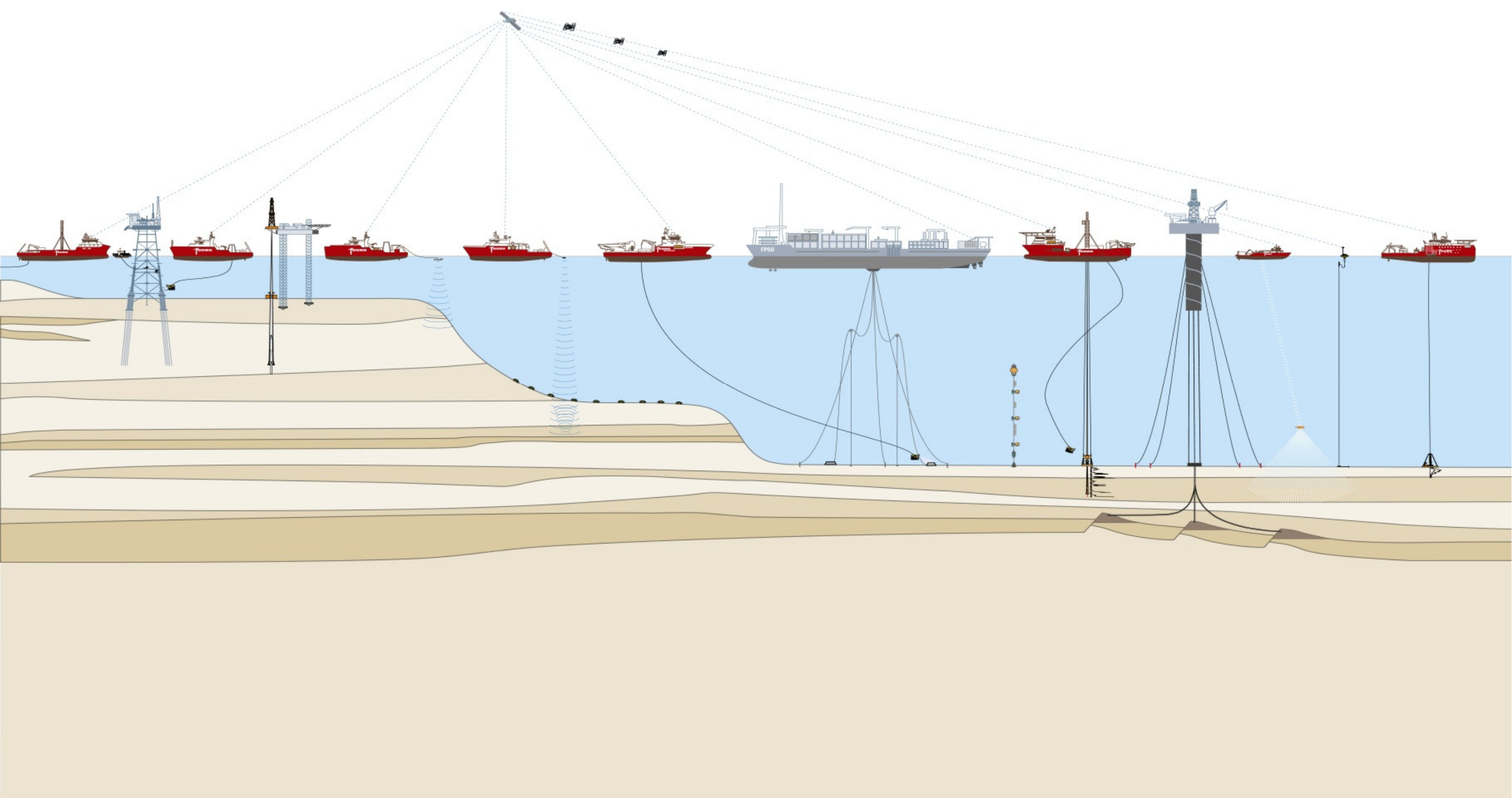
Geotechnical Report
Thermal Conductivity and
Seabed Temperature
Wind Farm Sites I to IV
Borssele Wind Farm Zone
Dutch Sector, North Sea

Client Reference No. WOZ1500010
Fugro Report No. N6170/01
Issue 2

Rijksdienst voor Ondernemend Nederland (RVO)



Geotechnical Report
Thermal Conductivity and
Seabed Temperature
Wind Farm Sites I to IV
Borssele Wind Farm Zone
Dutch Sector, North Sea

Client Reference No. WOZ1500010
Fugro Report No. N6170/01
Issue 2

Client	Rijksdienst voor Ondernemend Nederland (RVO)
Client Address	Croeselaan 15 3521 BJ Utrecht The Netherlands

Rijksdienst voor Ondernemend Nederland

Fugro Project Lead E. Schoute - Senior Project Engineer

Issue No.	Date	Report Status	Approved
2	28-Jun-2016	Final	LJP
1	29-Apr-2016	Fugro Approved Draft	LJP

Fugro Engineers B.V. P.O. Box 130, 2630 AC Nootdorp. ☎ +31703111444, ✉ febvinfo@fugro.nl
Trade Register Nr: 27114149 / VAT Nr: 005621409B07
A member of the Fugro group of companies with offices throughout the world

CONTENTS

1. INTRODUCTION

1.1 Purpose of Report

This report presents an assessment of thermal conductivity and seabed temperature of near-seafloor soils for Wind Farm Sites (WFS) I to IV of the Borssele Wind Farm Zone (WFZ). Plate 1 shows location information. The presented information serves as input for design, installation and maintenance of planned inter-array and export cable routes at WFS I to IV (site).

1.2 Scope of Report

The scope of this report is as follows:

- Assessment of near-seafloor soil conditions in view of their thermal properties, based on results of available geotechnical information (Fugro, 2015a-b; Fugro, 2016a-b);
- Recommended thermal conductivity values of near-seafloor soils, based on integrated information, i.e. public domain, Fugro database and regional experience and laboratory test results;
- Vertical profiles of seasonal seabed temperature.

This report considers ‘near-seafloor’ as the depth interval from seafloor to 3 m below seafloor (BSF).

1.3 Project Responsibilities and Use of Report

This report presents information according to a project specification determined and monitored by the Client.

This report must be read in conjunction with Section B, “Use of Report”. This section includes information about report issue control.

Fugro understands that this report will be used for the purpose described above. That purpose was a significant factor in determining the scope and level of the services. If the purpose for which the report is used or the Client’s proposed development or activity change, this report may no longer be valid.

2. RESULTS

2.1 Site Zonation

The near-seafloor soils in the Borssele WFZ comprise sediments deposited during the Quaternary, particularly the Southern Bight Formation (Holocene) and the Eemian and/or locally Kreftenheye Formations (Pleistocene). Fugro (2015a-b) and Fugro (2016a-b) assign the Holocene deposits to geotechnical Unit A and the Pleistocene deposits to geotechnical Unit B. These units are similar in terms of their geotechnical properties. The boundary between Unit A and Unit B is difficult to discriminate on basis of the available geotechnical data. The boundary lies within or below the depth range of interest (i.e. the top 3 m of the seabed).

The near-seafloor soils in the Borssele WFZ site consist of fine to medium SAND, locally (slightly) gravelly or clayey and locally with beds of (slightly) gravelly clayey sand or clay. Relative density varies from very loose to very dense. Microscopic analyses indicate that quartz is the main component. Grains are typically sub-angular to sub-rounded, with low to high sphericity. Section A presents further information.

Table 2.1 and Plate 2 provide a zonation for thermal conductivity assessment. The zonation specifically considers particle size distribution and in situ relative density of sand. These properties are important for thermal conductivity of soil.

Table 2.1: Zonation within the Borssele WFZ

Soil Property	ZONE 1	ZONE 2	ZONE 3
Relative Density	Very dense sand	Very loose to medium dense sand	Very loose to very dense sand
Particle Size Distribution	▪ Medium sand ▪ Well-graded to poorly graded ▪ Percentage fines <5 % ▪ Locally gravelly	▪ Fine to medium sand ▪ Well-graded to poorly graded ▪ Percentage fines 2 % to 10%	▪ Medium sand ▪ Well-graded to poorly graded ▪ Percentage fines <5 % ▪ Locally gravelly
Mineralogy*	▪ 80 % to 95 % of quartz ▪ Minor: lithic fragments, feldspar, mica, shell fragments	▪ 80 % to 95 % of quartz ▪ Minor: lithic fragments, feldspar, mica, shell fragments	▪ 80 % to 95 % of quartz ▪ Minor: lithic fragments, feldspar, mica, shell fragments
Water Content*	21 % to 25 %	23 % to 27 %	21 % to 27 %
Notes: * Values derived from laboratory test results and/or mineralogical analyses - Zonation applies to the top 3 m BSF			

Zone 1 follows approximately the trends of major sandbanks (or larger sand waves) and is typically characterised by relatively high soil densities (i.e. very dense sands) below the seafloor.

Zone 2 coincides roughly with bathymetrical lows between the sandbanks and is characterised by relatively low soil densities (i.e. very loose to medium dense sands) in the top 3 m of the soil profile.

Zone 3 is widespread across the Borssele WFZ. It is intermediate between Zones 1 and 2. In situ relative density varies considerably in a vertical soil profile.

2.2 Thermal Conductivity

2.2.1 Recommended Thermal Conductivity Values

A key consideration for thermal conductivity is the condition of the soil around the cable after cable burial. Trenching and trench backfilling result in a significant degree of remoulding and disturbance of the in situ soils. This generally results in soil around the cable having initially lower density and higher water content than the surrounding in situ material. Depending on the selected methods of trenching and backfilling, it is expected that relative density of the backfill material will vary between 30% and 90%. Also, trenching and trench backfilling can change soil composition, particularly amount of fines (particles of <0.06 mm size).

The recommended values for soil thermal conductivity are as follows:

- Low estimate: 1.8 W/(m.K)
- Best estimate: 2.2 W/(m.K)
- High estimate: 2.4 W/(m.K)

The recommended values for thermal conductivity are independent of zonation. This is because of the laboratory test results (Section A) showing (1) relatively uniform soil conditions across the Borssele WFZ, and (2) a relatively narrow range of soil densities as function of relative density. Particularly, dry density of soil typically ranges between 1.40 Mg/m^3 at a relative density of 30 % and 1.55 Mg/m^3 at a relative density of 90 %. Results of laboratory tests on saturated specimens at relative densities ranging from approximately 30% to 90% showed a range in thermal conductivities of 1.9 W/(m.K) to 2.2 W/(m.K). The physical condition of the disturbed soil changes with time due to consolidation processes, such as self-weight consolidation, ageing, cyclic (wave, tide, atmospheric) loading and temperature variations. Consolidation and accompanying chemical processes will slowly decrease water content over time and increase soil particle contacts.

Client-provided data (RVO, 2016) include a summary of thermal conductivity tests performed on specimens from vibrocores, taken from within the Borssele WFZ. The summary shows a reference to ASTM D5334-08 (ASTM, 2008) and “undisturbed” testing. The results have not been used. Comments are as follows:

- The presented information lacks documentation according to ASTM, particularly (a) which soil material type(s) apply to the test results, (b) test specimen preparation and test method(s) used, (c) specimen dry density or saturation.
- The available documentation and results are outside of expectations. For example, vibrocore samples are most likely to have a disturbed soil structure and therefore fall in sample class 3 (ISO, 2014), i.e. disturbed, not undisturbed. This may have an effect on the dry density of test specimens. Specimen saturation is of importance as an increase in saturation will result (similar to the effect of an increase in dry density) in an increase in thermal conductivity (Fricke et al., 1992). Also, test results show a broader range of thermal conductivity values than expected for soils in this region, with thermal conductivity values for 40 samples from seafloor to 3 m BSF ranging from 0.82 W/(m.K) to 3.23 W/(m.K).

2.2.2 Parameters Influencing Thermal Conductivity

Heat can be transferred by convection, radiation and conduction. Heat transfer in saturated soils is mainly due to thermal conduction through the solids framework and the pore water (Farouki, 1986). Radiation is not relevant for power cable engineering. Convection is generally negligible for heat transfer in a soil.

Thermal conductivity of saturated soil is influenced by the following parameters:

- Mineral composition;
- Soil density and water content;
- Particle size distribution;
- Ambient temperature.

Table 2.2 illustrates thermal conductivity values of selected soil components (Van Wijk, 1963; Lovell, 1985). Soil thermal conductivity varies with mineral composition. For example, sands with high quartz content generally have higher thermal conductivity than sands with high contents of feldspars or other minerals. In saturated soil, the ability to transmit heat is a function of the relative proportions of solids and water. The thermal conductivity of solids is significantly higher than that of water.

Table 2.2: Thermal Conductivity of Soil Components

Material	Thermal Conductivity [W/(m.K)]
Water	0.60
Organic matter	0.25
Feldspars (or clay)*	1.90
Mixture of minerals*	2.90
Shell material	3.32
Quartz	8.50
Note: * approximate values	

Soil thermal conductivity increases with the dry density of soil. With an increase in soil dry density, more soil particles are packed into a unit volume and the number of contact points between the particles increases. This increase in contact points provides a larger heat flow path and, thus, increases the soil thermal conductivity. Figure 1 shows how unit weight (and thus dry density) and soil water content are related for water-saturated soil. The parameter ρ_s refers to density of solid particles. For Borssele WFZ Units A and B, typical values for ρ_s are in the range of 2.65 Mg/m^3 to 2.70 Mg/m^3.

A soil possesses a certain distribution of grain sizes and shapes. As smaller grains are added to a uniform-sized material, they will tend to fill the space between the larger grains (Brandon & Mitchell, 1989), thus providing more grain-contact area and an increase of solid matter per unit volume (i.e. higher soil density or soil unit weight). Therefore, well-graded soils have potential to conduct heat better than poorly graded soils. The deviation of particle shape from spherical also affects the packing characteristics.

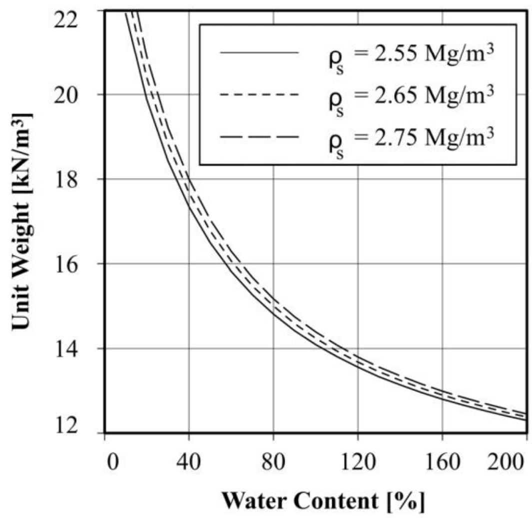


Figure 1: Relationship between soil water content and unit weight

Figure 2 summarises literature values and results from Fugro studies of North Sea sands. The presented information is approximate. As a general rule, thermal conductivity increases with increase in average particle size. Thermal conductivity of sand increases slightly with soil temperature. For the Borssele WFZ, this effect is assessed to be negligible, particularly because soil temperatures close to pore water freezing are unlikely.

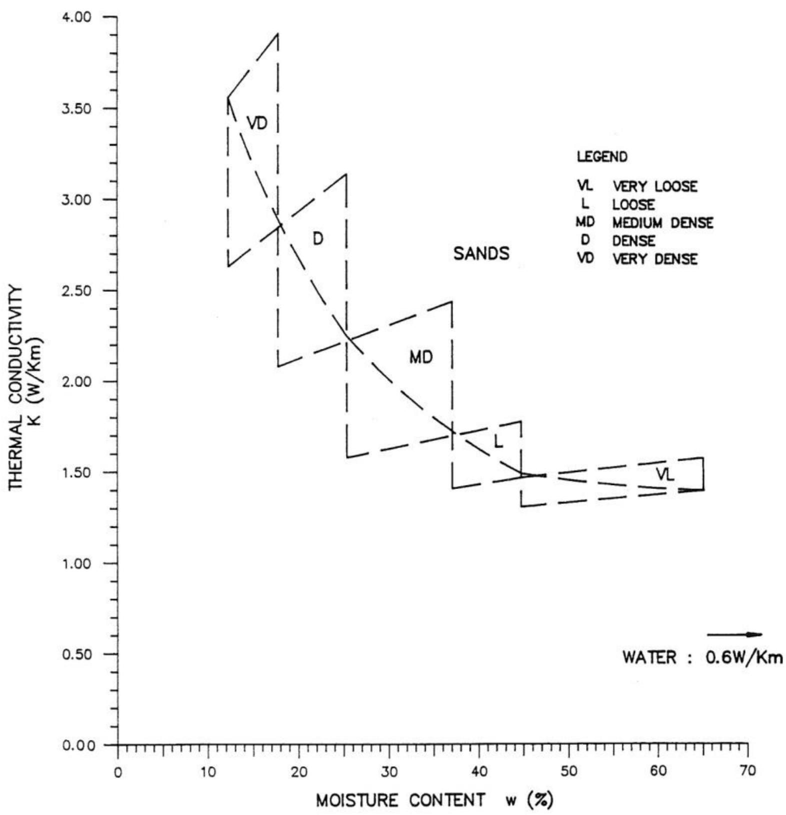


Figure 2: Thermal conductivity-moisture content relationships for sands at various relative densities

2.3 In situ Seabed Temperature

Figures 3 and 4 show recommended best-estimates for in situ seabed temperature distribution for the Borssele WFZ. The upper diagram in Figure 3 shows temperature change over one year at selected depths within the top 10 m BSF. The lower diagram shows temperature-depth profiles with temperature variation for each month of the year.

The presented values in Figure 3 and Figure 4 rely on database information on mean sea water temperatures close to seafloor (Fugro, 2016c).

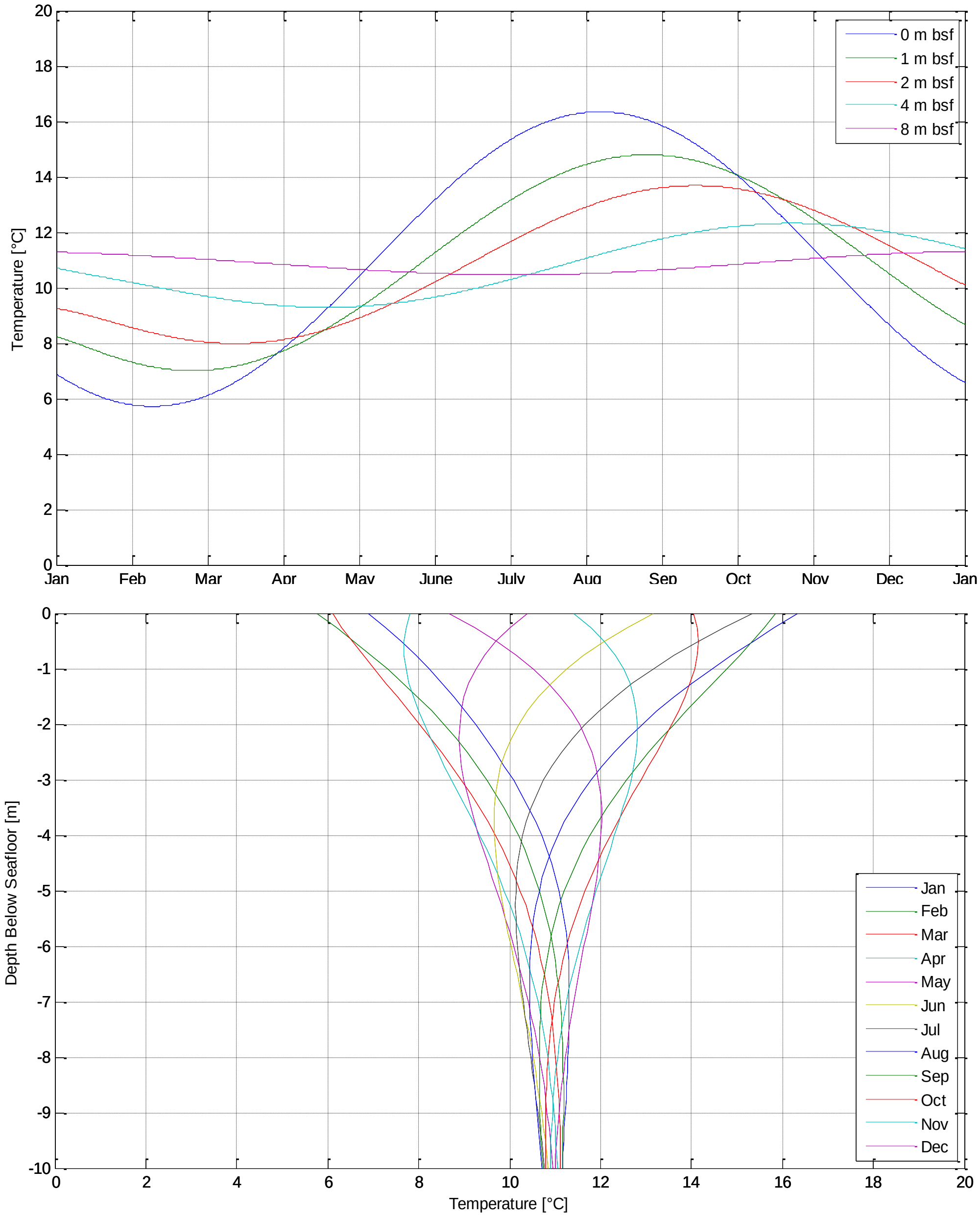


Figure 3: Modelled seasonal seabed temperature distribution

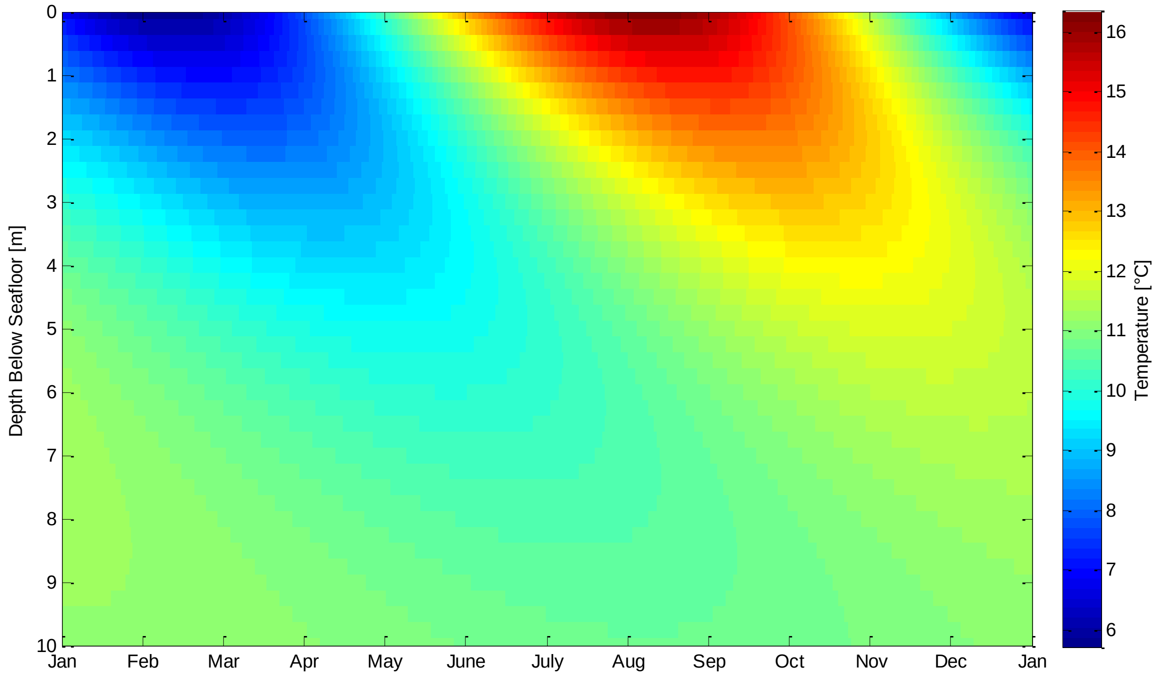


Figure 4: Heat transfer from sea water into soil

Comments are as follows:

- Fugro (2016c) considers monthly average values, monitored at various locations across the southern North Sea. As such, data are representative for a given year. Temperature anomalies within a year are possible. Public domain data sources indicate no significant long-term trends of bottom water temperature in the southern North Sea.
- Figure 5 shows variation of annual seawater temperature close to seafloor (IACMST, 2005). The data show a maximum difference of approximately 4°C between 1970 and 2002. This means that the peak-to-peak amplitudes of seasonal temperature variations can be higher than the approximately 10°C as given in Figure 3. Figure 5 combined with Figure 3 allow estimation of temperature design ranges for cable engineering.
- Figures 3 and 4 consider a one-dimensional heat transfer equation:

$$\frac{\partial T(t,z)}{\partial t} = \kappa \frac{\partial^2 T(t,z)}{\partial z^2}$$

where T(t,z) is the temperature at time t (s) and depth z (m BSF) and κ the thermal diffusivity (m^2/s). Thermal diffusivity is related to thermal conductivity, heat capacity and soil density. The selected values for thermal conductivity and volumetric heat capacity were 2.3 W/(m.K) and 2.5×10^6 J/(K.m^3) respectively.

- The model assumes bottom boundary conditions such that no heat is reaching deep layers from the seafloor. The boundary condition for the top is the bottom water temperature. It is equal to the seasonal variation over one year and can be approximated by a sinusoidal wave equation:

$$T(t,0) = A + B.\sin\left(\frac{2\pi t}{12} + \omega\right)$$

where A (°C) is the average temperature, B (°C) is the average amplitude and ω a time phase shift (month).

- The heat transfer equation requires correction for geothermal gradient $c_g(z)$. A value of 0.03 K/m for geothermal gradient was used, which represents an average temperature gradient near the surface of the earth (Miesner et al., 2015).
- Comparison of the results of the analytic model with measured in situ temperature-depth profiles presented by Mueller et al. (2013) shows a good correlation. It also shows dependence on accurate thermal parameter values.

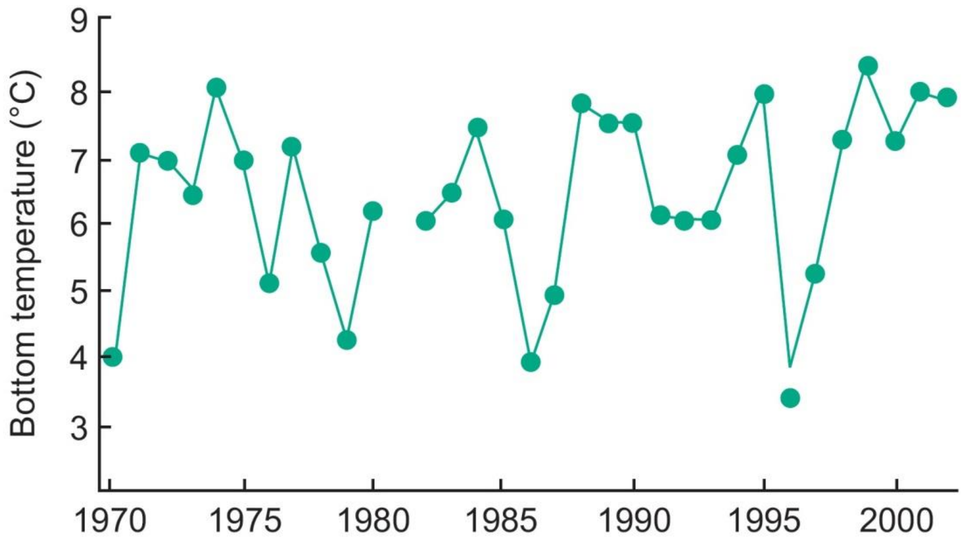


Figure 5: Bottom temperature time series, southern North Sea (IACMST, 2005)

3. SOURCES OF INFORMATION AND REFERENCES

3.1 Client-supplied Information

This report uses and summarises the following Client-supplied information:

- Rijksdienst voor Ondernemend Nederland (RVO), 2016. Email with subject: *"Thermal conductivity data TenneT BWFZ"*, from RVO to Fugro, 8 March 2016.

3.2 Fugro Information

This report uses and summarises Fugro-held information:

- Previous geotechnical investigations within the Borssele WFZ (Fugro, 2015a-b and Fugro, 2016a-b);
- Fugro experience and in-house knowledge on thermal conductivity measurements for North Sea soils (laboratory and in situ).

3.3 References of Main Text

ASTM International, 2008. *ASTM D5334-08 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure.* West Conshohocken: ASTM International.

Brandon, T.L. and Mitchell, J.K., 1989. Factors Influencing Thermal Resistivity of Sands, *ASCE Journal of Geotechnical Engineering*, Vol. 115, No. 12.

Farouki, O.T., 1986. *Thermal Properties of Soils*, Series on Rock and Soil Mechanics, Vol. 11, Trans Tech Publications.

Fricke, B.A., Misra, A., Becker, B.R. and Stewart, W.E., 1992. Soil Thermal Conductivity: Effects of Saturation and Dry Density. In: *Thermal Performance of the Exterior Envelopes of Buildings V,* pp. 158-168. Atlanta: ASHRAE.

Fugro, 2015a. *Geological Ground Model – Wind Farm Site I – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. TN48112/Fugro Report No. N6016/05, Issue 3, 15 July 2015.

Fugro, 2015b. *Geological Ground Model – Wind Farm Site II – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. TN48112/Fugro Report No. N6016/06, Issue 4, 3 August 2015.

Fugro, 2016a. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site III – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. WOZ1500010/Fugro Report No. N6083/01, Issue 4, 10 February 2016.

Fugro, 2016b. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site IV – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. WOZ1500010/Fugro Report No. N6083/03, Issue 4, 10 February 2016.

Fugro, 2016c. Monthly averages of seawater temperature from the southern North Sea, GDEM database, email sent to Fugro, 31 March 2016.

IACMST, 2005. *Marine Processes and Climate*. IACMST Contribution to Charting Progress – an Integrated Assessment of the State of the UK Seas. Report 2 of 5, London:DEFRA.

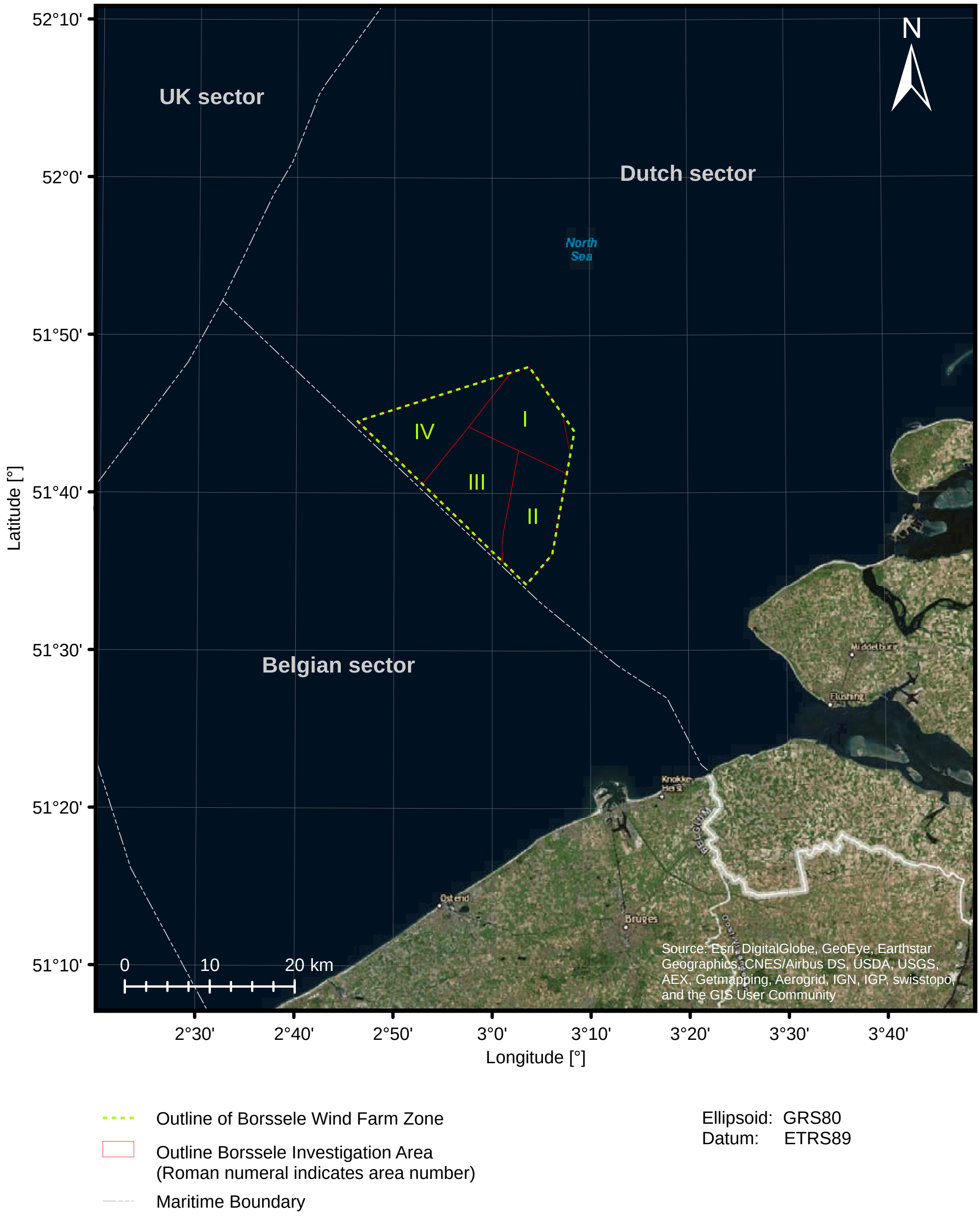
International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

Lovell, M.A., 1985. Thermal Conductivity of Marine Sediments, *Quarterly Journal of Engineering Geology*, Vol.18, pp. 437-441.

Miesner, F., Lechleiter, A. and Müller, C., 2015. Reconstructing bottom water temperatures from measurements of temperature and thermal diffusivity in marine sediments, *Ocean Science*, Vol. 11, No. 4, pp. 559-571.

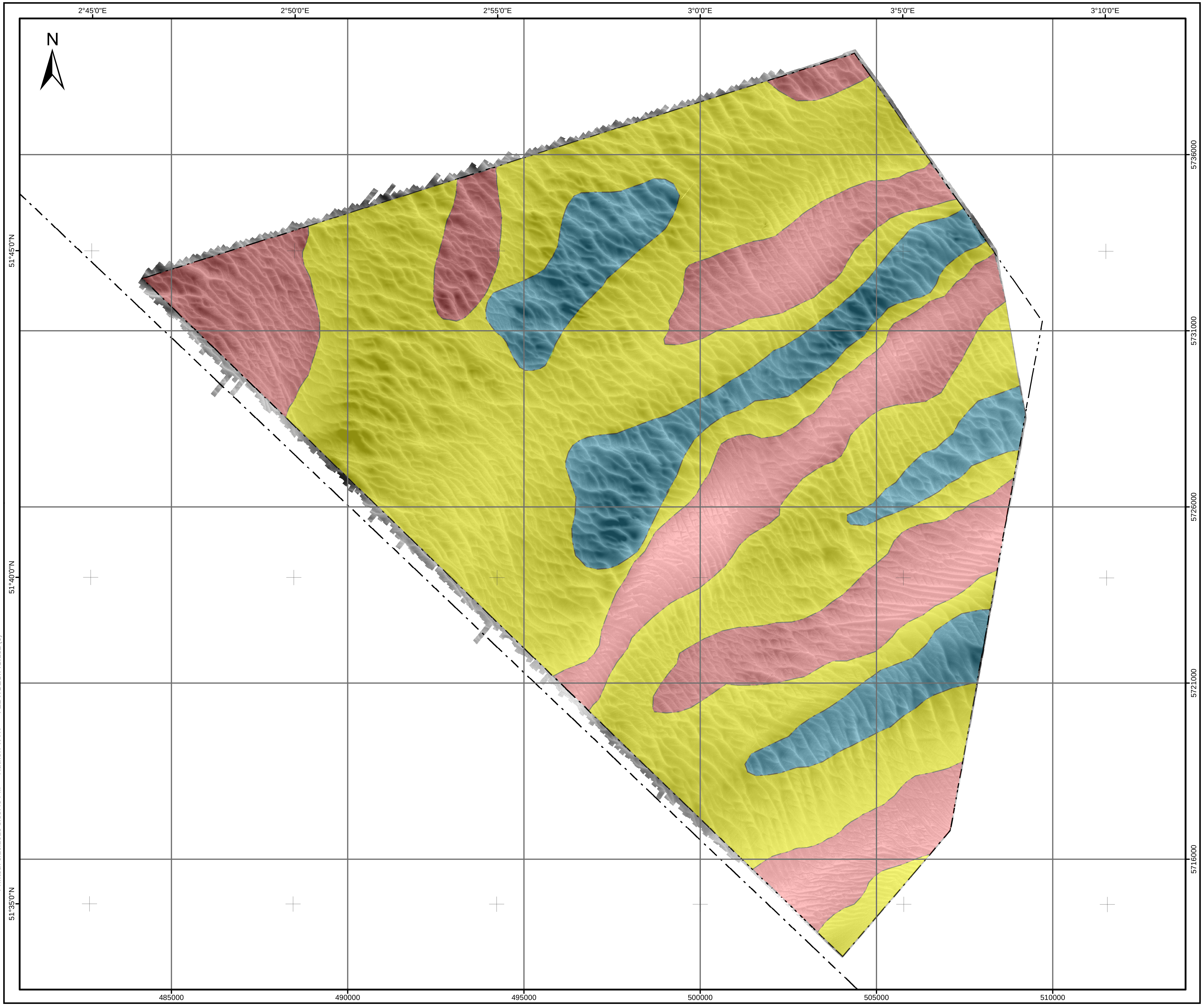
Mueller, Ch., Miesner, F., Usbeck, R. and Schmitz, T., 2013. 2K-criterion: measuring and modelling temperatures and thermal conductivities/diffusivities in shallow marine sediments. In: *Proc. Conference on Maritime Energy 2013,* TUHH, Hamburg, pp. 475-490.

Wijk, W.R. van, ed., 1963. *Physics of plant environment*, Amsterdam: North Holland Publishing.



Printed: 6/20/2016 3:31:30 PM

VICINITY MAP
BORSSELE WIND FARM ZONE, WFS I TO IV – DUTCH SECTOR, NORTH SEA



LEGEND:

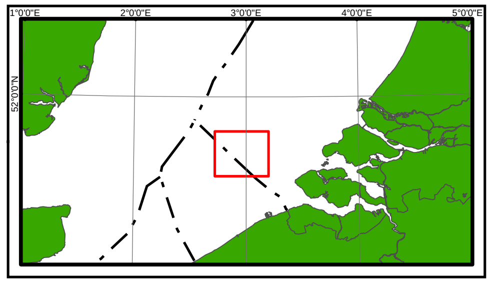
- Zone 1
- Zone 2
- Zone 3
- Maritime Boundary

NOTES:

- Background is bathymetry as presented in Fugro (2015a-h)
- Resolution cells 1 m x 1 m
- Data acquired by Multi-Beam Echo Sounder (MBES)

GEODETIC PARAMETERS:

DATUM	ETRS89
Ellipsoid	GRS80
Semi major axis	a = 6 378 137.000
Inverse flattening	1/f = 298.257222101
PROJECTION	UTM, Zone 31 North
Central Meridian (CM)	3° 00' 00" E
Latitude of Origin	0° 00' 00" N
False Easting	500 000 m
False Northing	000 000 m
Scale factor	0.9996
Units	metres/degrees



Rijksdienst voor Ondernemend Nederland (RVO)
Croeselaan 15, 3521 BJ, Utrecht - THE NETHERLANDS

Fugro
Prismastraat 4, 2631 RT, Nootdorp - THE NETHERLANDS

ZONATION OF NEAR-SEAFLOOR SOILS

BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

Scale 1:100,000
at original A3 page size

0 1,000 2,000 3,000 metres

Fugro Report No. N6170/02 | Issue 1 | Plate 2

Printed: 6/20/2016 3:35:46 PM Author: WVK FEBV/GEO/FIG/152 (0)

SECTION A: LABORATORY TEST RESULTS

SECTION A1: LABORATORY TESTING OVERVIEW

TEXT – SECTION A1: Page

LIST OF PLATES IN SECTION A1: Plate

A1. LABORATORY TESTING OVERVIEW

A1.1 TEST PROGRAMME AND SAMPLE SELECTION

Laboratory testing comprised geotechnical index and classification tests (Table A1.1). For further details on test procedure and methodology, refer to the document "Geotechnical Laboratory Tests" presented in Appendix 1. The laboratory test programme covered specimens of two coarse-grained cohesionless soil units recovered as part of a geotechnical investigation (Fugro, 2015a-b; Fugro, 2016a-b).

Table A1.1: Laboratory Test Programme

Test Type	No. of Tests	Test Procedure
Particle Size Distribution (Sieving)	6	BS1377:Part 2:1990:Clause 9.2 (BSI, 1990) / Fugro in-house procedure
Minimum and Maximum Index Dry Density	6	DGI #000 96-07-02 (DGI, 1996)
Microscopic Photography	6	Fugro in-house procedure
Thermal Conductivity	18	ASTM D5334-14 (ASTM, 2014) / Fugro in-house procedures

All tests were carried out on "batch" samples of coarse-grained cohesionless material. Two to four samples from the first approximately 3 m below seafloor from selected borehole locations were mixed to form a batch. For each zone, two batches were created (refer to Main Text of this report for details about zonation). Table A1.2 presents a summary of the selected samples used for batch sample preparation.

Table A1.2: Batch Samples

Batch ID	Zone	Soil Unit	Site	Borehole ID	Sample ID	Sample Depth [m BSF]
1-I	1	A (Southern Bight)	WFS I	BH-WFS1-2A	2BagA	1.0
					2BagB	1.3
					3BagA	2.0
					3BagB	2.3
1-II	1	A (Southern Bight)	WFS III	BH-WFS3-3	3BagA	1.5
					3BagB	1.8
					4BagA	2.5
2-I	2	A (Southern Bight)	WFS III	BH-WFS3-2	1BagB	0.5
					2BagA	1.0
2-II	2	A (Southern Bight)	WFS IV	BH-WFS4-7	1BagA	0.0
					2BagA	1.0
3-I	3	B (Eem/Kreftenheye)	WFS III	BH-WFS3-5	3BagA	1.5
					3BagB	1.8
					4BagA	2.5
					4BagB	2.7
3-II	3	A (Southern Bight)	WFS IV	BH-WFS4-5	2BagA	0.5
					3BagA	1.0
					4BagA	1.5

A1.2 PRACTICE FOR SAMPLE HANDLING AND LABORATORY TESTING

Practice

Procedure:	According to ISO (2014)
Data Processing and Management:	– Laboratory-specific software as applicable – Graphical scales selected to suit general presentation of data, as applicable – No display of data outside of chart limits, i.e. some values may not be shown, as applicable
Data Format(s):	PDF for viewing and printing (this primary document)

Initial Sample Handling - refer to geotechnical reports Fugro (2015a-b) and Fugro (2016a-b)

Site Geotechnical Laboratory - refer to geotechnical reports Fugro (2015a-b) and Fugro (2016a-b)

Office Geotechnical Laboratory

Test Programme:	Geotechnical index and classification
Programme Adjustment:	– Assessment of feasibility of a test (by sample inspection) prior to start of specimen preparation – Decision by laboratory: (1) to proceed with test, (2) not to proceed with test, (3) to advise adjustments to test procedure – Selection by laboratory of alternative test specimen if decision is “not to proceed” or when adequate test completion proves impracticable
Ground Description:	– No update of geotechnical (sample) ground descriptions made in site geotechnical laboratory, i.e. retention of originally recorded data – Geotechnical sample description is an interpretation of processed data available at the time of preparation, where performed in office geotechnical laboratory – Level of detail and accuracy in geotechnical sample description and interpretation depend on factors such as test data, sample size, quality, coverage, availability of supplementary information, and project requirements
Laboratory Air Temperature:	Typically about $20^{\circ}C$
Sample in Plastic Bags	– Geotechnical index and classification – If applicable, selection and labelling of left-over sample sections for disturbed preservation

Sample Storage and Disposition

Sample Storage:	– Storage period as per contract – Storage temperature within the range $+2^{o}C$ and $+35^{o}C$ – Protection from direct sunlight
Transport:	Not applicable
Final Disposition:	In accordance with office laboratory procedures

A1.3 INDEX LABORATORY TESTS

Particle size distributions were determined for the six individual batches. Results are presented on Plates A2-1 to A2-3 in Section A2.

Minimum and maximum index dry densities were determined for each batch, according the test methods outlined in DGI Product Sheet #000 96-07-02 (DGI, 1996). Results are summarized on Plate A1-1 in Section A1.

A1.4 SAMPLE MICRO PHOTOGRAPHS

Two reflected light micro photographs were taken per batch. One photograph was taken from an unwashed sample with a field of view (FOV) of 10 mm and one photograph was taken from a washed sample, with a FOV of 5 mm. Unwashed photographs were taken to provide a visual indication of clay content within the sample. Micro photographs are presented in Section A3.

A1.5 THERMAL CONDUCTIVITY TESTS

Thermal conductivity is defined as the amount of heat that passes through a unit cross-sectional area of a substance, under a unit thermal gradient, per unit time. After the sample reaches temperature equilibration, the thermal needle probe is inserted. After temperature equilibration is confirmed by probe measurements, a current is passed through the heater element in the probe. As a result, the temperature of the probe and the surrounding soil increases. Results are plotted as temperature versus logarithm of time for the heating and cooling phases. The thermal conductivity is calculated from a suitable linear part of the curve (last 2/3 of the measurements), using the following formula (Brandon & Mitchell, 1989):

$$k=\frac{P}{4\pi(T_2-T_1)}ln\left(\frac{t_2}{t_1}\right)$$

where:
k = Thermal conductivity [W/(m·K)]
P = Applied power [W/m]
T_2-T_1 = Temperature change over selected time interval [K]
t_1, t_2 = Start and end of time interval [s]

Specimens for thermal conductivity testing were prepared according a Fugro in-house method. The system comprises a pressure controller and a sample pedestal complete with porous stone and water inlet mounted on a vibrating table. A test specimen is prepared on the pedestal in a 66 mm inner diameter section of clear liner by using Ladd under-compaction in six layers (Ladd, 1978) at an initial water content of 10%. The specimen was then saturated by pumping water in from the bottom at a slow rate, with control of the pressure head used (approximately 12 kPa) and with measurement of the volume of water added, until water appeared at the upper surface of the sample. By this point, the specimen was assumed to have reached full saturation. After performing a thermal conductivity test at the lowest dry density test point (approximately 30% relative density) vibration was turned on to compact the specimen further to the next dry density test point. Dry densities for each test have been assessed by linear measurement, i.e. the height of the sand column in the liner after vibrating.

Thermal conductivity tests were performed using a KD2 Pro Thermal Properties Analyzer (Decagon Devices, Inc.) with a TR-1 thermal needle probe (100 mm length, 2.4 mm diameter) according procedures described in ASTM D5334-14 (ASTM, 2014) supplemented by various in-house methods. Using this method, a heating element and a temperature measuring element are inserted into a soil specimen. Heating and cooling intervals were one minute each. The tests were performed in duplicate on saturated reconstituted specimens from six batches. Per batch, three thermal conductivity tests were performed at dry densities corresponding to relative densities of approximately 30%, 60% and 90%. Dry densities were selected considering minimum/maximum index dry density test results (refer to Plate A1-1).

Results of the thermal conductivity tests are summarized on Plate A1-2. Heating and cooling curves are presented on plates in Section A4.

A1.6 REFERENCES

ASTM International, 2014. *ASTM D5334-14 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure.* West Conshohocken: ASTM International.

BSI, 1990. *BS 1377: Part 2:1990 British Standard Methods of Test for Soils for Civil Engineering Purposes. Part 2. Classification Tests.* London: BSI (Incorporating Amendment No. 1, published 15 May 1996).

DGI, 1996. *Minimum Index Void Ratio, e_{min} (Danish Method)/Maximum Index Void Ratio, e_{max} (Danish Method).* DGI Product Sheet #000 96-07-02. Lyngby: Danish Geotechnical Institute.

Fugro, 2015a. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site I – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. TN48112/Fugro Report No. N6016/01, Issue 4, 10 July 2015.

Fugro, 2015b. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site II – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. WOZ1500010/Fugro Report No. N6016/03, Issue 4, 10 July 2015.

Fugro, 2016a. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site III – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. WOZ1500010/Fugro Report No. N6083/01, Issue 4, 10 February 2016.

Fugro, 2016b. *Geotechnical Report – Investigation Data – Geotechnical Borehole Locations – Wind Farm Site IV – Borssele Wind Farm Zone – Dutch Sector, North Sea.* Client Reference No. WOZ1500010/Fugro Report No. N6083/03, Issue 4, 10 February 2016.

International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

Ladd, R.S., 1978. Preparing Test Specimens Using Undercompaction. *Geotechnical Testing Journal*, GTJODJ, Vol. 1, No. 1, March 1978, pp. 16-23.

Sample ID	Minimum Index Dry Density [Mg/m^3]	Maximum Index Dry Density [Mg/m^3]
BATCH1-I	1.34	1.54
BATCH1-II	1.38	1.58
BATCH2-I	1.34	1.65
BATCH2-II	1.35	1.64
BATCH3-I	1.33	1.59
BATCH3-II	1.31	1.61

SUMMARY OF MINIMUM AND MAXIMUM INDEX DRY DENSITY TEST RESULTS

Sample ID	Soil Type	Dry Density [Mg/m^3]	Relative Density [%]	P [W/m]	k [W/(m.K)]
BATCH1-I	Sand	1.40	33	4.22	2.03
				4.21	2.06
BATCH1-I	Sand	1.46	63	4.20	2.01
				4.19	2.04
BATCH1-I	Sand	1.52	91	4.18	2.17
				4.16	2.21
BATCH1-II	Sand	1.43	28	4.03	1.99
				4.02	2.12
BATCH1-II	Sand	1.49	59	4.01	2.19
				4.00	2.20
BATCH1-II	Sand	1.56	91	4.00	2.20
				4.00	2.21
BATCH2-I	Sand	1.43	34	4.08	2.05
				4.07	2.12
BATCH2-I	Sand	1.51	60	4.07	2.07
				4.05	2.09
BATCH2-I	Sand	1.61	89	4.05	2.21
				4.03	2.24
BATCH2-II	Sand	1.43	32	4.00	2.04
				4.00	2.07
BATCH2-II	Sand	1.51	60	3.99	2.11
				3.98	2.10
BATCH2-II	Sand	1.61	91	3.97	2.17
				3.96	2.24
BATCH3-I	Sand	1.40	30	4.10	1.91
				4.09	1.90
BATCH3-I	Sand	1.48	62	4.09	1.90
				4.07	1.98
BATCH3-I	Sand	1.50	70	4.07	1.96
				4.06	2.00
BATCH3-II	Sand	1.39	31	3.97	1.97
				3.96	1.98
BATCH3-II	Sand	1.52	74	3.96	2.15
				3.94	2.17
BATCH3-II	Sand	1.57	89	3.94	2.15
				3.93	2.22

Key
P = Applied power
k = Thermal conductivity

SUMMARY OF THERMAL CONDUCTIVITY TEST RESULTS

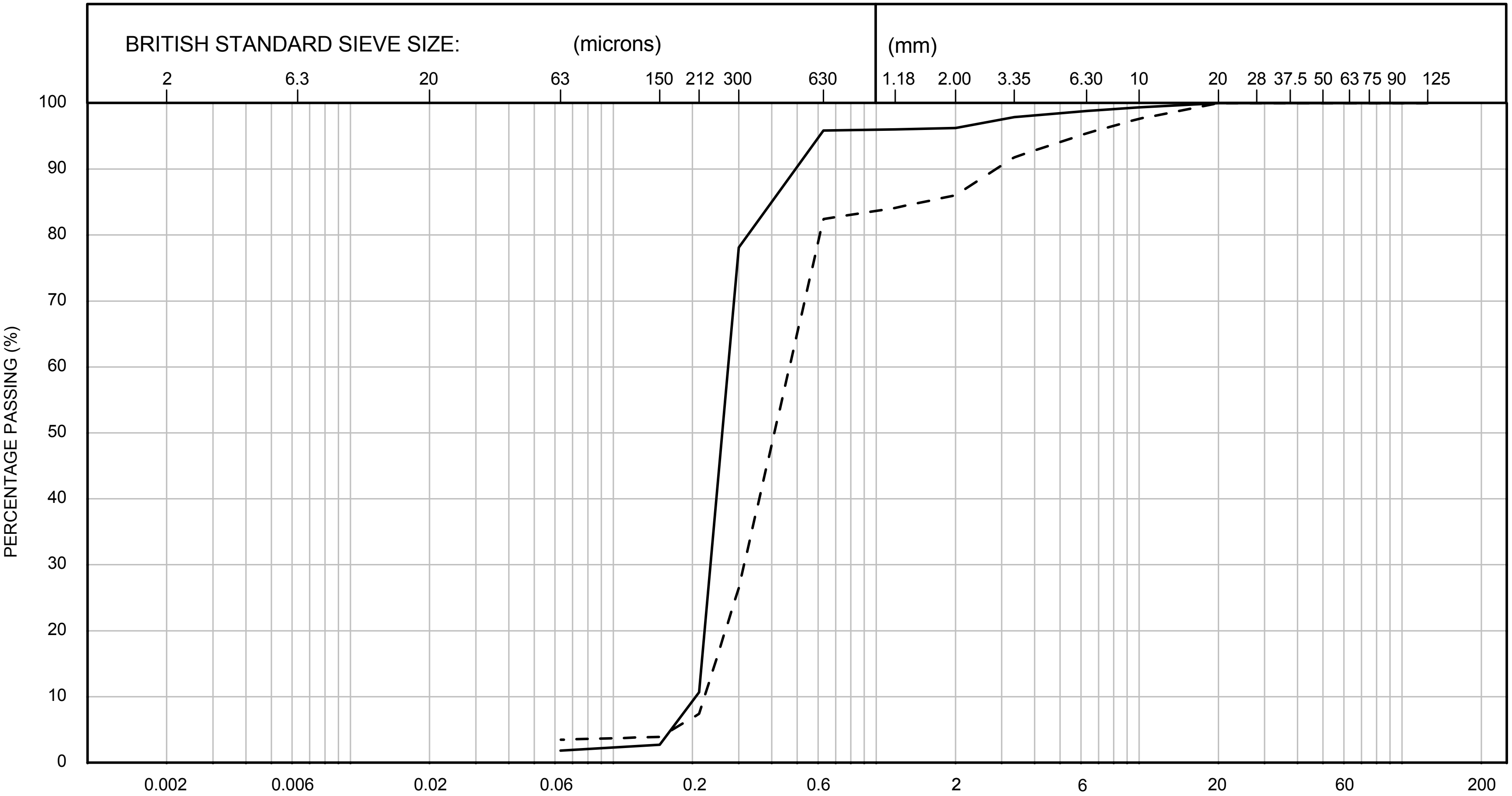
SECTION A2: INDEX LABORATORY TESTS

LIST OF PLATES IN SECTION A2: Plate

THERMAL CONDUCTIVITY AND SEABED TEMPERATURE BORSSELE WIND FARM ZONE

PARTICLE SIZE DISTRIBUTION TEST RESULTS

BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	COBBLES
	SILT			SAND			GRAVEL			

Curve	Batch	Sample	Depth (m)	BS Test * Method	Pretreatment Method	Percentage soil types: Clay	Silt†	Sand	Gravel	Cobbles
———	BATCH 1-I	-	-	9.2	N/A	0	1	96	3	0
– – –	BATCH 1-II	-	-	9.2	N/A	0	3	84	13	0

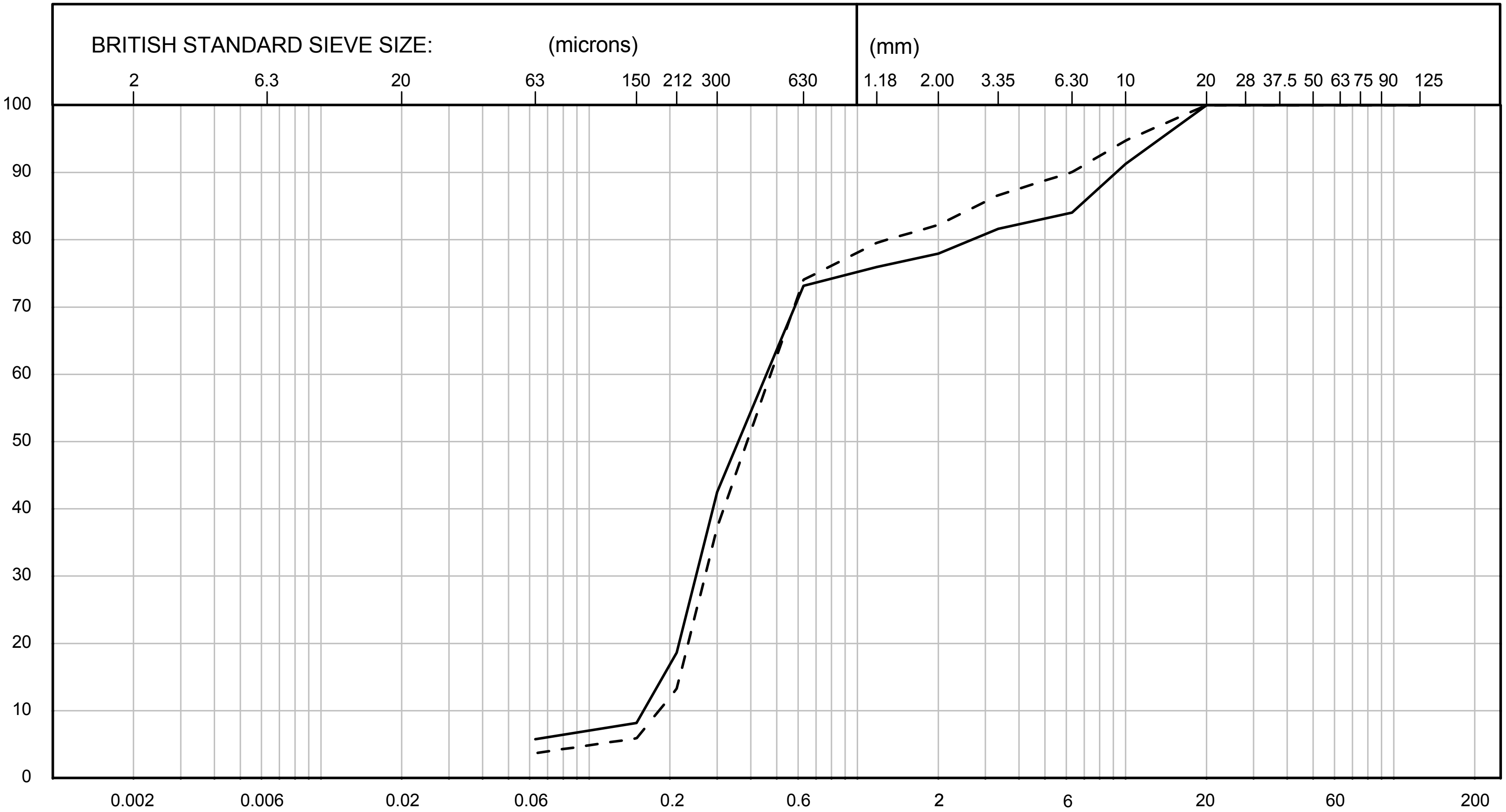
* Tested in accordance with the following clauses of BS 1377: Part 2: 1990:

9.2	Wet sieve
9.3	Dry sieve
9.4	Sedimentation by pipette
9.5	Sedimentation by hydrometer

† Note: Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

THERMAL CONDUCTIVITY AND SEABED TEMPERATURE
BORSSELE WIND FARM ZONE

PARTICLE SIZE DISTRIBUTION TEST RESULTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA



CLAY	SILT			SAND			GRAVEL			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	

Curve	Batch	Sample	Depth	BS Test * Method	Pretreatment Method	Percentage soil types: Clay	Silt†	Sand	Gravel	Cobbles
├──┤	BATCH 2-I	-	-	9.2	N/A	0	5	73	22	0
├ ─ ─┤	BATCH 2-II	-	-	9.2	N/A	0	3	80	17	0

* Tested in accordance with the following clauses of BS 1377: Part 2: 1990:

9.2	Wet sieve
9.3	Dry sieve
9.4	Sedimentation by pipette
9.5	Sedimentation by hydrometer

† Note: Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

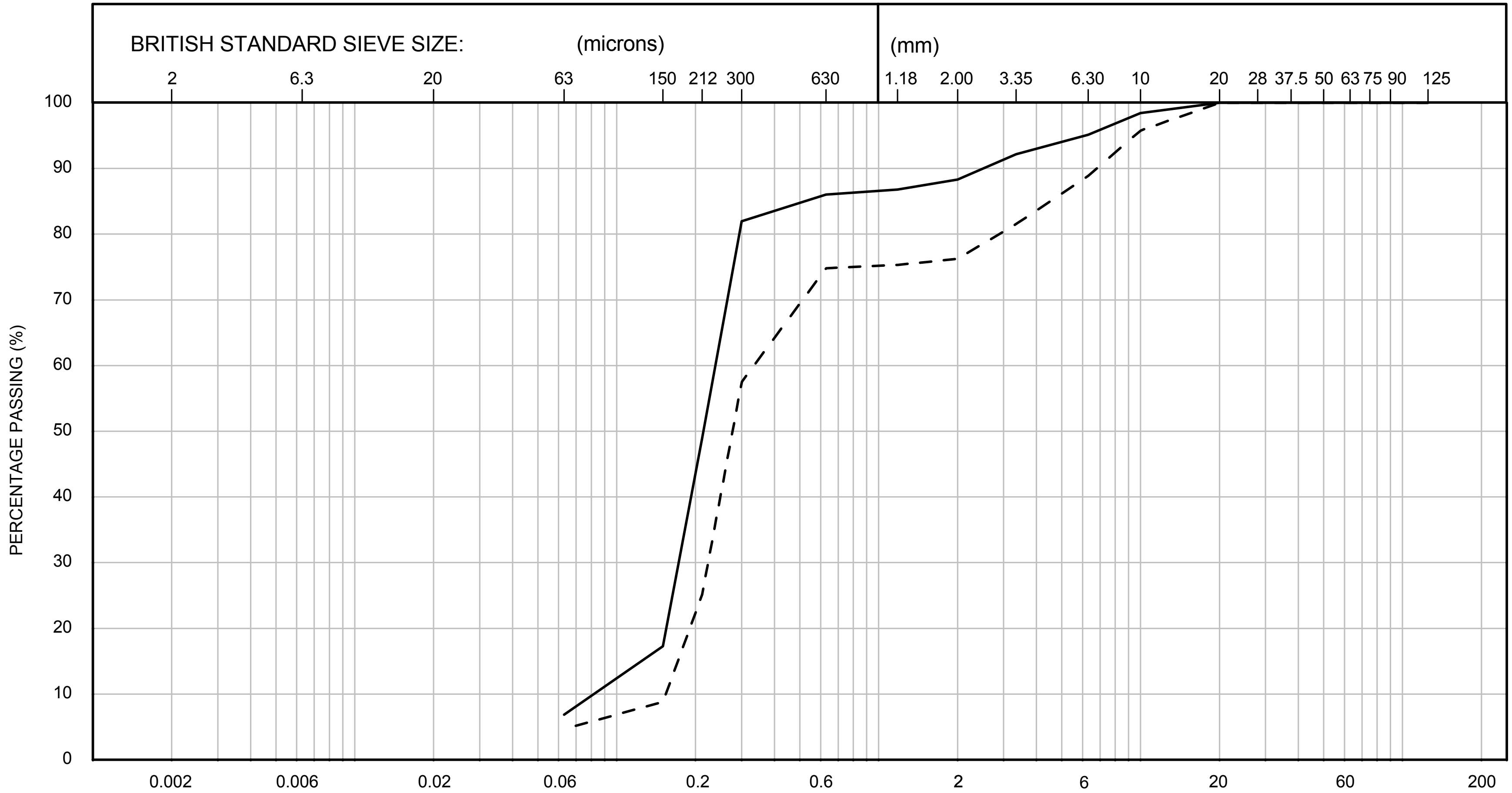
FUGRO

Fugro Report No. N6170/01 (2)

THERMAL CONDUCTIVITY AND SEABED TEMPERATURE
BORSSELE WIND FARM ZONE

PARTICLE SIZE DISTRIBUTION TEST RESULTS

BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA



CLAY	SILT			SAND			GRAVEL			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	

Curve	Batch	Sample	Depth	BS Test * Method	Pretreatment Method	Percentage soil types: Clay	Silt†	Sand	Gravel	Cobbles
———	BATCH 3-I	-	-	9.2	N/A	0	6	83	11	0
– – –	BATCH 3-II	-	-	9.2	N/A	0	4	73	23	0

* Tested in accordance with the following clauses of BS 1377: Part 2: 1990:

9.2	Wet sieve
9.3	Dry sieve
9.4	Sedimentation by pipette
9.5	Sedimentation by hydrometer

† Note: Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

FUGRO

SECTION A3: SAMPLE MICRO PHOTOGRAPHS

LIST OF PLATES IN SECTION A3: Plate

A - BH-WFS3 Batch 1-II
Unwashed
FOV - 10 mm

B - BH-WFS3 Batch 1-II
Washed
FOV - 5 mm

C - BH-WFS3 Batch 2-I
Unwashed
FOV - 10 mm

D - BH-WFS3 Batch 2-I
Washed
FOV - 5 mm

E - BH-WFS3-5 Batch 3-I
Unwashed
FOV - 10 mm

F- BH-WFS3-5 Batch 3-I
Washed
FOV - 5 mm

G- BH-WFS4-5 Batch 3-II
Unwashed
FOV - 10 mm

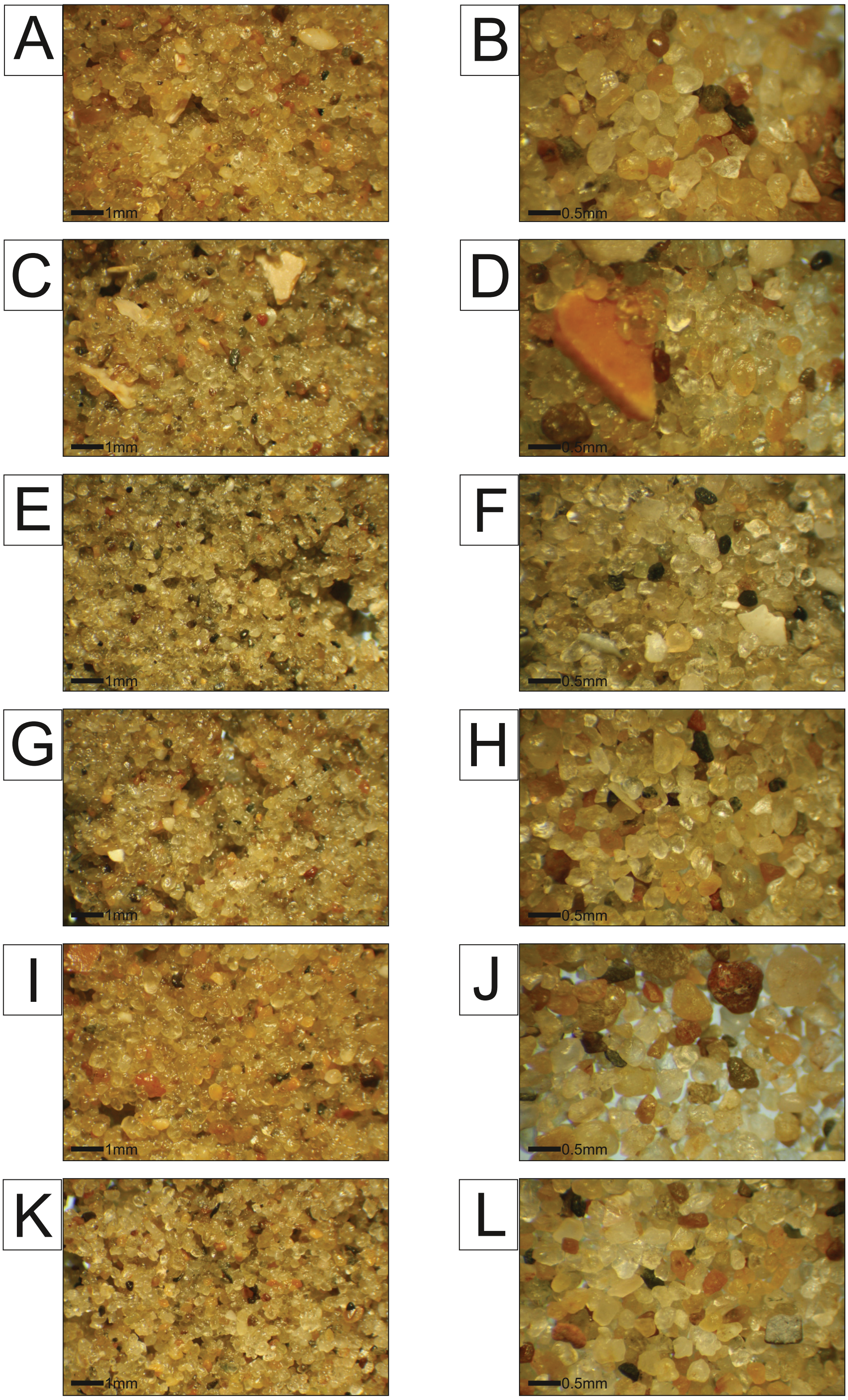
H - BH-WFS4-5 Batch 3-II
Washed
FOV - 5 mm

I - BH-WFS4-7 Batch 2-II
Unwashed
FOV - 10 mm

J - BH-WFS4-7 Batch 2-II
Washed
FOV - 5 mm

K - BH-WFS7 Batch 1-I
Unwashed
FOV - 10 mm

L - BH-WFS7 Batch 1-I
Washed
FOV - 5 mm

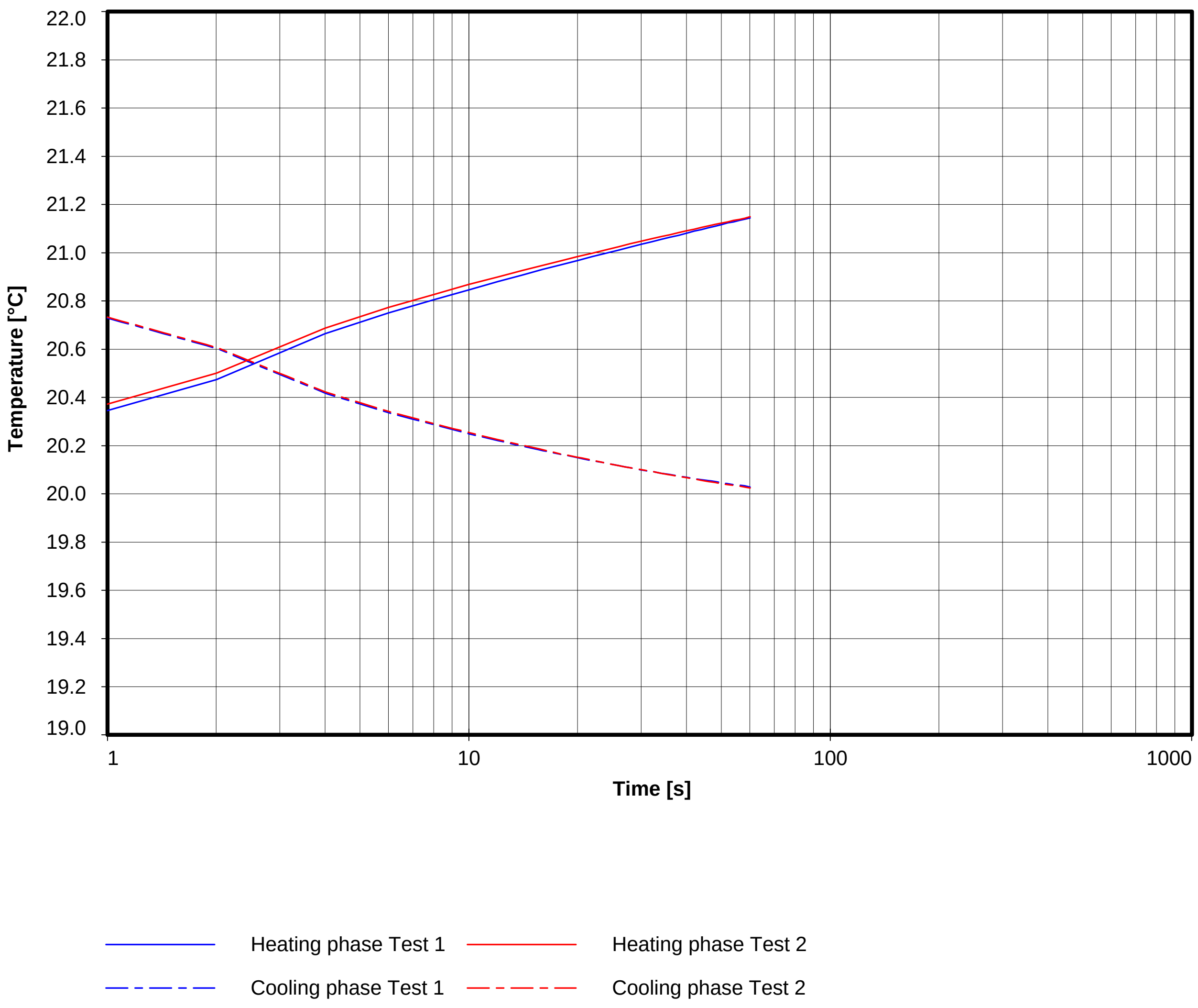


SAMPLE MICRO PHOTOGRAPHS

BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

SECTION A4: THERMAL CONDUCTVITY TESTS

LIST OF PLATES IN SECTION A4: Plate

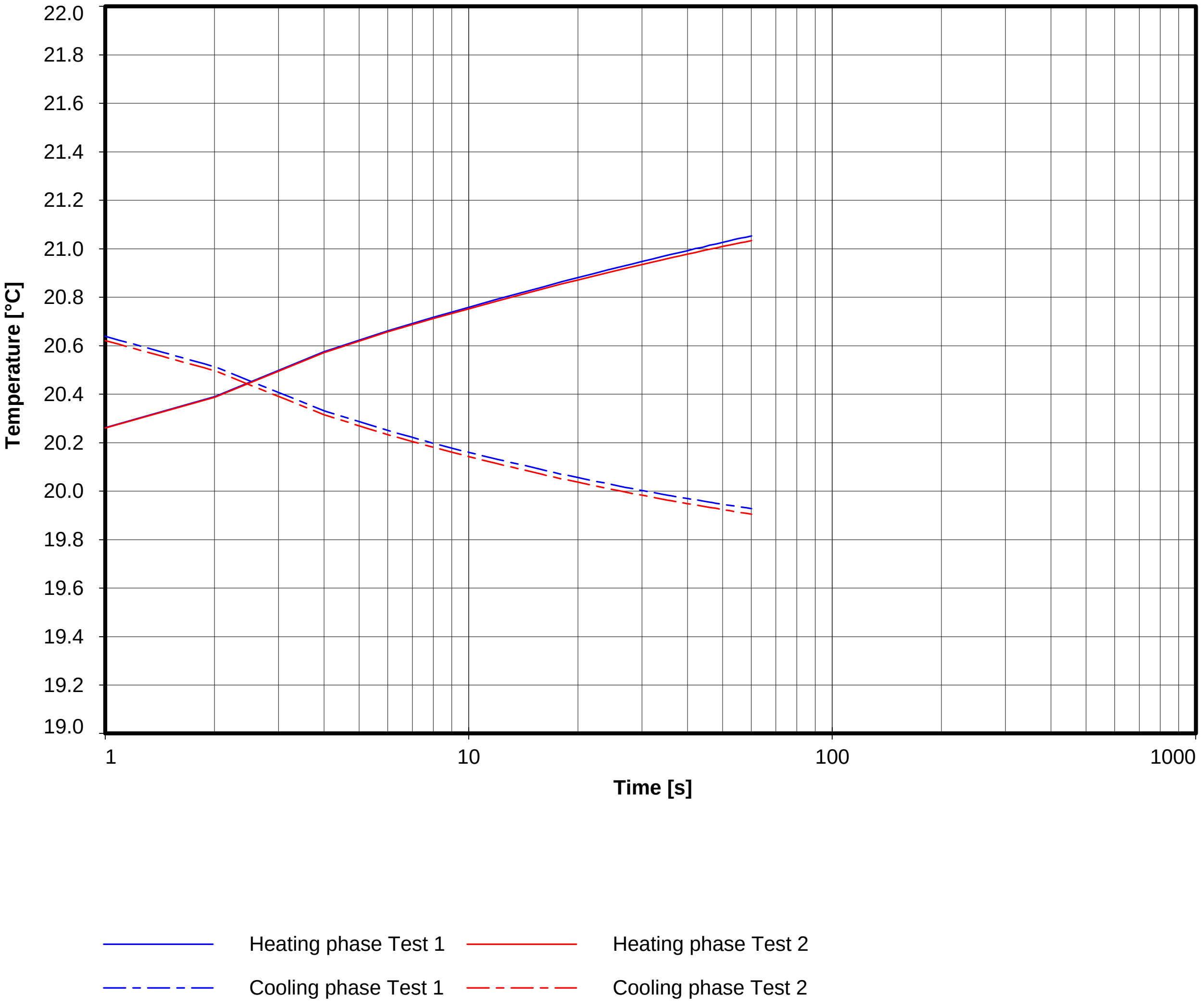


Test 1		**Test 2**	
Batch	: BATCH1-I	Batch	: BATCH1-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.4 Mg/m^3	Dry density	: 1.4 Mg/m^3
Relative density	: 33 %	Relative density	: 33 %
Error	: 0.0017	Error	: 0.0029
Date	: 24/05/2016	Date	: 24/05/2016
Applied power	: 4.22 W/m	Applied power	: 4.21 W/m
Thermal conductivity	: 2.03 W/(m.K)	Thermal conductivity	: 2.06 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 10:52:00

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

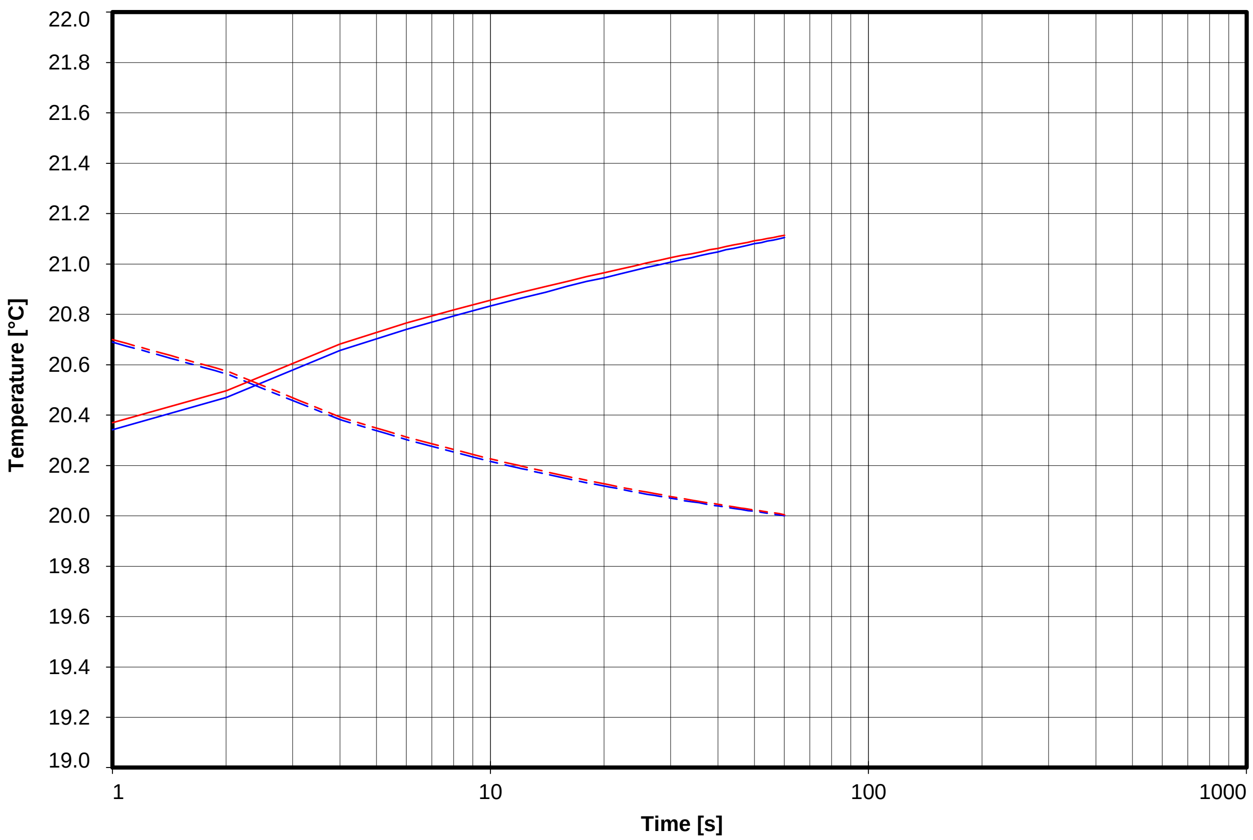


Test 1			**Test 2**		
Batch	:	BATCH1-I	Batch	:	BATCH1-I
Soil type	:	SAND	Soil type	:	SAND
Dry density	:	1.46 Mg/m^3	Dry density	:	1.46 Mg/m^3
Relative density	:	63 %	Relative density	:	63 %
Error	:	0.0026	Error	:	0.0033
Date	:	24/05/2016	Date	:	24/05/2016
Applied power	:	4.2 W/m	Applied power	:	4.19 W/m
Thermal conductivity	:	2.01 W/(m.K)	Thermal conductivity	:	2.04 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 10:54:25

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

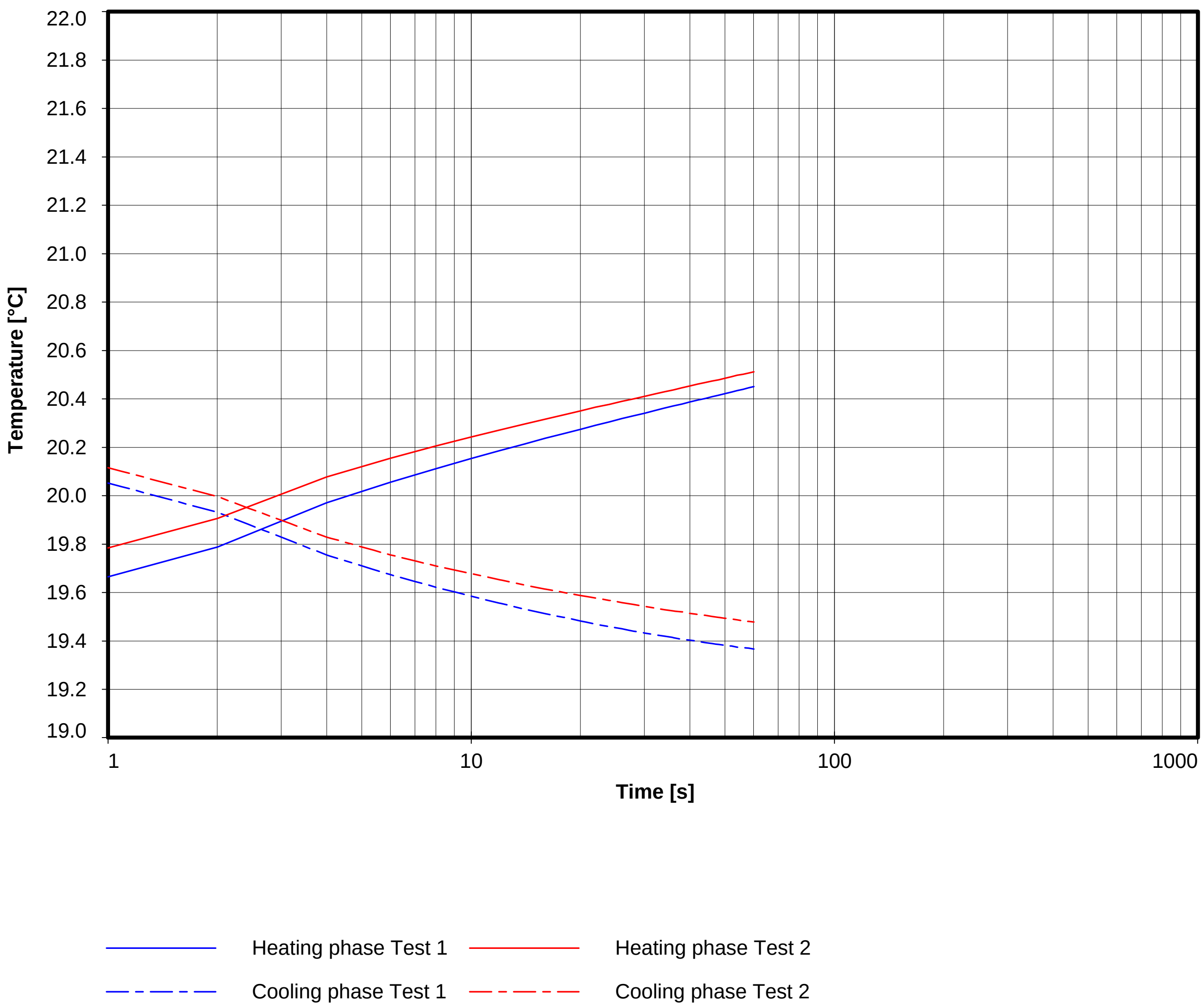


Test 1			**Test 2**	
Batch	: BATCH1-I		Batch	: BATCH1-I
Soil type	: SAND		Soil type	: SAND
Dry density	: 1.52 Mg/m^3		Dry density	: 1.52 Mg/m^3
Relative density	: 91 %		Relative density	: 91 %
Error	: 0.0032		Error	: 0.0036
Date	: 24/05/2016		Date	: 24/05/2016
Applied power	: 4.18 W/m		Applied power	: 4.16 W/m
Thermal conductivity	: 2.17 W/(m.K)		Thermal conductivity	: 2.21 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 10:56:34

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

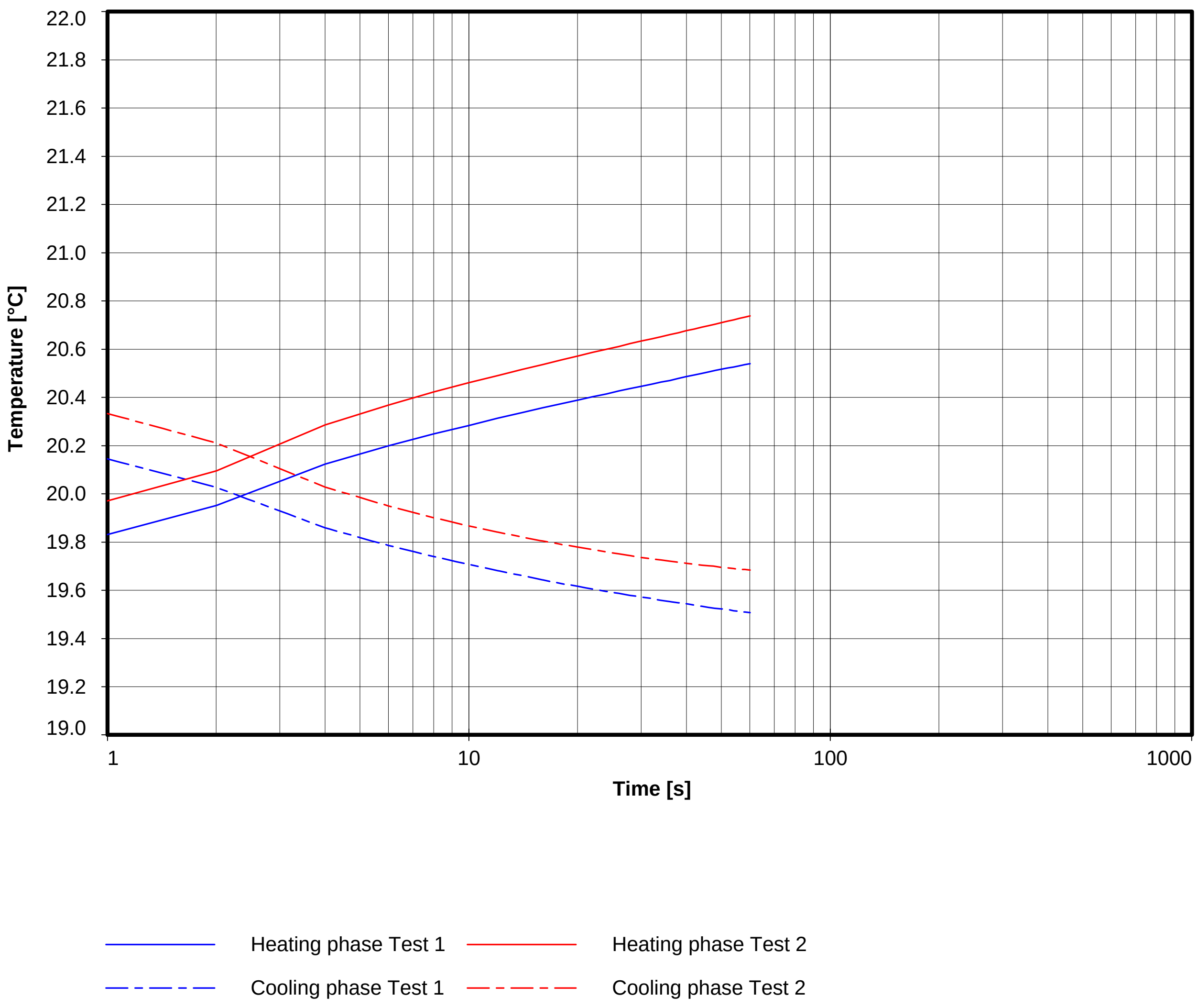


Test 1		**Test 2**	
Batch	: BATCH1-II	Batch	: BATCH1-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.43 Mg/m^3	Dry density	: 1.43 Mg/m^3
Relative density	: 28 %	Relative density	: 28 %
Error	: 0.0026	Error	: 0.0021
Date	: 27/05/2016	Date	: 27/05/2016
Applied power	: 4.03 W/m	Applied power	: 4.02 W/m
Thermal conductivity	: 1.99 W/(m.K)	Thermal conductivity	: 2.12 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 10:58:56

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

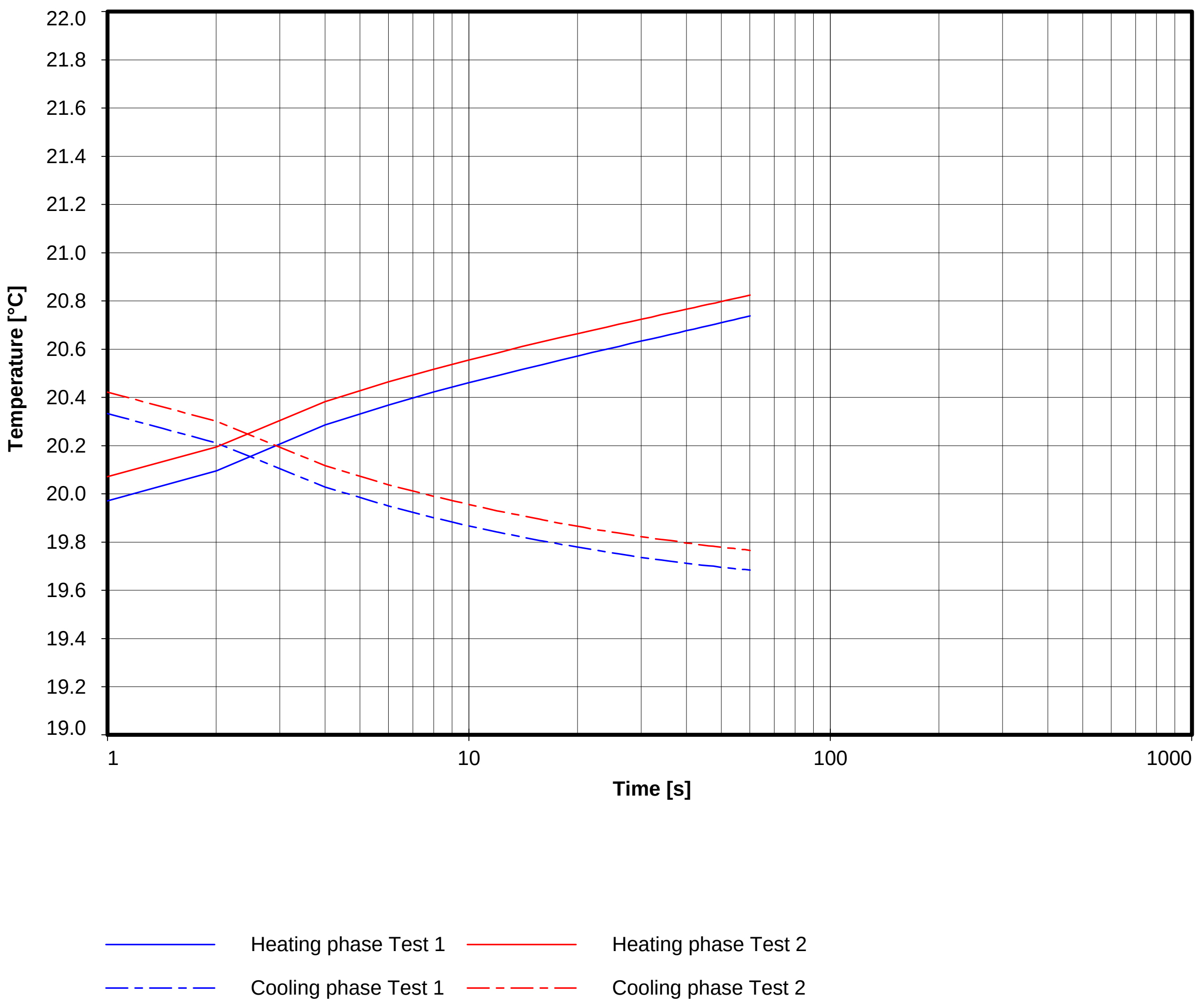


Test 1		**Test 2**	
Batch	: BATCH1-II	Batch	: BATCH1-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.49 Mg/m^3	Dry density	: 1.49 Mg/m^3
Relative density	: 59 %	Relative density	: 59 %
Error	: 0.0023	Error	: 0.0027
Date	: 27/05/2016	Date	: 27/05/2016
Applied power	: 4.01 W/m	Applied power	: 4 W/m
Thermal conductivity	: 2.19 W/(m.K)	Thermal conductivity	: 2.2 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:16:27

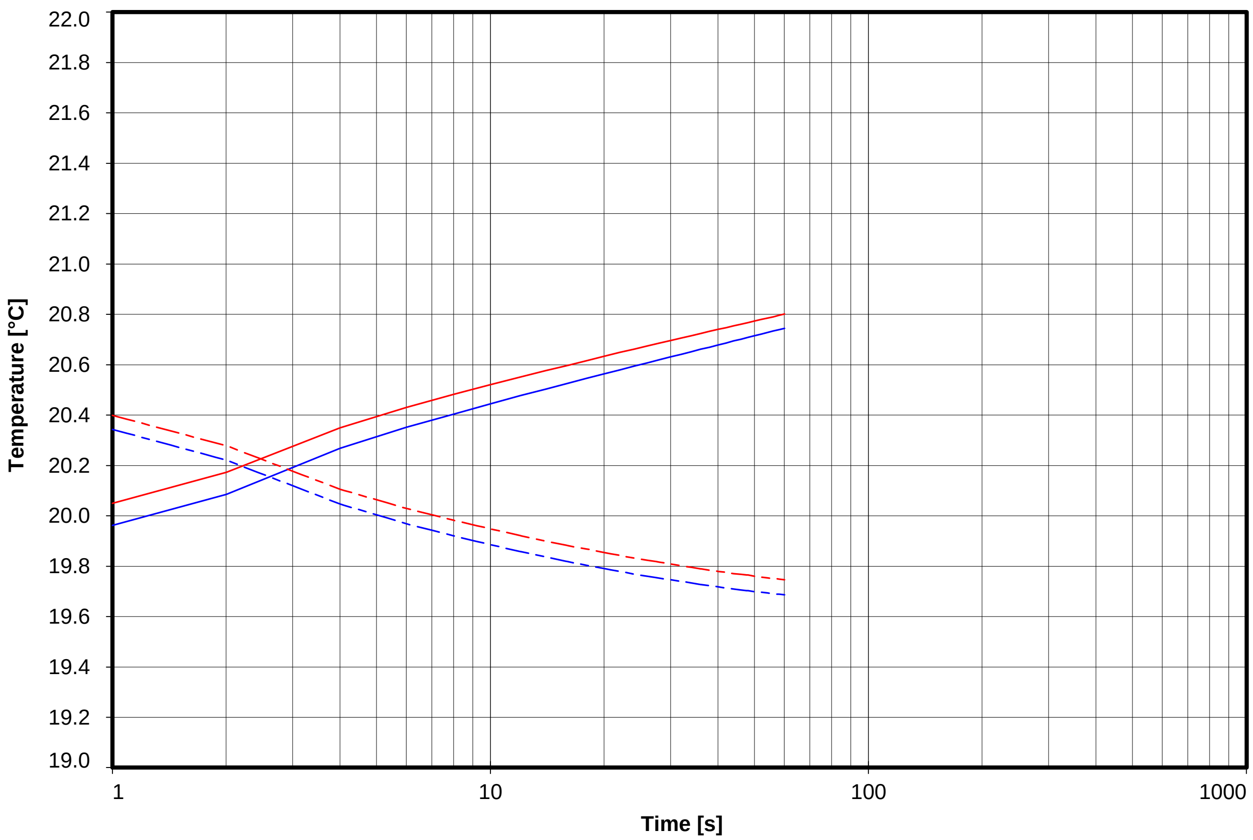
THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA



Test 1		**Test 2**	
Batch	: BATCH1-II	Batch	: BATCH1-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.56 Mg/m^3	Dry density	: 1.56 Mg/m^3
Relative density	: 91 %	Relative density	: 91 %
Error	: 0.0027	Error	: 0.0028
Date	: 27/05/2016	Date	: 27/05/2016
Applied power	: 4 W/m	Applied power	: 4 W/m
Thermal conductivity	: 2.2 W/(m.K)	Thermal conductivity	: 2.21 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

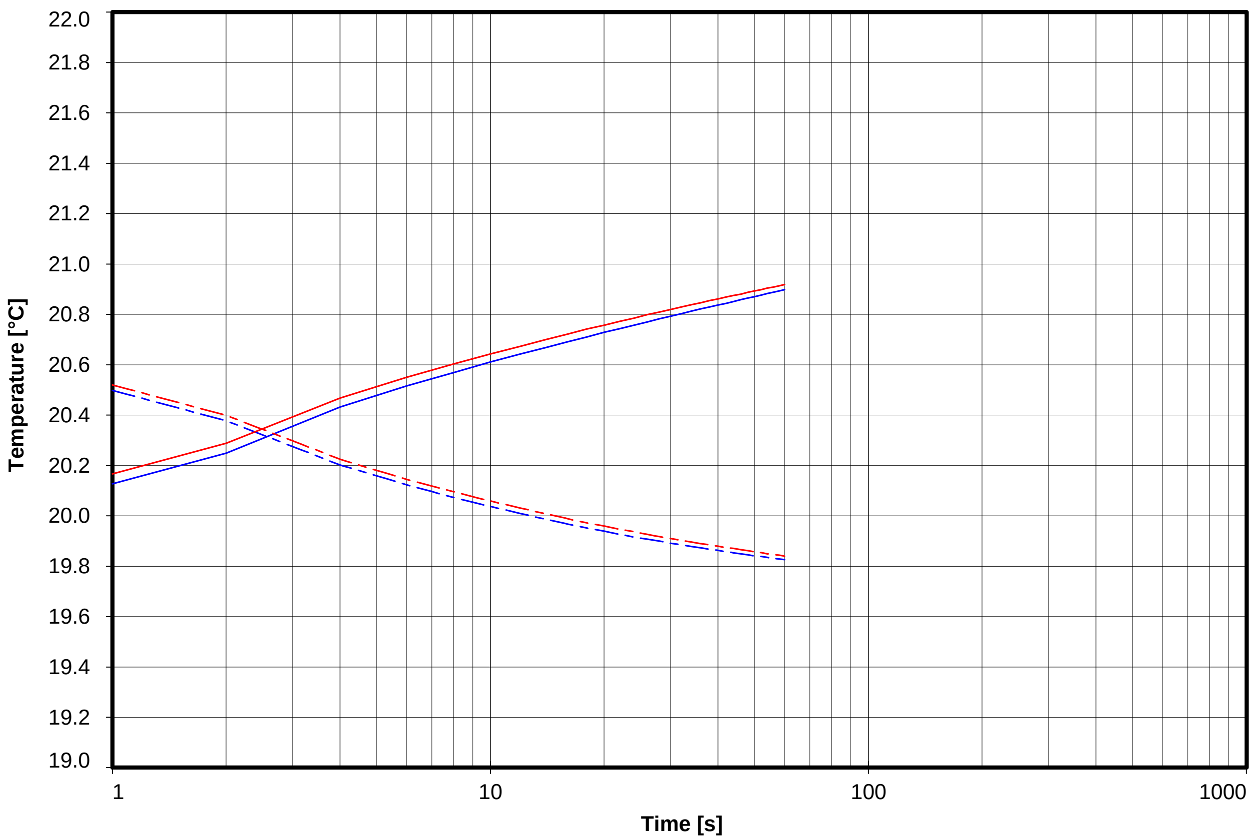


Test 1		**Test 2**	
Batch	: BATCH2-I	Batch	: BATCH2-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.43 Mg/m^3	Dry density	: 1.43 Mg/m^3
Relative density	: 34 %	Relative density	: 34 %
Error	: 0.0022	Error	: 0.0027
Date	: 26/05/2016	Date	: 26/05/2016
Applied power	: 4.08 W/m	Applied power	: 4.07 W/m
Thermal conductivity	: 2.05 W/(m.K)	Thermal conductivity	: 2.12 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:23:23

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

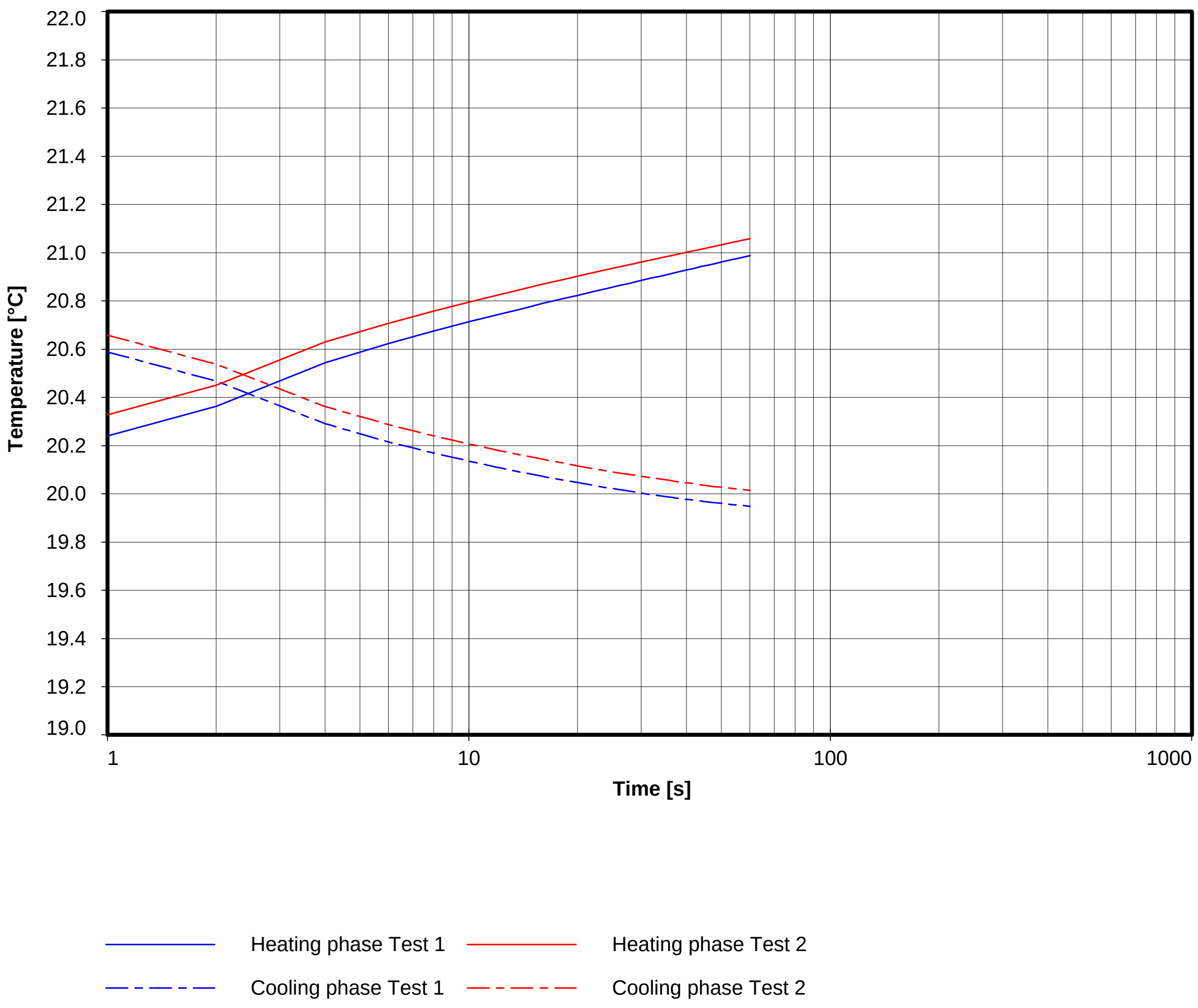


Test 1		**Test 2**	
Batch	: BATCH2-I	Batch	: BATCH2-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.51 Mg/m^3	Dry density	: 1.51 Mg/m^3
Relative density	: 60 %	Relative density	: 60 %
Error	: 0.0028	Error	: 0.0031
Date	: 26/05/2016	Date	: 26/05/2016
Applied power	: 4.07 W/m	Applied power	: 4.05 W/m
Thermal conductivity	: 2.07 W/(m.K)	Thermal conductivity	: 2.09 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:25:21

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

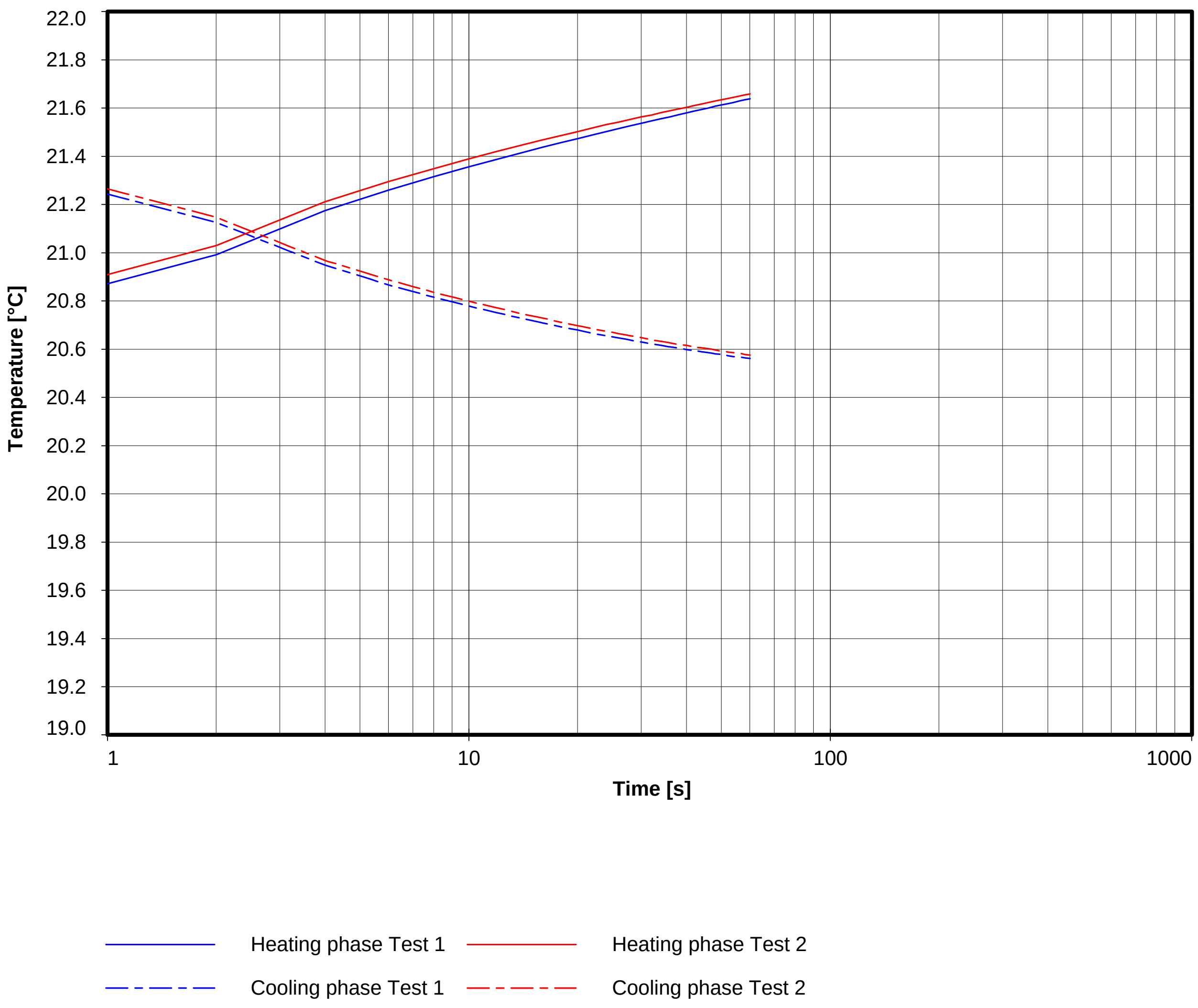


Test 1		Test 2	
Batch	: BATCH2-I	Batch	: BATCH2-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.61 Mg/m^3	Dry density	: 1.61 Mg/m^3
Relative density	: 89 %	Relative density	: 89 %
Error	: 0.0033	Error	: 0.0036
Date	: 26/05/2016	Date	: 26/05/2016
Applied power	: 4.05 W/m	Applied power	: 4.03 W/m
Thermal conductivity	: 2.21 W/(m.K)	Thermal conductivity	: 2.24 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:27:21

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

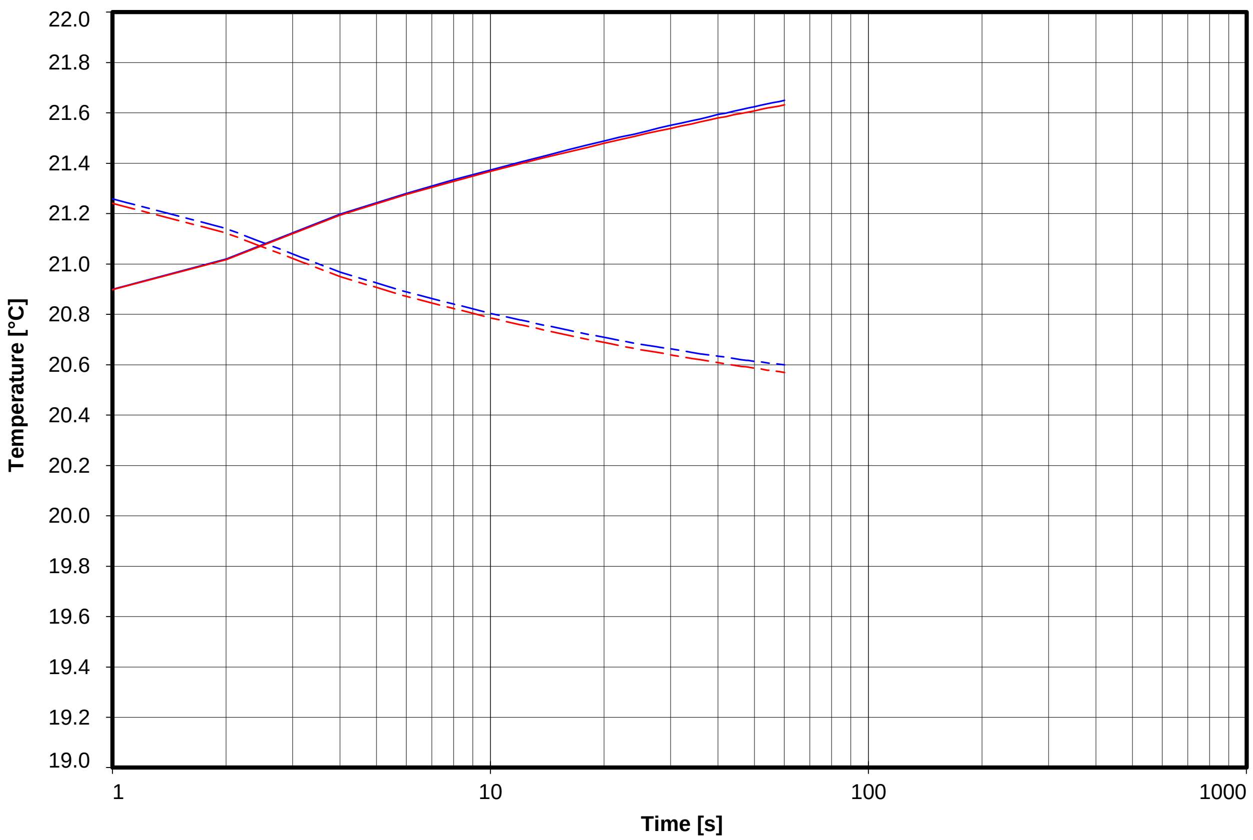


Test 1			**Test 2**		
Batch	: BATCH2-II		Batch	: BATCH2-II	
Soil type	: SAND		Soil type	: SAND	
Dry density	: 1.43	Mg/m^3	Dry density	: 1.43	Mg/m^3
Relative density	: 32	%	Relative density	: 32	%
Error	: 0.0032		Error	: 0.0032	
Date	: 27/05/2016		Date	: 27/05/2016	
Applied power	: 4	W/m	Applied power	: 4	W/m
Thermal conductivity	: 2.04	W/(m.K)	Thermal conductivity	: 2.07	W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:29:29

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

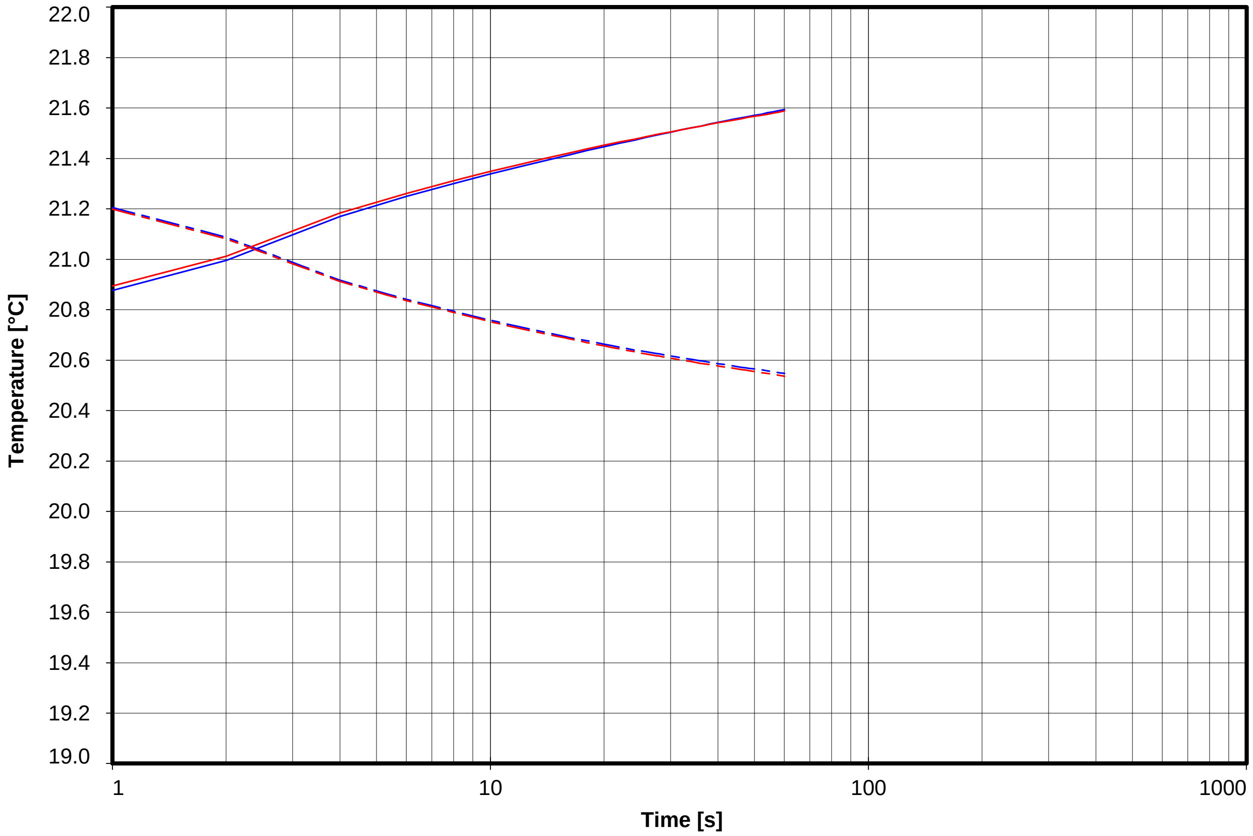


Test 1		**Test 2**	
Batch	: BATCH2-II	Batch	: BATCH2-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.51 Mg/m^3	Dry density	: 1.51 Mg/m^3
Relative density	: 60 %	Relative density	: 60 %
Error	: 0.0038	Error	: 0.0029
Date	: 27/05/2016	Date	: 27/05/2016
Applied power	: 3.99 W/m	Applied power	: 3.98 W/m
Thermal conductivity	: 2.11 W/(m.K)	Thermal conductivity	: 2.1 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:31:14

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

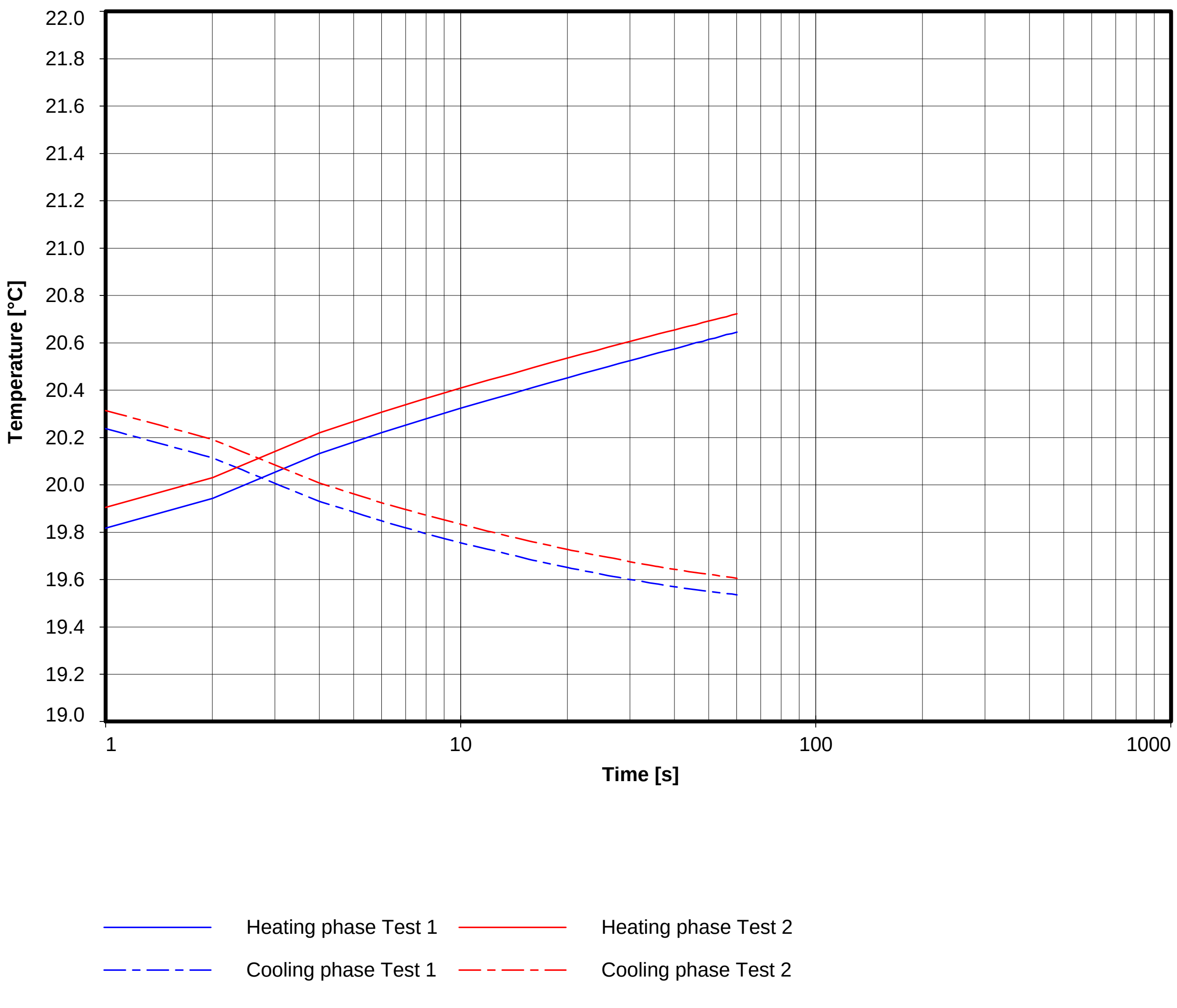


Test 1		**Test 2**	
Batch	: BATCH2-II	Batch	: BATCH2-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.61 Mg/m^3	Dry density	: 1.61 Mg/m^3
Relative density	: 91 %	Relative density	: 91 %
Error	: 0.0036	Error	: 0.004
Date	: 27/05/2016	Date	: 27/05/2016
Applied power	: 3.97 W/m	Applied power	: 3.96 W/m
Thermal conductivity	: 2.17 W/(m.K)	Thermal conductivity	: 2.24 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:32:57

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

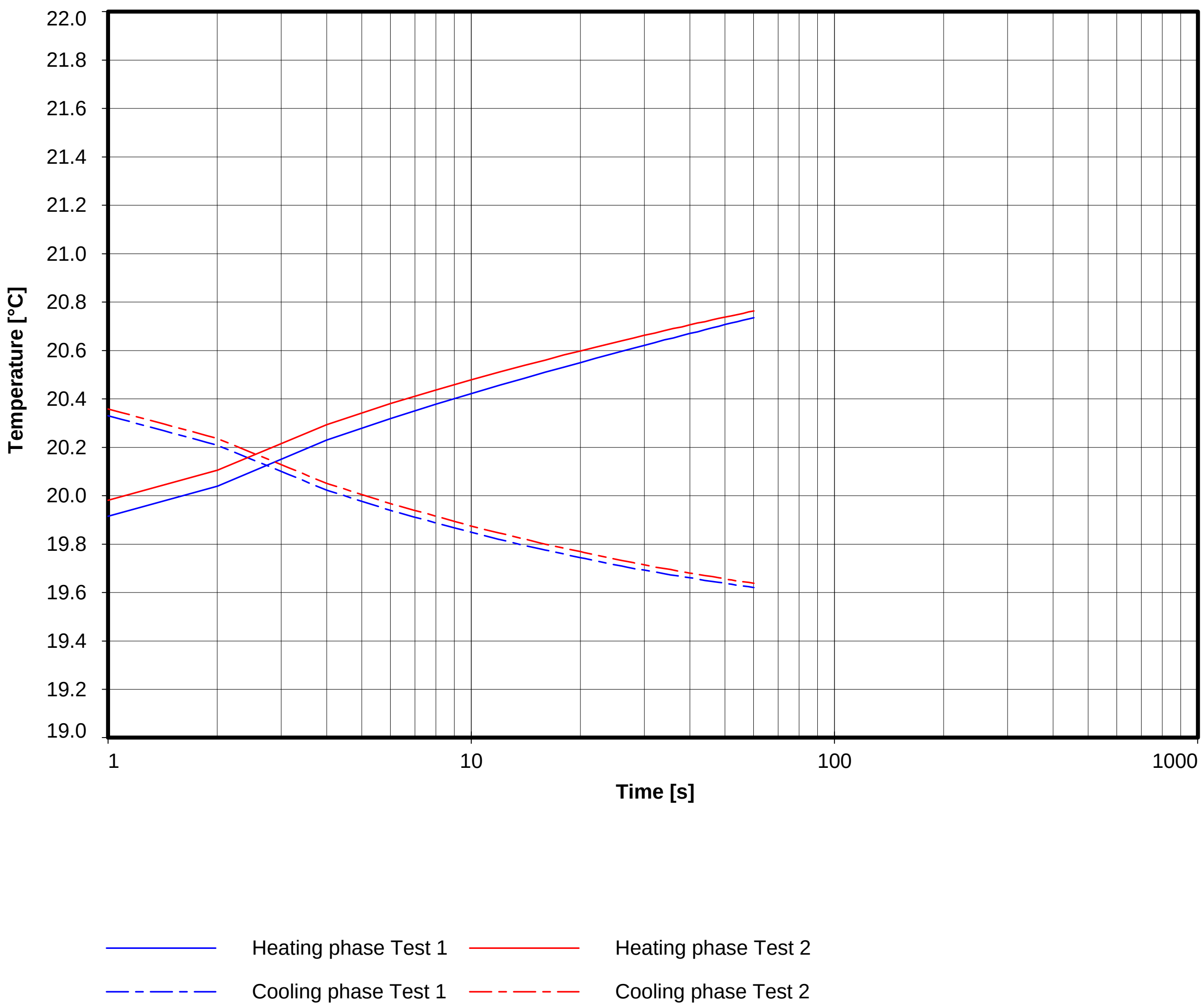


Test 1		**Test 2**	
Batch	: BATCH3-I	Batch	: BATCH3-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.4 Mg/m^3	Dry density	: 1.4 Mg/m^3
Relative density	: 30 %	Relative density	: 30 %
Error	: 0.0035	Error	: 0.0035
Date	: 25/05/2016	Date	: 25/05/2016
Applied power	: 4.1 W/m	Applied power	: 4.09 W/m
Thermal conductivity	: 1.91 W/(m.K)	Thermal conductivity	: 1.9 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:34:40

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

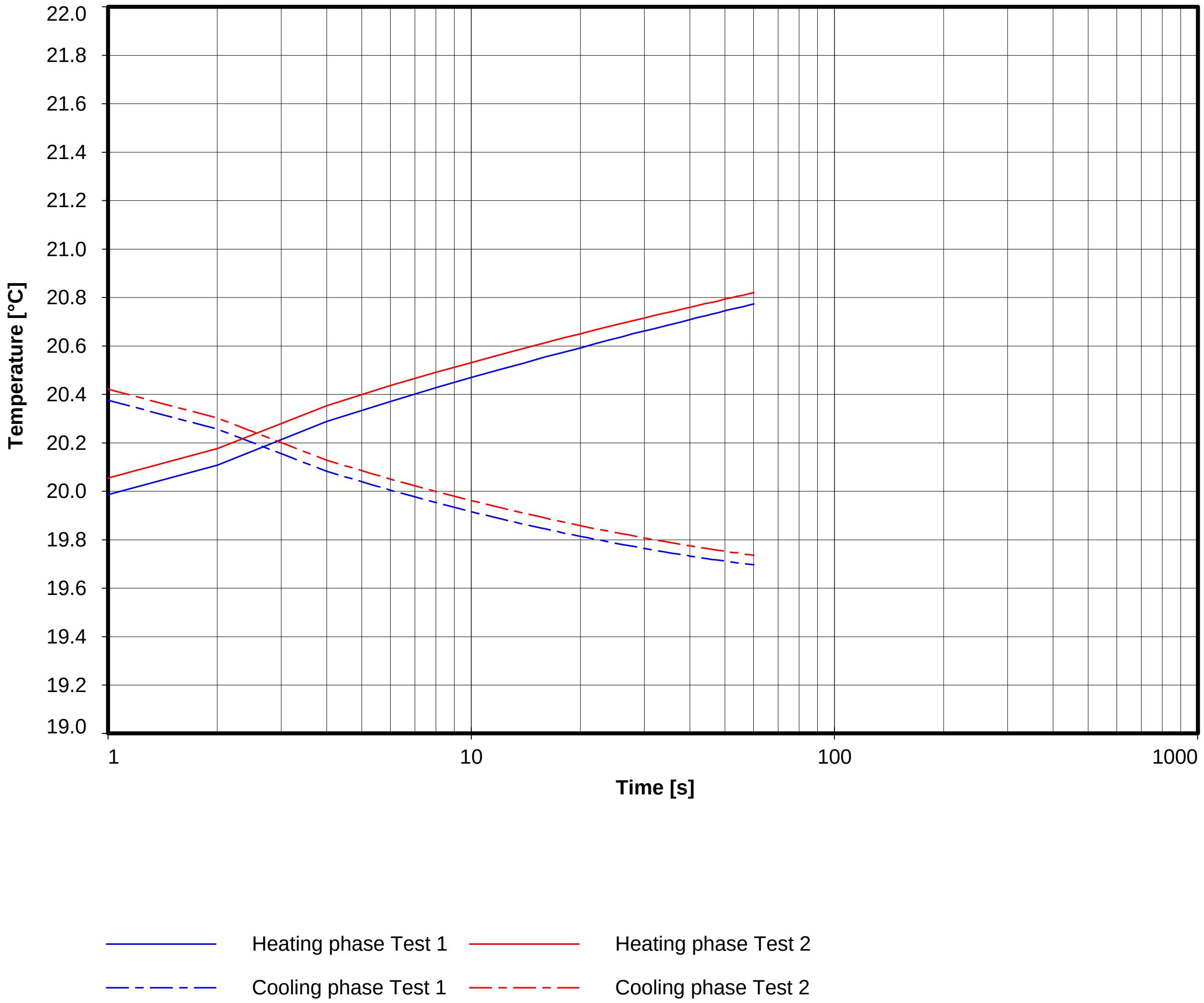


Test 1			**Test 2**		
Batch	:	BATCH3-I	Batch	:	BATCH3-I
Soil type	:	SAND	Soil type	:	SAND
Dry density	:	1.48 Mg/m^3	Dry density	:	1.48 Mg/m^3
Relative density	:	62 %	Relative density	:	62 %
Error	:	0.0029	Error	:	0.004
Date	:	25/05/2016	Date	:	25/05/2016
Applied power	:	4.09 W/m	Applied power	:	4.07 W/m
Thermal conductivity	:	1.9 W/(m.K)	Thermal conductivity	:	1.98 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 11:36:22

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

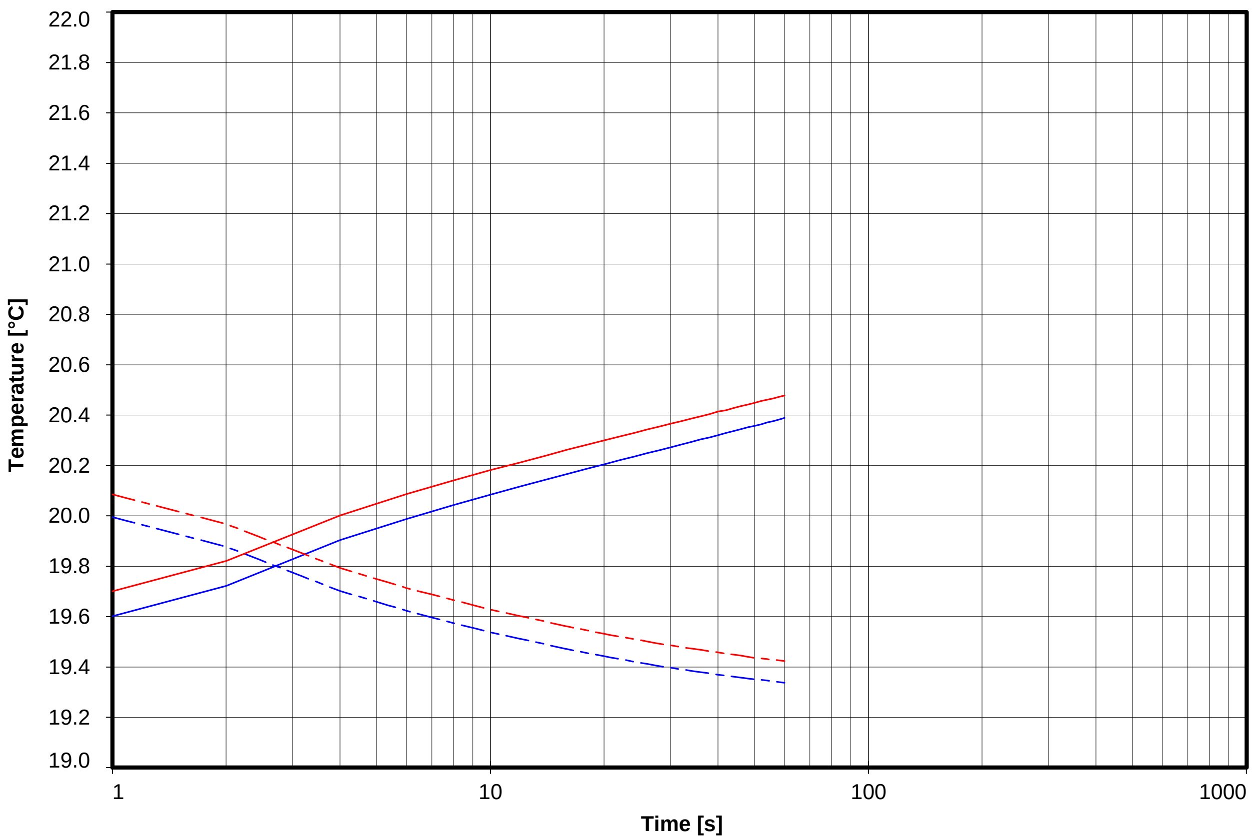


Test 1		**Test 2**	
Batch	: BATCH3-I	Batch	: BATCH3-I
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.5 Mg/m^3	Dry density	: 1.5 Mg/m^3
Relative density	: 70 %	Relative density	: 70 %
Error	: 0.0034	Error	: 0.0032
Date	: 25/05/2016	Date	: 25/05/2016
Applied power	: 4.07 W/m	Applied power	: 4.06 W/m
Thermal conductivity	: 1.96 W/(m.K)	Thermal conductivity	: 2 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 12:07:42

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

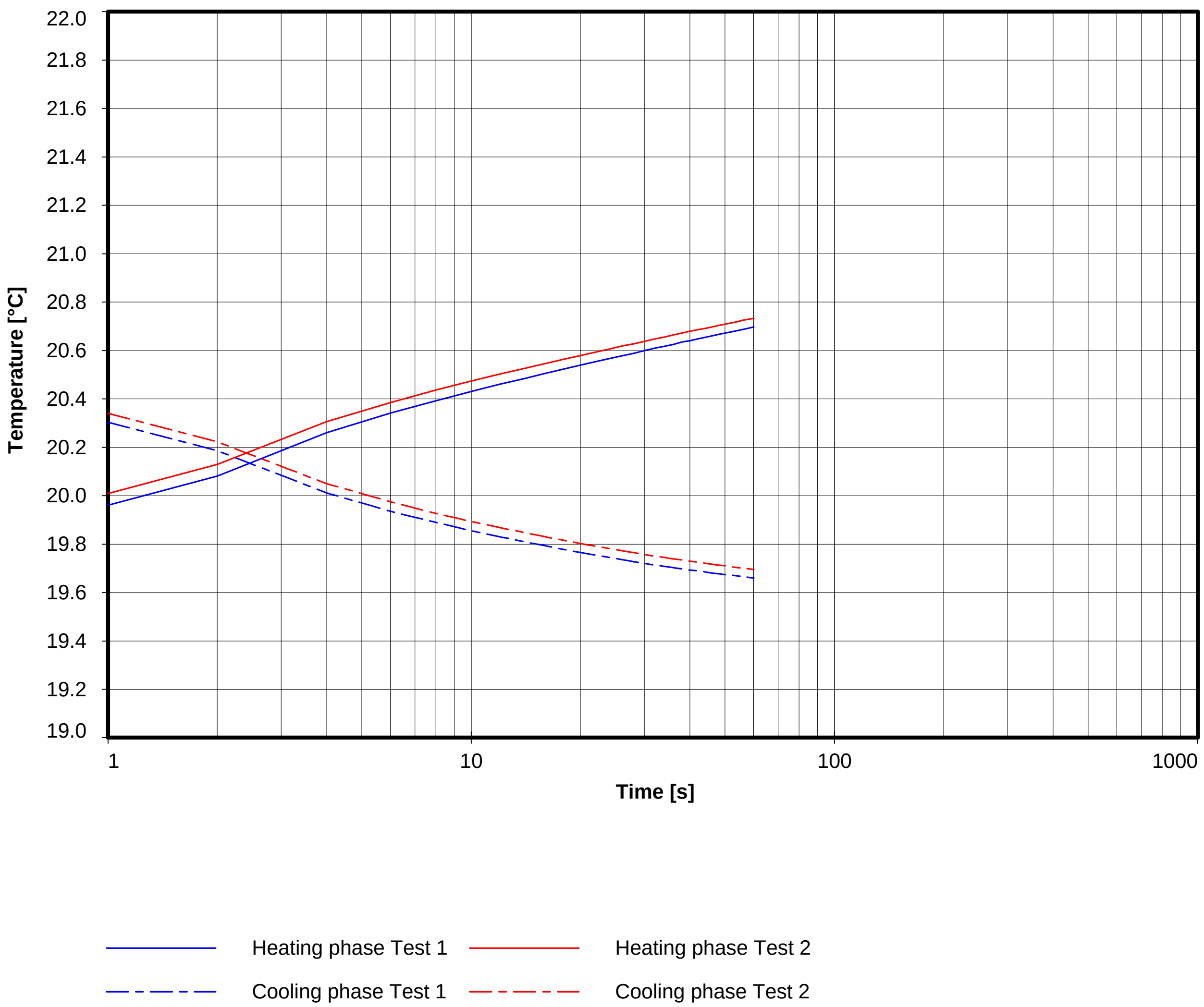


Test 1		**Test 2**	
Batch	: BATCH3-II	Batch	: BATCH3-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.39 Mg/m^3	Dry density	: 1.39 Mg/m^3
Relative density	: 31 %	Relative density	: 31 %
Error	: 0.0024	Error	: 0.0024
Date	: 31/05/2016	Date	: 31/05/2016
Applied power	: 3.97 W/m	Applied power	: 3.96 W/m
Thermal conductivity	: 1.97 W/(m.K)	Thermal conductivity	: 1.98 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 12:10:57

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

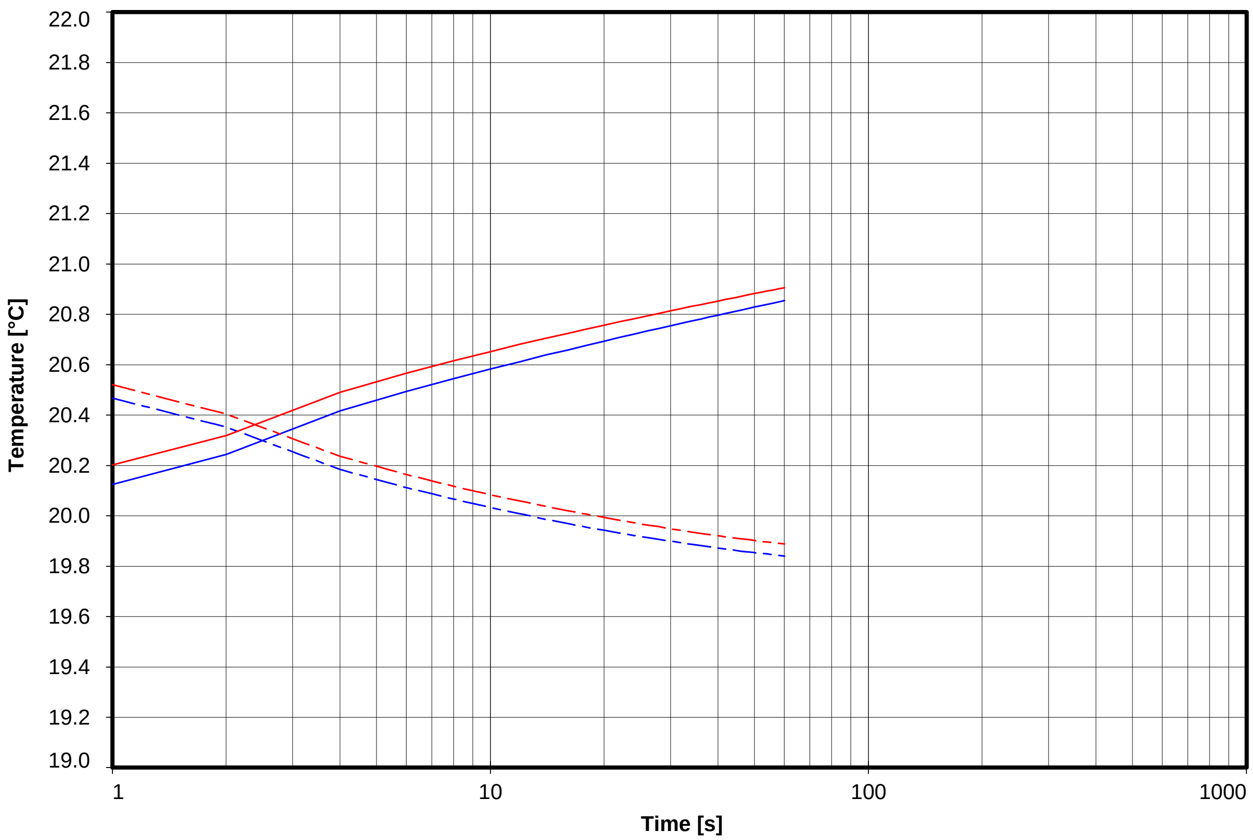


Test 1		**Test 2**	
Batch	: BATCH3-II	Batch	: BATCH3-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.52 Mg/m^3	Dry density	: 1.52 Mg/m^3
Relative density	: 74 %	Relative density	: 74 %
Error	: 0.0031	Error	: 0.0034
Date	: 31/05/2016	Date	: 31/05/2016
Applied power	: 3.96 W/m	Applied power	: 3.94 W/m
Thermal conductivity	: 2.15 W/(m.K)	Thermal conductivity	: 2.17 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 12:12:43

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA



Test 1		**Test 2**	
Batch	: BATCH3-II	Batch	: BATCH3-II
Soil type	: SAND	Soil type	: SAND
Dry density	: 1.57 Mg/m^3	Dry density	: 1.57 Mg/m^3
Relative density	: 89 %	Relative density	: 89 %
Error	: 0.0034	Error	: 0.0036
Date	: 31/05/2016	Date	: 31/05/2016
Applied power	: 3.94 W/m	Applied power	: 3.93 W/m
Thermal conductivity	: 2.15 W/(m.K)	Thermal conductivity	: 2.22 W/(m.K)

Note(s):
- Results calculated from final 2/3 of heating phase (measurements between 40 and 60 seconds)

PPPWIN 03.26.nl/Example_2 tests.ppp/2016-06-18 12:22:44

THERMAL CONDUCTIVITY MEASUREMENTS
BORSSELE WIND FARM ZONE, WFS I TO IV - DUTCH SECTOR, NORTH SEA

SECTION B: USE OF REPORT

CONTENTS Reference

REPORT ISSUE CONTROL

Section	Page No.	Plate No.	Issue No.	Revision
Main Text	All	-	2	Editorial corrections and additions; results of thermal conductivity testing added
Plates following Main Text	-	All	2	Editorial corrections
Section A1	All	-	2	Editorial corrections and additions; section on thermal conductivity testing added
Section A1	-	All	2	Plates A1-1 and A1-2 added
Section A2	-	All	2	Editorial corrections
Section A3	-	A3-1	2	Editorial corrections and additions
Section A4	-	All	2	Results of thermal conductivity testing added

1) The definitive copy of this report is held in Fugro's information system
2) Report distribution is restricted to project participants approved by the Client
3) The *report* issue number is the same as the highest issue number of any individual page
4) Pages of this report are at Issue 1 except those pages listed above
5) The reference at the bottom left-hand corner of each page shows the Fugro report ID and the page issue number (between brackets)

QUALITY MANAGEMENT RECORD

Fugro Project Lead: signed

Report Review and Approval: L.J. Peuchen – Principal Geotechnical Engineer

Report Section	Prepared By	Checked By
Main Text	BBK/ESE	VTT
Plates following Main Text	BBK	VTT
Section A	BBK/ESE	VTT

Person(s):

BBK: B. Klosowska
ESE: E. Schoute
VTT: V. Tertel

INTRODUCTION

This document provides guidelines, recommendations and limitations regarding the use of information in this report.

The cost of geotechnical data acquisition, interpretation and monitoring is a small portion of the total cost of a construction project. By contrast, the costs of correcting a wrongly designed programme or mobilising alternative construction methods are often far greater than the cost of the original investigation. Attention and adherence to the guidelines and recommendations presented in this guide and in the geotechnical report can reduce delays and cost overruns related to geotechnical factors.

This guide applies equally to the use of geotechnical and multi-disciplinary project information and advice.

REQUIREMENTS FOR QUALITY GEOTECHNICAL INFORMATION

Fugro follows ISO 9001 quality principles for project management and ISO 2394 for general principles on reliability for structures. Project activities usually comprise part of specific phases of a construction project. The quality plan for the entire construction project should incorporate geotechnical input in every phase - from the feasibility planning stages to project completion. The parties involved should do the following:

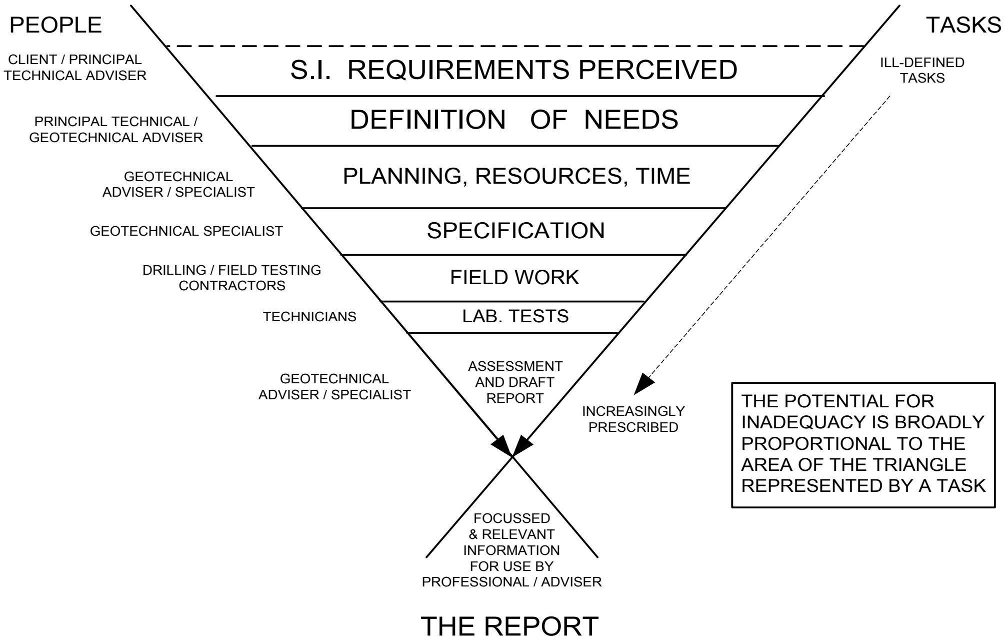
- Provide complete and accurate information necessary to plan an appropriate geotechnical site investigation.
- Describe the purpose(s), type(s) and construction methods of planned structures in detail.
- Provide the time, financial, personnel and other resources necessary for the planning, execution and follow-up of a site investigation programme.
- Understand the limitations and degree of accuracy inherent in the geotechnical data and engineering advice based upon these data.
- During all design and construction activities, be aware of the limitations of geotechnical data and geotechnical engineering analyses/advice, and use appropriate preventative measures.
- Incorporate all geotechnical input in the design, planning, construction and other activities involving the site and structures. Provide the entire geotechnical report to parties involved in design and construction.
- Use the geotechnical data and engineering advice for only the structures, site and activities which were described to Fugro prior to and for the purpose of planning the geotechnical site investigation or geotechnical engineering analysis programme.

AUTHORITY, TIME AND RESOURCES NECESSARY FOR GEOTECHNICAL INVESTIGATIONS

Adequate designation of authority and accountability for geotechnical aspects of construction projects is necessary. This way, an appropriate investigation can be performed, and the use of the results by project design and construction professionals can be optimised.

Figure 1 illustrates the importance of the initial project phases for gathering adequate geotechnical information for a project. The initial phases, when site investigation requirements are defined and resources are allocated, are represented by more than 50% of the Quality Triangle (Figure 1). Decisions and actions made during these phases have a large impact of the outcome and thus the potential of the investigation to meet project requirements.

Figure 1: Quality of Geotechnical Site Investigation (adapted from SISG[1]).



DATA ACQUISITION AND MONITORING PROGRAMMES

Geotechnical investigations are operations of discovery. Investigation should proceed in logical stages. Planning should allow operational adjustments deemed necessary by newly available information. This observational approach permits the development of a sound engineering strategy and reduces the risk of discovering unexpected hazards during or after construction.

GEOTECHNICAL INFORMATION – DATA TYPES AND LIMITATIONS

1. RELIABILITY OF SUPPLIED INFORMATION

Geotechnical engineering can involve the use of information and physical material that is publicly available or supplied by the Client. Examples are geodetic data, geological maps, geophysical records, earthquake data, earlier geotechnical logs and soil samples. Fugro endeavours to identify potential anomalies, but does not independently verify the accuracy or completeness of public or Client-supplied information unless indicated otherwise. This information, therefore, can limit the accuracy of the report.

2. COMPLEXITY OF GROUND CONDITIONS

There are hazards associated with the ground. An adequate understanding of these hazards can help to minimize risks to a project and the site. The ground is a vital element of all structures which rest on or in the ground. Information about ground behaviour is necessary to achieve a safe and economical structure. Often less is known about the ground than for any other element of a structure.

3. GEOTECHNICAL INVESTIGATION - SPATIAL COVERAGE LIMITATIONS

Geotechnical investigations collect data at specific test locations. Interpretation of ground conditions away from test locations is a matter of extrapolation and judgement based on geotechnical knowledge and experience, but actual conditions in untested areas may differ from predictions. For example, the interface between ground materials may be far more gradual or abrupt than a report indicates. It is not realistic to expect a geotechnical investigation to reveal or anticipate every detail of ground conditions. Nevertheless, an investigation can reduce the residual risk associated with unforeseen conditions to a tolerable level. If ground problems do arise, it is important to have geotechnical expertise available to help reduce and mitigate safety and financial risks.

1 Site Investigation Steering Group SISG 1993. *Site Investigation in Construction 2: Planning, Procurement and Quality Management*. London: Thomas Telford.

4. ROLE OF JUDGEMENT AND OPINION IN GEOTECHNICAL ENGINEERING

Geotechnical engineering is less exact than most other design disciplines, and requires extensive judgement and opinion. Therefore, a geotechnical report may contain definitive statements that identify where the responsibility of Fugro begins and ends. These are not exculpatory clauses designed to transfer liabilities to another party, but they are statements that can help all parties involved to recognise their individual responsibilities and take appropriate actions.

COMPLETE GEOTECHNICAL REPORT SHOULD BE AVAILABILE TO ALL PARTIES INVOLVED

To prevent costly construction problems, construction contractors should have access to the best available information. They should have access to the complete original report to prevent or minimize any misinterpretation of site conditions and engineering advice. To prevent errors or omissions that could lead to misinterpretation, geotechnical logs and illustrations should not be redrawn, and users of geotechnical engineering information and advice should confer with the authors when applying the report information and/or recommendations.

GEOTECHNICAL INFORMATION IS PROJECT-SPECIFIC

Fugro’s investigative programmes and engineering assessments are designed and conducted specifically for the Client described project and conditions. Thus this report presents data and/or recommendations for a unique construction project. Project-specific factors for a structure include but are not limited to:

- location
- size and configuration of structure
- type and purpose or use of structure
- other facilities or structures in the area.

Any factor that changes subsequent to the preparation of this report may affect its applicability. A specialised review of the impact of changes would be necessary. Fugro is not responsible for conditions which develop after any factor in site investigation programming or report development changes.

For purposes or parties other than the original project or Client, the report may not be adequate and should not be used.

CHANGES IN SUBSURFACE CONDITIONS AFFECT THE ACCURACY / SUITABILITY OF THE DATA

Ground is complex and can be changed by natural phenomena such as earthquakes, floods, seabed scour and groundwater fluctuations. Construction operations at or near the site can also change ground conditions. This report considers conditions at the time of investigation. Construction decisions should consider any changes in site conditions, regulatory provisions, technology or economic conditions subsequent to the investigation. In general, two years after the report date, the information may be considered inaccurate or unreliable. A specialist should be consulted regarding the adequacy of this geotechnical report for use after any passage of time.

APPENDIX 1: DESCRIPTIONS OF METHODS AND PRACTICES

CONTENTS

This appendix presents method statements and terminology that are generally familiar to expert users of the information.

INTRODUCTION

Fugro employs a range of industry-standard systems for soil description, with additional refinements. The more important systems are:

- British Standard 5930 (ground investigations).
- American Society for Testing and Materials (ASTM) Standards D 2487-11 (Classification of soils for Engineering Purposes) and D 2488-09a (Description and Identification of Soils – Visual-Manual Procedure).
- International Standard ISO 14688-1:2002 (Geotechnical Investigation and Testing - Identification and Classification of Soil: Identification and Description) and International Standard ISO 14688-2:2004 (Principles for a Classification).
- International Standard ISO 19901-8:2014 (Marine Soil Investigations).

The standards are similar, as they are (1) based on the Unified Classification System (Casagrande, 1947), (2) rely on a range of relatively simple visual and manual observations and (3) classify soils according to particle-size distribution and plasticity. Laboratory particle-size distribution and Atterberg limits tests are used to confirm the observations. In addition, the standards include organic soils characterization under soil particle type description.

Significant differences between the standards include the particle-size boundaries and the degree to which plasticity is used as a basis for description. Other differences include the format and order of the soil description.

This document describes a convention that is consistent with either the BS or ASTM standard, and that produces soil descriptions, which can be converted to the other standard. In addition, to describe calcareous soils, Fugro has integrated the carbonate classification system outlined by Clark and Walker (1977) with both British Standard and ASTM systems (Landva et al., 2007). No further information is given about the ISO standards.

British Standard and ASTM systems apply primarily to common terrestrial soils in temperate climates. However, construction activities in coastal areas and offshore can also encounter major carbonate soil deposits. The engineering characteristics of carbonate soil deposits can differ substantially from those of silica-based soil deposits, primarily because of cementation and differences in void ratios.

Appropriate description is necessary. A commonly accepted procedure for calcareous soil deposits is the Clark and walker system, originally developed for the Middle East. This considers particle size, carbonate content and material strength. The particle size classification fits both BS and ASTM system. The carbonate content is an additional feature and the material strength classification relates to common post-depositional alteration of calcareous soil.

This document does not include rock description or specific engineering geological classification systems, such as those for the detailed identification of peat, chalk or micaceous sand.

The main steps of the soil description system are:

1. Measure or estimate particle type as silica-based, organic, or calcareous.
2. For soils that are predominantly silica-based and organic, select BS 5930:1999 or ASTM D 2487 based on local geotechnical practice or project requirements, and follow the appropriate descriptive procedure. For calcareous soils, use the process described by Peuchen et al. (1999).
3. Measure or estimate the particle-size distribution and Atterberg limits (plasticity) for use in defining the principal and secondary soil fractions.
4. Measure or estimate soil strength according to one of the following: (1) relative density of coarse-grained soil, (2) consistency of fine-grained soil, (3) cementation of cemented soil, or (4) lithification of soil undergoing diagenesis.
5. Complete the description using the additional terms for the soil mass characteristics and other features such as bedding, colour, and particle shape.

CALCAREOUS SOIL DESCRIPTION

The procedure considers particle size, carbonate content and material strength. The particle-size classification follows the Unified Soil Classification System. The carbonate content is an additional feature and the material strength classification relates to common post-depositional alteration of calcareous soil.

PARTICLE TYPE

The first determinant for soil description is particle type using Table 1. It mainly differentiates between silica and carbonate soil compositions with organic content of less than 1% of the dry weight. Organic soils are further described in the soil description procedures for BS and ASTM (Table 4).

TABLE 1 - PARTICLE TYPE

Clay soil	Other Soils	Carbonate Content (by dry weight)	Reaction with HCl (10%)
--	Silica	< 10 %	In clays: no bubbles, or slowly forming bubbles. In sands: reaction often limited to some individual particles, or particle surface Residue - Nearly all soil remaining
Calcareous	Calcareous silica	10 to 50	In clays: clearly visible, prolonged reaction and foaming. In sand: violent reaction Residue - Large part of soil remaining
Carbonate	Siliceous carbonate	50 to 90	Violent reaction Residue - Only small part of soil remaining
Carbonate	Carbonate	> 90	Violent reaction Residue - Hardly any soil remaining

The description method does not distinguish between types of carbonate material, and assumes that non-carbonate particles are siliceous.

CEMENTATION AND LITHIFICATION

Cementation is the process by which a binding material precipitates in the voids between the grains or minerals. Lithification is the process by which a soil is hardened due to pressure solution and transformation or new grain or mineral growth. Both processes contribute to the formation of rock.

The descriptions for cementation follow rock strength classification (Table 2) expressed as uniaxial compressive strength σ_c:

TABLE 2 – CEMENTATION

Cementation	σ_c [MPa]
Slightly cemented	0.3 to 1.25
Moderately cemented	1.25 to 5.0
Well cemented	5.0 to 12.5

The term "well cemented" in Table 2 applies to soil, which also shows sublayers with little or no cementation. In case of further lithification, the soil description becomes a rock description using Table 3. The rock strength is only indicative.

TABLE 3 - LITHIFICATION

Carbonate content [%]	Dominant fraction: Clay	Silt	Sand	Gravel	Cobbles	Boulders	σ_c [MPa]
incomplete lithification							
< 10	CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE	CONGLOMERATE or BRECCIA		0.3 to 12.5
10 to 50	Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE			
50 to 90	Clayey CALCILUTITE	Siliceous CALCISILTITE	Siliceous CALCARENITE	Conglomeratic CALCIRUDITE			
> 90	CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE			
complete lithification							
< 50	CLAYSTONE	SILTSTONE	SANDSTONE	GRAVEL CONGLOMERATE	CONGLOMERATE or BRECCIA		>12.5
> 50	Fine-grained Argillaceous LIMESTONE	Fine-grained Siliceous LIMESTONE	Medium grained LIMESTONE	Conglomeratic LIMESTONE			

The Clark and Walker system does not include reef limestone (biolithite). **Reef limestone** represents an in situ accumulation of biological origin (e.g. coral reef) and consists largely of carbonate skeletal material of colonising organisms. The carbonate content normally exceeds 90%. Classification of strength follows rock description procedures.

SOIL DESCRIPTION USING BS 5930:1999

In the following sections, each of the main characteristics is described in the order most commonly used for soil identification, with some portions of the text quoted (shown within quotation marks) or paraphrased from the BS 5930.

SOIL GROUP (BS)

The soil group subdivides the soils into very coarse, coarse, fine, and organic soils.

<u>Very coarse</u> soils consist of cobbles and boulders, with particles larger than 60 mm in diameter. These soil particles are rarely sampled using standard soil sampling techniques. They are described separately, and not included when determining the proportions of the other soil components.

The initial classification of silica soils as <u>coarse</u> or <u>fine</u> is based on the percentage of fine particles after the very coarse particles are removed. In BS 5930, the boundary between coarse (i.e. sands and gravels) and fines (i.e. silts and clays) is 0.060 mm (60 µm). When the soil contains approximately 35% or more fines, it is described as a fine soil; further classification of the fine soil as a clay or silt depends on the plasticity of the soil. When the soil contains less than about 35% fine material, it is usually described as a coarse soil. “The boundary between fine and coarse soils is approximate, as it depends on the plasticity of the fine fraction and the grading of the coarse fraction.”

<u>Organic soils</u> contain usually small quantities of dispersed organic matter that can have a significant effect on soil plasticity. Organic soil descriptions in BS 5930 are based on an organic content by weight determined by loss on ignition. Where organic matter is present as a secondary constituent, the following terms are used:

TABLE 4 - ORGANIC SOIL DESCRIPTIONS

Term	Organic content [% by weight]	Typical colour
Slightly organic clay or silt Slightly organic sand	2 to 5 1 to 3	Grey Same as mineral
Organic clay or silt Organic sand	5 to 10 3 to 5	Dark grey Dark grey
Very organic clay or silt Very organic sand	> 10 > 5	Black Black

Soils with organic contents up to approximately 30% by weight and water contents up to about 250% behave as mineral soils and are described using the terms given in the lower portion of Table 4.

Peat consists predominantly of plant remains, is usually dark brown or black, and has a distinctive smell. It is generally classified according to the degree of decomposition (fibrous, pseudo-fibrous, or amorphous) and strength (firm, spongy, or plastic). When encountered, reference can also be made to the classification given in ASTM Standard Procedure D 4427.

PRINCIPAL SOIL TYPE AND PARTICLE SIZE (BS)

Coarse-Grained Soils

The principal soil type in coarse-grained soils is sand if the dry weight of the sand fraction (0.06 mm to 2 mm particle sizes) exceeds that of the gravel fraction (2 mm to 60 mm particle sizes), and vice versa for gravel.

As an addition to the BS 5930 classification, coarse-grained soils are described as well-graded or poorly-graded based on the grain-size distribution curve, using the coefficient of uniformity (C_U) and, to a lesser extent, the coefficient of curvature (C_C), as follows:
- Sands with ≤12% fines are <u>well-graded</u> when $C_U \geq 6$ and C_C is between 1 and 3.
- Sands are <u>poorly-graded</u> for other values of C_U and C_C.
- Gravels with ≤12% fines are <u>well-graded</u> when $C_U \geq 4$ and C_C is between 1 and 3.
- Gravels are <u>poorly-graded</u> for other values of C_U and C_C.

For coarse-grained soils with fines contents > 12%, these terms are not used.

Sands and gravels are sub-divided into coarse, medium, and fine, as defined in Table 5.

TABLE 5 - SIZE FRACTION DESCRIPTIONS FOR COARSE-GRAINED SOILS

Soil	Particle diameter range [mm]		
	Coarse	Medium	Fine
Gravel	60 to 20	20 to 6	6 to 2
Sand	2 to 0.6	0.6 to 0.2	0.2 to 0.06

Fine-Grained Soils

Fine-grained soils are classified as clay or silt according to the results of Atterberg limits tests. A fine-grained soil is classified as clay if:

$$I_P \geq 6 \text{ and } I_P \geq 0.73\,(w_L\text{-}20)$$

where:
I_P = plasticity index [%]
w_L = liquid limit [%]

Otherwise the dominant soil fraction is silt. The equation $I_P = 0.73\,(w_L-20)$ represents the "A-line" in a plasticity chart. The plasticity chart may also show a “U-line” defined as $I_P = 0.9\,(w_L-8)$ and $w_L \geq 16$, according to Casagrande (1948). The U-line represents an approximate upper limit of correlation between plasticity index and liquid limit for natural soils.

The following additional descriptors (as used in the ASTM soil description procedure) are added:
- Clays with liquid limits of 50% or higher are described as “fat.”
- Clays with liquid limits below 50% are described as “lean.”
- Silts with liquid limits of 50% or higher are termed “elastic silt.”
- Silts with liquid limits below 50% are simply “silts.”

The term “silty clay” is not used, since BS 5930 explicitly states that silt and clay “are to be mutually exclusive.”

Particle Shape

The description of particle shape includes terms for form, angularity, and surface texture. These terms are the same for BS 5930 as for ASTM D 2488. Reference should be made to the corresponding ASTM section of this document.

COMPOSITE (SECONDARY) SOIL TYPES (BS)

BS 5930 defines procedures for assigning secondary soil fractions to coarse-grained soils that are identical for sand and gravel, except that the secondary soil type is sandy when the principal soil type is gravel and vice versa. For fine-grained soils (silt and clay) there is a single procedure for assigning secondary soil fractions. The ranges for the percentages of the secondary constituents are similar to, though different from, those defined by ASTM.

If the principal soil type is sand, secondary soil fractions may be gravelly and silty or clayey (e.g. silty sand). Similarly, if the principal soil type is clay, secondary soil fractions may be sandy or gravelly. Table 6 (from BS 5930) gives the terms to be used for ranges of secondary constituents.

TABLE 6 - DESCRIPTIVE TERMS AND RANGES FOR SECONDARY CONSTITUENTS

Term	Principal soil type	Approximate proportion of secondary constituent	
		Coarse soil	Fine soil
Slightly clayey or silty	SAND and/or GRAVEL		< 5%
Clayey or silty			5% to 20%
Very clayey or silty			> 20% (1)
Slightly sandy or gravelly		< 5%	
Sandy or gravelly		5% to 20%	
Very sandy or gravelly		> 20%	
Slightly sandy and/or gravelly	SILT or CLAY	< 35%	
Sandy and/or gravelly		35% to 65%	
Very sandy and/or gravelly		> 65% (2)	

Notes: (1) or can be described as fine soil depending on engineering behaviour
(2) or can be described as coarse soil depending on engineering behaviour.

COLOUR (BS)

Soil colours are described using the Munsell Soil Colour Charts (Gretag-Macbeth, 2000).

The Munsell colour is arranged according to three variables known as Hue, Value and Chroma. The Hue notation of a colour indicates its relation to red, yellow, green, blue and purple. The Value notation indicates the relative lightness. The Chroma notation indicates the intensity of the colour.

BEDDING/STRATIGRAPHY (BS)

Layers of different soil types within a stratum are called bedding units, and are described in terms of the unit thickness. In an otherwise homogeneous soil, these can be identified as bedding planes or as colour changes, and not necessarily as discontinuities.

Table 7 (from BS 5930) gives terms for bedding/stratigraphy.

TABLE 7 - DESCRIPTIVE TERMS FOR BEDDING/STRATIGRAPHY

Stratified	Bedding	Interbedded	Thickness [mm]
Very thick beds	Very thick bedded	Very thickly interbedded	>2000
Thick beds	Thickly bedded	Thickly interbedded	600 to 2000
Medium beds	Medium bedded	Medium interbedded	200 to 600
Thin beds	Thinly bedded	Thinly interbedded	60 to 200
Very thin beds	Very thinly bedded	Very thinly interbedded	20 to 60
Thick laminae	Thickly laminated	Thickly interlaminated	6 to 20
Thin laminae	Thinly laminated	Thinly interlaminated	<6

Strata with alternating or different beds or laminations can be described as interbedded or interlaminated. Where the soil types are approximately equal, both terms can be used (e.g. thinly interlaminated SAND and CLAY).

Partings are bedding surfaces that separate easily, and typically are laminae of no appreciable thickness. The spacing between partings is described in the same terms as for spacing of discontinuities (Table 8).

DISCONTINUITIES/STRUCTURE (BS)

Discontinuities include fissures and shear planes, and the descriptor refers to the mean spacing between such discontinuities in a soil mass. A soil is “fissured” when it breaks into blocks along unpolished discontinuities, and “sheared” when it breaks into blocks along polished discontinuities (which is equivalent to a slickensided soil). The spacing description ranges from extremely closely spaced (less than 20 mm) to very widely spaced (over 2000 mm). No other descriptive terms are used. An example would be: Firm grey very closely fissured fine sandy calcareous CLAY with many silt partings.

The spacing terms are also used for distances between partings, isolated beds or laminae, desiccation cracks, rootlets, etc.

TABLE 8 - SPACING OF DISCONTINUITIES

Term	Mean spacing range [mm]
Very widely	Over 2000
Widely	600 to 2000
Medium	200 to 600
Closely	60 to 200
Very closely	20 to 60
Extremely closely	Under 20

DENSITY/COMPACTNESS OF GRANULAR SOILS (BS)

Usually, soil description offers little evidence about the density condition of coarse-grained cohesionless (granular) soil samples. The reason for this is the substantial sampling disturbance incurred during conventional sampling operations such as push sampling, percussion sampling, and vibrocoring. Complementary investigation techniques, such as Cone Penetration Tests (CPT), are usually necessary. The strength of a cohesionless soil is normally measured as a function of its relative density (also termed compactness or density index). Relative density is the ratio of the difference between the void ratios of a cohesionless soil in its loosest state and existing natural state to the difference between its void ratio in the loosest and densest states.

Relative density (compactness) is referred to in BS 5930:1999 only in terms of N-values obtained by the Standard Penetration Test (which is not conducted in offshore site investigations). Rather than using SPT-based values, it is common practice to interpret relative density on the basis of CPT results. Ranges of relative density are given in Table 9. These ranges are in common use in the industry. They were originally given in Lambe and Whitman (1979) and in the API RP 2A guidelines generally used for offshore pile design. These terms also apply to cohesionless fine-grained soils.

TABLE 9- RANGE OF RELATIVE DENSITY OF GRANULAR SOILS

Term	Range of relative density [%]
Very loose	Less than 15
Loose	15 to 35
Medium dense	35 to 65
Dense	65 to 85
Very dense	Greater than 85

STRENGTH OF COHESIVE SOILS (BS)

The strength of cohesive soils is given in terms of undrained shear strength, using the terms and ranges given in Table 10, with an additional level to cover "very hard" soils.

TABLE 10 - UNDRAINED SHEAR STRENGTH SCALE FOR COHESIVE SOILS (BS 5930:1999)

Term	Undrained shear strength	
	[kPa]	[ksf] (1)
Very soft	Less than 20	Less than 0.4
Soft	20 to 40	0.4 to 0.8
Firm	40 to 75	0.8 to 1.5
Stiff	75 to 150	1.5 to 3.0
Very stiff	150 to 300	3.0 to 6.0
Hard	300 to 600	6.0 to 12.0
Very hard (2)	Greater than 600	Greater than 12.0

Notes: (1) Unit conversion added to table
(2) Added for global practice.

MINOR CONSTITUENTS (BS)

Percentages of minor constituents within the soil, such as shell or wood fragments, or small soil inclusions (such as partings or pockets), can be quantified using the terms "with trace", "with few", "with" and "with many" (in increasing order). These terms are usually added at the end of the main soil description (e.g. with many shell fragments, with silt pockets, etc.); exceptions are terms such as "shelly", which are more appropriate before the soil group name. For beds of material within a soil matrix, the terminology for spacing and thickness of beds is used. For individual particles of soil or material within a soil matrix, the terms "partings" and "pockets" are used.

SOIL ODOUR (BS)

Describing the odour from soil samples as they are retrieved or extruded on board ship can be useful. Terms used to describe the odour are H_2S, "musty", "putrid" and "chemical". It must be emphasised that soil odour descriptions are unlikely to be fully consistent, because of factors such as variations in sample handling, ambient conditions at time of sample description, and strong dependence on a person's ability to detect and identify odour.

SOIL DESCRIPTION USING ASTM D 2487 AND D 2488

The identification and description of silica soils in the ASTM system consists primarily of a group name and symbol, which are based on the particle-size distribution and the Atterberg limits test results, and the results of other laboratory classification tests.

The main standard for soil description, D 2487 Classification of Soils for Engineering Purposes, is applicable to naturally-occurring soils passing a 3-in. (75-mm) sieve, and identifies three major soil types: coarse-grained, fine-grained, and highly organic soils. The major soil types are further subdivided into 15 specific basic soil groups.

An accompanying Standard, D 2488, outlines the Description and Identification of Soils using a Visual-Manual Procedure. This standard is used primarily in the field, where full particle-size distribution curves and Atterberg limits values are not available. It gives guidance for detailed descriptions of soil particles and soil conditions (e.g. colour, structure, strength, cementation, etc.), which are not included in D 2487.

Soil types with particles larger than 75 mm (i.e. cobbles and boulders) are not included in the Standards, but are identified.

SOIL TYPES (ASTM)

The initial classification of silica soils as coarse-grained or fine-grained is based on the percentage fines, expressed as the percentage of dry weight of the total sample after the very coarse particles are removed, as with BS 5930. However, ASTM has defined the coarse-fine boundary as 0.075 mm (75 μm).

The soil is coarse-grained (sand or gravel) if the percentage fines is 50% or less. Otherwise, the soil is fine-grained (silt or clay) – the classification is not based on plasticity.

Coarse-grained soils are classified further as either sand or gravel using the results of particle-size distribution tests.

Fine-grained soils are classified further as silt or clay on the basis of the liquid limit and plasticity index (from Atterberg limits tests).

The soil is an organic soil if it contains sufficient quantities of dispersed organic matter that it has an influence on the liquid limits of the fines component after oven-drying, as outlined in the BS Section. The definition of peat is similar to that in BS 5930 and it is generally classified according to the degree of decomposition and strength. When encountered, reference should be made to the classification given in ASTM D 4427.

SOIL GROUP NAME AND SYMBOL (ASTM)

Coarse-Grained Soils

For coarse-grained soils, the dominant soil fraction is sand if the dry weight of the sand fraction, i.e. particle sizes from 0.075 mm to 4.75 mm, exceeds that of the gravel fraction, i.e. particles ranging from 4.75 mm to 75 mm, and vice versa for gravel.

Coarse-grained soils with ≤12% fines are also described as well-graded or poorly-graded based on the particle-size distribution curve, using the coefficient of uniformity (C_U) and, to a lesser extent, the coefficient of curvature (C_C) as follows:
– Sands are well-graded when $C_U \geq 6$ and C_C is between 1 and 3.
– Sands are poorly-graded for other values of C_U and C_C.
– Gravels are well-graded when $C_U \geq 4$ and C_C is between 1 and 3.
– Gravels are poorly-graded for other values of C_U and C_C.

For coarse-grained soils with fines contents >12%, these terms are not used.

Sands and gravels are also sub-divided into coarse, medium, and fine, as defined in Table 11.

TABLE 11 - SIZE FRACTION DESCRIPTIONS FOR COARSE-GRAINED SOILS

Soil	Particle diameter range [mm]		
	Coarse	Medium	Fine
Gravel	75 to 19	-	19 to 4.75
Sand	4.75 to 2.0	2.0 to 0.425	0.425 to 0.075

The predominant size fractions present are identified, and the absence of size range descriptors means that fine, medium, and coarse fractions are all present in roughly equal proportions.

Fine-Grained Soils

Fine-grained soils are classified as clay or silt according to the results of Atterberg limits tests. A soil is inorganic clay if: $I_P \geq 6$ and $I_P \geq 0.73(w_L-20)$

where:
I_P = plasticity index [%]
w_L = liquid limit [%]

The A-line and U-line in a plasticity chart are as described in the BS section.

Clays with liquid limit $w_L < 50$ and plasticity index $I_P > 7$ are further classified as lean clay, and given the group symbol “CL”. Clays with liquid limits $w_L \geq 50$ are further classified as fat clay, and are given the group symbol “CH”.

A soil is classified as a silt when it plots below the A-line or the plasticity index $I_P < 4$. Silts with liquid limit $w_L < 50$ are given the group symbol “ML”. Silts with liquid limits $w_L \geq 50$ are further classified as elastic silt, and are given the group symbol “MH”.

Soils are classified as silty clay where the liquid limit versus plasticity index plots on or above the A-line but where the plasticity index falls within the range $4 \leq I_P \leq 7$, i.e. the hatched zone in the lower left-hand corner of the plasticity chart. Silty clays are given the Group Symbol “CL-ML”.

Organic Soils

For both clay and silt, or the fines component of a coarse-grained soil, the additional term organic applies if the ratio of the liquid limit of a sample (or the fines portion of the sample) after oven drying at 105° C to the liquid limit without oven drying is less than 0.75.

Organic soils are classified in a manner similar to that for inorganic soils for plots of the liquid limit (not oven dried) versus plasticity index with respect to the A-line. Organic clays and silts with liquid limit $w_L < 50$ are given the same group symbol “OL”. Organic clays and silts with liquid limits $w_L \geq 50$ are given the group symbol “OH”.

Coarse-grained soils containing fine organic material are described using the term “with organic fines”.

SECONDARY SOIL TYPE (ASTM)

Secondary soil type descriptions follow the ranges given in Table 12. No other terms are used, though combinations of these terms are.

TABLE 12 - DESCRIPTIVE TERMS AND RANGES FOR SECONDARY CONSTITUENTS

Term	Principal soil type	Term	Approximate proportion of secondary constituent	
			Coarse soil	Fine soil
	SAND and/or GRAVEL(1)			< 5%
	SAND and/or GRAVEL(1)	with clay or silt		5% to 12%
Clayey or Silty	SAND and/or GRAVEL(1)			> 12%
	SAND and/or GRAVEL(1)		<15% gravel or sand	
	SAND and/or GRAVEL(1)	with gravel or sand	≥15% gravel or sand	
	SILT or CLAY		< 15%	
	SILT or CLAY	with sand or gravel(1)	15% to 29%	
Sandy and/or gravelly(1)	SILT or CLAY		≥30%	

Note: (1) choice depends on which has higher percentage.

PARTICLE SHAPE (ASTM)

The description of particle shape includes references to form, angularity, and surface texture. These terms are normally used only for gravels, cobbles, and boulders, though in some cases for coarse sands.

The form (or shape) of coarse particles is described as flat, elongated, or both.

Flat: Width/Thickness > 3
Elongated: Length/Width > 3
Flat and elongated meets both criteria. These terms are not used if the criteria are not strictly met.

Angularity terms are usually only applied to particles gravel-size and larger (Table 13, from ASTM D 2488).

TABLE 13 - ANGULARITY OF COARSE-GRAINED PARTICLES

Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

The surface texture of coarse particles are described as rough or smooth.

COLOUR (ASTM)

As noted for BS 5930 (BS section), soil colours are described using the Munsell Soil Colour Charts (Gretag-Macbeth, 2000).

SOIL ODOUR (ASTM)

The same descriptive terms suggested for BS 5930 (BS Section) are used with the ASTM Standards. It must be emphasised that soil odour descriptions are unlikely to be fully consistent, because of factors such as variations in sample handling, ambient conditions at time of sample description, and strong dependence on a person's ability to detect and identify odour.

STRENGTH OF COHESIVE SOILS (ASTM)

Descriptions of cohesive soil strength are not part of the ASTM classification system; however soil strength is incorporated whenever available from laboratory or in situ test results and interpretation. The boundaries for undrained shear strength ranges in current use in North American practice are given in Table 14. These boundaries are lower than those used with BS 5930.

TABLE 14 - UNDRAINED SHEAR STRENGTH SCALE FOR COHESIVE SOILS (1)

Term	Undrained shear strength	
	[kPa]	[ksf] (2)
Very soft	Less than 12.5	Less than 0.25
Soft	12.5 to 25	0.25 to 0.50
Firm	25 to 50	0.50 to 1.0
Stiff	50 to 100	1.0 to 2.0
Very stiff	100 to 200	2.0 to 4.0
Hard	200 to 400	4.0 to 8.0
Very hard (3)	Greater than 400	Greater than 8.0

Notes: 1) from Terzaghi and Peck (1967)
2) ksf used primarily for US projects
3) the upper boundary for "Hard", and the "Very hard" range have been added

DENSITY/COMPACTNESS OF GRANULAR SOILS (ASTM)

Tables of recommended values and descriptors for relative density are not provided in the ASTM Standards, but in practice relative density is often interpreted on the basis of cone penetration test results. The same ranges of relative density (compactness) as those recommended for use with BS 5930 (see BS Section) are used.

DISCONTINUITIES/STRUCTURE (ASTM)

Criteria for describing soil structure are provided in ASTM D 2488, and in Table 15 along with additional terms in use in the geotechnical industry.

TABLE 15 - DESCRIPTIVE TERMS FOR SOIL STRUCTURE

Term	Description
Slickensided	Fracture or shear planes (or planes of weakness) that appears slick and glossy.
Fissured	Cohesive soil that breaks into blocks along unpolished planes (discontinuities), often filled with a different material. The fill material is noted.
Blocky	Cohesive soil that breaks into small angular lumps along polished planes (discontinuities) which resist further breakdown.
Gassy	Soil has a porous nature and there is evidence of gas, such as blisters.
Expansive	Visibly expands after sampling. Degree of expansion is estimated and noted.
Platy	A stratified appearance when the soil can be broken into thin horizontal plates.
Cemented	Material grains bound together forming an intact mass.

The distance between the fissures, shear planes and expansion cracks is noted using the terms in Table 8.

BEDDING/STRATIGRAPHY (ASTM)

The terminology for bedding thickness and stratigraphic description used in North American offshore practice is more detailed than outlined in ASTM D 2488, and is different from BS 5930. In Table 16, the descriptive terms have been further defined and integrated with BS 5930 terminology.

TABLE 16 - DESCRIPTIVE TERMS FOR BEDDING THICKNESS AND INCLUSIONS

Term	Bedding thickness	
	[mm]	[inch]
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample	
Parting	< 3	1/8
Lamina	3 to < 6	1/8 to < 0.25
Laminated(1)	Alternating partings or laminae of different soil types in equal proportion	
Lens	6 to < 20	0.25 to < 0.75
Seam	20 to < 76	0.75 to < 3
Layer	Greater than 76	Greater than 3
Stratified(2)	Alternating lenses, seams or layers of different soil types in equal proportion	
Intermixed	Soil sample composed of pockets of different soil types, and laminated or stratified structure is not evident	

Notes: (1) Equivalent to “Interlaminated” term used in BS 5930:1999
(2) Equivalent to “Interbedded” term used in BS 5930:1999.

MINOR CONSTITUENTS (ASTM)

Minor constituents within a soil, such as shell or wood fragments, or small quantities of soil particles (not secondary soil types), are typically more relevant to the site geology or to laboratory testing procedures than to soil behaviour. Since the terms and percentages are not defined in either BS 5930 or ASTM D 2487/8, the terms “with trace”, “with few”, “with”, “with many” are used as a guide.

WRITTEN SOIL DESCRIPTIONS

Although soils are classified in the order of the characteristics described in the preceding sections, written descriptions are given in a different order in both Standards. To bring as much consistency as possible to the soil descriptions, Fugro selected a single preferred order of terms, which most closely resembled the majority of the descriptions used in Fugro offices around the world.

In this description, the principal soil type is given last as the soil name, with most other terms written as adjectives. The principal soil type is given in upper-case.

The preferred order of terms for a soil description are:
1. Density/compactness/strength.
2. Discontinuities.
3. Bedding.
4. Colour.
5. Secondary (composite) soil types.
6. Particle shape.
7. Particle size.
8. PRINCIPAL SOIL TYPE.

with:
9. Minor constituents (can be inserted in front of the principal soil type, such as “shelly”).
10. Soil odour.

For example: Firm closely-fissured dark olive grey sandy calcareous CLAY with few silt pockets. Where used, the Group Symbol is part of the soil description, e.g. loose poorly-graded fine to medium SAND with silt (SP-SM).

PARTICULATE DEPOSITS

The geological origin of a single particle type allows the following descriptions (optional):

Clastic: sediment transported and deposited as grains of inorganic origin. Typical clastic particles are:
- quartz grains: clear or milky white and ranging from very angular to very rounded; commonly a frosted surface for wind-blown grains
- feldspar grains: varying in colour from milky white to light yellowish brown
- mica flakes: varying in colour from gold-coloured to dark brown
- dark mineral grains: usually of igneous or metamorphic origin with undetermined mineralogy
- silicate grains: undetermined mineralogy
- rock fragments: including fragments of carbonate rock
- debris: deposit of rock fragments of a variety of particle sizes which may include sand and finer fractions; typical examples are rock debris and coral debris.

Organic: remains of plants and animals that consists mainly of carbon compounds

Bioclastic: sediment transported and deposited as grains of organic origin. Examples of bioclastic particles are:
- Calcareous algae: crustal or nodular growths or erect and branching forms produced by lime-secreting algae; microstructures include layered, rectangular structures and internal fine tube-like structures.
- Foraminifera: hard sediment test (external skeleton) consisting of calcite or aragonite and produced by unicellular organisms; commonly less than 1 mm in diameter, multi-chambered and intact.
- Sponge spicules: spicules of siliceous sponges in a variety of rayed shapes; dimensions ranging from less than 1 mm to over 1 cm in length but usually less than 1 mm in width.
- Corals: commonly consisting of small fibres set perpendicular to the walls and septal surfaces; mainly aragonite composition for relatively recent forms; conversion of aragonite to calcite for earlier corals, usually with consequent loss of original structural details.
- Echinoids: hard part of echinoids consisting of a plate or skeletal element forming a single crystal of calcite; five-rayed internal symmetry for spines of echinoids; typical widths ranging from several mm to a few cm.
- Bryozoans: chambered cell-like structures that are considerably coarser than those of calcareous algae; either aragonite or calcite composition; possible cell in-fill consisting of clear calcite and/or micrite.
- Bivalves and Gastropods: Mollusk shells, chiefly of aragonite composition; inner layer of aragonite protected by an outer layer of calcite for some bivalve shells and gastropods.

Oolitic: sediment consisting of solid, round or oval, highly polished and smooth coated grains, which may or may not have a nucleus. The coating consists of chemically precipitated aragonite, possibly converted to calcite. Ooliths have concentric structures and may also have radial structures. The grains are generally less than 2 mm diameter.

Pelletal: sediment consisting of well-rounded grains of ellipsoidal shape and no specific internal structure. The composition is clay to silt-sized carbonate material, which is probably the excretion product of sediment eating organisms. Pellets may have an oolitic crust. The grains are generally less than 2 mm diameter.

STRUCTURE OF NON PARTICULATE DEPOSITS

Reef: soil or rock formed by in situ accumulation or build-up of carbonate material by colonial organisms such as polyps (coral), algae (algal mats or balls) and sponges.

Orthochemical: orthochemical components precipitated during or after deposition. These components can include: (1) pyrite spherulites and grains, (2) crystal euhedra of anhydride or gypsum, (3) replacement patches and nodular masses of anhydrite and gypsum. Single grains are rare.

GEOLOGICAL INFORMATION

Specific geological terms can assist the geotechnical soil description by providing information on stratigraphy, origin (genesis) or regional significance (optional). Examples are:

- time stratigraphy, such as Eemian and Pleistocene
- lithostratigraphy, such as Yarmouth Roads Formation
- depositional environment, such as Marine, Glacio-lacustrine and Residual Soil
- regional significance, such as Chalk and Mud.

REFERENCES

ASTM International, 2011. *ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes Unified Soil Classification System).* West Conshohocken: ASTM International.

ASTM International, 2009. *ASTM D2488-09a Standard Practice for Description and Identification of Soils Visual-Manual Procedure).* West Conshohocken: ASTM International.

ASTM International, 2007. *ASTM D4427-07 Standard Classification of Peat Samples by Laboratory Testing.* West Conshohocken: ASTM International.

British Standards Institution, 1999. *BS 5930:1999 Code of practice for ground investigations*. London: BSI.

Casagrande, A. 1947. Classification and Identification of Soils*. Proceedings of the American Society of Civil Engineers,* Vol. 73, No. 6, pp. 783-810.

Clark, A.R. and Walker, B.F. 1977. A Proposed Scheme for the Classification and Nomenclature for Use in the Engineering Description of Middle Eastern Sedimentary Rocks. *Géotechnique*, Vol. 27, No. 1, pp. 94 99.

Gretag-Macbeth, 2000. *Munsell Soil Color Charts.* Year 2000 revised washable ed., New Windsor: Gretag-Macbeth.

International Organization for Standardization, 2002. *ISO 14688-1:2002 Geotechnical Investigation and Testing - Identification and Classification of Soil - Part 1: Identification and Description*. Geneva: ISO.

International Organization for Standardization, 2004. *ISO 14688-2:2004 Geotechnical Investigation and Testing - Identification and Classification of Soil - Part 2: Principles for a Classification.* Geneva: ISO.

International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

Landva, J., Remijn, M. and Peuchen, J. 2007. Note on Geotechnical Soil Description. In *Offshore Site Investigation and Geotechnics: Confronting New Challenges and Sharing Knowledge: Proceedings of the 6th International Conference, 11–13 September 2007, London, UK,* London: Society for Underwater Technology, pp. 505-514.

Peuchen, J., De Ruijter, M. and Goedemoed, S. 1999. Commercial Characterisation of Calcareous Soils. In Al-Shafei, K.A. Ed. *Engineering for Calcareous Sediments: Proceedings of the Second International Conference on Engineering for Calcareous Sediments, Bahrain, 21-24 February 1999, Vol. 1,* Rotterdam: Balkema, pp. 113-121.

ISSUE 26

TESTING PRACTICE

GENERAL

This document summarises geotechnical laboratory test methods for soil.

Fugro strives to arrange testing in registered laboratories with formal accreditation. This document summarises test methods used by Fugro geotechnical laboratories in the Netherlands. Test methods used by on-site laboratories and other office laboratories are often identical or generally equivalent.

Laboratory tests are carried out in general accordance with standards published by ASTM International (ASTM), British Standards Institution (BSI) and International Organization for Standardization (ISO). Note that ISO (2014) refers mainly to test procedures published in other documents, with some additional recommendations. In-house test procedures adopted for some tests are generally based on published recommendations for which no standards are available. Detailed work instructions and calibration details are available for inspection at the laboratory.

Some of the laboratory tests allow various optional procedures. These procedures are not applicable, unless specifically agreed.

Soil parameter values can vary with temperature. Tests are generally conducted at laboratory temperatures of around 20^{o}C. Any tests conducted at specific temperatures and any corrections for temperature are explicitly reported.

Laboratory test results show depth defined as vertical distance between ground surface or seafloor and top of the laboratory test specimen, unless indicated otherwise.

SAMPLE REQUIREMENTS

The feasibility of a particular laboratory test relates to the sampling practice and sample handling for a particular soil and depends on factors such as soil type, available amount of sample material and sample quality. Usually, a reasonable estimate of test feasibility is possible at the time of sampling. A further refinement is possible in the laboratory prior to testing and, in some cases, only after testing. The limitations of feasibility estimates may lead to rejection of samples for testing upon inspection in the laboratory or may result in appropriate comments on test results after completion of testing.

The adopted classification system for sample quality is according to BSI (2015) and ISO (2006, 2014). The classification system recognises 5 classes on the basis of feasibility of specific geotechnical identification and laboratory tests. A summary of these classes is as follows:
Class 1: undisturbed: strength, stiffness and consolidation
Class 2: undisturbed: permeability, unit weight, boundaries of strata - fine
Class 3: disturbed: water content
Class 4: disturbed: particle size analysis, Atterberg limits, boundaries of strata - broad
Class 5: disturbed: sequence of layers
The higher class includes the laboratory tests of the lower class.

An indication of intact (undisturbed) sample quality may be obtained from re-compression of a test specimen, for example in an oedometer or triaxial cell. Table 1 presents a method recommended by ISO (2014) based on $\Delta e/e_0$. Here, Δe represents the change in void ratio Δe from an initial laboratory value (e_0) at atmospheric conditions to the specimen void ratio upon re-compression to in situ stress conditions.

TABLE 1 - INTACT SAMPLE QUALITY - $\Delta e/e_0$

Overconsolidation Ratio	$\boldsymbol{\Delta e/e_0}$			
	1 (very good to excellent)	**2** (good to fair)	**3** (poor)	**4** (very poor)
1 to 2	< 0.04	0.04 to 0.07	0.07 to 0.14	> 0.14
2 to 4	< 0.03	0.03 to 0.05	0.05 to 0.10	> 0.10

The presented criteria are based on tests on marine clays in the depth range 4 m to 25 m, with plasticity index in the range 6% to 43%, water content 20% to 67% and overconsolidation ratios of 1 to 4. The criteria must be used with caution for soils outside this range.

Table 2 presents an alternative indication of intact (undisturbed) sample quality according to Terzaghi et al. (1996). Here, volumetric strain is derived from an initial laboratory specimen volume and the specimen volume upon re-compression to in situ stress conditions. The criteria apply to clays with an overconsolidation ratio of less than about 3 to 5. Parameters such as effective preconsolidation pressure σ'_p and undrained shear strength c_u preferably require laboratory specimen with SQD equal to B or better (DeGroot et al., 2005).

TABLE 2 – INTACT SAMPLE QUALITY - SDQ

Volumetric Strain ε_v [%]	**SDQ**
< 1	A
1 to 2	B
2 to 4	C
4 to 8	D
> 8	E
Note: SDQ: Sample Quality Designation	

The $\Delta e/e_0$ and ε_v criteria represent a simplification, as they ignore important soil changes during the process of sampling and sample handling up to specimen preparation in a geotechnical laboratory. This simplification avoids interpretation anomalies related to uncertainties in laboratory values for soil unit weights, water contents and density of solid particles.

The $\Delta e/e_0$ and ε_v criteria assume no-gas within the pore water. Gas can cause an increase in void ratio when recovering samples to surface. The result is a correspondingly larger change in void ratio when returning a specimen back to the estimated effective stress conditions in situ. In such case, it is likely that the undrained shear strength would be less affected than soil stiffness, as the void ratio in situ has been partially restored. Changes in soil fabric remain.

Values for $\Delta e/e_0$ and ε_v should exclude secondary consolidation. In practice, no correction for secondary consolidation will be applied. This practice underestimates undisturbed sample quality, particularly for incremental loading oedometer tests with 24 hour load increments and longer.

GEOTECHNICAL INDEX TESTING

WATER CONTENT

The water content is determined by drying selected moist/wet soil material for at least 18 hours to a constant mass in a 110°C drying oven. The difference in mass before and after drying is used as the mass of the water in the test material. The mass of material remaining after drying is used as the mass of the solid particles. The ratio of the mass of water to the measured mass of solid particles is the water content of the material. This ratio can exceed 1 (or 100%).

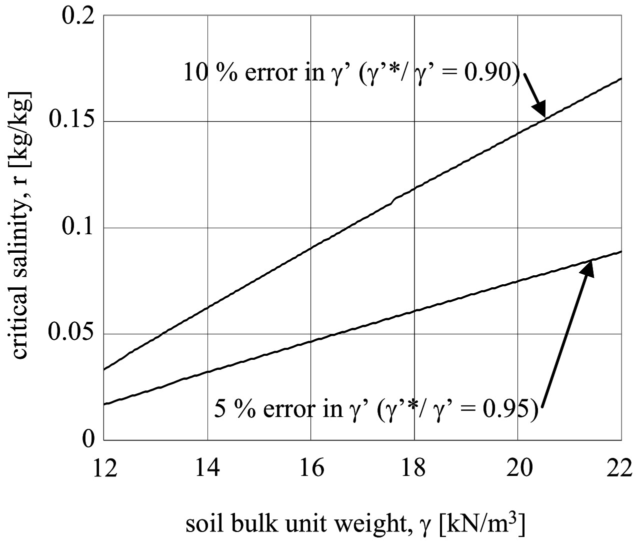
Test references: ASTM D2216-10, BS 1377: Part 2: 1990, ISO/TS 17892-1:2014, ISO 19901-8:2014

UNIT WEIGHT – VOLUME-MASS CALCULATION

Measurement of volume and mass of a soil sample allows calculation of unit weight. For fine-grained (cohesive) soils, a soil specimen is generally obtained from a standard steel cylinder with cutting edge, which is pushed manually into the extruded soil sample. Preference is given to a 100 ml cylinder (area ratio of 12%), but a volume of 33.3 ml (area ratio of 21%) may be used when insufficient homogeneous sample is available. If possible, a specimen of coarse-grained (non-cohesive) soil is obtained by selecting a part of a cylindrical soil sample, trimming the end surfaces, and measuring height and diameter. This method also applies to fine-grained specimens selected for strength and/or stiffness (e.g. triaxial and oedometer) tests.

Unit weight γ (kN/m^3) refers to unit weight of the soil specimen at the water content at the time of test.

The method excludes correction for pore water salinity r (contains dissolved solids), in situ pressure and temperature. The diagram below provides an indication of error in calculated submerged unit weight γ' versus submerged unit weight corrected for salinity, γ'^* (Kay et al., 2005). Typical seawater salinity is 35 g salt per kg seawater (r = 0.035). Correction for salinity is optional.

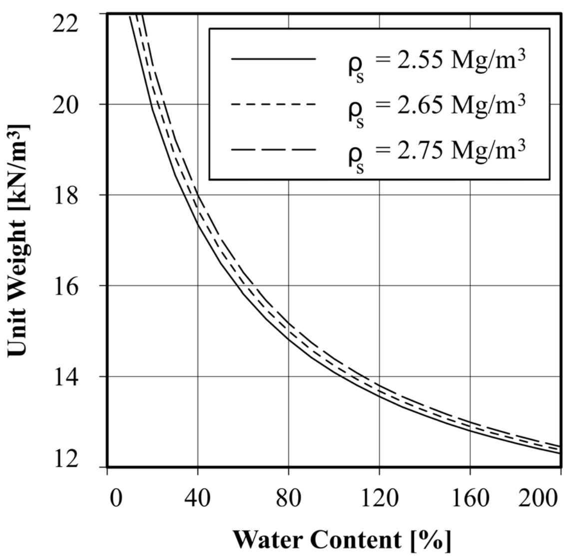


Optionally, dry unit weight γ_d, is calculated from the mass of oven-dried soil and the initial specimen volume.

Test references: BS 1377: Part 2: 1990, ASTM D7263-09, ISO 19901-8:2014

UNIT WEIGHT – DERIVED FROM WATER CONTENT

Water content (w) measurement allows estimation of soil unit weight (γ) on fully saturated samples. This practice requires input on density of solid particles (ρ_s) and presumes saturation of non-saline pore water.



Correction for (high) pore water salinity (contains dissolved solids) is optional.

Test reference: In-house

DENSITY OF SOLID PARTICLES – CONVENTIONAL PYCNOMETER

The density of the solid particles of an oven-dried soil sample is determined by means of a stoppered-bottle pycnometer, using distilled water. The method is considered applicable to solid particles that are not soluble in water. For soils with a high organic content, a different liquid may be selected. Soils with high pore water salinity (contain dissolved solids) require use of a gas pycnometer. This is optional.

Test references: BS 1377: Part 2: 1990, ASTM D854-14, ISO 19901-8:2014

GRAIN SHAPE

Grain shape is determined by microscopic comparison of both grain roundness and sphericity with standard grain shapes. The standard shapes are presented together with the test results.

Test reference: In-house

PARTICLE SIZE ANALYSIS

Particle size analysis can be performed by means of sieving and/or hydrometer readings. Sieving is carried out for particles that would be retained on a 0.063 mm (ISO and BS) or 0.075 mm (ASTM) sieve, while additional hydrometer readings may be carried out when a significant fraction of the material passes a 0.063 mm (0.075 mm) sieve.

In a sieve analysis, the mass of soil retained on each sieve is determined, and expressed as a percentage of the total mass of the sample. Prior to sieving, samples are treated with a dispersing agent (sodium hexameta-phosphate), rinsed on a 0.063 mm (0.075 mm) sieve and dried.

The hydrometer method allows measurement of the density of a suspension consisting of fine-grained soil particles and distilled water, to which a dispersion agent is added. This suspension is mixed using a high speed stirrer. Testing is performed in a thermostatically controlled water bath (25° ± 0.5°). The particle size is calculated according to Stokes' Law for a single sphere, on the basis that particles of a particular diameter were at the surface of the suspension at the beginning of sedimentation and had settled to the level at which the hydrometer is measuring the density of the suspension. These calculations require a value for the density of solid particles. Generally, a value of 2.65 t/m^3 is assumed. When other values are used, this is included in the laboratory report. The hydrometer results for selected particle sizes are presented as a percentage of the total mass of the soil sample.

Particle size is presented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot. The general slope of the distribution curve may be described by the coefficient of uniformity C_u, where $C_u = D_{60}/D_{10}$, and the coefficient of curvature C_c, where $C_c = (D_{30})^2/D_{10} \times D_{60}$. D_{60}, D_{30}, and D_{10} are effective particle sizes indicating that 60%, 30%, and 10% respectively of the particles (by weight) are smaller than the given effective size.

Combined presentation of results from hydrometer readings and sieving normally requires data harmonising in the area of overlap, i.e. near the 0.06 mm particle size.

Test references: ISO/TS 17892-4:2004, BS 1377: Part 2: 1990, ASTM D422-63 (2007)e2 (withdrawn, no replacement), ASTM D6913-04(2009)e1, ISO 19901-8:2014

PERCENTAGE FINES

The Percentage Fines test identifies the proportions of fine grained (< 0.06 mm for BS/ISO and < 0.075 mm for ASTM) and coarse-grained (> 0.06 mm) particle sizes of a soil sample by wet sieving through a 0.063 mm (0.075 mm) sieve. Prior to sieving, the sample is treated with a dispersing agent. The Percentage Fines is defined as the ratio of dry mass of soil passing the 0.063 mm (0.075 mm) sieve to the dry mass of the total soil sample, expressed as a percentage.

Test references: ISO/TS 17892-4:2004, BS 1377: Part 2: 1990, ASTM D6913-04(2009)e1, ISO 19901-8:2014

ATTERBERG LIMITS

Atterberg limits are determined on soil specimens with a particle size of less than 0.425 mm. If necessary, coarser material is removed by dry sieving. The Atterberg limits refer to arbitrarily defined boundaries between the liquid and plastic states (Liquid Limit, w_L), and between the plastic and brittle states (Plastic Limit, w_P) of fine grained soils. They are expressed as water content, in percent.

The liquid limit is defined as the water content at which a part of soil is placed in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove, when the cup is subjected to 25 standard shocks. The one-point liquid limit test is usually carried out. Distilled water may be added during soil mixing to achieve the required consistency.

The plastic limit is defined as the water content at which a soil can no longer be deformed by rolling into 3 mm diameter threads without crumbling.

The range of water contents over which a soil behaves plastically is the Plasticity Index, I_P. This is the difference between the liquid limit and the plastic limit (w_L-w_P).

Test references: BS 1377: Part 2: 1990, ASTM D4318-10e1, ISO 19901-8:2014

MINIMUM INDEX UNIT WEIGHT

The minimum index unit weight (γ_{dmin}) of cohesionless soil is determined from the mass of oven-dry material that is deposited by slowly withdrawing a soil-filled funnel from a standard mould of either 70 ml or 550 ml volume.

Test reference: In-house

MAXIMUM INDEX UNIT WEIGHT - IMPACT COMPACTION

The maximum index unit weight (γ_{dmax}) of cohesionless soil is determined from the mass of oven-dry, compacted soil in a standard mould. The soil is compacted in 5 layers, with each layer being subjected to respectively 5, 10, 20, 40 and 80 blows from a standard, hand-held hammer.

Equipment dimensions are as follows. Preference is given to the large mould, but application depends on size of sample.

		70.5 ml mould	**554 ml mould**
Hammer mass	[g]	185	750
Drop height	[mm]	300	390
Cross-sectional area	[mm^2]	1006	38,500

Reference: In-house, DGI Product Sheet #000 96-07-02

MAXIMUM INDEX UNIT WEIGHT – VIBRATING HAMMER

The maximum index unit weight (γ'_{dmax}) is obtained by compacting soil that has been passed through a 4 mm sieve into a mould at a range of water contents. The first sample is thoroughly mixed with water, to produce a soil with a 4% water content, and then compacted in three equal layers using a vibrating hammer for a period of 10 seconds per layer. The top section of the mould is removed and the sample levelled in the bottom section of the mould. The unit weight of the sample is calculated and a representative portion of soil is removed for water content determination.

The test is repeated at four further water contents. By determining the dry unit weight achieved at each water content, a maximum dry unit weight may be estimated. There is evidence of breakdown of crushable particles during a test.

ISSUE 43

Equipment dimensions are as follows:

- Volume of mould: 96.4 ml
- Hammer: Milwaukee heavy duty 545S
  - 1300 W nominal / 650 W release
  - rotation/min: 300
  - hammer force: 8.5 J
  - mass: 6.7 kg

Test reference: In-house

GEOCHEMICAL TESTING

ORGANIC MATTER CONTENT – DICHROMATE OXIDATION METHOD

An oven-dried (50°C) soil sample is mixed with potassium dichromate solution and left for 30 min to allow the oxidation of organic matter to proceed. The solution is titrated with a ferrous sulphate solution (to determine the amount of excess potassium dichromate). The organic matter content is defined as the ratio of the total volume of potassium dichromate solution used to oxidize the organic matter in the soil sample to the mass of the initial dried soil sample (Walkley and Black's method). It is expressed as a percentage.

Note: soils containing sulphides or chlorides have been found to yield inaccurate (too high) organic matter content measurements using this procedure.

Test references: BS 1377: Part 3: 1990:3

ORGANIC MATTER CONTENT – LOSS ON IGNITION

An oven-dried (105°C) soil sample is heated to 550°C for 2 hours. The mass is measured before and after heating. The organic matter content is defined as the ratio of the mass loss due to heating to the original mass of the dried soil sample, and is expressed as a percentage.

Note: the mass loss on ignition test is reliable for (1) sandy soils that contain little or no clay and no carbonate and (2) peats and organic clays containing more than 10% organic matter.

Test references: BS 1377: Part 3: 1990:4, ASTM D2974-14, NEN 5754, ISO 19901-8:2014

CARBONATE CONTENT – GAS VOLUME

The carbonate content is determined by drying selected soil material to a constant mass in a 110°C drying oven, and measuring the volume of dissipated carbon dioxide (CO_2) upon reaction of the soil with hydrochloric acid (HCl). The carbonate content is calculated from calibration values, and expressed as a percentage of dry mass of the original soil.

Test reference: ISO 10693:2004, ISO 19901-8:2014

CARBONATE CONTENT - GAS PRESSURE

The carbonate content is determined by using a dried or a natural soil specimen and measuring the pressure of dissipated carbon dioxide (CO_2) upon reaction of the soil with hydrochloric acid (HCl). The carbonate content is calculated from the mass of the specimen and the pressure increase after reaction by comparison with calibration values. For a natural soil, a correction factor is applied to correct for water content. Carbonate content is expressed as a percentage of dry mass of the original soil.

Test reference: ASTM D4373-14, ISO 19901-8:2014

WATER-SOLUBLE SULPHATE CONTENT – GRAVIMETRIC METHOD

The water-soluble sulphate content of a soil sample is determined on a test portion that has been sieved and crushed through a 2 mm sieve and oven dried to 110°C. The test portion is mixed with distilled water to prepare a 2:1 water:soil extract.

In the gravimetric method, barium chloride solution is added to the water:soil extract and the precipitated barium sulphate is collected, dried and weighted. The sulphate content is then calculated from the mass of the material used in the analysis and the mass of the barium sulphate precipitated. BS presents the results in SO_3 [g/l] and AASTHO in SO_4 [mg/kg].

If a 2:1 water:soil extract is prepared, one can convert sulphites (SO_3) into sulphates (SO_4) by multiplying SO_3 by a factor 1.2. For extractions other than a 2:1 the multiplying factor is different.

Test reference: BS 1377: Part 3:1990, AASHTO T290-95-UL (2007)

WATER-SOLUBLE CHLORIDE CONTENT – MOHR'S METHOD

The water-soluble chloride content of a soil sample is determined on a test portion that has been sieved and crushed through a 2 mm sieve and oven dried to 110°C. The test portion is mixed with distilled water to prepare a 2:1 water:soil extract.

In the Mohr's method chloride ion will precipitate with silver nitrate. The chloride reacts with the silver ion before any silver chromate forms, due to the lower solubility of silver nitrate. The potassium chromate indicator reacts with excess silver ion to form a red silver chromate precipitate. The end point is the appearance of the first permanent orange colour. The chloride content is expressed as a percentage by mass of dry soil.

This test method is suitable for analysing solutions with a pH between 6.0 and 8.5.

Test reference: BS 1377: Part 3: 1990, AASHTO T291-94-UL (2008)

PERMEABILITY TESTING

CONSTANT HEAD PERMEABILITY: TRIAXIAL CELL

The effect of stress level on the coefficient of permeability may be estimated from constant head tests in a triaxial cell – flexible wall permeameter. The specimen is compacted in a split mould by tamping of thin layers of moist soil to the required initial density, and subsequently mounted in the triaxial cell. Filter screens or porous disks are placed at both ends of the specimen. The required stress level is applied and saturation is achieved by flushing with CO_2 gas followed by controlled flow of de-aired water and the application of backpressure. The degree of saturation is checked by the pore water pressure response to small variations in cell pressure. A hydraulic gradient is applied and the rate of flow is recorded for various time steps. The permeability is calculated in accordance with Darcy's equation for laminar flow.

References: BS 1377: Part 6: 1990, ASTM D5084-10, ISO 19901-8:2014

COMPRESSIBILITY TESTING

OEDOMETER - INCREMENTAL LOADING

The oedometer test covers determination of the rate and magnitude of consolidation of a laterally restrained soil specimen, which is axially loaded in increments of constant stress until the excess pore water pressures have dissipated for each increment. Normally, each load increment is maintained for 24 hours.

The test is generally carried out on undisturbed (intact) cohesive specimens using a consolidometer (oedometer) apparatus, which is placed in a thermostatically controlled room (10°C). Selection of mounting method depends on soil characteristics. Soils that show a tendency to swell, such as peat or overconsolidated clays, are mounted dry. Moist sponges are placed in the oedometer cell to retain sample moisture conditions. Other samples are usually mounted using the wet mounting method. Distilled water is added to the cell when loads are applied to the loading arm. When required, the initial load is increased to prevent swell.

Key parameters that can be obtained from this test are the preconsolidation pressure σ'_p and the coefficient of consolidation c_v. The preconsolidation pressure is estimated using the graphical Casagrande construction. The root time method or the log time method is used for determination of c_v. Other parameters that may be derived from this test are the compression index C_c, the coefficient of volume compressibility m_v and the vertical permeability k_v.

Test references: ASTM D2435/D2435M -11, BS 1377: Part 5: 1990, ISO 19901-8:2014

OEDOMETER - CONSTANT RATE OF STRAIN

The Constant Rate of Strain (CRS) oedometer test covers determination of the rate and magnitude of consolidation of a laterally restrained soil specimen when it is drained axially and subjected to controlled deformation loading. The rate of deformation is selected so that excess pore water pressures are between 3% and 20% of the applied axial stress. Drainage of pore water is permitted from the top of the specimen and pore water pressures are measured at the bottom of the specimen. The test is generally carried out on undisturbed (intact) cohesive specimens using a consolidometer, in a thermostatically controlled room (20°C).

Key parameters that can be obtained from this test are the preconsolidation pressure σ'_p and the coefficient of consolidation c_v as a function of axial stress. The preconsolidation pressure is estimated using the graphical Casagrande procedure, while the coefficient of consolidation is determined analytically from the measurements of axial stress, strain and excess pore water pressure. Other parameters that may be derived from this test are the compression index C_c, the coefficient of volume compressibility m_v and the coefficient of vertical permeability k_v.

Test reference: ASTM D4186/D4186M-12, ISO 19901-8:2014

STRENGTH INDEX TESTING

TORVANE AND POCKET PENETROMETER

The torvane and pocket penetrometer are small hand-held instruments for rapid strength index testing of fine grained (cohesive) soils. The torvane test is carried out by pressing a standard vane into the soil and measuring the minimum torque required to rotate the vane. The vane size can be selected to suit the expected torque up to an equivalent undrained shear strength of the soil of 250 kPa. The undrained shear strength is correlated to the measured torque by vane size and torvane spring constant.

The pocket penetrometer test consists of pressing a small solid cylinder into the soil, to a specified penetration. The maximum force required for penetration is correlated to the undrained shear strength. The size of the cylinder can be selected so that undrained shear strength readings of up to 900 kPa can be taken.

Test reference: ISO 22475-1:2006, ISO 19901-8:2014

FALL CONE

The fall cone is a rapid index test for determining undrained shear strength of undisturbed or remoulded specimens of cohesive soil. The test consists of suspending a standard cone of a specified mass and apex angle vertically over and just touching the surface of the specimen. Subsequently, the cone is released and penetrates into the sample under its self-weight. The depth of penetration for the selected cone is correlated to the undrained shear strength of the soil. Several correlations exist. The cone size and shape can be selected to suit the expected undrained shear strength of the specimen.

Reference: ISO/TS 17892-6, ISO 19901-8:2014

HAND VANE

The hand vane allows index testing for undrained shear strength of cohesive soil. The tool is similar to the laboratory miniature vane except for reduced control: manual penetration and rotation of the vane.

Several different measurements of undrained shear strength are possible:

a) Intact: undisturbed undrained shear strength as measured on an intact specimen.
b) Intact–residual: measured post-peak during initial shearing of an intact specimen.
c) Intact–vane-remoulded: measured after multiple rotations of the hand vane after completion of the intact test.
d) Hand-remoulded: steady state (post-peak if exists) resistance of a hand-remoulded test specimen.
e) Hand-remoulded–cane-remoulded: steady state resistance of a hand-remoulded specimen measured after applying multiple vane rotations.

Different values of the remoulded shear strength are often obtained from the different measurement methods.

A specimen may be tested in the sample tube in which it was taken, in a block sample or in a mould after removal from a sampler. The test apparatus consists of a rectangular vane with a short push rod for penetration into the soil. The vane is then slowly rotated by hand and the maximum torsional moment is recorded. Various vane sizes can be selected depending on the consistency of the specimen. Calculation of undrained shear strength is based on a cylindrical failure surface for which uniform stress distributions are assumed. The equation for undrained shear strength is as follows:

$$c_u = \frac{T_{max}}{\pi D^2 \left(\frac{1}{2} H + \frac{1}{6} D \right)}$$

where:

c_u	= peak undrained shear strength	[kPa]
T_{max}	= maximum torsional moment	[kNm]
D	= vane diameter	[m]
H	= vane height	[m]

Test reference: in-house

LABORATORY MINIATURE VANE

The laboratory miniature vane test allows determination of undrained shear strength of cohesive soil. CEN (2007) classifies the laboratory miniature vane as a strength index test.

Several different measurements of undrained shear strength are possible:

a) Intact: undisturbed undrained shear strength as measured on an intact specimen.
b) Intact–residual: measured post-peak during initial shearing of an intact specimen.
c) Intact–vane-remoulded: measured after multiple rotations of the vane after completion of the intact test.
d) Hand-remoulded: steady state (post-peak if exists) resistance of a hand-remoulded test specimen.
e) Hand-remoulded–vane-remoulded: steady state resistance of a hand-remoulded specimen measured after applying multiple vane rotations.

Different values of the remoulded shear strength are often obtained from the different measurement methods.

A specimen may be tested in the sample tube in which it was taken or in a mould after extrusion from the sample tube. The sample tube or mould is mounted in the test apparatus and a rectangular vane is lowered into the soil. The vane is then rotated at 10°/min (BS 1377) or at 60°/min to 90°/min (ASTM D4648) and the maximum torsional moment is recorded. A continuous record of rotation versus torsional moment can also be made if required (optional). Various vane sizes can be selected depending on the consistency of the specimen. Calculation of undrained shear strength is based on a cylindrical failure surface for which uniform stress distributions are assumed. The equation for undrained shear strength is as follows:

$$c_u = \frac{T_{max}}{\pi D^2 \left(\frac{1}{2} H + \frac{1}{6} D \right)}$$

where:

c_u	= peak undrained shear strength	[kPa]
T_{max}	= maximum torsional moment	[kNm]
D	= vane diameter	[m]
H	= vane height	[m]

Test references: BS 1377: Part 7: 1990, ASTM D4648/D4648M-16, ISO 19901-8:2014

UNCONSOLIDATED UNDRAINED TRIAXIAL (UU)

This type of test is usually performed on undisturbed (intact) samples of cohesive soils. Depending on the consistency of the cohesive material, the test specimen is prepared by trimming the sample or by pushing a mould into the sample. A latex membrane with a thickness of approximately 0.2 mm is placed around the specimen. A lateral confining pressure of 600 kPa to 1000 kPa is maintained during axial compression loading of the specimen. Some test procedures consider lateral confining pressures that are equivalent to total in situ vertical stress. Consolidation and drainage of pore water during testing is not allowed. The test is deformation controlled (strain rate of 60%/h), single stage, and stopped when an axial strain of 15% or 20% is achieved. The deviator stress is calculated from the measured load assuming that the specimen deforms as a right cylinder.

The presentation of test results includes a plot of deviator stress versus axial strain. The undrained shear strength, c_u, is taken as half the maximum deviator stress. The stress at 15% or 20% strain is used to calculate undrained shear strength if a maximum stress has not been reached earlier.

To determine strength sensitivity, the test may be repeated on remoulded (compacted) specimens. When possible, the tested undisturbed specimen is kneaded in the membrane, and then reshaped in a mould prior to testing. Stiff to hard specimens are cut into pieces, and reconstituted (compacted) by tamping the pieces in layers into a mould, until the original specimen dimensions are obtained. The sensitivity is the ratio of shear strength of undisturbed soil to shear strength of remoulded soil, $c_u/c_{u;r}$.

Test references: ASTM D2850-15 (2015), BS 1377: Part 7: 1990 (Clause 8), ISO 19901-8:2014

STRENGTH TESTING

RING SHEAR - SOIL/STEEL INTERFACE

Ring shear interface tests are performed on remoulded or reconstituted (compacted) soils to infer the residual friction angle, also called the constant volume friction angle (δ_{cv}), on a soil-steel interface.

The ring shear apparatus enables an annular specimen of soil, 5 mm thick with internal and external diameters of 70 mm and 100 mm, respectively, to be subjected to rotational shear.

First, the sample is consolidated to selected stress conditions. Then, it is sheared at a rate of 500 mm/min (fast shear), followed by 50 mm/min, up to a relative displacement of at least one metre. The sample is then resting for a period of 24h and after that is again consolidated to its selected stress conditions. Finally, the sample is sheared at a slower rate of 0.018 mm/min under drained conditions.

The presentation of the test results includes a plot of stress ratio and angle of shearing resistance versus displacement, both for fast and slow shear.

Test reference: BS 1377: Part 7: 1990, Jardine et al. (2005) (Appendix A), ISO 19901-8:2014

DIRECT SIMPLE SHEAR (DSS)

Simple shear tests provide a simulation of the plane strain mode of shearing for undisturbed (intact), remoulded or reconstituted (compacted) specimens. Key features of the DSS test are essentially constant horizontal dimensions of the specimen in the direction of shear, and a constant volume during shear to simulate undrained behaviour for a saturated test specimen. A constant volume is achieved by maintaining a constant specimen height. A constant specimen height is achieved by varying the normal load applied to the specimen or by fixing the vertical loading ram in place.

The direct simple shear test is carried out on a cylindrical specimen of 66 mm diameter and 16 mm to 19 mm height depending on the test apparatus. Lateral confinement of the specimen is provided by a membrane in combination with a stack of brass shearing washers, or by a reinforced membrane. There are no facilities for applying back pressure and control of drainage.

The stress state within a test specimen is insufficiently uniform to allow fundamental processing of test results. Nevertheless, data are commonly presented by shear stresses and strains for the horizontal plane and by equivalent pore pressures.

The peak horizontal shear stress is inferred as the undrained shear strength.

Test reference: ASTM D6528-07, ISO 19901-8:2014

ISSUE 43

DIRECT SHEAR – SOIL/SOIL INTERFACE

Direct shear testing (or shear box testing) is a method for determining drained soil resistance (angle of internal friction, φ') for cohesionless and cohesive soils.

The soil to be tested is placed in a split mould, with internal dimensions of 60 mm by 60 mm. A porous stone and loading plate are placed on top of the specimen and a normal load is applied to the specimen. The sample is then sheared, by displacing the top half of the split mould relative to the bottom half, at a rate of displacement preventing significant excess pore-water pressures to be generated. During the test, horizontal displacement, load and vertical displacement are recorded.

On completion of the first stage, the specimen is removed from the mould and the unit weight and water content are determined. Two further tests may then be performed, at the same unit weight, but with increased normal loads.

The test results are presented in the form of graphs of horizontal displacement versus shear stress and normal stress versus maximum shear stress.

Test reference: BS1377: Part 7: 1990, ISO 19901-8:2014

CONSOLIDATED UNDRAINED TRIAXIAL (CIU AND CAU)

The consolidated undrained triaxial test offers the opportunity to derive both undrained and drained strength parameters for undisturbed (intact) or remoulded (compacted) specimens. Specimens are generally prepared by trimming cohesive samples to the required dimensions. The wet mounting method is used, which includes use of wet porous disks and a water-filled drainage system.

Test procedures include specimen saturation, consolidation and compression loading. For cohesive soils, filter paper strips are attached to the specimen circumference to promote drainage during consolidation. Saturation is obtained by incrementing cell pressure and back pressure. The degree of saturation is checked by the pore water pressure response to small variations in cell pressure.

In case of isotropic consolidation (CIU) the specimen is usually consolidated to a stress level equivalent to the mean in situ stress estimated for the appropriate sample depth. For anisotropic consolidation (CAU), the specimen is consolidated to the estimated vertical and horizontal effective stresses. Various consolidation stages may be adopted to simulate the consolidation history and the effects of the expected loading sequence.

Specimen shearing is carried out under conditions of constant axial strain rate, while monitoring axial load and pore water pressure. A strain rate of 4%/h is generally applied, except when consolidation was slow, in which case a smaller strain rate is applied. The deviator stress is calculated from the measured load assuming the specimen deforms as a right cylinder. The shearing stage is terminated on the basis of effective principal stress ratio (ratio of effective axial stress to effective lateral stress σ'_1/σ'_3), or when an axial strain of 15% or 20% is reached. The CIU test may consist of three consolidation and shearing stages of increasing stress level. These stages may be performed on a single specimen or on three separate specimens.

The presentation of test results includes stress-strain data, effective stress paths, pore water pressures and shear strength parameters. Stress paths may be presented in terms of a mean effective stress (p' or s') and the principal stress difference or deviator stress (q or t) as follows:

- Cambridge p'-q space and ASTM p'-q space, with p' defined as $(\sigma'_1+2\sigma'_3)/3$ and q as $\sigma_1-\sigma_3$;
- BSI (1990) s'-t space, with s' defined as $(\sigma'_1+\sigma'_3)/2$ and t as $(\sigma_1-\sigma_3)/2$.

The undrained shear strength is defined as half the deviator stress at failure, c_u = q/2 and is reported for the following failure criteria:
1) maximum deviator stress
2) maximum stress ratio q/p'.

The stress at 15% or 20% strain is used to calculate undrained shear strength when a maximum stress has not been reached. A secant angle of internal friction, φ', is determined from q = Mp' where M = (6sinφ')/(3-sinφ') or sin φ' = 3M/(6+M) for compression conditions. This definition assumes a zero effective cohesion intercept and may be applied to M_{max} but also to other values of M and corresponding values of q and p'. Similarly, sin φ' = t/s'. For tests with three shearing stages, angles of internal friction may be determined for each stage separately, and from a straight line approximation of the failure points of the three stages. The latter method also provides a value for effective cohesion intercept c'.

Test references: NEN 5117, ASTM D4767-11, BS 1377: Part 8: 1990 (Clause 4, 5, 6, 7), ISO 19901-8:2014

CONSOLIDATED DRAINED TRIAXIAL (CID AND CAD)

Consolidated drained triaxial compression tests are generally performed on samples of cohesionless soils. The specimen of dry soil is prepared in the rubber membrane on the base of the triaxial cell, without the use of side drains. Soil particles larger than 20% of the diameter of the specimen are removed. Specimens are prepared by tamping thin layers of soil to a density approximating the estimated in situ dry density. To saturate the specimen, CO_2 gas is used to expel the air and subsequently de-aired water is used to expel the CO_2 gas. The specimen is further saturated by incrementing cell pressure and back pressure, until the pore pressure response to a cell pressure increment (B-factor) indicates saturation is complete. The specimen is then isotropically or anisotropically consolidated (CID and CAD respectively).

After consolidation the sample is sheared by applying axial load at a sufficiently slow rate to permit drainage (usually 6%/h). The lateral confining pressure is kept constant during each loading stage. Pore pressure measurements are made at the bottom to check if the test is fully drained. The deviator stress is calculated from the measured load assuming the specimen deforms as a right cylinder. The CID test may have three consolidation and loading stages of increasing pressure performed on either a single specimen or on three separate specimens. The CAD test is limited to a single shearing stage. A shearing stage is terminated on the basis of effective stress ratio (ratio of effective axial stress to effective lateral stress, σ'_1/σ'_3), or when an axial strain of 15% or 20% is reached.

Results include stress-strain data, stress paths, and volumetric/shear strain of each loading stage. Stress paths may be presented in terms of a mean effective stress (p' or s') and the principal stress difference or deviator stress (q or t) as follows:

- Cambridge p'-q space and ASTM p'-q space, with p' defined as $(\sigma'_1+2\sigma'_3)/3$ and q as $\sigma_1-\sigma_3$;
- BSI (1990) s'-t space, with s' defined as $(\sigma'_1+\sigma'_3)/2$ and t as $(\sigma_1-\sigma_3)/2$.

A secant angle of internal friction, φ', is determined from q = Mp' where M = (6sinφ')/(3-sinφ') or sin φ' = 3M/(6+M) for compression conditions. This definition assumes zero effective cohesion intercept and may be applied to M_{max} but also to other values of M and corresponding values of q and p'. Similarly, sin φ' = t/s'. For tests with three shearing stages, angles of internal friction may be determined for each stage separately, and from a straight line approximation of the failure points of the three stages. The latter method also provides a value for effective cohesion intercept c'.

Test reference: ASTM D7181-11; BS 1377: Part 8: 1990 (Clause 4, 5, 6, 8), ISO 19901-8:2014

REFERENCES

American Association of State and Highway Transportation Officials, 2007. *T290-95-UL Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil*. Washington, D.C., AASHTO

American Association of State and Highway Transportation Officials, 2008. *AASHTO T291-94-UL Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil*. Washington, D.C., AASHTO

ASTM International, 1963. *ASTM D422-63(2007) Standard Test Method for Particle-Size Analysis of Soils*. West Conshohocken: ASTM International.

ASTM International, 2014. *ASTM D854-14 Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.* West Conshohocken: ASTM International.

ASTM International, 2010. *ASTM D2216-10 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.* West Conshohocken: ASTM International.

ASTM International, 2011. *ASTM D2435/D2435M-11 Standard Test Methods for One-dimensional Consolidation Properties of Soils Using Incremental Loading.* West Conshohocken: ASTM International.

ASTM International, 2015. *ASTM D2850-15 Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.* West Conshohocken: ASTM International.

ASTM International, 2014. *ASTM D2974-14 Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils.* West Conshohocken: ASTM International.

ASTM International, 2012. *ASTM D4186/D4186M-12 Standard Test Method for One-dimensional Consolidation Properties of Saturated Cohesive Soils using Controlled-strain Loading.* West Conshohocken: ASTM International.

ASTM International, 2010. *ASTM D4318-10e1 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.* West Conshohocken: ASTM International.

ASTM International, 2014. *ASTM D4373-14 Standard Test Method for Rapid Determination of Carbonate Content of Soils.* West Conshohocken: ASTM International.

ASTM International, 2016. *ASTM D4648/D4648M-16 Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-grained Clayey Soil.* West Conshohocken: ASTM International.

ASTM International, 2011. *ASTM D4767-11 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.* West Conshohocken: ASTM International.

ASTM International, 2010. *ASTM D5084-10 Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.* West Conshohocken: ASTM International.

ASTM International, 2007. *ASTM D6528-07 Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils.* West Conshohocken: ASTM International.

ASTM International, 2009. *ASTM D6913-04(2009)e1 Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.* West Conshohocken: ASTM International.

ASTM International, 2011. *ASTM D7181-11 Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils.* West Conshohocken: ASTM International.

ASTM International, 2009. *ASTM D7263-09 Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens.* West Conshohocken: ASTM International.

British Standards Institution, 1990. *BS 1377:1990 British Standard Methods of Test for Soils for Civil Engineering Purposes.* London: BSI. (Parts 1-9, with Amendments)

British Standards Institution, 2015. *BS 5930:2015 Code of practice for ground investigations*. London: BSI.

DeGroot, D.J., Poirier, S.E. and Landon, M.M. 2005. Sample Disturbance - Soft Clays. *Studia Geotechnica et Mechanica,* Vol. 27, No. 3-4, pp. 91-105.

Danish Geotechnical Institute (DGI), *Minimum Index Void Ratio, e_{min} (Danish Method).* DGI Product Sheet #000 96-07-02.

International Organization for Standardization, 2004. *ISO 10693:2004 Soil quality - Determination of Carbonate Content - Volumetric Method.* Geneva: ISO.

International Organization for Standardization, 2004. *ISO/TS 17892-1:2014 Geotechnical Investigation and Testing - Laboratory Testing of Soil - Part 1: Determination of Water Content.* Geneva: ISO.

International Organization for Standardization, 2004. *ISO/TS 17892-4:2004 Geotechnical Investigation and Testing – Laboratory Testing of Soil - Part 4: Determination of Particle Size Distribution.* Geneva: ISO.

International Organization for Standardization, 2006. *ISO 22475-1:2006 Geotechnical Investigation and Testing - Sampling Methods and Groundwater Measurements - Part 1: Technical Principles for Execution.* Geneva: ISO.

International Organization for Standardization, 2014. *ISO 19901-8:2014 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures – Part 8: Marine Soil Investigations.* Geneva: ISO.

Jardine, R., Chow, F.C., Overy, R.F. and Standing, J.R. 2005. *ICP Design Methods for Driven Piles in Sands and Clays*. London: Thomas Telford.

Kay, S., Goedemoed, S.S. and Vermeijden, C.A. 2005. Influence of Salinity on Soil Properties. In Gourvenec, S. and Cassidy, M. Eds., *Frontiers in Offshore Geotechnics ISFOG 2005: Proceedings of the First International Symposium on Frontiers in Offshore Geotechnics, University of Western Australia, Perth, 19-21 September 2005,* London: Taylor & Francis, pp. 1087-1093.

Lunne, T., Berre, T., Andersen, K.H., Strandvik, S. and Sjursen, M. 2006. Effects of Sample Disturbance and Consolidation Procedures on Measured Shear Strength of Soft Marine Norwegian Clays. *Canadian Geotechnical Journal*, Vol. 43, No. 7, pp. 726-750.

Nederlands Normalisatie-instituut, 1991. NEN 5117 Geotechnics - Determination of the Shear Resistance and Deformation Parameters of Soil - Triaxial Test.Delft: NEN. (With Amendment NEN 5117/A1, May 1997) (in Dutch)

Terzaghi, K. 1943, 1946. *Theoretical Soil Mechanics*. New York: Wiley.

INTRODUCTION

Site characterisation may be defined as a fit-for-purpose model of seabed conditions at a geographical location in a sea or ocean. Seabed is the ground below seafloor, including pore fluid and gas. The model is fundamental to managing ground risks and optimizing opportunities. The model is a prediction and a reduction of reality:

- Providing sound information with which to define and assess the suitability of a site for proposed facilities
- Detecting and assessing the possible effects of geohazards and changes in seabed conditions with time
- Choosing parameter values for assessment of limit states and assess the feasibility of building/installing, operating and/or decommissioning a structure.

Other terms used in practice for (parts of) site characterisation include integrated study, integrated geosciences, desk study, and seabed characterisation.

Site characterisation can also refer to the activities required to create the model of seabed conditions (e.g. Evans, 2010; Peuchen, 2014).

The terms seabed and seafloor are according to ISO (2003):

- Seabed comprises materials below the sea in which a structure is founded, whether of soils such as sand, silt or clay, cemented materials or, of rock
- Seafloor is defined as the interface between the sea and the seabed.

This document focuses on offshore projects. Site characterisation is an integral part of offshore structure design and operation according to reliability principles covered by standards and codes of practice; for instance API (2011, 2014 and 2015), RenuwableUK (2013), CEN (2004 and 2011); ISO (2002, 2003, 2004, 2009, 2013 and 2016), Osborne et al. (2011) and SNAME (2008).

The following sections provide further information.

SITE HAZARDS

TYPES OF HAZARDS, RISK AND MITIGATION

Site hazards may be grouped into:

- natural geohazards
- man-made hazards.

Natural geohazards are commonly referred to as geohazards or geological hazards. They are about past geological processes and events have shaped the seafloor and seabed. Some of these processes may still be active today. The resulting seafloor topography, and geological and geotechnical conditions within the seabed can be hazardous when installing offshore structures including infrastructure (e.g. Clayton and Power, 2002; OGP, 2009; API, 2011).These processes.

Man-made hazards include shipwrecks, fallen objects, seafloor debris and unexploded ordnance. Within the context of this document, man-made hazards exclude accidental events such as vessel impact, sabotage, well drilling problems and fishing activities.

In relation of offshore activities, geohazards can be defined as local and/or regional site and soil conditions having a potential of developing into a condition (e.g. irregular seafloor topography) or process (e.g. currents, submarine slides) that could cause loss of life or damage to health, environments and/or assets. The event-triggering sources can be ongoing geological processes or human induced changes (OGP, 2009). Figure 1 presents a schematic overview of offshore geohazards.

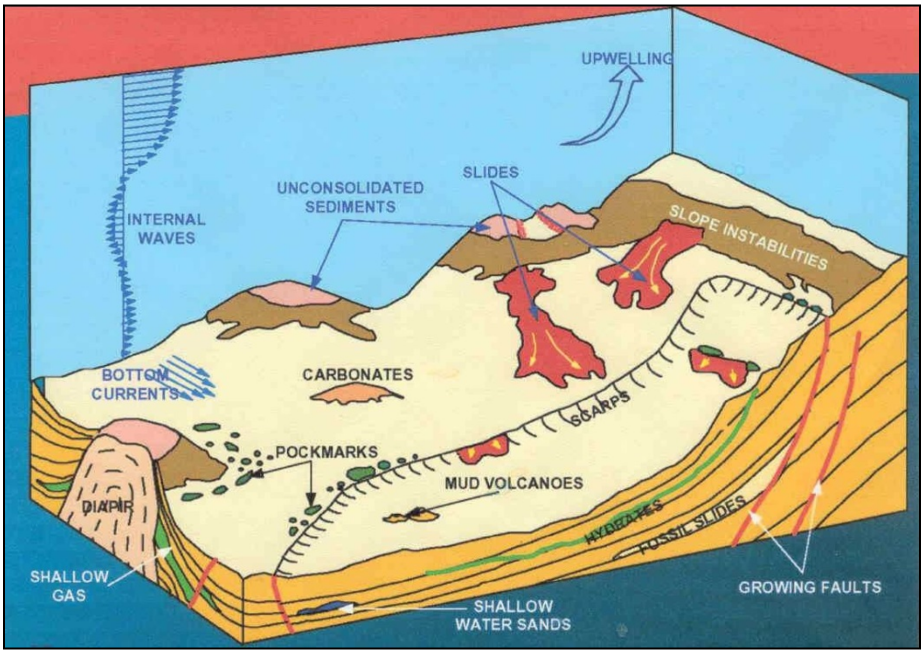


Figure 1: Offshore natural geohazards in deep water settings (modified after Campbell et al., 1986)

The damage potential of site hazards can range from, for example, local effects on pipelines and subsea structures to complete loss of all installations in a license areas and 3rd party losses (OGP, 2009).

The table below presents an overview of potential impacts and/or consequence associated with natural geohazards (and man-made hazards) occurring offshore.

Table 1: Potential Impact/Consequence Associated with Site Hazards

Impact / Consequence	Natural Geohazards and Man-made Hazards															
	Irregular Seafloor Topography	Seafloor Bedforms	Seafloor Outcrops and Hard Seafloor	Soil Liquefaction	Shallow Gas & Gassy Soils	Gas Hydrates	Gas and Fluid Seepage	Diapirs (e.g. Mud /Salt) and Mud Volcanoes	Earthquakes	Faults	Tsunami	Slope Failure	Submarine Mass Movement	Wind, Waves and Currents	Seafloor Scour and Sediment Mobility	Man-Made Hazards
Uneven support (foundation instability)		x				x				x	x				x	
Loss of support (structural stresses)				x			x		x		x	x	x			
Spanning (pipeline & flowlines)	x	x	x							x						
Increased foundation settlements, reduced access				x	x											
Burial / embedment leading to additional loading and reduced access		x		x									x		x	
Reduced soil strength and bearing resistance				x	x		x									

Impact / Consequence	Natural Geohazards and Man-made Hazards															
	Irregular Seafloor Topography	Seafloor Bedforms	Seafloor Outcrops and Hard Seafloor	Soil Liquefaction	Shallow Gas & Gassy Soils	Gas Hydrates	Gas and Fluid Seepage	Diapirs (e.g. Mud /Salt) and Mud Volcanoes	Earthquakes	Faults	Tsunami	Slope Failure	Submarine Mass Movement	Wind, Waves and Currents	Seafloor Scour and Sediment Mobility	Man-Made Hazards
Lateral loading of structure leading to overstressing of foundation / structure components									x		x	x	x	x		x
Structure displacement and structural damage				x					x	x	x	x	x			x
Increased potential for soil liquefaction					x	x	x		x		x			x		
Increased potential for shallow soil instability and submarine sliding					x	x	x	x	x		x			x	x	
Foundation and structure installation difficulties	x	x	x		x	x	x									x
Steel abrasion, gouging and denting; excessive wear trenching equipment			x													
Gas and fluid migration (excess pore pressures)					x	x	x	x		x	x			x		
Corrosion of steel structures, pipelines, flowlines					x		x	x								
Well (borehole) instability					x	x	x			x						
Mud losses (well/borehole drilling)										x						
Damage to casing string and pile foundations										x						
Presence of environmentally protected chemosynthetic communities					x		x	x								
Explosions leading to changed site conditions																x

Site hazards can generally not be treated on a statistical basis applying solely historical data. The nature of a hazard is often site and time dependent. In addition, natural geohazards are often interrelated. This may be due to a common trigger mechanism (e.g. earthquake, slope failure), or that one geohazard occurrence or process forms a trigger for other geohazards.

For instance:

- Earthquakes will induce dynamic actions on a structure and may induce elevated pore pressures leading to increased susceptibility to soil liquefaction;
- Slope failures and their deposits may result in irregular seafloor topography;
- Mud and salt diapirs are commonly associated with radial fault patterns, and continuous diapirism may result in (shallow) slope failures.

Table 2 highlights some relations between natural geohazards.

Table 2: Related Offshore Natural Geohazards

	Irregular Seafloor Topography	Seafloor Bedforms	Seafloor Outcrops and Hard Seafloor	Soil Liquefaction	Shallow Gas & Gassy Soils	Gas Hydrates	Gas and Fluid Seepage	Diapirs (e.g. mud /salt) and Mud volcanoes	Earthquakes	Faults	Tsunamis	Slope Failure	Submarine Mass Movement	Wind, Waves and Currents	Seafloor Scour and Sediment Mobility
Irregular Seafloor Topography		x	x							x		x	x	x	x
Seafloor Bedforms	x													x	x
Seafloor Outcrops and Hard Seafloor	x				x		x	x				x			x
Soil Liquefaction					x	x	x	x	x					x	
Shallow Gas & Gassy Soils			x	x		x	x	x		x		x	x		
Gas Hydrates				x	x		x					x	x		
Gas and Fluid Seepage			x	x	x	x		x		x		x	x		
Diapirs (e.g. mud /salt) and Mud volcanoes			x	x	x		x			x		x			
Earthquakes				x						x	x	x	x		
Faults	x				x		x	x	x		x	x	x		
Tsunamis									x	x		x	x	x	x
Slope Failure	x		x		x	x	x	x	x	x	x		x	x	x
Submarine Mass Movement	x				x	x	x		x	x	x	x		x	x
Wind, Waves and Currents	x	x		x							x	x	x		x
Seafloor Scour and Sediment Mobility	x	x	x								x	x	x	x	

Assessment of hazard probability of occurrence and frequency can be based on geomechanical modelling taking into account uncertainty in modelling of site conditions, soil parameter values, ongoing geological processes, actions and applied analysis methods (Clayton and Power, 2002; OGP, 2009).

The risk of a site hazard is the sum of the product of the probability of a hazard event affecting a structure and damage consequence. The damage consequence can depend on factors such as structure robustness and vulnerability. The information in this document covers the nature of hazards and their potential implications, not the risk. Power et al. (2005) and Galavazi et al. (2006) describe risk analysis methodology.

Risk mitigation can include avoidance (e.g. a certain standoff distance to avoid structure interaction) and design for robustness.

IRREGULAR SEAFLOOR

Seafloor morphology can be irregular as a result of past or present geological processes. Human activities can also affect the seafloor topography. Irregular seafloor may be caused by (or be associated with) a number of natural and man-made phenomena. These include:

- Canyons and channels
- Boulders (e.g. drop stones)
- Spudcan footprints
- Anchor scars
- Trawl marks and scars
- Drill cuttings.

The scale of morphological features varies (e.g. scour marks, submarine canyons). The impact can differ per structure type and geometry.

SEABED SCOUR AND SEDIMENT MOBILITY

Seabed scour relates to the erosion of seabed sediments. Such erosion can occur under normal metocean conditions or can be enhanced as a result of a structure or multiple structures interrupting a natural flow regime above seafloor, thereby increasing flow velocities. Scour can be enhanced or initiated by secondary processes such as rocking of a structure.

Especially non-cohesive sandy (and silty) sediments are susceptible to scour. Erosion and transport of fine sand can start at a flow velocity in excess of 0.2 m/s. Local scour pits (or scour holes) can form shortly after installation of a structure. Their dimensions will usually vary in time depending on the flow regime.

Scour can occur in any water depth (from shoreline to deep sea). The flow regime due to wave- and tidal-influence is generally stronger in shallow water than in deep water (Soulsby, 1997; Sumer & Fredsoe, 2002). In general, tide- and wave-action, in combination with fluvial discharge of fresh water determine the natural flow regime in coastal areas. Deepwater bottom current activity may result from density differences between water masses and from global thermohaline ocean circulation. Resulting sedimentary accumulations are known as contourite drifts (Faugeres et al., 1999).

Seafloor variation can usually be characterized as some combination of the following Whitehouse (1998):

- Local scour and sedimentation; usually a steep sided scour pit around a structure or structural element
- Global (or general) scour; a (shallow) scoured basin of large extent around a structure, possibly due to overall structure effects, multiple structure interaction, or wave-soil-structure interaction
- Overall seabed movement; erosion, deposition, bedform migration that would also occur in the absence of a structure (i.e. regional scour).

SEAFLOOR BEDFORMS

A seafloor bedform is a morphological feature formed by interaction of wave-action and (tidal-) currents and cohesionless sediment (i.e. sand/silt). Bedforms are typically found on sandy areas of continental shelves.

Bedforms can be grouped into:

- Ripples: wave length about 0.3 m to 0.6 m, height up to 0.05 m
- Mega ripples: wave length 0.3 m to 1 m, height 0.05 m to 0.2 m
- Sand waves (dunes): wave length 30 m to several hundreds of metres, height between 1 m to 2 m and 10 m to15 m
- Sand banks: wave length 1 km to tens of km, width 0.5 km up to 10 km, height up to tens of metres.

A characteristic of bedforms is their mobility. Sand waves tend to move slowly (metre per year) or flex their crests with tidal currents. Ripples tend to be more mobile, in the order of a metre per day (Morelissen et al., 2003).

For structure design it is important to identify which part of the seabed and/or the bedforms is actually mobile. The rate at which bedforms recover after having been modified by, for example, cable trenching mainly depends on sediment transport rate and supply of sediment.

SEAFLOOR OUTCROPS AND HARD SEAFLOOR

Seafloor outcrops and hard seafloor ground conditions commonly include:
- Shell and coral banks, reefs, which are common in shallow waters in the tropical zones.
- Local patches of cemented soil (e.g. hard ground, cap rock). Examples are authigenic carbonates around pockmarks, Kurkar ridges (cemented aeolian dunes) in the eastern Mediterranean Sea, beach rocks (cemented beach sediments) in the Caribbean Sea, sabkha deposits (evaporitic-tidal floodplain deposits) in the Arabian/Persian Gulf and Gulf of Suez.
- Crust composed of precipitated metal sulphides associated with hydrothermal activity (e.g. black and white smokers) in vicinity of tectonic plate boundaries and faults.
- Outcrops of rock. Examples are pre-Quaternary sand- and limestone beds offshore West Africa, sedimentary and metamorphic rocks exposed in the Irish Sea.

It should be noted that seafloor outcrops and hard seafloor may have environmental protection status or legislative implications.

Cementation of soil may result from sub-marine cementation processes. Cementation may also have resulted from past sub-aerial exposure of a continental shelf during low sea level stands under arid climate conditions. Cementation generally occurs in carbonate-rich and hyper-saline environments.

DIAPIRS AND MUD VOLCANOES

A diapir is a domal upwelling of sediment, rock or salt that forms in response to tectonic forces, density differences and high overburden pressures. Diapirs can pierce through a stratigraphic overburden and create an envelope of overconsolidated soils, deformed rock and sediments around a diaper core (e.g. salt). Generally, a circular dome-shaped topographic feature develops when a diapir approaches the seafloor. Diapirs are commonly associated with radial faulting patterns and locally increased seafloor slopes.

Salt diapirs are known to be present in, for example, the Gulf of Mexico, offshore Brazil and West Africa, and the North Sea.

Mud diapirs and mud volcanoes are usually associated with rapidly-deposited sediments and in situ pore pressure conditions significantly higher than hydrostatic (overpressured). Additionally, high vertical and horizontal stresses typically apply, caused by faulting, folding and uplift processes.

Mud diapirs and mud volcanoes occur mostly in (historic) delta areas: Nile Delta (offshore Egypt), Absheron Ridge (offshore Azerbaijan, Caspian Sea), Makran Ridge (offshore Iran, Arabian Sea), and Niger Delta (offshore Nigeria).

Release of pressure is commonly provided by faults and folding of the strata. Sediments mixed with over-pressured fluid and gas (mud) migrate upward through the stratigraphic overburden in vertical columnar zones (diapirs). Usually the over-pressured muds enter fault planes, thus causing diapirism along faults. A mud volcano can form when a mud diapir breaks the seafloor.

In general, mud volcanoes are conical, as tall as 65 m and up to 2 km across. The size and shape of a mud volcano depends on the frequency of expulsion and the type of material ejected. This can be unconsolidated soils, overconsolidated material, fractured rock (e.g. breccia), oil, gas and water (Snead, 1972; Newton et al., 1980; Delisle et al., 2002; Delisle, 2004; Delisle, 2005). Not all offshore mud volcanoes are active. Eruptions are believed to be episodic.

SHALLOW GAS & GASSY SOILS

Gas may be present (trapped) in the seabed (e.g. gassy soils). Shallow gas can comprise a mixture of different gases, such as carbon dioxide, hydrogen sulphide, ethane and methane. In general, the gases originate from bacterial decay of organic matter (biogenic gases) within a few metres of the seafloor. Gas may also come from sources much deeper in the stratigraphy and migrate upwards through pores and cracks in the soil and rock (petrogenic gases).

Shallow gas may be present dissolved in pore water, as free gas in gas-filled voids or bubbles, and as gas hydrates. Over time, gas in soil may increase the in-situ pore pressures and result in excess pore pressures.

Migration of gas in soil can result in accumulation of gas in seabed below a foundation. Shallow gas in the pore water can have a serious effect on foundation behaviour.

In addition, shallow gas can be toxic to humans, can combust and explode.

Soil property measurements on geotechnical samples containing shallow gas may not be representative of in situ properties.

GAS HYDRATES

Gas hydrates are ice-like crystalline solids composed of water molecules surrounding a molecule of gas, generally methane. Gas hydrates can only form when gas is over-saturated in water. Gas hydrates are stable under high pressure and low temperature conditions, and may be present at seafloor and in shallow sediments, generally in deep water environments in excess of 500 m below Mean Sea Level (Rastogi et al., 1999; Von Rad et al., 2000).

Stable gas hydrate acts as cement and increases strength and rigidity of soil.

Natural gas hydrates are regarded as a geohazard when they dissociate, start “melting”. Both water and gas are released into soil when gas hydrates dissociate. This can result in formation of “gassy soils”. The addition of water and gas may decrease soil strength and form a weak layer (Orange and Breen, 1992; Judd and Hovland, 2007). Gas hydrate dissociation may be initiated by human activities, e.g. flow of “hot” hydrocarbons through well production casings, pipelines and flowlines.

Gas hydrates may for as a result of human activity. Gas hydrates can be a by-product of hydrocarbon production, forming hydrate plugs in the wellbore, around leaking joints and in pipelines. If a deep water exploration or production well is leaking, gas introduced into the shallow soils may react with water molecules to form hydrate layers or nodules.

GAS AND FLUID SEEPAGE

Gas and fluid seepage at seafloor is commonly associated with pockmarks. Pockmarks are roughly circular or conical depressions in the seafloor, generally 1 m to 350 m wide and up to 35 m deep (Newton et al., 1980; Von Rad et al., 2000; Judd and Hovland, 2007).

Pockmarks form by disruption of a pore pressure environment. This disruption may be triggered by natural or human causes, and can form on time scales of less than a year. Pockmarks can be intermittently active over long periods of time or can grow with explosive eruption events. The sediments in a pockmark are generally variable and may be overconsolidated.

When gas seeps continue over a long period of time, biological processes may cause cementation of the seabed sediments. Formation of authigenic carbonates can take place around the seeps (Judd and Hovland, 2007; Ding, 2008). In some cases, unique ecological habitats form in and around pockmarks. Such habitats may be protected by environmental legislation.

Authigenic carbonates may form thin crusts of weakly cemented sediments (hard grounds). They can be continuous over distances of several hundreds of metres (Von Rad et al., 2000). Locally more massive, competent layers of authigenic carbonates can be present as hard cemented layers or ‘lenses’. They may form large build-ups and seafloor mounts (Judd and Hovland, 2007).

Apart from natural seeps, gas seepage may also be induced by drilling activities (e.g. geotechnical drilling, hydrocarbon exploration drilling). The drilling process may cause fracturing of soil and rock, when drilling mud pressures exceed the fracture pressure of the soil or rock (i.e. hydraulic fracturing). These fractures may form pathways for fluid and gas migration into the wellbore and up to seafloor. A wellbore or leaking well casing may form a pathway to the surrounding rock and soil formations, introducing gas into sand layers in the shallow subsurface. Overtime, the introduced gas may affect the geotechnical properties of a soil and have serious effects on foundation behaviour.

Drilling-induced fluid flows (e.g. shallow water flows) occur when a pressurised sand body (aquifer) encapsulated in clay is penetrated by the drilling process. Shallow water flows are common offshore large river deltas, such as the Mississippi Delta (Gulf of Mexico) and the Nile Delta (offshore Egypt). The sand bodies are commonly derived from sediment deposition out of turbidity currents.

EARTHQUAKES

An earthquake, or seismic event, occurs after stresses in the earth's crust that have gradually built up, are suddenly released by movements along a fault. The movement generates seismic waves which propagate away from the earthquake epicentre. Most earthquakes occur along tectonic plate boundaries.

The location, magnitude and frequency (recurrence) of earthquakes cannot be reliably predicted. The probability of seismic events can be assessed on the basis of historic records of earthquake activity.

Seismic impact depends on geotechnical conditions at the site and structure design. Seismic activity may induce faulting, soil liquefaction, slope failure, and tsunamis.

SOIL LIQUEFACTION

Two types of liquefaction may be distinguished:
- gravitational (sometimes called static or flow) liquefaction, usually occurring in submerged slopes;
- cyclic liquefaction, usually generated through strong cyclic forces.

Soil liquefaction or cyclic mobility represents a decrease of soil strength and stiffness caused by an increase in pore water pressure in saturated soil. Soil liquefaction usually occurs in response to sudden change in stress condition, causing it to behave like a liquid. Examples of cyclic and dynamic actions include earthquake shaking, storm wave loading, structure displacements upon cyclic load application, pile installation by driving and vortex vibrations due to fluid flow around a structure.

Liquefaction potential can be significant for loose cohesionless soils present close to ground surface (seafloor) and below the water table. Dense sands, loose unsaturated sands and some sensitive cohesive materials can also liquefy under some conditions. In addition, the presence of gas in loose sands can change soil behaviour and may potential for liquefaction (Grozic, 2003).

FAULTS

A fault is a planar fracture or discontinuity in a volume of soil or rock along which significant vertical and/or horizontal displacement has occurred (Figure 2) (i.e. faulting). Fault zones are areas where multiple fractures and faults occur in close proximity, with similar moment direction.

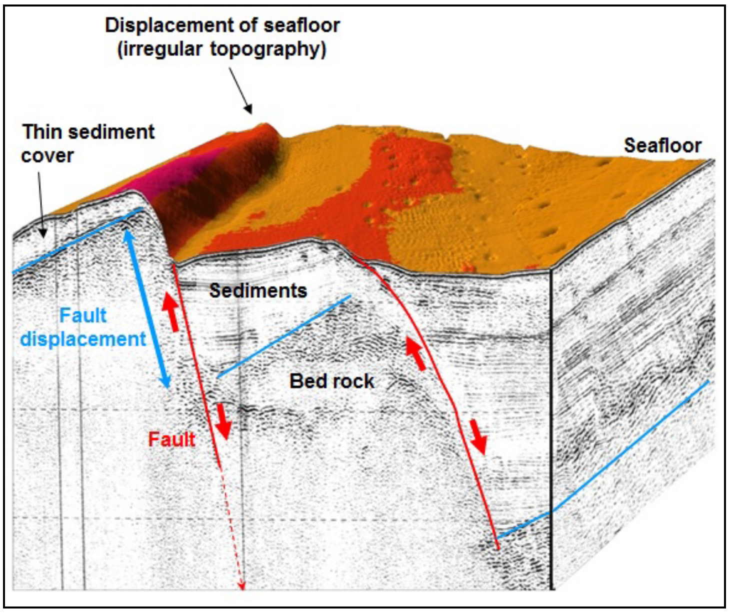


Figure 2: Surface and subsurface expression of fault displacement

Faults can be associated with:

- Tectonic activity (e.g. at tectonic plate boundaries, earthquake zones);
- Laterally variable soil subsidence and compaction;
- Soil contractions (e.g. polygonal faulting in North Sea and West African seabed sediments);
- Diapirism (e.g. radial faulting);
- Slope failure (e.g. headwall scarp, failure planes, tension cracks).

Movement along the fault plane (and hence soil displacement) is a semi-continuous process acting on time scales ranging from years to millions of years. Faults are commonly considered to be in-active if there has been no observed movement or evidence of seismic activity during the last 10,000 years. In this case a fault can be covered by a uniform layer of soil (i.e. without a clear discontinuity surface being present). Depending on crustal stresses and changes therein, apparently in-active faults may be reactivated causing further soil displacements and even seismic events.

Faults may result in a displaced, stepped seafloor and/ or irregular linear topographic features on the seafloor (e.g., headwall scarps). In addition, stratigraphic sequences are displaced in the seabed.

Deep-seated faults, with lengths of 100's to 1000's of metres, may be associated with earthquakes. The build-up of stresses due to differential movement in the earth's crust may be released along these deep-seated faults, whereby large amounts of energy move through rock and soils in the form of pressure waves and shear waves. These deep-seated, earthquake generating, faults are sometimes referred to as seismic faults.

TSUNAMIS

A tsunami (or surge wave) is a series of ocean waves of long wave lengths, which are created when a large volume of water is suddenly displaced by a submarine earthquake, landslide or volcanic eruption (Figure 3). In the open ocean, tsunami waves travel at high speeds (in excess of 800 km/h) with heights of, say, less than 0.05 m. As they approach the coast, the velocity decreases (to approximately 50 km/h) and the wave height increases up to several metres or tens of metres. At the coastline, the force of a tsunami wave can cause loss of life, damage to buildings and infrastructure, large scale erosion (scour) and flooding of low-lying areas.

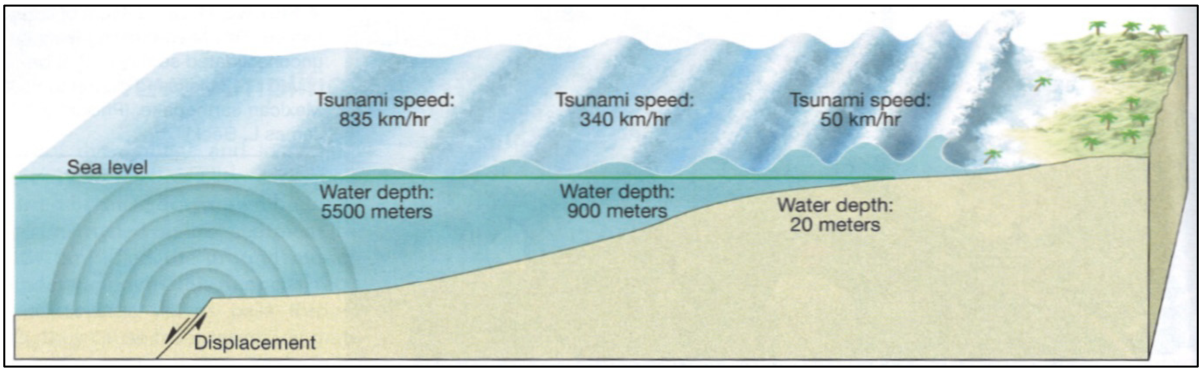


Figure 3 Tsunami generated by fault displacement offshore

SLOPE FAILURE

Slope failure occurs when downslope driving forces acting on seabed exceed resistance. In general, slope failure results in the down-slope movement of a soil mass (see section titled Submarine Mass Movements). Slopes may be unstable at any water depth.

Slopes may develop due to tectonics, high sedimentation rates or incision and erosion by seafloor currents and flows.

Slope failure can be triggered by earthquakes, strong currents, storms (wave actions), tsunamis, volcanism and human activity (Hampton et al., 1996; Mulder and Cochonat, 1996; Locat and Lee, 2005; Judd and Hovland, 2007; Rogers and Goodbred, 2010).

Usually, a combination of two or more factors influence slope failure, e.g. presence of shallow gas and an earthquake (Orange and Breen, 1992; Judd and Hovland, 2007). Slopes can be unstable due to low shear strength and overpressured strata (e.g. shallow gas). Seabed may fail on slight slopes as little as 0.5˚ (Hampton et al., 1996; Judd and Hovland, 2007).

Failure scarps and oversteepened slopes are commonly associated with past slope failures. Past slope failures may be reactivated if a trigger (e.g. pore pressure build-up, earthquake) is present. The seafloor morphology resulting from a slope failure may be irregular and undulating (see section titled Irregular Seafloor Topography).

SUBMARINE MASS MOVEMENTS

A submarine mass movement is a displacement of seabed material driven directly by gravity or other body forces, rather than stresses associated with fluid flow. The deposits of submarine mass movements are commonly referred to as mass transport deposits, MTDs.

Submarine mass movements commonly follow from slope failures and include the following processes (Figure 4) (Lee et al., 2007):

- Slides:
  - Translational slide
  - Rotational slide
- Mass flows:
  - Debris flow
  - Debris avalanche
  - Mud flow
  - Liquefaction flow
  - Turbidity current

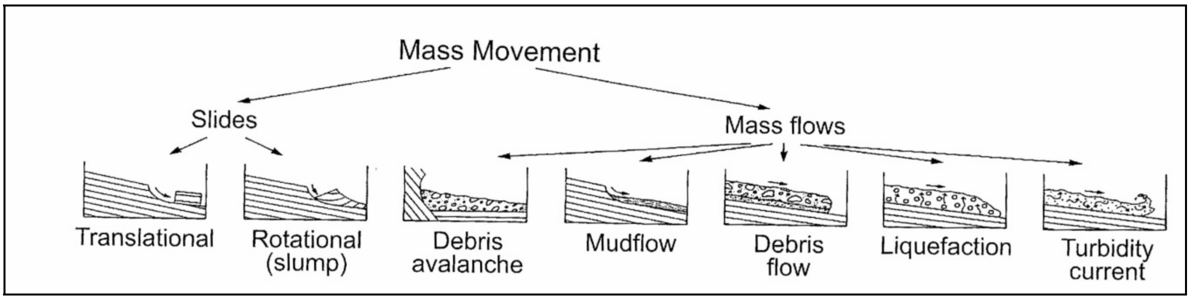


Figure 4: Submarine mass movement classification (after Lee et al., 2007)

Slides are movements of essentially rigid, undeformed masses along discrete failure/slip planes. If slip occurs along a planar surface the slide is referred to as a translational slide. If slip occurs along a curved failure plane and the rigid mass shows rotation, the slide is referred to as rotational.

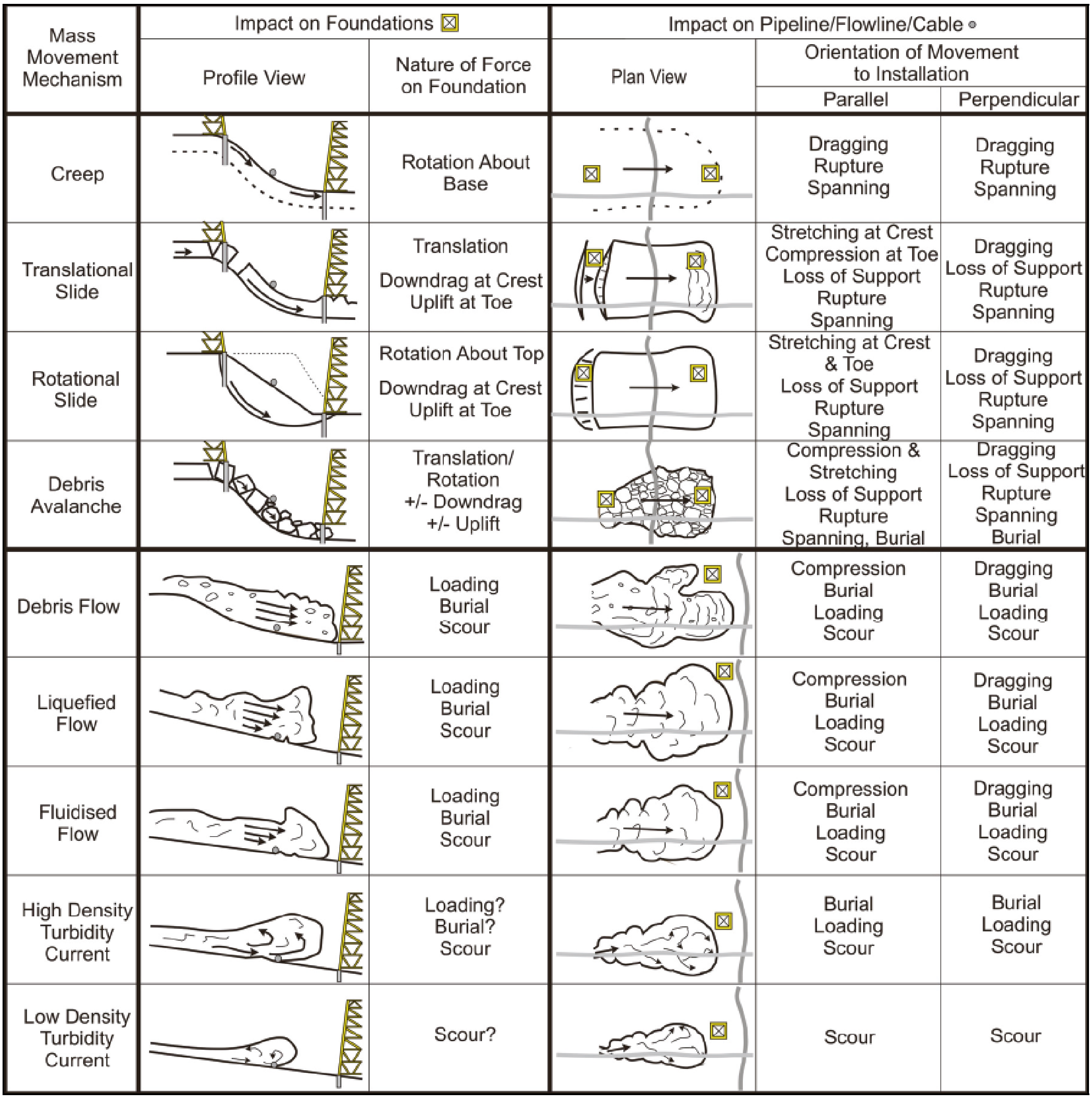
If moving sediments take a form of viscous fluid, the feature is referred to as mass flow or gravity flow. Mass flow deposits show considerable internal deformation with many invisible or short-lived internal slip surfaces. Submarine slides can become mass flows as the failed material progressively disintegrates, gets entrained with surrounding water and moves downslope.

Debris flows are mass flows in which sediments are heterogeneous and may include larger clasts supported by a fine-grained soil matrix. Mud flows involve predominantly fine-grained (mud) sediments. Turbidity currents involve downslope transport of a relatively dilute suspension of sediment grains that are supported by an upward component of fluid turbulence. Turbidity currents often evolve from disintegration and dilution

ISSUE 03

of debris and mud flows. Liquefaction flows occur when loosely packed sandy sediments collapse under environmental conditions (e.g. cyclic actions by waves or earthquakes; see section titled Soil Liquefaction. Debris avalanches occur where slides collapse and disintegrate into smaller pieces. They move rapidly without following pre-existing channels or valleys.

The potential impact of submarine mass movements on a structure depends upon the location or orientation of the structure in relation to the movement direction (Figure 5).



Mass Movement Mechanism	Impact on Foundations ⊠		Impact on Pipeline/Flowline/Cable ∘		
	Profile View	Nature of Force on Foundation	Plan View	Orientation of Movement to Installation: Parallel	Orientation of Movement to Installation: Perpendicular
Creep		Rotation About Base		Dragging Rupture Spanning	Dragging Rupture Spanning
Translational Slide		Translation Downdrag at Crest Uplift at Toe		Stretching at Crest Compression at Toe Loss of Support Rupture Spanning	Dragging Loss of Support Rupture Spanning
Rotational Slide		Rotation About Top Downdrag at Crest Uplift at Toe		Stretching at Crest & Toe Loss of Support Rupture Spanning	Dragging Loss of Support Rupture Spanning
Debris Avalanche		Translation/ Rotation +/- Downdrag +/- Uplift		Compression & Stretching Loss of Support Rupture Spanning, Burial	Dragging Loss of Support Rupture Spanning Burial
Debris Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
Liquefied Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
Fluidised Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
High Density Turbidity Current		Loading? Burial? Scour		Burial Loading Scour	Burial Loading Scour
Low Density Turbidity Current		Scour?		Scour	Scour

Figure 5: Potential impacts of submarine mass movements on platform foundation and pipeline (modified after Thomas et al., 2009)

WIND, WAVES, CURRENTS AND TIDES

Periods of extreme weather conditions, such as (tropical) storms, monsoons, peak wind, waves and current regimes, can cause lateral and cyclic actions on the seafloor and any seabed-supported structure. In addition, adverse weather conditions may complicate structure installation activities.

Peak wave and (seafloor/bottom) current regimes can also cause changes in seafloor conditions due to scour and burial (i.e. sediment remobilisation), winnowing of seafloor sediments (i.e. removal of fine/clay-size materials) and development of irregular seafloor topography.

Tidal variation and atmospheric pressure fluctuations as a result of storms are known to change pore pressures conditions in the seabed, potentially creating circumstances leading to soil failure and liquefaction.

Estimation of environmental actions is relatively inaccurate. It normally involves statistical data for a specific geographic region and various procedures for modelling the interaction of a structure and its environment.

MAN-MADE HAZARDS

Human activities and anthropogenic (i.e. man-made/man-induced) features, debris or obstructions can have an adverse effect on an offshore structure.

Seafloor features and objects have been left by human activities since the dawn of mankind. Ship wrecks can form archaeological sites, war graves, enhance ecological diversity and may be restricted areas.

In addition, offshore energy activities, such as drilling, (jack-up) platform installation and decommissioning and resulting footprints may alter seafloor topography and/or potentially alter seabed conditions (e.g. drill spoils, gas charging as a result gas migration along exploration wells).

Commonly encountered man-made hazards include:
- Unexploded ordnance (UXO);
- Existing energy facilities (e.g. fixed platforms, pipelines, manifolds, wellheads, power cables etc.);
- Telecommunication cables;
- Ship wrecks;
- Fallen objects (e.g. shipping containers).

These hazards can complicate structure installation and design if not identified at an early stage.

Activities such as hydrocarbon extraction and deep salt mining can change site conditions, for example causing regional subsidence of the seabed and/or trigger fault activity (Barton et al., 1987; Broughton et al., 1998; Broughton et al., 1997, Gebara et al., 2000). Subsidence can range from millimetres to 10’s of metres. It typically depends on reservoir size, mechanical properties of reservoir and overlying ground, reservoir depth, production rate, pressure drawdown and duration.

REFERENCES

American Petroleum Institute, 2011. *ANSI/API RP 2GEO, First Edition Geotechnical and foundation design considerations: ISO 19901-4:2003 Modified), Petroleum and natural gas industries—Specific requirements for offshore structures, Part 4—Geotechnical and foundation design considerations*. Washington, D.C.: API.

American Petroleum Institute, 2015. *API RP 1111 Design, Construction, Operation, and Maintenance of Offshore Hydrocarbon Pipelines (Limit State Design*. Washington, D.C.: API), 5th Edition.

American Petroleum Institute, 2014. *API RP 2A-WSD, 22nd Edition Planning, designing and constructing fixed offshore platforms - Working Stress Design*. Washington, D.C.: API.

Barton, N., Hårvik, L., Christianson, M., Bandis, S.C., Makurat, A., Chryssanthakis, P. and Vik, G., 1987. Rock mechanics modelling of the Ekofisk reservoir subsidence. *Proceedings of the 27th Symposium on Rock Mechanics, Alabama, 1986*, pp. 267-274.

Broughton, P., Aldridge, T.R. and Nagel, N.B., 1997. Geotechnical aspects of subsidence related to the foundation design of Ekofisk platforms, *Proceedings of the Institution of Civil Engineers. Geotechnical Engineering*, Vol. 125, No. 3, pp. 129-140.

Broughton, P., Aldridge, T.R. and Komaromy, S., 1998. Steel jacket structures for the new Ekofisk complex in the North Sea., *Proceedings of the ICE - Civil Engineering*, Vol. 126, No. 2, pp. 54-64.

Campbell, K.J., Hooper, J.R. and Prior, D.B. 1986. Engineering Implications of Deepwater Geologic and Soil Conditions, Texas-Louisiana Slope. In *Eighteenth Annual Offshore Technology Conference, May 5-8, 1986, Houston, Texas: Proceedings, Vol. 1, OTC Paper 5105,* pp. 225-232.

Clayton, C. and Power, P. 2002. Managing Geotechnical Risk in Deepwater. In Cook, M. et al. Eds., *Offshore Site Investigation and Geotechnics: 'Diversity and Sustainability': Proceedings of an International Conference Held in London, UK, 26-28 November 2002,* London: Society for Underwater Technology, pp. 425-443.

Delisle, G. 2004. The Mud Volcanoes of Pakistan, *Environmental Geology*, Vol. 46, No. 8, pp. 1024-1029.

Delisle, G., Von Rad, U., Andruleit, H., Von Daniels, C.H., Tabrez, A.R. and Inam, A. 2002. Active mud volcanoes on- and offshore eastern Makran, Pakistan. *International Journal of Earth Sciences*, Vol. 91, No. 1, pp. 93-110.

Delisle, G. 2005. Mud Volcanoes of Pakistan — an Overview: a report on three centuries of historic and recent investigations in Pakistan. In Martinelli, G., Panahi, B. Eds. *Mud Volcanoes, Geodynamics and Seismicity*, Dordrecht: Springer, NATO Science Series IV. Earth and Environmental Sciences, Vol. 51, pp. 159-169.

Ding, F. 2008. *Near-surface Sediment Structures at Cold Seeps and their Physical Control on Seepage: a Geophysical and Geological Study in the Southern Gulf of Mexico and at the Frontal Makran Accretionary Prism/Pakistan*. Dissertation, Universität Bremen.

European Committee for Standardization, 2004. *EN 1997-1:2004 Eurocode 7: Geotechnical Design - Part 1: General Rules*. Brussels: CEN. (With Corrigendum EN 1997-1:2004/AC, February 2009).

European Committee for Standardization, 2011. *EN 14161 Petroleum and Natural Gas Industries - Pipeline Transportation Systems ISO 13623:2009 Modified).* Brussels: CEN.

Evans, T.G. 2010. A Systematic Approach to Offshore Engineering for Multiple-project Developments in Geohazardous Areas. In Gourvenec, S. and White, D. Eds., *Frontiers in Offshore Geotechnics II: Proceedings of the 2nd International Symposium on Frontiers in Offshore Geotechnics, Perth, Australia, 8-10 November 2010,* Boca Raton: CRC Press, pp. 3-32.

Faugères, J.-C., Stow, D.A.V., Imbert, P. and Viana, A. 1999. Seismic features diagnostic of contourite drifts. *Marine Geology*, Vol. 162, No. 1, pp. 1-38

Galavazi, M., Moore, R., Lee, M., Brunsden, D. and Austin, B. 2006. Quantifying the Impact of Deepwater Geohazards. In *2006 Offshore Technology Conference, 1-4 May, Houston, OTC Paper 18083.*

Gebara, J.M., Dolan, D., Pawsey, S., Jeanjean, P. and Dahl-Stamnes, K.H.. 2000. Assessment of Offshore Platforms Under Subsidence—Part I: Approach, *Journal of Offshore Mechanics and Arctic Engineering*, Vol. 122, No. 4, pp. 260-266.

Grozic, J.L.H. 2003. *Liquefaction Potential of Gassy Marine Sands*. In Locat, J. and Mienert, J. (Eds.), Submarine Mass Movements and Their Consequences, Kluwer Academic Publishers, Dordrecht, pp. 37-45.

Hampton, M.A., Lee, H.J. and Locat, J. 1996. *Submarine Landslides*. Review of Geophysics, Vol. 34, No. 1, pp. 33-59.

International Organization for Standardization, 2013. *ISO 19900:2013 Petroleum and Natural Gas Industries - General Requirements for Offshore Structures.* Geneva: ISO.

International Organization for Standardization, 2002. *ISO 14688-1:2002 Geotechnical Investigation and Testing - Identification and Classification of Soil - Part 1: Identification and Description*. Geneva: ISO.

International Organization for Standardization, 2004. *ISO 14688-2:2004 Geotechnical Investigation and Testing - Identification and Classification of Soil - Part 2: Principles for a Classification.* Geneva: ISO.

International Organization for Standardization, 2003. *ISO 19901-4:2003 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures - Part 4: Geotechnical and Foundation Design Considerations.* Geneva: ISO.

International Organization for Standardization, 2009*. ISO 13623:2009 Petroleum and Natural Gas Industries - Pipeline Transportation Systems*. Geneva: ISO

International Organization for Standardization, 2016. *ISO 19905-1:2016 Petroleum and Natural Gas Industries – Site Specific Assessment of Mobile Offshore Units - Part 1: Jack-Ups.* Geneva: ISO.

Judd, A.G. and Hovland, M. 2007. *Seabed Fluid Flow: the Impact on Geology, Biology, and the Marine Environment.* Cambridge: Cambridge University Press.

Lee, H.J., Locat, J., Desgagnés, P., Parsons, J.D., McAdoo, B.G., Orange, D.L., Puig, P., Wong, F.L., Dartnell, P. and Boulanger, E. 2007. Submarine Mass Movements on Continental Margins. In Nittrouer, C.A. et al. Eds. *Continental Margin Sedimentation: from Sediment Transport to Sequence Stratigraphy,* Oxford: Blackwell, Special Publication of the International Association of Sedimentologists, No. 37, pp. 213-274.

Locat, J. and Lee, H.L. 2005. *Subaqueous Debris Flows*. In Jakob, M. and Hungr, O. (Eds.), Debris-flow Hazards and Related Phenomena, Springer-Verlag, Berlin, Springer-Praxis Books in Geophysical Sciences, pp. 203-245.

Morelissen, R., Hulscher, S., Knaapen, M.A.F., Németh, A.A. and Bijker, R. 2003. Mathematical Modelling of Sand Wave Migration and the Interaction with Pipelines. *Coastal Engineering*, Vol. 48, No. 3, pp. 197-209.

Mulder, T. and Cochonat, P. 1996. Classification of Offshore Mass Movements. *Journal of Sedimentary Research*, Vol. 66, No. 1, pp. 43-57.

Newton, R.S., Cunningham, R.C. and Schubert, C.E. 1980. Mud Volcanoes and Pockmarks: Seafloor Engineering Hazards or Geological Curiosities? In *Twelfth Annual Offshore Technology Conference, May 5-8, Houston, Texas: Proceedings, Vol. 1, OTC Paper 3729,* p. 425-429.

OGP International Association of Oil and Gas Producers, 2009. *Geohazards from seafloor instability and mass flow.* London: International Association of Oil and Gas Producers, OGP Report No. 425.

OGP International Association of Oil and Gas Producers, 2013. *Guidelines for the Conduct of Offshore Drilling Hazard Site Surveys.* London: International Association of Oil and Gas Producers, OGP Report No. 373-18-1.

Osborne, J.J., Teh, K.L., Houlsby, G.T., Cassidy, M.J., Bienen, B. and Leung, C.F. 2011. *InSafeJIP: Improved Guidelines for the Prediction of Geotechnical Performance of Spudcan Foundations during Installation and Removal of Jack-up Units: Joint Industry-funded Project.* Woking: RPS Energy, Report No. EOG0574-Rev1c.

ISSUE 03

Parker, E.J., Traverso, C., Moore, R., Evans, T. and Usher, N. 2008. Evaluation of Landslide Impact on Deepwater Submarine Pipelines. *OTC.08: Proceedings 2008 Offshore Technology Conference, 8 May, Houston, Texas, USA, OTC Paper 19459.*

Power, P., Galavazi, M. and Wood, G. 2005. Geohazards Need Not Be: Redefining Project Risk. In *Offshore Technology Conference, 2-5 May 2005, Houston, Texas, U.S.A., OTC Paper 17634.*

Rastogi, A., Deka, B., Budhiraja, I.L. and Agarwal, G.C. 1999. Possibility of Large Deposits of Gas Hydrates in Deeper Waters of India. *Marine Georesources & Geotechnology*, Vol. 17, No. 1, pp. 49-63.

RenewableUK, 2013. *Guidelines for the Selection and Operation of Jack-ups in the Marine Renewable Energy Industry: version 2: full technical and regulatory update.* London: RenewableUK.

Rogers, K.G. and Goodbred, S.L. 2010. Mass Failures Associated with the Passage of a Large Tropical Cyclone over the Swatch of No Ground Submarine Canyon Bay of Bengal*. Geology,* Vol. 38, No. 11, pp. 1051-1054.

Skipp, B.O. and Mallard, D.J. 1995. Site Investigations with Earthquakes in Mind. In Elnashai, A.S. Ed. *European Seismic Design Practice: Research and Application: Proceedings of the Fifth SECED Conference on European Seismic Design Practice, Chester, United Kingdom, 26-27 October 1995,* Rotterdam: Balkema, pp. 151-161.

Snead, R.E. 1972. Mud Volcanoes of the Makran Coast. *Explorers Journal*, Vol. 50, No. 1, pp. 22-28.

Society of Naval Architects and Marine Engineers 2008. *Recommended Practice for Site Specific Assessment of Mobile Jack-up Units.* First Edition – May 1994 (Revision 3 – August 2008). Jersey City: SNAME, Technical & Research Bulletin, 5-5A.

Soulsby, R. 1997. *Dynamics of marine sands: A manual for practical applications.*London: Thomas Telford.

Sumer, B.M. and Fredsøe, J. 2002. *The mechanics of scour in the marine environment*. World Scientific, Singapore, Advanced Series on Ocean Engineering; Vol. 17.

Thomas, S., Hooper, J.R. and Clare, M. 2009. Constraining Geohazards to the Past: Impact Assessment of Submarine Mass Movements on Seabed Developments. Mosher, D.C. et al. Eds., *Submarine Mass Movements and their Consequences: 4th International Symposium,* Dordrecht: Springer, Advances in Natural and Technological Hazards Research, Vol. 28, pp. 387-398.

Von Rad, U., Berner, U., Delisle, G., Doose-Rolinksi, H., Fechner, N., Linke, P., Lückge, A., Roeser, H.A., Schmaljohann, R., Wiedicke, M. and SONNE 122/130 Scientific Parties 2000. Gas and Fluid Venting at the Makran Accretionary Wedge off Pakistan, *Geo-Marine Letters*, Vol. 20, No. 1, pp. 10-19.

Wells, D.L. and Coppersmith, K.J. 1994. New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement. *Bulletin of the Seismological Society of America,* Vol. 84, No. 4, pp.974-1002.

Whitehouse, R. 1998. *Scour at Marine Structures: a Manual for Practical Applications.* London: Thomas Telford.

Yuan, F., Wang, L., Guo, Z. and Shi, R. 2012. A Refined Analytical Model for Landslide or Debris Flow Impact on Pipelines. Part I: Surface Pipelines, *Applied Ocean Research*, Vol. 35, pp. 95-104.

APPROACH

A geotechnical design situation or a re-assessment of an existing structure requires geotechnical analysis, including evaluation of hazards and verification of relevant limit states. Geotechnical analysis follows design philosophies included in standards and codes of practice, where available. All consider that the resistance (or capacity) of a geotechnical system must be greater than the actions (demands or loads) on the system for an acceptable or required level of safety or reliability (ISO 2394, 2015).

The approach for geotechnical analysis typically includes these steps:
- selection of procedures and models for geotechnical analysis
- processing and integration of geotechnical information, e.g. by preparation of geotechnical logs, cross sections, geographical information system GIS and/or 3D ground model
- site characterisation including hazard identification
- selection of geotechnical parameter values for calculation models
- application of calculation models and evaluation of results.

The approach for geotechnical analysis includes assumptions and premises. One premise is that the Client's activities are state-of-the-practice in all areas, including planning, engineering, construction, operation and maintenance of a geotechnical system or structure.

HAZARD EVALUATION

Hazards are situations or events with potential to cause damage (ISO 2000, 2013). Hazard evaluation typically includes classification, estimation of probability of occurrence and measures for countering the hazard. Examples of hazards are abnormal environmental events, accidental events, geohazards and man-made site hazards. Note that event probability differs from risk, where risk is defined as the product of probability and consequence.

In many geotechnical situations, hazard evaluation will not be complete and exact. It will be necessary to draw on so-called tacit expert knowledge. This means senior expertise, with access to geotechnical knowledge and experience. Judgement and opinion are inevitable and a senior expert or a team of senior experts is more likely to arrive at a correct understanding and an appropriate way forward. Judgement is qualitative and subjective. Table 1 shows probability expressions intended for a context of approximate and subjective probability of the occurrence of a hazardous event or phenomena during a defined exposure period (Peuchen et al., 2015).

Table 1. Expressions for approximate and subjective probability

Term	Verbal descriptor	Approximate probability for exposure period
Negligible	unlikely, although the possibility cannot be ruled out completely	0 to 0.01
Low	not probable, although uncertain	0.01 to 0.1
High	credible, possibility can be described with reasonable confidence by known physical conditions or processes	0.1 to 1

Measures for countering a hazard include source elimination, avoidance, implementation of a barrier, minimising consequences and design for the hazard.

LIMIT STATES

Limit states may be grouped into Ultimate Limit States (ULS, for example structure stability), Serviceability Limit States (SLS, for example for avoiding excessive settlement), Fatigue Limit States (FLS) and Accidental Limit States (ALS). Verification of a limit state usually involves one or more of the following approaches:
- calculation models
- prescriptive measures
- experimental models and load tests
- observational method.

Features of a calculation model include:
- method of analysis typically including simplifications and modification of the results where necessary to improve accuracy or to allow for uncertainty and systematic error
- actions, such as (a sequence of) imposed loads or imposed displacements
- geometrical data, such as the shape of a geotechnical structure, geometry of the ground surface, water levels and interfaces between ground strata
- characteristic values of geotechnical parameters of ground (soil, rock, pore fluid, pore gas) and other materials
- limiting values of, for example, deformations and vibrations
- partial factors or safety factors.

The common analytical models rely on semi-empirical and direct methods of analysis.

Prescriptive measures generally involve (1) conventional and conservative details in the design and (2) attention to specification and control of materials, workmanship, protection and maintenance procedures. Their use is often applicable where calculation models are not available or not necessary. Examples are prescriptive measures for ensuring durability against chemical attack or frost action.

Experimental models and load tests can help to justify a design approach. Important considerations for evaluation of the results include differences in ground conditions, time effects and scale effects.

Prediction of geotechnical behaviour is often difficult. The observational method allows carefully planned monitoring during construction and includes planned contingency measures where necessary. Assessment of the monitoring results takes place at appropriate stages.

DESIGN PHILOSOPHIES

Design philosophies typically incorporate geotechnical calculation models and corresponding (partial) factors. These partial factors or safety factors may vary depending on the specific design scenario.

Design philosophies for the ULS may be grouped as follows:
1. Working Stress Design (WSD) or Allowable Stress Design (ASD).
2. Partial Factor Design (PFD) or Limit State Design (LSD).
   a. Factored material properties.
   b. Factored resistance.

The WSD method uses global safety factors applied to characteristic values (or ultimate values) of resistance.

The PFD methods use partial action factors and partial factors applied to resistance. The partial action factors are applied to characteristic or representative values of actions. This results in design values for actions. The factored material properties and factored resistance methods differ by their calculation of resistance. The method for factored material properties applies partial material factors to characteristic values of material properties such as undrained shear strength of soil. The factored values are then used in the calculation model to obtain a design value for resistance (factored resistance). The factored resistance method uses characteristic values of material properties in the calculation model and then applies a partial resistance factor to obtain a design value for resistance. An additional factor γ_d can be considered to account for model uncertainty or other uncertainties not covered by other partial factors (ISO, 2013).

API Recommended Practice RP 2A-WSD (API, 2014) is an example of the WSD approach. Eurocode 7 Geotechnical Design (CEN, 2004; 2007), ISO 19900 (2013), ISO 19901-4 (2003) and API RP 2GEO Geotechnical and Foundation Design Considerations (API, 2011 and 2014) provide design principles according to the PFD approaches.

Design philosophies for the ALS, SLS and FLS are similar. Global safety factors and partial factors will differ from the ULS.

GEOTECHNICAL PARAMETER VALUES

DESIGN PROCESS

Assignment of geotechnical parameter values or soil property values is according to the following steps:
1. Site characterisation and stratigraphic schematisation.
2. Evaluation of derived values of geotechnical parameters.
3. Selection of characteristic values of geotechnical parameters and application in a calculation model.

The selection of characteristic values of geotechnical parameters takes place within the context of a calculation model and thus includes consideration of limit states, actions, geometry, limiting values and partial factors or safety factors. Divorcing the selection of characteristic values from the actual use and evaluation of a calculation model may lead to errors.

STRATIGRAPHIC SCHEMATISATION

General site characterisation is necessary before selection of geometrical data for the ground and before evaluation of the results of specific tests and observations. Such site characterisation comprises a general assessment of the character and basic constituents of the ground (soil and rock classification) and their possible change in time.

Typical parameters for soil classification include particle size distribution, water content, carbonate content, Atterberg limits, unit weight, relative density and undrained shear strength. Typical parameters for rock classification include mineralogy, water content, unit weight and uni-axial compressive strength.

Stratigraphic schematisation depends on the nature of the actions, geometrical quantities of the structure that interacts with the ground, volume of ground that represents the domain of influence with respect to the limits state, spatial ground variability, simplification of ground conditions, e.g. undrained versus drained foundation response.

Two competing factors apply to spatial ground variability: (1) the spatial averaging of properties over a potential failure surface, which reduces the coefficient of variation of property values (i.e. with respect to that for the location under consideration) and (2) the tendency for a failure surface to follow the path of least resistance.

Stratigraphic schematisation can include evaluation of:
- basic parameters such as undrained shear strength and relative density on the basis of derived values of geotechnical parameters (refer following section)
- geological and hydro-geological setting
- results of a geophysical survey
- hazards such as potential instability of the ground
- water levels
- aggressiveness of ground and ground water.

DERIVED VALUES OF GEOTECHNICAL PARAMETERS

A derived value of a geotechnical parameter or coefficient is obtained from test results by theory, correlation or empiricism. In situ test and laboratory test measurements and other relevant data provide a basis for obtaining derived values of geotechnical parameters.

Laboratory test standards often specify procedures for obtaining derived values, in particular where it is possible to obtain a derived value by means a of a conversion model or theory. Such derived values are thus part of the laboratory test report. An example is the unconsolidated undrained triaxial compression test. Normalised load and displacement data are the basic measured values. The measured values and the use of theory allow the calculation of a derived value of undrained shear strength by consideration of principal stress conditions and a theoretical deformation model.

Standards for in situ tests usually require reporting of (normalised) measured values only. Examples of measured values are cone resistance and sleeve friction for a Cone Penetration Test (CPT). Measured values can serve as input for some calculation models that rely on empirical relationships. An example is the use of CPT cone resistance for the calculation of axial pile resistance. A more common approach is to obtain derived values of geotechnical parameters from in situ tests on the basis of empiricism or (simplified) theory or a combination thereof. Evaluation of derived values of geotechnical parameters will usually comprise undrained shear strength (c_u) and relative density (D_r) according to a single interpretation method, where appropriate.

Many empirical correlations and theoretical interpretation models are available for obtaining specific derived values of geotechnical parameters from the results of laboratory and in situ tests. Evaluation of various sets of derived values by engineering judgement or statistical methods can be considered, whereby one method is selected as reference.

Measured values and derived values may be represented by low estimate, best estimate and high estimate values. In statistical terms, a best estimate value aims to represent a mean value of a geotechnical parameter for a stratum or multiple soil layers. Low and high estimates aim for the quantile associated with the 5% fractile. Comments are as follows:

- Low, best and high estimates usually consider a reference method or procedure, if values from multiple methods or procedures are combined. This is because a test result or a derived value can depend on the method(s) selected to obtain the parameter value. For example, a value of undrained shear strength derived from a triaxial test can depend on the sampling method, sample handling practice, laboratory test procedure and whether undrained shear strength is derived from maximum deviator stress or maximum principal stress ratio.
- Low, best and high estimates can include judgement and opinion, particularly for a limited quantity or absence of test results and derived values. This implies that outliers may be ignored and that a bias may be introduced relative to the available data. Judgement and opinion consider physically credible values, comparison of data with results from other tests and *a priori* knowledge such as geological setting and comparable experience.
- A wide spread of data can indicate spatial variability of soil. This means that averaging of test results and derived values can obscure a weaker or stronger zone.
- A calculation model can require specific schematisation of soil stratigraphy and model-specific selection of parameter values. This is not covered by low, best and high estimates.

CHARACTERISTIC VALUES OF GEOTECHNICAL PARAMETERS

A characteristic value of a geotechnical parameter represents a ***cautious estimate*** for the value affecting the occurrence of a limit state (CEN, 2004). The selection of a characteristic value takes account of possible differences between derived values of geotechnical parameters and geotechnical parameters representative of the behaviour of a geotechnical structure. Reasons for differences can include non-homogeneity of the ground, extent of the zone governing a particular limit state, uncertainties in geometrical data and analytical model, time effects, brittle or ductile response of the ground, influence of construction activities.

Characteristic values may be lower values, which are less than the most probable value, or upper values, which are greater. Each calculation requires the most unfavourable combination of lower and/or upper values for independent geotechnical parameters.

Statistical methods may be appropriate for selection of a characteristic value (Hicks, 2013; Baecher and Christian, 2003). Usually, they should allow for incorporation of a-priori knowledge of comparable experience with geotechnical parameters, for example by Bayesian methods, as necessary. Selection of a statistical characteristic value is typically such that the calculated probability of a worse value governing the occurrence of a limit state is not greater than 5%. Variance reduction methods may be applied where appropriate.

In principle, spatial ground variability affects:

- The mean (X_m), Standard Deviation (SD) and probability density function (pdf) of the ground property for the location under consideration, including any depth trend.
- The scale of fluctuation (θ) of the ground property, which is the distance over which the property values are significantly correlated; the scale of fluctuation in the (near) horizontal plane is often much larger than in the vertical direction, i.e. $\theta_h >> \theta_v$, for example due to the process of deposition.
- The limit state under consideration, particularly relating to the geometrical quantities of the structure that interacts with the ground, the nature of the applied actions and the volume of ground that represents the domain of influence with respect to the limit state.

The pdf required for the characteristic value should take account of the spatial variability of ground property values and the limit state under consideration, and thus may differ considerably from the underlying pdf for the location under consideration (Figure 1). If the domain of influence is represented by the dimension D, the characteristic value will be a function of the ratio θ/D and will generally lie within the following limits:

- For relatively large values of θ/D, there may be considerable uncertainty regarding the property value governing the structure response. Specifically, although the occurrence of the limit state will generally be governed by the "local" mean, there will be uncertainty about what that mean actually is. The characteristic value may then be represented by the 5 percentile of the underlying pdf. (Figure 1a)
- For intermediate values of θ/D, the characteristic value may be estimated from a pdf with a reduced variance to account for averaging of properties. However, account should also be taken of any apparent reduction in the property mean due to the tendency for failure to follow the path of least resistance. (Figure 1b)
- For small values of θ/D, there is considerable averaging of property values over potential failure surfaces and the response of the structure may be reasonably represented by a cautious estimate of the mean over the failure surface. For the assumption of a normal distribution of X, this is equivalent to a cautious estimate of X_m, the mean of the underlying distribution. (Figure 1c).

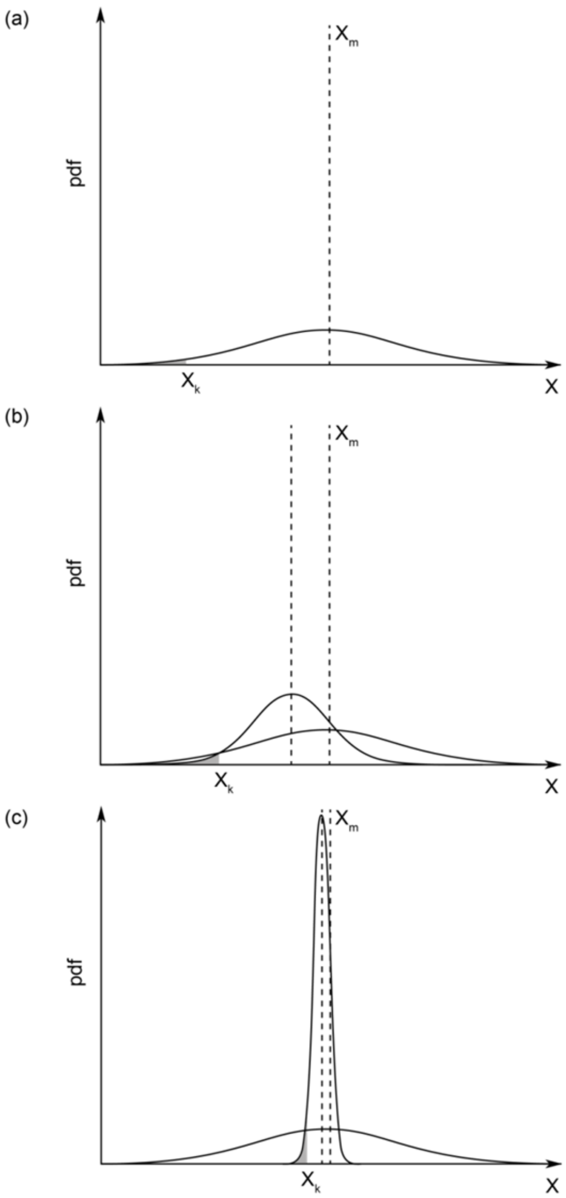


Figure 1. Estimation of characteristic value and pdf (after Hicks, 2013): (a) X_k based on underlying pdf (for large θ/D); (b) X_k based on modified pdf (for intermediate θ/D); (c) X_k based on modified pdf (for small θ/D)

REFERENCES

American Petroleum Institute, 2014. *API RP 2A-WSD, 22nd Edition Planning, designing and constructing fixed offshore platforms - Working Stress Design*. Washington, D.C.: API.

American Petroleum Institute, 2011. *ANSI/API RP 2GEO, First Edition Geotechnical and foundation design considerations: ISO 19901-4:2003 Modified), Petroleum and natural gas industries—Specific requirements for offshore structures, Part 4—Geotechnical and foundation design considerations*. Washington, D.C.: API.

Baecher, G.B., and Christian, J.T. 2003. *Reliability and Statistics in Geotechnical Engineering*. London and New York: Wiley.

European Committee for Standardization, 2004. *EN 1997-1:2004 Eurocode 7: Geotechnical Design - Part 1: General Rules*. Brussels: CEN. (With Corrigendum EN 1997-1:2004/AC, February 2009).

European Committee for Standardization, 2007. *EN 1997-2:2007 Eurocode 7 - Geotechnical Design – Part 2: Ground Investigation and Testing.* Brussels: CEN. (With Corrigendum EN 1997-2:2007/AC, June 2010).

Hicks, M.A. 2013. An Explanation of Characteristic Values of Soil Properties in Eurocode 7. In P. Arnold, G.A. Fenton, M.A. Hicks, T. Schweckendiek and B. Simpson Eds. *Modern Geotechnical Design Codes of Practice: Implementation, Application and Practice.* IOS Press, pp. 36-45.

International Organization for Standardization, 2015. *ISO 2394:2015 General Principles on Reliability for Structures.* Geneva: ISO.

International Organization for Standardization, 2003. *ISO 19901-4:2003 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures - Part 4: Geotechnical and Foundation Design Considerations.* Geneva: ISO.

International Organization for Standardization, 2013. *ISO 19900:2013 Petroleum and Natural Gas Industries - General Requirements for Offshore Structures.* Geneva: ISO.

International Organization for Standardization, 2000. *ISO 17776:2000 Petroleum and Natural Gas Industries – Offshore Production Installations – Guidelines on Tools and Techniques for Hazard Identification and Risk Assessment*. Geneva: ISO.

Peuchen, L.J., Meijninger, B.M.L. and Drummen, T.W.A. 2015. Reassessment of geotechnical conditions after an offshore well incident. In *Proceedings of the XVI ECSMGE: Geotechnical Engineering for Infrastructure and Development,* ICE Publishing, 2015, pp. 195-206.

ISSUE 14

Symbol	Unit	Quantity
I - GENERAL		
L	m	Length
B	m	Width
D	m	Diameter
d	m	Depth
h	m	Height or thickness
z	m	Penetration or depth below reference level (usually ground surface)
A	m^2	Area
V	m^3	Volume
W	kN	Weight
t	s	Time
v	m/s	Velocity
a	m/s^2	Acceleration
g	m/s^2	Acceleration due to gravity ($g = 9.81\ m/s^2$)
m	kg	Mass
ρ	kg/m^3	Density
π	-	Mathematical constant (= 3.14159)
e	-	Base of natural logarithm (= 2.71828)
ln	-	Natural logarithm
log	-	Logarithm base 10
II - STRESS AND STRAIN		
P_a	kPa	Atmospheric pressure
u	MPa	Pore water pressure
u_o	MPa	Hydrostatic pore pressure relative to seafloor or phreatic surface
σ	kPa	Total stress
σ'	kPa	Effective stress
τ	kPa	Shear stress
t	kPa	Shear stress in s'-t space [= $(\sigma'_1 - \sigma'_3)/2$] or [= $(\sigma_1 - \sigma_3)/2$]
$\sigma_1,\sigma_2,\sigma_3$	kPa	Principal stresses
σ'_{ho}	kPa	Effective in situ horizontal stress
σ_{vo}	kPa	Total in situ vertical stress relative to ground surface or phreatic surface
σ'_{vo}	kPa	Effective in situ vertical stress (or p'_o)
σ'_h	kPa	Effective horizontal stress
σ'_v	kPa	Effective vertical stress
r_u	-	Pore pressure ratio [= u/σ_{vo}]
p'	kPa	Mean effective stress [= $(\sigma'_1 + \sigma'_2 + \sigma'_3)/3$]
q	kPa	Principal deviator stress [= $\sigma'_1 - \sigma'_3$] or [= $\sigma_1 - \sigma_3$]
s'	kPa	Mean effective stress in s'-t space [= $(\sigma'_1 + \sigma'_3)/2$]
ε	-	Linear strain
$\varepsilon_1,\varepsilon_2,\varepsilon_3$	-	Principal strains
ε_v	-	Volumetric strain
γ	-	Shear strain
ν	-	Poisson's ratio
ν_u	-	Poisson's ratio for undrained stress change
ν_d	-	Poisson's ratio for drained stress change
E	MPa	Modulus of linear deformation (Young's modulus)
E_u	MPa	Modulus of linear deformation (Young's modulus for undrained stress change)
E_d	MPa	Modulus of linear deformation (Young's modulus for drained stress change)
G	MPa	Modulus of shear deformation (shear modulus)
G_{max}	MPa	Shear modulus at small strain
I_r	-	Rigidity index [= G/τ_{max} or G/s_u]
K	MPa	Modulus of compressibility (bulk modulus)
M	MPa	Constrained modulus [= $1/m_v$]
μ	-	Coefficient of friction
η	kPa.s	Coefficient of viscosity

Symbol	Unit	Quantity

III - PHYSICAL CHARACTERISTICS OF GROUND

(a) Density and Unit weights

γ	kN/m^3	Unit weight of ground (or bulk unit weight or total unit weight)
γ_d	kN/m^3	Unit weight of dry ground
γ_s	kN/m^3	Unit weight of solid particles
γ_w	kN/m^3	Unit weight of water
γ_{pf}	kN/m^3	Unit weight of pore fluid
γ_{dmin}	kN/m^3	Minimum index (dry) unit weight
γ_{dmax}	kN/m^3	Maximum index (dry) unit weight
γ' or γ_{sub}	kN/m^3	Unit weight of submerged ground
ρ	Mg/m^3 [= t/m^3]	Density of ground
ρ_d	Mg/m^3 [= t/m^3]	Density of dry ground
ρ_s	Mg/m^3 [= t/m^3]	Density of solid particles
ρ_w	Mg/m^3 [= t/m^3]	Density of water
D_r	-, %	Relative density [= $I_D = \gamma_{dmax}\,(\gamma_d-\gamma_{dmin})/\gamma_d(\gamma_{dmax}-\gamma_{dmin}) = (e_{max}-e)/(e_{max}-e_{min})$]
v	-	Specific volume [= 1+e]
e	-	Void ratio
e_o	-	Initial void ratio
e_{max}	-	Maximum index void ratio
e_{min}	-	Minimum index void ratio
I_D	-, %	Density index [= D_r]
R_D	-, %	Dry density ratio [= γ_d/γ_{dmax}]
n	-, %	Porosity
w	%	Water content
S_r	%	Degree of saturation
r	-, g/kg	Salinity of pore fluid [= ratio of mass of salt to mass of pore fluid]
R	g/l	Salinity of fluid [= ratio of mass of salt to volume of distilled water]
s	g/l	Salinity of fluid [= ratio of mass of salt to volume of fluid]
S	g/kg	Salinity of seawater [= ratio of mass of salt to mass of seawater]

(b) Consistency

w_L	%	Liquid limit
w_P	%	Plastic limit
I_P	%	Plasticity index [= $w_L - w_P$]
I_L	%	Liquidity index [= $(w - w_P)/(w_L - w_P)$]
I_C	%	Consistency index [= $(w_L - w)/(w_L - w_P)$]
A	-, %	Activity [= ratio of plasticity index to percentage by weight of clay-size particles]

(c) Particle size

D	mm	Particle diameter
D_n	mm	n percent diameter [n% < D]
C_u	-	Uniformity coefficient [= D_{60}/D_{10}]
C_c	-	Curvature coefficient [= $(D_{30})^2/D_{10}D_{60}$]

(d) Dynamic Properties

v_p	m/s	P-wave velocity (compression wave velocity)
v_s	m/s	S-wave velocity (shear wave velocity)
v_{s1}	m/s	S-wave velocity normalised to 100 kPa in situ vertical stress
D	-, %	Damping ratio of ground

ISSUE 44

Symbol	Unit	Quantity
(e) Hydraulic properties		
k	m/s	Coefficient of permeability
k_v	m/s	Coefficient of vertical permeability
k_h	m/s	Coefficient of horizontal permeability
i	-	Hydraulic gradient
(f) Thermal and Electrical properties		
T	K, °C	Temperature
k	W/(m·K)	Thermal conductivity
a_L	1/°C	Thermal expansion coefficient (linear)
α	m^2/s	Thermal diffusion coefficient
ρ	Ω.m	Electrical resistivity
K	S/m	Electrical conductivity
(g) Magnetic properties		
B	T	Magnetic flux density (or magnetic induction)
(h) Radioactive properties		
γ	CPS	Natural gamma ray

IV - MECHANICAL CHARACTERISTICS OF GROUND

Symbol	Unit	Quantity
(a) Cone Penetration Test (CPT)		
q_c	MPa	Cone resistance
q_{c1}	MPa	Cone resistance normalised to 100 kPa effective in situ vertical stress
f_s	MPa	Sleeve friction
f_t	MPa	Sleeve friction corrected for pore pressures acting on the end areas of the friction sleeve
R_f	%	Ratio of sleeve friction to cone resistance
R_{ft}	%	Ratio of sleeve friction to corrected cone resistance (f_s/q_t or f_t/q_t)
u_1	MPa	Pore pressure at the face of the cone
u_2	MPa	Pore pressure at the cylindrical extension above the base of the cone or in the gap between the friction sleeve and the cone
u_2*	MPa	Pore pressure u_2, but derived rather than measured
u_3	MPa	Pore pressure immediately above the friction sleeve or in the gap above the friction sleeve
K	-	Adjustment factor for ratio of pore pressure at u_1 to u_2 location
q_n	MPa	Net cone resistance
q_t	MPa	Corrected cone resistance (or total cone resistance)
B_q	-	Pore pressure ratio
Q_t	-	Normalized cone resistance [= q_n/σ'_{vo}]
Q_{tn}	-	Normalized cone resistance with variable stress exponent
F_r	%	Normalized friction ratio [= f_t/q_n]
N_c	-	Cone factor between q_c and s_u
N_k	-	Cone factor between q_n and s_u
I_c	-	Soil behaviour type index (for Q_{tn} and F_r)
I_{SBT}	-	Soil behaviour type index (for q_c and R_f)
(b) Standard Penetration Test (SPT)		
N	Blows/0.3 m	SPT blowcount
N_{60}	Blows/0.3 m	SPT blowcount normalised to 60% energy
$N_{1,60}$	Blows/0.3 m	SPT blowcount normalised to 60% energy and to 100 kPa effective in situ vertical stress

Symbol	Unit	Quantity
(c) Strength of soil		
s_u	kPa	Undrained shear strength (or c_u)
s_u/σ'_{vo}	-	Undrained strength ratio
κ	kPa/m	Rate of increase of undrained shear strength with depth (linear)
c'	kPa	Effective cohesion intercept
φ'	°(deg)	Effective angle of internal friction
φ'_{cv}	°(deg)	Effective angle of internal friction at large strain
ε_{50}	%	Strain at 50% of peak deviator stress (or ε_c)
E_{50}	MPa	Young's modulus at 50% of peak deviator stress
$s_{u;r}$	kPa	Undrained shear strength of remoulded soil
$s_{u;ar}$	kPa	Undrained shear strength of aged remoulded soil
s_R	kPa	Undrained residual shear strength
S_t	-	Sensitivity [= $s_u/s_{u;r}$ or s_u/s_R]
T_x	-	Thixotropy strength ratio [$T_x(t) = s_{u;ar}(t)/s_{u;r}$]
σ'_c	kPa	Effective consolidation pressure
M	-	Gradient of critical state line when projected onto a constant volume plane
A	-	Pore pressure coefficient for anisotropic pressure increment
B	-	Pore pressure coefficient for isotropic pressure increment
(d) Strength of rock		
$I_{s(50)}$	MPa	Point load strength index
σ_c	MPa	Uni-axial compressive strength
(e) Consolidation (one dimensional)		
σ'_p	kPa	Effective preconsolidation pressure (or effective vertical yield stress in situ)
σ^*_{ve}	kPa	Effective vertical stress on ICL at e_0
σ'_{vy}	kPa	Effective vertical yield stress in situ (or effective preconsolidation pressure)
C_c	-	Compression index
C^*_c	-	Intrinsic compression index [= $e^*_{100} - e^*_{1000}$]
C_s	-	Swelling index (or re-compression)
CR	-	Primary compression ratio [= $C_c/(1+e_0)$]
RR	-	Recompression ratio [= $C_s/(1+e_0)$]
e_0	-	Void ratio at σ'_{vo}
e_L	-	Void ratio at liquid limit w_L
e^*_{100}	-	Void ratio at σ'_v = 100 kPa during one-dimensional intrinsic compression
e^*_{1000}	-	Void ratio at σ'_v = 1000 kPa during one-dimensional intrinsic compression
C_α	-	Coefficient of secondary compression (primary compression)
$C_{\alpha s}$	-	Coefficient of secondary compression (swelling/re-compression)
c_v	m^2/s	Coefficient of consolidation
H	m	Drainage path length
ICL	-	Intrinsic compression line (Burland 1990)
I_v	-	Void index [= $(e_0 - e^*_{100})/C^*_c$]
m_v	m^2/MN	Coefficient of volume compressibility
M	MPa	Constrained modulus [= $1/m_v$]
p	kPa	Vertical pressure
OCR	-	Overconsolidation ratio [= σ'_p/σ'_{vo}]
SCC	-	Sedimentation compression curve
SCL	-	Sedimentation compression line (Burland 1990)
S_σ	-	Stress sensitivity [= $\sigma'_{vy}/\sigma^*_{ve}$]
YSR	-	Yield stress ratio [= $\sigma'_{vy}/\sigma'_{vo}$]

ISSUE 44

V - GEOTECHNICAL DESIGN

(a) Partial factors

γ_d	-	Factor related to model uncertainty or other circumstances
γ_f	-	Partial action factor (load factor)
γ_m	-	Partial material factor (partial safety factor)
γ_R	-	Partial resistance factor (partial safety factor)

(b) Seismicity

a_g	m/s^2	Effective peak ground acceleration (design ground acceleration)
d_g	m	Peak ground displacement
α	-	Acceleration ratio [= a_g/g]
τ_c	kPa	Seismic shear stress

(c) Compaction

ρ_{dmax}	Mg/m^3 [= t/m^3]	Maximum dry density
ρ_{max}	Mg/m^3 [= t/m^3]	Maximum density
w_{opt}	%	Optimum moisture content

(d) Earth pressure

δ	°(deg)	Angle of interface friction (between ground and foundation)
K	-	Coefficient of lateral earth pressure
K_a	-	Coefficient of active earth pressure
K_{ac}	-	Coefficient of active earth pressure for total stress analysis
K_p	-	Coefficient of passive earth pressure
K_{pc}	-	Coefficient of passive earth pressure for total stress analysis
K_o	-	Coefficient of earth pressure at rest
K_{onc}	-	K_o for normally consolidated soil
K_{ooc}	-	K_o for overconsolidated soil

(e) Foundations

A	m^2	Total foundation area
A'	m^2	Effective foundation area
B'	m	Effective width of foundation
E_s	MN/m^3	Modulus of subgrade reaction
k	MPa/m	Rate of change of modulus of subgrade reaction E_s with depth z
L'	m	Effective length of foundation
H	MN	Horizontal external force or action
V	MN	Vertical external force or action
M	MN.m	External moment
T	MN.m	External torsion moment
Q	MN	Total vertical resistance of a foundation/pile
Q_p	MN	End-bearing of pile
Q_s	MN	Shaft resistance of pile
q_p	MPa	Unit end-bearing
q_{lim}	MPa	Limit unit end-bearing
f	kPa	Unit skin friction (or q_s)
f_{lim}	kPa	Limit unit skin friction
p	MN/m	Lateral resistance per unit length of pile
p_{lim}	MN/m	Limit lateral resistance per unit length of pile
s	m	Settlement
t	MN/m	Skin friction per unit length of pile
y	mm	Lateral pile deflection
z	mm	Axial pile displacement
α	-	Adhesion factor between ground and foundation (= f/s_u)
β	-	Adhesion factor between ground and foundation (= f/σ'_v or f/σ'_{vo})

Symbol	Unit	Quantity
δ	°(deg)	Angle of interface friction (between ground and foundation)
δ_{cv}	°(deg)	Constant volume or critical-state angle of interface friction (between ground and foundation)
N_c,N_q,N_γ	-	Bearing capacity factors
K_c,K_q,K_γ	-	Bearing capacity correction factors for inclined forces or actions, foundation shape and depth of embedment
i_c,i_q,i_γ	-	Bearing capacity correction factors for external force inclined from vertical shape
s_c,s_q,s_γ	-	Bearing capacity correction factors for foundation shape
d_c,d_q,d_γ	-	Bearing capacity correction factors for foundation embedment

Signs:
- A "prime" applies to effective stress.
- A "bar" above a symbol relates to average properties.
- A "dot" above a symbol denotes derivative with respect to time.
- The prefix "Δ" denotes an increment or a change.
- A “star” after a symbol denotes value corrected for pore fluid salinity.

BIBLIOGRAPHY

Burland, J.B. 1990. On the Compressibility and Shear Strength of Natural Clays. *Géotechnique,* Vol. 40, No. 3, pp. 329-378.

Det Norske Veritas, 1992. *Classification Notes No. 30.4 Foundations*. Oslo: DNV.

European Committee for Standardization, 2004. *EN 1997-1:2004 Eurocode 7: Geotechnical Design - Part 1: General Rules*. Brussels: CEN. (With Corrigendum EN 1997-1:2004/AC, February 2009).

European Committee for Standardization, 2007. *EN 1997-2:2007 Eurocode 7 - Geotechnical Design – Part 2: Ground Investigation and Testing.* Brussels: CEN. (With Corrigendum EN 1997-2:2007/AC, June 2010).

International Organization for Standardization, 2013. *ISO 19900:2013 Petroleum and Natural Gas Industries - General Requirements for Offshore Structures.* Geneva: ISO.

International Organization for Standardization, 2003. *ISO 19901-4:2003 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures - Part 4: Geotechnical and Foundation Design Considerations.* Geneva: ISO.

International Organization for Standardization, 2004. *ISO 14688-2:2004 Geotechnical Investigation and Testing - Identification and Classification of Soil - Part 2: Principles for a Classification.* Geneva: ISO.

ISSMFE Subcommittee on Symbols, Units, Definitions 1978. List of Symbols, Units and Definitions. In *Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, 1977, Tokyo, Vol. 3,* Tokyo: Japanese Society of Soil Mechanics and Foundation Engineering, pp. 156-170.

ISRM Commission on Terminology, Symbols and Graphic Representation, 1970. List of Symbols.

Noorany, I. 1984. Phase Relations in Marine Soils. *Journal of Geotechnical Engineering*, Vol. 110, No. 4, pp. 539-543.

ISSUE 44