Neptun Deep Project

Geotechnical Interpretative Report for Domino Drill Centers

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Neptun Deep Project

Geotechnical Interpretative Report for Domino Drill Centers

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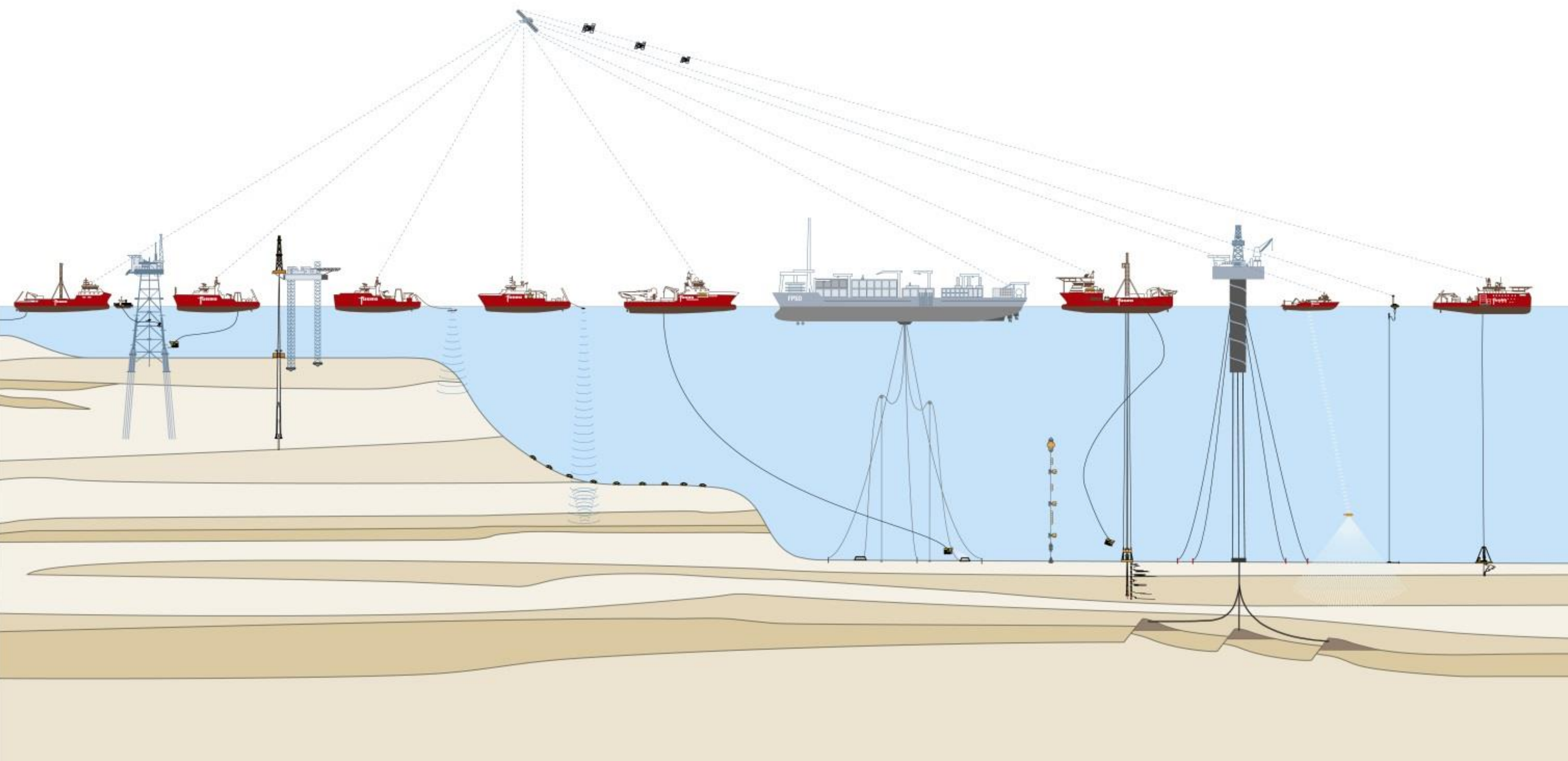
FUGRO

Domino Drill Center Geotechnical Interpretive Report
Neptun Deep Survey
Pelican South Field

Fugro Document No.: 173570-05c(02)
Issue Date: 14 June 2018

ExxonMobil Exploration and Production Romania Limited

Final Report



FUGRO

Domino Drill Center Geotechnical Interpretive Report
Neptun Deep Survey
Pelican South Field

Black Sea, Romania

Fugro Document No: 173570-05c(02)
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Prepared for:

ExxonMobil Exploration and Production Romania Limited

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Dear Yvonne Moret and Patrick Lee,

Neptun Deep Survey
Pelican South Field, Black Sea, Romania

We have the pleasure of submitting the Domino Drill Center Geotechnical Interpretive Report for the Neptun Deep Survey. This report presents the interpreted geotechnical soil parameters, the mudmat analyses and suction pile analyses for the Domino Drill Centers 1 and 2.

This report was prepared by Martin Gichura and Charles Bloore under the supervision of Mike Rattley.

We hope that you find this report to your satisfaction; should you have any queries, please do not hesitate to contact us.

Yours sincerely,
Fugro GB Marine Limited

Martin Gichura
Geotechnical Engineer

Distribution: One electronic copy to Yvonne Moret and Patrick Lee

QUALITY ASSURANCE RECORD

Section	Prepared By	Checked By	Approved By
Main text	MG	DB	MR
Plates following the main text	MG	DB	MR
Appendix A – Guidelines On Use Of Report	Fugro	Fugro	Fugro

Notes:
The personnel stated above were responsible for preparing, checking and approving this report
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REPORT ISSUE LOG

Issue No.	Status	Reason for Issue/Reissue	Comments on Content	Date
01	Draft	First issue to client	Awaiting client comments	23 May 2018
02	Final	Report updated for client comments	Various report sections modified, including minor technical clarifications and typos	14 June 2018

FRONTISPIECE

EXECUTIVE SUMMARY

Introduction

ExxonMobil Exploration and Production Romania Limited (ExxonMobil) is developing the Pelican South field and Domino field in the Romanian sector of the Black Sea.

ExxonMobil requested Fugro GB Marine Limited (Fugro) to perform a geotechnical site investigation to provide soils information at the Pelican South and Domino Field. The fieldwork was performed from the MV Fugro Synergy from 28 December 2017 to 8 February 2018.

At the Domino Deep water locations, ExxonMobil requested Fugro to derive design soil parameters for input into engineering analyses at two locations:

i. Domino Drill Center 1 (DODC-1);
ii. Domino Drill Center 2 (DODC-2).

For the two planned drill center locations this report presents:

i. Design soil parameters for preliminary mudmat and suction pile stability and installation analyses derived from in situ and laboratory testing data report (Fugro, 2018a);
ii. Preliminary mudmat stability and installation analyses;
iii. Preliminary suction pile vertical bearing capacity and installation analyses.

Geotechnical Data

The following data sources were used to derive the geotechnical data presented in this report.

i. Neptun Deep integrated report (Fugro, 2016a);
ii. Laboratory and in situ testing data report (Fugro 2015c);
iii. Laboratory and in situ testing data report (Fugro 2018a).

Geological Setting

The planned Domino drill centers are located in the deepwater area of the Neptun block. The sediments within the foundation zone comprise lacustrine clays deposited in a freshwater environment.

Global sea level rise and the reconnection of the Bosphorus Strait led to the flooding of the Black Sea and the deposition of organic rich clay (sapropel) and coccolith ooze.

During periods of sea-level lowstand, the canyons acted as the main source of sediment transport, with sand and silt layers deposited during periods of high canyon activity. This is observable in the boreholes drilled at the two planned drill centers. DODC-2 is located closer to the canyon and as a result has more sand and silt layers within Geotechnical Soil Unit 5 than at the DODC-1 Location. Geotechnical Soil Unit 5 sediments at DODC-2 are also more overconsolidated than the sediments at DODC-1; this is interpreted to be due to erosion by downcanyon flows. The canyons are not interpreted to be active at present.

Design Soil Parameters

Mudmat and suction pile foundations are planned to be installed to support In-line tee assembly (ITA), flowline end termination (FLET) and manifold structures. Derivation of the geotechnical design soil parameters for preliminary mudmat and suction pile analyses is discussed in this report and presented on the plates following the main text. The following design soil parameters were derived:

- Water content (w);
- Total unit weight (γ);
- Plasticity Index (I_p);
- Cone penetration test (CPT) cone resistance (q_c);
- Undrained shear strength (s_u);
- Remoulded strength (s_{ur});
- Strength sensitivity (S_t);
- Overconsolidation ratio (OCR).

Low estimate (LE), best estimate (BE) and high estimate (HE) design soil profiles were derived to the investigation depth of 30 m. The design profiles presented in this report are specific to the analyses presented in this report and should be carefully reviewed for any other purpose.

Chemical Testing Results

Chemical composition from the chemical tests presented by Fugro (2018a) are briefly discussed in this report. The observed changes in chemistry are interpreted to have been caused by the transition from freshwater to marine environments and agree with the geological model for the Neptun block.

Engineering Analysis

General

Mudmat and suction pile vertical bearing capacity and installation analyses are presented in this report. Mudmat and suction pile analysis was performed according to an API (2011) working stress design approach applying a global factor of safety of 2.0 to unfactored loads and unfactored resistances.

The following foundation design risks were identified at the Domino drill center locations:

i. Very soft sediments at seafloor comprising Coccolith ooze and sapropel formations;
ii. Gas hydrates;
iii. Buried mass transport deposits (MTDs).

The identified engineering risks should be mitigated by performing a detailed review of the impact these risks pose on the designed foundations. The following mitigations measures are proposed for the above geotechnical risks.

Coccolith ooze and sapropel formation: The coccolith ooze and sapropel formations are observed in the top 2.5 m below mudline (BML) at both drill center locations. Due to the extremely weak strength of the formation and high liquidity index of the soils. Fugro recommends that a detailed engineering assessment of both formations be performed including:

- Detailed foundation set down analyses to assess the risk of soil wash out, fluidisation and excessive settlements occurring due to disturbance of these highly sensitive soils;
- Detailed assessment of the geotechnical interaction between these formations and underlying formations and the potential effects on foundation stability outside of conventional design practice.

Landing impact and settlement analyses were not performed as part of this report. However, Fugro recommends that the landing impact of the mudmat and settlement analysis should be further reviewed in detailed design.

Alternatively, dredging or otherwise would mitigate the design risks associated with these soils given the extremely low strength of the highly sensitive coccolith ooze and sapropel formations.

Shallow gas: Shallow gas is not present as free gas at the Domino drill center locations and the biogenic methane is interpreted to be stable at the current temperatures and pressures. However, should the temperatures and pressures change during installation or over the operational lifetime of the well, the biogenic methane may dissociate and become free gas. Dissociation of the biogenic methane can significantly reduce sediment strength and stiffness. The reduction in sediment strength will lower the suction pile bearing capacity and mudmat stability. A reduction in soil stiffness will increase the pile displacement under load and settlement under structure self-weight, and increase the settlement of mudmats. Therefore, to account for the possible significant reduction in sediment strength due to dissociation of biogenic methane, a 30 % cautionary reduction in the undrained shear strength was applied for foundation analyses. This differs to the reductions associated with free gas at the platform location (~10 % assumed). Fugro recommends further review of the risk of gas dissociation in the detailed design stage.

Buried mass transport deposits: Buried mass transport deposits (MTD) were observed at the DODC2 location. The MTD layer was observed to be of higher localised strength in comparison to the surrounding geotechnical soil units. The MTD layer was observed between 3.8 m and 5.8 m BML. There is the potential for unexpected over-penetration or under-penetration of foundations where these stronger blocks of sediment are present.

Mudmat Analysis

Mudmat stability and installation analyses were performed for the ITA structure at the DODC-1 location and for the FLET at the DODC-2 location. Loads used in the analysis were as specified by ExxonMobil (2017). The preliminary mudmat analyses presented in the report provide a cautious upper bound of foundation geometries. These preliminary analyses adopt the following:

i. For mudmat stability analyses, low estimate s_u considering historic data, reductions in s_u due to the presence of hydrates and application of a global factor of safety of 2.0;
ii. For mudmat installation analyses, high estimate s_u considering historic data;
iii. Soil layering effects.

Mudmat vertical, horizontal, moment and torsion stability analyses were performed according to Feng at al. (2014) recommendations. Results of the mudmat analyses shows that:

i. At the DODC-1 location, a 20.0 m by 12.0 m ITA mudmat with a skirt height of 1.2 m is required to support the applied loads;

ii. At the DODC-2 location, a 29.3 m by 16.0 m FLET mudmat with a skirt height of 1.0 m is required to support the applied loads.

The mudmat sizings above are considered impractically large and are governed by the extremely low strength of the highly sensitive coccolith ooze and sapropel formations. In this case dredging or otherwise removal of these formations should be considered.

If the coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2) were removed, a 12.5 m by 7.5 m ITA mudmat with a skirt height of 0.1 m at the DODC-1 location and a 19.3 m by 10.5 m FLET mudmat with a skirt height of 0.2 m at the DODC-2 location would be required to support the applied loads. Based on the cautious design assumptions applied in this report.

Should the removal of the formations not be considered feasible, Fugro recommends that the mudmat analyses presented in this report are refined for detailed design considering the potentially highly sensitive Coccolith ooze and Sapropel formations (Geotechnical Soil Units 1 and 2).

Mudmat skirt penetration analysis was performed according to the following methods:
i. API RP 2GEO (API, 2011);
ii. DNV GL-RP-C212 (DNV GL, 2017b).

The results of the mudmat skirt penetration analysis show that full skirt penetration can be achieved using mudmat self-weight. An exceptional case is, for the 22 m by 12.0 m FLET mudmat with a skirt of 1.7 m, the mudmat is not installable according to the upper bound DNV (2017b) recommendations..

If the coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2) were removed, the 0.1 m and 0.2 m skirt heights analysed in this report would be installable under self-weight of the mudmats.

Detailed analyses considering, but not limited to, the following example effects will allow refinement of design conclusions on mudmat feasibility and geometry:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Soil layering interaction effects between weaker soil layers (coccolith ooze and sapropel) and more competent layers (Geotechnical Unit 3) on stability analyses, if these units are not removed;
iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, consolidation parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u.

It is recommended that these effects are quantified and considered in detail during detailed design in accordance with any specific ExxonMobil design basis requirements.

Suction Pile Analysis

Vertical suction pile bearing capacity analyses were performed according to DNV GL (2017c) for the manifold structures at both DODC-1 and DODC-2 locations. Loads used in the analysis were provided by ExxonMobil.

It should be noted that the suction pile bearing capacity and installation analyses presented consider that the Coccolith ooze and Sapropel formations are present i.e. no dredging or otherwise removal of the formations was considered.

Suction pile installation analyses were performed according to Houlsby and Byrne (2005) method. Table S.1 summarises the results of the suction pile vertical bearing capacity and installation analyses.

The preliminary suction pile analyses presented in the report provide a cautious upper bound of foundation geometries. These preliminary analyses adopt cautious estimates of:

i. For suction pile vertical capacity analyses; low estimate s_u considering historic data, reductions in s_u due to the presence of gas hydrates and application of a global factor of safety of 2.0;
ii. For suction pile installation analyses; high estimate s_u considering historic data;
iii. Soil thixotropy effects;
iv. End bearing resistance applicability (no end bearing was cautiously considered).

No base end bearing was considered in the vertical bearing capacity analyses of the suction pile. This is a conservative assumption that takes into consideration no full contact between the base plate and seafloor due to seafloor slope or tilting of the caisson following installation. Annulus (tip) end bearing was also not considered as a conservative assumption due to the extremely low strength of the soil.

As described above, the preliminary analyses performed represent an upper bound of the suction pile geometry. Detailed analyses are expected to allow optimisation of the suction pile geometry. The following example considerations are recommended for detailed design:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Considering thixotropic effects for increased time intervals on a per unit basis in accordance with operation schedules and based on site-specific thixotropy data;
iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, S_t, consolidation parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u;
vi. Consideration of suction pile annulus and base resistance mobilisation with load rate and duration.
vii. The potentially highly sensitive Cocolith ooze and Sapropel formations (Geotechnical Soil Units 1 and 2).

It is recommended that these effects are quantified and considered for detailed design in accordance with any specific ExxonMobil design basis requirements.

Table S.1: Suction Pile Penetration and Installation Analyses Results

OD	Domino Drill Center 1				Domino Drill Center 2			
	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction
[m]	[m]	[kN]	[m]	[kPa]	[m]	[kN]	[m]	[kPa]
6	26.3 (4.4)	0.828	4.7	875	25.1 (4.2)	0.790	4.6	815
7	24.4 (3.5)	0.897	4.6	635	23.1 (3.3)	0.849	4.4	582
8	22.6 (2.8)	0.950	4.4	470	21.3 (2.7)	0.895	4.2	425
9	20.7 (2.3)	0.979	4.2	344	19.5 (2.2)	0.923	4.0	310
10	18.6 (1.9)	0.978	4.0	244	18.0 (1.8)	0.947	3.9	232
11	16.8 (1.5)	0.973	3.8	174	15.7 (1.4)	0.909	3.6	153

Notes:
OD = Outer Diameter
SWT = Self weight

CONTENTS

APPENDICES

TABLES IN THE MAIN TEXT

FIGURES IN THE MAIN TEXT

PLATES FOLLOWING THE MAIN TEXT

ABBREVIATIONS

BE	Best estimate
BH	Borehole
BML	Below mudline
BSR	Bottom-simulating reflectors
CAU	Anisotropically consolidated undrained triaxial test
CIU	Isotropically consolidated undrained triaxial test
CPT	Cone Penetration Test
CRS	Constant rate of strain
DODC-1	Domino Drill Center 1
DODC-2	Domino Drill Center 2
DSS	Direct simple shear test
ERP	Emergency Response Plan
ExxonMobil	ExxonMobil Exploration Production Romania Ltd
FC	Fallcone
FLET	Flow Line End Termination
FOS	Factor of safety
Fugro	Fugro GB Marine Limited
GSU	Geotechnical soil unit
HE	High estimate
ITA	In-line Tee Assembly
LDPC	Large diameter piston core
LE	Low estimate
LV	Laboratory vane
MBES	Multibeam echo sounder
MMO	Marine Mammal Observation Report
MTD	Mass transport deposit
NE	North east
OCR	Overconsolidation ratio
OD	Outer diameter
PEP	Project Execution Plan
PP	Pocket penetrometer
PSHA	Probabilistic seismic hazard analysis
SBP	Sub-bottom profiler
SEM	Scanning electron microscope
SGMP	Shallow Gas Management Plan
SRA	Site response analysis
SSHE	Safety, Security, Health and Environment Plan
SWT	Self-weight
TM	Transverse Mercator
TN	Technical Note

TV	Torvane
UU	Unconsolidated undrained triaxial test
VHMT	Vertical, horizontal, moment and tension
WSD	Working stress design

1. INTRODUCTION

1.1 Project Setting

The Neptun Deep development area is located within the Neptun block, Black Sea, offshore Romania. The planned development comprises the Domino Drill Centers 1 and 2. These will be positioned in approximately 900 m water depth, 23 km south-east of a planned platform location and the Pelican South Drill Center. The Domino Drill Centers are tied back to the Platform on the shelf at 123 m water depth by a flowline. A second flowline runs from the Pelican Drill Center on the shelf to the platform. A production pipeline will run from the planned platform location to shore. Figure 1.1 presents an overview of the planned development.

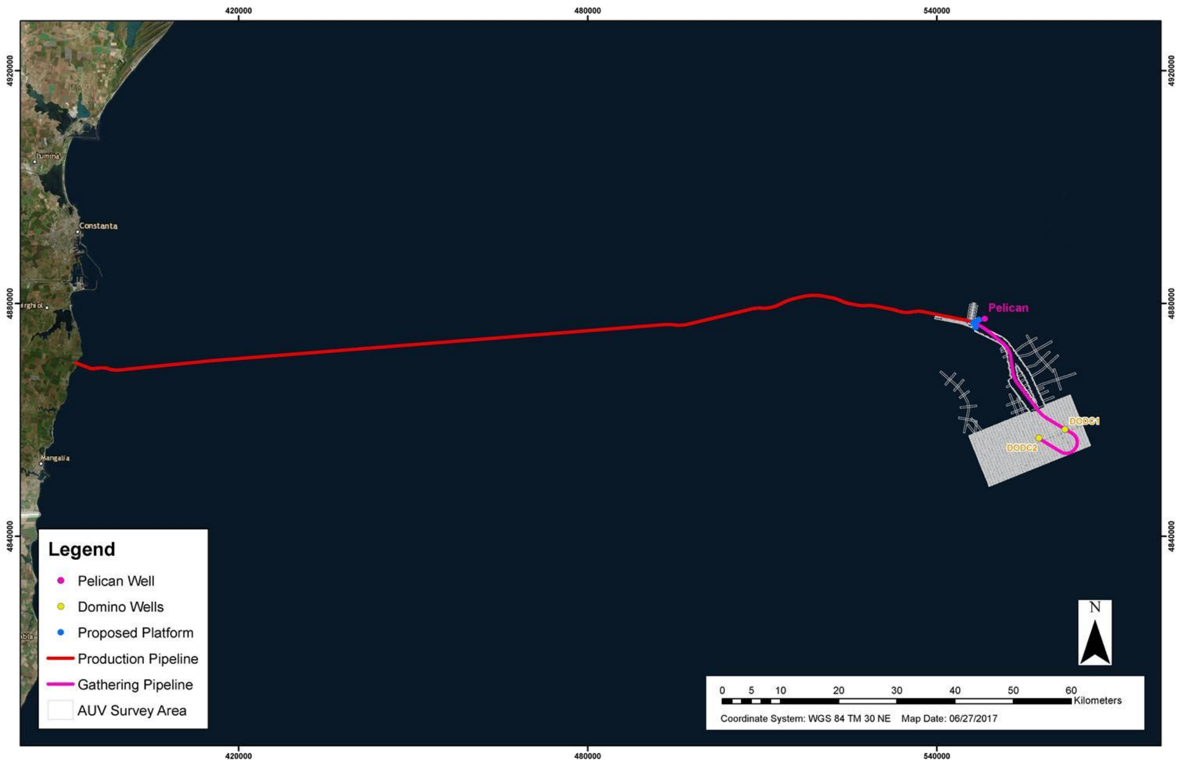

Figure 1.1: Main planned infrastructure associated with the Neptun Deep development area

1.2 Project Summary

ExxonMobil Exploration Production Romania Ltd (ExxonMobil) contracted Fugro to acquire and report a geotechnical site survey for a planned platform location, flowline route and three drill centres in the Neptun block, Black Sea, offshore Romania. This work was carried out under Marine Site Survey order A2552390. Call Off 2 Change Order 6.

The scope of work comprised:

i. 4 seabed cone penetration tests (CPT);
ii. 9 sampling boreholes;
iii. 7 CPT boreholes;
iv. 14 combined sampling and in situ testing boreholes;
v. 4 Pilotholes.

The site investigation was performed from the Fugro Synergy between 28 December 2017 and 8 February 2018.

The geotechnical data were acquired to assess the sub-seafloor conditions and to provide data for input to preliminary foundation design. This report forms part of a series reports for the geotechnical site investigation, as detailed in Table 1.1.

Table 1.1: Reporting Structure

<table>
<tr><th>Type</th><th colspan="2">Deliverable</th></tr>
<tr><td rowspan="5">Engineering / Interpretive</td><td colspan="2">WORK PACKAGE 4
INTERPRETIVE REPORTS</td></tr>
<tr><td>Integrated Report Update
Report Number: 173570-08</td><td>Slope Stability and debris flow run-out modelling Update Report
Report Number: 173570-09</td></tr>
<tr><td>Geological Interpretative Report
Report Number: 173570-06</td><td>Site Response Analysis
Report Number: 173570-07</td></tr>
<tr><td>Geotechnical Interpretive Report
Pelican Drill Center
Report Number: 173570-05a</td><td>Geotechnical Interpretive Report Platform
Report Number: 173570-05b</td></tr>
<tr><td>Geotechnical Interpretive Report
Domino Drill Center
Report Number: 173570-05c</td><td>Geotechnical Interpretive Report
Pipeline and Flowlines
Report Number: 173570 -05d</td></tr>
<tr><td rowspan="5">Factual</td><td colspan="2">WORK PACKAGE 3
FACTUAL/LABORATORY REPORT</td></tr>
<tr><td colspan="2">Laboratory and In situ Testing Data report
Report Number: 173570-04</td></tr>
<tr><td colspan="2">WORK PACKAGE 3
FIELD/RESULTS REPORTS</td></tr>
<tr><td>Operations Report
Report No.: 173570-01</td><td>MMO Report
Report No.: 173570-02</td></tr>
<tr><td colspan="2">Field Data Report
Report No.: 173570-03</td></tr>
<tr><td>Preliminary Data</td><td colspan="2">Preliminary Interpretation Technical Note
TN-173570-05</td></tr>
<tr><td rowspan="2">Execution</td><td>Project Execution Plan
Document No.: 173570-PEP</td><td>Safety, Security, Health and Environmental Plan
Document No.: 173570-SSHE</td></tr>
<tr><td>Emergency Response Plan
Document No.: 173570 -ERP</td><td>Shallow Gas Management Plan
Document No.: 173570-SGMP</td></tr>
</table>

1.3 Planned Drill Center Locations

Two deepwater drill center locations are planned at the Neptun block:

i. Domino Drill Center 1 (DODC-1);
ii. Domino Drill Center 2 (DODC-2).

It is understood that subsea manifolds are planned at the DODC-1 and DODC-2 locations. An in-line tee assembly (ITA) is understood to be planned for the DODC-1 location and a flowline end termination (FLET) is planned for the DODC-2 location. A single suction pile foundation is under consideration for each subsea manifold. Mudmats are under consideration for the ITA and FLET.

Table 1.2 summarises the coordinates of the above structures as provided by ExxonMobil. Plate 1 presents the detailed location plan of the Domino infield location. Fugro (2018a) presents in situ and laboratory test results.

Table 1.2: Location of Subsea Structures at the Domino Drill Center Locations

Description	Easting [m]	Northing [m]
In-line Tee Assembly (ITA)	562 476	4 857 931
Flowline End Template (FLET)	557 284	4 857 171
Manifold at DODC-1	562 446	4 857 885
Manifold at DODC-2	557 315	4 857 216

Notes:
Please refer to Table 1.3 for the appropriate coordinates reference system
a = Manifolds assumed to be installed at both Domino drill center 1 and 2

1.4 Scope of Report

This report presents the geotechnical soil parameters for each defined geotechnical soil unit for use in preliminary mudmat and suction pile, stability and installation analyses. This report should be read in conjunction with the laboratory and in situ testing data report (Fugro, 2018a). The following tasks were performed to produce the results presented this report:

i. Evaluation and interpretation of the geotechnical data for the drill center locations from the laboratory and in situ testing data report (Fugro, 2018a);
ii. Derivation of representative design soil parameters for preliminary engineering analysis;
iii. Determination of preliminary suction pile size based on stability and installation analyses using preliminary load data;
iv. Determination of preliminary mudmat sizes based on stability and installation analyses using preliminary load data.

1.5 Project Coordinate Reference System

Table 1.3 presents the geodetic parameters for this project.

Table 1.3: Project Coordinate Reference System Parameters

Geodetic Datum	
Datum	World Geodetic System 1984 (WGS84)
Ellipsoid	WGS84
Semi-major axis	6 378 137 m
Semi-minor axis	6 356 752 m
Inverse flattening	$^1/_f$ = 298.257223563
Angular unit	Degrees
Map Projection	
Projection system	Transverse Mercator (TM) 30° NE
Central meridian	30º 00′ 00.00″ east
Latitude of origin	0º north
False easting	500 000.0 m
False northing	0.0 m
Scale factor on central meridian	0.9996
Linear unit	Metres

1.6 Guidelines on Use of Report

Appendix A (guidelines on use of report) outlines the limitations of this report, in terms of a range of considerations including, but not limited to, its purpose, its scope, the data on which it is based, its use by third parties, possible future changes in design procedures and possible changes in the conditions at the site with time. It represents a clear exposition of the constraints which apply to all reports issued by Fugro. It should be noted that the Guidelines do not in any way supersede the terms and conditions of the contract between Fugro and ExxonMobil.

2. GEOLOGICAL SETTING

2.1 General

This section details the geological setting at the Domino drill center locations (Fugro, 2015a and 2016) and provides a summary of the geological model of the site based on a literature review and the results of the geohazard core logging (Fugro, 2014; 2015a). An updated geological setting will be presented in the updated integrated report for the site (Fugro 2018c: in press).

Geological processes in the Neptun block were controlled by global sea level change during the Quaternary. Figure 2.1 presents the sea level curve for the late Quaternary showing the changing water level in the Black Sea and environmental conditions over the last 30,000 years.

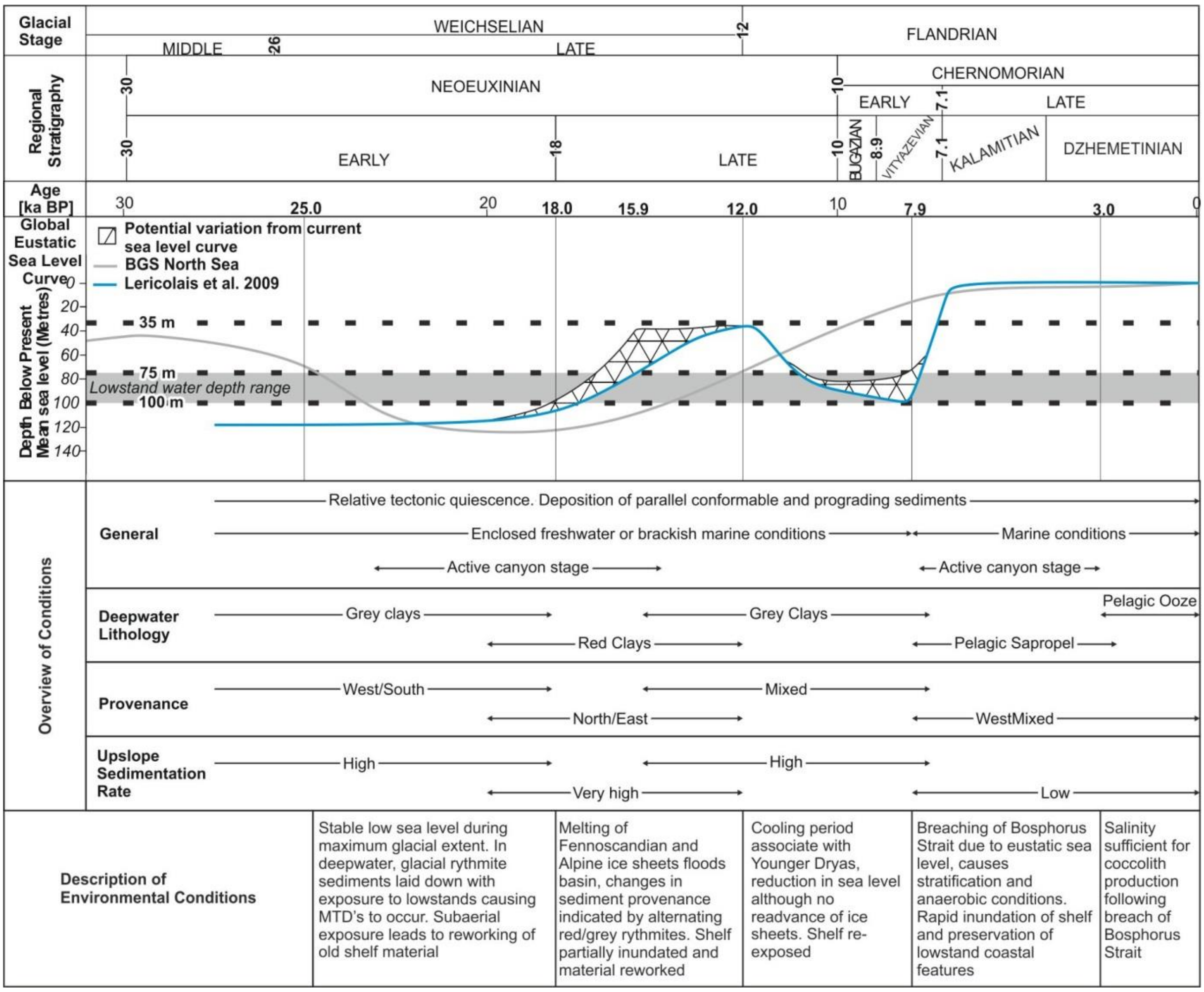

Figure 2.1: Sea level curve for the Neptun block in the late Quatenary

2.2 Site-specific Geological Setting

2.2.1 General

The planned Domino drill centers are located in the deepwater area of the Neptun block. Figure 2.2 shows the bathymetry in the vicinity of DODC-1 and DODC-2 and the main geomorphological features.

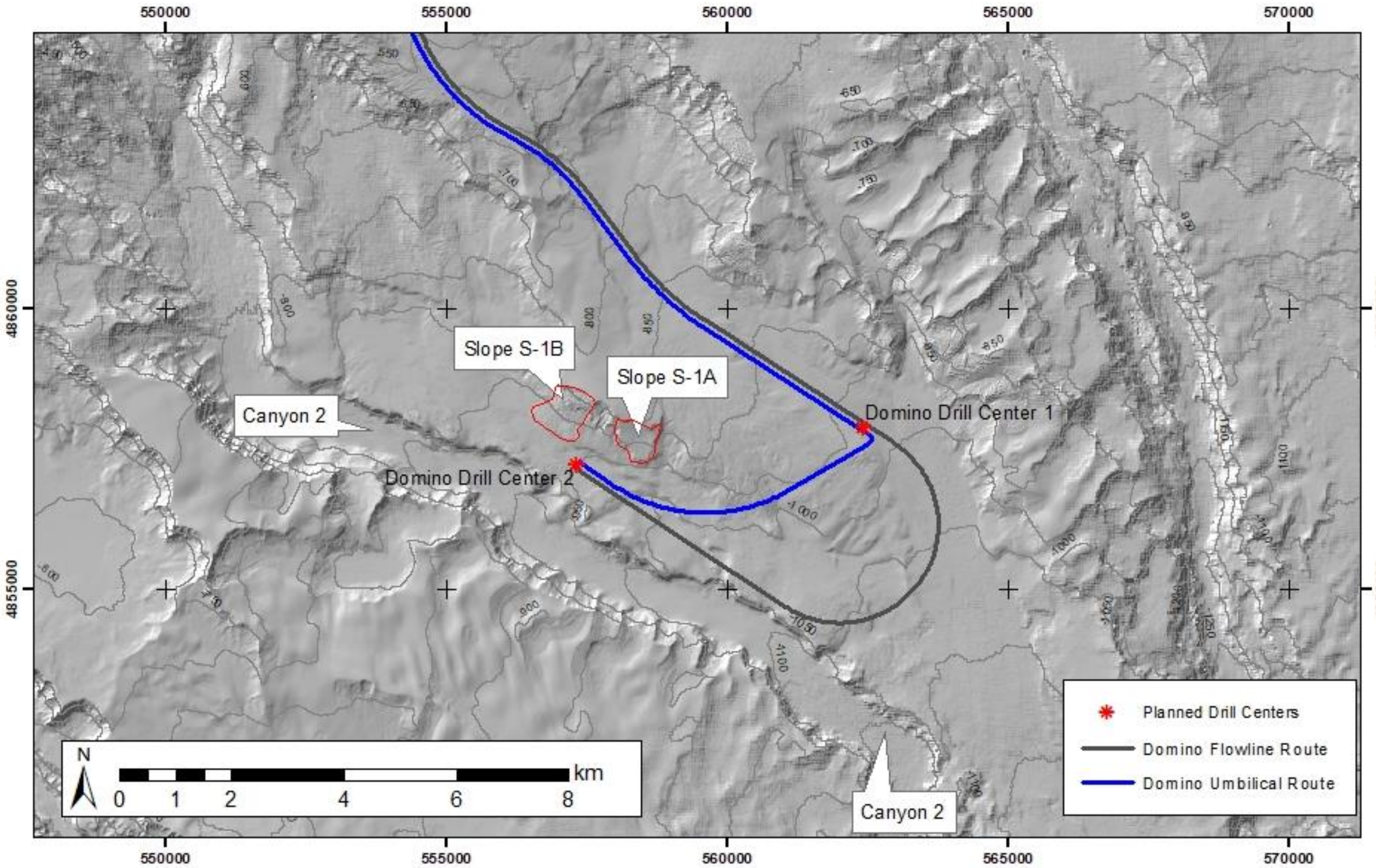

Figure 2.2: Bathymetry Map Showing the Main Geomorphological Features Within the Vicinity of Domino Drill Centers 1 and 2

The sediments within the foundation zone comprise lacustrine clays deposited in a freshwater environment between 25 thousand years before present (ka) and 8 ka.

Global sea level rise at 8 ka and the reconnection of the Bosphorus Strait led to the flooding of the Black Sea and the deposition of organic rich clay (sapropel) and coccolith ooze.

During periods of sea-level lowstand, the canyons acted as the main source of sediment transport, with sand and silt layers deposited during periods of high canyon activity. This is observable in the boreholes drilled at the two planned drill centers. DODC-2 is located closer to the canyon and as a result has more sand and silt layers within Geotechnical Soil Unit 5 than at the DODC-1 Location. Geotechnical Soil Unit 5 sediments at DODC-2 are also more overconsolidated than the sediments at DODC-1; this is interpreted to be due to erosion by downcanyons flows. The canyons are not interpreted to be active at present.

Table 2.1 presents the interpreted depositional environment for each geotechnical soil unit based on the geotechnical model presented In Fugro (2016a).

Table 2.1: Interpreted Depositional Environment for Each Geotechnical Soil Unit

Geotechnical Soil Unit	Depth to Base of Unit [m BML]		Generalised Soil Description	Interpreted Depositional Environment
	DODC-1	DODC-2		
1	1.9 to 3.0	1.5 to 2.8	Extremely low strength dark greenish grey CLAY with many extremely closely spaced planar parallel thin laminae of white calcareous silt	Clay with coccolith ooze: interpreted to represent deposition in a recent marine environment
2			Extremely low strength very dark greenish grey organic CLAY with many extremely closely spaced planar parallel thin laminae of very organic very dark grey and black clay	Sapropel: deposited during the mixing of saline water from the Mediterranean Sea with the freshwater of the Black Sea
3	8.0 to 9.3	8.8 to 10.0	Extremely low strength to low strength (normally consolidated) dark grey to reddish brown CLAY Becoming low strength (normally consolidated) grey CLAY with pockets and thin beds of silt and/ or sand with depth below seafloor	Lacustrine sediment with terrigenous input (reddish brown clays) Higher energy terrigenous input (silt and/or sand beds) e.g. background location B35T[a] shows a complete GU3 profile, with silt and/or sand beds at 25 m BML, ~40 ka)
4	12.0 to 13.0	11.6 to 12.1	Low strength to medium strength (lightly overconsolidated) grey slightly silty CLAY with many extremely closely spaced to very closely spaced planar inclined	GSU3 modified by slope failure processes (possible mass transport deposits)

Geotechnical Soil Unit	Depth to Base of Unit [m BML]		Generalised Soil Description	Interpreted Depositional Environment
	DODC-1	DODC-2		
			thin to thick laminae of black silt and dark grey clay	
5	14.9[b] to 35.0[b]	15.0[b] to 35.0[b]	Low strength to medium strength (lightly overconsolidated) CLAY with closely spaced to medium spaced very thin to thin beds of sand	GSU3 modified by slope failure processes (possible mass transport deposits) and retention of the silt and/or sand beds from older GSU3 sediments. May also occur as in situ overconsolidated older GU3 sediments with silt and/or sand beds
A	0.0 to 1.2	0.2 to 1.2	Low strength to medium strength (lightly overconsolidated) grey lightly silty CLAY with many extremely closely spaced to very closely spaced planar inclined thin to thick laminae of black silt and dark grey clay	GSU3 as hydrotroilite: diagenetic front with increased hydrogen sulphide, biogenic methane and amorphous iron sulphide content overprinting existing sediment (Jorgensen et al., 2004) Generally identified between 3 m and 5 m BML and generally < 0.5 m thick; therefore, most commonly associated with GSU3 but also identified in GSU4

Notes:
a = Location B35T was reported in Fugro (2015a)
b = End of borehole
GSU = Geotechnical Soil unit
BML = Below mudline
ka = A thousand years before present

2.2.2 Gas Hydrates

The presence of sediments with an elevated organic content has resulted in the production of methane. At the water depths associated with the Domino Drill Centers these are interpreted to be present as gas hydrates. Gas hydrates are inferred to be present at water depths greater than 600 m to 750 m water depth extending to the base of the hydrate stability zone for the Black Sea at approximately 1900 m water depth (Popescu et al., 2007). Bottom-simulating reflectors (BSR) are usually associated with the presence of gas hydrates and have been observed at other deepwater developments. No BSRs were observed within the sub-bottom profiler (SBP) data in the Infield Area, however are observable at depths between 100 m and 200 m BML below seafloor in the 3D Seismic data (Fugro, 2013).

2.2.3 Mass Transport Deposits

Mass Transports Deposits (MTDs) are a feature of the sediments at the Domino Drill Centers. Fugro, (2016a and 2016b) MTDs were mapped on the bathymetry data and SBP data as part of the geophysical interpretation for the Infield Area (Fugro, 2014a). In the MBES data, MTDs are observed as areas of irregular seafloor with a distinct headwall and often with blocks of material that were mobilised downslope.

Within the SBP geophysical data, MTDs are present as a series of acoustically structureless seismic packages that thin on the slopes and then thicken at the base. These have sharp basal contacts and irregular upper contacts. The relative ages of the MTDs were classified based on the relative thickness of the overlying deposits (Fugro, 2014a).

The MTDs within the Infield Area are classified based on their stratigraphic occurrence. Table 2.2 summarises the three classes of MTD. The MTDs observed in the geophysical data were confirmed during geohazard core logging (Fugro, 2015a). Level 1 and Level 2 MTDs are sampled at both DODC-1 and DODC-2.

Table 2.2: Classification of MTDs within the Infield Area and Upslope Areas

Level	Description	MBES Observations	Interpreted Environment of Deposition	Distribution
0	MTD occurs within surficial sediments	Rough seafloor – MTD well defined in MBES bathymetry data	Mass transport in recent marine sediments < 7.9 ka	Canyon 2
1	MTD occurs directly below surficial sediments	Rough seafloor – MTD well defined in MBES bathymetry data; deeper older features may be observed at seafloor	Mass transports in lacustrine sediments during period of sea level lowstand, high canyon activity and sediment input (12.0 ka to 7.9 ka)	East of Infield Area and in base of gullies and Canyon 2
2	MTD occurs below surficial sediments and acoustically well-bedded sediments	Irregular seafloor; less well defined than Level 1 due to burial by thicker acoustically well-bedded sediments	Mass transports in lacustrine sediments (40 ka to 20 ka)	Across Infield Area

Notes:
MBES = Multibeam echo sounder
MTD = Mass transport deposit
ka = A thousand years before present

DODC-2 is located at the base of Slope S1B (see Figure 3.6). This slope is a level 1 Mass transport deposit. Dating of the slope failure on S1B shows date this slope failed at approximately 5500 years before present (Fugro, 2018d).

3. GEOTECHNICAL PROFILE

3.1 General

This section details the geotechnical soil units observed in the boreholes at the Domino Drill Center locations.

3.2 Geotechnical Data

3.2.1 General

Eight geotechnical data locations were primarily considered at the two drill center locations. Table 3.1 summarises the details of the primary borehole used in this study. Fugro (2018a) presents the geotechnical borehole logs, laboratory data and in situ CPT data at the Domino Drill Center locations.

Table 3.1: Primary Borehole Data Considered

Domino Location	Borehole Name	Easting[a] [mE]	Northing[a] [mN]	Water Depth[b] m [MSL]			Termination Depth [m BML]	Comment
				Pressure Sensor	Echo Sounder	Drill string		
Drill Center 1 (DODC-1)	DD1-BH-01	562 441	4 857 887	974.6	974.1	972.1	35.0	CPT and sampling BH
	DD1-BH-02	562 468	4 857 934	973.8	974.0	972.8	20.6	Sampling BH
	DD1-BH-02a	562 465	4 857 931	974.8	974.0	973.4	34.5	
	DD1-CPT-03	562 414	4 857 845	973.1	973.1	972.4	14.9	CPT only BH
Drill Center 2 (DODC-2)	DD2-BH-01	557 322	4 857 211	950.1	949.3	948.0	35.0	CPT and sampling BH
	DD2-BH-02	557 290	4 857 171	950.6	950.0	948.8	26.6	Sampling BH
	DD2-BH-02a	557 287	4 857 167	950.6	950.0	950.2	34.6	
	DD2-CPT-03	557 332	4 857 265	949.1	948.3	947.5	14.8	CPT BH

Notes:
a = Coordinate system WGS84 TM 30E
b = Water depth measured using echosounder
MSL = Mean sea level
BML = Below mudline
CPT = Cone penetration test
BH = Borehole

Table 3.2 summarises the supplementary geotechnical data used in the evaluation of the geotechnical soil properties to help identify data trends and reduce uncertainty caused by data gaps. These data were selected because the water depth and interpreted geotechnical conditions are similar to the Domino drill center locations.

Table 3.2: Additional Borehole Data Evaluated

Geotechnical Location	Description	Reference
B19CPT/LDPC	Seabed CPT and large diameter piston core	Fugro (2015a)
B20CPT/LDPC	Seabed CPT and large diameter piston core	
B22CPT/LDPC	Seabed CPT and large diameter piston core	
A36CPT/LDPC	Seabed CPT and large diameter piston core	
S1A-BH-01	Continuous sampling borehole	Fugro (2018a)
S1A-CPT-01	Continuous CPT borehole	
S1A-BH-02	Continuous sampling borehole	
S1A-CPT-02	Continuous CPT borehole	

Notes:
CPT = Cone penetration tests
LDPC = Large diameter piston cores
BH = Borehole

3.2.2 Geotechnical Soil Units

Table 3.3 presents the depth of the interpreted geotechnical units for the Infield Area locations. There is generally good agreement with the geotechnical model and limited variability between the locations at each of the planned developments.

Table 3.3: Geotechnical Units Interpreted to be Present

Geotechnical Soil Unit	Depth to Base of Unit [m BML]		Generalised Soil Description
	DODC-1	DODC-2	
1	1.9 to 3.0	1.5 to 2.8	Extremely low strength dark greenish grey CLAY with many extremely closely spaced planar parallel thin laminae of white calcareous silt
2			Extremely low strength very dark greenish grey organic CLAY with many extremely closely spaced planar parallel thin laminae of very organic very dark grey and black clay
3	8.0 to 9.3	8.8 to 10.0	Extremely low strength to low strength (normally consolidated) dark grey to reddish brown CLAY Becoming low strength (normally consolidated) grey CLAY with pockets and thin beds of silt and/ or sand with depth below seafloor
4	12.0 to 13.0	11.6 to 12.1	Low strength to medium strength (lightly overconsolidated) grey slightly silty CLAY with many extremely closely spaced to very closely spaced planar inclined thin to thick laminae of black silt and dark grey clay
5	14.9[a] to 35.0[a]	15.0[a] to 35.0[a]	Low strength to medium strength (lightly overconsolidated) CLAY with closely spaced to medium spaced very thin to thin beds of sand
A	0.0 to 1.2	0.2 to 1.2	Low strength to medium strength (lightly overconsolidated) grey lightly silty CLAY with many extremely closely spaced to very closely spaced planar inclined thin to thick laminae of black silt and dark grey clay

Notes:
BML = Below mudline
DODC-1 = Domino drill center 1
DODC-2 = Domino drill center 2
a = Depth of borehole

3.3 Geological Considerations

3.3.1 Coccolith Ooze

The Coccolith ooze (Geotechnical Soil Unit 1) is present from seafloor to 0.5 m BML at DODC-1 and DODC-2 locations. Geotechnical Soil Unit 1 was sampled across the infield area by Fugro (2015c) and is discussed in detail by Fugro (2016a). It was difficult to sample Geotechnical Soil Unit 1 due to its extremely low strength and high water content. Piston sampling was performed at the Drill Center locations. From the samples recovered, it was observed that this geotechnical soil unit was not as well preserved. The piston samples resulted in low sample quantities of Geotechnical Soil Unit 1 being recovered with high sample disturbance.

The observed sample disturbance within Geotechnical Soil Unit 1 is most likely to have caused variability in water content and unit weight measured results rather than the composition of the unit i.e. varying coccolith ooze content. The coccolith ooze formation has a high voids ratio. During sampling the material may have been slightly compressed resulting in potential changes in volume and break down of the soil fabric, which may in turn may affect the water content and unit weights measured from recovered samples.

A review of the Fugro (2015a) data shows that the sample quality, based on visual inspection, is best in the box core subsamples and poorest in piston samples.

When sampling at the Domino Drill Center boreholes it was not possible to recover samples of Coccolith ooze from seafloor using the standard piston sampling equipment with a 1 m tube attached. Therefore, a 3 m liner was used where Geotechnical Soil Unit 1 were prevented from falling out of the liner by the more competent samples of Geotechnical Soil Unit 2 and 3.

The laminations of coccoliths (white stripes in the samples) are referred to as a coccolith ooze and are more prevalent towards the top of the unit (i.e. seafloor) as shown in Figure 3.1. Coccolith ooze occurs in nested stacks with no identified cementation or overgrowths of calcite at points of grain contact.

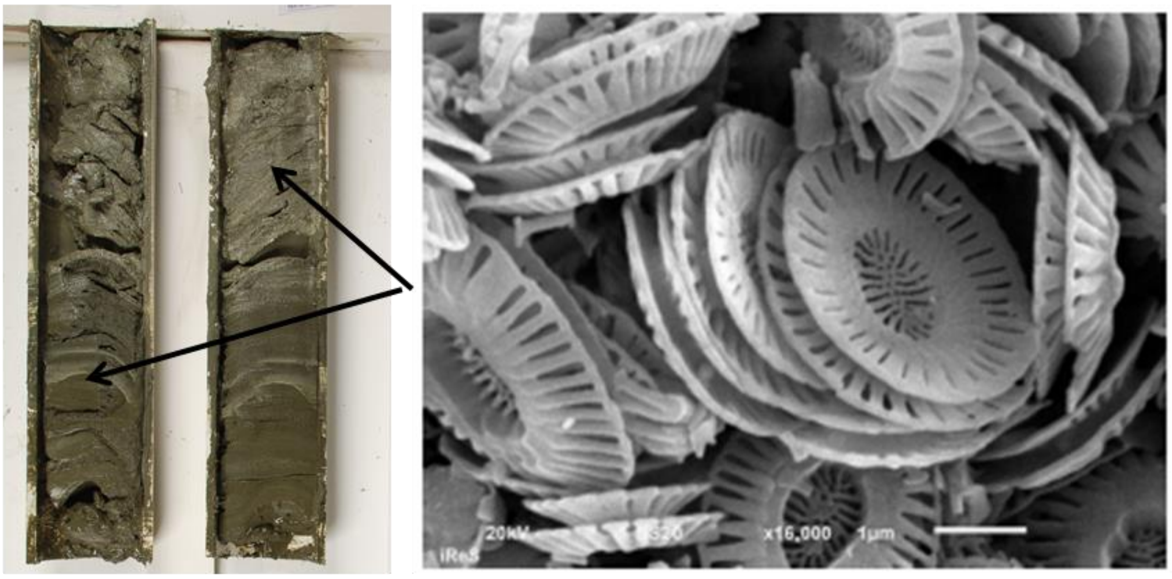

Figure 3.1: (Left) Photograph of GU1 clay with coccolith ooze (white stripes) between 0.0 m and 0.4 m BML from location A16 (Fugro, 2015a) and (Right) a scanning electron microscope image of individual calcite coccoliths

It was difficult to identify Geotechnical Soil Unit 1 using in situ test data due to the settlement of the seabed frame at mudline. Combinations of low undrained shear strengths (<1 kPa) and low unit weights at the seafloor caused seabed frame settlements (e.g.>35 cm) in 2014 (Fugro, 2015a).

3.3.2 Sapropel

The Sapropel formation (Geotechnical Soil Unit 2) consists of organic clay and is observed from 0.5 m to 2.5 m BML at both the DODC-1 and DODC-2 locations.

The average of organic content from GU2 is up to 11 %. Geotechnical Soil Unit 2 is classified as an amorphous organic soil (H10) using the Von Post classification included in ASTM D5715 (2006). The organic clay is a black soil that is referred to as a sapropel as shown in Figure 3.2. The scanning electron microscope (SEM) image of the sapropel presents a fibrous material comprising abundant tubular biogenic structures which may be radiolaria or sponge spicules composed of biogenic silica and has a relatively low particle density and a very high porosity (visual assessment from SEM).

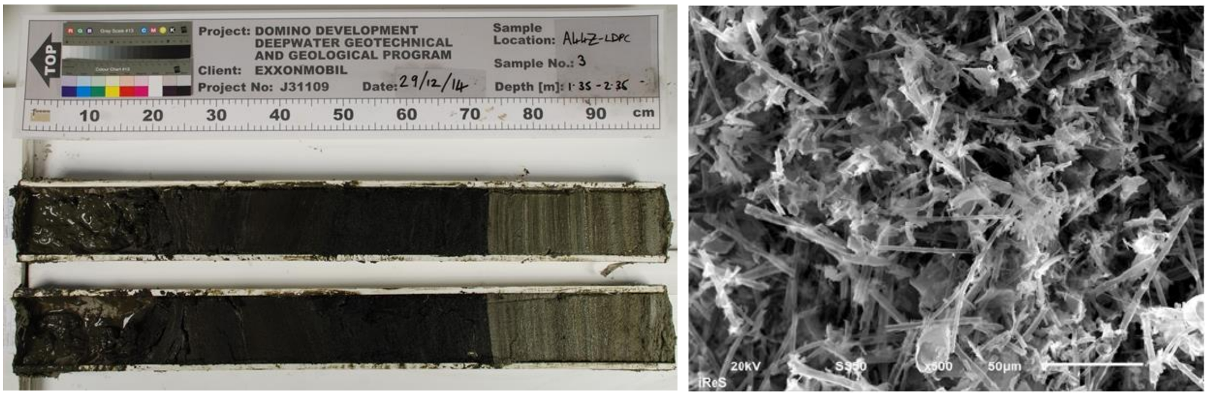

Figure 3.2: (Left) Sample photograph from A44Z-LDPC (Fugro, 2015c) showing GU2 black organic clay (sapropel) between 1.35 m and 2.05 m BML and (Right) a scanning electron microscope image of abundant fibrous tubular biogenic structures from GU2 organic clay (sapropel)

The lateral variability of the organic content is assumed to be negligible across the Infield Area due to uniform pelagic sedimentation. The organic content test results show a reasonable correlation between the thickness of the sapropel and increasing organic content.

The variability of measured water content and unit weight results for Geotechnical Soil Unit 2 is likely to be caused by the sample quality and in the latter case, difficulty in sample preparation. Such effects are common in high voids ratio and extremely low strength soils.

The sample quality based on visual inspection of Geotechnical Soil Unit 2 is good to fair in piston liner samples, but the unit is still susceptible to sample separation (voids within the sample liner). The sample separation in Geotechnical Soil Unit 2 noted by Fugro (2018a) is possibly related to the extremely low strength soil and the recovery methods of the piston samples, and because samples are held horizontally during subsampling.

The difficulties in identifying Geotechnical Soil Unit 2 from the in situ test data is exacerbated by the seabed frame interacting with the near-seafloor sediment (i.e. seabed frame settlement) similar to the Coccolith Ooze.

3.3.3 Gas Hydrates and Gas Hydrate Dissociation

In Geotechnical Soil Units 3, 4 and 5, biogenic gas (methane) was observed in the boreholes. Due to the water depth and pressures at the DODC-1 and DODC-2 Locations this is interpreted to occur as stable gas hydrates. The gas hydrates are interpreted to have disassociated and expanded following sampling and recovery. This sample expansion resulted in the opening of voids and fissures within the samples. Sample tubes which appear to have greater than 90 cm recovery before extrusion had less than 60 cm of sediment within the tube once extruded. Section 4.6.5 discusses the results of the headspace gas testing where the gas was detected from the DODC locations. Section 4 discusses the effects on the derived design soil parameters.

Evidence of gas hydrates as recorded in two geohazard cores (A24Z and A45Z) during the geohazard core logging (Fugro, 2015a). Although these cores are located up to 12 km from the proposed drill centers and at shallower water depths, the geotechnical soil conditions are similar to those at the DODC-1 and DODC-2 locations. Therefore, these cores provide indirect evidence of the disassociation hydrates within the Infield Area; this process may also occur at the Domino Drill Centers.

Cores A24Z at 13.10 m BML and A45Z at 7.05 m BML have a high moisture content and a disturbed (crumb) texture attributed to the past presence of gas hydrates. The crumb soil texture has a large number of visible voids and has a remoulded/disturbed character. Figure 3.3 presents photographs of the crumb soil texture. The soil texture at these locations and depths are both present within Geotechnical Soil Unit 4. This soil texture has only been identified at these two sample locations and cannot be reliably predicted using geophysical data or extrapolated to the Domino Drill Center Locations. Corresponding CPT data at A24Z displays a reduction in undrained shear strength near the same depth below seafloor however shows no significant changes in pore pressure or sleeve friction. The reduction in undrained shear strength and the presence of the crumb soil texture are considered to be linked. The crumb soil texture is not interpreted to extend laterally from the core location where it was tested however may be locally present in other parts of the Infield Area.

The crumb soil texture is thought to be linked to the dissociation of gas hydrates in the sample caused by an increase in temperature and decrease in pressure following recovery of the sample to sea level.

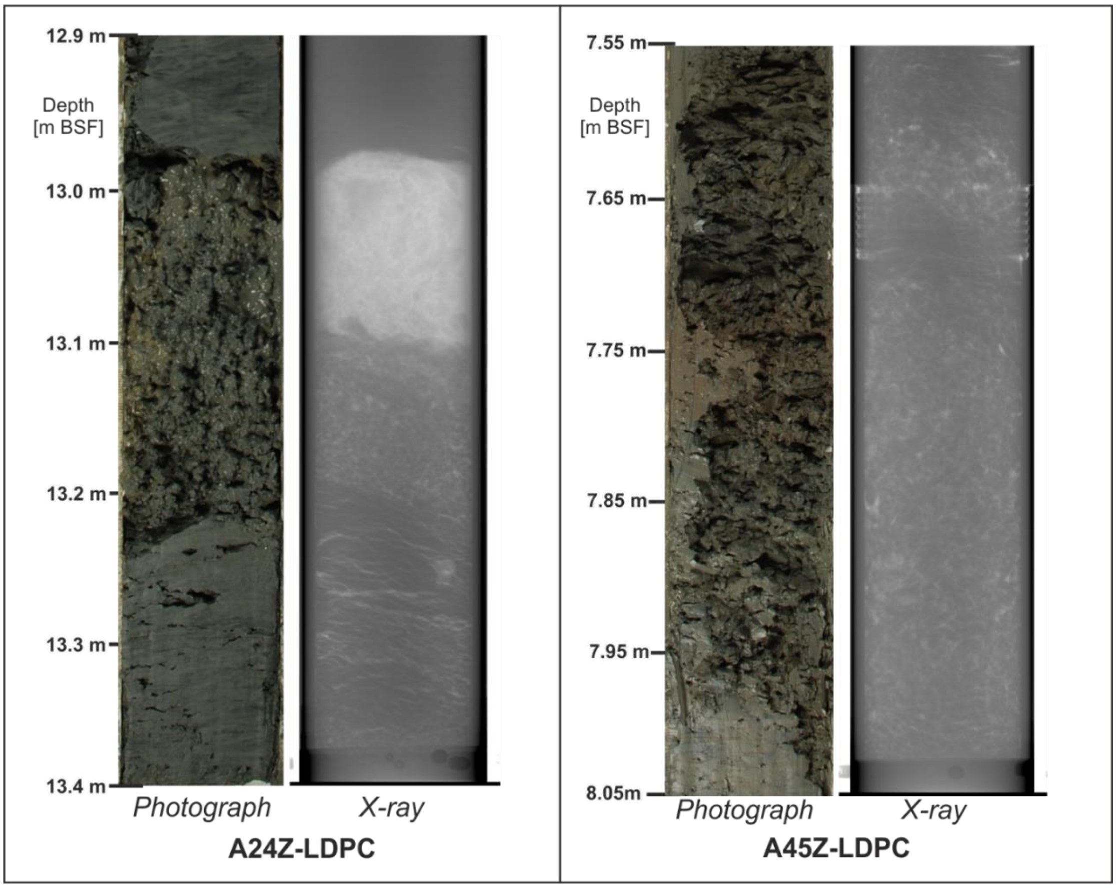

Figure 3.3: Geohazard cores A24Z-LDPC and A45Z-LDPC (Fugro, 2015a) showing crumb texture and evidence of interpreted gas hydrates within the sediment

3.3.4 Soil Structure Observations

During offshore laboratory tests, the sample failed prematurly along fissures and voids resulting in reduced undrained shear strength values from laboratory vane (LV), torvane (TV) and pocket penetrometer (PP) tests. The LV, TV and PP results are lower than the onshore strength tests and therefore are not considered representative of the sediment strength. The difference in sediment strength is most noticeable in Geotechnical Soil Unit 5 where higher quantities of gas and gas indictors were measured (Section 4.6.5) and where there was more sample expansion.

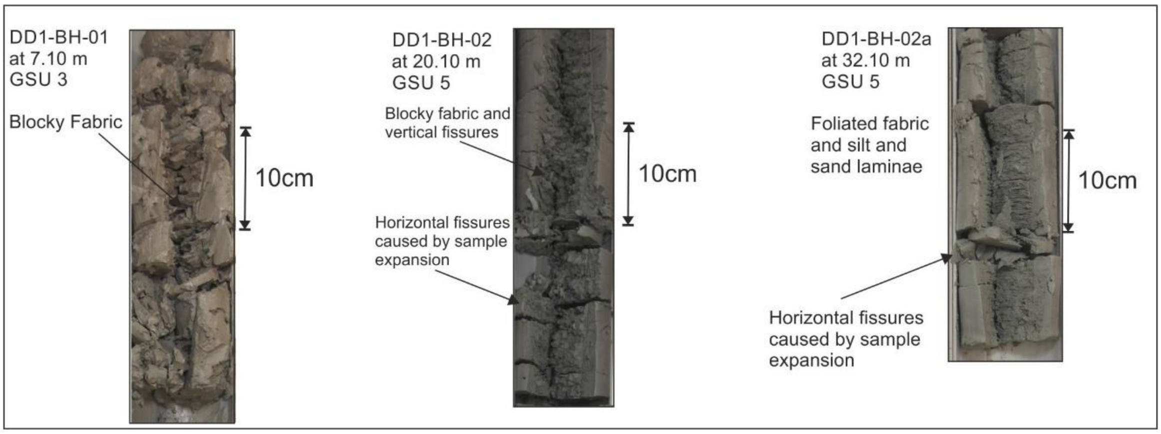

Figure 3.4: Examples of blocky and fissured soil fabric observed at DODC-1 and DODC-2

3.4 Lateral Variability and Buried Mass Transports

Geotechnical Soil Unit 3 typically comprises normally consolidated buried mass transport deposits and lacustrine sediment deposited before 8 Ka (Fugro, 2018c: in press). DD2-BH-01 and DD2-CPT03 samples mass transport deposits comprising fully mobilised sediment that in indistinguishable from the underlying lacustrine sediment.

Borehole DD2-BH-02 shows an increase in undrained shear strength between 3.8 m and 5.8 m BML (Figure 3.5) and corresponding increase in unit weight and decrease in moisture content. This is interpreted to be an overconsolidated intact block within a buried mass transport deposit matrix from a failure of slope S1B located approximately 1 km North-west of DODC-2 (Figure 3.6). The sub-bottom profiler data shows a block of sediment close to seafloor at the DD2-BH-02 borehole location, with a blocky seafloor caused by buried blocks of sediment observable in the bathymetry data (Figure 3.7).

This increase in undrained shear strength is not interpreted to be related to Geotechnical Unit A as the soil does not show the characteristic black colour with iron sulphide nodules; the undrained shear strength is also higher than usually observed in Geotechnical Soil Unit A.

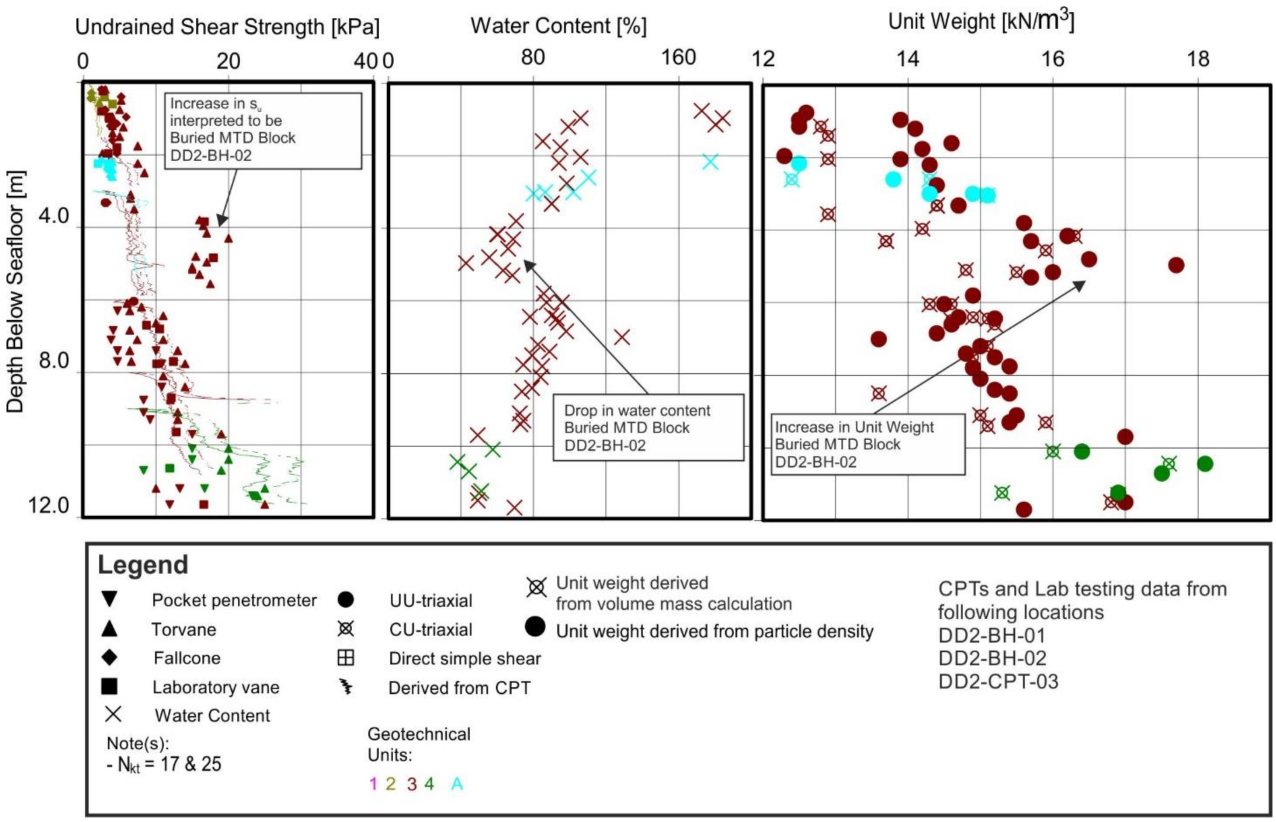

Figure 3.5: Increase in undrained shear strength in DD2-BH-02

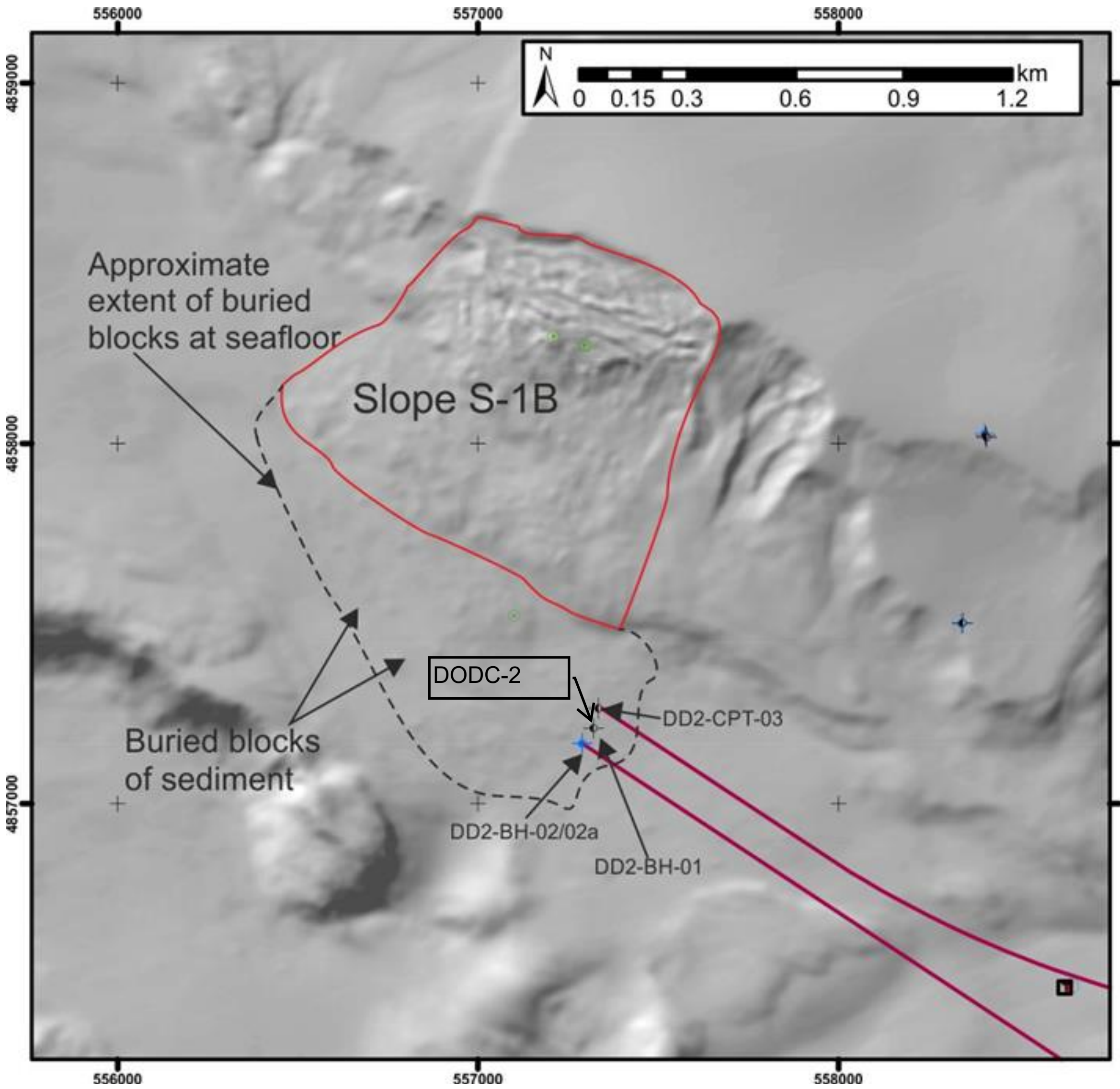

Figure 3.6: Shaded bathymetry showing buried mass-transport block

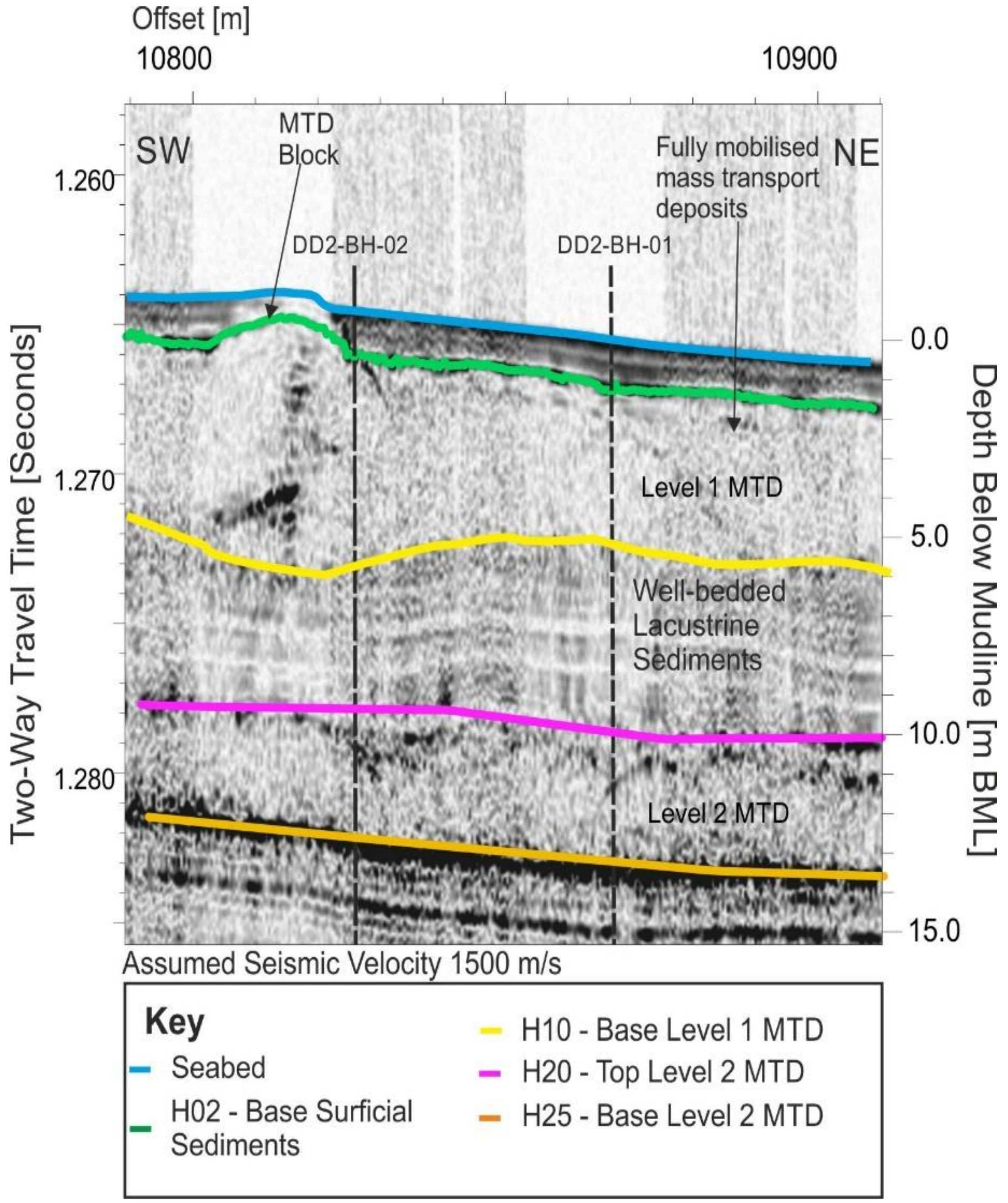

Figure 3.7: Sub-bottom profiler data example

4. INTERPRETATION AND EVALUATION OF GEOTECHNICAL DATA

4.1 Introduction

This section presents an interpretation and evaluation of the soil parameters for the DODC-1 and DODC-2 locations within the Domino field. The soil parameters discussed in this section are summarised in the plates following the main text of this report. The plates present a selection of individual and/or composite plots with recommended representative low estimate (LE), best estimate (BE) and high estimate (HE) parameter profiles applicable for preliminary suction pile and preliminary mudmat sizing. The parameters profiles were derived statistically according to DNV GL (2017a) and using engineering judgement.

The soil parameters discussed in this report were evaluated based on the geotechnical soil unitisation as described in Section 3.

LE and HE terms are used to represent a credible indication of the low and high distribution of the representative geotechnical parameters of the soil, with engineering judgement applied, or particular geotechnics risks. It should be noted that the LE and HE terms are not necessarily lower or upper bound soil properties but rather recommended low or high values, which could be used as reference during derivation of soil parameters for preliminary mudmat sizing and preliminary suction pile sizing.

The BE profile for a soil parameter is typically based on a statistical average and engineering judgement of the available data from the geotechnical site investigations and subsequent laboratory testing.

4.2 Basic Soil Physical Properties

4.2.1 General

The following basic physical soil properties were discussed in this section:

i. Water content;
ii. Particle density;
iii. Total unit weight;
iv. Plasticity data.

Sections 4.2.1 to 4.2.4 discusses the above the basic physical soil properties derived from field data and laboratory test data. In general, the basic physical soil properties are relatively consistent within the geotechnical soil units and only minor data scatter is observed.

4.2.2 Water Content

Plate 2 presents water content (w) data versus depth plot for the DODC-1 location and Plate 3 presents w data versus depth plot for the DODC-2 location.

The LE, BE and HE design lines were derived statistically from data presented by Fugro (2016a and 2018) and using engineering judgement for each geotechnical soil unit. A single design LE, BE and HE design soil profile applicable at the DODC-1 and DODC-2 locations was derived for Geotechnical Soil Units 1 and 2. Current and historical data were considered in deriving the design water content values; however, the historical data is not presented on Plates 2 and 3. The data presented indicates that:

i. The highest water content is observed in Geotechnical Soil Unit 1 (Coccolith ooze formation) at both drill center locations. The water content ranges from 220 % to 932 %, the latter exceeding the plate scale. Geotechnical Soil Unit 1 is the coccolith ooze and therefore the observed high water content are consistent with the geological interpretation;
ii. The elevated water content in the upper part of Geotechnical Soil Unit 2 (Sapropel formation) at both drill center locations is expected to have occurred as a result of the influx of saline rich pore-water from higher salinity seawater following the breach of the Bosphorus Strait and is reflected in the design lines provided. The water content in Geotechnical Soil Unit 2 ranges from 74 % to 727 %;
iii. The water content in Geotechnical Soil Unit 3 at the DODC-1 location is consistent. The water content at the DODC-2 location varies between 3.75 m and 5.50 m BML. This variation is interpreted to be due to the presence of a rafted block of overconsolidated material from the failure of slope S1B located approximately 1 km North-west (See Section 3.4). Related design soil parameters such as unit weight (see Section 4.2.4) and undrained shear strength (see Section 4.4.2) show corresponding data trends;
iv. The water contents in Geotechnical Soil Units 4 and 5 at both drill center locations are observed to be generally constant with depth within their respective units.

4.2.3 Particle Density

Plate 4 presents the particle density data versus depth profile for the DODC-1 and DODC-2 locations. The LE, BE and HE soil profiles were determined by engineering judgement.

Limited particle density data were available in Geotechnical Unit 1, 2 and 4. Therefore, a review of the current and historical data at both drill center locations was performed to assist in determining the particle density profiles. A single combined LE, BE and HE design profile applicable at both the DODC-1 and DODC-2 locations was derived.

Only two data points are available for Geotechnical Soil Units 1 and 2. Therefore, a constant soil profile based on the two data points was derived, which is generally consistent with previous datasets (Fugro, 2016a).

Particle density data for Geotechnical Soil Units 3 to 5 are relatively constant with depth as might be expected for similar depositional environments.

The particle density BE profile was used to derive unit weight from water content data.

4.2.4 Total Unit Weight

Plate 5 presents the total unit weight (γ) data with depth at DODC-1 location. Plate 6 presents the γ data with depth at DODC-2 location. The unit weight data were determined from:

i. Volume-mass calculations from undisturbed samples;
ii. Measured water content and unitised BE particle density values.

The LE, BE and HE design lines were derived statistically from data presented by Fugro (2016a and 2018a) and using engineering judgement for each geotechnical soil unit. A single design LE, BE and

HE design soil profile applicable at the DODC-1 and DODC-2 locations was derived for Geotechnical Soil Units 1 and 2. Current and historical data were considered in deriving the design water content values; however, the historical data is not presented on Plates 5 and 6.

Measured water content unit weights were generally higher than the volume-mass unit weights. The volume-mass unit weights may have been affected by the presence of shallow gas in the sand and silt partings, as noted in Section 3.3.3, causing volume expansion and therefore affecting the volume calculation. In Geotechnical Soil Unit 5, a larger difference is observed between unit weights determined from volume-mass and measured water content at DODC-2 in comparison to DODC-1. This larger difference is due to a higher content of silt and sand partings being present, which is reflected in the CPT data. Therefore, unit weight design lines were generally biased to data calculated from measured water content and particle density measurements.

The unit weight data within Geotechnical Soil Unit 3 at the DODC-1 location is generally consistent. The unit weight values at the DODC-2 location increase between 3.75 m and 5.50 m BML. This is interpreted to be due to the presence of a rafted block of overconsolidated material from the failure of slope S1B located approximately 1 km North-west (See Sections 3.4 and 4.2.2).

BE submerged unit weight (γ') was determined from γ and used in mudmat and suction pile stability and installation analysis. Equation 4.1 describes the calculation of γ'.

$$\gamma' = \gamma - \gamma_w$$

Equation 4.1

Where:
γ_w = Unit weight of seawater (taken as ~ 10.0 kN/m^3)

4.2.5 Plasticity Data

Plasticity profiles were determined using data presented by Fugro (2018) and historical data presented by Fugro (2016a). Composite plots of plastic limit (W_p) and liquid limit (W_L) data were delineated into soil units and used to determine representative parameter profiles of plasticity index (I_p). The LE, BE and HE design lines for I_p were derived from the available data for each soil unit. These data were derived for input to the DNV GL (2017c) suction pile vertical bearing capacity analyses.

Plate 7 presents the plastic limit and liquid limit plot for DODC-1 location. Plate 8 presents the plastic limit and liquid limit plot at DODC-2 location.

Plate 9 presents the plasticity index versus depth plot for the DODC-1 location. Plate 10 presents the plasticity index versus depth plot for the DODC-2 location. Equation 4.2 describes the calculation of I_p from the W_L and W_p.

$$I_p = W_L - W_p$$

Equation 4.2

Plate 11 presents the BS 5930 (2015) plasticity chart for the DODC-1 location. Plate 12 presents the BS 5930 (2015) plasticity chart for the DODC-2 location. Limited data are available in Geotechnical Soil Unit 1; therefore, plasticity data for Geotechnical Soil Unit 1 and 2 were determine as a single unit. Table 4.1 summarises the plasticity ranges for the DODC-1 and DODC-2 locations as observed from the plasticity charts.

Table 4.1: Plasticity Ranges of Geotechnical Soil Units

Geotechnical Soil Unit	Plasticity Range
1 and 2	Extremely high plasticity
3	High to very high plasticity
4	High to very high plasticity
5	Intermediate to high plasticity

For Geotechnical Soil Unit 1 and 2, in the top 2.5 m BML at both drill center locations, an extremely high soil plasticity is observed from the measured data, which is consistent with higher water content and lower unit weight values. This is consistent with the expected properties of the coccolith ooze (Geotechnical Soil Unit 1) and the sapropel formation (Geotechnical Soil Unit 2).

The I_p in Geotechnical Soil Units 3 to 5 generally shows a decreasing trend with depth. This reduction in I_p with depth is due to the increasing sand and silt content within the deeper geotechnical soil units as can be observed from the borehole data. An outlying data point was noted within Geotechnical Unit 3 at approximately 12 m BML. This data point appears to have been affected by increased sand and silt content leading to an anomalously low $I_p$and was therefore ignored in the interpretation.

Liquidity Index (I_L) was derived from the plasticity data and water content (w). It relates the water content of a fine-grained soil to its plasticity data. Equation 4.3 describes the calculation of I_L:

$$I_L = \frac{w - W_P}{I_P}$$

Equation 4.3

Plate 13 presents I_L versus depth for the DODC-1 location. Plate 14 presents I_L versus depth for the DODC-2 location. The I_L plot shows that the I_L generally decreases with depth.

For Geotechnical Soil Units 1 to 3 the I_L is high (greater than 1). This is expected of the coccolith and sapropel formations which make up these units and can be related to the extremely high water contents observed in these three geotechnical units of between 80 % and 575 %. This is expected to be indicative of extremely high sensitivities and correspondingly low remoulded undrained shear strengths.

The I_L within Geotechnical Soil Unit 4 is observed to be approximately one and, as noted for Geotechnical Units 1 to 3, is expected to be indicative of potential for high undrained shear strength sensitivity.

The I_L within Geotechnical Soil Unit 5 is observed to be approximately 0.5. This lower I_L can be attributed to the higher sand content of this geotechnical soil unit.

Further discussion relating to soil sensitivity is provided in section 4.4.5.

4.3 In Situ Testing

4.3.1 General

CPT data acquired in the boreholes for the DODC-1 and DODC-2 locations were used in deriving cone resistance (q_c) and sleeve friction (f_s) profiles. CPT q_c data correlations were used for foundation installation analyses and to derive indicative undrained shear strength (s_u), overconsolidation ratio (OCR) and soil strength sensitivity (S_t) data.

4.3.2 Measured Cone Resistance Profile

Measured cone resistance data were derived from CPT data presented in Fugro (2018a). Plate 15 presents the unitised q_c data and the determined LE, BE and HE q_c profiles for the DODC-1 location. Plate 16 presents the unitised q_c on an enhanced scale for the DODC-1 location. Plate 17 presents the unitised q_c data and the determined LE, BE and HE q_c profiles for the DODC-2 location. Plate 18 presents the unitised q_c on an enhanced scale for the DODC-2 location.

The design profiles were determined based on engineering judgement. The LE q_c profile was used in deriving the s_u for mudmat stability and suction pile vertical capacity analyses. The HE profile was considered in mudmat and suction pile installation analyses and to derive HE s_u for input to the same analyses. It should be noted, however, that because this profile is not necessarily location-specific, the HE q_c profile reflects the risk of high cone resistance due to the presence of sand and silt layers that may be expected within the immediate vicinity and may not necessarily reflect a continuous penetration resistance profile. This risk should be considered by the designer on a case-by-case basis. The HE q_c design profiles used in the engineering analysis, which in some instances differ to Plates 16 to 18, are presented on Plates 37 and 38.

The measured q_c data were used to derive OCR as described in Section 4.5.2.

Geotechnical Soil Units 1 to 4 are observed to be generally consistent at both DODC-1 and DODC-2 locations.

Wider CPT profile bounds are presented for Geotechnical Soil Unit 5 at the DODC-2 location compared to the DODC-1 location. This is due to the presence of more sand and silt partings at the DODC-2 location. DODC-2 is located near a canyon, as described in Section 2, and is therefore expected to have a higher sand and silt content transported from the canyon.

4.3.3 Sleeve Friction

Plate 19 presents the CPT sleeve friction with depth for the DODC-1 location. Plate 20 presents the CPT sleeve friction with depth for the DODC-2 location. BE design profiles derived using engineering judgement are presented.

The f_s data was used as a reference in deriving remoulded undrained shear strength and strength sensitivity (see Section 4.4.4 and Section 4.4.5, respectively).

4.4 Monotonic Undrained Shear Strength

4.4.1 General

This section details the methods used to determine monotonic undrained shear strength (s_u) for each soil unit present for the DODC-1 and DODC-2 locations. LE, BE and HE s_u profiles were derived based on engineering judgement. The design profiles represent a credible range of the high quality laboratory test data. LV, FC, DSS, UU, CAU and CIU laboratory test data presented by Fugro (2018a) supplemented with additional data, as described in Table 3.2, were used to derive the design profiles for each geotechnical soil unit. s_u from q_c was also used to inform the design profiles. The geotechnical soil properties at the DODC-1 and DODC-2 locations were considered to be similar. Therefore, a single combined LE, BE and HE design profile applicable at both the DODC-1 and DODC-2 locations was derived based on a review of current and historical data at the domino infield area. These design soil profiles are presented on Plates 21 to 24.

A LE design line of the s_u data was derived for mudmat stability and suction pile vertical bearing capacity analyses, considering the inherent variability in the data. A HE design line of the s_u data was derived for mudmat and suction pile installation analyses. Plate 21 and 22 presents the s_u data and the derived LE, BE and HE design lines for the DODC-1 location on different scales. Plates 23 and 24 present the s_u data and the derived LE, BE and HE design lines for the DODC-2 location on different scales. Plates 21 to 24 contain the unfactored s_u design soil parameters and do not include any reductions due to shallow gas effects. For the preliminary engineering analyses, a cautionary 30 % reduction was applied to the LE undrained shear strength parameters presented on Plates 21 to 24 (Section 5.2.3) to account for biogenic gas dissociation. However, detailed analyses should be performed during detailed design to appropriately quantify the effects of shallow gas and gas hydrates on the soil strength.

LE s_u design soil profiles in Geotechnical Soil Units 1 and 2 were based on trends inferred from CPT data and HE design lines are based on the LV and fallcone data. Due to the limited amount of test data in Geotechnical Soil Units 1 and 2, additional data as summarised in Table 3.2 was used to profile the design bounds.

For Geotechnical Soil Unit 5, the s_u was based on the consolidated laboratory test data. The LE profile derived reflects the approximate lower limit of the DSS and UU testing. The HE profiles derived is expected to be representative of the increase in strength which may occur due to increases in shear-induced dilation as observed from CIU tests. The spikes in the CPT data within this unit is reflective of the sand and silt beds, and therefore the design profiles within this unit are biased towards the consolidated laboratory test data. Design profiling within this unit should be carefully reviewed based on the objective of the engineering analyses for which they are being considered.

4.4.2 Undisturbed Undrained Shear Strength from Laboratory Data

The s_u data were obtained from LV and unconsolidated undrained (UU) triaxial tests performed in the offshore laboratory. DSS test data, and UU triaxial test data from the onshore laboratory testing were also used in determining s_u in clay soil units.

The TV, LV and DSS test results at the DODC-2 location between 3.75 m and 5.50 m BML (Geotechnical Soil Unit 2) were all higher than the CPT data. This is interpreted to be a rafted block of overconsolidated material (see Section 3.4 and Figure 3.5).

Index strength test data from pocket penetrometer (PP) and torvane (TV) data were not considered representative of the soil strength within Geotechnical Soil Unit 5 due to the influence of soil structure on the measurements from these tests. A generally lower s_u was obtained from index tests (PP and TV) than from DSS, UU and CPT (Section 3.3.4). This is interpreted to be due to the PP and TV prematurely failing at lower shear strengths along the fissures and in sand and silt partings within in the sediments. Additionally, expansion of the sample due to gases in the soil in Geotechnical Soil Unit 5 may have affected the results. Therefore, PP, TV and LV measurements were not considered in deriving the s_u design profiles provided in Geotechnical Soil Unit 5.

4.4.3 Undisturbed Undrained Shear Strength from Cone Penetration Test Data

Undrained shear strength (s_u) was measured directly from laboratory testing and was also inferred from CPT data using Equation 4.4:

$$s_u = q_n / N_{kt}$$

Equation 4.4

Where:
q_n= Net cone resistance [kPa]
N_{kt}= Empirical factor relating cone resistance to undrained shear strength

N_{kt} factors of 17 to 25 were used to derive characteristic LE and HE s_u values from q_c data for input to capacity and installation analyses. The N_{kt} factors used are based on an N_{kt} assessment undertaken by Fugro (2016a). Further review of the N_{kt} factors and detailed calibration of these values should be considered in the sandy and fissured clays (Geotechnical Units 3 to 5) as part of detailed foundation design.

4.4.4 Remoulded Undrained Shear Strength

The remoulded undrained shear strength (s_{ur}) was measured using remoulded LV (LVr) and remoulded UU (UUr) test results. Plate 25 presents the LE, BE and HE s_{ur} for the DODC-1 location. Plate 26 presents the LE, BE and HE s_{ur} for the DODC-2 location.

The residual LV test is prepared using the vane to remould the soil after the undisturbed test, as outlined in ASTM D4648 (1982). The LVr test consists of removal of the soil from the sample tube, physically remoulding the soil with a spatula, replacing the remoulded soil into a suitable container and testing as outlined in ExxonMobil G004 (2015).

Values of s_{ur} were also calculated from I_L according to Wroth (1979). Equation 4.5 describes the calculation of s_{ur} from I_L.

$$s_{ur} = 1.7\left[10^{2(1-I_L)}\right]$$

Equation 4.5

Values of s_{ur} were also determined from CPT f_s. According to Lunne et al., (1997) f_s from an electric cone is approximately equal to the s_{ur}. In this report s_{ur} for design profiling is cautiously considered to be inferred from two thirds of f_s based on Fugro experience.

A large degree of variability is observed between the LVr, UUr and f_s datasets. Generally, the results from LVr are low relative to the other datasets. This is interpreted to be due to the LVr susceptibility to moisture content redistribution along the failure plane. Further, this failure mechanism is considered less representative of the failure modes expected for the foundations considered in this report. Therefore, the LE $s_{ur}$was derived based on the higher LVr values. The f_s dataset is considered a better indicator of s_{ur} for the purposes of shallow foundation design, due to the mechanisms related to remoulding soil around the cone sleeve being broadly similar to remoulding of soil around a foundation skirt or pile shaft. The BE s_{ur} was therefore bias towards the f_s derived values for DODC-1. However, at DODC-2, in Geotechnical Soil Unit 5, the BE s_{ur} was derived based on engineering judgement to give due consideration to the higher sand and silt content. Generally, the UUr tests plot towards the upper bound of the dataset and therefore were typically used to define the HE design lines.

4.4.5 Strength Sensitivity

The strength sensitivity (S_t) is calculated from the ratio of s_u to s_{ur}. The S_t was assessed based on undisturbed and remoulded LV, UU and fall cone test results. Equation 4.6 was used to derive S_t from CPT data based on the recommendations of Schmertmann (1978).

$$S_t = {}^{N_s}\!/_{R_f}$$

Equation 4.6

Where:
N_s = Factor relating S_t to R_f
R_f = Measured friction ratio as determined from q_c and f_s

At the DODC-1 and DODC-2 locations N_s factors of 3.5 and 9 were considered for derivation of S_t. N_s factors considered are determined following a correlation of S_t data from laboratory tests data and CPT sleeve friction.

Plates 27 and 28 present the strength sensitivity data and LE, BE and HE design profiles for the DODC-1 and DODC-2 locations, respectively. These design profiles were derived based on engineering judgement. The S_t profile derived from CPT data generally correlates with the lower and upper bounds inferred from laboratory test data. The LE design lines were generally based on the UU data. The high estimate design lines were tentatively based on the LV data noting that some LV values were ignored based on the CPT based correlation and the knowledge that preferential failure can occur in fissured soils or soils with silt and sand laminae and pockets. The BE S_t profile for DODC-2 considered the higher s_{ur} from cone sleeve friction data due to silt and sand laminae.

The high S_t values in Geotechnical Units 1 and 2 are consistent with expectations due to the high liquidity indices.

BE S_t values were used in combination with LE $s_u$values for suction pile vertical bearing capacity analysis.

4.5 In Situ Stresses and Stress History

4.5.1 General

This section presents the inferred stress history parameters for the Domino drill center locations. The following one-dimensional consolidation laboratory tests were performed to determine the stress history at the Domono Drill Center locations:

i. Constant rate of strain (CRS) consolidation tests;
ii. Incremental oedometer consolidation tests results.

Results of the consolidation tests are presented in the laboratory and in situ testing data report (Fugro, 2018a) and were used to determine the stress history parameters.

4.5.2 Overconsolidation Ratio

Overconsolidation ratio (OCR) was derived from the preconsolidation pressure (p'_c) or maximum additional overburden pressure ($\Delta p'$) and estimated effective overburden pressure (p'_0). Equation 4.7 describes the calculation of OCR.

$$OCR = \frac{p'_c}{p'_0} = \frac{(p'_0 + \Delta p)}{p'_0}$$

Equation 4.7

The one-dimensional consolidation test data were used to derive p'_c based on the Casagrande (1936) method. Values of p'_0 were determined from the BE submerged unit weight assuming fully saturated soils and hydrostatic soil conditions.

In addition to the oedometer test data, OCR was also indirectly assessed from p'_0 and s_u determined from UU, CAU and DSS tests. Equation 3.4 describes the relationship used to estimate OCR, from s_u and p'_0 (Mayne, 1980):

$$OCR = \left(\frac{\left({}^{s_u}/_{p'_0}\right)_{oc}}{\left({}^{s_u}/_{p'_0}\right)_{nc}}\right)^{1/\lambda_0}$$

Equation 4.8

Where:

s_u = Undrained shear strength [kPa]
p'_0 = Effective overburden pressure [kPa]
$(s_u/p'_0)_{OC}$ = Ratio for overconsolidated soil
$(s_u/p'_0)_{NC}$ = Ratio for normally consolidated soil (taken as ~0.25)
λ_0 = 0.85

OCR was also inferred from CPT data using the method outlined by Powell et al. (1988) where the shape of the normalised cone resistance (Q_t) profile is taken into account. Equation 4.9 and Equation 4.10 describe the calculation of OCR according to Powell et al. (1988).

$$OCR = Q_t \times k$$

Equation 4.9

Where:
k = An empirical constant [~ 0.22]
Q_t = Normalised cone resistance [MPa]

$$Q_t = \left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}}\right)$$

Equation 4.10

Where:
q_t = Total cone resistance [MPa]
σ_{v0} = Total overburden pressure [kPa]
σ'_{v0} = Effective overburden pressure [kPa]

Plates 29 and 30 present the measured and derived apparent OCR for the DODC-1 and DODC-2 locations, respectively. The BE design line was primarily derived based on incremental and CRS consolidation test data and is broadly consistent with the interpreted geological stress history. The BE apparent OCR was considered in suction pile vertical bearing capacity analyses. It should be noted that the CRS data may be subject to some uncertainty depending on the rate dependency characteristics of the soil. That is, the OCR predicted from CRS test data increases with increasing strain rate (Sheahan et al., 1996). However, the preconsolidation stresses determined from the CRS and incremental oedometer tests are generally consistent and were therefore considered appropriate for determining a design OCR profile. A detailed review and comparison between the CRS and incremental oedometer data is recommended during detailed design to assess the consolidation rate dependency of the soil.

Geotechnical Soil Units 1 and 2 can be assessed to have an OCR of 1 for the purposes of geological stress history although their compression and shear characteristic would be indicative of under consolidated soils.

4.6 Chemical Composition

The chemical composition and salinity content tests were generally performed in accordance with the procedures presented in BS 1377. Table 4.2 to Table 4.6 summarise the chemical content results per geotechnical soil unit.

The testing performed at the Domino drill centers supplements the chemical testing performed on samples obtained in the vicinity of the planned Domino drill centers (Fugro, 2015b) and discussed by Fugro (2016a).

The results of the chemical composition testing are briefly discussed in Sections 4.6.1 to 4.6.5 of this report, where these tests can be used to further update the geological model for the site, they will be discussed in the updated integrated report for the site (Fugro, 2018c: *in press*). The observations that can be made in the data verify the geological model. The observed changes in chemistry are interpreted to have been caused by the transition from freshwater to marine environments and agree with the geological model for the Neptun block.

Table 4.2: Carbonate Content Test Results

Geotechnical Soil Unit	Carbonate Content [%]	Number of Tests
1	-	-
2	5.7 to 34.1	3
3	5.2 to 16.4	7
4	6.4	2
5	4.1 to 10.2	15
A	5.7 to 12.5	2

Table 4.3: Organic Content Test Results

Geotechnical Soil Unit	Organic Content [%]	Number of Tests
1	2.2	1
2	2.3 to 11.0	3
3	0.7 to 2.0	6
4	0.8 to 1.1	2
5	0.2 to 2.7	14
A	1.5 to 2.0	2

Table 4.4: Water Soluble Chloride Content

Geotechnical Soil Unit	Water Soluble Chloride Content [mg/l]	Number of Tests
1	-	-
2	3000 to 12000	3
3	730 to 4600	7
4	560 to 1100	2
5	54 to 2100	15
A	2700 to 3600	2

Table 4.5: Water Soluble Sulphate Content Test Results

Geotechnical Soil Unit	Water Soluble Sulphate Content [mg/l]	Number of Tests
1	-	-
2	450 to 750	3
3	46 to 2800	7
4	58 to 59	2
5	29 to 160	15
A	750 to 1200	2

Table 4.6: pH Test Results

Geotechnical Soil Unit	pH [-]	Number of Tests
1	-	-
2	7.8 to 8.6	3
3	6.9 to 8.4	7
4	8.0	2
5	7.8 to 8.3	15
A	8.5 to 8.6	2

4.6.1 Carbonate Content

Plate 31 and Table 4.2 presents the composite carbonate content versus depth plot for all geotechnical units. Carbonate content tests were performed with the results expressed as a percentage by mass of carbonate (CO_3). The results range from 4.1 % to 34.1 %. A higher carbonate content is observed within Geotechnical Soil Unit 2 and this is interpreted to be due to the higher percentage of coccoliths as a result of marine conditions.

4.6.2 Organic Content

Plate 32 and Table 4.3 present the results of total organic content testing. Total organic content ranges from 0.2 % to 11.0 %. Based on the BS 5930 (2015) soil classification, the measured range indicates that the samples tested are inorganic (< 2 %) to organic (6 % to 20 %) The inorganic nature of most of the sediments suggests that they were deposited in an oxygenated shallow water environment without the stratification that is now present in the Black Sea. Geotechnical Unit 2 is an exception to this with higher levels of organic material (between 2.3 % and 11.0 %). This occurred as organic material did not breakdown during the mixing of saline waters and freshwater during the deposition of organic-rich sediments.

4.6.3 Chloride Content

Plate 33 and Table 4.4 present the composite plot of water soluble chloride content. Chloride content decreases with depth: this is due to the transition from marine sediments to lacustrine sediments. Geotechnical Soil Unit 2 has chloride content between 3000 mg/l and 12000 mg/l while the under lying sediments do not exceed 4600 mg/l.

This trend is consistent with the sediments of Geotechnical Soil Unit 2 deposited during a period of transition following the breach of the Bosporus and the migration of chloride-rich pore-water through the sediment column over the past 8200 years (Riboulot et al., 2018).

4.6.4 Sulphate Content and pH

Plate 34 and Table 4.5 summarise the sulphate content. The same trend noted in Chloride testing is observable in the sulphate content results. Geotechnical Soil Unit 2 show elevated sulphate content between 450 to mg/l and 750 mg/l; this drops in the underlying Geotechnical Soil Units 3, 4 and 5 sediments consistent with the change in pore-water chemistry associated with the breach of the Bosporus. Sulphate content value associated with DD2-BH-02 at 1.75 m should be viewed with caution as it exceeds the expected range within Geotechnical Soil Unit 2.

Plate 35 and Table 4.6 summarise the pH for the Eastern Fault and Central Fault Locations. The pH ranges between 6.90 to 8.60 across all of the samples; this is consistent with samples deposited in in a freshwater to marine environment.

4.6.5 Headspace Gas

Headspace gas analysis was carried out on three samples for the Domino drill. Carbon Isotope testing to identify the origin of the gas in the Domino Drill Centers was not completed for this phase of work. Table 4.7 summarises the headspace gas analysis and carbon isotope analysis test results. Plate 36 presents the headspace gas results versus depth.

Table 4.7: Headspace Gas Analysis and Carbon Isotope Analysis Test Results at DODC-1 and 2

Geotechnical Soil Unit	Headspace Gas Analysis (Methane C1) [ppm]	
	Result Range	Number of Tests
5	3364 to 23696	3

The headspace gas values show that methane (C1) is present in Geotechnical Soil Unit 5. A full headspace sampling and testing programme was not completed for the Domino drill centers. Previous analysis (Fugro 2015b) suggests that biogenic methane is present within the sediments, but the Domino drill centers are located within the hydrate stability zone therefore free gas is not expected. Headspace gas specimens were collected on samples where high volumes of gas were detected using a portable gas detector, and where there was significant sample expansion.

The results show that elevated levels of methane are present within Geotechnical Soil Unit 5; these values are similar to those measured in previous studies (Fugro, 2015b). It is interpreted that the methane present within the deepwater area of the site is in the form of methane hydrate. Although the methane is stable as a hydrate at the current temperatures and pressures it may disassociate and become free gas if the local temperature and pressure conditions change during the structure design life (e.g. by well temperature influences, cyclic load-induced stress changes, suction pile installation under pressure). The highest concentrations in Unit 5 are interpreted to be the result of gas hydrate accumulating in sand and silt layers within the predominantly clay unit.

5. PRELIMINARY ENGINEERING ANALYSES

5.1 General

This section summarises the mudmat stability and suction pile vertical bearing capacity analyses, and foundation installation analyses for the DODC-1 and DODC-2 locations.

Section 4 presents the derivation of the design soil parameters used in the engineering analyses. Plate 37 presents the tabulated design soil parameters for mudmat and suction pile foundation analyses for the DODC-1 location. Plate 38 presents the tabulated design soil parameters for mudmat and suction pile foundation analyses for the DODC-2 location.

5.2 Foundation Design Risks

5.2.1 General

The following risks are identified at the Domino drill center locations, which may have an impact on the foundation design analysis:

i. Coccolith ooze and sapropel formations;
ii. Gas hydates and gas hydrate dissociation;
iii. Buried mass transport deposits.

Sections 5.2.2 and 5.2.3 discuss the identified risks and associated mitigation measures applied in the foundation design analysis.

5.2.2 Coccolith Ooze and Sapropel Formations

5.2.2.1 General

Geotechnical Soil Unit 1, coccolith ooze formation, consists of extremely low strength clay and is observed from mudline to 0.5 m BML at DODC-1 and DODC-2 locations.

Geotechnical Soil Unit 2, sapropel formation, consists of extremely low strength organic clay and is observed from 0.5 m to 2.5 m BML at both DODC-1 and DODC-2 locations.

Due to the extremely low strength of both formations observed at the top 2.5 m BML the following foundation design implications should be considered as a minimum, particularly where partial or complete removal of these formations is not considered.

5.2.2.2 High Soil Strength Sensitivity

When foundations, particularly suction piles, are installed, the soil around the skirts or pile is remoulded. The strength may then regain with time due to thixotropic effects. Due to the highly sensitive nature of these soils, the remoulded undrained shear strength is expected to be exceptionally low immediately after installation. Thixotropy testing indicates that over a period of 10 days no strength gain may be recorded (i.e. thixotropy = 1.0). Therefore, further consideration may be required regarding the time between installation of the foundation and loading of the foundation with the structure self weights and when the foundation may experience full operational loading. Depending on this planned duration for a given contractor, other (potentially more favourable) thixotropy values based on the laboratory test data may be justified which will in turn optimise the foundation size.

5.2.2.3 Seabed Mobility

Both Geotechnical Unit 1 and 2 comprise extremely low strength soils. These extremely low soil strengths will lead to susceptibility to scour (soil removal) around the installed foundation due to the combination of noted seabed currents and the likely current acceleration and vortices after foundation installation. Scour will reduce the effective foundation embedment and reduce the vertical, horizontal and moment bearing capacity.

5.2.2.4 Landing of Mudmats and Suction Piles

Landing of mudmat and suction pile foundations on these formations may induce fluidisation of soils due to their extremely low strength, high liquidity index (greater than 2.0), high strength sensitivity, and low remoulded undrained shear strength. The effect of landing of the mudmat and suction pile foundations was not analysed for this report. However, for detailed design, the landing of the mudmat and suction pile foundations will need to consider:

- Slack-line condition: water pressure increase as the foundation is lowered towards the seabed in relation to the foundation weight;
- Scour due to water escaping from beneath the foundation;
- Bearing failure due to water pressure increases loading the seabed as touchdown is approached;
- Vent-hole extrusion: where soil extruding through the holes in the mudmat baseplate during set-down.

5.2.2.5 Settlement

The short-term and long-term settlement of the mudmat and suction pile foundations on both formations must be considered in detail due to the extremely low strength, extremely high moisture contents correspondingly very high void ratios and unconsolidated in situ state. Foundations, particularly mudmats, may experience large settlement. Due to the large settlement expected, settlement analysis may govern the design of the mudmat and suction pile foundations, and should be considered in the detailed design stage.

5.2.2.6 Excessive Mudmat Skirt Lengths

Due to the extremely low strength soils within the upper 2.5 m BML mudmats may experience very low, vertical, horizontal and moment stability if skirts are embedded within this zone. Increasing the skirt lengths to ‘key in’ to more competent underlying strata (e.g. Geotechnical Unit 3), may result in impractically long mudmat skirts. Therefore, a detailed analysis is recommended during detailed design to assess if mudmats of a practical geometry are feasible also considering the recommendation in Section 5.2.2.4 and 5.2.2.5 above.

5.2.3 Gas Hydrates and Gas Hydrate Dissociation

Free gas was not detected or observed at the Domino drill center locations but biogenic methane is present within the sediments (See Section 4.6.5) and noted in the borehole logs at both Domino drill center locations (Fugro, 2018a). The biogenic methane observed during borehole logging is interpreted to be the presence and dissociation of methane hydrates that are stable at the in situ temperatures and pressures. During sampling and the return of the samples to the vessel, the pressure and temperature change is interpreted to have led to dissociation of the methane hydrates. This caused expansion of ‘gassy’ samples (Fugro, 2018a). If the in situ pressure and/ or temperatures were to change over the operational lifetime of the structure (e.g. suction installation, foundation stressing the seafloor, compressive and tensile loads), the biogenic methane may dissociate and become free gas. Free gas may lead to:

i. Increased pore pressure in low permeability soils and increase moisture content as gas hydrates dissociate into free gas and water. In turn, these effects may significantly reduce the strength and stiffness of the soil and therefore reduce the vertical, horizontal and moment resistance offset by the soil;
ii. Acceleration of foundation corrosion due to changes in the pore-water chemistry by increasing potentially corrosive chemicals such as, sulphates and chlorides or changing the pH of the pore water;
iii. Accumulation of methane gas in foundation voids (e.g. suction pile head) during suction installation.

The dissociated free gas may migrate into and around some foundations where it could accumulate or reform into hydrates. The presence of stable methane hydrates may also lead to significant increases in soil strength.

In the preliminary mudmat and suction pile analyses presented in this report a cautionary 30 % reduction to the low estimate undrained shear strength presented on Plates 21 to 24 was applied to account for dissociation of the biogenic methane which can significantly reduce sediment strength and stiffness. The 30 % reduction is expected to be cautious, in liew of a detailed risk assessment and geotechnical impact appraisal, and based on Fugro’s experience of dissociation impacts in similar soils.

A larger reduction of 30 % due to dissociation of gas hydrates is expected at the DODC locations relative to the 10 % reduction applied to the shaft friction at the platform location. This is due to free gas interpreted as existing at the platform location and stable hydrates being interpreted to be present at the DODC locations. Free gas is expected to use sand and silt as migration pathways and potentially reduce undrained shear strength less than dissociated gas hydrates.

The presence of stable gas hydrates existing within a soil may lead to significant increases in the soil strength. The stable gas hydrates may dissociate due to ground disturbance, pressure changes or temperature changes resulting in a significant decrease in the soil strength. The reduction in sediment strength will lower the suction pile capacity and mudmat stability and a reduction in sediment stiffness will increase the pile displacement under load and settlement under structure self-weight and increase the settlement of mudmats. A detailed review regarding the effects of methane hydrate gas dissociation on the soil geotechnical properties is recommended during detailed design and is expected to allow design refinement.

5.2.4 Seismicity

Seismic stability, post-seismic stability and post-seismic settlement checks may be required for the Domino drill centre locations using site-specific probabilistic seismic hazard analysis (PSHA) and site response analysis (SRA).

Formal unity checks on seismic stability may result in excessive foundation dimensions. An alternative approach considering evaluation of foundation displacements under seismic loading and the associated impact on structure operability may often lead to a reduced foundation size relative to unity checks.

5.3 Structures

5.3.1 General

Fugro understand the following structures are to be installed in the Domino Infield Areas:

i. ITA at the DODC-1 location;
ii. FLET and pig launcher at the DODC-2 location;
iii. Manifolds at both DODC-1 and DODC-2 locations.

Fugro understands that a mudmat or a mudmat seated on a single suction pile foundation are under consideration for each of the Domino FLET and ITA structures and that a single suction pile is proposed for each manifold.

The ITA and FLET structures are understood to be designed to move axially and laterally on the mudmat foundation to accommodate pipeline movements.

5.3.2 Initial Mudmat Geometry

An initial mudmat geometry was provided by ExxonMobil (2017) for each structure. Table 5.1 summarises the mudmat initial geometry of the FLET and ITA structures.

Table 5.1: Initial Mudmat Geometries

Structure	Length [m]	Breadth [m]
Flow line end termination (FLET)	22.0	12.0
In-line tee tie-in assembly (ITA)	20.0	12.0

The mudmat was assumed to have a 10 mm thickness perimeter skirt plus two internal skirts in each direction of the same thickness. Figure 5.1 shows the idealised mudmat foundation base plan.

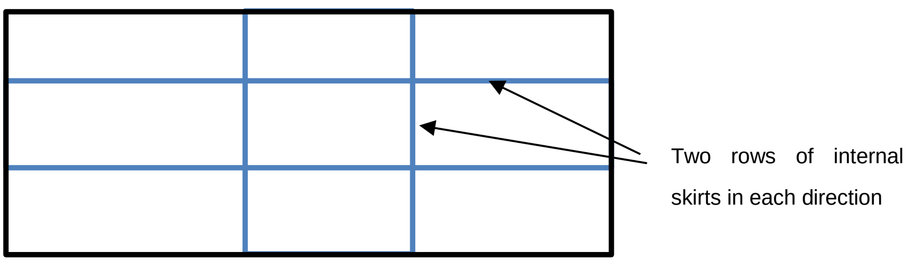

Figure 5.1: Indicative Mudmat Foundation Base Plan

Where required, wall thickness was reduced to 8 mm and the number of internal skirts were reduced to 1 row in each direction to achieve full skirt penetration. Section 5.5.3 describes these instances.

5.3.3 Initial Suction Pile Geometry

An initial suction pile geometry was as provided by ExxonMobil (2017). The base case suction pile modelled was 6.0 m outer diameter with 25 mm wall thickness. The required length of the suction pile based on the assumed loads was determined in the analyses. The top cap of the suction pile was also assumed to have the same thickness for this analysis, though Fugro appreciate that, dependent on the assigned contractor, this may differ appreciably compared the value assumed.

No internal skirts or stiffeners were assumed.

5.4 Preliminary Load Data

5.4.1 General

Suction pile analyses for the manifold structures considered vertical load of the manifold and assumed self weight of the suction pile dependent on determined penetration depth of the suction pile. Horizontal loads were assumed equal to 20% of the manifold self-weight.

Mudmat analyses considered vertical, horizontal, moment and torsion (VHMT) loading. The load data applied in the engineering analyses was provided by ExxonMobil (2017). The load data is based on a pipe stress analysis reaction loads which includes the submerged weight of the considered structures. ExxonMobil (2017) provides further details of the specific load components included in the design loads considered in this report.

A working stress design (WSD) method was applied where a global safety factor of 2 was assumed and neither partial material nor load safety factors were applied.

No seafloor slope was assumed for all structures.

5.4.2 Mudmat Loads

5.4.2.1 <u>FLET Load Components</u>

The following load components were considered for the FLET preliminary design load cases, understood to be consistent with data provided by ExxonMobil (2017).:

i. FLET mudmat self-weight (load varied based on size), applied in the mudmat geometric centre at skirt tip level;
ii. Combined self-weight of the FLET and pig launcher (308 kN + 107 kN = 415 kN), applied at a 1.5 m lateral offset (parallel to mudmat width) and either a 3.0 m or 5.5 m axial offset (parallel to mudmat length) from the mudmat geometric center. Exxon Mobil (2017) also states that pipe support forces are included in the loads adopted in this report.

Design load cases were derived by combining individual load components each with a given magnitude, direction and coordinate of application relative to the geometric centre of the mudmat at skit tip level. Each load component is then resolved to the geometric centre of the foundation at either skirt tip level.

When a larger mudmat that that specified in Table 5.1 was analysed, the mudmat self-weight was estimated based on an additional 1.4 kN/m^2, plus the associated additional skirt self-weight based on that assumptions in Section 5.3.2 and a 67 kN/m^3 submerged steel self-weight. Table 5.2 summarises the resolved static preliminary base case design loads to be adopted in the FLET.

Table 5.2: Preliminary Design Loads for FLET and Pig Launcher Mudmat Analyses

Load Case Description	Mudmat Geometry	Unfactored Global Foundation Loads					
		V [kN]	H_x [kN]	H_y [kN]	M_x [kNm]	M_y [kNm]	T [kNm]
Load case with extreme offsets of 3 m (X) and 1.5 m (Y)	22.0 m (L) × 12 m (B)	795	0	208	1245	-623	312
Load case with extreme offsets of 5.5 m (X)[a] and 1.5 m (Y)	22.0 m (L) × 12 m (B)	795	0	208	2283	-623	312
Load case with extreme offsets of 3 m (X) and 1.5 m (Y)	29.3 m (L) × 16.0 m (B)	1163	0	208	1245	-623	312
Load case with extreme offsets of 5.5 m (X)[a] and 1.5 m (Y)	29.3 m (L) × 16.0 m (B)	1163	0	208	2283	-623	312
Load case with extreme offsets of 3 m (X) and 1.5 m (Y)[b]	19.3 m (L) × 10.5 m (B)	701	0	208	1245	-623	312
Load case with extreme offsets of 5.5 m (X)[a] and 1.5 m (Y)[b]	19.3 m (L) × 10.5 m (B)	701	0	208	2283	-623	312

Notes:
a = Includes an additional 2.5 m in the axial direction (X) to account for pipeline movements due to expansion
b = Mudmat weights where the coccolith ooze and sapropel formations (top 2.5 m BML) are fully removed
FLET = Flow Line End Termination
V, H, M, T = Vertical, horizontal, moment and torsion (refer to Figure 5.2 for sign convention)

Figure 5.2 presents the sign convention used for preliminary mudmat foundation design. All loads provided by ExxonMobil were converted into this format.

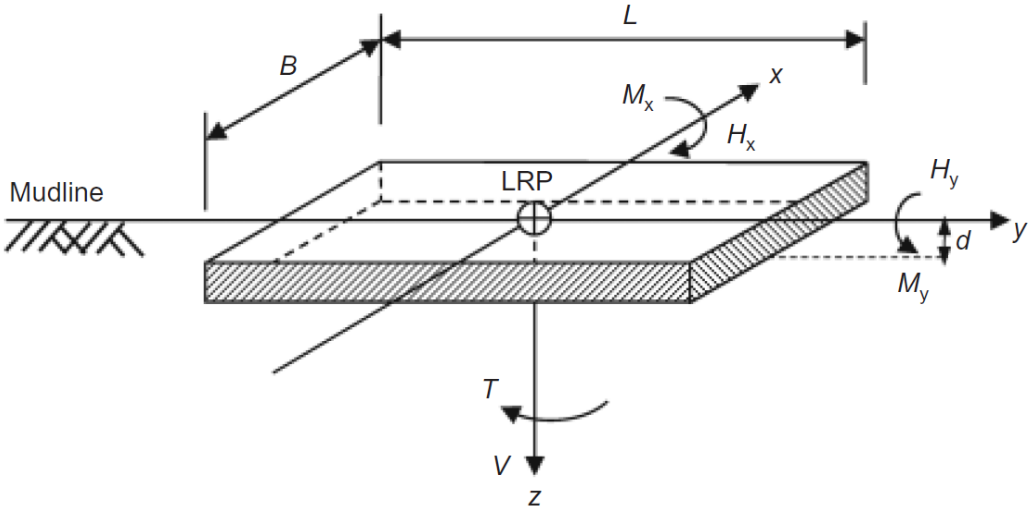

Figure 5.2: Sign convention used for foundation analyses (Feng et al. 2014)

5.4.2.2 <u>ITA Load Components</u>

The following load components were considered for the ITA preliminary design load cases, understood to be consistent with data provided by ExxonMobil (2017):

i. ITA mudmat self-weight (load varied based on size), applied in the mudmat geometric centre at skirt tip level;
ii. Combined self-weight of the ITA (177 kN), applied at a 1.5 m lateral offset (parallel to mudmat width) and either a 3.0 m or 5.5 m axial offset (parallel to mudmat length) from the mudmat geometric center. Exxon Mobil (2017) also states that pipe support forces are included in the loads adopted in this report.

The design load cases were derived as described in Section 5.4.2.1; However, each additional mudmat m^2 was assumed to be 1.4 kN/m^2, reflecting the different mudmat self-weight and geometry. Table 5.3 summarises the static preliminary design loads to be adopted in the FLET and ITA structures.

Table 5.3: Preliminary Design Loads for ITA Mudmat Analyses

Load Case Description	Mudmat Geometry	Unfactored Global Foundation Loads					
		V [kN]	H_x [kN]	H_y [kN]	M_x [kNm]	M_y [kNm]	T [kNm]
Load case with extreme offsets of 3 m (X) and 1.5 m (Y)	20.0 m (L) × 12 m (B)	526	0	88	528	-264	132
Load case with extreme offsets of 5.5 m (X)[a] and 1.5 m (Y)	20.0 m (L) × 12 m (B)	526	0	88	968	-264	132
Load case with extreme offsets of 3 m (X) and 1.5 m (Y)[b]	12.5 m (L) × 7.5 m (B)	308	0	88	528	-264	132
Load case with extreme offsets of 5.5 m (X)[a] and 1.5 m (Y)[b]	12.5 m (L) × 7.5 m (B)	308	0	88	968	-264	132

Notes:
a = Includes an additional 2.5 m in the axial direction (X) to account for pipeline movements due to expansion
b = Mudmat weights where the coccolith ooze and sapropel formations (top 2.5 m BML) are fully removed
ITA = In-line Tee Assembly
V, H, M, T = Vertical, horizontal, moment and torsion (refer to Figure 5.2 for sign convention)

5.4.2.3 Manifold Load Components

The following load components were considered for the ITA preliminary design load cases, understood to be consistent with data provided by ExxonMobil (2017):

i. Suction pile self-weight, based on an assumed 25 mm wall thickness and top plate thickness and a 67 kN/m^3 submerged steel self-weight. The self-weight was varied with suction pile penetration and applied at the geometric centre of the suction pile head, assumed to be at seafloor;
ii. Manifold self-weight (981 kN), applied at the geometric centre of the suction pile head, assumed to be at seafloor;
iii. Assumed horizontal load of 196 kN (20% of 981 kN), applied at the geometric centre of the suction pile head, assumed to be at seafloor.

5.5 Preliminary Mudmat Analyses

5.5.1 General

Preliminary foundation design analyses were performed for the ITA and FLET structures at the Domino drill center locations according to Feng et al. (2014) recommendations. The preliminary mudmat analyses presented in the report provide a cautious upper bound of foundation geometries. Table 5.2 and Table 5.3 summarises the preliminary design loads used in the mudmat analyses.

5.5.2 Foundation Vertical Horizontal Moment Torsion Stability

5.5.2.1 General

The undrained stability of the mudmat foundations was analysed considering multidirectional foundation loading. Multidirectional foundation loading comprises vertical dead loads (V), biaxial live horizontal loads (Hx and Hy), moments (My and Mx), and torque loading (T) collectively referred to as V-H^2-M^2-T or simply VHMT loading.

The VHMT stability analyses considered the LE s_u profiles shown on Plates 21 and 22. A further 30 % reduction was applied to the LE s_u profiles to allow for a potential reduction in sediment strength due to dissociation of biogenic methane.

The analyses considered the VHMT method described by Feng et al. (2014). The method involves defining the ultimate limit state of mudmats under combined multidirectional foundation loading through a nest of two-dimensional failure envelopes.

Plates 37 and 38 present the design soil parameters used in the mudmat analyses. The derivation of the design soil parameters is described in Sections 4. It should be noted that the top 2.5 m BML consists of the extremely weak Coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2). The mudmat capacity analyses were performed assuming that the mudmat will be landed on top of Geotechnical Soil Unit 1. Therefore, to perform the mudmat stability analyses and determine the embedment estimate the preliminary geometry, a linear undrained shear strength profile based on the strengths of Geotechnical Soil Units 1 and 2 was used. The linear profile is applied conservatively to account for the extremely weak strength of Geotechnical Soil Units 1 and 2 and may result in larger mudmat geometries and longer skirt penetrations being predicted.

Where the specified mudmat sizes could not support the applied vertical loads, iterative mudmat sizing analyses were performed to determine mudmat size and skirt penetration depth required to support the loads. The sizing of the mudmat was confirmed by ensuring the preliminary design loads (see Section 5.4) remained within the VHMT capacity envelopes with a factor of safety of greater than or equal to 2.0.

A mudmat stability analyses was also performed considering a scenario where the extremely low strength highly sensitive Geotechnical Soil Units 1 and 2 were removed and the mudmat was installed at the top of Geotechnical Soil Unit 3.

5.5.2.2 Method

The Feng et al. (2014) failure envelope method firstly entails the calculation of the uniaxial capacities; namely, those for which a single component of load or moment acts on the foundation in isolation. The available maximum horizontal, moment and torsional capacities were then examined according to the mobilised vertical resistance, involving identification of the V-H_x, V-H_y, V-M_x, V-M_y and V-T interaction diagrams. Subsequent analyses are performed to investigate the interaction diagrams under Hx-Hy and Mx-My loading, allowing estimation of the maximum horizontal and moment capacity for any angle of horizontal and moment loading, respectively. These maximum values are then reduced according to the mobilised torsion by considering the interaction diagram for H-T and M-T loading. Ultimately, a general formulation of a failure envelope is developed for the VHMT loading.

5.5.2.3 Results

Table 5.4 summarises the results of the mudmat foundation analyses for the DODC-1 and DODC-2 locations.

Table 5.4: Mudmat Foundation Analyses Results

Domino Drill Center Location	Structure	Length [m]	Breadth [m]	Skirt Penetration Depth [m BML]	FOS [-]
DODC-1	ITA	20.0	12.0	1.0	< 2.0
				1.2	> 2.0
		12.5[a]	7.5[a]	0.1	> 2.0
DODC-2	FLET	22.0	12.0	1.0	< 2.0
				1.7	> 2.0
		29.3[b]	16.0[b]	1.0	> 2.0
		19.3[a]	10.5[a]	0.2	> 2.0

Notes:
a = Mudmat geometries where the coccolith ooze and sapropel formations (top 2.5 m BML) are fully removed
b = This mudmat geometry is considered impractically large
DODC-1 = Domino Drill Center 1
DODC-2 = Domino Drill Center 2
ITA = In-line Tee Assembly
FLET = Flow Line End Termination
FOS = Global factor of safety of 2.0

Results of the mudmat foundation analysis shows that:

i. At the DODC-1 location, a 20.0 m by 12.0 m ITA mudmat with a skirt height of 1.2 m is required to support the applied loads;
ii. At the DODC-2 location, a 22.0 m by 12.0 m FLET mudmat with a skirt height of 1.7 m is required. However, should the skirt height be reduced to 1.0 m a FLET mudmat of 29.3 m by 16.0 m would be required;

Given the impractically large mudmats that may be required to support the structures due to the extremely low strength of the highly sensitive coccolith ooze and sapropel formations, removal of these formations should be considered as one of the mitigation measures.

If the coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2) were removed, a 12.5 m by 7.5 m ITA mudmat with a skirt height of 0.1 m at the DODC-1 location and a 19.3 m by 10.5 m FLET mudmat with a skirt height of 0.2 m at the DODC-2 location would be required to support the applied loads.

5.5.3 Mudmat Skirt Penetration Resistance

5.5.3.1 General

Skirt penetration analyses were performed to ensure full embedment of the mudmat skirts, and therefore full contact between the mudmat plate base and seafloor surface.

HE design soil parameters were used in mudmat installation analyses.

The analyses considered all components expected to penetrate the seafloor, including internal and perimeter skirts assumed and described in Section 5.3.2.

Skirt penetration resistance was calculated for the self-weight of the mudmat for the given size analysed (Section 5.4.2) in general accordance with following analysis methods:

i. API RP 2GEO (API, 2011);
ii. DNV GL-RP-C212 (DNV GL, 2017b).

In general, the resistance to skirt penetration was calculated from static bearing capacity theory. The penetration resistance (Q) at a given skirt penetration is the sum of the skin friction (Q_s) and the end bearing resistance (Q_p). Equation 5.1 describes the calculation of Q.

$$Q = \sum_{i=1}^{n} f_i A_{si} + \sum_{i=1}^{n} q_i A_{pi}$$

Equation 5.1

Where:

n = Total number of skirt elements
i = ith skirt element
f_i = Average unit skin friction
A_{si} = Embedded surface area
q_i = Unit end bearing
A_{pi} = Gross end area

The gross end bearing area considered in the installation analysis is the skirt tip end bearing area. The skirt tip end bearing area was determined by considering the equivalent skirt perimeter taking into account two rows of internal skirts in each direction (Figure 5.1). Equation 5.2 describes the calculation of skirt end bearing area from the equivalent skirt perimeter.

$$End\ Bearing\ Area = Equivalent\ skirt\ perimeter \times skirt\ thickness$$

Equation 5.2

5.5.3.2 API RP 2GEO (API, 2011)

Plates 37 and 38 presents the design soil parameters used in determining the unit end bearing and unit skin friction.

The unit frictional resistance was calculated by multiplying the HE undrained shear strength by a skin friction factor α. The α value was calculated according to API (2011) recommendations.

The unit end bearing resistance was calculated by multiplying the HE undrained shear strength by a bearing capacity factor of 9.

5.5.3.3 DNV GL-RP-C212 (DNV GL, 2017b)

Unit skin friction and unit end bearing values were calculated according to DNV GL (2017b) recommendations. Empirical coefficient (k_p) relating q_c to end bearing and empirical coefficient (k_f) relating q_c to skin friction were applied according to DNV GL (2017b) recommendations. Most probable and highest expected penetration resistances were determined, each adopting different k_p and k_f coefficients. The empirical coefficients were reduced by 50 % for the top 1.5 m BML following the recommendations of DNV GL (2017b) to account for minor lateral movement and piping during lowering, which is considered cautious for these very low strength high liquidity index soils. Further reduction may be anticipated depending on the outcome of detailed landing analyses.

5.5.3.4 Results

Table 5.5 summarises the results of the foundation skirt penetration assessment.

Table 5.5: Mudmat Skirt Installation Assessment Results

Structure	Simplified dimension for Analyses				Total Vertical Load [kN]	HE Soil Resistance at Full Penetration		
	Equivalent Skirt Perimeter [m]	Skirt Length [m]	Skirt Thickness [m]	Skirt End Bearing Area [m^2]		API (2011) [kN]	DNV GL (2017b) MP [kN]	DNV GL (2017b) UB [kN]
ITA Mudmat	128.0	1.2	0.01	1.3	427.0	359.9 (pass)	68.7 (pass)	113.7 (pass)
	80.0[a]	0.1	0.01	0.8	132	14.1[a] (pass)	23.4[a] (pass)	78.9[a] (pass)
FLET Mudmat	102.0	1.7	0.01	0.8	507.0	488.6[b] (pass)	307.1[b] (pass)	509.2[b] (fail)
	181.3	1.0	0.01	2.3	748.0	405.8 (pass)	228.4 (pass)	377.7 (pass)
	119.0[a]	0.2	0.01	1.5	286.0	187.1[a] (pass)	111.3[a] (pass)	184.1[a] (pass)

Notes:
Skirt end bearing area equates to: equivalent skirt perimeter multiplied by the wall thickness
HE = High estimate
MP = Most probable
UB = Upper bound
ITA = In-line Tee Assembly
FLET = Flow Line End Termination Assembly
Pass = Mudmat can be installed using its own self weight
Fail = Mudmat cannot be installed using its own self weight
a = Mudmat geometries where the coccolith ooze and sapropel formations (top 2.5 m BML) are fully removed
b = Mudmat internal skirts reduced to 1 row in each direction to assist with installation

The installation analyses indicate that:

i. The ITA mudmat at the DODC-1 location can be installed under self-weight for the geometries and skirt penetration depths analysed;
ii. The FLET mudmat at the DODC-2 location can generally be installed under self-weight for the geometries and skirt penetration depths analysed. An exceptional case is for the 22 m by 12.0 m mudmat with a skirt of 1.7 m, the mudmat is not installable according to the upper bound DNV (2017b) recommendations.

5.5.4 Discussion of Mudmat Results

This section discusses the effects of the assumptions applied in the mudmat stability and installation analyses. These preliminary analyses adopted the following:

i. For mudmat stability analyses, low estimate s_u considering historic data, reductions in s_u due to the presence of hydrates and application of a global factor of safety of 2.0;
ii. For mudmat installation analyses, high estimate s_u considering historic data;
iii. Soil layering effects.

Stability analysis indicates that a skirt height of 1.7 m is required to maintain the ExxonMobil (2017) proposed mudmat footprint of the FLET. However, preliminary installation analysis indicates that the mudmat cannot be installed using its own self-weight based on the assumed two internal skirts and a 1.7 m skirt height. Therefore, the internal skirts were reduced to one row in each direction and the skirt wall thickness was reduced from 10 mm to 8 mm. The mudmat therefore appears be installable for two installation methods, but appears to be marginal for one method. The issue is overcome by reducing the internal skirts and wall thickness. However, this may affect the foundation stability and increase the risk of scooping failure where the failure plane transfers from the mudmat skirt tip to the mudmat base plate. Fugro recommends that skirt spacing optimisation be performed as part of the detailed design to mitigate against any stability and installation risks of the mudmat.

The FLET mudmat proposed at the DODC-2 is considered to be large based on Fugro experience. This large mudmat size is governed by the extremely low strength of Geotechnical Soil Units 1 and 2 (Coccolith ooze and Sapropel formations). Fugro recommends further review of the mudmat and seabed surface interaction be performed in the detailed design considering the potentially highly sensitive Coccolith ooze and Sapropel formations (See Section 5.2) should this formation not be excavated.

The mudmat sizings above are considered impractically large and are governed by the extremely low strength of the highly sensitive coccolith ooze and sapropel formations. In this case dredging or otherwise removal of these formations should be considered.

Detailed analyses considering, but not limited to, the following example effects are expected to provide more representative conclusions on mudmat feasibility and geometry:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Soil layering interaction effects between weaker soil layers (coccolith ooze and sapropel) and more competent layers (Geotechnical Unit 3) on stability analyses;

iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, compression parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u.

It is recommended that these effects are quantified and considered in detail during detailed design in accordance with any specific ExxonMobil design basis requirements.

5.6 Preliminary Suction Pile Analyses

5.6.1 General

Preliminary suction pile capacity analyses were performed to estimate the pile penetration required for 6.0 m outer diameter suction pile to withstand the applied preliminary design vertical loads. Additional suction pile diameters were also considered. Section 5.4.1 describes the loads used in the suction pile capacity analyses.

Vertical caisson capacity was calculated using the DNV (2017c) method. Tension loading was not considered because Fugro understands tension loading is not applicable. The suction pile analyses considered the LE s_u profiles shown on Plates 23 and 24. A further 30 % reduction was applied to the LE s_u profiles to allow for a potential reduction in sediment strength due to dissociation of biogenic methane.

5.6.2 Vertical Capacity

5.6.2.1 <u>General</u>

The vertical capacity of suction piles is calculated using Equation 5.3:

$$Q_t = Q_s + Q_b$$

Equation 5.3

Where:

Q_t = Caisson vertical capacity
Q_s = Caisson frictional capacity
Q_b = Caisson end bearing capacity

Fugro understands suction piles will be installed to support the manifolds in the DODC-1 and DODC-2 locations. The vertical load of the manifolds is 981 kN while the self-weight of the suction pile is dependent on the pile penetration depth BML. Section 5.6.2.2 summarises the calculation procedures adopted to calculate caisson vertical capacity.

5.6.2.2 <u>Method</u>

The frictional capacity of the suction piles was calculated using the method presented by DNV (2017c) for suction anchors in clay. Equation 5.4 describes the calculation of the caisson frictional capacity.

$$Q_s = A_s . f_s$$

Equation 5.4

Where:

A_s = Area of caisson shaft in contact with soil

f_s = Unit shaft friction, calculated using Equation 5.5

$$f_s = \tau_{f,cy}^{DSS}.\alpha$$

Equation 5.5

Where:

$\tau_{f,cy}^{DSS}$ = Cyclic direct simple shear (DSS) shear strength of intact clay

α = Friction factor (limited to a maximum value of 1.0)

The DNV GL (2017c) method was developed for use with suction anchors that are expected to experience significant cyclic loads over their lifetime. To account for this, the DNV GL (2017c) method uses a cyclic shear strength ($\tau_{f,cy}^{DSS}$) in its calculation procedures. This $\tau_{f,cy}^{DSS}$ is calculated by modifying s_u to account for the effects of cyclic loading, such as increased s_u due to loading rate effects and decreased s_u due to cyclic degradation.

Since the suction piles in this report are expected to experience predominantly monotonic loading from structure weights, $\tau_{f,cy}^{DSS}$ was replaced with the monotonic undrained shear strength for the calculations described in this report.

Due to the nature of suction pile installation, different friction factors (α) are stipulated by DNV GL (2017c) for different components of the caisson shaft friction.

For the soil down to the self-weight penetration depth, the value of α immediately after installation is calculated using Equation 5.6:

$$\alpha = C_t.\frac{1}{S_t}$$

Equation 5.6

Where:

C_t = Time-dependent thixotropy factor (assumed equal to 1.0, until laboratory test data are reported)

S_t = Soil sensitivity

Immediately after caisson installation, the use of Equation 5.6 results in the unit shaft friction being equal to the remoulded undrained shear strength.

Where the plasticity index (I_p) is more than 20 %, α from Equation 5.6 is limited to the following:

$$\alpha = 0.5\left(\frac{s_u}{\sigma'_{v0}}\right)^{-0.5} \; for \; \frac{s_u}{\sigma'_{v0}} \leq 1.0$$

Equation 5.7

$$\alpha = 0.5\left(\frac{s_u}{\sigma'_{v0}}\right)^{-0.25} \quad for \quad \frac{s_u}{\sigma'_{v0}} > 1.0$$

Equation 5.8

Where:

s_u = Undrained shear strength

σ'_{v0} = In situ vertical effective stress

As noted previously, the above calculations only apply to soil down to the self-weight penetration depth. The self-weight penetration depth is calculated in accordance with DNV GL (2017c) recommendations.

DNV GL (2017c) defines a transitional zone between penetration by self-weight and penetration by suction. The transitional zone is defined from the self-weight penetration depth to one caisson diameter below the self-weight penetration depth. In this transitional zone, DNV GL (2017c) states that it can be assumed that the effect of self-weight penetration decreases linearly with depth and the α value calculated for self-weight penetration should be applied because it provides more representative results.

For piles penetrated using suction, Equation 5.9 is applied:

$$\alpha = C_t.\frac{1}{S_t}.\frac{\alpha^{OC}}{\alpha^{NC}}$$

Equation 5.9

Where:

$\frac{\alpha^{OC}}{\alpha^{NC}}$ = Correction of α due to the effects of overconsolidation

Equation 5.9 is the same as Equation 5.6, except for the correction for the effects of overconsolidation. DNV GL (2017c) recommends using curve A in Figure 5.3 for the correction.

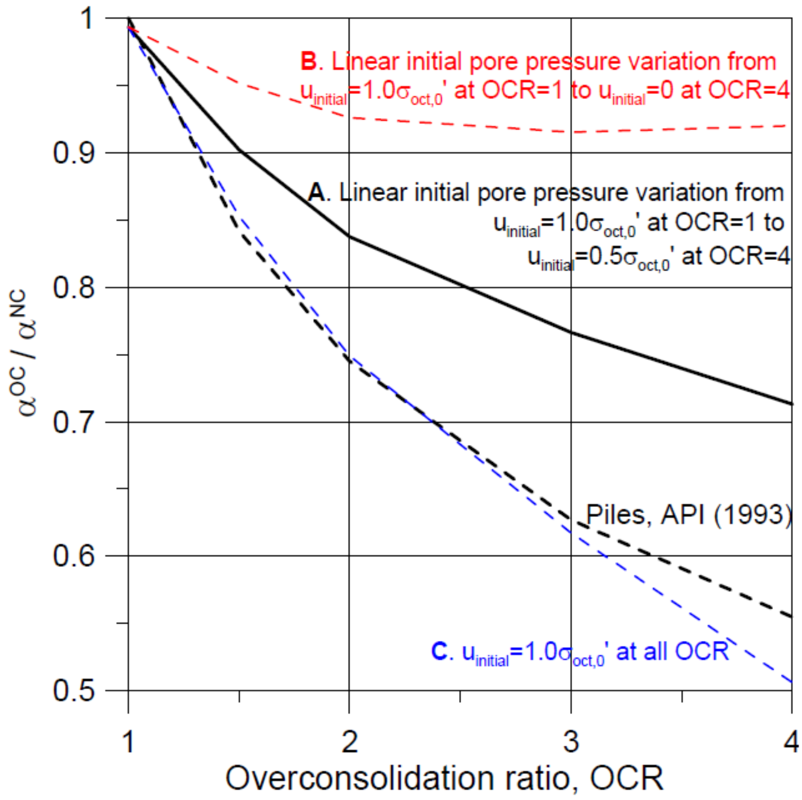

Figure 5.3: Correction of α as a function of overconsolidation ratio

For the purpose of this report the DNV GL (2017c) provided lower bound correction for α values for the suction pile shaft was used.

DNV GL (2017c) provides lower bound α values for the inside of piles that are penetrated by suction. DNV GL (2017c) also states that the inside friction factor increases with increasing OCR but it does not provide specific guidance. Available geotechnical data were used with Equation 5.6 to calculate α for inside the suction pile.

Horizontal load checks were performed for the horizontal loads considered here. Horizontal loads were not considered governing.

5.6.2.3 Results

Plates 39 and 40 present preliminary suction pile sizing charts for pile length versus diameter for various suction pile geometries. These plots represents the various caisson geometries required to support the manifold with a global factor of safety of 2.0 and considering the analysis approach adopted in this report. Table 5.6 summarises the outer diameter and suction pile lengths required to achieve the required capacities.

Table 5.6: Suction Pile Vertical Bearing Capacity Analyses Results

Outer Diameter [m]	Suction Pile Penetration Depth [m BML]	
	DODC-1	DODC-2
6.0	26.3	25.1
7.0	24.4	23.1
8.0	22.6	21.3
9.0	20.7	19.5
10.0	18.6	18.0
11.0	16.8	15.7

Notes:
BML = Below mudline
DODC-1 = Domino Drill Center 1
DODC-2 = Domino Drill Center 2

The suction pile lengths stated in Table 5.6 do not include an allowance for seafloor slope or plug heave. For a caisson outer diameter of 6.0 m, a suction pile penetration depth of 26.3 m BML is required at the DODC-1 location. For a caisson outer diameter of 6.0 m, a suction pile penetration depth of 25.1 m BML is required at the DODC-2 location.

The axial vertical capacity calculations are subject to the following conservatisms based on the design assumptions used for the analyses presented in this report:

- No base end bearing component was included in the soil resistance calculation. This is cautious for the condition where a fully penetrated or grouted caisson can transfer load via the soil plug. This cautious assumption takes into consideration incomplete contact between the base plate and seafloor due to seafloor slope, tilting of the caisson following installation or generation of voids during installation. In addition, no pile annulus end bearing was considered for the caisson. Consideration of end bearing may reduce the pile penetration depth requirement, but this should be considered on a load case by load case basis and considering the installation methodology and mitigations (i.e. underbase grouting);
- The design soil parameters used are discussed in Section 4. At the Domino drill center locations, a reduction of the LE undrained shear strength by a cautionary 30 % was applied to account for dissociation of the biogenic methane which can significantly reduce sediment strength and stiffness. The 30 % reduction is expected to be cautious, in liew of a detailed risk assessment and geotechnical impact appraisal, and based on Fugro’s experience of dissociation impacts in shallow soils.

5.6.3 Suction Pile Installation

5.6.3.1 <u>General</u>

Suction pile installation analyses were performed using the Houlsby and Byrne (2005) method. This method is used to predict:

- Self-weight penetration;
- Required suction to install the caisson.

The analyses were performed for the outer diameters and penetration depths presented in Table 5.6. A suction pile wall thickness of 25 mm was used in the preliminary installation assessment which is consistent with ExxonMobil (2017) specification. Suction pile installation analysis should be updated once the suction pile geometry and make-up are confirmed by the installation contractors.

HE soil strength parameters were applied in the suction pile penetration analyses. LE parameters were applied in the self-weight penetration assessment to predict the maximum self-weight penetration.

5.6.3.2 Method

The suction pile foundation self-weight penetration resistance was calculated as the sum of internal (Q_{si}) and external shaft resistance (Q_{se}) and end bearing resistance (Q_b):

$$Q_{tot} = Q_{s,e} + Q_{s,i} + Q_b = h\alpha_0 s_{u1}(\pi D_o) + h\alpha_i s_{u1}(\pi D_i) + (\gamma' h N_q + s_{u2} N_c)(\pi D t)$$

Equation 5.10

Where:

Q_{tot} = Total penetration resistance
D_o = Outer diameter
D_i = Internal diameter
D = Mean diameters
s_{u1} = Average undrained shear strength between mudline and depth h
s_{u2} = Undrained shear strength at depth h
α_0 and α_0 = External and internal adhesion factors as used in pile design
N_c = Bearing capacity factor
h = Penetration Depth
t = Suction pile wall thickness

Self-weight penetration ends when static equilibrium is achieved. This occurs at the depth where soil resistance is equal to the self-weight of the structure. At the end of self-weight penetration, suction is applied to penetrate the suction pile to the required penetration depth.

During suction-assisted penetration, a seal is formed at the tip of the suction pile foundation. Suction is applied within the caisson to reduce the water pressure to lower than the surrounding seawater. This forces the caisson to embed itself due to the differential pressure created on the top plate of the caisson.

Suction-assisted penetration resistance was predicted based on the Houlsby and Byrne (2005) method. The required suction is taken as the calculated resistance minus the structure self-weight. The maximum applicable suction is dependent on:

i. The absolute pressure at which the water cavitates;
ii. The minimum absolute pressure that can be achieved by the given pump design;
iii. The minimum relative pressure that can be achieved by the pump.

Equation 5.11 describes the suction pressure (s) required to install the caisson to a specified depth.

$$s = \frac{\left(h\alpha_0 s_{u1}(\pi D_o) + h\alpha_i s_{u1}(\pi D_i) + (\gamma' h N_q + s_{u2} N_c)(\pi D t)\right) - V'}{\pi D^2{}_o/_4}$$

Equation 5.11

Where:
V' = Vertical effective load

5.6.3.3 Results

Table 5.7 summarises the results of the suction pile installation analysis. The required suctions for installation, are within the allowable limits based on water depth.

Table 5.7: Suction Pile Analyses Results

OD	Domino Drill Center 1				Domino Drill Center 2			
	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction
[m]	[m]	[kN]	[m]	[kPa]	[m]	[kN]	[m]	[kPa]
6	26.3 (4.4)	0.828	4.7	875	25.1 (4.2)	0.790	4.6	815
7	24.4 (3.5)	0.897	4.6	635	23.1 (3.3)	0.849	4.4	582
8	22.6 (2.8)	0.950	4.4	470	21.3 (2.7)	0.895	4.2	425
9	20.7 (2.3)	0.979	4.2	344	19.5 (2.2)	0.923	4.0	310
10	18.6 (1.9)	0.978	4.0	244	18.0 (1.8)	0.947	3.9	232
11	16.8 (1.5)	0.973	3.8	174	15.7 (1.4)	0.909	3.6	153

Notes:
OD = Outer Diameter
SWT = Self weight

5.6.4 Discussion of Suction Pile Results

This section discusses the effects of the assumptions applied in the suction pile stability and installation analyses. The preliminary suction pile analyses presented in the report provide a cautious upper bound of foundation geometries. These preliminary analyses adopts the following estimates:

i. For suction pile vertical capacity analyses; low estimate s_u considering historic data, reductions in s_u due to the presence of gas hydrates and application of a global factor of safety of 2.0;
ii. For suction pile installation analyses; high estimate s_u considering historic data,;
iii. Soil thixotropy effects;
iv. End bearing resistance applicability (no end bearing was considered).

No base end bearing was considered in the vertical capacity analyses of the suction pile. This is a conservative assumption that takes into consideration no full contact between the base plate and seafloor due to seafloor slope or tilting of the caisson following installation. Annulus (tip) end bearing was also not considered as a conservative assumption due to the extremely low strength of the soil.

As described above, the preliminary analyses performed represent an upper bound of the suction pile geometry. Detailed analyses are expected to allow optimisation of the suction pile geometry. The following example considerations are recommended for detailed design:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Considering thixotropic effects for increased time intervals on a per unit basis in accordance with operation schedules and based on site-specific thixotropy data;
iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, S_t, compression parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u;
vi. Consideration of suction pile annulus and base resistance mobilisation with load rate and duration.

It is recommended that these effects are quantified and considered for detailed design in accordance with any specific ExxonMobil design basis requirements.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

ExxonMobil requested Fugro to derive geotechnical design soil parameters for use in mudmat and suction pile stability and installation assessment.

6.2 Geotechnical Data

The following data sources were used to derive the geotechnical data presented in this report.

i. Neptun Deep integrated report (Fugro, 2016a);
ii. Laboratory and in situ testing data report (Fugro 2015a);
iii. Laboratory and in situ testing data report (Fugro 2018a).

6.3 Geological Setting

The planned Domino drill centers are located in the deepwater area of the Neptun block. The sediments within the foundation zone comprise lacustrine clays deposited in a freshwater environment.

Global sea level rise and the reconnection of the Bosphorus Strait led to the flooding of the Black Sea and the deposition of organic rich clay (sapropel) and coccolith ooze.

During periods of sea-level lowstand, the canyons acted as the main source of sediment transport, with sand and silt layers deposited during periods of high canyon activity. This is observable in the boreholes drilled at the two planned drill centers. DODC-2 is located closer to the canyon and as a result has more sand and silt layers within Geotechnical Soil Unit 5 than at the DODC-1 Location. Geotechnical Soil Unit 5 sediments at DODC-2 are also more overconsolidated than the sediments at DODC-1; this is interpreted to be due to erosion by downcanyons flows. The canyons are not interpreted to be active at present.

6.4 Geotechnical Design Soil Parameters

Geotechnical design soil parameters were derived for mudmat and suction pile stability and installation analyses at the DODC-1 and the DODC-2 locations. Mudmat and suction pile foundations are to be installed to support ITA at the DODC-1 location, FLET at the DODC-2 location and manifold structures at both DODC-1 and DODC-2 locations.

LE, BE and HE design soil profiles were derived to the depth of investigation. The design profiles presented in this report are applicable specifically to the analyses presented in this report and should be carefully reviewed for any other purpose.

6.5 Engineering Analysis

6.5.1 General

Concept level mudmat and suction pile stability and installation analyses are presented in this report. Mudmat and suction pile analyses were performed according to API (2011) working stress design approach applying a global factor of safety of 2.0 to unfactored loads and unfactored resistances.

The following foundation design risks are identified:

i. Coccolith ooze and sapropel formations;
ii. Gas hydrates;
iii. Buried mass transport deposits.

The identified foundation design risks should be mitigated by performing a detailed review of the impact these risks which may affect the design of the foundations. The following mitigations measures were planned for the above geotechnical risks.

Coccolith ooze and sapropel formation: Due to the extremely weak strength of the formation and liquidity index, a detailed review of the foundation design analyses of both formations is required to review:

i. Detailed mudmat setdown analysis or landing impact to assess the risk of soil wash out, fluidisation and excessive settlement occurring due to disturbance of these highly sensitive soils;
ii. Detailed assessment of the geotechnical soil interaction of the coccolith ooze and sapropel formations with the underlying formations and the potential effects on foundation stability outside of conventional design practice.

Landing impact and settlement analyses were not performed as part of this report. However, Fugro recommends that the landing impact of the mudmat and settlement analysis should be further reviewed in detailed design.

Alternatively, dredging or otherwise would mitigate the design risks associated with these soils given the extremely low strength of the highly sensitive coccolith ooze and sapropel formations.

Shallow gas: Shallow gas is not present as free gas at the Domino drill center locations. Gas (biogenic methane) is present as gas hydrates which is stable at the current temperatures and pressures. However, should the temperatures and pressures change over the operational lifetime of the well, the gas hydrates may dissociate and become free gas. Therefore, to account for the possible significant reduction in sediment strength due to dissociation of biogenic methane, a 30 % cautionary reduction in the undrained shear strength was applied for foundation analyses. This differs to the reductions associated with free gas at the platform location (~10 % assumed). Fugro recommends further review of the risk of gas dissociation in the detailed design stage.

Buried mass transport deposits: Buried mass transport deposits (MTD) were observed at the DODC2 location. The MTD layer was observed to be of higher localised strength in comparison to the surrounding geotechnical soil units. The MTD layer was observed between 3.8 m and 5.8 m BML. There is the potential for unexpected over-penetration or under-penetration of foundations where these stronger blocks of sediment are present.

6.5.2 Mudmat Analysis

Mudmat stability and installation analyses were performed for the ITA structure at the DODC-1 location and for the FLET at the DODC-2 location. Loads used in the analysis were as specified by

ExxonMobil (2017). The preliminary mudmat analyses presented in the report provide a cautious upper bound of foundation geometries. These preliminary analyses adopt cautious estimates of:

i. For mudmat stability analyses, low estimate s_u considering historic data, reductions in s_u due to the presence of hydrates and application of a global factor of safety of 2.0;
ii. For mudmat installation analyses, high estimate s_u considering historic data;
iii. Soil layering effects.

Preliminary mudmat vertical, horizontal, moment and torsion stability analysis were performed according to Feng at al. (2014) recommendations. Results of the mudmat analyses shows that:

i. At the DODC-1 location, a 20.0 m by 12.0 m mudmat with a skirt height of 1.2 m is required to support the applied loads;
ii. At the DODC-2 location, a 29.3 m by 16.0 m mudmat with a skirt height of 1.0 m is required to support the applied loads.

The mudmat sizings above are considered impractically large and are governed by the extremely low strength of the highly sensitive coccolith ooze and sapropel formations. In this case dredging or otherwise removal of these formations should be considered.

If the coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2) were removed, a 12.5 m by 7.5 m ITA mudmat with a skirt height of 0.1 m at the DODC-1 location and a 19.3 m by 10.5 m FLET mudmat with a skirt height of 0.2 m at the DODC-2 location would be required to support the applied loads based on the cautious design assumptions applied in this report.

Should the removal of the formations not be considered feasible, Fugro recommenda that the mudmat analyses presented in this report are refined for detailed design considering the potentially highly sensitive coccolith ooze and sapropel (Geotechnical Soil Units 1 and 2).

Mudmat skirt penetration analyses were performed according to the following methods:

i. API RP 2GEO (API, 2011);
ii. DNV GL-RP-C212 (DNV GL, 2017b).

The results of the mudmat skirt penetration analysis shows that the skirts can be installed using self-weight to required depth without exceeding the soil resistance. An exceptional case is, for the 22 m by 12.0 m FLET mudmat with a skirt of 1.7 m, the mudmat is not installable according to the upper bound DNV (2017b) recommendations.

If the coccolith ooze and sapropel formations (Geotechnical Soil Units 1 and 2) were removed, the 0.1 m and 0.2 m skirt heights analysed in this report would be installable under self-weight of the mudmats.

Fugro recommends that the mudmat analyses presented in this report are refined for detailed design considering but not limited to the following:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Soil layering interaction effects between weaker highly sensitive soil layers (coccolith ooze and sapropel) and more competent layers (Geotechnical Unit 3) on stability analyses;
iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, compression parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u.

It is recommended that these effects are quantified and considered in detailed during detailed design in accordance with any specific ExxonMobil design basis requirements.

6.5.3 Suction Pile Analysis

Vertical suction pile stability analyses were performed according to DNV GL (2017c) for the FLET structure at both DODC-1 and DODC-2 locations. Loads used in the analysis were provided by ExxonMobil.

It should be noted that the suction pile bearing capacity and installation analyses presented consider that the Coccolith ooze and Sapropel formations are present i.e. no dredging or otherwise removal of the formations was considered.

Suction pile installation analyses were performed according to Houlsby and Byrne (2005) method. Table 6.1 summarises the results of the suction pile vertical stability and installation analyses.

Table 6.1: Suction Pile Vertical Bearing Capacity and Installation Analyses Results

OD	Domino Drill Center 1				Domino Drill Center 2			
	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction	Pile Penetration Depth (L/D)	Installation Load	SWT Penetration	Required Suction
[m]	[m]	[kN]	[m]	[kPa]	[m]	[kN]	[m]	[kPa]
6	26.3 (4.4)	0.828	4.7	875	25.1 (4.2)	0.790	4.6	815
7	24.4 (3.5)	0.897	4.6	635	23.1 (3.3)	0.849	4.4	582
8	22.6 (2.8)	0.950	4.4	470	21.3 (2.7)	0.895	4.2	425
9	20.7 (2.3)	0.979	4.2	344	19.5 (2.2)	0.923	4.0	310
10	18.6 (1.9)	0.978	4.0	244	18.0 (1.8)	0.947	3.9	232
11	16.8 (1.5)	0.973	3.8	174	15.7 (1.4)	0.909	3.6	153

Notes:
OD = Outer Diameter
SWT = Self weight

The preliminary suction pile analyses presented in the report provide a cautious upper bound of foundation geometries. These preliminary analyses adopt the following:

i. For suction pile vertical capacity analyses; low estimate s_u considering historic data, reductions in s_u due to the presence of gas hydrates and application of a global factor of safety of 2.0;
ii. For suction pile installation analyses; high estimate s_u considering historic data;
iii. Soil thixotropy effects;

iv. End bearing resistance applicability (no end bearing was considered).

No base end bearing was considered in the vertical capacity analyses of the suction pile. This is a conservative assumption that takes into consideration no full contact between the base plate and seafloor due to seafloor slope or tilting of the caisson following installation. Annulus (tip) end bearing was also not considered as a conservative assumption due to the extremely low strength of the soil.

Fugro recommends that the suction pile analyses presented in this report are refined for detailed design considering but not limited to the following:

i. Structure-location specific design soil parameterisation as far as is possible with the available dataset;
ii. Considering thixotropic effects for increased time intervals on a per unit basis in accordance with operation schedules and based on site-specific thixotropy data;
iii. Quantifying the effects of gas hydrate dissociation on key design soil parameters (e.g. s_u, S_t, compression parameters);
iv. Consideration of consolidated strength increase;
v. Rate effects on s_u;
vi. Consideration of suction pile annulus and base resistance mobilisation with load rate and duration.
vii. The potentially highly sensitive Coccolith ooze and Sapropel formations (Geotechnical Soil Units 1 and 2).

It is recommended that these effects are quantified and considered for detailed design in accordance with any specific ExxonMobil design basis requirements.

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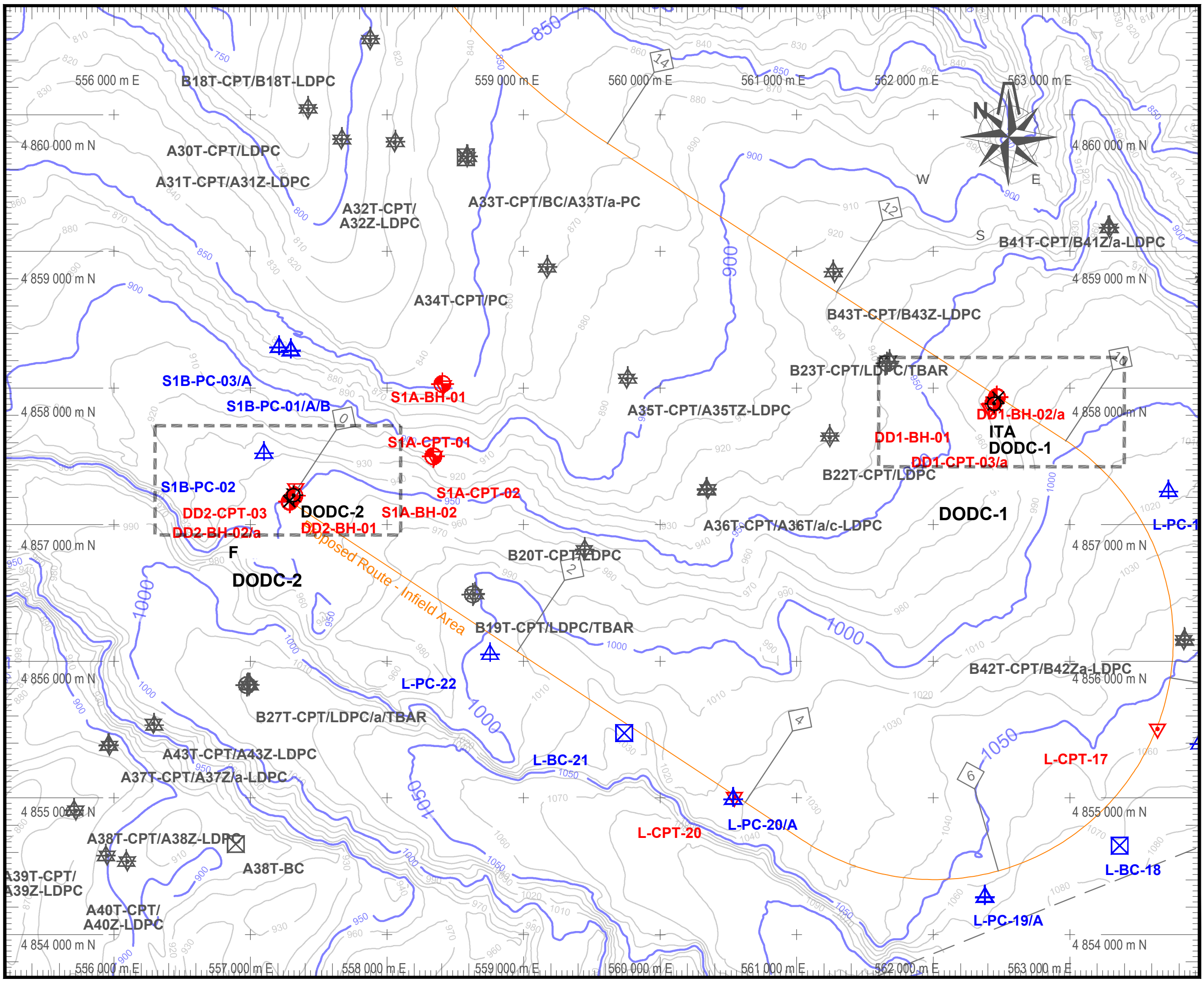
LIST OF PLATES



GEODETIC PARAMETERS:

Geodetic datum:	WGS84
Ellipsoid:	WGS84
Projection:	Transverse Mercator
Zone:	30 NE
Central Meridian:	30° 00' 00" E

LEGEND:

- Proposed Pipeline Route
- Bathymetric contours at 10 m intervals with index contour at 50 m intervals reduced to MSL
- ITA In-line Tee Assembly
- FLET Flowline End Template
- DODC Manifold at DODC-1 and 2

Fugro report J31109 (2014/2015):

- BH Borehole Location - CPT Only
- CPT Cone Penetration Test
- VC Vibrocore
- BC Box Core
- TBAR T-Bar Test
- PC Piston Core
- LDPC Large Diameter Piston Core

Geoquip report (2017):

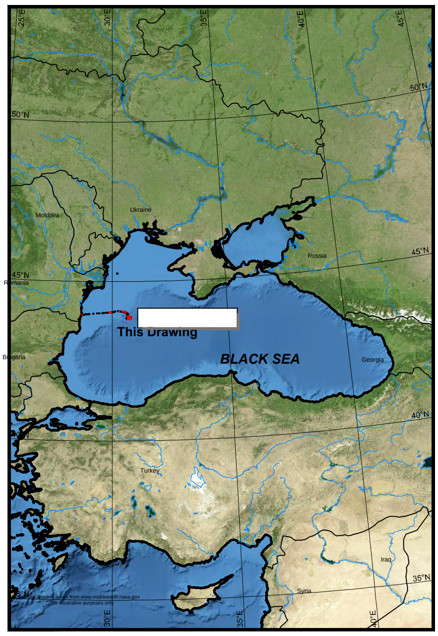
- BC Box Core
- PC Piston Core

Fugro report 173570 (2017):

- BH Borehole Location - Combined
- CPT Cone Penetration Test
- PH Pilot Hole

Scale 1:50 000

0 500 1 000 1 500 m



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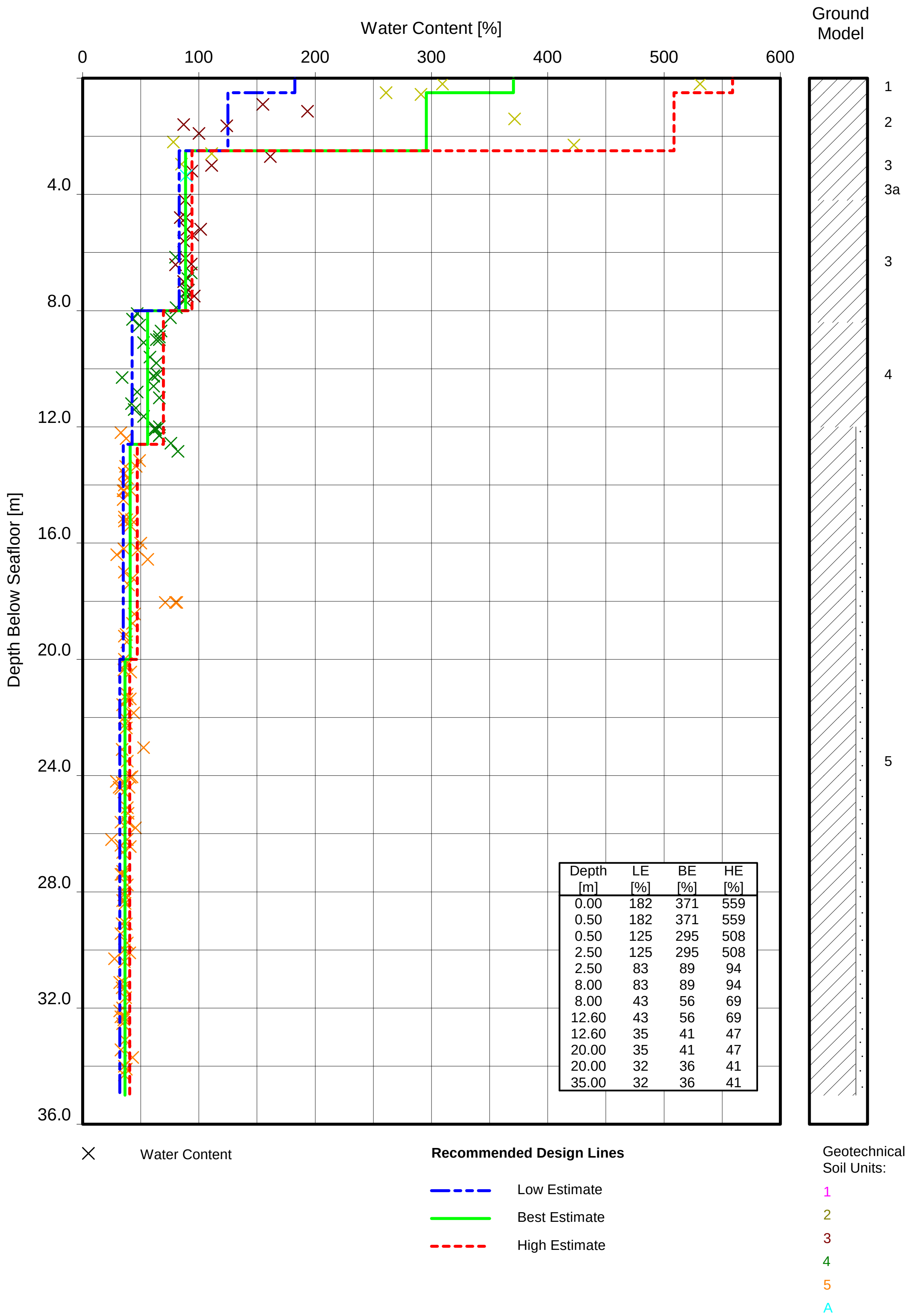
Issue:	Description:	File: 173570_Detailed_Plan.dwg	Drawn:	Interpreted:	Checked:	Approved:	Date:

DETAILED LOCATION PLAN - DOMINO INFIELD AREA



Depth [m]	LE [%]	BE [%]	HE [%]
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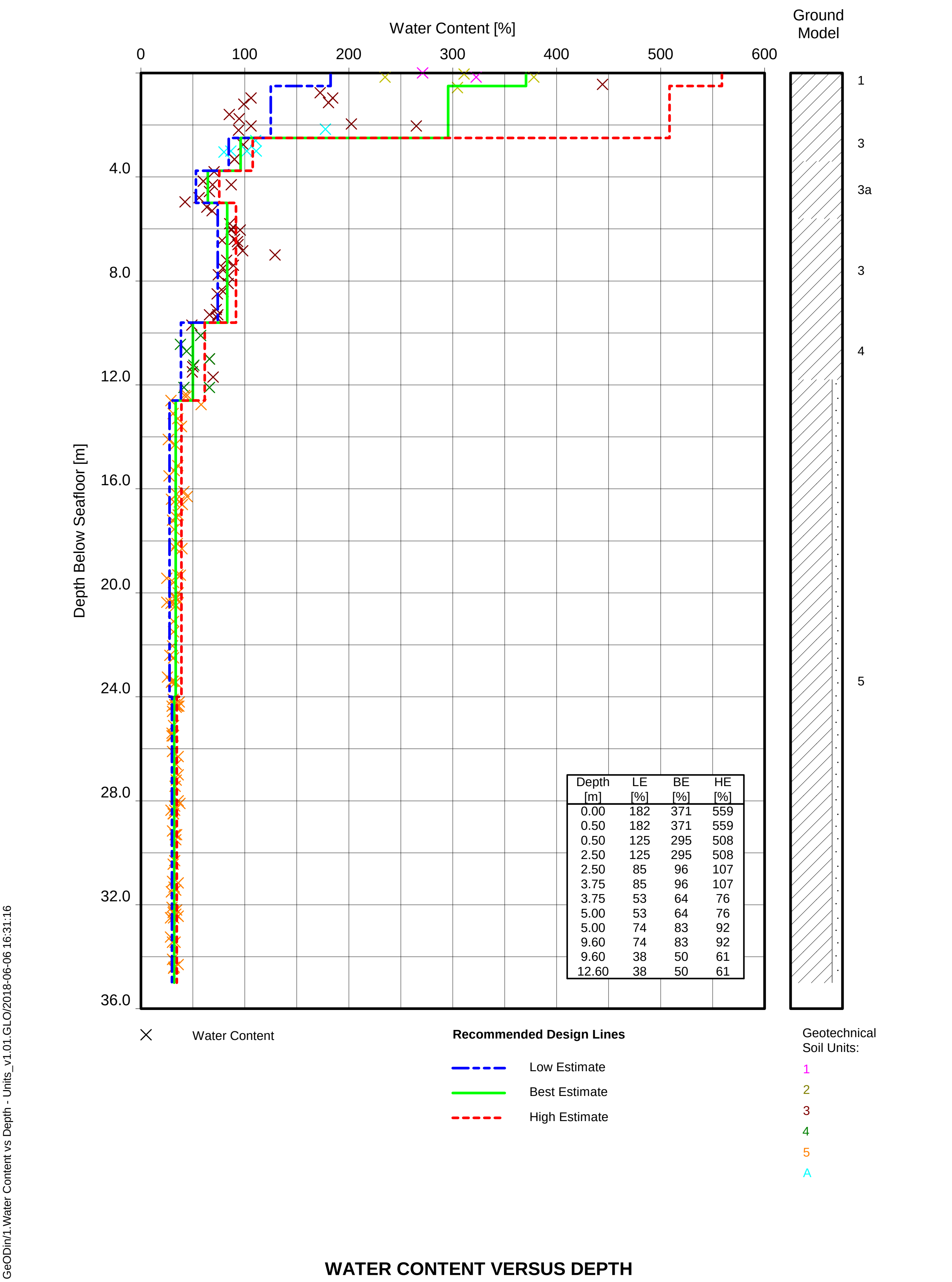
0.50	182	371	559
0.50	125	295	508
2.50	125	295	508
2.50	83	89	94
8.00	83	89	94
8.00	43	56	69
12.60	43	56	69
12.60	35	41	47
20.00	35	41	47
20.00	32	36	41
35.00	32	36	41

GeODin/1.Water Content vs Depth - Units_v1.01.GLO/2018-06-06 16:30:13

WATER CONTENT VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [%]	BE [%]	HE [%]
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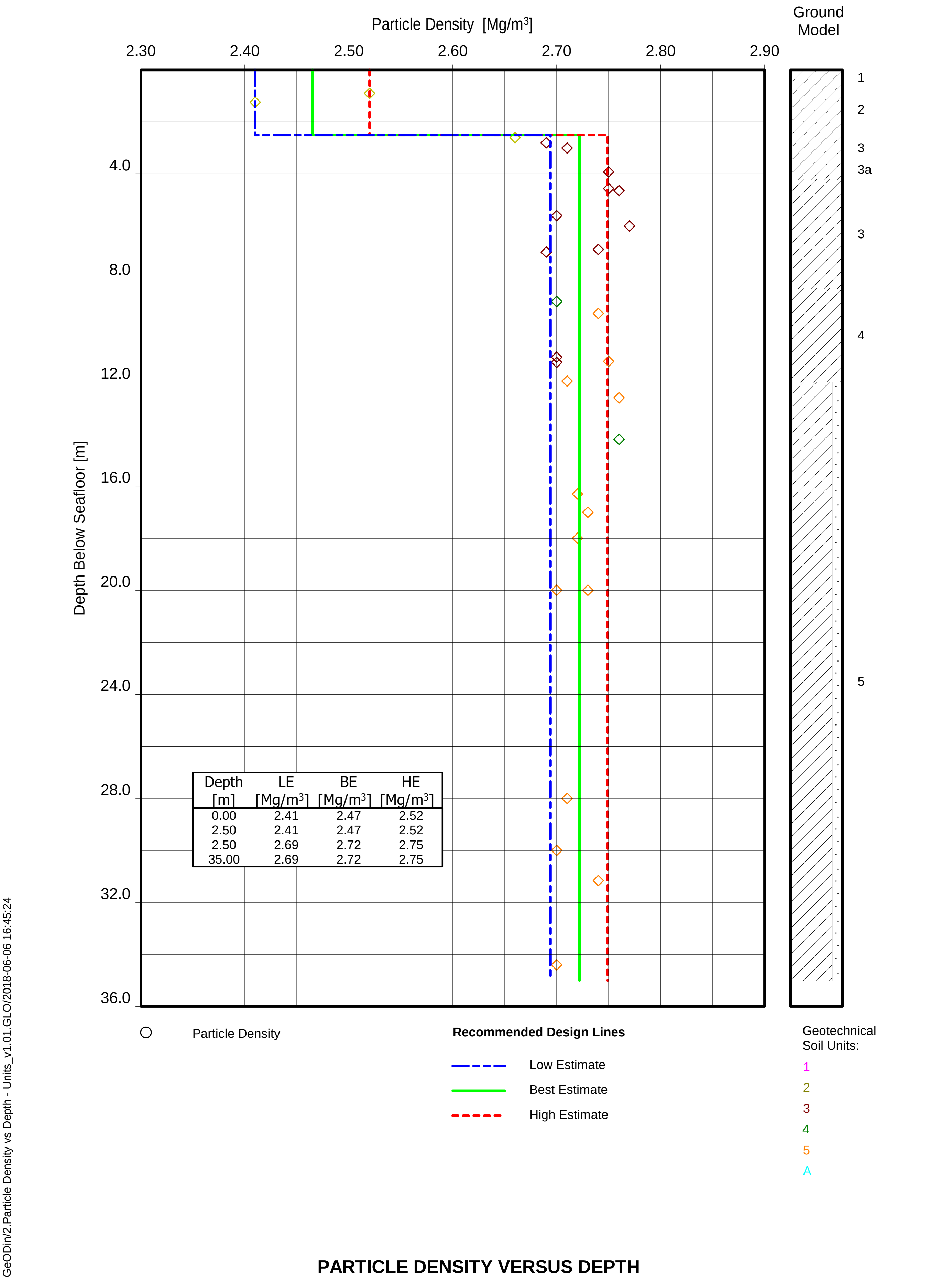
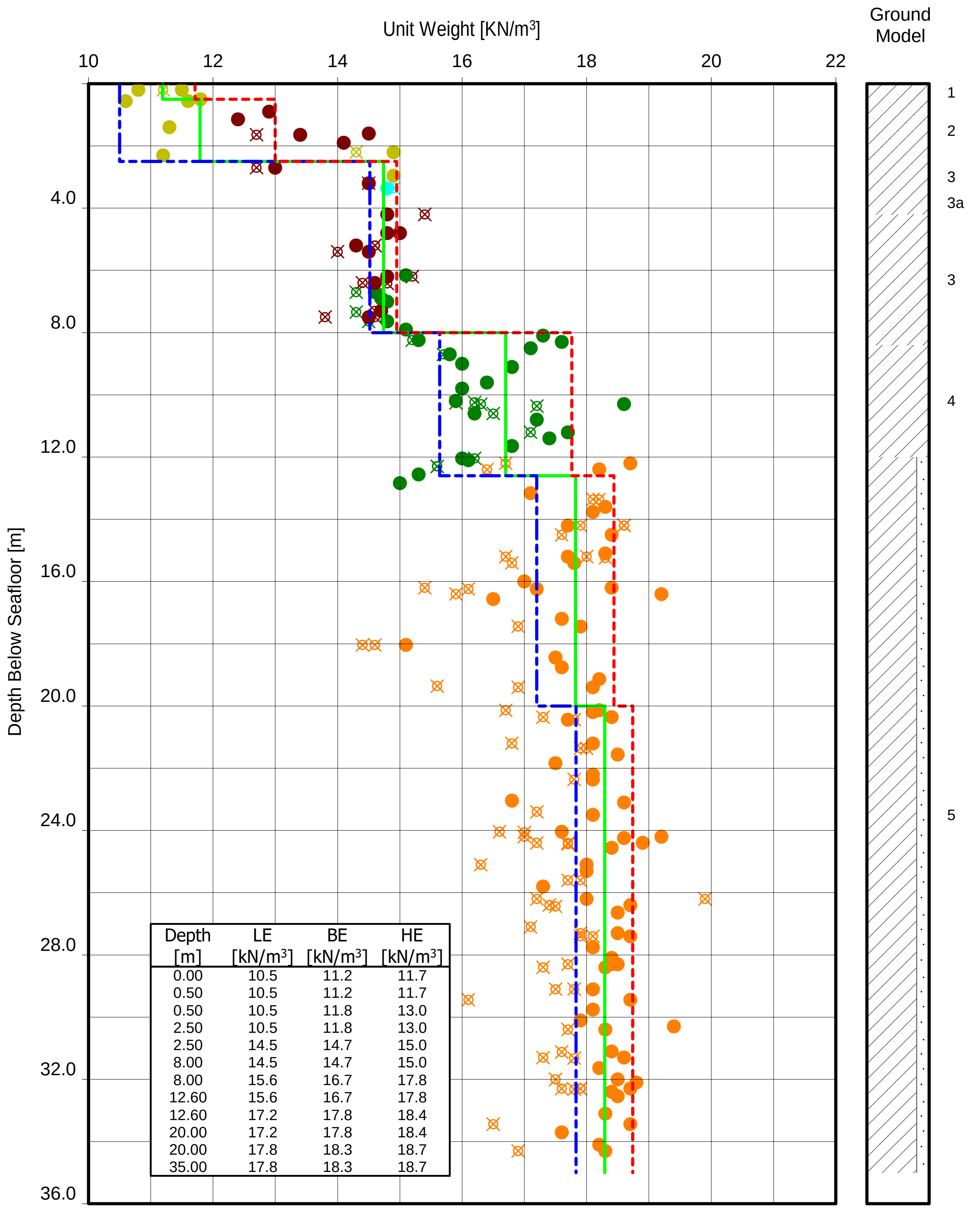
0.50	182	371	559
0.50	125	295	508
2.50	125	295	508
2.50	85	96	107
3.75	85	96	107
3.75	53	64	76
5.00	53	64	76
5.00	74	83	92
9.60	74	83	92
9.60	38	50	61
12.60	38	50	61

WATER CONTENT VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [Mg/m³]	BE [Mg/m³]	HE [Mg/m³]
0.00	2.41	2.47	2.52
2.50	2.41	2.47	2.52
2.50	2.69	2.72	2.75
35.00	2.69	2.72	2.75

PARTICLE DENSITY VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey



Depth [m]	LE [kN/m³]	BE [kN/m³]	HE [kN/m³]
0.00	10.5	11.2	11.7
0.50	10.5	11.2	11.7
0.50	10.5	11.8	13.0
2.50	10.5	11.8	13.0
2.50	14.5	14.7	15.0
8.00	14.5	14.7	15.0
8.00	15.6	16.7	17.8
12.60	15.6	16.7	17.8
12.60	17.2	17.8	18.4
20.00	17.2	17.8	18.4
20.00	17.8	18.3	18.7
35.00	17.8	18.3	18.7

● Derived from water content and particle density

⊠ Derived from volume mass calculation

Recommended Design Lines

Low Estimate

Best Estimate

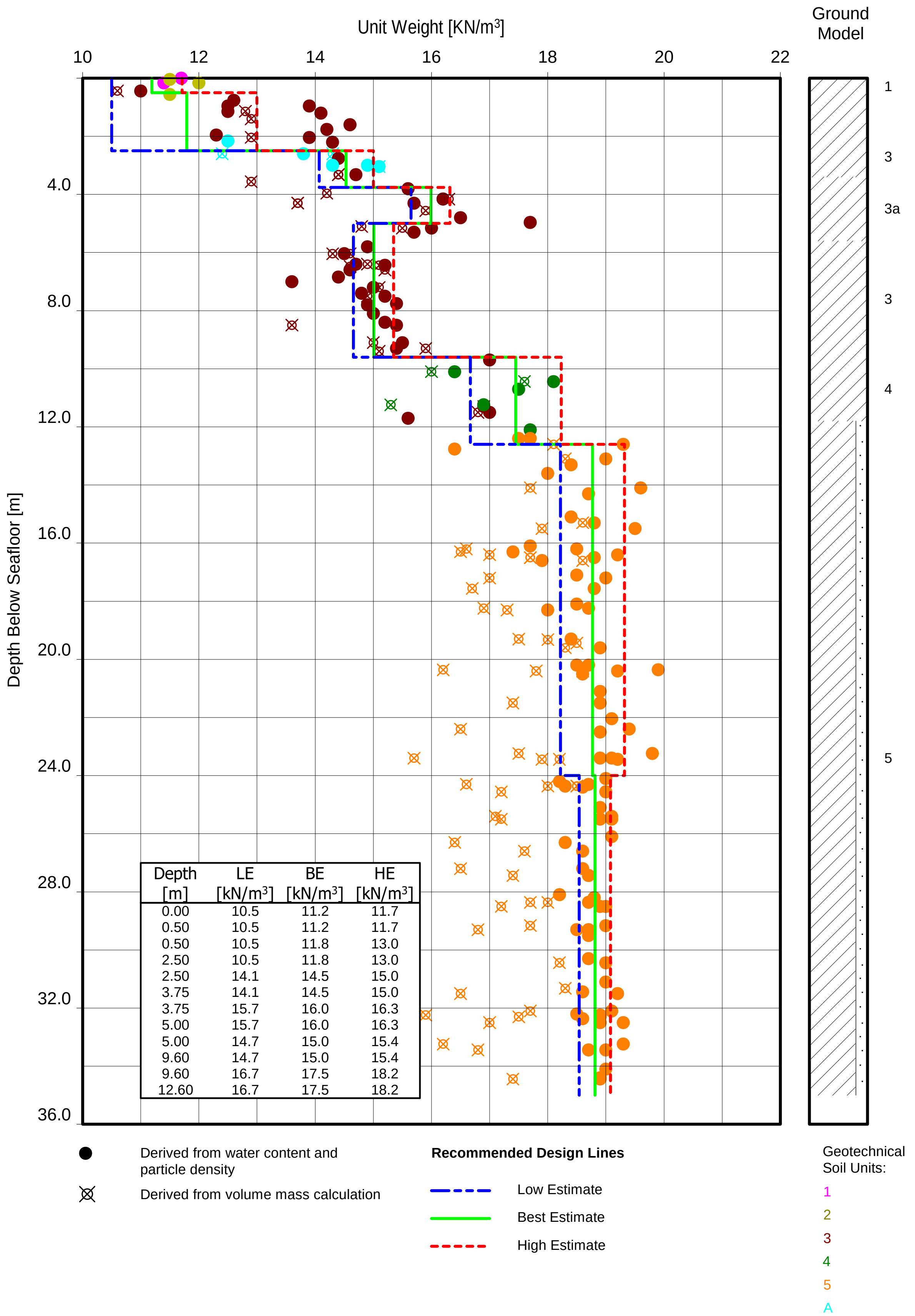
High Estimate

Geotechnical Soil Units:

1
2
3
4
5
A

GeODin/3.Unit Weight versus Depth - Units_v1.01.GLO/2018-06-06 17:16:28

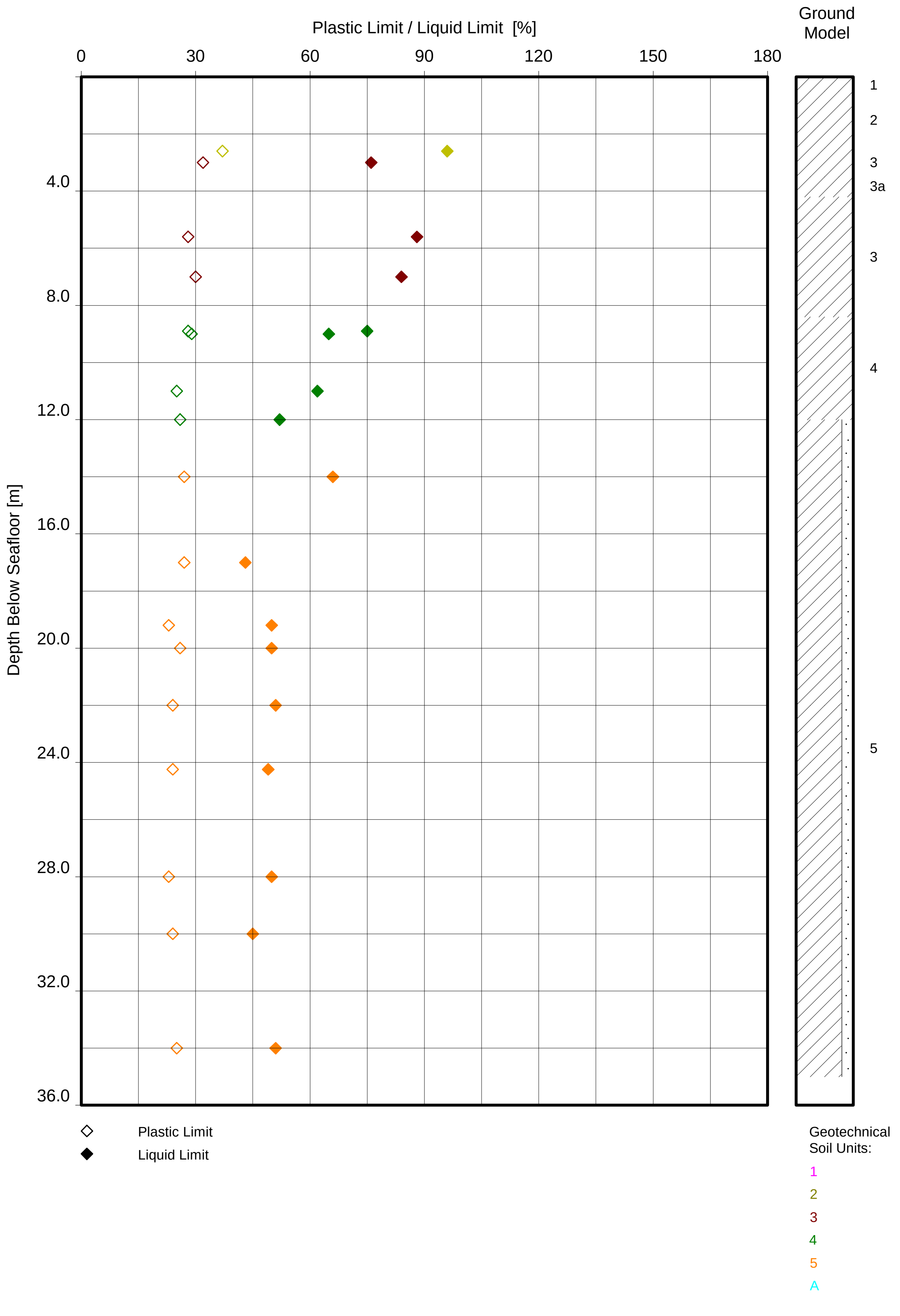
UNIT WEIGHT VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [kN/m³]	BE [kN/m³]	HE [kN/m³]
0.00	10.5	11.2	11.7
0.50	10.5	11.2	11.7
0.50	10.5	11.8	13.0
2.50	10.5	11.8	13.0
2.50	14.1	14.5	15.0
3.75	14.1	14.5	15.0
3.75	15.7	16.0	16.3
5.00	15.7	16.0	16.3
5.00	14.7	15.0	15.4
9.60	14.7	15.0	15.4
9.60	16.7	17.5	18.2
12.60	16.7	17.5	18.2

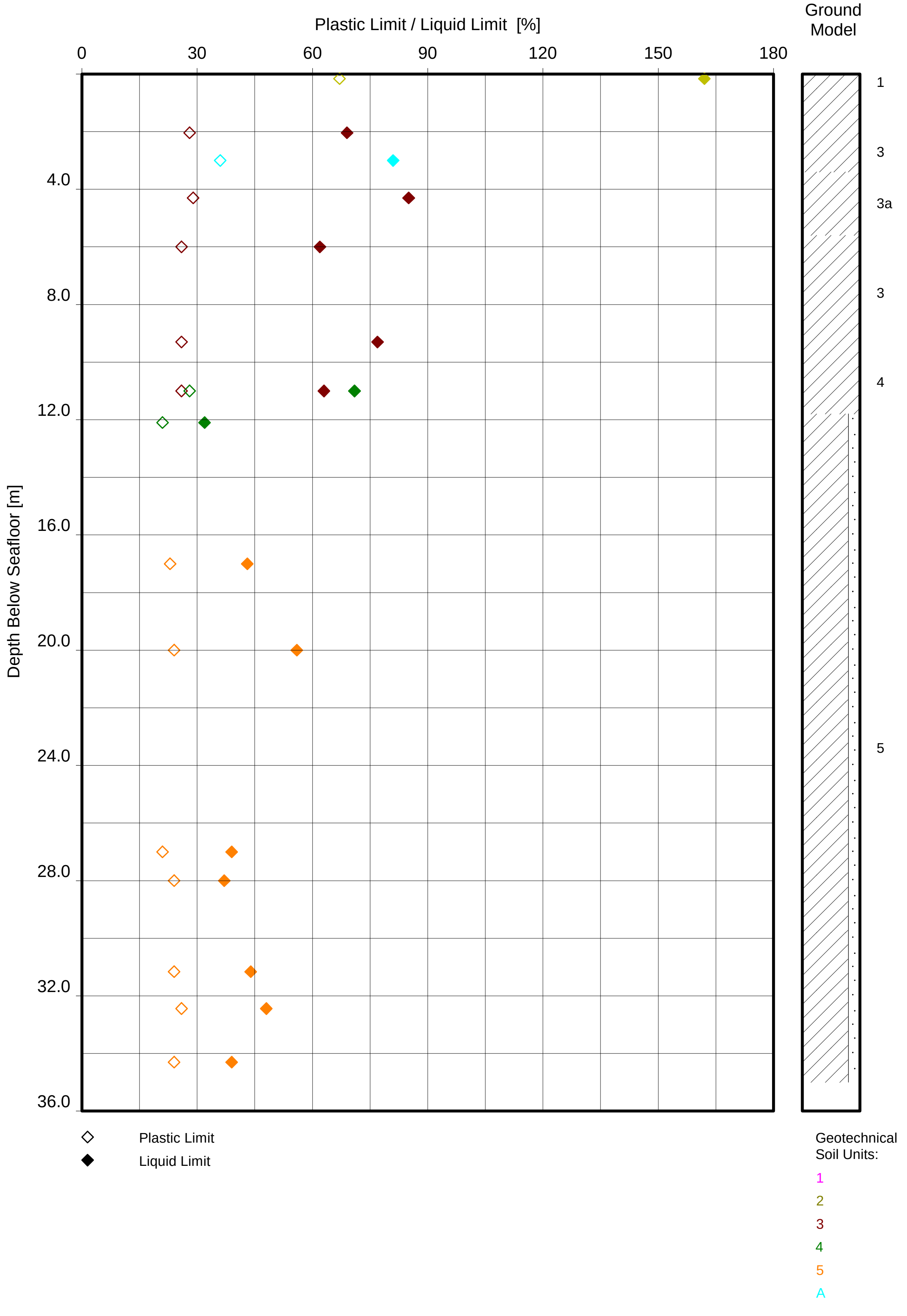
GeODin/3.Unit Weight versus Depth - Units_v1.01.GLO/2018-06-06 17:17:15

UNIT WEIGHT VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



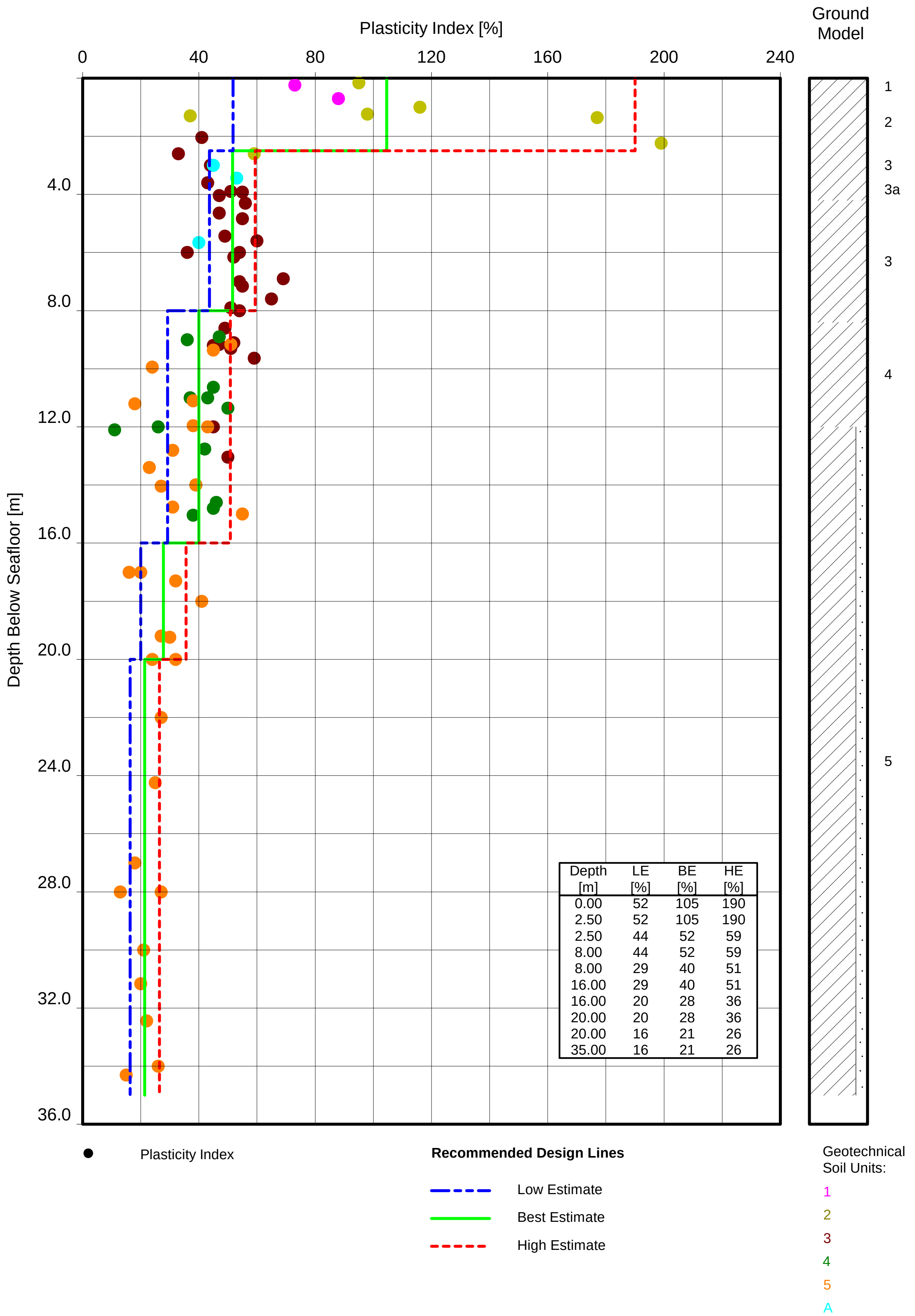
GeODin/4.Plastic Limit _ Liquid Limit vs Depth - Units_v1.01.GLO/2018-06-06 17:18:11

PLASTIC LIMIT / LIQUID LIMIT VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



GeODin/4.Plastic Limit _ Liquid Limit vs Depth - Units_v1.01.GLO/2018-06-06 17:18:38

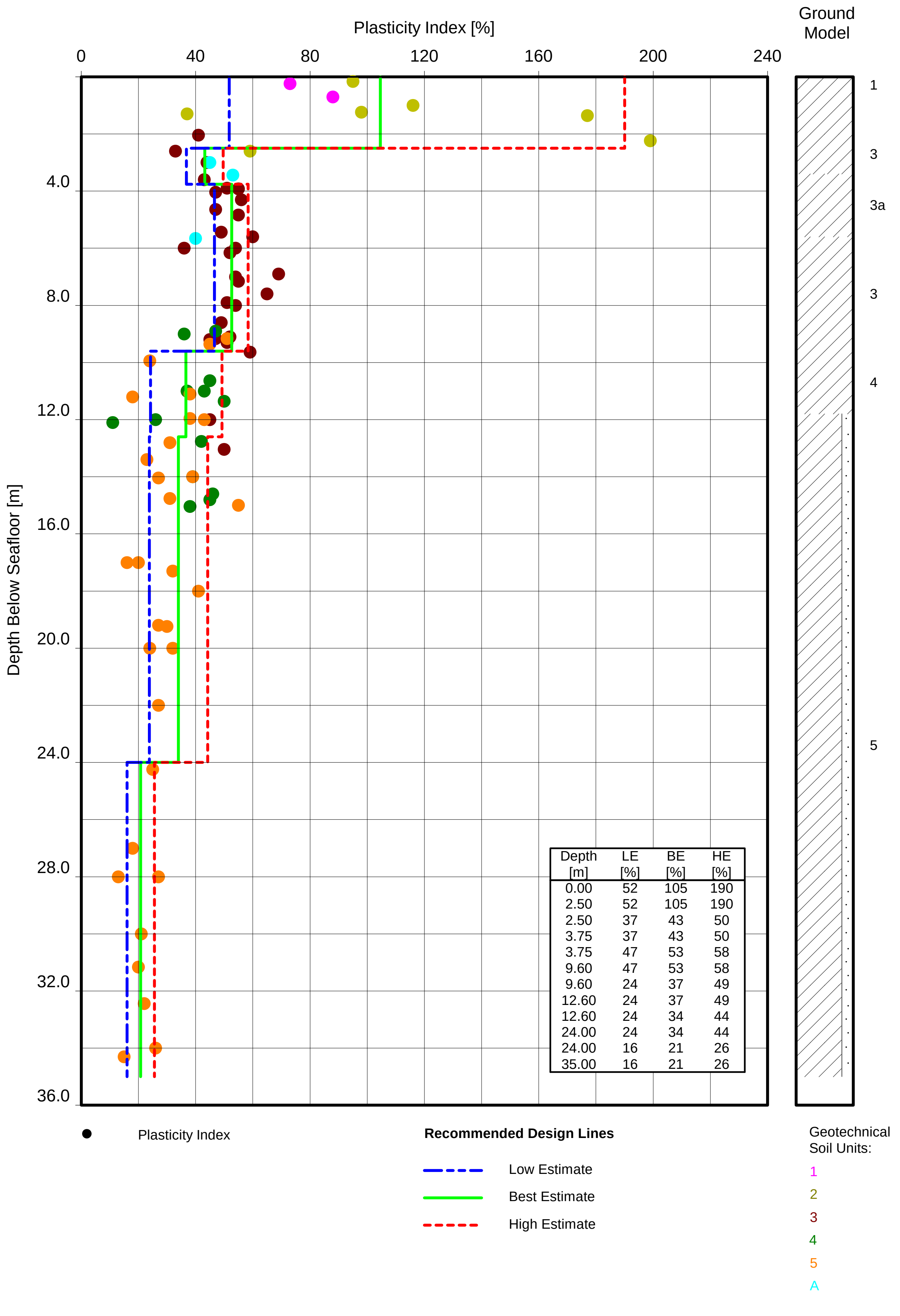
PLASTIC LIMIT / LIQUID LIMIT VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [%]	BE [%]	HE [%]
0.00	52	105	190
2.50	52	105	190
2.50	44	52	59
8.00	44	52	59
8.00	29	40	51
16.00	29	40	51
16.00	20	28	36
20.00	20	28	36
20.00	16	21	26
35.00	16	21	26

GeODin/5.Plasticity Index vs Depth - Units_v1.01.GLO/2018-06-13 10:20:32

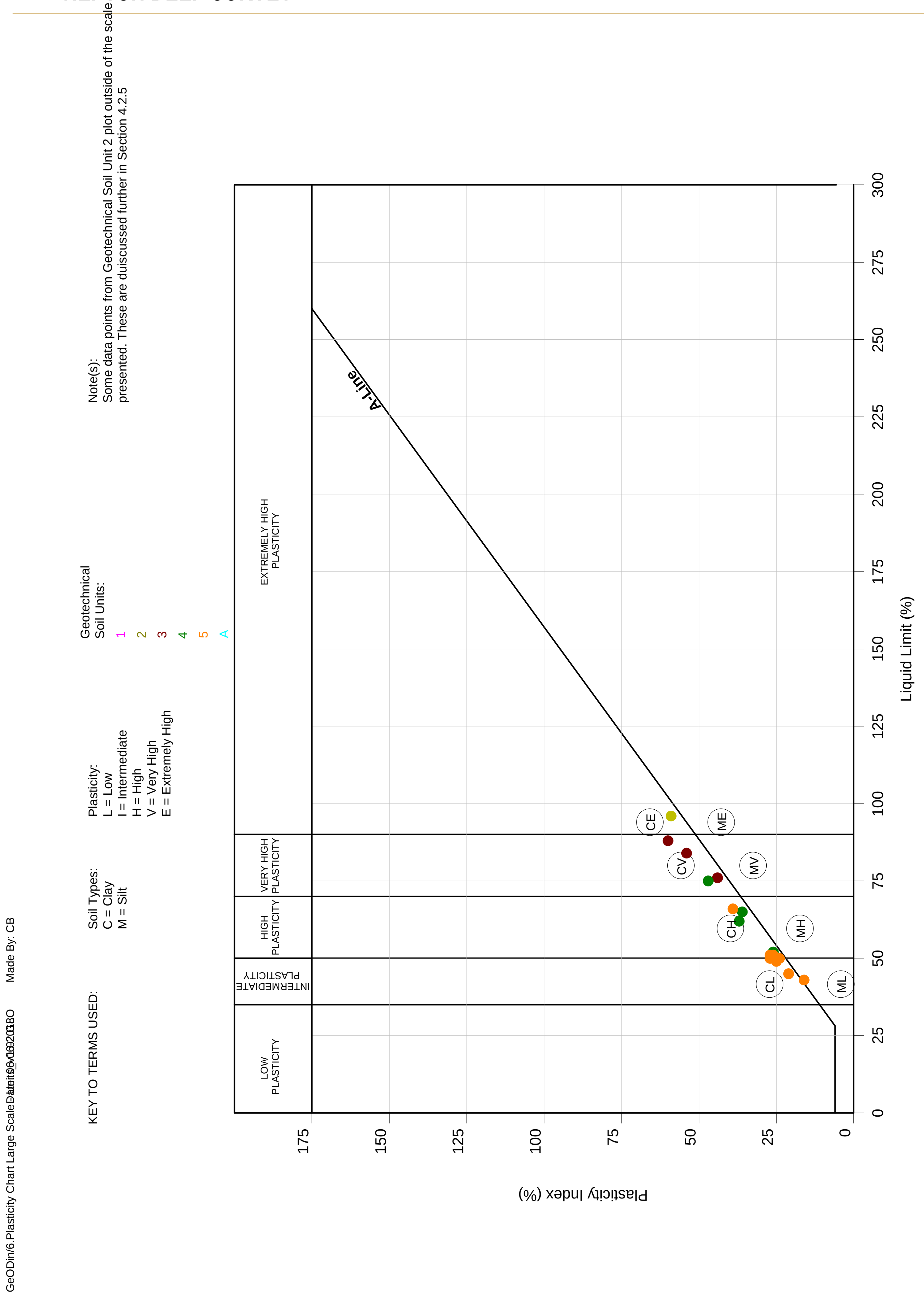
PLASTICITY INDEX VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



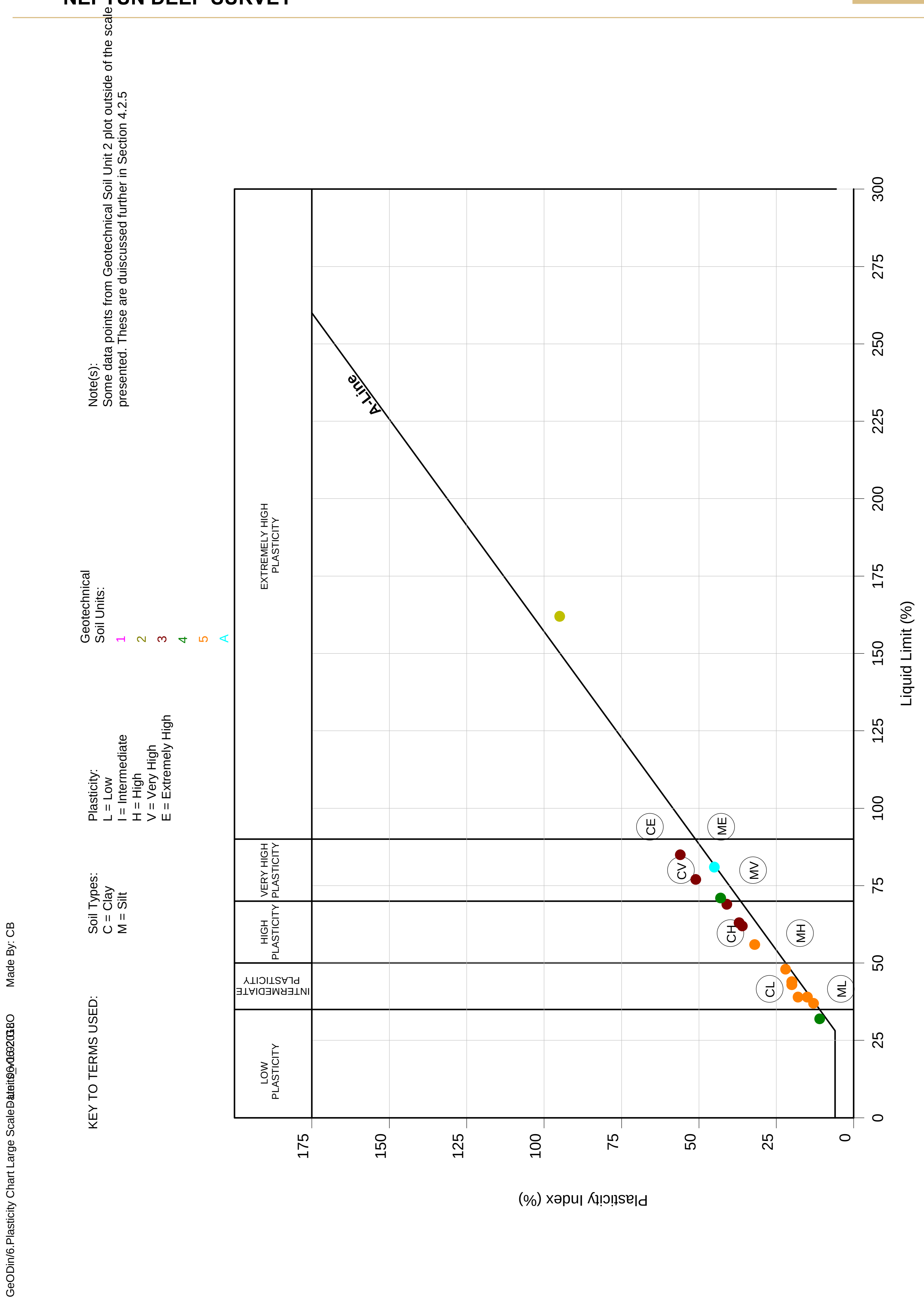
Depth [m]	LE [%]	BE [%]	HE [%]
0.00	52	105	190
2.50	52	105	190
2.50	37	43	50
3.75	37	43	50
3.75	47	53	58
9.60	47	53	58
9.60	24	37	49
12.60	24	37	49
12.60	24	34	44
24.00	24	34	44
24.00	16	21	26
35.00	16	21	26

GeODin/5.Plasticity Index vs Depth - Units_v1.01.GLO/2018-06-13 10:21:02

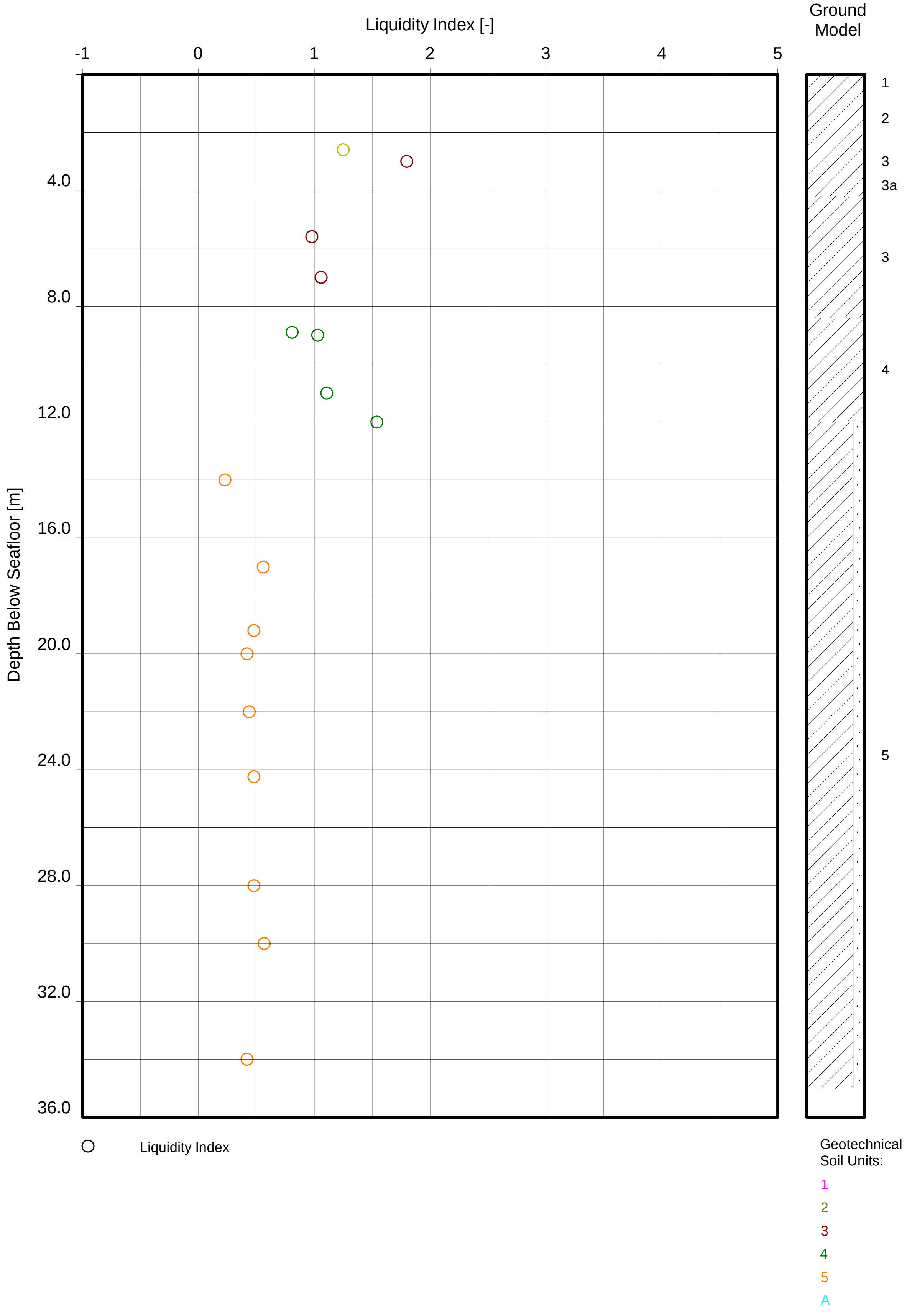
PLASTICITY INDEX VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



PLASTICITY CHART (BS 5930)
Domino Drill Center 1, Neptun Deep Survey

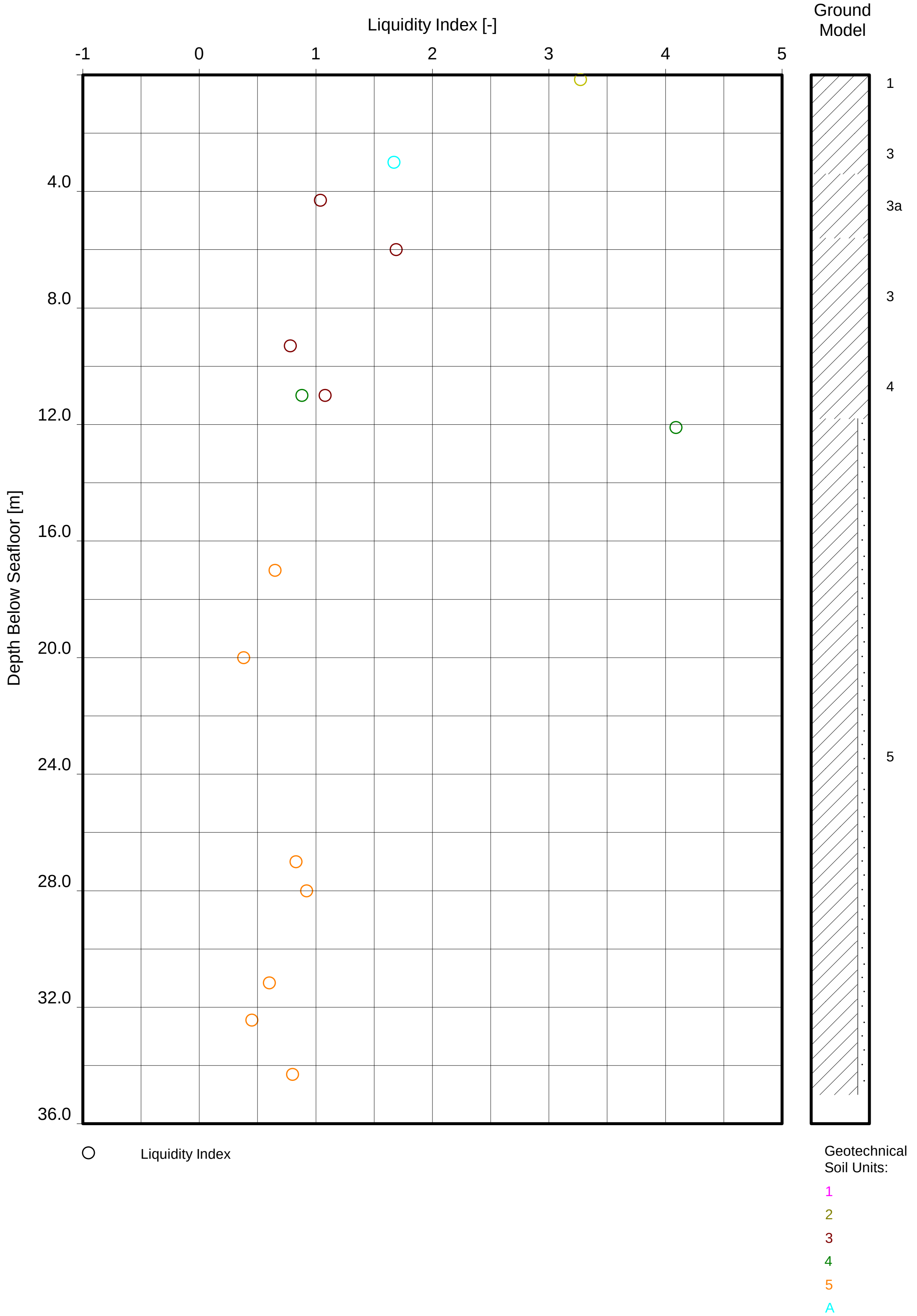


PLASTICITY CHART (BS 5930)
Domino Drill Center 2, Neptun Deep Survey



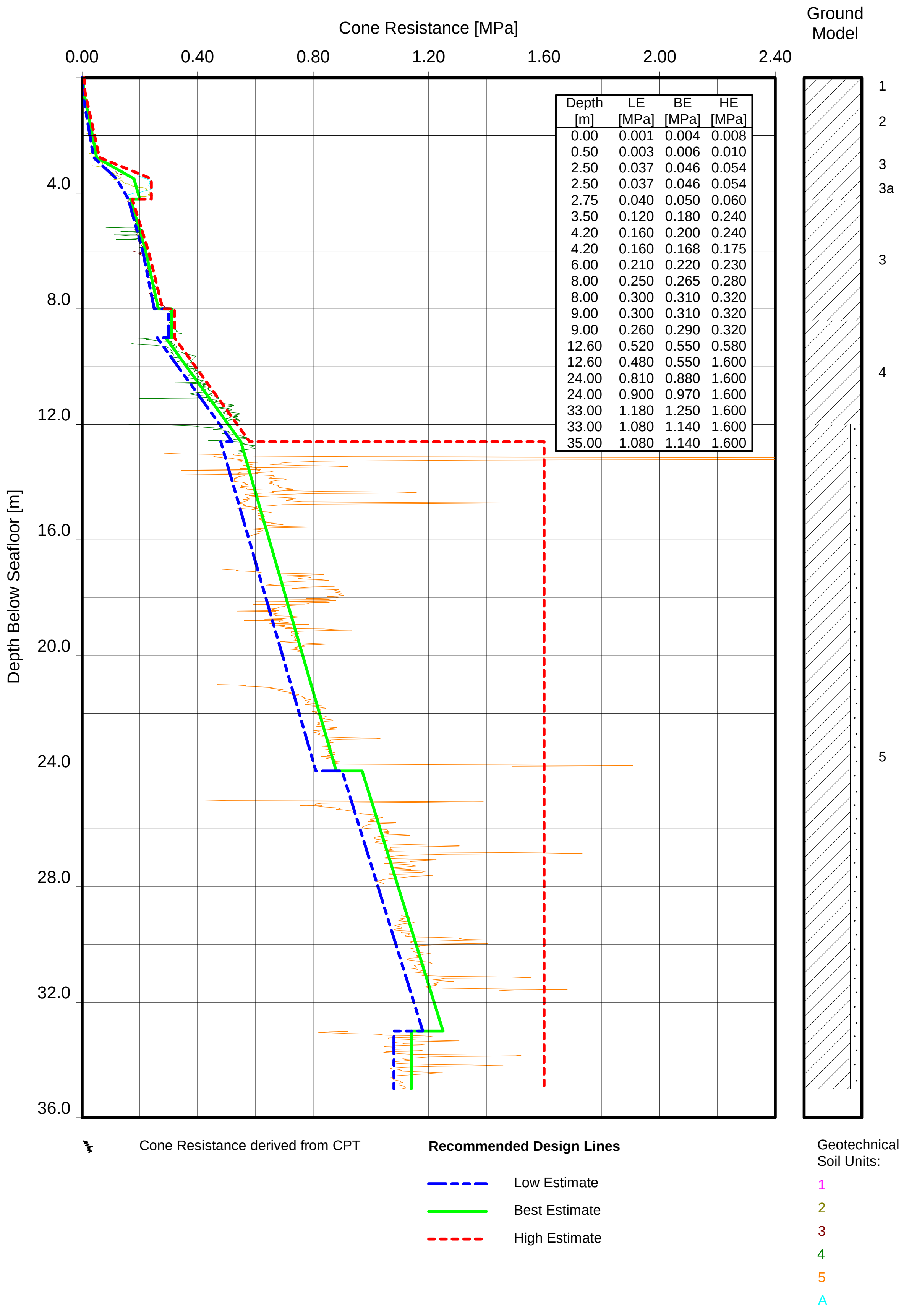
GeODin/7.Liquidity Index vs Depth - Units_v1.02.GLO/2018-06-06 17:47:02

LIQUIDITY INDEX VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



GeODin/7.Liquidity Index vs Depth - Units_v1.02.GLO/2018-06-06 17:47:27

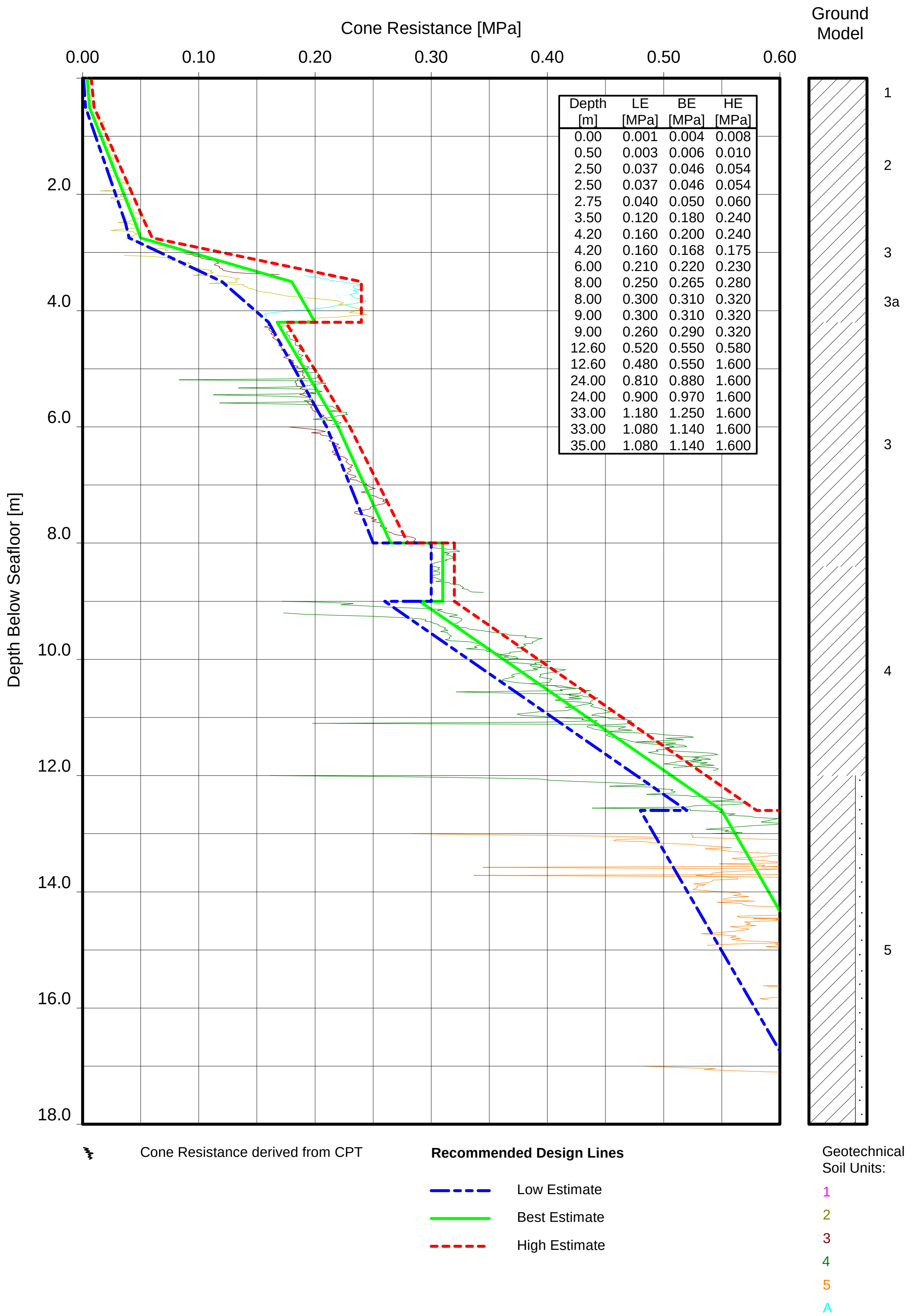
LIQUIDITY INDEX VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [MPa]	BE [MPa]	HE [MPa]
0.00	0.001	0.004	0.008
0.50	0.003	0.006	0.010
2.50	0.037	0.046	0.054
2.50	0.037	0.046	0.054
2.75	0.040	0.050	0.060
3.50	0.120	0.180	0.240
4.20	0.160	0.200	0.240
4.20	0.160	0.168	0.175
6.00	0.210	0.220	0.230
8.00	0.250	0.265	0.280
8.00	0.300	0.310	0.320
9.00	0.300	0.310	0.320
9.00	0.260	0.290	0.320
12.60	0.520	0.550	0.580
12.60	0.480	0.550	1.600
24.00	0.810	0.880	1.600
24.00	0.900	0.970	1.600
33.00	1.180	1.250	1.600
33.00	1.080	1.140	1.600
35.00	1.080	1.140	1.600

GeODin/8.Cone Resistance vs Depth - Units_v1.01.GLO/2018-06-06 17:50:23

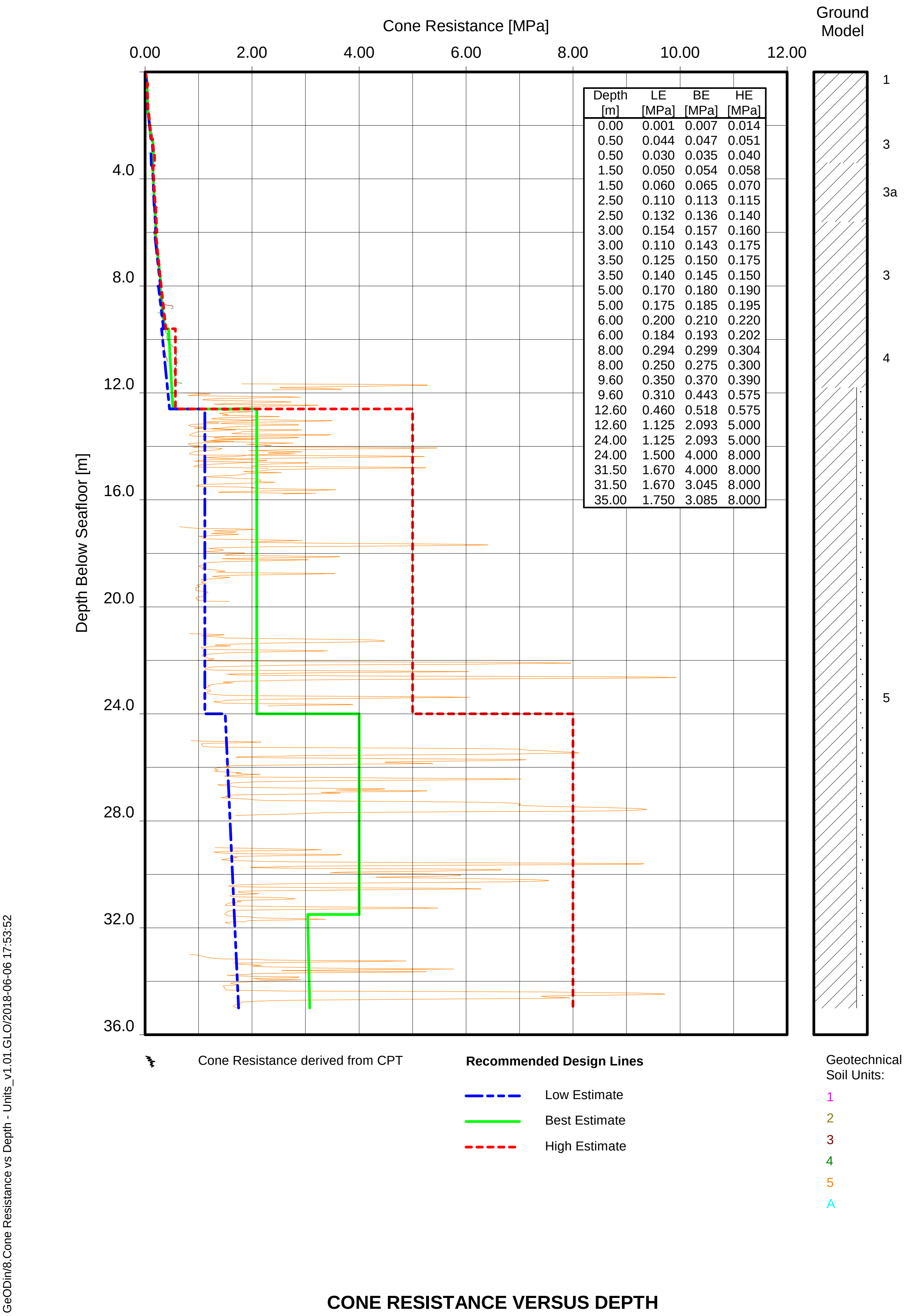
CONE RESISTANCE VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [MPa]	BE [MPa]	HE [MPa]
0.00	0.001	0.004	0.008
0.50	0.003	0.006	0.010
2.50	0.037	0.046	0.054
2.50	0.037	0.046	0.054
2.75	0.040	0.050	0.060
3.50	0.120	0.180	0.240
4.20	0.160	0.200	0.240
4.20	0.160	0.168	0.175
6.00	0.210	0.220	0.230
8.00	0.250	0.265	0.280
8.00	0.300	0.310	0.320
9.00	0.300	0.310	0.320
9.00	0.260	0.290	0.320
12.60	0.520	0.550	0.580
12.60	0.480	0.550	1.600
24.00	0.810	0.880	1.600
24.00	0.900	0.970	1.600
33.00	1.180	1.250	1.600
33.00	1.080	1.140	1.600
35.00	1.080	1.140	1.600

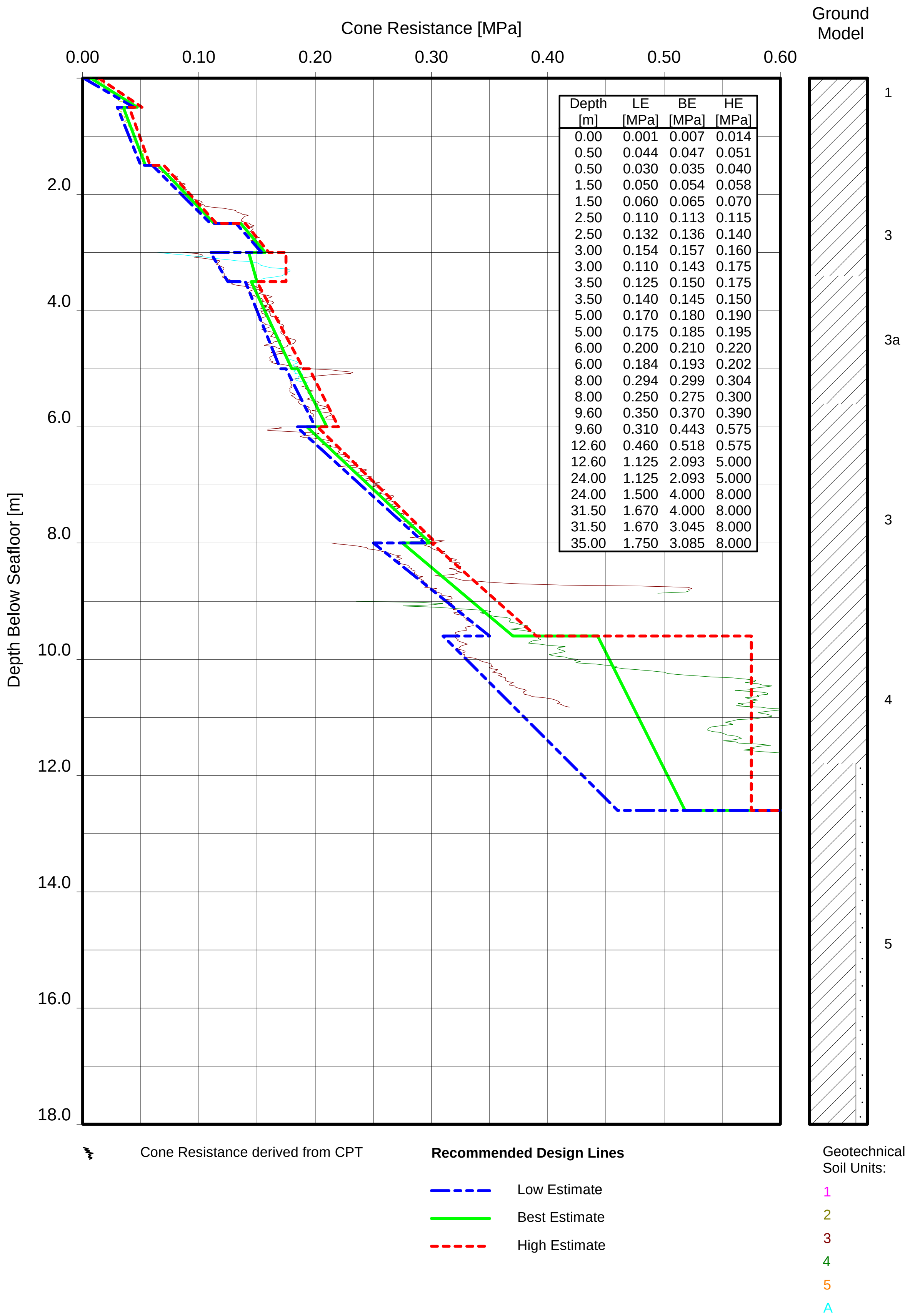
GeODin/8.Cone Resistance vs Depth - Units_v1.01.GLO/2018-06-06 17:52:01

CONE RESISTANCE VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [MPa]	BE [MPa]	HE [MPa]
0.00	0.001	0.007	0.014
0.50	0.044	0.047	0.051
0.50	0.030	0.035	0.040
1.50	0.050	0.054	0.058
1.50	0.060	0.065	0.070
2.50	0.110	0.113	0.115
2.50	0.132	0.136	0.140
3.00	0.154	0.157	0.160
3.00	0.110	0.143	0.175
3.50	0.125	0.150	0.175
3.50	0.140	0.145	0.150
5.00	0.170	0.180	0.190
5.00	0.175	0.185	0.195
6.00	0.200	0.210	0.220
6.00	0.184	0.193	0.202
8.00	0.294	0.299	0.304
8.00	0.250	0.275	0.300
9.60	0.350	0.370	0.390
9.60	0.310	0.443	0.575
12.60	0.460	0.518	0.575
12.60	1.125	2.093	5.000
24.00	1.125	2.093	5.000
24.00	1.500	4.000	8.000
31.50	1.670	4.000	8.000
31.50	1.670	3.045	8.000
35.00	1.750	3.085	8.000

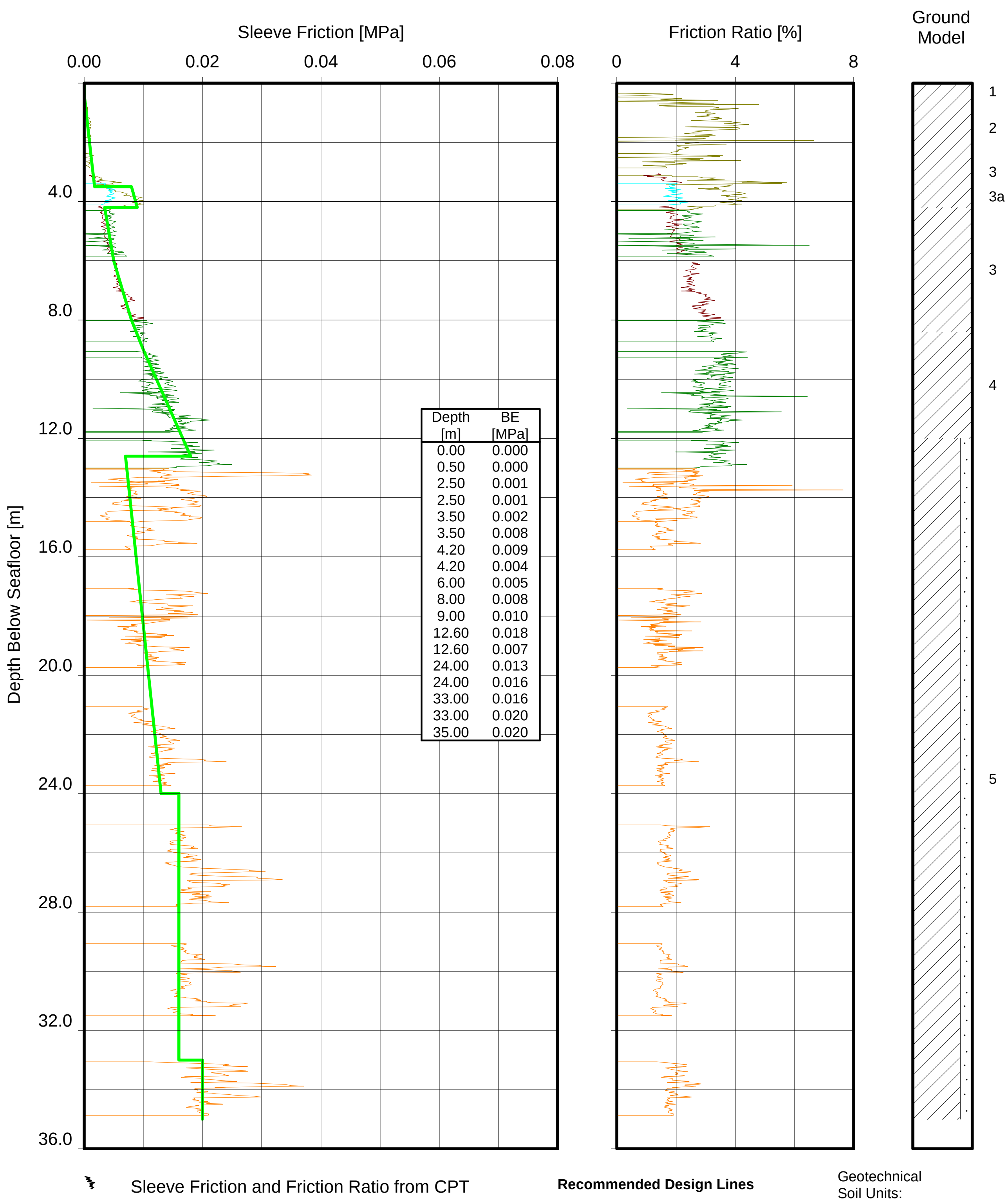
CONE RESISTANCE VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [MPa]	BE [MPa]	HE [MPa]
0.00	0.001	0.007	0.014
0.50	0.044	0.047	0.051
0.50	0.030	0.035	0.040
1.50	0.050	0.054	0.058
1.50	0.060	0.065	0.070
2.50	0.110	0.113	0.115
2.50	0.132	0.136	0.140
3.00	0.154	0.157	0.160
3.00	0.110	0.143	0.175
3.50	0.125	0.150	0.175
3.50	0.140	0.145	0.150
5.00	0.170	0.180	0.190
5.00	0.175	0.185	0.195
6.00	0.200	0.210	0.220
6.00	0.184	0.193	0.202
8.00	0.294	0.299	0.304
8.00	0.250	0.275	0.300
9.60	0.350	0.370	0.390
9.60	0.310	0.443	0.575
12.60	0.460	0.518	0.575
12.60	1.125	2.093	5.000
24.00	1.125	2.093	5.000
24.00	1.500	4.000	8.000
31.50	1.670	4.000	8.000
31.50	1.670	3.045	8.000
35.00	1.750	3.085	8.000

GeODin/8.Cone Resistance vs Depth - Units_v1.01.GLO/2018-06-06 17:52:49

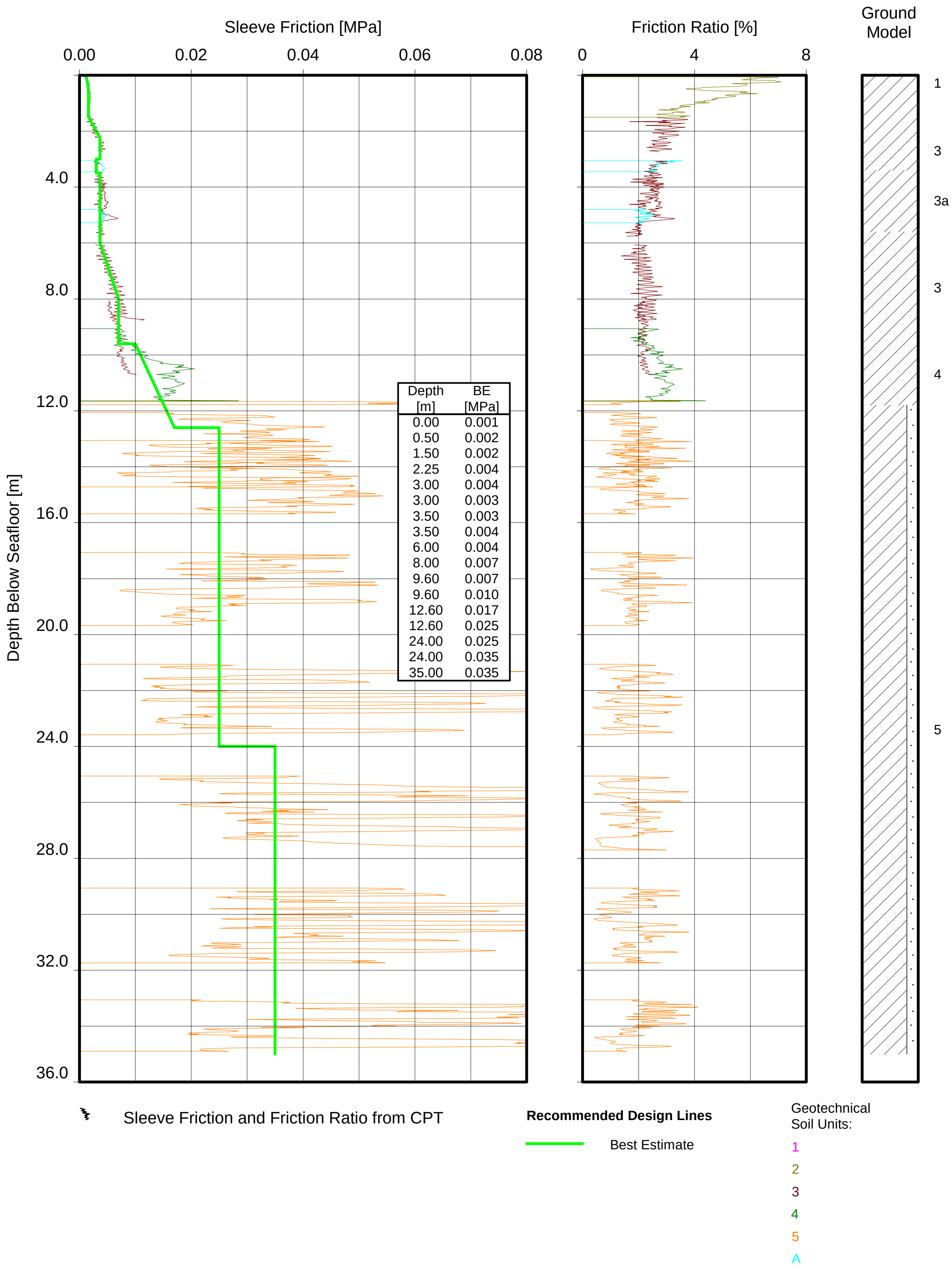
CONE RESISTANCE VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	BE [MPa]
0.00	0.000
0.50	0.000
2.50	0.001
2.50	0.001
3.50	0.002
3.50	0.008
4.20	0.009
4.20	0.004
6.00	0.005
8.00	0.008
9.00	0.010
12.60	0.018
12.60	0.007
24.00	0.013
24.00	0.016
33.00	0.016
33.00	0.020
35.00	0.020

GeODin/Sleeve Friction.GLO/2018-06-06 17:55:39

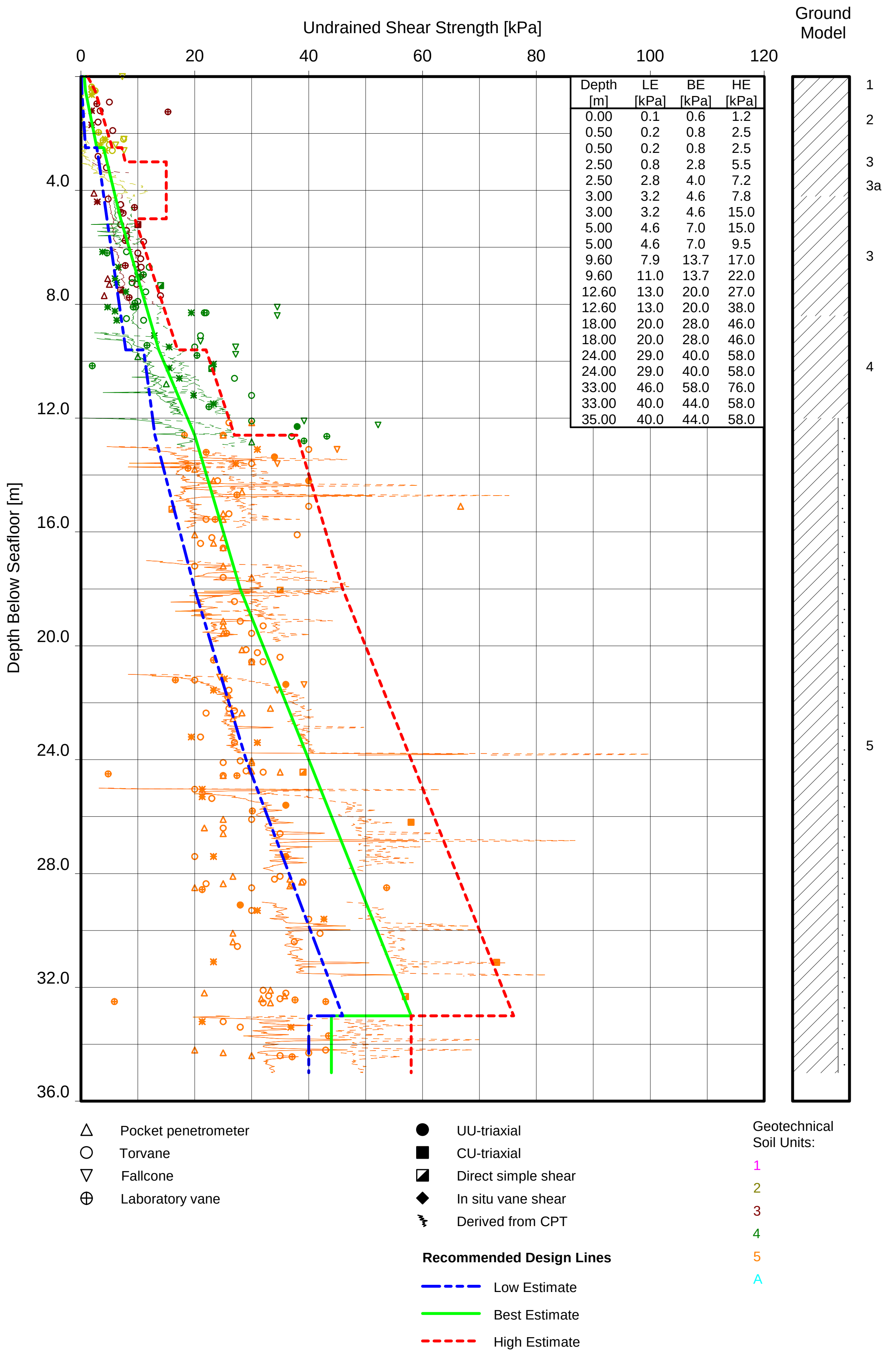
SLEEVE FRICTION AND FRICTION RATIO VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	BE [MPa]
0.00	0.001
0.50	0.002
1.50	0.002
2.25	0.004
3.00	0.004
3.00	0.003
3.50	0.003
3.50	0.004
6.00	0.004
8.00	0.007
9.60	0.007
9.60	0.010
12.60	0.017
12.60	0.025
24.00	0.025
24.00	0.035
35.00	0.035

GeODin/Sleeve Friction.GLO/2018-06-06 17:56:14

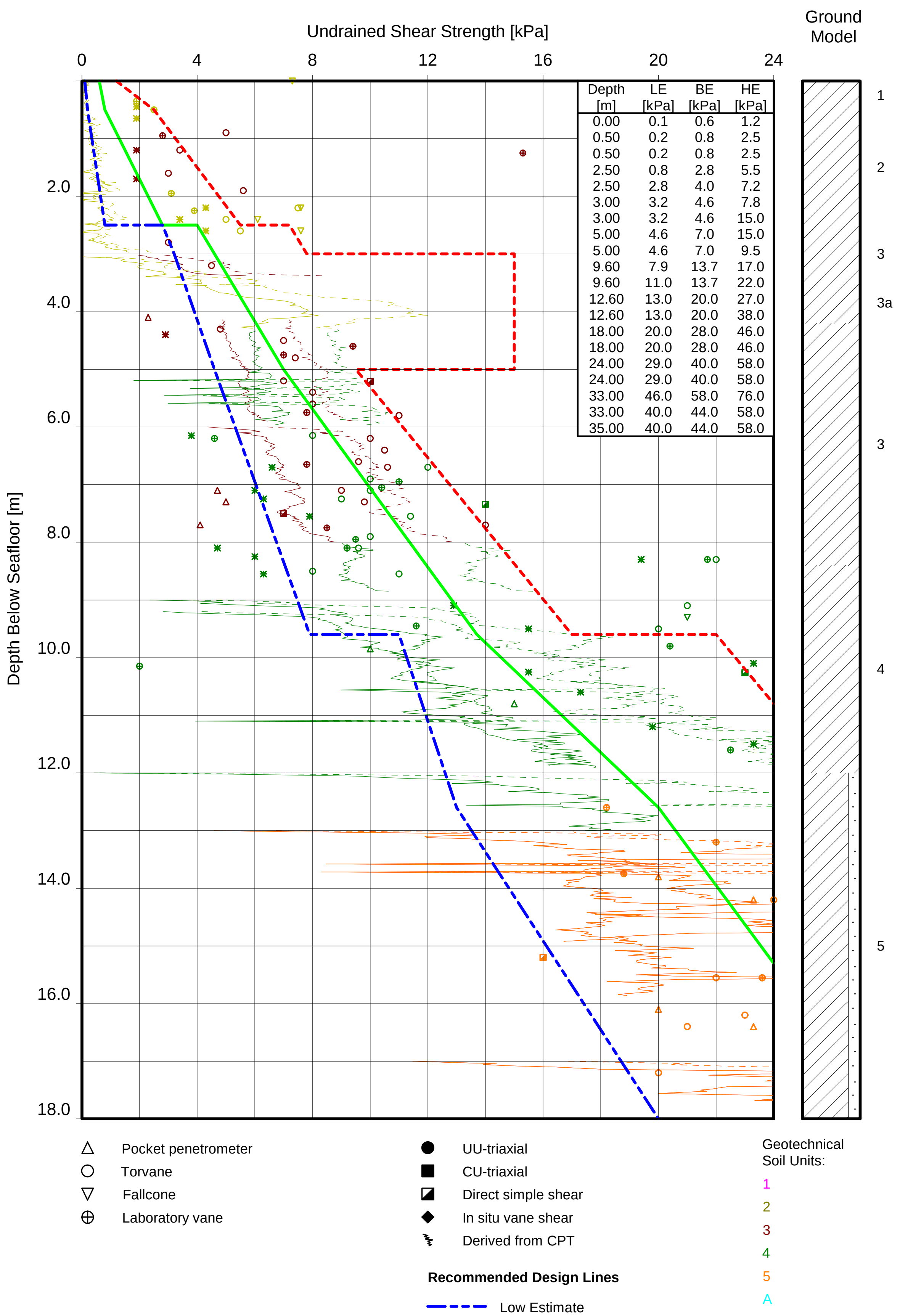
SLEEVE FRICTION AND FRICTION RATIO VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.1	0.6	1.2
0.50	0.2	0.8	2.5
0.50	0.2	0.8	2.5
2.50	0.8	2.8	5.5
2.50	2.8	4.0	7.2
3.00	3.2	4.6	7.8
3.00	3.2	4.6	15.0
5.00	4.6	7.0	15.0
5.00	4.6	7.0	9.5
9.60	7.9	13.7	17.0
9.60	11.0	13.7	22.0
12.60	13.0	20.0	27.0
12.60	13.0	20.0	38.0
18.00	20.0	28.0	46.0
18.00	20.0	28.0	46.0
24.00	29.0	40.0	58.0
24.00	29.0	40.0	58.0
33.00	46.0	58.0	76.0
33.00	40.0	44.0	58.0
35.00	40.0	44.0	58.0

GeODin/9.Undrained Shear Strength versus Depth - Units_v1.01.GLO/2018-06-12 20:41:17

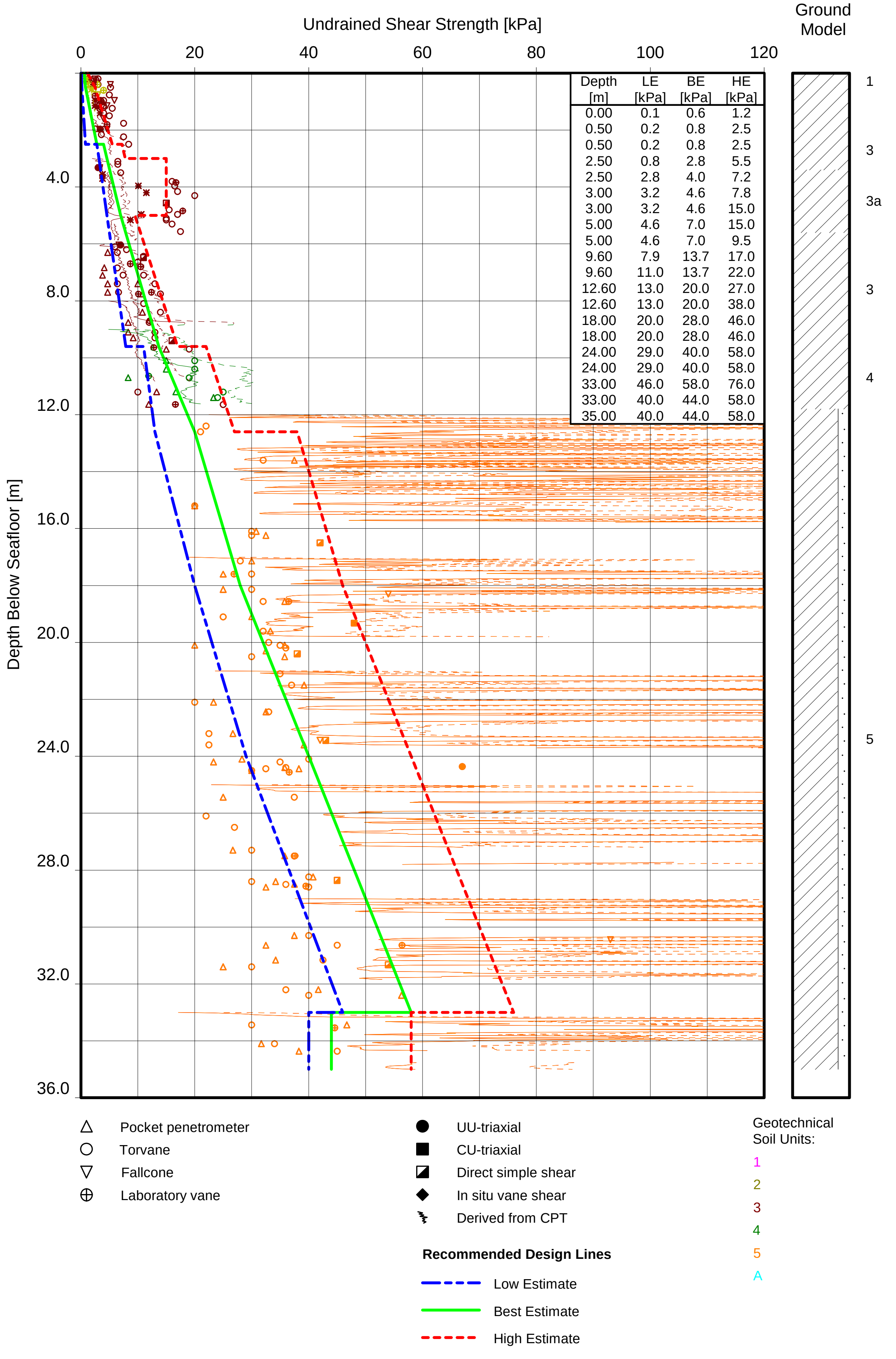
UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.1	0.6	1.2
0.50	0.2	0.8	2.5
0.50	0.2	0.8	2.5
2.50	0.8	2.8	5.5
2.50	2.8	4.0	7.2
3.00	3.2	4.6	7.8
3.00	3.2	4.6	15.0
5.00	4.6	7.0	15.0
5.00	4.6	7.0	9.5
9.60	7.9	13.7	17.0
9.60	11.0	13.7	22.0
12.60	13.0	20.0	27.0
12.60	13.0	20.0	38.0
18.00	20.0	28.0	46.0
18.00	20.0	28.0	46.0
24.00	29.0	40.0	58.0
24.00	29.0	40.0	58.0
33.00	46.0	58.0	76.0
33.00	40.0	44.0	58.0
35.00	40.0	44.0	58.0

GeODin/9.Undrained Shear Strength versus Depth - Units_v1.01.GLO/2018-06-12 20:42:59

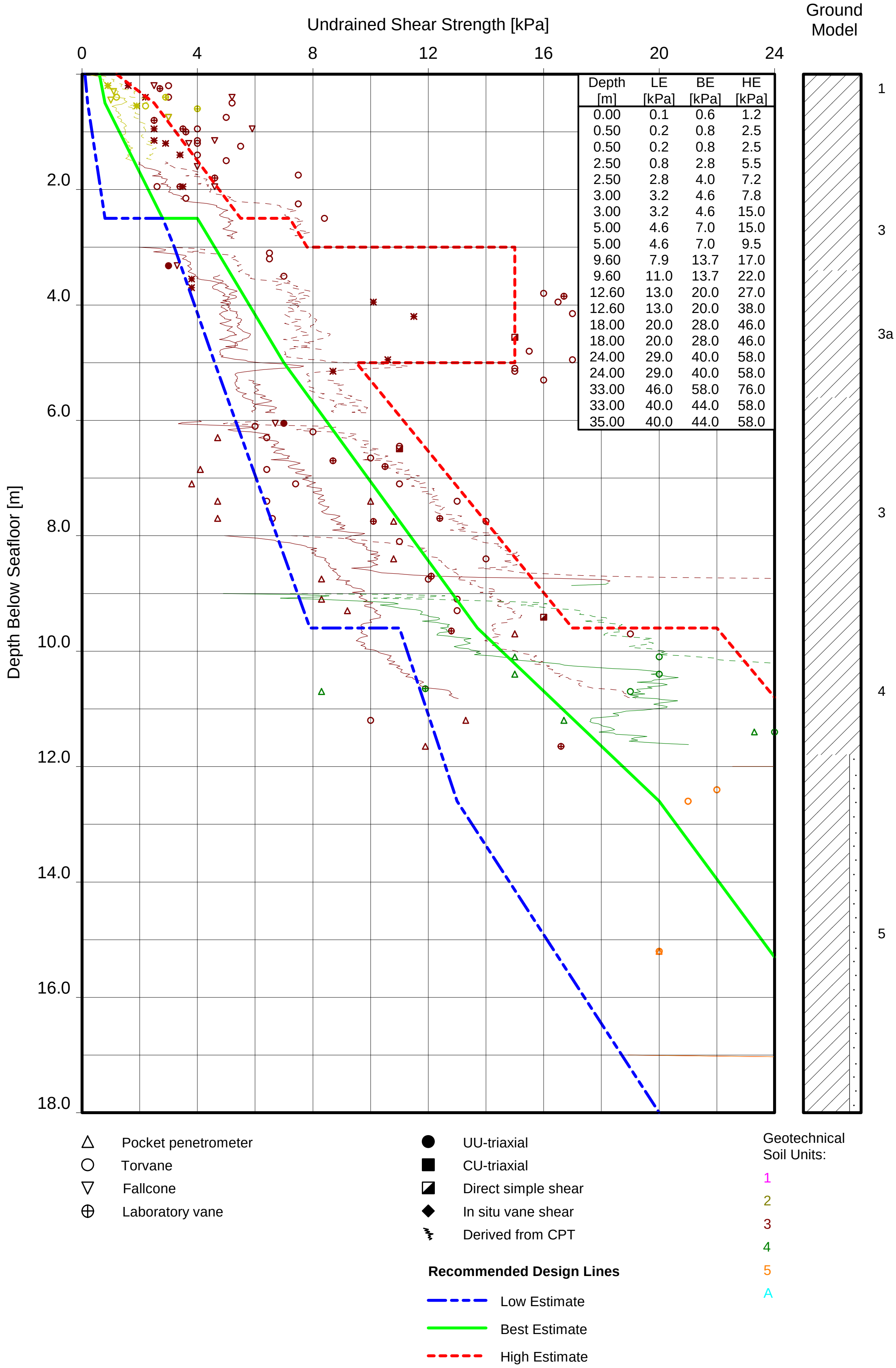
UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.1	0.6	1.2
0.50	0.2	0.8	2.5
0.50	0.2	0.8	2.5
2.50	0.8	2.8	5.5
2.50	2.8	4.0	7.2
3.00	3.2	4.6	7.8
3.00	3.2	4.6	15.0
5.00	4.6	7.0	15.0
5.00	4.6	7.0	9.5
9.60	7.9	13.7	17.0
9.60	11.0	13.7	22.0
12.60	13.0	20.0	27.0
12.60	13.0	20.0	38.0
18.00	20.0	28.0	46.0
18.00	20.0	28.0	46.0
24.00	29.0	40.0	58.0
24.00	29.0	40.0	58.0
33.00	46.0	58.0	76.0
33.00	40.0	44.0	58.0
35.00	40.0	44.0	58.0

GeODin/9.Undrained Shear Strength versus Depth - Units_v1.01.GLO/2018-06-12 20:41:58

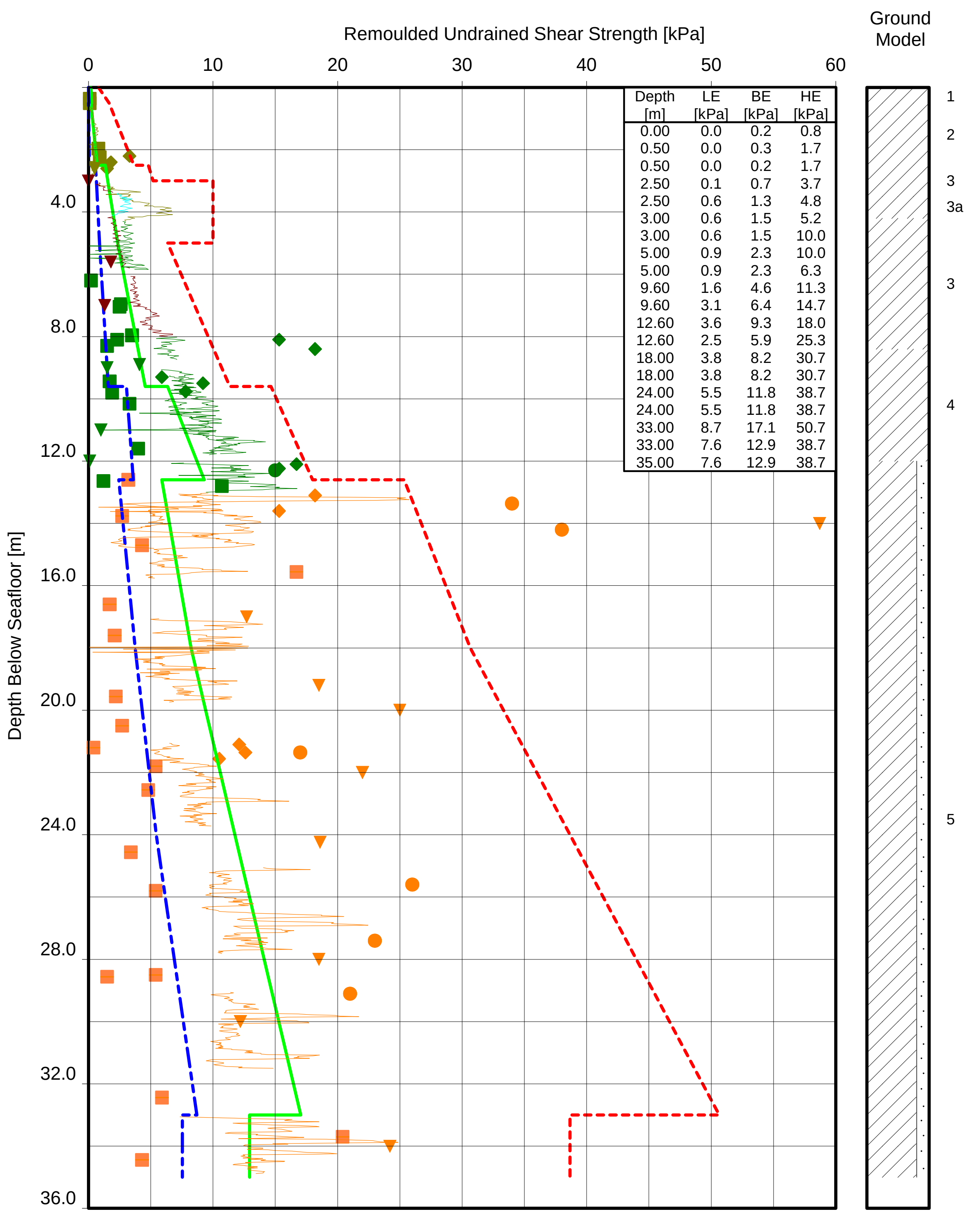
UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.1	0.6	1.2
0.50	0.2	0.8	2.5
0.50	0.2	0.8	2.5
2.50	0.8	2.8	5.5
2.50	2.8	4.0	7.2
3.00	3.2	4.6	7.8
3.00	3.2	4.6	15.0
5.00	4.6	7.0	15.0
5.00	4.6	7.0	9.5
9.60	7.9	13.7	17.0
9.60	11.0	13.7	22.0
12.60	13.0	20.0	27.0
12.60	13.0	20.0	38.0
18.00	20.0	28.0	46.0
18.00	20.0	28.0	46.0
24.00	29.0	40.0	58.0
24.00	29.0	40.0	58.0
33.00	46.0	58.0	76.0
33.00	40.0	44.0	58.0
35.00	40.0	44.0	58.0

GeODin/9.Undrained Shear Strength versus Depth - Units_v1.01.GLO/2018-06-12 20:43:40

UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.0	0.2	0.8
0.50	0.0	0.3	1.7
0.50	0.0	0.2	1.7
2.50	0.1	0.7	3.7
2.50	0.6	1.3	4.8
3.00	0.6	1.5	5.2
3.00	0.6	1.5	10.0
5.00	0.9	2.3	10.0
5.00	0.9	2.3	6.3
9.60	1.6	4.6	11.3
9.60	3.1	6.4	14.7
12.60	3.6	9.3	18.0
12.60	2.5	5.9	25.3
18.00	3.8	8.2	30.7
18.00	3.8	8.2	30.7
24.00	5.5	11.8	38.7
24.00	5.5	11.8	38.7
33.00	8.7	17.1	50.7
33.00	7.6	12.9	38.7
35.00	7.6	12.9	38.7

Key of Symbols:

- ◆ Fallcone
- ■ Laboratory Vane
- ● UU-triaxial
- ▼ Liquidity Index

In situ data:

2/3 Sleeve friction (fs) - Cone penetration test (CPT) data

Note(s):
For the calculation of suR from the liquidity index, a representative value from the water content design profile was used.

Recommended Characteristic Profiles

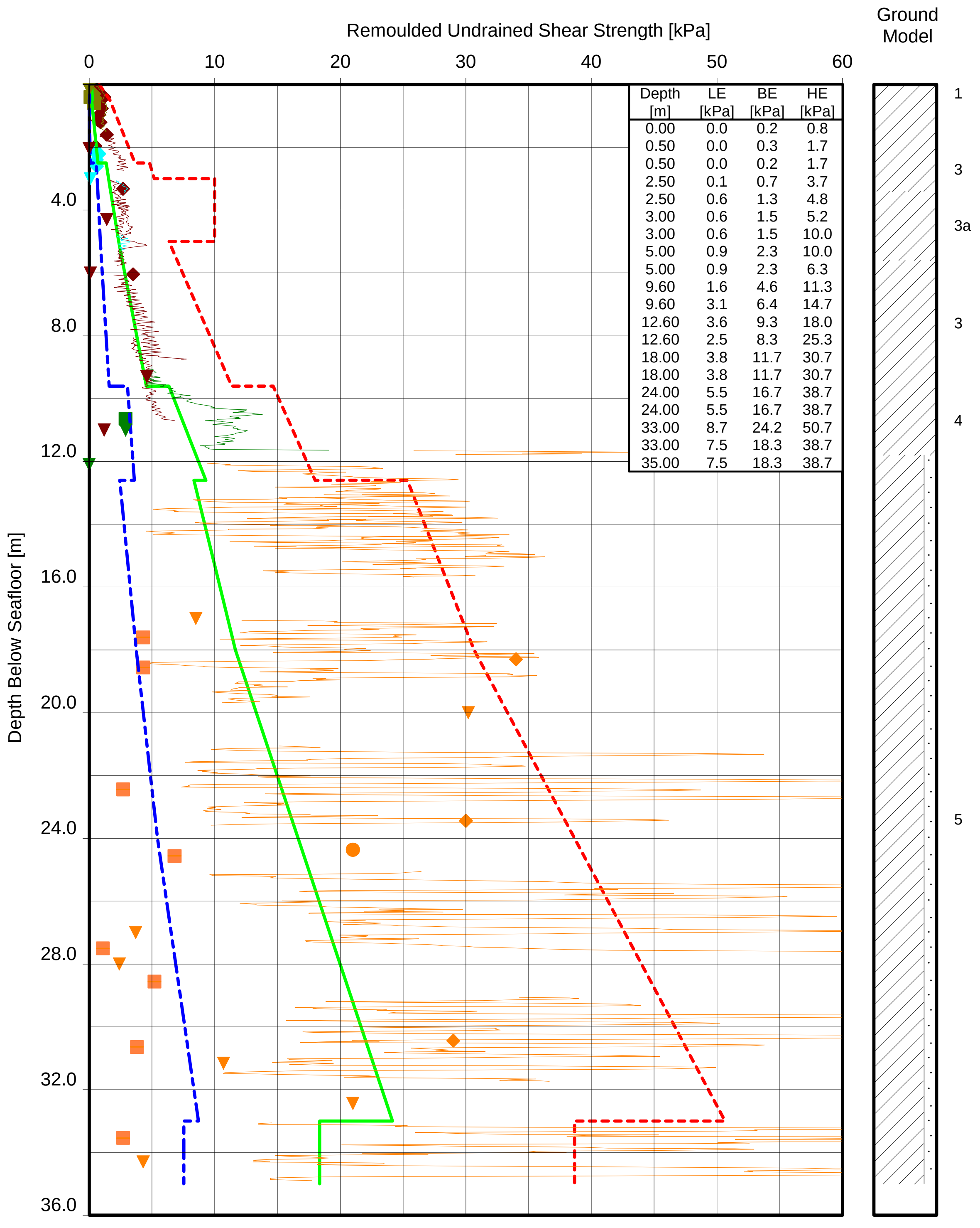
- Low Estimate
- Best Estimate
- High Estimate

Geotechnical Soil Units:

1
2
3
4
5
A

GeODin/SuR.GLO/2018-06-13 15:12:48

REMOULDED UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [kPa]	BE [kPa]	HE [kPa]
0.00	0.0	0.2	0.8
0.50	0.0	0.3	1.7
0.50	0.0	0.2	1.7
2.50	0.1	0.7	3.7
2.50	0.6	1.3	4.8
3.00	0.6	1.5	5.2
3.00	0.6	1.5	10.0
5.00	0.9	2.3	10.0
5.00	0.9	2.3	6.3
9.60	1.6	4.6	11.3
9.60	3.1	6.4	14.7
12.60	3.6	9.3	18.0
12.60	2.5	8.3	25.3
18.00	3.8	11.7	30.7
18.00	3.8	11.7	30.7
24.00	5.5	16.7	38.7
24.00	5.5	16.7	38.7
33.00	8.7	24.2	50.7
33.00	7.5	18.3	38.7
35.00	7.5	18.3	38.7

Key of Symbols:

- ◆ Fallcone
- ■ Laboratory Vane
- ● UU-triaxial
- ▼ Liquidity Index

In situ data:

2/3 Sleeve friction (fs) - Cone penetration test (CPT) data

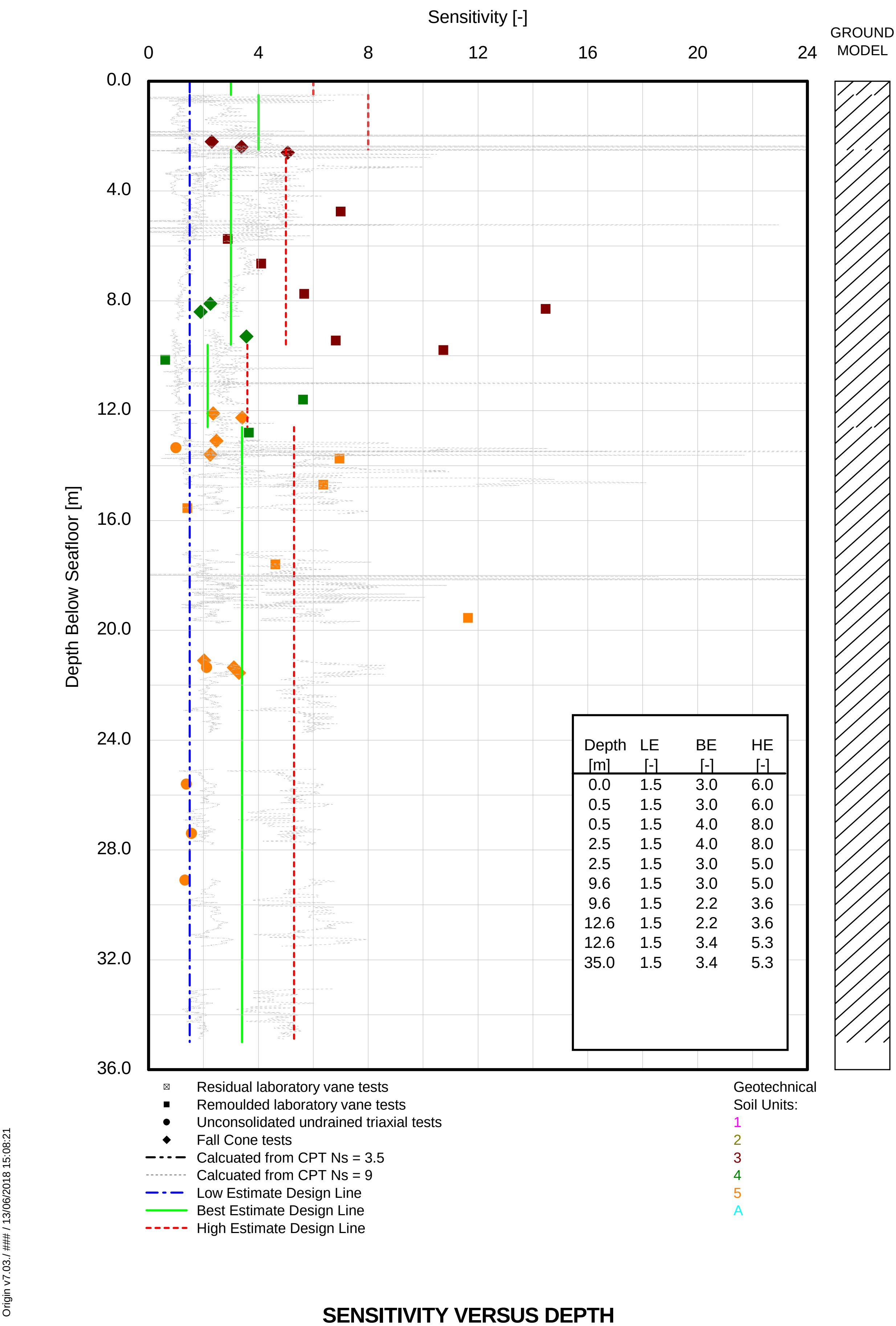
Note(s):
For the calculation of suR from the liquidity index, a representative value from the water content design profile was used.

Recommended Characteristic Profiles

- Low Estimate
- Best Estimate
- High Estimate

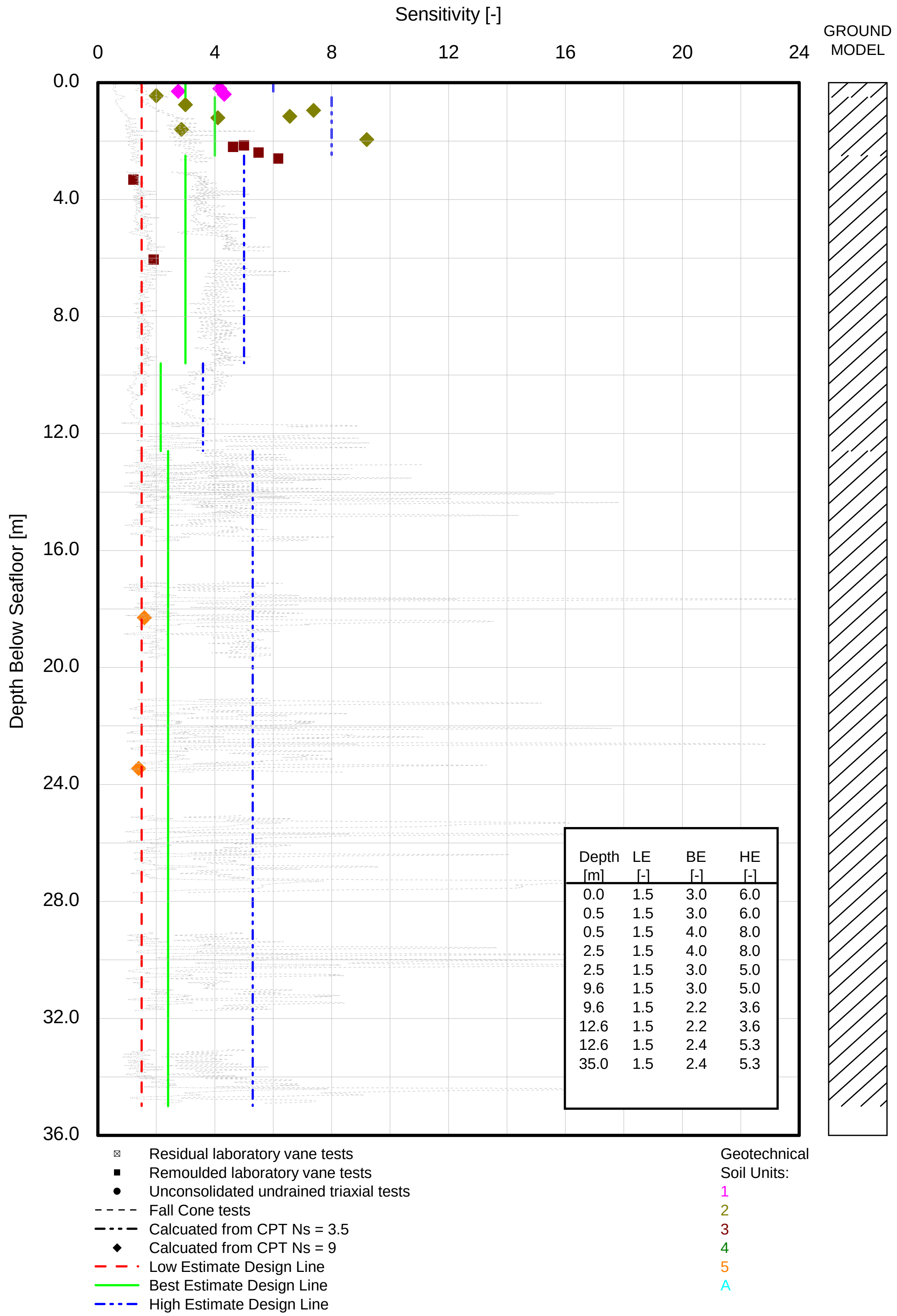
GeODin/SuR.GLO/2018-06-13 15:15:14

REMOULDED UNDRAINED SHEAR STRENGTH VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



Depth [m]	LE [-]	BE [-]	HE [-]
0.0	1.5	3.0	6.0
0.5	1.5	3.0	6.0
0.5	1.5	4.0	8.0
2.5	1.5	4.0	8.0
2.5	1.5	3.0	5.0
9.6	1.5	3.0	5.0
9.6	1.5	2.2	3.6
12.6	1.5	2.2	3.6
12.6	1.5	3.4	5.3
35.0	1.5	3.4	5.3

SENSITIVITY VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



Depth [m]	LE [-]	BE [-]	HE [-]
0.0	1.5	3.0	6.0
0.5	1.5	3.0	6.0
0.5	1.5	4.0	8.0
2.5	1.5	4.0	8.0
2.5	1.5	3.0	5.0
9.6	1.5	3.0	5.0
9.6	1.5	2.2	3.6
12.6	1.5	2.2	3.6
12.6	1.5	2.4	5.3
35.0	1.5	2.4	5.3

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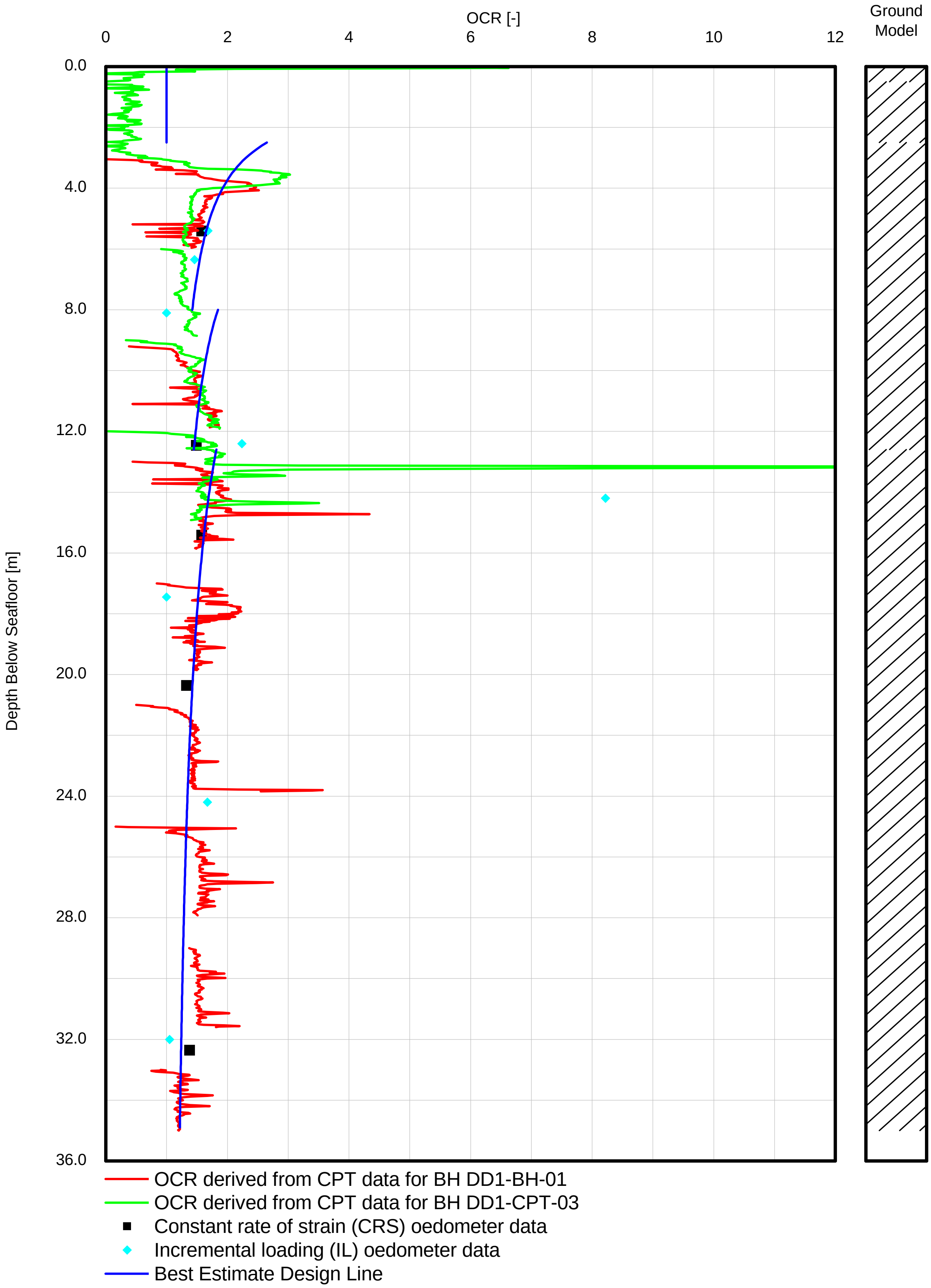
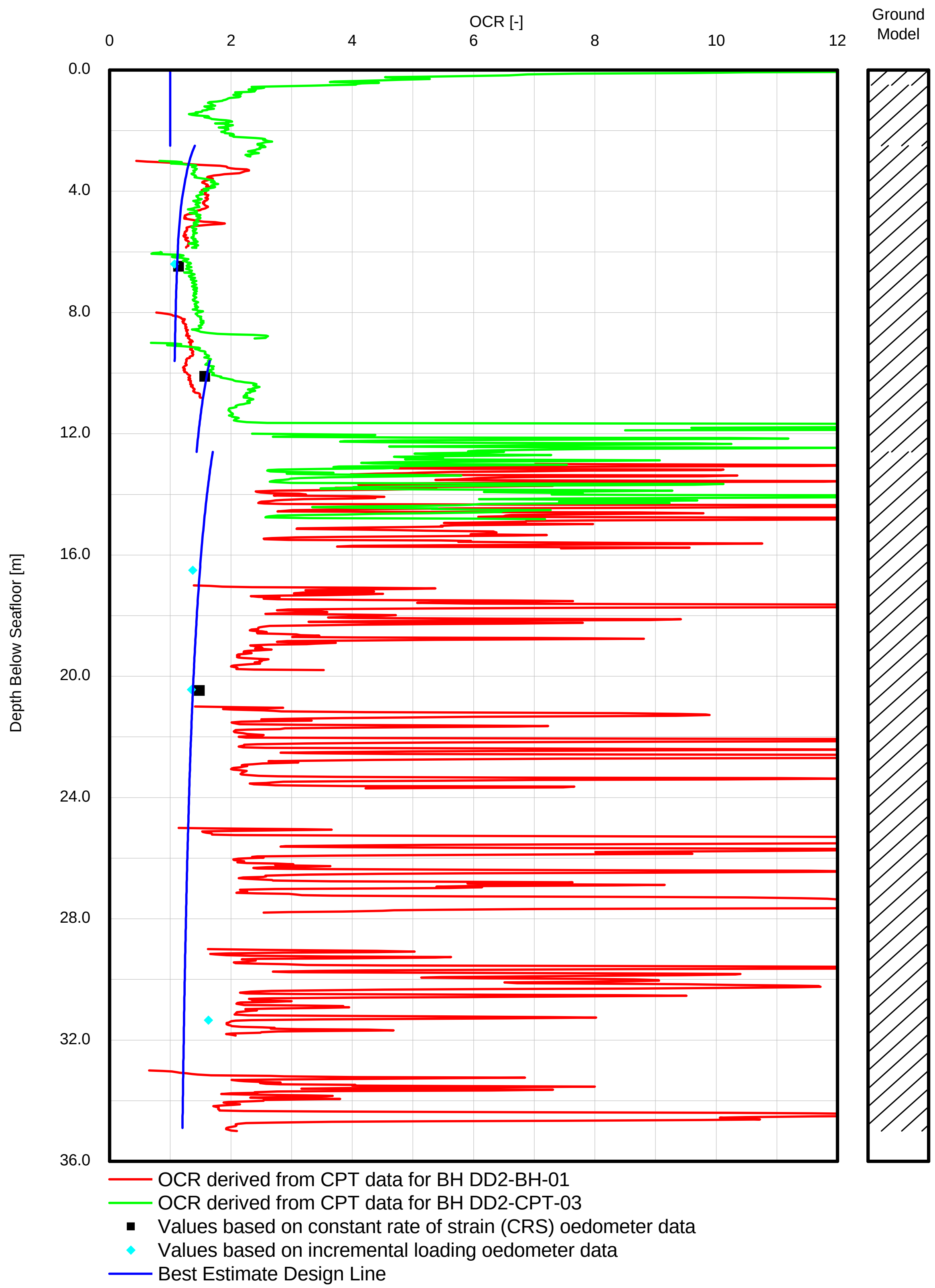
SENSITIVITY VERSUS DEPTH
Domino Drill Center 2, Neptun Deep Survey



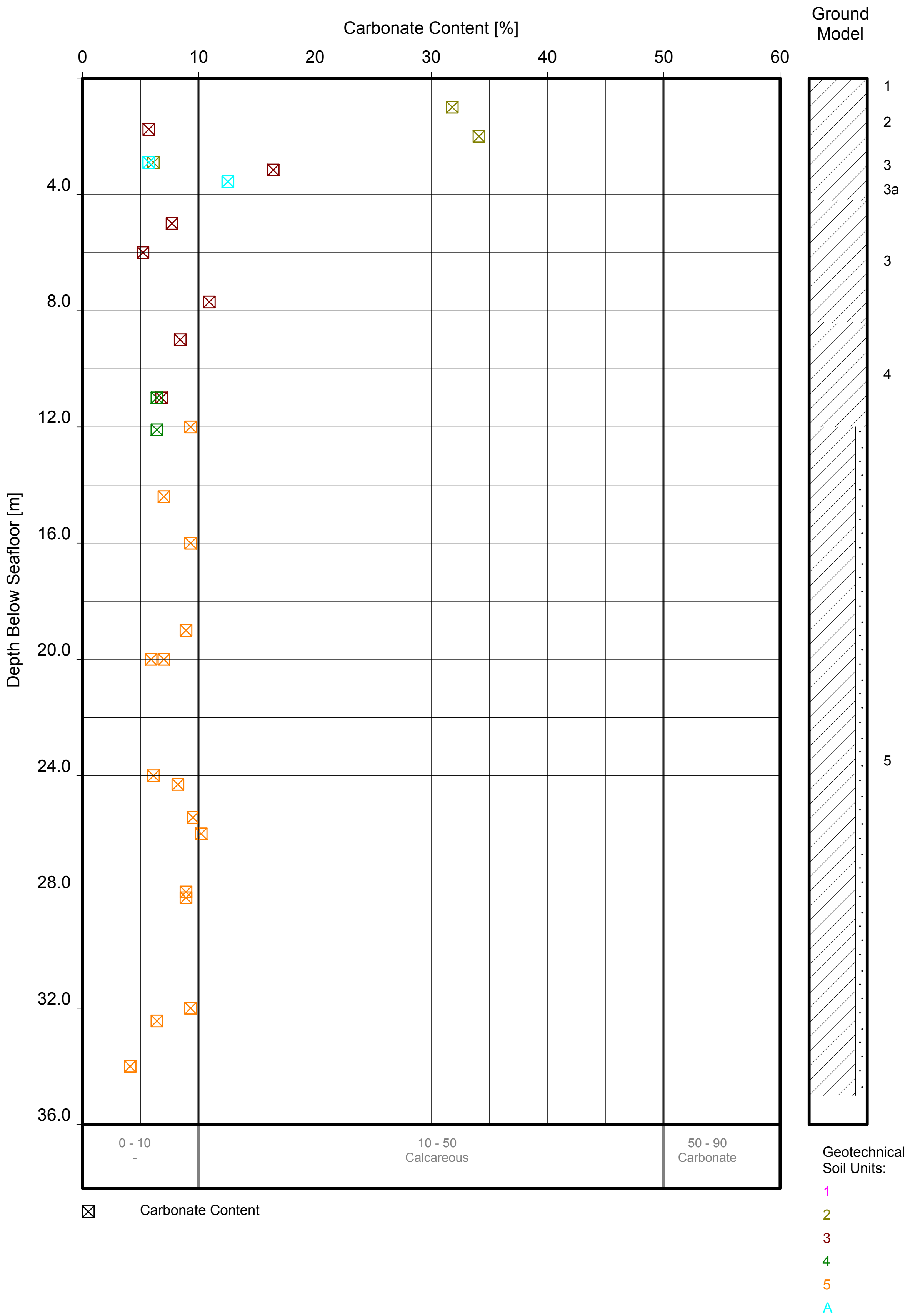
Note:
IL oedometer test at 14.20 m is considered an outlier
and not used in the derivation of a best estimate design line

OVERCONSOLIDATION RATIO (OCR) VERSUS DEPTH
Domino Drill Center 1, Neptun Deep Survey



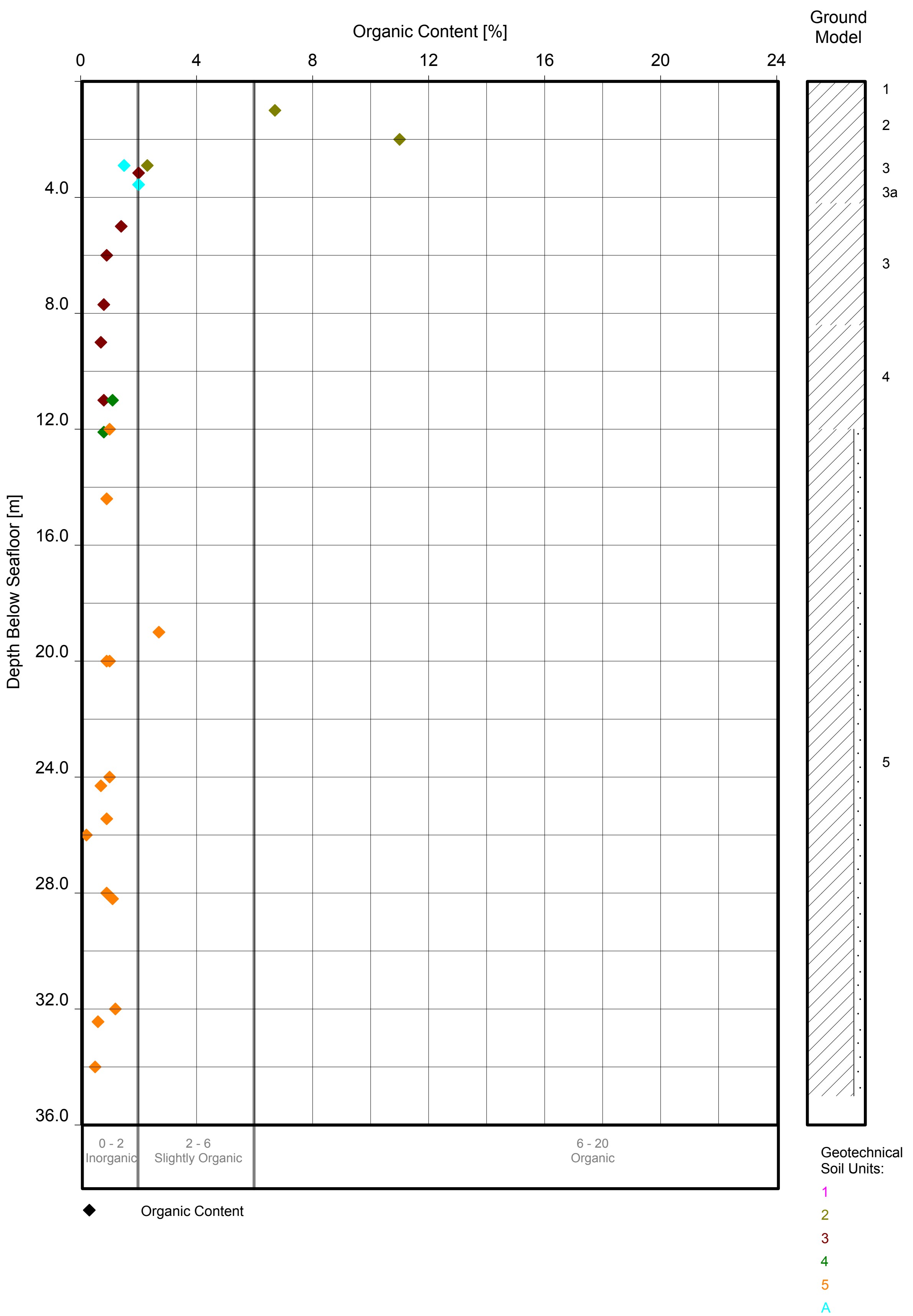
OVERCONSOLIDATION RATIO (OCR) VERSUS DEPTH

Domino Drill Center 2, Neptun Deep Survey



GeODin/Carbonate Content.GLO/2018-05-22 16:16:04

CARBONATE CONTENT VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey



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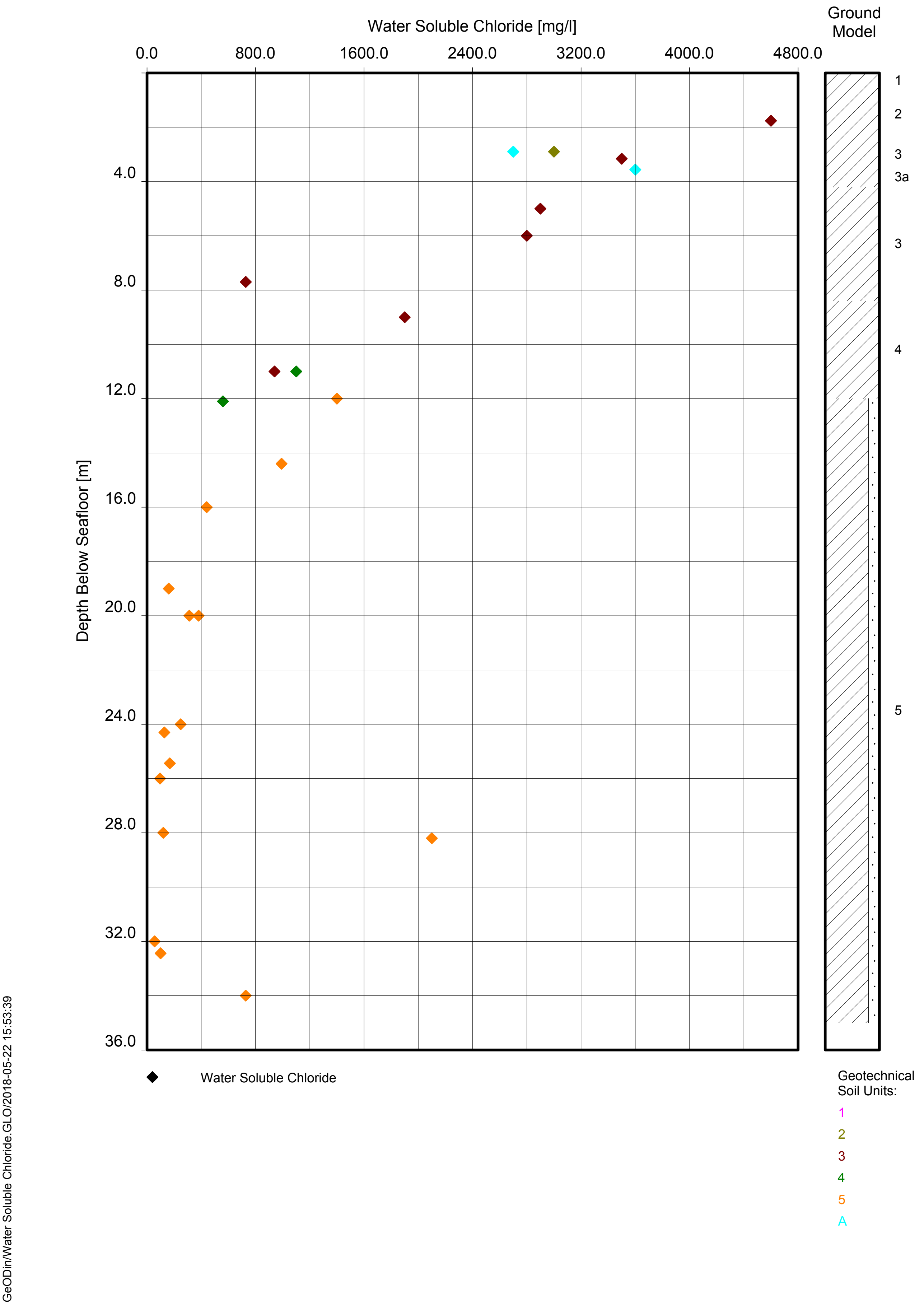
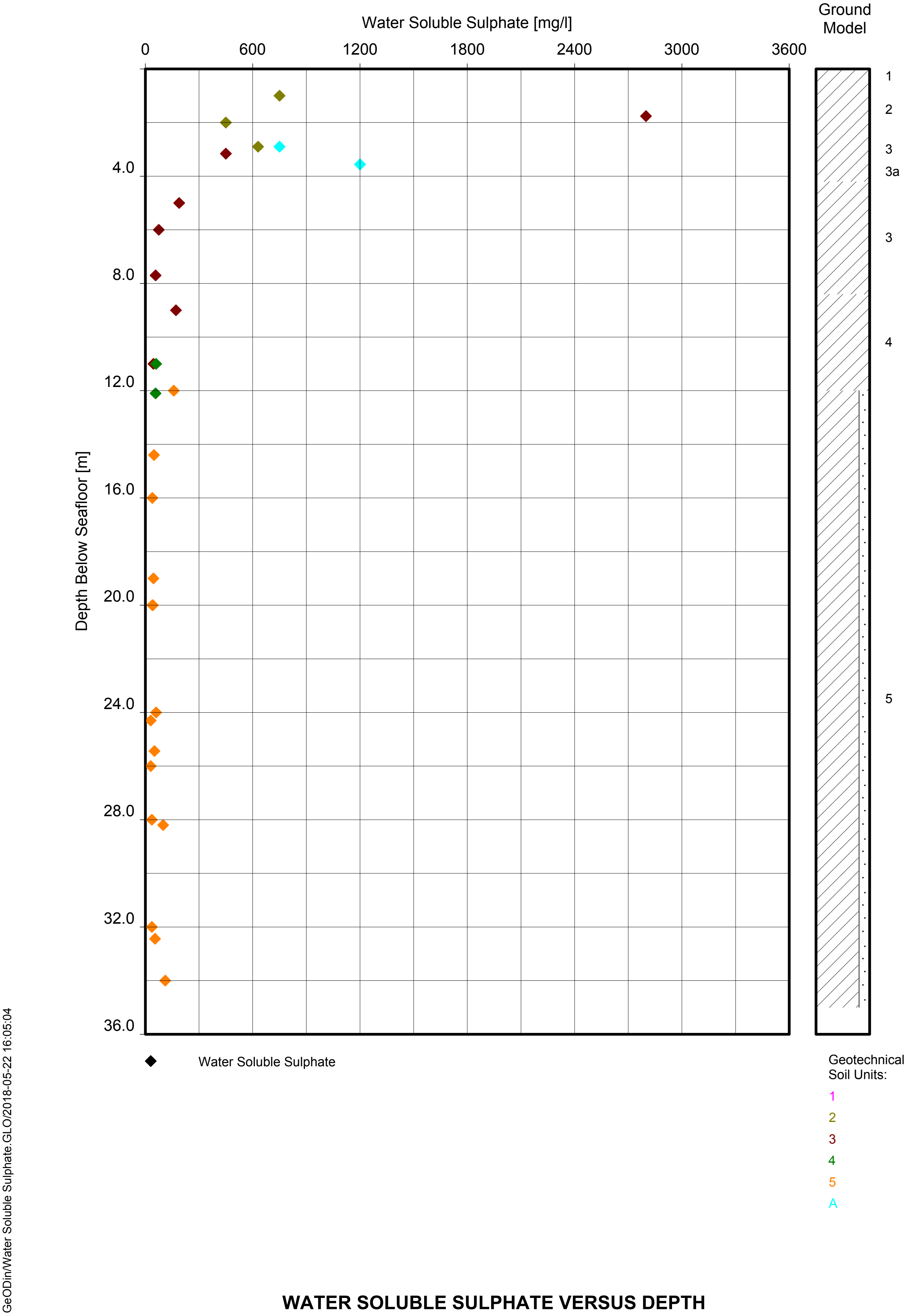
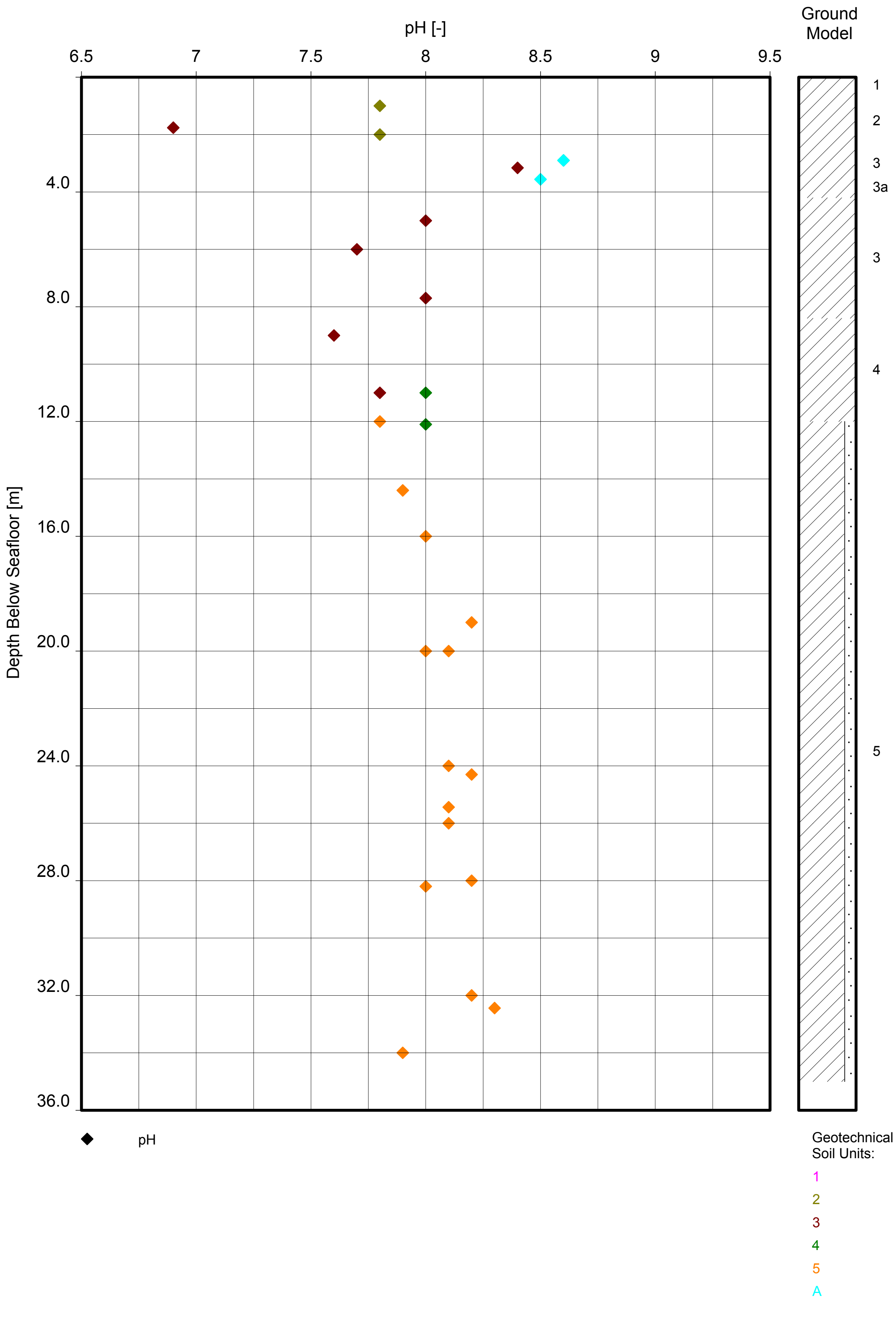
ORGANIC CONTENT VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey



WATER SOLUBLE CHLORIDE VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey

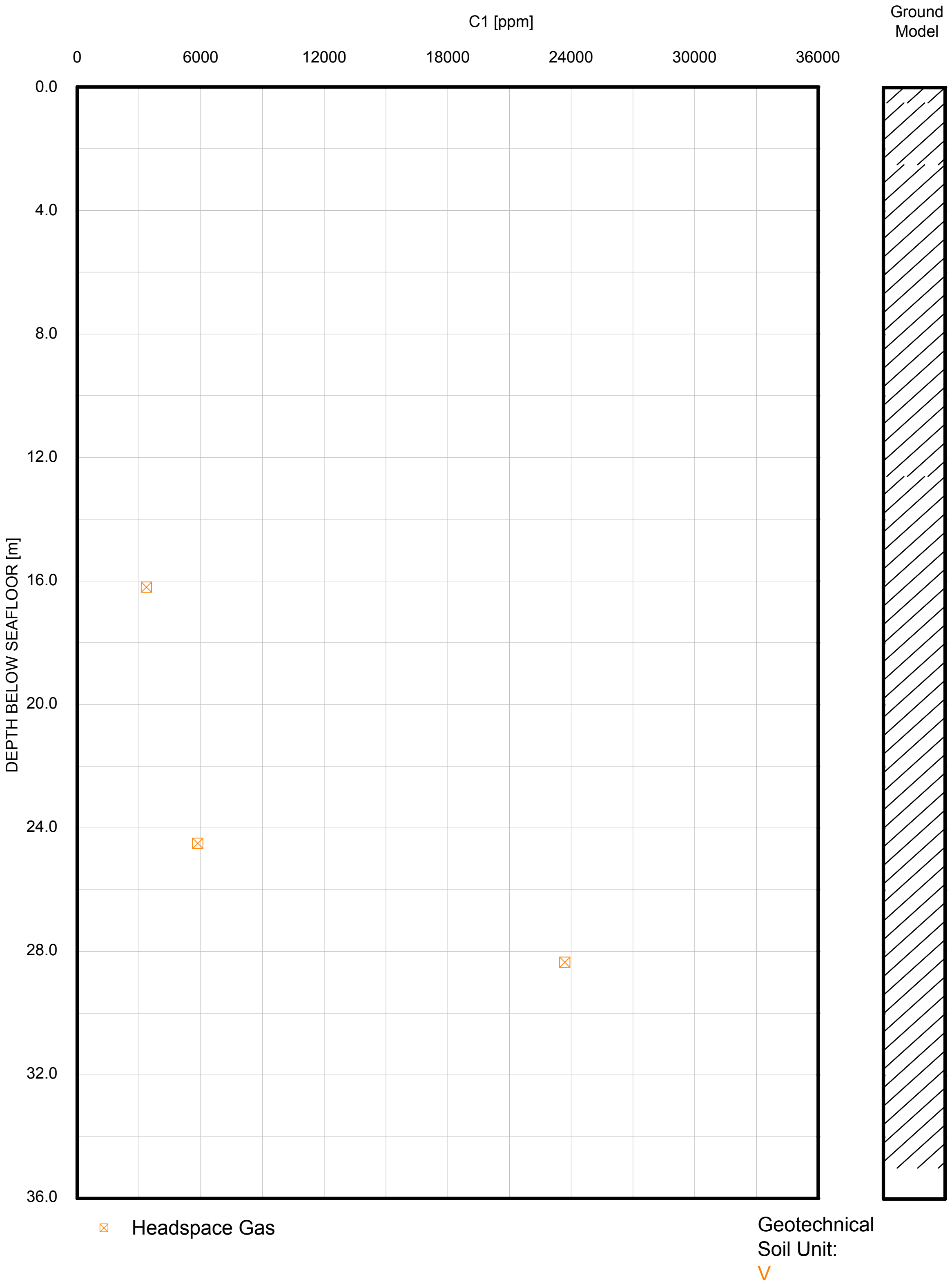


WATER SOLUBLE SULPHATE VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey



GeODin/pH.GLO/2018-05-22 16:11:24

pH VERSUS DEPTH
Domino Drill Center 1 and 2, Neptun Deep Survey



HEADSPACE GAS VERSUS DEPTH
C1 Parts Per Million
Domino Drill Center 1 and 2, Neptun Deep Survey

Depth	Soil Description	Water Content [%]			Unit Weight [kN/m3]			Plasticity Index [%]			Measured Cone Resistance (q_c) [Mpa]			Undrained Shear Strength [kPa]			Sensitivity [-]			Remoulded Undrained Shear Strength [kPa]		
		LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE
0.00	Extremely low strength CLAY	182.3	370.7	559.0	10.5	11.2	11.7	51.8	104.7	157.5	0.00	0.00	0.01	0.1	0.6	1.2	1.5	3.0	6.0	0.0	0.2	0.8
0.50		182.3	370.7	559.0	10.5	11.2	11.7	51.8	104.7	157.5	0.00	0.01	0.01	0.2	0.8	2.5	1.5	3.0	6.0	0.0	0.3	1.7
0.50	Extremely low strength CLAY (Organic rich sapropel)	125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.00	0.01	0.01	0.2	0.8	2.5	1.5	4.0	8.0	0.0	0.2	1.7
2.50		125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.04	0.05	0.05	0.8	2.8	5.5	1.5	4.0	8.0	0.1	0.7	3.7
2.50	Extremely low strength to low strength block light olive grey to very dark grey CLAY, with traces of black mottling	83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.04	0.05	0.05	2.8	4.0	7.2	1.5	3.0	4.6	0.6	1.3	4.8
2.75		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.04	0.05	0.06	3.0	4.3	7.5	1.5	3.0	4.6	0.7	1.4	5.0
3.50		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.12	0.18	0.24	3.5	5.2	15.0	1.5	3.0	4.6	0.8	1.7	10.0
3.50		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.12	0.18	0.24	3.6	5.2	15.0	1.5	3.0	4.6	0.8	1.7	10.0
4.20		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.16	0.20	0.24	4.0	6.0	15.0	1.5	3.0	4.6	0.9	2.0	10.0
4.20		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.16	0.17	0.18	4.0	6.0	15.0	1.5	3.0	4.6	0.9	2.0	10.0
6.00		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.21	0.22	0.23	5.3	8.5	11.1	1.5	3.0	4.6	1.2	2.8	7.4
8.00		83.1	88.6	94.1	14.5	14.7	15.0	43.6	51.5	59.4	0.25	0.27	0.28	6.8	11.4	14.4	1.5	3.0	4.6	1.5	3.8	9.6
8.00		42.6	56.0	69.4	15.6	16.7	17.8	29.3	40.1	50.9	0.30	0.31	0.32	6.8	11.4	14.4	1.5	3.0	4.6	1.5	3.8	9.6
9.00		42.6	56.0	69.4	15.6	16.7	17.8	29.3	40.1	50.9	0.30	0.31	0.32	7.5	12.8	16.0	1.5	3.0	4.6	1.6	4.3	10.7
9.00		42.6	56.0	69.4	15.6	16.7	17.8	29.3	40.1	50.9	0.26	0.29	0.32	7.5	12.8	16.0	1.5	3.0	4.6	1.6	4.3	10.7
12.60		42.6	56.0	69.4	15.6	16.7	17.8	29.3	40.1	50.9	0.52	0.55	0.58	13.0	20.0	27.0	1.5	3.0	4.6	2.8	6.7	18.0
12.60	Low strength foliated dark greenish grey silty CLAY with closely spaced thin laminae of fine sand	35.0	41.0	47.1	17.2	17.8	18.4	29.3	40.1	50.9	0.48	0.55	1.60	13.0	20.0	38.0	1.5	3.4	5.3	2.5	5.9	25.3
16.00		35.0	41.0	47.1	17.2	17.8	18.4	29.3	40.1	50.9	0.58	0.65	1.60	17.4	25.0	43.0	1.5	3.4	5.3	3.3	7.4	28.7
16.00		35.0	41.0	47.1	17.2	17.8	18.4	19.9	27.8	35.6	0.58	0.65	1.60	17.4	25.0	43.0	1.5	3.4	5.3	3.3	7.4	28.7
20.00		35.0	41.0	47.1	17.2	17.8	18.4	19.9	27.8	35.6	0.69	0.76	1.60	23.0	32.0	50.0	1.5	3.4	5.3	4.3	9.4	33.3
20.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	0.69	0.76	1.60	23.0	32.0	50.0	1.5	3.4	5.3	4.3	9.4	33.3
24.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	0.81	0.88	1.60	29.0	40.0	58.0	1.5	3.4	5.3	5.5	11.8	38.7
24.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	0.90	0.97	1.60	29.0	40.0	58.0	1.5	3.4	5.3	5.5	11.8	38.7
33.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	1.18	1.25	1.60	46.0	58.0	76.0	1.5	3.4	5.3	8.7	17.1	50.7
33.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	1.08	1.14	1.60	40.0	44.0	58.0	1.5	3.4	5.3	7.5	12.9	38.7
35.00		32.2	36.4	40.6	17.8	18.3	18.7	16.4	21.4	26.4	1.08	1.14	1.60	40.0	44.0	58.0	1.5	3.4	5.3	7.5	12.9	38.7

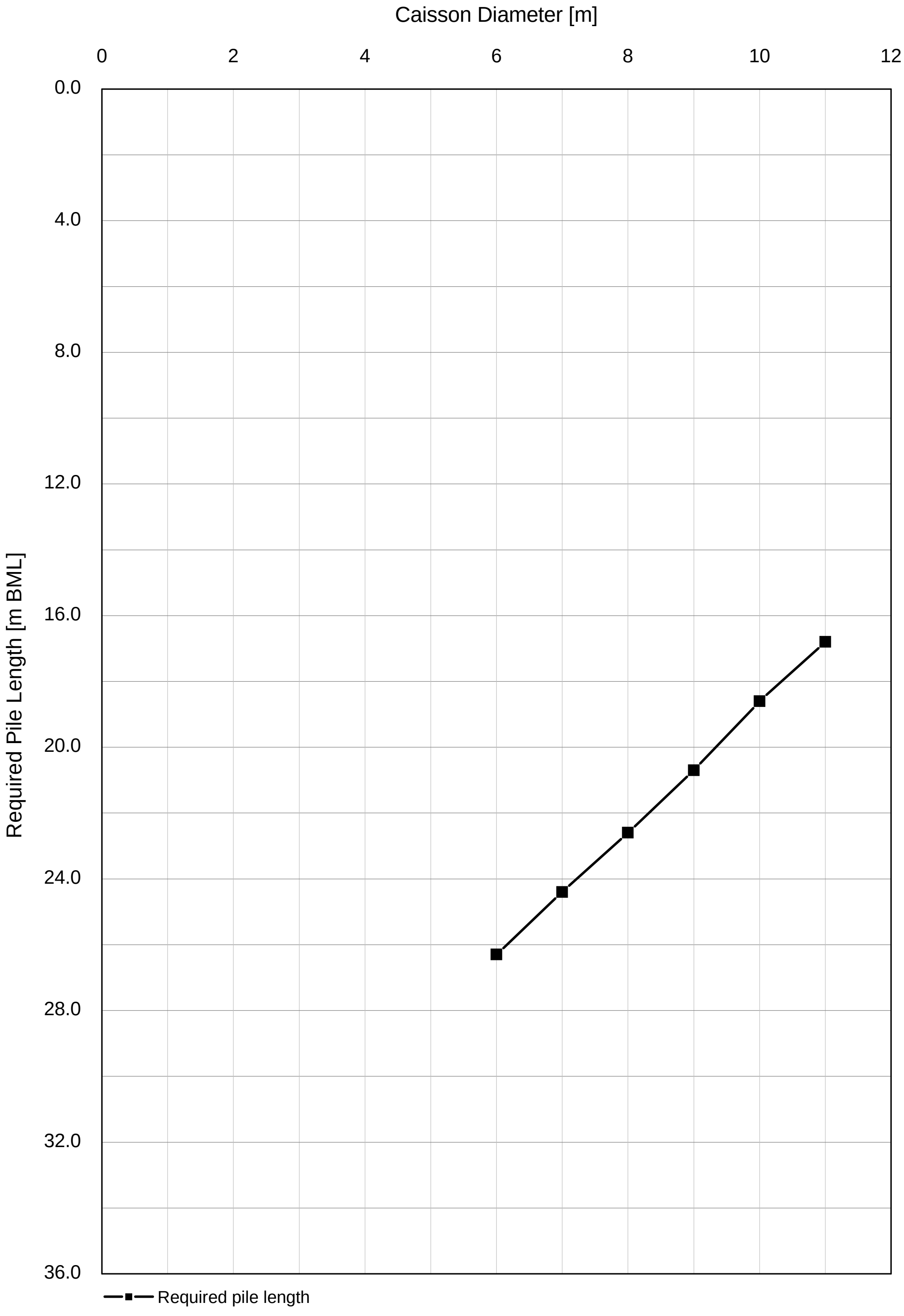
Notes:
LE = Low Estimate
BE = Best Estimate
HE = High Estimate

DESIGN SOIL PARAMETERS
Domino Drill Center 1
Neptun Deep Survey

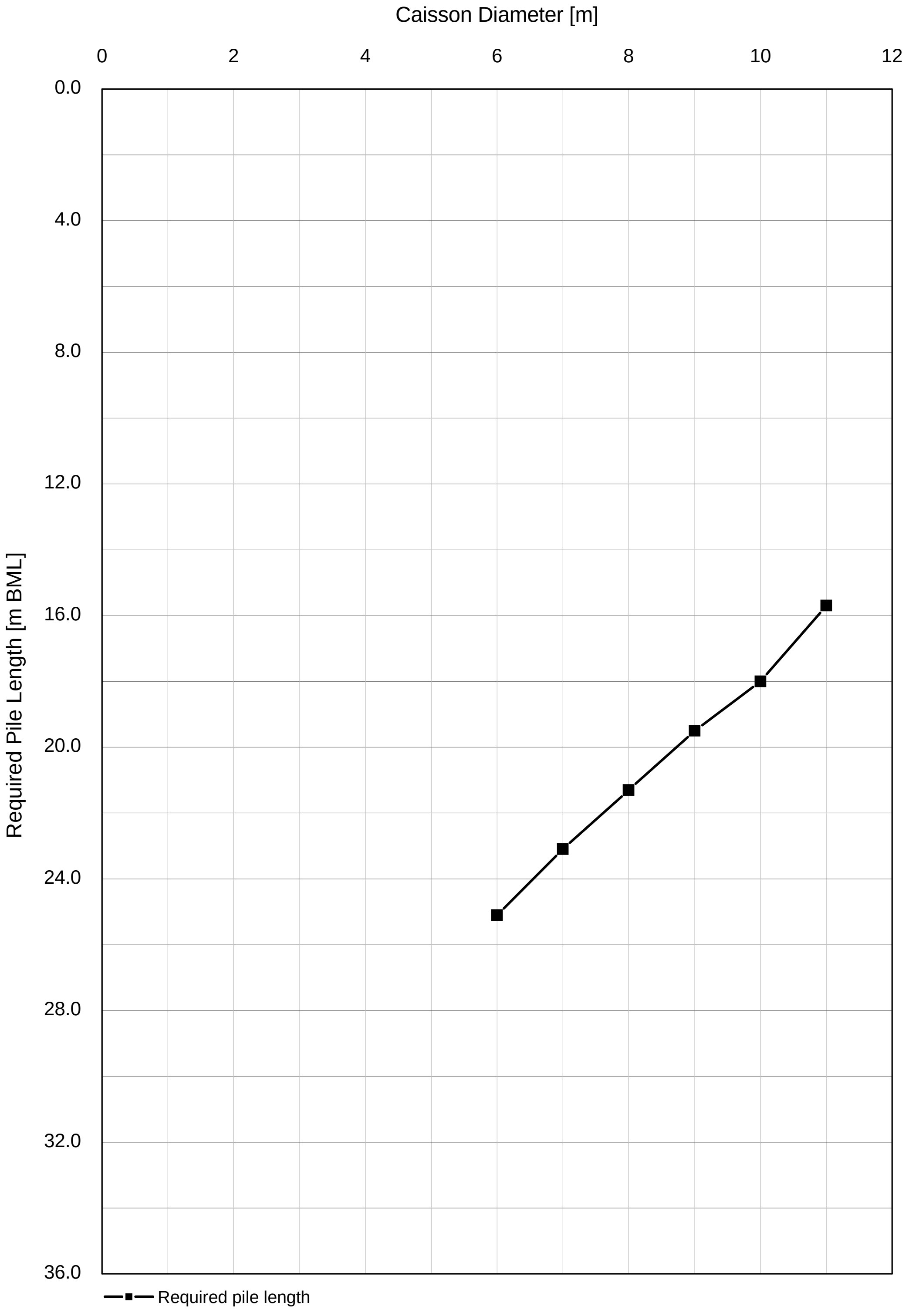
Depth	Soil Description	Water Content [%]			Unit Weight [kN/m3]			Plasticity Index [%]			Measured Cone Resistance (q_c) [Mpa]			Undrained Shear Strength [kPa]			Sensitivity [-]			Remoulded Undrained Shear Strength [kPa]		
		LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE	LE	BE	HE
0.00	Extremely low strength CLAY	182.3	370.7	559.0	10.5	11.2	11.7	51.8	104.7	157.5	0.00	0.01	0.01	0.1	0.6	1.2	1.5	3.0	6.0	0.0	0.2	0.8
0.50		182.3	370.7	559.0	10.5	11.2	11.7	51.8	104.7	157.5	0.04	0.05	0.05	0.2	0.8	2.5	1.5	3.0	6.0	0.0	0.3	1.7
0.50	Extremely low strength CLAY (Organic rich sapropel)	125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.03	0.04	0.04	0.2	0.8	2.5	1.5	4.0	8.0	0.0	0.2	1.7
1.50		125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.05	0.05	0.06	0.5	1.8	4.0	1.5	4.0	8.0	0.1	0.4	2.7
1.50		125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.06	0.07	0.07	0.5	1.8	4.0	1.5	4.0	8.0	0.1	0.5	2.7
2.50		125.0	295.5	508.3	10.5	11.8	13.0	51.8	104.7	157.5	0.11	0.11	0.12	0.8	2.8	5.5	1.5	4.0	8.0	0.1	0.7	3.7
2.50	Extremely low strength to very low strength block dark greenish grey CLAY	84.7	96.1	107.4	14.1	14.5	15.0	36.7	43.2	49.6	0.13	0.14	0.14	2.8	4.0	7.2	1.5	3.0	4.6	0.6	1.3	4.8
3.00		84.7	96.1	107.4	14.1	14.5	15.0	36.7	43.2	49.6	0.15	0.16	0.16	3.2	4.6	7.8	1.5	3.0	4.6	0.7	1.5	5.2
3.00		84.7	96.1	107.4	14.1	14.5	15.0	36.7	43.2	49.6	0.11	0.14	0.18	3.2	4.6	15.0	1.5	3.0	4.6	0.7	1.5	10.0
3.75		84.7	96.1	107.4	14.1	14.5	15.0	36.7	43.2	49.6	0.13	0.15	0.18	3.7	5.5	15.0	1.5	3.0	4.6	0.8	1.8	10.0
3.75		81.6	92.9	104.2	14.2	14.7	15.1	46.7	52.5	58.4	0.14	0.15	0.15	3.7	5.5	15.0	1.5	3.0	4.6	0.8	1.8	10.0
5.00		52.9	64.3	75.7	15.7	16.0	16.3	46.7	52.5	58.4	0.17	0.18	0.19	4.6	7.0	15.0	1.5	3.0	4.6	1.0	2.3	10.0
5.00		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.18	0.19	0.20	4.6	7.0	9.5	1.5	3.0	4.6	1.0	2.3	6.3
6.00		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.20	0.21	0.22	5.3	8.5	11.1	1.5	3.0	4.6	1.2	2.8	7.4
6.00		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.18	0.19	0.20	5.3	8.5	11.1	1.5	3.0	4.6	1.2	2.8	7.4
8.00		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.29	0.30	0.30	6.8	11.4	14.4	1.5	3.0	4.6	1.5	3.8	9.6
8.00		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.25	0.28	0.30	6.8	11.4	14.4	1.5	3.0	4.6	1.5	3.8	9.6
9.60		74.2	82.8	91.5	14.7	15.0	15.4	46.7	52.5	58.4	0.35	0.37	0.39	7.9	13.7	17.0	1.5	3.0	4.6	1.7	4.6	11.3
9.60		38.3	49.9	61.5	16.7	17.5	18.2	24.2	36.7	49.2	0.31	0.44	0.58	11.0	13.7	22.0	1.5	3.0	4.6	2.4	4.6	14.7
12.60		38.3	49.9	61.5	16.7	17.5	18.2	24.2	36.7	49.2	0.46	0.52	0.58	13.0	20.0	27.0	1.5	3.0	4.6	2.8	6.7	18.0
12.60	Low strength to medium strength dark greenish grey CLAY, with closely to medium spaced thin to medium beds of sand	27.5	33.3	39.1	18.2	18.8	19.3	23.9	34.1	44.2	1.13	2.09	5.00	13.0	20.0	38.0	1.5	3.4	5.3	2.5	5.9	25.3
24.00		27.5	33.3	39.1	18.2	18.8	19.3	23.9	34.1	44.2	1.13	2.09	5.00	29.0	40.0	58.0	1.5	3.4	5.3	5.5	11.8	38.7
24.00		30.1	32.2	34.4	18.5	18.8	19.1	15.9	20.8	25.6	1.50	4.00	8.00	29.0	40.0	58.0	1.5	3.4	5.3	5.5	11.8	38.7
33.00		30.1	32.2	34.4	18.5	18.8	19.1	15.9	20.8	25.6	1.67	4.00	8.00	46.0	58.0	76.0	1.5	3.4	5.3	8.7	17.1	50.7
33.00		30.1	32.2	34.4	18.5	18.8	19.1	15.9	20.8	25.6	1.67	3.05	8.00	40.0	44.0	58.0	1.5	3.4	5.3	7.5	12.9	38.7
35.00		30.1	32.2	34.4	18.5	18.8	19.1	15.9	20.8	25.6	1.75	3.09	8.00	40.0	44.0	58.0	1.5	3.4	5.3	7.5	12.9	38.7

Notes:
LE = Low Estimate
BE = Best Estimate
HE = High Estimate

DESIGN SOIL PARAMETERS
Domino Drill Center 2
Neptun Deep Survey



REQUIRED PILE LENGTH FOR SUCTION PILE
DNV (2005) ASSESSMENT METHOD
DOMINO DRILL CENTER 1



REQUIRED PILE LENGTH FOR SUCTION PILE
DNV (2005) ASSESSMENT METHOD
DOMINO DRILL CENTER 2

APPENDICES

A. GUIDELINES ON USE OF REPORT

A. GUIDELINES ON USE OF REPORT

This report (the "Report") was prepared as part of the services (the "Services") provided by Fugro GB Marine Limited ("Fugro") for its client (the "Client") under terms of the relevant contract between the two parties (the "Contract"). The Services were performed by Fugro based on requirements of the Client set out in the Contract or otherwise made known by the Client to Fugro at the time.

Fugro's obligations and liabilities to the Client or any other party in respect of the Services and this Report are limited in time and value as defined in Contract (or in the absence of any express provision in the Contract as implied by the law of the Contract) and Fugro provides no other representation or warranty whether express or implied, in relation to the Services or for the use of this Report for any other purpose. Furthermore, Fugro has no obligation to update or revise this Report based on changes in conditions or information which emerge following issue of this Report unless expressly required by the Contract.

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