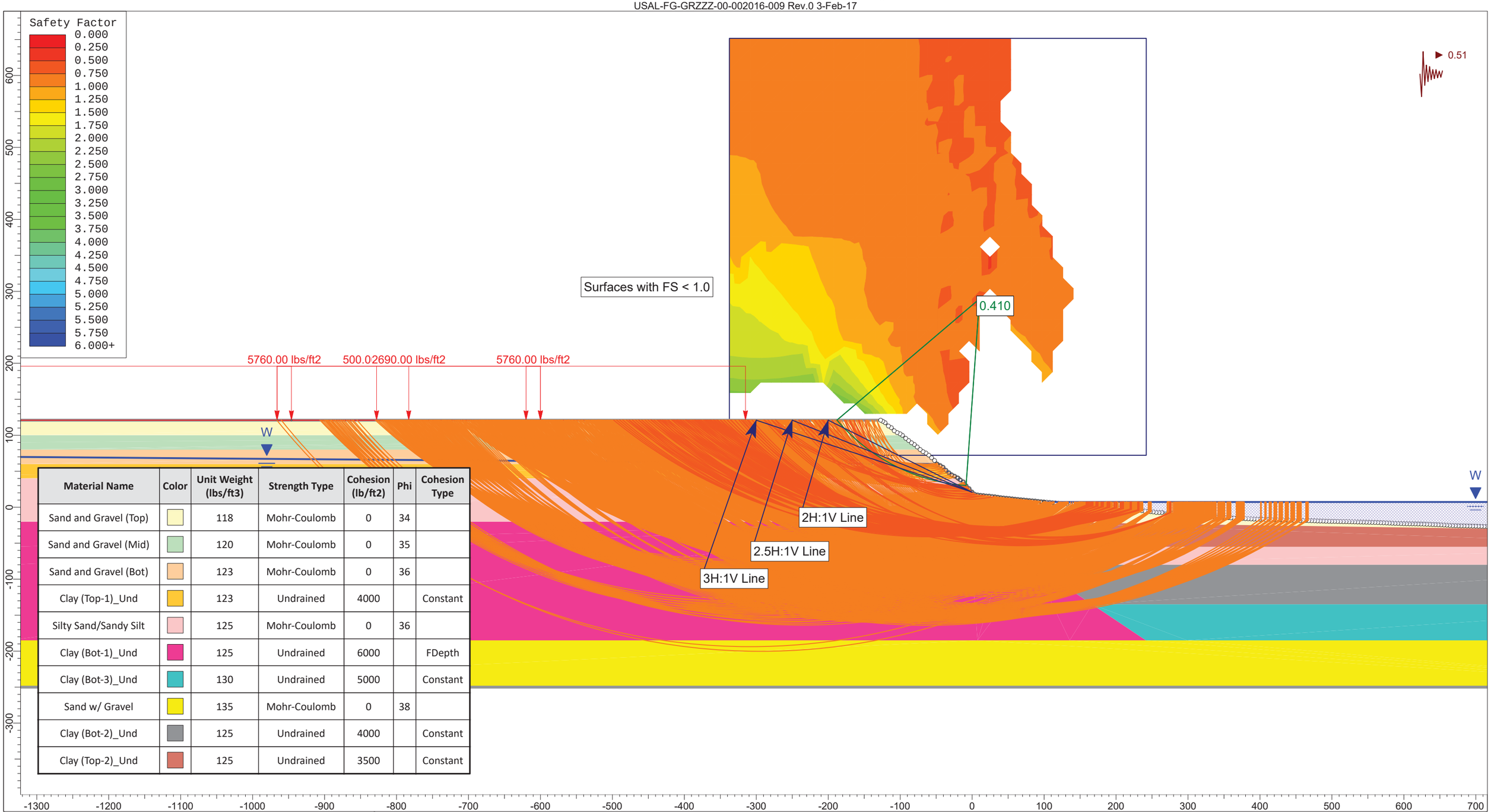


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		123	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bot-1)_Und		125	Undrained	6000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Existing slope (1.4H:1V), reference (3H:1V) line, 300 ft from Tank (North-Area 2) with operating load, seismic (OBE, undrained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-21 (DP2A-S Tank North)_operating (OBE 0.33).slim



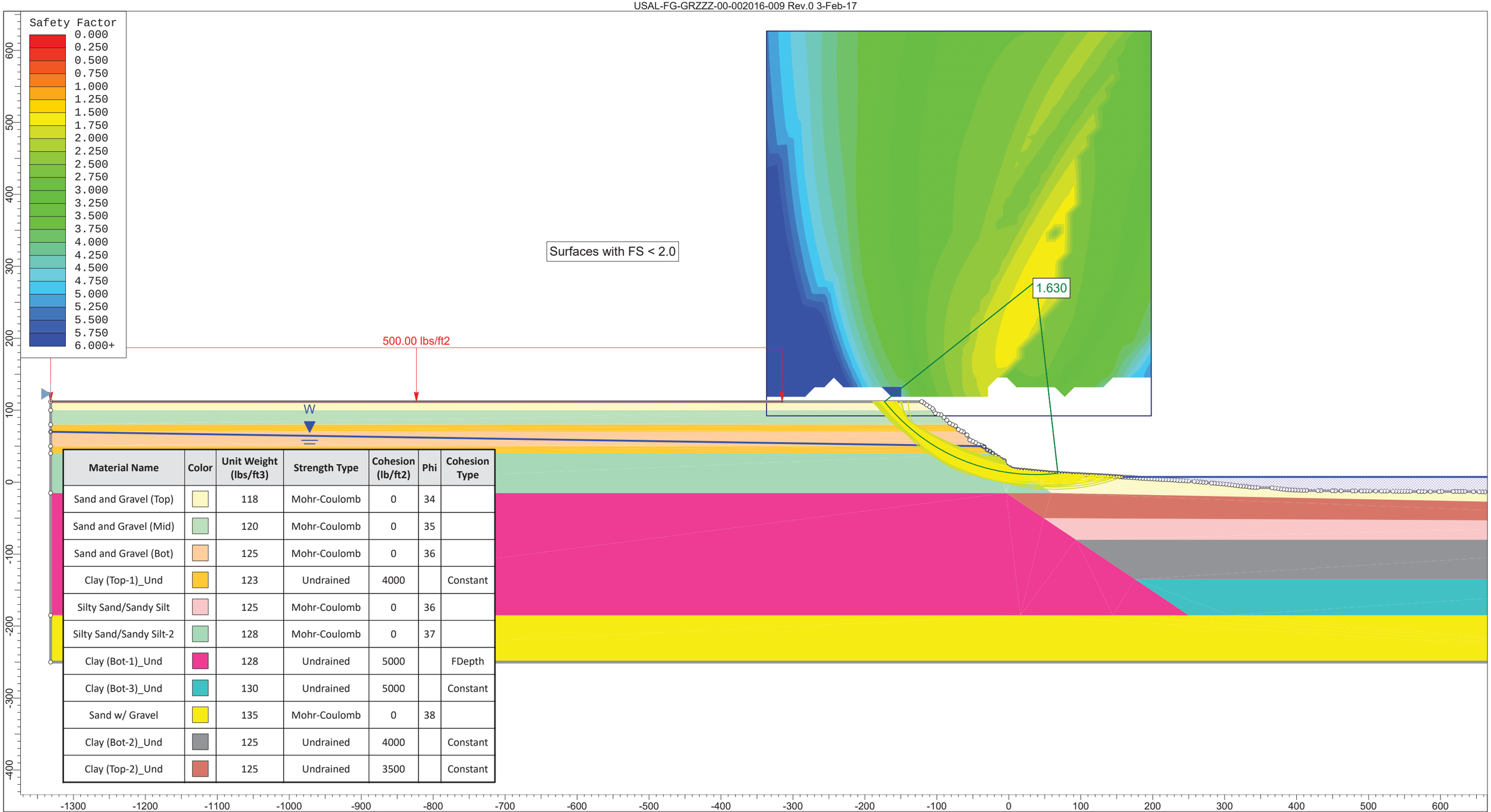
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		123	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Clay (Bot-1)_Und		125	Undrained	6000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

FUGRO

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Existing slope (1.4H:1V), reference (3H:1V) line, 300 ft from Tank (North-Area 2) with operating load, seismic (SSE, undrained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-22 (DP2A-S Tank North)_operating (SSE 0.51).slim

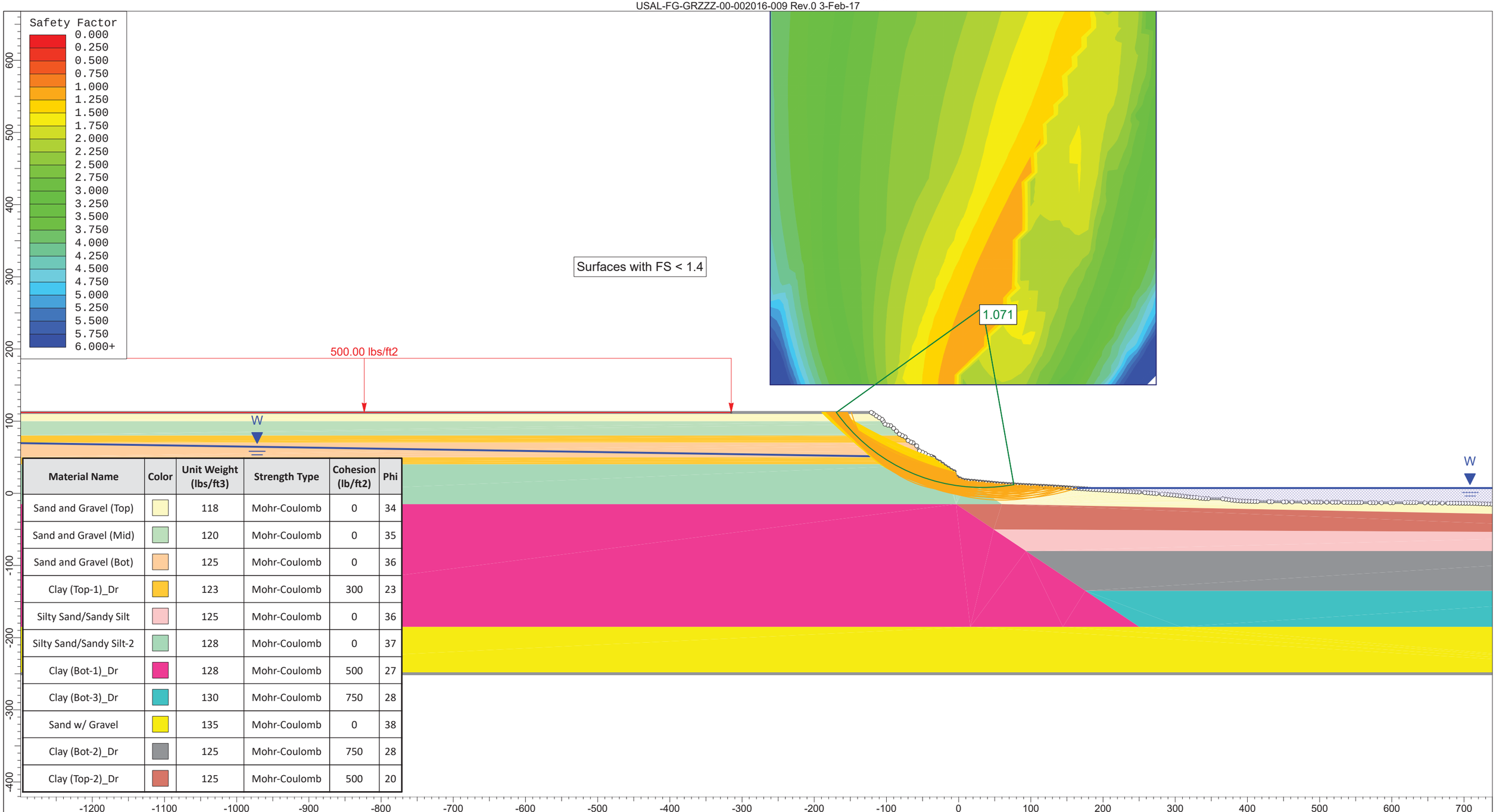
AREA 3 (OUTFALL) – SLOPE STABILITY RESULTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

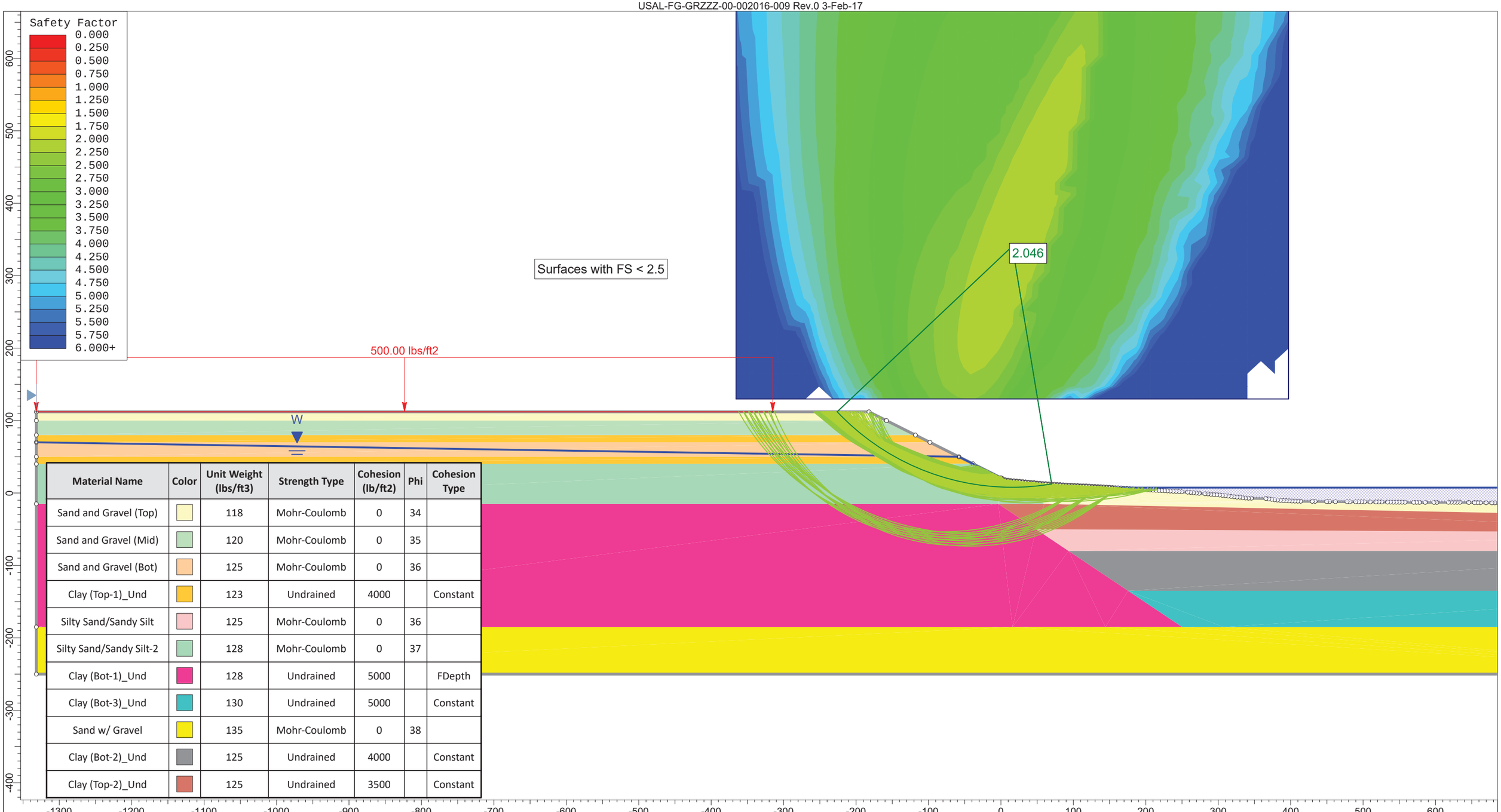
Project	04.10160001-12		
Analysis Description	Existing slope (1.3H:1V), Outfall (Area 3) with load, static (undrained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-23 (DP3-S_existing)_undrained (static).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36
Clay (Top-1)_Dr		123	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37
Clay (Bot-1)_Dr		128	Mohr-Coulomb	500	27
Clay (Bot-3)_Dr		130	Mohr-Coulomb	750	28
Sand w/ Gravel		135	Mohr-Coulomb	0	38
Clay (Bot-2)_Dr		125	Mohr-Coulomb	750	28
Clay (Top-2)_Dr		125	Mohr-Coulomb	500	20

SLIDEINTERPRET 6.012

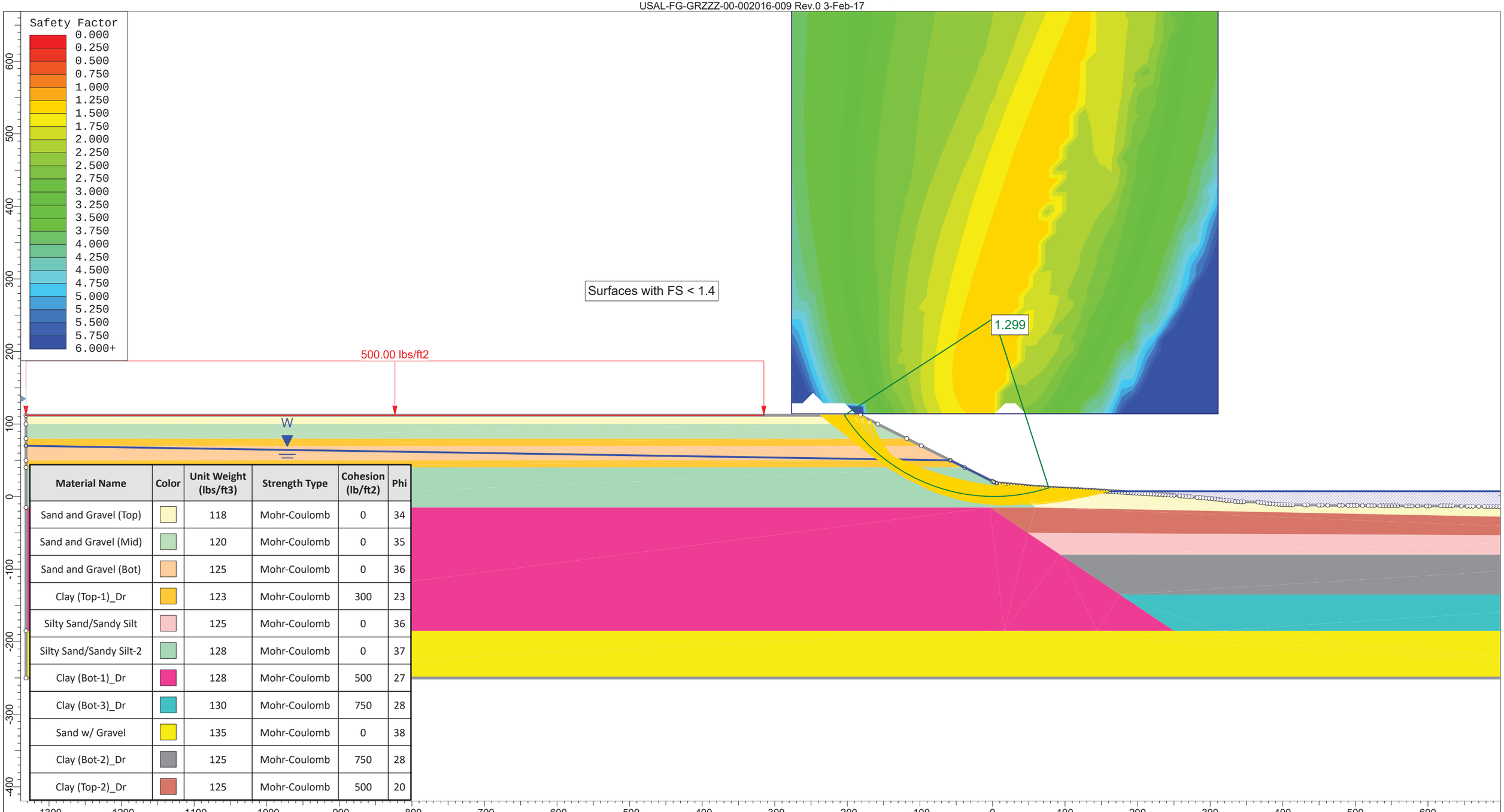
Project	04.10160001-12		
Analysis Description	Existing slope (1.3H:1V), Outfall (Area 3) with load, static (drained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-24 (DP3-S_existing)_drained (static).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

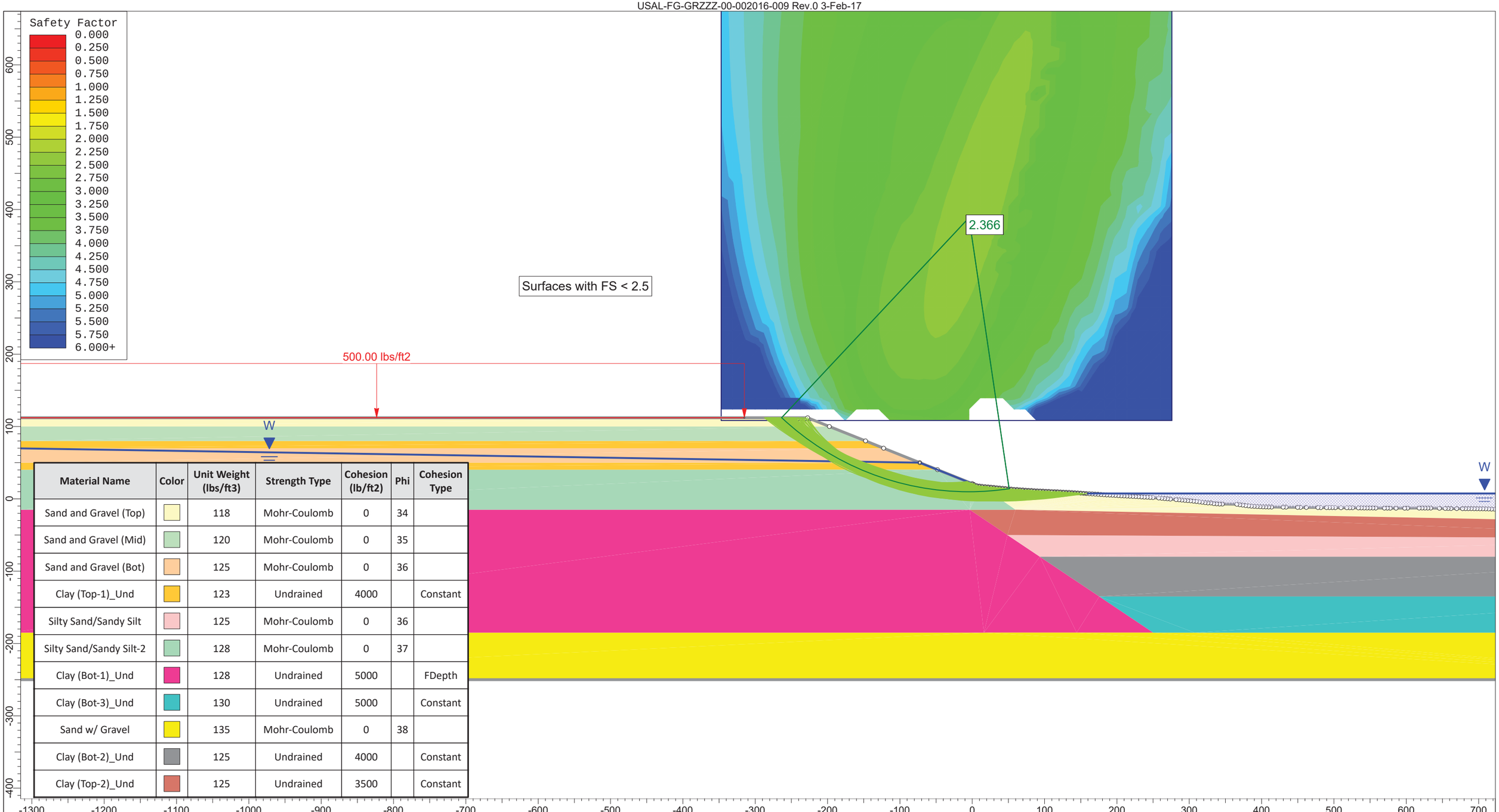
Project	04.10160001-12		
Analysis Description	Recommended slope (2H:1V), Outfall (Area 3) with load, static (undrained) condition		
Drawn By	Sochan Jung	Scale 1:1500	Company Fugro Consultants
Date	12/1/2016, 5:46:56 PM		File Name Plate D-25 (DP3-S_2H1V)_undrained (static).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36
Clay (Top-1)_Dr		123	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37
Clay (Bot-1)_Dr		128	Mohr-Coulomb	500	27
Clay (Bot-3)_Dr		130	Mohr-Coulomb	750	28
Sand w/ Gravel		135	Mohr-Coulomb	0	38
Clay (Bot-2)_Dr		125	Mohr-Coulomb	750	28
Clay (Top-2)_Dr		125	Mohr-Coulomb	500	20

SLIDEINTERPRET 6.012

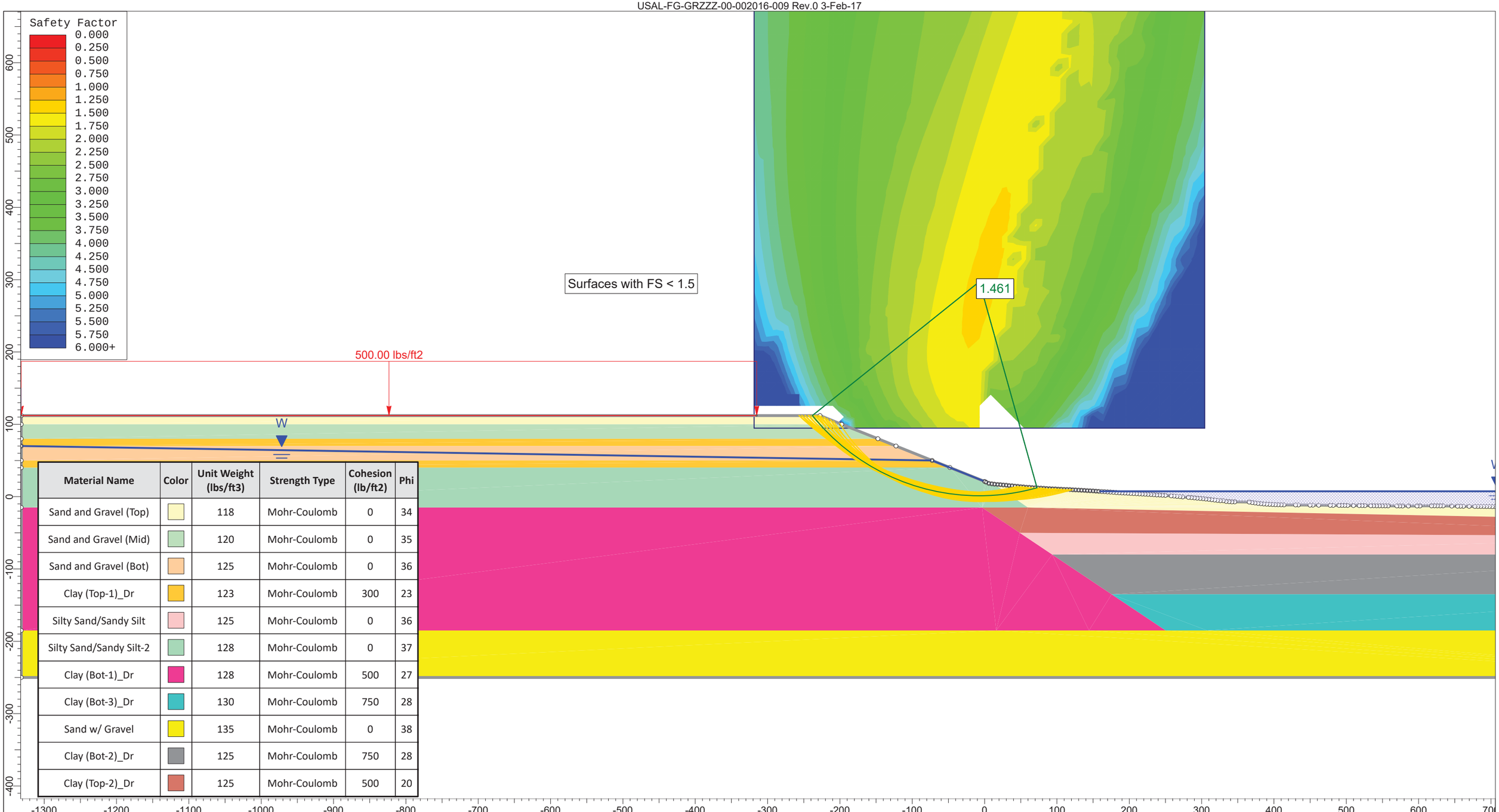
Project	04.10160001-12		
Analysis Description	Recommended slope (2H:1V), Outfall (Area 3) with load, static (drained) condition		
Drawn By	Sochan Jung	Scale	1:1500
Company	Fugro Consultants		
Date	12/1/2016, 5:46:56 PM	File Name	Plate D-26 (DP3-S_2H1V)_drained (static).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

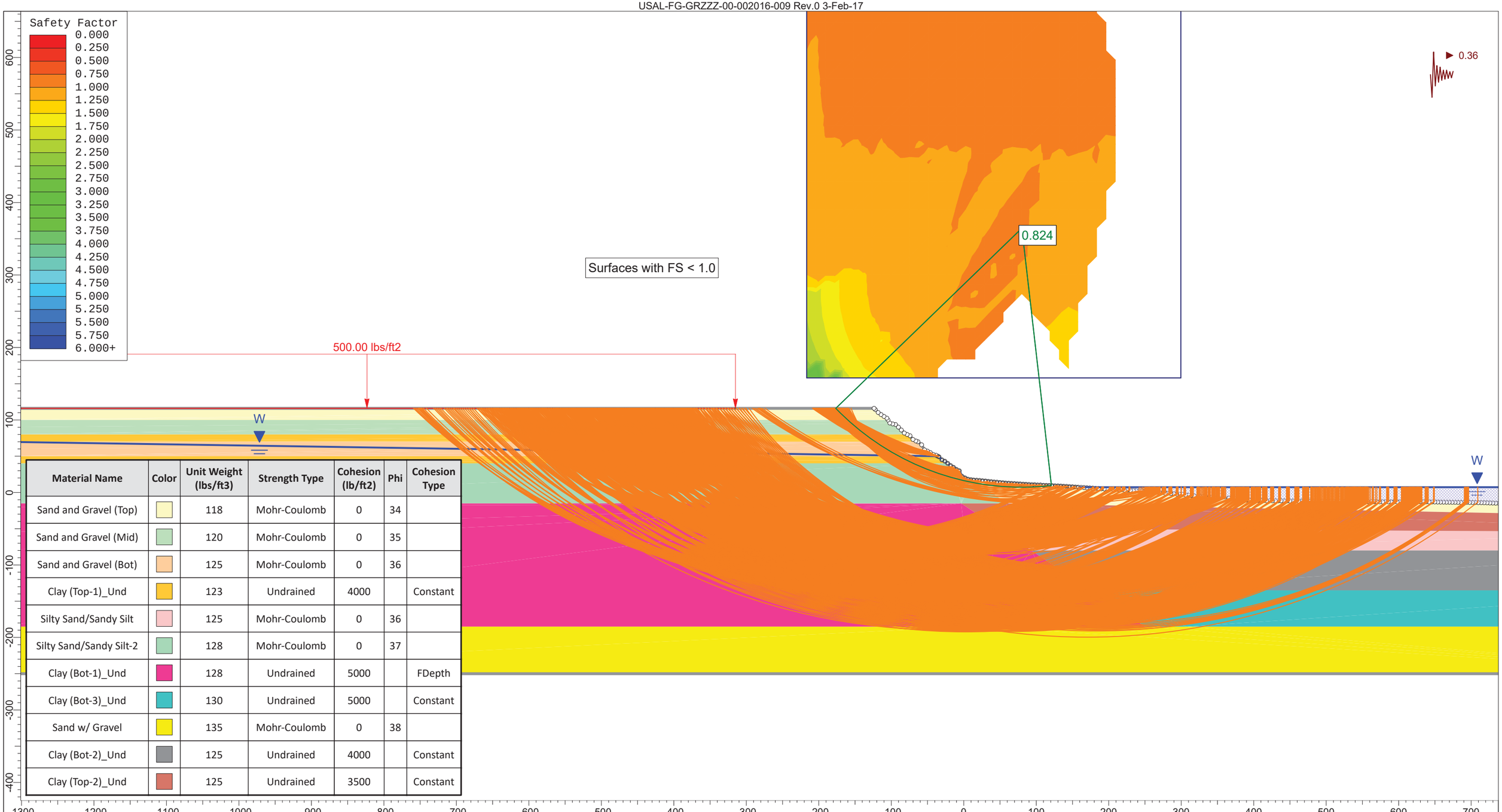
Project	04.10160001-12		
Analysis Description	Recommended slope (2.5H:1V), Outfall (Area 3) with load, static (undrained) condition		
Drawn By	Sochan Jung	*Scale* 1:1500	*Company* Fugro Consultants
Date	12/1/2016, 5:46:56 PM		*File Name* Plate D-27 (DP3-S_2.5H1V)_undrained (static).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36
Clay (Top-1)_Dr		123	Mohr-Coulomb	300	23
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37
Clay (Bot-1)_Dr		128	Mohr-Coulomb	500	27
Clay (Bot-3)_Dr		130	Mohr-Coulomb	750	28
Sand w/ Gravel		135	Mohr-Coulomb	0	38
Clay (Bot-2)_Dr		125	Mohr-Coulomb	750	28
Clay (Top-2)_Dr		125	Mohr-Coulomb	500	20

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Recommended slope (2.5H:1V), Outfall (Area 3) with load, static (drained) condition		
Drawn By	Sochan Jung	Scale 1:1500	Company Fugro Consultants
Date	12/1/2016, 5:46:56 PM		File Name Plate D-28 (DP3-S_2.5H1V)_drained (static).slim

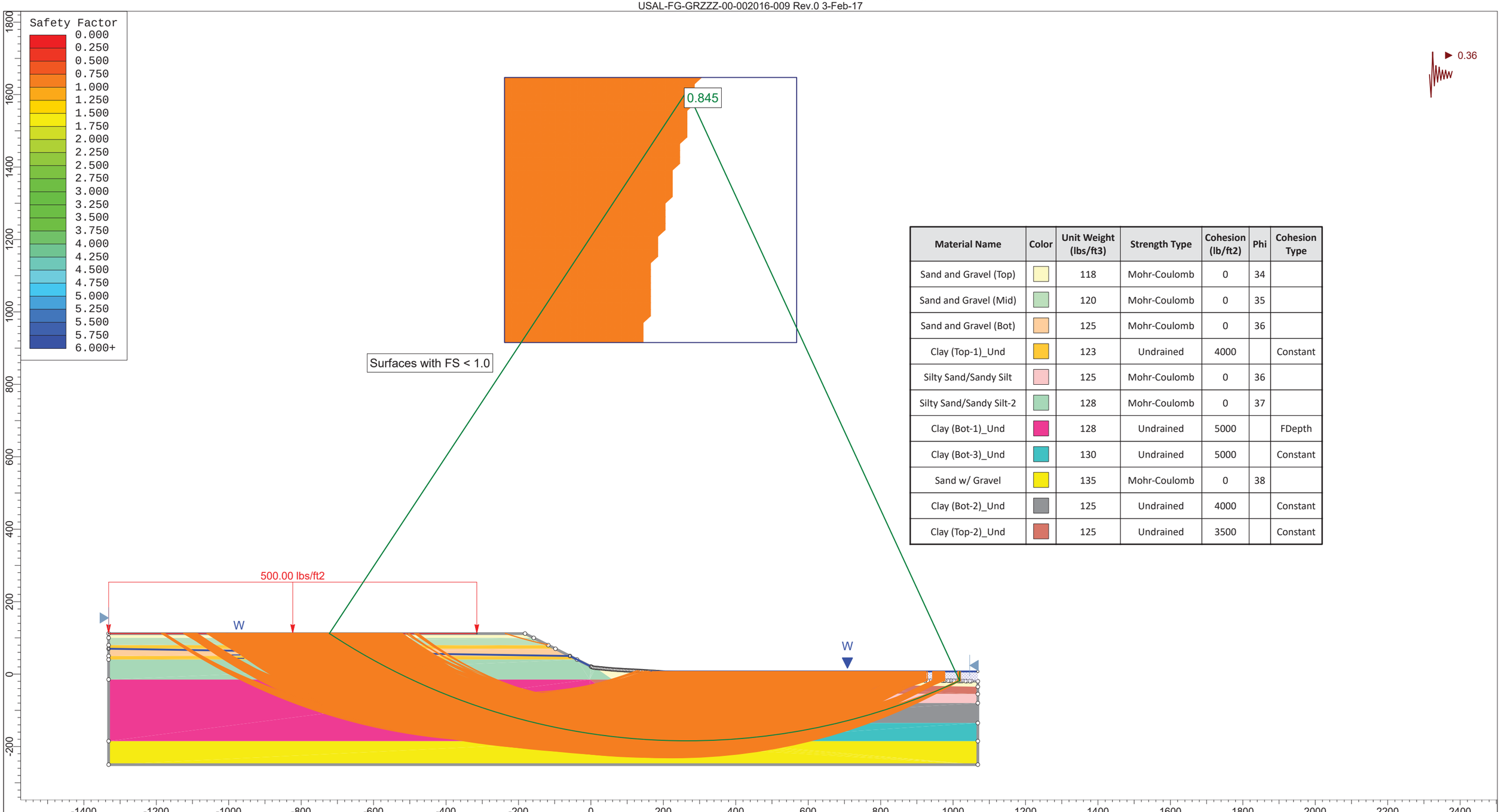


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

FUGRO

SLIDEINTERPRET 6.012

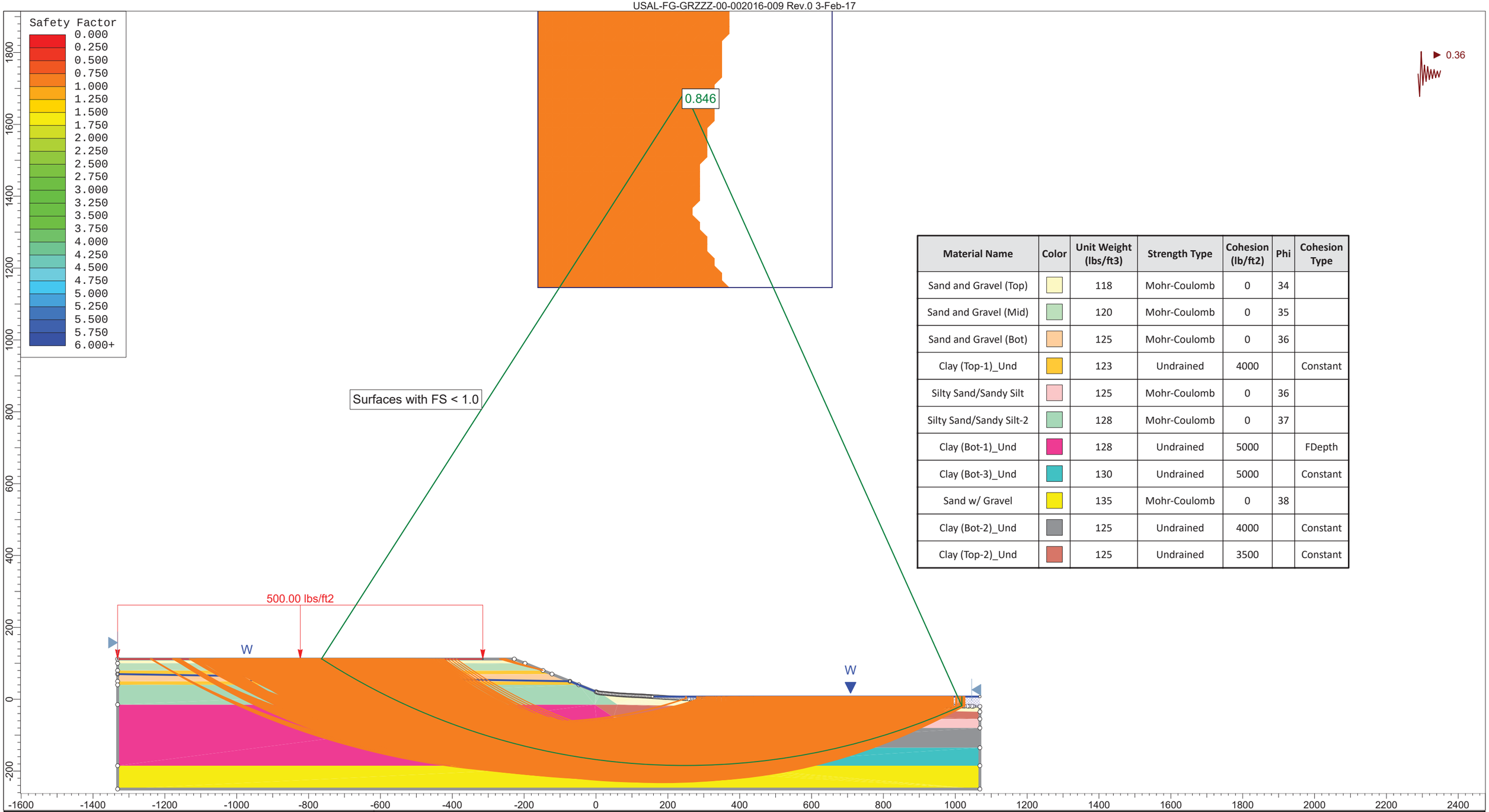
Project	04.10160001-12		
Analysis Description	Existing slope (1.3H:1V), Outfall (Area 3) with load, seismic (undrained) condition		
Drawn By	Sochan Jung	Scale 1:1500	Company Fugro Consultants
Date	12/1/2016, 5:46:56 PM		File Name Plate D-29 (DP3-S_existing)_undrained (seismic).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Recommended slope (2H:1V), Outfall (Area 3) with load, seismic (undrained) condition		
Drawn By	Sochan Jung	Scale	1:3000
Company	Fugro Consultants		
Date	12/1/2016, 5:46:56 PM	File Name	Plate D-30 (DP3-S_2H1V)_undrained (seismic).slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type
Sand and Gravel (Top)		118	Mohr-Coulomb	0	34	
Sand and Gravel (Mid)		120	Mohr-Coulomb	0	35	
Sand and Gravel (Bot)		125	Mohr-Coulomb	0	36	
Clay (Top-1)_Und		123	Undrained	4000		Constant
Silty Sand/Sandy Silt		125	Mohr-Coulomb	0	36	
Silty Sand/Sandy Silt-2		128	Mohr-Coulomb	0	37	
Clay (Bot-1)_Und		128	Undrained	5000		FDepth
Clay (Bot-3)_Und		130	Undrained	5000		Constant
Sand w/ Gravel		135	Mohr-Coulomb	0	38	
Clay (Bot-2)_Und		125	Undrained	4000		Constant
Clay (Top-2)_Und		125	Undrained	3500		Constant

SLIDEINTERPRET 6.012

Project	04.10160001-12		
Analysis Description	Recommended slope (2.5H:1V), Outfall (Area 3) with load, seismic (undrained) condition		
Drawn By	Sochan Jung	Scale	1:3000
Company	Fugro Consultants		
Date	12/1/2016, 5:46:56 PM	File Name	Plate D-31 (DP3-S_2.5H1V)_undrained (seismic).slim