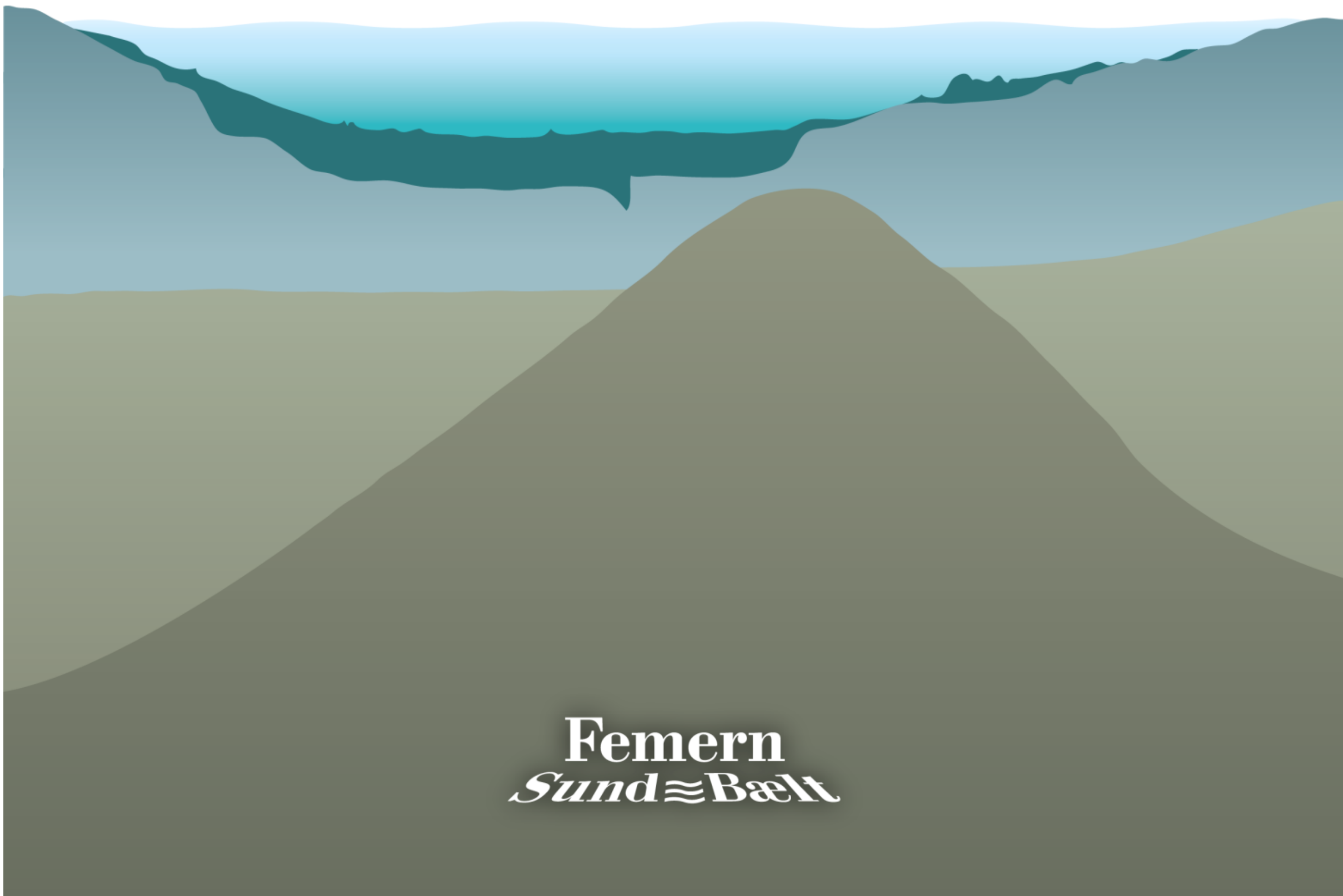
GDR 00.1-001

May 2011

Ground Investigation Report



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Femern A/S
Vester Søgade 10
1601 København V

Tel.: +45 3341 6300
Fax.: +45 3341 6301
www.fehmarnlink.com
CVR no. 28 98 65 64

Prepared by

Rambøll Arup Joint Venture
c/o
Rambøll Danmark A/S
Hannemanns Allé 53
DK-2300 Copenhagen S
Danmark

Phone +45 51611000
Rambøll Arup Joint Venture
Danish reg. no: CVR-NR 31749077
Member of FRI

This report is based on the geological/geotechnical knowledge gathered by Femern A/S until May 2011.
As the investigations have not been completed,
an update of this report is planned for end 2012.

Prepared	**RMH, GLH, UTN, PSK, JRF, NLSM, CH, DJ**	**2011-05-01**
Checked	**PM**	**2011-05-01**
Approved	**JRF**	**2011-05-01**

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1. Introduction

This report summarizes the geophysical, geological and geotechnical investigations performed in 1995/96 and in the period from 2008 to May 2011 for the future Fehmarnbelt Fixed Link. The design solution (bridge or immersed tunnel) for the fixed link had not been decided before the different investigations were initiated and as such the performed investigations were directed against both a bridge and an immersed tube tunnel solution.

The investigations for the Fehmarnbelt Fixed Link in 2008, 2009 and 2010 included:

- Establishment of a positioning reference system by AXIONET mainly to be used during the construction of the fixed link.
- Geophysical offshore investigations performed by Rambøll Arup Joint Venture (RA) comprising marine shallow seismic investigations, marine side scan investigations, marine magnetic measurements and bathymetric measurements. The main survey was performed in 2008. In 2009 an additional bathymetric survey covering a wide area on both sides of the Rødbyhavn to Puttgarden ferry route was carried out and in 2010 a supplementary survey using similar tools as used for the 2008 survey was carried out.
- Geophysical onshore investigations performed by RA comprising onshore seismic reflection seismic investigations and Continuous Vertical Electrical Sounding (CVES) measurements.
- Onshore and offshore geotechnical type A-borings for sampling and geotechnical type B-borings for down-the-hole CPTs performed by Fugro.
- Down-hole geophysical borehole logging performed by RA.
- Seabed CPTs (type C-borings) performed by Fugro.
- Onshore Fehmarn CPT campaign performed by Fugro.
- Advanced Laboratory Testing performed by GEO/Deltares on samples recovered during the Boring Campaign.
- Geological (micropalaeontologic) dating of selected samples performed by GEUS.
- Large Scale Testing performed by Aarsleff/GEO.
- Establishment of an overall geological model and detailed description of the ground conditions, focusing on each detected geological unit. This part of the work includes also an assessment of the groundwater conditions, of the dome structures and of the seismicity in Fehmarnbelt as treated in separate Geotechnical Data Reports, prepared by RA.

It is the objective that the data and test results from the combined geophysical, geological and geotechnical investigations will enable an integrated approach to the interpretation of ground conditions and derivation of geotechnical properties covering the specified concepts for the fixed link between Germany and Denmark.

The investigations undertaken from 2008 to May 2011 have focused on an area approximately 2 km by 27 km situated east of the ferry route between Rødbyhavn in Denmark and Puttgarden in Germany.

This report includes the following:

- an executive summary is included in Chapter 2;

- the investigations from 2008 to May 2011 are summarised in Chapters 3 through 10; and
- the main findings of the investigations are summarised in Chapter 11.

Supplementary information, which is normally much more detailed, is presented in the Appendices GDR 00.1-001-A through GDR 00.1-001-F to the report and in the references listed in Chapter 12. The Geotechnical Data Reports (GDRs) listed in Chapter 12 are available in the Geo Information System /22/. A total list of all GDRs produced within the Geotechnical Services Agreement is attached as enclosure III.

The geotechnical background material and data are available within Femerns Geo Information System /22/.

For abbreviations and definitions not separately defined in the present report, reference is made to Femerns Geo Nomenclature (/12/) and to Femerns Legend and definitions (/42/).

2. Executive Summary

The investigations have included the below activities:

- **Seismic surveys**

The deep seismic survey of 1995 delivered an overall picture of the stratigraphy and tectonic situation in the investigated area; this interpretation is still valid. A comprehensive, more shallow survey was performed in 2008, and a supplementary survey in an area right west of the 2008-area was performed in 2010. This confirmed and detailed the picture arrived at in 1995. However, observations of disturbances in the layering and special point-reflections from something interpreted as cobbles in the Palaeogene layers south of the dome created the – wrong – idea that the upper series of layers south of the dome should be dominated by floes of Palaeogene clay interlayered by clay till.

Two further seismic surveys have been performed as part of the 2008-2010 investigations: The one was a bathymetric survey performed by RA; the other was a near-shore survey at both coastlines performed by GEUS.

- **Boring campaigns**

The interpretation of the seismic surveys provided the ground model allowing selection of locations for borings in the Boring Campaigns of 2009 and 2010, including mainly 36 boring locations off-shore in Fehmarnbelt and 12 boring locations inside the coastlines. The information obtained from the high quality deep borings has allowed a very significant increase in the understanding of the stratigraphic, tectonic and geotechnical conditions in the area. The different units identified on the seismic profiles have all been recognised as well defined geologic formations/units.

- **Geophysical borehole logging**

Geophysical borehole logging has been performed in almost all the boreholes, but problems with the stability of the walls have largely prevented the use of the optical televiewer, which can only be run in open holes. The original aim to obtain acoustic “pictures” of the borehole wall to document the condition of the chalk and to illustrate strike and dip of the layers in the folded part of the Palaeogene clay layers has unfortunately not been fulfilled. Moreover, the search for characteristic peaks or patterns, especially in the Palaeogene strata, in the profiles for other log tools has not been successful. However, from the logging important information on details of the series of layers has been gained, helping to correlate the upper layers between the boreholes; the logging has also been an important tool to secure the quality of the boring works.

- **Advanced laboratory testing**

The advanced laboratory testing has included pilot and production testing. The objective of the pilot testing has been to clarify procedures to be applied during production testing. The objective of the production testing has been to establish soil properties as measured in the laboratory as small scale testing with the identified procedures.

A considerable number of advanced laboratory tests such as different kinds of oedometer tests, triaxial tests and direct simple shear tests have been performed for the different soil units.

The major part of the advanced tests has been performed for the clays of Palaeogene origin, being the most challenging soil unit. However, also a considerable number of advanced tests have been performed for Glacial tills and for the Cretaceous chalk.

- **Large scale testing**

The large scale testing has been concentrated in an area with Palaeogene folded Røsnæs clay close to the seabed at a location in the shallow waters off the Fehmarn coast approximately 1 km east of Puttgarden ferry harbour and has so far mainly included:

- Excavation with base area 30 m × 30 m from elevation c. -10 m to elevation -20 m, installation of extenso-piezometers from 3 m to 25 m below excavation surface and installation of surface benchmark and performance of multibeam surveys.
- Cone penetration tests around the planned groups of test piles and ground anchors.
- Installation of driven steel tube piles and bored cast-in-situ test piles as well as reaction piles.
- Tension load testing of two of the driven piles and two of the bored piles.
- Instrumentation monitoring of the extenso-piezometers (P1-P9) and surface benchmarks.

- **Seismicity**

A desk study of the **seismicity** of the area has been conducted. It is concluded that the fixed link will be constructed in an area classified as a very low seismicity area. This means that the provision of the Eurocode 8 on Earthquake Risk does not need to be observed. It is, however, from the study concluded that the peak horizontal acceleration at the soil surface with a return period of 475 years is estimated to be in the range of 0.014 to 0.036 g.

INVESTIGATION RESULTS

From a morphologic point of view, the area can be divided into a central basin area with a seabed elevation below c. -24 m and two gently dipping coastal slope areas (Figure 11.1.1-1). As can be seen from the following descriptions this zoning of the site is important, as soft Postglacial and Lateglacial deposits are, with a few exceptions, only present within the central basin.Working from the surface downwards the deposits detected are:

- ***The Quaternary***

Postglacial marine sand. The deposit is dominated by clean, sorted sand with shells locally with a small amount of gyttja. The sand is encountered in local accumulations dispersed across the area; at least part of the deposit is mobile.

Postglacial marine gyttja. This soft organic deposit has been detected in the central part of the basin, where it, according to the borings, is locally up to 6 m thick. Marine gyttja has also been found in a former bay inside the coast line on Lolland. Peat, possibly Allerød age, has also been detected in a few borings below the marine deposits in a small depression in the till surface on the southern coastal slope.

Postglacial/Lateglacial deposits from the different marine and freshwater stages of the Baltic are present in the basin below and around the gyttja deposit. These include bodies of well to poorly sorted sand without any signs of organic activity which are presumed to be Lateglacial meltwater deposits, laminated ("varved") clay/silt supposed to be a Late-glacial meltwater/freshwater deposit and more massive, layered clay/silt deposits of more uncertain age and depositional environment.

Glacial deposits. During the 1995 to 1996 investigation campaign it became clear that the glacial deposits below the area could be divided into an "upper till" and a "lower till" with rather different appearance and geotechnical properties. The 2008 to 2010 campaign has revealed that both of the "tills" probably includes components of tills from more than one glacial event. As there is still a clear difference between the tills included in the original "Upper till" compared to those included in the original "Lower till" it has been decided to keep the original division, but to rename the original "Upper and Lower till" to "Upper and Lower till Unit" and only to use the term "till deposit" for the material left by the glaciers in a single glacial event.

Upper till Unit consists of very hard clay till, often with "normal" clay till composition but also often very silty and/or very sandy. Locally it is described as sand till. A number of observations indicate that the Upper till Unit consists of till deposits from two different glacial events, both with almost identical grain size distribution and both being very hard. The Upper till Unit is both most outspread and thickest in the northern part of the area where it often has its surface almost at seabed level in the marine locations or immediately below topsoil where encountered onshore on Lolland.

Meltwater deposits dominated by sand but locally including significant silt- and clay bodies have been detected within the glacial deposits both in the seismic surveys and in a number of borings. They are mainly concentrated in two separated parts of the investigated area. The meltwater deposits are considered to belong to at least two, maybe three different glacial events; one being within the lower part of the Upper till Unit and the other being located between the Upper and Lower till units.

Lower till Unit. This Unit is dominated by medium plasticity clay till, significantly it also contains both floes of high plasticity Palaeogene clay and meltwater deposits. The 2009 and 2010 Boring Campaigns have made it obvious that the Unit includes at least three different till deposits. By far most widespread of these, and which also seem to be the upper of them, is a medium plasticity clay till present all over the area. The second deposit is a high plasticity clay till, lowermost till, which has mostly been found in the southern part of the area. Where both the high and medium plasticity till have been detected in a single one and the same boring, the medium plasticity clay till is always situated above the high plasticity clay till. A third deposit belonging to the Lower till Unit is the "chalk till" with an almost white colour and a $CaCO_3$ content of more than 50%. This till deposit has only been found in rather few borings. It has been shown that it always appears above and therefore is younger than the high plasticity clay till, while its relation to the medium plasticity clay till is more uncertain.

- ***The Palaeogene***

Palaeogene clay. The Palaeogene clay is located directly below the Quaternary deposits and locally as floes within the Lower till Unit (albeit those floes are correctly described as Quaternary age deposits of Palaeogene clay). The microfossil analyses of a significant number of samples from the borings have shown that all of the Palaeogene formations from the Æbelø Fm., the Holmehus clay Fm., the Ølst clay Fm., the Røsnæs clay Fm. and to the Lillebælt clay Fm., and maybe also the Søvind marl Fm. are present in the borings. Moreover, it seems likely that the very uppermost part of the Røsnæs clay Fm. together with the lower part of the Lillebælt Clay Fm. dominate the floes, while the Røsnæs Clay formation totally dominates the very important part of the area immediately north of the German coast where Palaeogene clay is folded up almost exposed at seabed level.

- ***The Cretaceous***

Cretaceous chalk would have been deeply buried below younger deposits in the area if it had not been lifted up by a rising salt pillow to its present position less than 16 m below the sea bottom in the culmination point. The Chalk is a typical Danish chalk, very muddy and only slightly indurated. The flint content is typically about 5%, and the flint is present as nodules- often concentrated in layers - and not as plates as known from other (younger) limestone deposits in Denmark.

Glacial tectonics

A number of different observations indicate that the strata below the area have been heavily disturbed by ice pressure in one or more glaciations in the Quaternary. The uppermost part of the Palaeogene clay has been pressed up in big floes, and most of these floes have been transported south and west by the glaciers, finally to be left as isolated floes within the till deposits. Especially in the south-western part of the investigated area floes of Palaeogene clay are common.

The upper part of the Palaeogene clay below the till with the clay floes has been pressed up into a giant fold system, and only below elevation -70 m or deeper has the deposit not been affected by glacier pressure. The folding has clearly weakened the clay compared to the intact layers below, with the CPTs showing the exact level for the surface of undisturbed clay.

Salt tectonics

Two salt pillows are situated at depth below the general area of the site: One is located immediately north of the midpoint of the investigated area in Fehmarnbelt. The second salt pillow has its culmination point on land north of Rødbyhavn but its southern flank is within the area of interest for the fixed link. Salt pillows are formed by salt deposited in an 800 to 1000 m thick layer and covered by kilometre thick layers of younger deposits. Under such pressure the salt behaves in a plastic fashion, and as its density is lower than the density of the covering, heavily consolidated clay and sand deposits, the salt seeks an upwards path forming local salt pillows (thickenings). Concern has been directed both to the up-doming movement and the associated downwards movement in the area surrounding the dome itself and to the fact that the process must lead to stretching of the chalk present in the upper part of the covering series of layers, which again might weaken the chalk and maybe forms active faults.

It is concluded that the upwards movement of the central part of the pillow and downwards movement in an area surrounding the culmination point is likely still to be ongoing, but only at a maximum rate of approximately 1 mm/year rise in the culmination point.

Geotechnical parameters
The basic key to the geotechnical parameters is:
1. The combined plan and longitudinal section Drawing no. 070-02-09, included as Enclosure I (simplified geological profile appears from Fig. 2-1).
2. The soil type.
3. The CPT value.

It has been decided that the choice of geotechnical parameters for the deposits relevant for excavation/foundation of constructions shall be based on the combined results of CPT tests performed in the borings, from advanced laboratory tests and from plate load tests conducted as part of the Large Scale Testing. At the time for writing this report, the plate load tests have not yet been performed (programmed for later in 2011). The results/evaluations described in Chapter 11 are therefore still based on incomplete data sets, and also the "typical values" in the table (Enclosure II) might be slightly revised when the results of the plate load tests are available.

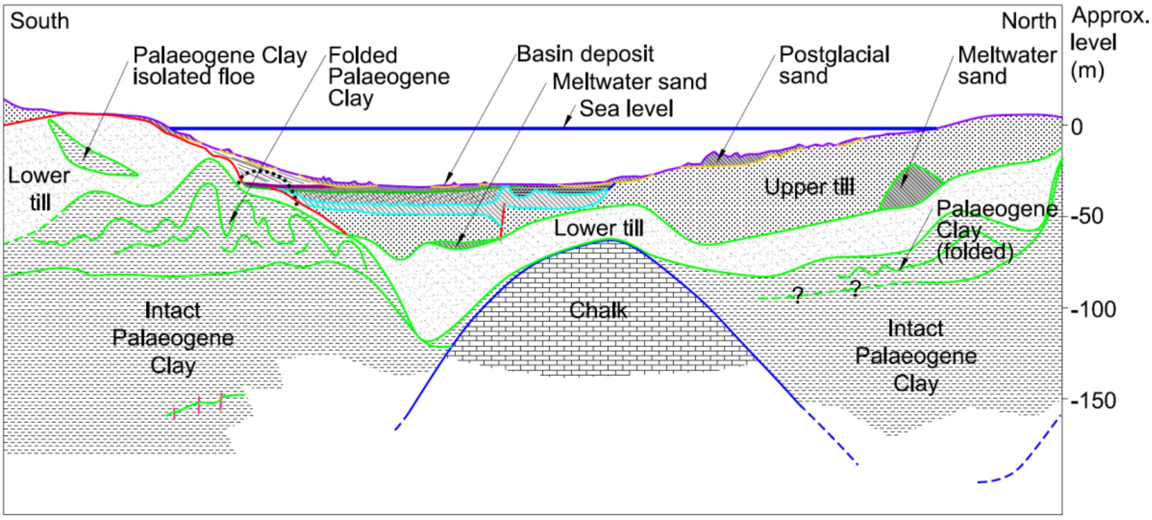

Figure 2-1. Simplified geological profile across Fehmarnbelt

POSITIONING SYSTEM
In both the planning phase and in the construction phase it is important to be able to accurately and unambiguously coordinate and position objects. It is also necessary to have means of obtaining accurate coordinates anywhere in the project area. To fulfil these requirements, a project related positioning reference system, the Fehmarnbelt Coordinate System (FCS) has been established, and a positioning reference frame (the physical realisation of the reference System) has been build up.

3. Positioning System

3.1 General

For the main part of the field investigations performed during the years 2008 to 2010 the UTM zone 32 N with EUREF89 datum has been used as plane coordinate system during survey operations. In addition, geographical coordinates in WGS 84 system were also used. Elevations and levels have been referred to DVR90.

Very early in the planning phase it was realised that the ability to accurately and unambiguously coordinate and position objects are a key requirement; a single 'coordinate system' must therefore be adopted for the project. This system could be either the German or the Danish national system or it could be a project specific system. Given the very tight requirements for reliability for such a construction project, the project related system was chosen.

Late in 2010 Femerns own project related coordinate system (FCS) was ready for use (/40/). The system included both a Positioning System and a Positioning Frame. It was developed by RA in close collaboration with KMS, BKG and DTU Space and with Axionet as a Contractor for the Positioning Frame.

Programs for conversion between the systems have now been developed, and in the Geo Information System (/22/) the new coordinates have been added where relevant to the coordinate related information.

3.2 Equipment and procedures

The project specific geodetic infrastructure established for project comprises:

- A *Reference System* (coordinate system and vertical reference definition),
- A *Reference Frame* (the physical realisation of the Reference System) and
- A *Positioning Service* (RTK service) providing high precision positioning and navigation.

The reference system of the FCS is defined as the International Terrestrial Reference System which is also used for GNSS.

The Femern Map Projection (FMP) is defined to be used within the construction area for the fixed link. The height system, FSCVR10, is defined as Mean Sea Level at Rødbyhavn in 2010. The defining stations have been connected to the tide gauge by levelling – using the 1989 hydrostatic levelling between Marienleuchte and Rødbyhavn for the connection across Fehmarnbelt. A geoid model has been developed and fitted to the FCSVR10 by levelled heights of the GNSS stations, and to the ITRF2005 by ellipsoidal heights of the GNSS stations.

Software has been developed for conversion of coordinates between the FCS, the German and Danish realisations of the ETRS89, as well as for conversion of ellipsoidal and orthometric heights within the FCS.

Last but not least a continuous positioning service has been established. This is an RTK GNSS service based on GPS and GLONASS with the option of including Galileo if this system becomes operational. The service transmits RTK network data by UHF radio. The estimated obtainable position accuracy (RMS) with the FBPS RTK service is 0.8 cm in the horizontal and 1.5 cm in the vertical for static applications under normal operating conditions. If one (or more) of the FBPS GNSS stations for any reason loose the communication line with the control centre, the station will automatically switch to a single station RTK service. The estimated obtainable accuracy (RMS) with this backup service is 1.2 cm in the horizontal and 1.7 cm in the vertical for static applications under normal operating conditions.

The combination of the 3D GNSS-based reference frame, the vertical reference frame and the geoid model are referred to as the reference frame. It is realised physically by determination of coordinates of four permanent GNNS stations (2 on Lolland and 2 on Fehmarn) and a number of 3D control points.

All details regarding reference system, map projection, geoid model, coordinate transformation and RTK service can be found in /40/.

4. Offshore Geophysical Investigations

4.1 General

In the 1995/96 investigations, a comprehensive survey including reflection seismic tools was performed (/13/). This included both a shallow seismic and a deeper seismic survey. The idea was that major, deep seated structures would be more easily observed on the deeper profiles. Once the deep seated structures had been identified it would then be easier to observe and understand their continuation through the upper layers from the shallow seismic surveys.

As part of the 2008 2010 investigations, surveys in the offshore area have been performed by RA in three separated campaigns. In addition, GEUS, in 2009, performed a nearshore survey along both the German and the Danish coasts.

The first and largest of the seismic surveys was the 2008 survey. This included both an offshore and a nearshore campaign. The offshore survey covered the area with a water depth of 5 m and deeper in a 2 km wide corridor; the vessel R/V Madoc was used as platform. Data acquisition was performed along lines extending north-south and parallel with the centreline of the investigated area. The original distance between the survey lines was 50 m. In order to accommodate the need for more detailed data for archaeological assessments, fill-in lines with focus on marine magnetometry and shallow seismic were added, thus narrowing the line distance to 25 m.

In the nearshore section the programme was limited to the acquisition of single beam bathymetry with 10 m line spacing. The dinghy ‘Rambunctious’ was used as platform for nearshore bathymetry acquisition.

The investigations are reported in /4/ comprising the bathymetry, magnetic anomalies, seabed classification, selected seismic profiles, and contours of surface as well as thickness of selected units.

In 2009 a bathymetric survey was performed within a much bigger area (c. 836 km^2) between Fehmarn and Lolland (/38/). Three vessels were used for the survey (M/V Triad, M/V Ping and M/V Seabeam.

Also in 2009, GEUS/DHI has performed a number of nearshore surveys in water depths between 2 and 6 m along the German and the Danish coasts (/39/). The three purposes for the surveys were archaeological investigations, marine biological mapping and coastal profiling. The lines were in most of the surveys sailed with an internal distance of 25 m in the direction along the coast, with a little overlap to the area surveyed in 2008 for QA reasons. However, for the coastal profiling the lines were sailed perpendicular to the coast.

Finally, in May 2010 a supplementary survey was performed immediately west of the area covered by the 2008 investigations (/4/).

4.2 Equipment and procedures

The 2008 survey was performed with the vessel R/V Madoc as platform; the following was carried out:

- Multi beam echo sounder bathymetric measurements.
- Marine shallow seismic using low and high frequency sources.
- Side scan sonar recordings.
- Marine magnometry.

The low frequency shallow seismic profiling was acquired using a Georesources Geospark 200 sparker source and a five element hydrophone streamer; the high frequency seismic was recorded using a Benthos Chirp III. The instrument was controlled from a Benthos CL-160 top-site unit.

The magnetometer was a Geometrics G882. A Reson SeaBat 8125 multibeam echo sounder was employed for acquisition of the offshore bathymetry. This instrument emits 512 beams in a beam angle of 60° and thus provides a swathe width of approximately three times the depth below the transducer.

In order to obtain valid depth measurements from the echo sounder, profiles of the sound velocities in the water were measured regularly. For this purpose an FSI CTD was used.

The position of the vessels was obtained using Javad GPS in RTK mode. A base station was installed on top of a silo next to Rødby Havn.

The bathymetric Large Area survey in 2009 (/38/) was performed with Kongsberg Multibeam EM3002D instruments as echo sounders.

The Supplementary survey in 2010 (/4/) was performed with the ship R/V Madoc as a platform and a Benthos SIS 1625 was employed for both high frequency seismic investigations and side scan recording. As magnetometer was used a Geometrics G882 instrument, and a Reson SeaBat 8125 was employed for the offshore bathymetric survey.

The nearshore survey performed in 2009 by GEUS used the ship Føniks Miljø as a platform for the part of the investigations that was related to archaeological purposes. A Geometrics G882 magnetometer was employed, and a Benthos SIS 1626 was used as combined side scan/sub-bottom profiler. For the part of the investigations that was conducted for coastal profiling purposes, the small ship GEUS II was used as a platform and bathymetric data was collected with a Navisound Reson instrument.

5. Onshore Geophysical Investigations

5.1 General

The onshore geophysical investigations performed by RA in 2008 consisted of reflection seismic investigations on Fehmarn and Lolland and Continuous Vertical Electrical Sounding (CVES) on Fehmarn.

The reflection seismic survey included two lines with a total nominal length of 4.5 km near Rødby and three lines with a length of 5.3 km near Puttgarden. The CVES survey included 11 lines near Puttgarden, with a total nominal length of 9.4 km. The investigations are reported in /3/. This report includes detailed maps of the investigation site, seismic profiles, CVES profiles and maps.

5.2 Equipment and procedures

5.2.1 Onshore seismic

The Pulled Array Seismic method is based on a seismic vibrator as energy source and a land streamer with geophones mounted on steel plates as receivers.

The investigations were performed using an IVI Minivib T7000 seismic vibrator as energy source. The land streamer is fitted with geophones from Mark Products. The total length of the land streamer is 222.5 m.

Distance between vibration points	10 m
Distance between vibrator and first group	6.25 m
Distance between group 2–49 (first channel is used for correlation of the sweep)	1.25 m
Distance between group 50-112	2.5 m
Sample distance	0.5 ms

Table 5.2.1-1 Recording parameters

5.2.2 Continuous Vertical Electrical Sounding (CVES)

CVES is an automatised geoelectrical measuring method which allows rapid collection of large datasets from electrical soundings. The principle is that a current applied to two electrodes generates an electrical potential in the ground which can then be measured using two other electrodes. When the distance between the power electrodes is increased, the penetration into the ground will also be increased. Using varying geometries it is then possible to obtain information of the variation of the resistivity with depth as well as laterally.

The equipment used for the CVES measurement is from the Swedish company ABEM.

All CVES lines are recorded with the 400 m setup, giving an investigation depth of approximately 60m. The interpreted CVES are in /3/ reported as profiles and as contoured maps for the depth to and the level for the Palaeogene Clay surface.

6. Boring Campaign

6.1 Onshore borings and CPTUs

6.1.1 General

The onshore Boring Campaign performed by Fugro in 2009 and 2010 included borings in 8 locations on Fehmarn and 4 locations on Lolland (in total 12 type A-borings and 10 type B-borings). Maximum depth of boring was 100 m. In addition to the A and B-borings, single push CPTUs were performed at 30 test locations on Fehmarn at which a total of 58 attempts were performed.

Except for the 30 test locations with single push CPTs on Fehmarn the onshore locations are shown in Enclosure I (Drawing no. 070-02-09). All onshore borings performed in 2009 have “09” and borings performed in 2010 have “10” as the two first figures in the boring number followed by “.A.” or “.B.” for the boring type and then by a serial number, “6NN” for borings on Fehmarn and “7NN” for borings on Lolland.

Type A-borings provide continuous or nearly continuous core while type B-borings provide continuous or nearly continuous CPTU readings from downhole testing.

The geotechnical logs for each individual type A and type B-boring appear from profiles provided in /2/. The single push CPTUs appear from logs provides in /24/.

For each type B-boring one set of measured values is given (depth, cone resistance, sleeve friction and pore pressure) together with a set of derived values defined by corrected cone resistance, net cone resistance, pore pressure ratio and friction ratio.

6.1.2 Equipment and procedures

The type A-borings at all 12 locations were initially drilled by cable percussion techniques and when coreable layers were met, the drilling work was continued using Geobor-S wireline drilling technique. However, in boring 09.A.601, 09.A.607 and 10.AB.610, Symmetrix destructive drilling techniques were used to overcome cohesionless layers with stones and cobbles.

During drilling in non-coreable strata, tube samples were recovered with a push sampler and samples were recovered with a bailer sampler or hammer sampler. Coring was performed with the Geobor-S equipped with a triple barrel coring device producing cores with 101 mm diameter.

In the onshore type A-borings (except for 09.A.601) a 88 mm PVC tube liner was installed and sealed with a bentonite grout enabling geophysical borehole logging – including VSPs – to be performed (see Chapter 8). Additionally, slotted standpipes were installed in two of the borings (09.A.602 and 09.A.607E) for water level observations.

The type B-borings at the 10 onshore locations were performed in a distance of c. 5 m from the corresponding A-boring, using a truck mounted CPTU system alternating with another truck mounted drilling system. The procedure included a CPTU push generally followed by drill-outs until the end of the borehole was reached. The CPTU truck penetrated CPTU rods to the depth of refusal in each stroke (stroke up to c. 25 m) alternating with clean-out boring and succeeding stroke etc. For some of the onshore CPTUs like 09.B.604A only one initial stroke has been performed and reported.

All CPTUs used a 10 cm^2 piezo-cone measuring cone tip resistance (q_c), the sleeve friction (f_s) and the pore pressure (u_2). Additionally, the inclination of the cone was measured. CPTUs were performed according to /17/ with target class 2 accuracy.

6.2 Offshore borings and CPTs

6.2.1 General

The offshore Boring Campaign performed by Fugro in 2009 and 2010 included boring in 36 offshore locations (36 type A-borings and 30 type B-borings). The boring depth has been up to 100 m. The offshore locations appear from Enclosure I (Drawing no. 070-02-09).

6.2.2 Equipment and procedures

All offshore type A-borings, except 09.A.004, were performed from Jack-up drilling platforms “SKATE III” or “Deep Diver”. The borings 09.A+B.004 were performed from the geotechnical drilling vessel “Highland Eagle”. The borings 09.B.007, 09.B.008 and 09.B.017 were performed from the geotechnical drilling vessel “Gargano”. The type B-boring at a location was generally performed within a distance of 5 m from the corresponding type A-boring.

Type A-borings were drilled using cable percussion techniques followed by core drilling using Geobor-S. After completion, the boreholes were sealed with a cement/bentonite grout.

Type B-borings (down-the-hole CPTU borings) were performed with Geobor-S in combination with a non-coring device deployed in the drill bit. The downhole CPTUs employed the WISON XP system on the “SKATE III” and “Deep Diver” and the WISON MkIII system on the “Gargano” and “Highland Eagle”. The CPTUs have been performed with down-the-hole equipment with maximum stroke length of 1.5–3.0 m alternating with clean-out boring and succeeding stroke etc.

All CPTU tests used a 10 cm^2 piezo-cone measuring cone tip resistance (q_c), the sleeve friction (f_s) and the pore pressure (u_2). CPTs were performed according to /17/ with target class 2 accuracy (namely data which allows, with rigorous procedures applied, derivation of geotechnical parameters).

The type B boreholes were sealed with a cement/bentonite grout.

6.3 Sample handling and laboratory works

6.3.1 General

The sample handling and laboratory works included mainly:

- On-site sample handling
- On-site geotechnical classification testing
- Off-site geotechnical laboratory testing (in Fugros main laboratory in The Netherlands).

The geological descriptions, the sample colour photographs and results of Fugros on-site and off-site classification tests for the borings appear in detail in /2/ and /23/.

6.3.2 Equipment and procedures

The samples retrieved from the borings were handled as follows:

- Hammer samples and push samples taken with Shelby tubes were after measurement of the recovery, cleaned from drill cuttings and sealed with plugs.
- Bailer and percussion samples taken with a split spoon were transferred to plastic bags and sealed.
- Core samples were, after removal from the barrel, immediately transferred to a split liner, cleaned (by removal of drill fluids and cuttings) and recovery was then determined.

The samples were then transported to the on-site geotechnical laboratories, where the testing/activities mainly included:

- Geological sample description in accordance with Femerns Geo Nomenclature and DGF Bulletin 1.
- Sample colour photography.
- Water content determinations in accordance with CEN ISO/TS 17892-1 for soils and ISRM Part 1 for chalk.
- Unit weight determination in accordance with CEN ISO/TS 17892-2 for soils and ISRM Part 1 for chalk.
- Selection, labelling, preservation of samples for Femerns Advanced Laboratory Testing, for geological dating, for Fugro off-site laboratory testing and for Femerns bulk material.
- Transportation of samples to Femerns sample containers in Rødbyhavn and to Fugro laboratory in the Netherlands.

The off-site geotechnical laboratory testing/activities mainly included:

- Particle density test in accordance with CEN ISO/TS 17892-3.
- Particle size analysis in accordance with CEN ISO/TS 17892-4.
- Atterberg limits in accordance with CEN ISO/TS 17892-12.
- Organic content in accordance with BS1377, Part 3, Clause 4 and BS1377, Part 3, Clause 3.

6.4 Correlation borings Lillebælt

The purpose of the correlation borings was to evaluate the influence of the old Lillebælt Bridge on the ground conditions. The following investigations were performed:

- A 10 m deep boring 10.A.803 was performed beneath the concrete slab of pillar 3. This investigation was performed by GEO, cf. Appendix GDR 00.1-001-F.
- Prior to the commencement of the offshore borings a munition survey in a minor area to the west of the Bridge was carried out. The munition survey included magnetometer and side scan investigations performed by GEUS. /42/
- 2 offshore sampling type A-borings 10.A.801 and 10.A.802 and two Type B-borings 10.B.801 and 10.B.802 were performed from the Jack-up drilling platform “Deep Diver” by FUGRO. Boring 10.A.801 and 10.B.801 were performed adjacent to pier 1 to a depth of 75 m below seabed and Boring 10.A.802 and 10.B.802 close to pier 3 to a depth of 40 m below seabed. The geological descriptions, the sample colour photographs and results of Fugro on-site and off-site classification tests for the borings appear in detail in /25/.

6.5 Correlation borings Fehmarnsund

The purpose of the correlation borings was to evaluate the influence of the northern embankment of the Fehmarnsund Brücke on the ground conditions beneath it. The following investigations were performed:

- One type A sampling borehole 10.A.901 and one type B CPTU boring 10.B.901, both to c. 80 m depth. The investigation was performed by the Large Scale Testing contractor Aarsleff/GEO. The geological descriptions, the sample colour photographs and results of Aarsleff/GEOs on-site and off-site classification tests for the borings appear in detail in /34/.

7. Seabed CPTU-campaign

7.1 General

The seabed CPTU campaign performed by Fugro in 2009 consisted of 41 in-situ cone penetration tests using the Wheeldrive system with a maximum penetration thrust capacity of 200 kN. The target CPTU depth was 25 m. The CPTU locations appear from Enclosure I (Drawing no. 070-02-09). All seabed CPTUs performed in 2009 have “09” as the first two figures in the CPTU number followed by “.C.” and then by a serial number “4NN”. The fieldwork operations were carried out from M/V Fugro Commander.

The in-situ test results of the individual CPTUs appear from the CPTU profiles provided in /1/. For each seabed CPTU one set of measured values is given (depth, cone resistance, sleeve friction and pore pressure) together with a set of derived values defined by total (=corrected) cone resistance, net cone resistance, pore pressure ratio and friction ratio.

7.2 Equipment and procedures

All seabed CPTUs (type C-borings) were performed using a 10 cm^2 cone measuring cone resistance (q_c), the sleeve friction (f_s) and the pore pressure (u_2).

The seabed CPTUs were performed according to /17/ with target class 2 accuracy. The stop criteria for each CPTU test occurred when the thrust capacity of the push system was insufficient to overcome the soil resistance. In case an obstruction was encountered during testing the cone and CPTU string were withdrawn 0.5 m and a second attempt was initiated to overcome the obstruction.

In /1/ a simplified interpretation of the dominant soil types has been presented.

8. Geophysical Borehole Logging Campaign

8.1 General

The geophysical borehole logging has been performed by RA in 33 offshore boreholes in Fehmarnbelt, in 7 onshore boreholes on Fehmarn and in 4 onshore boreholes on Lolland.

Vertical Seismic Profiling, VSP, has been performed in 7 onshore boreholes on Fehmarn and in 2 boreholes on Lolland.

The borehole logging has been performed as an integral part of the follow-up of the geological description and classification. Furthermore, the logging has been carried out for the following purposes:

- To outline the detailed stratigraphy and contribute to the geological model.
- To verify the depth to significant markers as registered during the boring process.
- To identify geophysical soil properties relevant for the geotechnical appraisal.
- To contribute to the correlation between boreholes and other geophysical data.

The geophysical borehole logging is reported in /5/.

8.2 Equipment and procedures

The logging programme included a number of different logging probes and the VSPs were performed with a hydrophone array and with an array of two 3D geophones. The choice of log-suite is very dependent on the stability of the borehole wall and on the installation in the borehole. The different logs are to a different degree influenced by the fluid (mud or polymer) in the borehole, by plastic or steel casings and by the borehole diameter.

Based on this fact, the chosen logging strategy has been to try to keep the borehole open without any kind of casing through the chalk and Palaeogene clay, because in an open borehole a wide range of logs can be recorded without any influence from the installation. Only a reduced log programme can be conducted in a plastic liner with or without slots or in a steel casing. However, in most of the borings there were stability problems, and in only one borehole was the stability adequate for the optical televiewer to be used.

The offshore logging programme included:

- Natural Gamma
- Single Induction Conductivity
- Focused Guard Log Resistivity, Deep and Shallow
- Compensated Neutron-Neutron Porosity
- Compensated Gamma-Gamma Density
- Calliper 1-arm or 3-arms
- Sonic P-Wave Velocity
- Acoustic Hardness (only in the open boreholes)
- Acoustic Radius (only in the open boreholes)

The onshore boreholes were initially logged through a cemented plastic liner. The total onshore logging programme included:

- Natural Gamma
- Dual Induction Conductivity
- Compensated Neutron-Neutron Porosity
- Compensated Gamma-Gamma Density
- Calliper 1-arm
- Sonic P-Wave Velocity
- VSP S-wave velocity
- VSP-P wave velocity

The complete log suite for each borehole is presented as composite log and VSP sheets. The interpreted geological profile supplemented with information on the technical completion of the borehole during the logging operation in terms of steel casing and/or plastic liner is also included in the composite log sheets in /5/.

9. Advanced Laboratory Testing

9.1 General

The Advanced Laboratory Testing including pilot and production testing has been performed by GEO with Deltares, NGI and Fugro as subcontractors.

The objective of the pilot testing has been to clarify procedures to be applied during production testing. The objective of the production testing has been to establish soil properties as measured in the laboratory with the identified procedures. The total number and type of tests for the different soil units reported and considered in this Ground Investigation Report are shown in Table 9.1-1.

The advanced laboratory tests included:

- IL: Oedometer, incremental loading.
- IL K0: Oedometer, incremental loading, determination of K_0.
- CRS: Oedometer, constant rate of strain.
- CPR: Oedometer, constant pore pressure ratio.
- CADc: Triaxial, anisotropic consolidation, drained failure, compression.
- CAUc: Triaxial, anisotropic consolidation, undrained failure, compression.
- CAUe: Triaxial, anisotropic consolidation, undrained failure, extension.
- CAUcy: Triaxial, anisotropic consolidation, undrained failure, cyclic.
- DSSst: Direct simple shear box test, static.
- DSScy: Direct simple shear box test, cyclic.
- UU: Triaxial, unconsolidated, undrained failure.
- RC: Resonant Column.
- UCS: Unconfined compression test.
- Brazil: Brazil test

The major part of the advanced tests has been performed for the clays of Palaeogene origin, being the most challenging soil unit. However, also a considerable number of advanced tests have been performed for Glacial tills and the Cretaceous chalk.

It must furthermore be noted that:

- Glacial melt water sand was only recovered in 09.A.018. Hence, limited testing.
- All testing in Æbelø has been performed in 09.A.007, where this formation has been encountered at approximately 20 m depth. Formation is only 1 m thick, so very limited testing.

The detailed Advanced Laboratory Testing has been reported in /26/ through /32/, where /26/ includes the details about the applied laboratory procedures.

Advanced laboratory tests, Numbers of conducted tests

Soil units excluding chalk	IL	IL_K0	CRS	O_CP	CADc	CAUc	CAUe	CAUcy	DSSst	DSScy	UU	RC	**TOTAL**
Postglacial marine sand/gravel unit													**0**
Postglacial marine/freshwater gyttja unit	5				2	3							**10**
Postglacial/lateglacial marine/freshwater, clay/silt unit	6				2	5	1		1				**15**
Postglacial/lateglacial marine/freshwater, sand/gravel unit													**0**
Glacial, Upper clay till	8	1	3		3	5	1						**21**
Glacial, Meltwater unit					2	1							**3**
Glacial, Chalk till	5	1			2	4		3					**15**
Glacial, Lower and Lowermost clay till	25	2	2	1	3	14	5	6	7	6		1	**72**
Palaeogene Lillebælt/Røsnæs unit, Floe	4		2						1				**7**
Palaeogene Lillebælt/Røsnæs unit, unclassified		1		1		2	1		2				**7**
Palaeogene Røsnæs unit - folded	68	13	14	12	4	58	13	4	46	10	24	5	**271**
Palaeogene Røsnæs unit - floe	2			1		2							**5**
Palaeogene Røsnæs unit - intact	9	1				6	1		1				**18**
Palaeogene Røsnæs unit - unclassified						1							**1**
Palaeogene Ølst unit - folded	3	1		1	2	3	1	1	1	3	1		**17**
Palaeogene Ølst unit - floe													**0**
Palaeogene Ølst unit - intact						1							**1**
Palaeogene Holmehus unit - unclassified	4	1		1	2	4	1	2	3	4			**22**
Palaeogene Æbelø unit	1					1							**2**
Borings from the old Lillebælt Bridge	5	1		2		1							**9**
Borings from the northern embankment at the Fehmarnsund Bridge	8			2									**10**
SUM	**153**	**22**	**21**	**21**	**22**	**111**	**24**	**16**	**62**	**23**	**25**	**6**	**506**

Chalk	IL	CADc	CAUc	CAUe	UCS	Brazil	**TOTAL**
Chalk	22	9	26	16	20	19	**112**
Cement stabilised soil (clay of Palaeogene origin)					4		**4**

Table 9.1-1 Summary of performed advanced laboratory tests

X-Ray photos have generally been used for selecting cores for testing. Routine classification tests have been performed for the cores selected for Advanced Laboratory Testing. X-Ray diffraction analyses have been performed for selected cores for determination of clay mineralogy.

Water used during laboratory testing for saturation and within the triaxial cells is artificially established pore water with a ion composition similar to the ion composition identified in the in-situ pore water.

9.2 Equipment and procedures

9.2.1 General

Details about the equipment and procedures can be found in /26/. Some of the recognised more specific issues are addressed below. The geological dating of selected samples has been performed by GEUS and the results are included in /6/.

9.2.2 Oedometer testing

Swelling pressure

During oedometer tests it is found that the clays of Palaeogene origin, when mounted in the cell without access to water and loaded to its in-situ vertical effective stress and then saturated, will absorb water and try to swell. This behaviour has been observed for swelling tests and traditional oedometer tests, and it indicates that the clays of Palaeogene origin in nature should expand for its in-situ stress. This is, however, not believed to be realistic. If the clay expands when saturated at the vertical effective in-situ stress in the laboratory, the water content increases relative to the in-situ state, implying that the properties measured in the test may not be representative.

The observations described above are most likely a consequence of a too low mean stress in the specimen at the time of saturation, meaning that the horizontal effective stresses are not fully regenerated when loading the specimen to its vertical effective in-situ stress. Saturation in the oedometer cells is therefore performed at a higher stress level (typically 1.5 to 2.0 times the vertical effective in-situ stress at shallow depth and decreasing with increasing depth).

Pre-consolidation pressure

The methods requested for estimating the pre-consolidation pressure is described by Akai /18/, Becker /19/, Casagrande /20/ and Janbu /21/. For both the clay till and for the clays of Palaeogene origin it has been a challenge to identify a clear and consistent pre-consolidation pressure.

Clay till is a stiff to very stiff material containing rather coarse grains. Trimming of the specimens will therefore usually imply an uneven perimeter and full contact between the soil and the Oedometer ring is therefore not established from the beginning of the test. The stiffness of the material combined with the lack of full contact between the specimen and the ring implies that the measured vertical strain will include an effect of the total specimen volume being squeezed laterally towards the Oedometer ring. A clear break down of the soil skeleton is therefore not seen in all the tests. It has been found that the method of Casagrande will imply a reasonable estimate of the pre-consolidation pressure, σ'_{pc} provided that σ'_{pc} does not exceed 1.5 to 2.0 MPa (a maximum of 4.8 MPa can be applied in the apparatus).

For clays of Palaeogene origin the rate of secondary compression, $C_{\alpha\varepsilon}$ increases with the axial stress and the virgin curve therefore keeps curving in a bilogarithmic plot. As a consequence of the curving virgin line, the methods of Becker and Casagrande will lead to pre-consolidation pressures being very dependent of the stress interval used along the virgin line. The pre-consolidation pressure as determined by the method of Janbu is located at a stress being slightly lower than where the minimum tangent modulus value is identified and the estimate of the pre-consolidation pressure is therefore not influenced by the stress-strain behaviour for large axial stresses where the creep contribution may be significant.

In order to clarify which method that would lead to the most consistent result, reloading loops from a series of CRS-tests were analysed. For these tests, a maximum axial stress exceeding any possible pre-consolidation pressure was applied before unloading was initiated. When analysing a reloading loop, the corresponding “pre-consolidation pressure” must be close to the maximum pressure experienced during previous phases of the test. It appeared that Janbu came out with a realistic “pre-consolidation pressure” for each test, whereas the Casagrande and the Becker methods were very sensitive with respect to modelling a straight line with measured data along a curve.

9.2.3 Triaxial testing

Triaxial testing is generally performed in accordance with Danish practice of using test specimens with a height to diameter ratio of unity combined with the use of lubricated ends. The triaxial tests on clay till and Palaeogene specimens have generally been carried out as consolidated undrained tests with pore pressure measurements (measurement on back pressure line).

The two different approaches on how samples are consolidated prior to carrying out undrained compressions are:

- Danish experience with triaxial testing on over-consolidated soils is that the specimen should be K_0-consolidated to a low estimate of the pre-consolidation pressure σ'_{pc} and then unloaded under K_0 conditions to the stress state, from which shearing will be initiated.
- An alternative procedure is to load the specimen stress controlled directly to the in-situ stress state, from which shearing will be initiated.

Two different procedures have been tested on the soil types encountered on the project and the measured undrained shear strengths have been compared with test results from constant volume Direct Simple Shear test and field measurements using CPTU.

The findings may be summarised as:

- For Lower Clay Till and for clays of Palaeogene origin, the specimen is loaded stress controlled directly to the in-situ stress from which shearing is initiated.
- For Upper Clay Till the specimens are loaded stress controlled to a low estimate of the pre-consolidation pressure before a stress controlled unloading is initiated, followed by shearing.

The in-situ stress state was estimated combining the vertical effective in-situ stress (hydrostatic pore pressure distribution) with correlations between the net cone resistance from CPTU and the pre-consolidation pressure, leading to the over-consolidation ratio, OCR. The lateral effective in-situ stress was defined using the coefficient of lateral earth pressure at rest, as measured in the IL K_0 tests.

For clay till a strain rate of 0.3 %/h is used during undrained shearing, whereas 0.05 %/h is used for the clays of Palaeogene origin (undrained shearing). The pressure heads in the triaxial cell have a diameter of 70 mm. For clay till it has been found that a specimen diameter with 70 mm will suffice whereas a specimen diameter of 68 mm for clays of Palaeogene origin must be used to ensure that the footprint of the specimen will not exceed the area of the pressure head during shearing.

9.2.4 Geological dating of samples

Geological dating based on microfossils has been performed on 163 samples of clays of Palaeogene origin and 7 samples of chalk. The chosen strategy for the dating has been to start with coccoliths and foraminifera (with $CaCO_3$ shells) and to supplement with the more expensive palynological analyses on samples where the results of the first analyses have not given sufficiently reliable results. Methods and results are further described in /6/ and Enclosure II, respectively.

10. Large Scale Testing Campaign

10.1 General

The Large Scale Testing has been performed by Per Aarsleff with GEO as subcontractor and with NGI as a supplier for most of the instrumentation. The works carried out to date (May 2011) include:

- Phase 1 excavation (base 30 m × 8 m) to elevation -20 m.
- Piezometer and extensometer installation (extenso-piezometers 3, 9 and 25 m below excavation surface).
- Phase 2 excavation (base 30 m × 30 m) .
- Surface benchmark installation (11 surface benchmarks).
- Four multibeam surveys of the 200 × 200 m2 survey area.
- Cone penetration tests at 3 locations (10.C.451-453).
- Driven pile installation of 5 test piles (DP1-DP5) and 4 reaction piles.
- Bored cast-in-situ pile installation of 5 test piles (BP1-BP5) and 4 reaction piles.
- Driven pile tension load testing of two piles (DP1 and DP2).
- Bored pile tension load testing of two piles (BP1 and BP2).
- Instrumentation monitoring.

The works performed by Per Aarsleff and GEO are reported in /33/-/37/.

10.2 Equipment and procedures

10.2.1 Trial excavation works and surveys

The excavation has been performed by the excavator CAT 385 mounted on a spud barge. Tugboat and splitbarges have been used for transporting the excavated material to the dump site.

Slopes were excavated with inclination 1 (vertical) to 2 (horizontal).

Four multibeam surveys have until now been performed using a Reson Seabat 7125 multibeam echosounder.

10.2.2 Instrumentation and monitoring

The six medium depth and deep extenso-piezometers were installed in partly predrilled boreholes while the three shallow extensor-piezometers were pushed in from the base of the phase 1 excavation. All nine extenso-piezometers were pushed to final depth using GEOs surface rig GEOTop with thrust capacity 20 tons. Automatic logging of data started 2010-09-30.

The four pile piezometers were placed at 14 m (PP1 and PP3) and 19 m (PP3 and PP4) below top of driven pile number DP4.

The reference measuring device (RMD) has been used to measure the relative height difference of the pedestals, extensometer anchors below pedestals and surface benchmarks. After each measurement the height of each pedestal/extensometer anchor/surface benchmark is calculated relative to pedestal P5 which is assumed to be a stable reference point.

10.2.3 Pile installation

The steel tube piles DP1-DP4 being OD508 × 20 mm and DP5 being OD508 × 22.2 mm were driven to 25 m below seabed. Additionally, four reaction piles DR1-DR4 were driven to 22 m below seabed. During pile driving the piles were checked for plugging and plugs were removed before piling was resumed.

The bored cast-in-place piles BP1-BP5 being OD610 mm on the upper 10 m below seabed and OD500mm below that depth were installed to 25 m below seabed using a hydraulic drilling rig mounted on a jack-up. Additionally, four reaction piles BR1-BR4 were installed to 22 m below seabed. The drilling with casing, bucket and auger was continued until the casing was firmly embedded in the clay. Final drilling, installation of reinforcement cage and concreting by tremmie tube was performed in a “dry” borehole.

All piles were installed through a piling guide frame with a 3 × 3 pile configuration.

10.2.4 Pile load testing - tension

Two of the driven piles and two of the bored cast-in-place piles have been tension load tested by means of a large reaction frame placed on top of the four reaction piles. The tension load is applied by a hydraulic jack mounted on the top of the reaction frame.

Displacements of pile heads were measured by displacement transmitters installed on a separate reference girder beam. Supplementary measurements of displacements were performed by means of the RMD.

10.2.5 CPTU

The CPTUs have been performed using GEOs surface rig GEOTop with a thrust capacity of 200 kN. The hydraulic CPTU press was mounted on a movable working platform. The CPTU testing could alternate with a truck mounted drilling system. The general procedure included a CPTU push in one stroke to refusal depth (up to 25.6 m) with possible need for extra attempts.

All CPTU tests used a 10 cm^2 piezo-cone measuring cone resistance (q_c), the sleeve friction (f_s) and the pore pressure (u_2). Additionally the inclination of the cone was measured. CPTU’s were performed according to /17/ with target class 2 accuracy (namely data which allows, with rigorous procedures applied, derivation of geotechnical parameters).

11 Ground Conditions

11.1 Geologic setting

The geologic and tectonic framework for the Fehmarnbelt area is described in /7/ and /8/. In those reports the deeper lying deposits including the thick Permo/Triassic halite (i.e. salt) layers, the main fault systems and the halokinetic structures (salt pillows) are described. The conclusions from the reports are summarised in the below Sections 11.2 and 11.3.

The ground water conditions described in detail in /9/ are summarised in Section 11.4 below.

The near surface Postglacial/Lateglacial deposits will be of direct interest in connection with the foundation works for the fixed link constructions and so will possibly also the deposits from the youngest part of the Cretaceous period, from the Palaeogene period and from the rest of the Quaternary period. (Neogene deposits have – at least not yet – been found in the area). These deposits have been described in detail in /6/ and the main observations and conclusions are summarised in Section 11.5.

11.1.1 Morphology and seismic stratigraphy

The seabed in the investigated area between Fehmarn and Lolland has been divided into a central basin with water depths above c. 24 m and two gently sloping areas from the coasts to the basin with water depths from 0 to c. 24 m (Figure 11.1.1-1). In the following sections these areas are termed "the basin" and "the slopes".

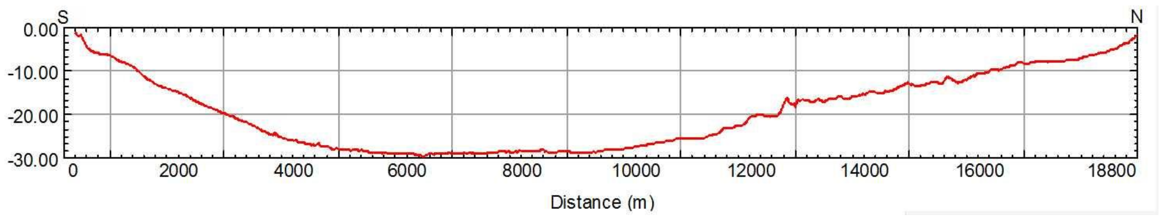

Figure 11.1.1-1 Depth profile across Fehmarnbelt. All units in metres.

Based on the seismic investigations performed in 1995/96 and in 2008 a number of seismic units have been defined. The Boring Campaigns in 1996, 2009 and 2010 revealed the geologic character of the units mapped from the seismograms, and Figure 11.1.1-2 illustrates the connection between seismic and geological units. The seismic Units used in earlier reports will therefore in this and future reports be replaced with true geological/sedimentological units.

Unit	Subunit	Geological description
Postglacial/Lateglacial deposits *("Upper Quaternary unit")*	*SU_1*	Postglacial marine sand with shells
	SU_{2a}	Marine gyttja (includes also freshwater peat and gyttja)
	SU_{2b}	Silty clay and silt, often laminated
	SU_{2c}	Mostly silty clay. Local sand layers
	SU_3	Sorted sand. Locally with silt and clay layers.
	SU_4	Sorted sand. Locally with silt and clay layers. Located in the northern part of the Basin.
Glacial deposits *("Lower Quaternary unit")*	*Upper till*	Sandy to very sandy clay till
	Meltwater sand	Sorted sand. In some areas with thick silt layers
	Lower till	Medium plasticity clay till. Locally with floes of Palaeogene deposits.
Palaeogene deposits (Both folded and intact)	Søvind Marl Formation	Most often very calcareous or calcareous light grey clay
	Lillebælt Clay Formation	Greenish, very high plasticity clay. Often non-calcareous
	Røsnæs Clay Formation	Often reddish or brownish, very high plasticity clay. Normally calcareous.
	Ølst Formation	Dark or even black, high to very high plasticity clay with pyrite concretions. Numerous ash layers
	Holmehus Formation	Multicoloured (reddish, bluish, greenish) very high plasticity, non-calcareous clay
	Æbelø Formation	Grey, non-calcareous, often very silty clay or even silt.
Cretaceous chalk	Maastrichtien and Campanian deposits	White chalk

Figure 11.1.1-2 Relation between seismic units and deposits detected in the borings. Names used for seismic units are written in *italic* in the figure.

A 3D digital geologic model of the area has been set up, illustrating the distribution of and the shape of the surfaces of the most important layers. This model is available in the Femern Geo Information System (/22/).

11.2 Salt pillow structures

During the last part of the Permian, Northern Europe including the Danish area was covered by a more or less isolated, wide sea. The climate was warm and dry and the intense evaporation from the sea surface resulted in the water becoming saturated with salt. The salt precipitated out of solution and collected as deposits on the sea bed. In the deepest part of the sea, salt layers of more than 1000 m thickness were deposited. The Fehmarnbelt area was, however, situated immediately south of the Jylland/Fyn ridge, which during this period was an island in the central part of the sea. The thickness of the salt layer is therefore 200-300 m less below Fehmarnbelt than in the central part of the former sea.

Because the density of the salt is less than the density of the consolidated sedimentary soil/rock covering the salt deposits and because the salt can deform in a viscous manner under pressure, it has a tendency to move upwards towards the surface forming salt pillows, sometimes further developing to domes and even to diapirs. A very large number of salt structures formed by this mechanism are found in the former Zechstein Sea area. Two of them are situated within or so close to the investigated area that their presence may be of importance for the fixed link project. Both of them are in the pillow stage, in which the lateral movement of salt from surrounding areas simply bends up the covering layers. The process is presumed to be still ongoing.

The presence of one salt pillow below the eastern part of the investigated area for the Fehmarnbelt Fixed Link and another one immediately north of the land connection in Rødbyhavn might influence the project in three different ways:

1. The vertical movements and the variations therein. It is important to be aware that surface movements above salt structures can be directed both upwards and downwards. This is a result of the salt flowing against the central part of the dome and lifting the surface above. This is flowing in from a “peripheral rim” around the culmination area, making the surface above subsidise. In /8/ information from literature on measured and calculated rates of heave/settlement of the surface above a number of dome structures in Northern Europe are presented. Moreover, information/indications on heave rate of the Fehmarnbelt dome have been collected from the seismic profiles. It is concluded that a heave rate of approximately 1.3 mm/year is a very conservative (upper bound) estimate of the ongoing movements of the surface above the Fehmarnbelt dome, and that the movements are likely ten times smaller than this value.

   The investigations have revealed that the northernmost part of the fixed link area is situated on the southern flank of the Rødby dome. As result of evaluations of the thickness and variation in the surface level of the two till units, the annual heave of this area might be similar to or slightly larger than the heave of the area above the Fehmarnbelt dome.

The deep “hole” in the surface of the Palaeogene clay in 09.A.004 indicates that this boring might be situated in a “peripheral rim” area around the Fehmarnbelt dome. If this is the case, the "rim" should be clearly visible in the chalk surface too, but the 1996 deeper seismic profiles give no clear evidence for that. Details on the 2008/2010 seismic profiles indicate that the subsidence rate of the surface above the area for the possible “rim” might have been significant earlier in the Quaternary period but has been very small or even absent in the last 20,000 years.

2. As the salt lifts the overlying strata they may experience tensile fractures (circumferential and radial to the axis of the dome) induced by the anticlinal movement generated by the salt at depth.

   In the Boring Campaigns in 2009 and 2010 attention has been directed to the character of the chalk, but neither in the CPTs nor in the observations during the core descriptions has any sign of karst phenomena or very dense fracturing been observed. However, two “abnormal observations” have been registered: the first is that thin streaks of “black matter” have been found in quite a number of the chalk cores. Microfossil analyses performed by GEUS indicate that the material could be dark clay from the Æbelø Formation, which is the deposit locally situated directly above the chalk. The second observation is that the chalk in some of the samples shows signs of possibly to have undergone slumping after deposition.

   It was hoped that the televiewer logs would have delivered important information regarding the character and quality of the chalk. However, in only one of the (shorter) chalk borings was the logging with the optical televiewer successful. The logging profiles for this boring showed no “abnormal” structures/fabric.

3. A pattern of normal faults often develop in the layers above and beside a dome. The 1996 seismic investigations and to a lesser extent the 2008 and 2010 seismic investigations have shown a number of smaller faults at the Fehmarnbelt dome. Theoretically, it is a possibility that sudden local movements along faults accompanied by smaller earthquakes may happen. Even though the risk for such movements is evaluated to be very small, it is proposed, when the final alignment has been decided, that the seismic profile for this line should be reinterpreted with special focus on faults.

In this connection it should, however, be observed that the area, as described in chapter 11.3, has been assessed to be a very low seismicity area, and that no significant earthquakes with epicentres close to the Fehmarnbelt dome have ever been registered.

11.3 Seismicity

The region including the Fehmarnbelt area is considered to be a stable continental region, meaning that there are no active tectonic plate boundaries close to the area. Tectonic maps show that the regional tectonics almost only contains historic failed rifts and also historic sutures. However, the area could contain unidentified zones of weakness where future earthquakes could occur. A number of small scale faults have been identified within the Fehmarnbelt Fixed Link alignment during the ground investigation for the project. However, these small scale faults are not of sufficient dimensions to generate significant earthquakes.

A study of readily available seismic hazard assessments carried out for the region has been performed and reported in /7/. The conclusion is very clearly that the Fehmarnbelt location is distant from any active zone and can be considered as located in a very low seismicity area. In Eurocode 8 EN 1998:2004 clause 3.2 states that the provision of EN 1998 does not need to be observed for such areas.

The status as a very low seismicity area has been confirmed by the study of historic earthquakes registered in relevant international and European earthquake catalogues that has been undertaken and reported in /7/. Only one single event from areas within a distance of 150 km from the fixed link is registered. This was an earthquake that occurred in 1629, but according to the USGS Significant Event register there is uncertainty both with regard to the location and magnitude of this event.

It is estimated that the peak horizontal acceleration on rock in the region with a return period of 475 years is in the range of 0.01 g to 0.02 g. Amplification of the bedrock ground motion would be expected to occur in the overlying soils above bedrock. Eurocode 8 recommends site amplification factors of 1.35 to 1.80 depending on the ground conditions. The peak horizontal acceleration at the soil surface with a return period of 475 years is therefore estimated to be in the range of 0.014 g to 0.036 g.

It is noted that the Fehmarnbelt Fixed Link does not readily fall within the scope of normal codes such as EN 1998 and that there is either no Danish National Annex for implementation of Eurocode 8 or national seismic hazard map for Denmark. A project specific seismic design basis will need to be developed.

11.4 Groundwater conditions

It is anticipated to be important for the fixed link project to obtain information on the presence of water bearing sand layers in the upper part of the series of layers in one or both of the coastal zones. From the general knowledge on the Quaternary series of layers it was expected that permeable meltwater sand layers might be present at relevant depths in the investigated area on Fehmarn, while it was evaluated as less probable that such would show up at relevant depths in Lolland (/9/).

11.4.1 Lolland

Most of the land borings on Lolland have shown the expected series of layers with a thin, upper postglacial marine sand layer above a layer of clay till that continues down to the surface of the Palaeogene deposits. An exception to this pattern is boring 09.A.701 which encountered an almost 2.5 m thick meltwater sand deposit between the depths of 15.0 m and 17.5 m. Furthermore in 09.A.702 two very thin meltwater sand layers have been detected in the clay till. To get a better idea if significant waterbearing layers are present, boring 10.A.071 was performed in 2010. This boring passed an approximately 2.5 m thick meltwater deposit at 20 m depth, but this was dominated by clayey silt and contained only a 10 cm thick sand layer. The conclusion is that there are probably rather outspread meltwater deposits in the glacial series of layers in the coastal area of Lolland, but that they only include thin high permeability waterbearing horizons.

Two of the offshore borings, 09.A.018 and 09.A.014 are located rather close to but still more than 1 km from the Lolland coast. 09.A.018 is situated closest to the coast; it has between elevation -18.5 and -33.0 m encountered a thick deposit that seems to be dominated by meltwater sand. Unfortunately, it is uncertain how much meltwater sand is actually present in the unit, as there has been significant core loss ("no recovery") at this location. The same unit seems to be present in 09.A.014 between elevations -23.0 m and -35.3 m. Also in the older boring 96.0.006 situated a little further from land than the above mentioned borings, the special meltwater sand dominated unit is present.

In conclusion, no thick, outspread aquifers are present in the coastal area, but smaller groundwater reservoirs might be present below geological formations that might be present in the coastal area right east of Rødbyhavn on Lolland:

- In the abutment/portal area at Lolland a postglacial marine deposit dominated by sorted, water bearing sand but with significant gyttja layers may be present. The layer can have a total thickness of 4.5 m in the area close to the coast line.
- Furthermore, a thick and outspread unit dominated by meltwater sand layers but interrupted by thin clay till occurrences seems to be present in elevations typically between -22 m and -35 m in the area close to land in the Rødbyhavn area. The layer may be present at the shore as a thin layer of meltwater sand as seen in borings 10.A.071 and 96.0.001.

11.4.2 Fehmarn

Seven borings have been performed on land on Fehmarn as part of the 2009 Boring Campaign; two further borings were added in 2010. Most of the borings have encountered variable thicknesses of up-thrusted floes of Palaeogene clay within the clay dominated till deposits.

According to the borings, layers of meltwater sand/gravel are only present in the glacial till in small parts of the coastal land area. This conclusion is based on the fact that the deep 09.A.603 has met no meltwater sand at all, 09.A.604 has only passed a very thin sand layer at a depth of 42 m (below the thick floes of Palaeogene clay) and 09.A.605 which was taken down to a depth of 50.5 m without reaching the surface of the Palaeogene clay, only has met a 0.6 m thick sand layer. On the western side of the railroad, 09.A.606 has detected a rather thick unit dominated by meltwater sand but including clay till layers at a depth between 8.0 and 12.5 m, while further inland 10.A.610/610A only encountered three very thin sand layers in the clay dominated till. In contrast to the above locations, quite a different situation is encountered in the area where 09.A.601, 09.A.602 and 09.A.607+10.A.607/607A are located. In this area the disturbed series of layers below the upper clay till contains major floes of both meltwater sand/gravel and Palaeogene clay.

In the offshore borings located closest to the Fehmarn coast the only sand detected is Postglacial marine sand in the surface layers in 09.A.001, 09.A.002, 09.A.003, 09.A.004 10.A.051, 10.A.052 and possibly as a very thin layer in 09.A.005. In the older part of the series of layers no sand seems to be present in the nearshore coastal area. Also boring 10.A.072, which was performed especially to look for possible waterbearing sand layers, only found clay deposits below the upper Postglacial marine sand.

There is no indication of sand layers in the geophysical CVES/Mep lines except for in the north-western part of the mapped area where the surface of the glacially disturbed Palaeogene clay deposits is estimated to be at a deep level. This sand layer has been verified by boring 09.A.606 in which sand was found between c. level -8 m and -12 m.

In the area where the borings 09.A.601, 09.A.602, 09.A.607 and 10.A.607/607A are located, rather thick floes of meltwater sand/gravel seem to be present in depths bigger than 10 to 12 m. The horizontal extent of these water bearing zones is not known, albeit sand layers of several hundreds of meters maybe encountered in the area. However, given that the glacial layers are considered to be disturbed, folded and faulted, it is not likely that sand layers of much larger extent may be met.

11.5 The series of layers

11.5.1 Postglacial and Lateglacial deposits

11.5.1.1 Geological description

An overview of the Postglacial/Lateglacial history of the Baltic and the proposed relations between the historic episodes and the deposits in the area is illustrated in Table 11.5.1.1-1.

It should be noted that, except for the Littorina Sea/present day sea stage in Table 11.5.1.1-1, the described episodes and related deposits are only relevant to the areas within the "basin". The reason for this is that the relative water level was so low in the Lateglacial and early Postglacial time that the basin slopes were dry land at that time. No deposits were formed there except for in local lakes/bogs, where freshwater gyttja and peat were deposited. Such a bog has been detected below the present day sea bed in the southern slope area. Finally, in the period for the Ancylus Lake the water level was still rising, and the slope areas nearest to the basin were incorporated in the lake.

Table 11.5.1.1-1 Lateglacial/Postglacial development of the Baltic, including the actual area

Stage	**Period (before present)**	**Typical deposit**
Littorina Sea/Present sea	7,800–now	Sand in exposed areas. Gyttja very locally in the deepest, low oxygen parts.
Ancylus Lake	9,000–7,800	Mostly silty clay. Some-times layered/laminated.
Yoldia Sea, higher salinity	9,500–9,000	Medium plasticity clays, often without visible layering. A few shells. Maybe near-shore sand deposits.
Yoldia Sea, low salinity	10,250–9,500	Mostly silty clay and silt without a distinct layering. Local sand deposits in areas where rivers entered the lake. No shells.
Baltic Ice Lake	13,000-10,250	Distinctly layered/lamina-ted ("varved") clay/silt. Maybe sand deposits in areas for inflow to the sea. Inside the shoreline Allerød peat and freshwater gyttja.
Meltwater stage	15,000–13,000	Meltwater sand and gravel.
End of glaciation in area	15,000	Clay till deposition stops.

11.5.1.2 Geophysical properties

Neither the Postglacial sand nor the basin deposits have been characterised by downhole geophysical log data. The main reason for this is the fact that these deposits are generally located within the length of bore that is stabilised with a steel casing; the steel casing reduces the quality of the data obtained.

11.5.1.3 Geotechnical properties

General

The geotechnical properties in these soil strata have been investigated through:

- Classification testing by Fugro of samples from the type A-borings (/2/, /23/).
- In-situ testing (CPT) by Fugro in the type B-borings (/2/, /23/).
- In-situ testing (CPT) by Fugro in the type C-borings (seabed CPTUs) (/1/).
- Classification testing by GEO of selected samples from the type A-borings for Advanced Laboratory Testing (/27/).
- Advanced geotechnical testing by GEO of selected samples from the type A-borings (/27/).

Details of the geotechnical properties for Postglacial and Lateglacial deposits can be found in Appendix GDR 00.1-001-B.

Classification properties

An overview of the geotechnical classification properties for the Postglacial and Lateglacial deposits appears from Table 11.5.1.3-1 and Table 11.5.1.3-2.

Table 11.5.1.3-1 Basic geotechnical classification properties for Postglacial and Lateglacial deposits

Soil type		w	γ	w_L	w_P	I_P
Postglacial marine sand/gravel	Arithm. mean Standard dev. No. of tests	25.3% 11.0% 67	18.3 kN/m^3 1.6 kN/m^3 19	-	-	-
Postglacial marine/freshwater gyttja and peat	Arithm. mean Standard dev. No. of tests	107.1% 71.4% 45	13.7 kN/m^3 2.0 kN/m^3 30	99.0% 59.5% 18	46.8% 36.5% 18	52.2% 27.3% 18
Postglacial/Lateglacial marine/freshwater clay/silt	Arithm. mean Standard dev. No. of tests	33.6% 14.1% 205	18.6 kN/m^3 1.8 kN/m^3 145	42.4% 14.2% 59	18.3% 4.9% 59	24.1% 10.5% 59
Postglacial/Lateglacial marine/freshwater sand/gravel	Arithm. mean Standard dev. No. of tests	20.2% 4.8% 35	20.5 kN/m^3 2.5 kN/m^3 13	-	-	-

Table 11.5.1.3-2 Additional geotechnical classification properties for Postglacial and Lateglacial deposits

Soil type		d_s	e	Clay T	$CaCO_3$	gl
Postglacial marine sand/gravel	Arithm. mean Standard dev. No. of tests	2.62 0.03 11	0.76 0.5 8	8.2% 8.5% 31	-	1.1% 1.4% 15
Postglacial marine/freshwater gyttja and peat	Arithm. mean Standard dev. No. of tests	2.45 0.31 11	3.41 1.37 15	13.9% 8.4% 36	9.8% 13.4% 4	12.6% 17.5% 17
Postglacial/Lateglacial marine/freshwater clay/silt	Arithm. mean Standard dev. No. of tests	2.67 0.04 30	0.95 0.43 72	30.0% 15.5% 122	17.2% 4.6% 7	2.5% 1.2% 42
Postglacial/Lateglacial marine/freshwater san/gravel	Arithm. mean Standard dev. No. of tests	2.67 0.02 8	0.68 0.20 8	7.8% 8.0% 22	-	0.7% 0.7% 9

The plasticity chart for the Postglacial marine/freshwater gyttja and marine/freshwater clay/silt is included as Fig. 11.5.1.3-1.

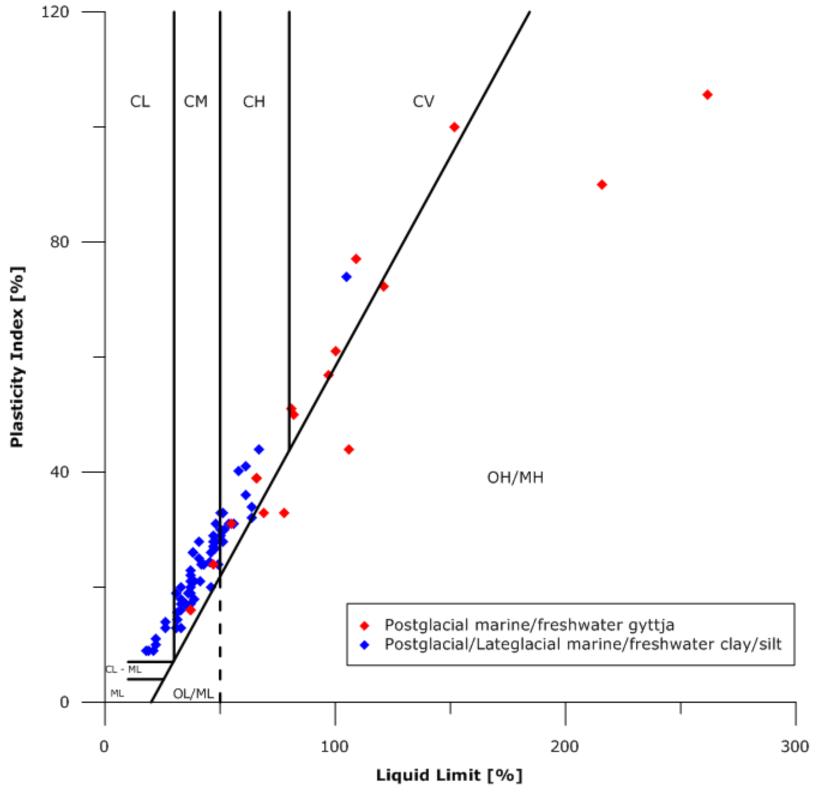

Figure 11.5.1.3-1 Plasticity chart for Postglacial/Lateglacial deposits

CPTU

A considerable number of the type B-borings with CPTU and the seabed CPTUs (type C-borings) have penetrated the Postglacial and Postglacial/Lateglacial deposits.

It appears that q_{net} generally varies:

- Between 0.5 and 20 MPa for the Postglacial marine sand.
- Between 10 and 300 kPa and generally increasing with depth for the Postglacial marine gyttja and peat.
- Between 50 and 1500 kPa for the Postglacial/Lateglacial marine/freshwater clay/silt.
- Between 1 and 30 MPa for the Postglacial/Lateglacial marine/freshwater sand/gravel.

Sand layers with higher values of q_{net} have been observed in several intervals within the Postglacial/Lateglacial marine/freshwater clay/silt.

Stress and stress history

Based on the oedometer tests the pre-consolidation stresses may be assessed. Although these stresses correspond to overconsolidation ratios (OCRs) equal to one or above, these deposits should generally be anticipated to be normally consolidated corresponding to OCR = 1.

The coefficient of earth pressure at rest K_0 has for the Postglacial and Lateglacial deposits been assumed to:

- 0.7 for gyttja, peat and clay/silt,
- 0.5 for sand/gravel.

Consolidation properties
The laboratory coefficient of consolidation c_k has for the deposits been determined between c. 10^{-8} and 10^{-5} m^2/sec.

The compression ratios are varying and the results of the tests are included in Table 11.5.1.3-3.

Table 11.5.1.3-3 Compression ratios for Postglacial and Lateglacial deposits

Soil type		Q
Postglacial marine/freshwater gyttja and peat	Arithmetic mean Standard dev. Number of tests	32.7% 15.0% 5
Postglacial/Lateglacial marine/freshwater clay/silt	Arithmetic mean Standard dev. Number of tests	9.2% 3.7% 6

An initial very rough prediction of the compression ratio Q in % may be obtained through the correlation q_{net}/7 for Postglacial marine/freshwater gyttja and peat and q_{net}/76 for Postglacial/Lateglacial marine/freshwater clay/silt where q_{net} has units of kPa.

The rate of secondary consolidation $C_{\alpha\varepsilon}$ (or ε_s) must generally be expected to be considerable. For a peat specimen $C_{\alpha\varepsilon}$ has been determined to vary between 2.0 and 3.8%/lct, increasing with increasing vertical effective stress level.

The laboratory coefficient of permeability k has been determined at the vertical effective in-situ stress level σ'_{v0} to be:
- Between c. 10^{-9} and $6 \cdot 10^{-9}$ m/sec for the Postglacial marine gyttja and peat.
- Between c. $0.1 \cdot 10^{-9}$ and $2 \cdot 10^{-9}$ m/sec for the Postglacial/Lateglacial marine/freshwater clay/silt.

Static shear strength
The undrained shear strengths as determined in the Advanced Laboratory Testing are summarised in Table 11.5.1.3.3-4.

Table 11.5.1.3.3-4 Undrained shear strength for Postglacial and Lateglacial deposits

Soil type		c_u^C	c_u^E	c_u^{DSS}
Postglacial marine/freshwater gyttja and peat	Arithmetic mean Standard dev. Number of tests	32 kPa 25 kPa 3	-	-
Postglacial/Lateglacial marine/freshwater clay/silt	Arithmetic mean Standard dev. Number of tests	33 kPa 12 kPa 5	13 kPa 1	13 kPa 1

With due regards to the rather few tests and the considerable variation of the cone factors when predicting the undrained shear strength, a cone factor N_{kt} of 20 (related to c_u^C) is suggested to be applied for Postglacial marine/freshwater gyttja and peat as well as for Postglacial/Lateglacial marine/freshwater clay indicating $c_u^C = q_{net}/20$.

The lower bound value of the effective strength parameters for Postglacial marine/freshwater gyttja and peat as well as for Postglacial/Lateglacial marine/freshwater clay can be estimated to $\varphi' = 20.4°$ and $c' = 8$kPa.

Based on both cone resistance in the B-borings and in the seabed CPTs (C-borings) in Postglacial marine sand/gravel and Postglacial/Lateglacial marine/freshwater sand/gravel the typical triaxial peak friction angles have been assessed to $\varphi' = 25°$ to $42°$. The lower part of these friction angles may be due to the presence of silt or clay in the strata or deviations between the interpreted deposit and the deposit actually present at the location of the B-boring and seabed CPTU.

Anisotropy factors have been addressed for a single series of tests and it appears that $c_u^E \approx c_u^{DSS} \approx 0.5\ c_u^C$.

11.5.2 Glacial deposits

11.5.2.1 Geological description

The glacial deposits have been divided into two main till units and two, maybe three, meltwater deposits: There is, however, clear evidence that both of the till units include deposits from more than one glacial event.

The upper till is typically a hard to very hard, sandy to very sandy clay till. Locally it includes meltwater sand layers/floes, and also floes of the lower till seem to be present locally within it. A number of observations indicate that the unit includes two till deposits with remnants of a separating meltwater deposit present locally.

Meltwater sand and silt separate the two till units in parts of the area. The sand varies widely both in grain size and degree of sorting over the area and both coarse, very gravelly sand and fine grained, very silty sand is included in the unit. Because of poor recovery, the more precise composition of the layer is only partly known.

The lower till unit typically consists of medium plasticity clay till, but local occurrences of both high plasticity clay till and very calcareous clay till as well as floes of meltwater sand and of clay of Palaeogene origin is present within it. The high plasticity clay till is almost consistently situated below the medium plasticity clay till with a well defined boundary, this indicates that it probably is a separate “lowermost till” deposited during a separate, older glacial event.

The upper, folded part of the underlying Palaeogene clay could also be regarded as a part of the lower till unit, but the authors have chosen to include it in the Palaeogene Clay Unit, even though it has been affected by glacial disturbance.

It is noted that stones (cobbles) are rare and boulders absent from the borings, however it is important to be aware that these are certainly present in the till deposit.

11.5.2.2 Geophysical properties

In the geophysical log profiles, the lower till exhibits constant medium to high natural gamma values, which are slightly higher than those in the overlying upper till. Porosity and density measurements are fairly constant throughout the profile. Where the lower till is covered by the meltwater sand, the boundary is often marked by a distinct decrease in natural gamma measurements. Where the lower till is immediately overlain by the upper till, the layer boundary is often difficult to define on logs, but usually it appears as a subtle decrease in natural gamma readings and induction conductivity values.

In the logs from borings with floes of Palaeogene clay, this clay is typically characterised by very high natural gamma readings, high neutron porosity and high induction conductivity. There is a clear boundary to the surrounding units when the floes are of considerable size. A good example of this is seen in borehole 09.A.704 where a clay floe is seen between 36 and 40 m depth.

The meltwater sand unit between the two tills is characterised by uneven readings in the gamma density and neutron porosity logs. In general, it has lower conductivities than the underlying till and lower natural gamma radiation values. The boundary to the overlying upper till unit is often fairly subtle on the logs but can be seen as an increase in natural gamma radiation values.

The P-wave velocities measured in the glacial deposits are in Table 11.5.2.2-1 separated into three parts: a part that is mainly floes of Palaeogene clay and upper and lower parts that are mostly till and sand. The differentiation between the upper and lower parts is possible due to the velocities being affected by the burial depth and water content.

The change in measured velocities from the VSP does not always match the interpreted lithology. This could be due to the limited precision of the VSP method and the difficulties with interpreting velocity intervals in grading lithologies.

Table 11.5.2.2-1 Derived Sonic and VSP interval velocities in the glacial deposits.

Geology		Sonic		P-VSP		S-VSP	
		Int. Vel. [m/s]		Int. Vel. [m/s]		Int. Vel. [m/s]	
		Range	Average	Range	Average	Range	Average
Glacial deposits	Upper till	-	-	1300-1800	1400	275-525	375
	Lower till	1770-2140	1900	1875-2075	2000	300-700	450
	Floes of Palaeogene clay	1570-1600	1585	1675-1725	1700	175-400	300

The shear modulus (G_0), Poisson's ratio (ν) and Young's modulus (E) have been calculated using the results from the VSP measurements and the density log. Stiffness values are small strain values, reflecting a drained response.

Table 11.5.2.2-2 Calculated parameters in the different glacial deposits from VSP.

Geology		G_0 [MPa]		Poisson's ratio [-]		Young's module [MPa]	
		Range	Average	Range	Average	Range	Average
Glacial deposits	Upper till	70-680	350	0.419-0.490	0.453	205-1930	1005
	Lower till	85-1090	400	0.433-0.492	0.473	250-3120	1180
	Floes of Palaeogene clay	55-345	150	0.470-0.494	0.486	170-1010	435

The range in velocities reflects the heterogeneity of the deposits which also appears from the full wave form data showing a very inconsistent pattern.

11.5.2.3 Geotechnical properties

General

The geotechnical properties of the Glacial deposits have been investigated through:

- Classification testing by Fugro of samples from the type A-borings /2/ and /23/.
- In-situ testing (CPT) by Fugro in the type B-borings /2/ and /23/.
- In-situ testing (CPT) by Fugro in the type C-borings (seabed CPTUs) /1/.
- Classification testing by GEO of selected samples extracted from the type A-borings for Advanced Laboratory Testing /28/.
- Advanced geotechnical testing by GEO of selected samples extracted from the type A-borings /28/.

Details of the geotechnical properties for Glacial deposits can be found in Appendix GDR 00.1-001-C.

It must be noted that the geotechnical properties for the floes of clays of Palaeogene origin have been described under the geotechnical properties for clays of Palaeogene origin.

Classification properties

Review of the results of the laboratory classification testing, undertaken on samples of the Glacial deposits, confirms the presence of individual Glacial units. These units are particularly well defined from a review of the plasticity chart, plasticity index and clay content test results, where the units fall into groupings or are evident through stepped variations of geotechnical parameters.

Assessment of the plasticity test results reveals a significant variance of plasticity between Glacial units. The test results indicate that the Upper and Chalk till are typically of low plasticity, whilst the Meltwater silt/clay and the Lower till are of low to medium plasticity. The plastic properties of the Lowermost till are noted to be highly variable, and range from medium to very high plasticity.

The plastic and liquid limit results within the Upper till, Chalk till and Lower till are shown to be relatively consistent within each of unit; with the exception of occasional elevated liquid limit results. Few tests were undertaken on samples of the Meltwater silt/clay; with the results indicating irregular ranges of plastic and liquid limits. The results of Atterberg Limits tests undertaken on samples of the Lowermost till reveal a significant range between the liquid and plastic limits which appear variable with depth.

With the exception of the Lowermost Till, the typical values of activity within the Glacial units are less than 1.25, and thus, the soil can be classified as ‘in-active’ or ‘normal’. However, the range of activity values within the Lowermost till typically range between 0.90 and 2.00 indicating proportions of the unit are ‘active’. Given this, the Lowermost Till may be prone to swelling and/or shrinkage with changes in water content.

An overview of the geotechnical classification properties for the Glacial deposits appears from Table 11.5.2.3-1 and Table 11.5.2.3-2.

It should be noted that the unit Meltwater sand includes thin layers of clay tills and very silty meltwater sands.

Table 11.5.2.3-1 Basic geotechnical classification properties of Glacial deposits

Glacial Unit		w	w_L	w_P	I_P	I_L	γ
Upper till	Arithmetic mean	9.6%	20.4%	11.5%	8.9%	-0.28	23.0 kN/m^3
	Standard dev.	1.6%	3.5%	1.6%	3.0%	0.23	1.1 kN/m^3
	Number of tests	467	123	123	123	39	421
Meltwater Silt/Clay	Arithmetic mean	22.4%	38.7%	18.6%	20.1%	0.10	20.7 kN/m^3
	Standard dev.	7.8%	18.0%	8.6%	11.0%	0.02	1.3 kN/m^3
	Number of tests	40	12	12	12	2	25
Meltwater Sand	Arithmetic mean	18.4%	-	-	-	-	21.1 kN/m^3
	Standard dev.	5.6%					1.8 kN/m^3
	Number of tests	78					44
Chalk till	Arithmetic mean	11.9%	22.3%	13.7%	8.6%	-0.36	22.4 kN/m^3
	Standard dev.	4.6%	2.7%	1.9%	2.9%	0.23	1.2 kN/m^3
	Number of tests	74	33	33	33	12	63
Lower till	Arithmetic mean	11.5%	28.1%	12.3%	15.8%	-0.05	22.6 kN/m^3
	Standard dev.	3.2%	9.4%	2.4%	7.8%	0.23	1.0 kN/m^3
	Number of tests	629	185	185	185	63	609
Lowermost till	Arithmetic mean	17.7%	58.3%	18.7%	39.6%	-0.04	21.4 kN/m^3
	Standard dev.	5.9%	22.7%	5.0%	18.9%	0.15	1.4 kN/m^3
	Number of tests	264	94	94	94	18	200

Table 11.5.2.3-2 Additional geotechnical classification properties of Glacial deposits

Glacial Unit		e	d_s	Clay T	Activity	$CaCO_3$
Upper till	Arithmetic mean	0.27	2.65	14.6%	0.61	26.0%
	Standard dev.	0.10	0.04	7.0%	0.27	8.4%
	Number of tests	117	75	224	79	9
Meltwater Silt/Clay	Arithmetic mean	0.58	2.68	20.9%	0.92	-
	Standard dev.	0.11	0.04	11.9%	0.42	
	Number of tests	12	7	25	9	
Meltwater deposits	Arithmetic mean	0.49	2.64	11.9%	-	30.8%
	Standard dev.	0.17	0.04	13.0%		-
	Number of tests	23	16	51		1
Chalk till	Arithmetic mean	0.31	2.68	27.3%	0.44	57.3%
	Standard dev.	0.04	0.04	7.6%	0.38	9.3%
	Number of tests	26	17	36	21	7
Lower till	Arithmetic mean	0.30	2.66	20.2%	0.75	22.3%
	Standard dev.	0.07	0.04	7.8%	0.28	10.1%
	Number of tests	235	98	258	123	29
Lowermost till	Arithmetic mean	0.46	2.66	29.7%	1.38	21.7%
	Standard dev.	0.21	0.03	11.4%	0.67	6.6%
	Number of tests	58	35	136	62	6

The plasticity chart for the Glacial deposits is included as Fig. 11.5.2.3-1.

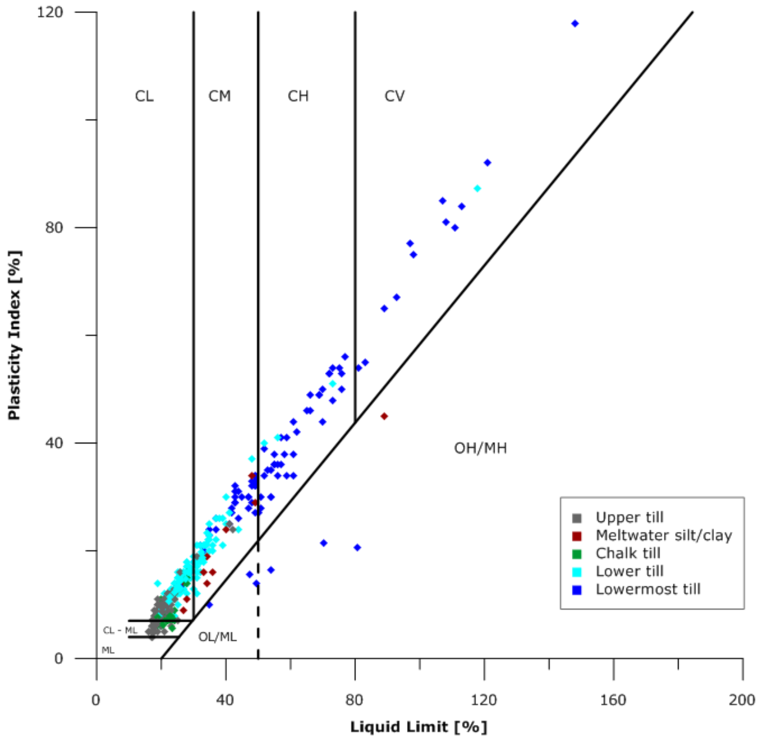

Figure 11.5.2.3-1 Plasticity chart for Glacial deposits

CPTU
In order to identify the geology within cone penetration test profiles, and to allow comparisons between laboratory test results with cone resistance profiles, a number of CPTUs were undertaken adjacent to A-borings. Consequently, the following typical ranges of net cone resistance, q_{net}, have been determined for each of the following Glacial units:

- Upper till – typically between 5 MN/m^2 and 40 MN/m^2 (see following text)
- Meltwater silt/clay – typically between 12 MN/m^2 and 55 MN/m^2
- Meltwater sand – typically between 3 MN/m^2 and 40 MN/m^2
- Lower till – typically between 2 MN/m^2 and 20 MN/m^2
- Lowermost till – typically between 2 MN/m^2 and 10 MN/m^2

It should be noted that a significant number of CPTUs undertaken within the Upper till met refusal. Review of cone resistance profiles suggest that between 40 % and 70 % of CPTU pushes typically met refusal in the Upper till. Given that the area where the piezocone met refusal was drilled out, it is unknown whether these refusals were a result of the piezocone encountering cobbles/boulder obstructions or whether the soil strength exceeds the limit of the test equipment. As a result, the reported upper limit of the typical range of the Upper till should be adopted with significant caution.

An accurate profile of cone resistance could not be determined within the Chalk Till as that the piezocone repeatedly met refusal. As a result, a limited number of net cone resistance values were obtained and are not representative of the strata strength.

Stress and stress history
In order to identify the stress history, a number of oedometer tests were carried out on samples of the till deposits.

It has been attempted to correlate the net cone resistance and the estimated pre-consolidation pressure for the tills. Besides the challenges already pointed out in Chapter 9.2.2, the correlations have also been influenced by the net cone resistance that carries a large scatter with jumps, reflecting stones and variability within the soil formation. In light of this, no correlation between pre-consolidation and net cone resistance can be established for the Upper till. However, with regards to the Lower till, an estimate of the pre-consolidation pressure may be found using the correlation: $\sigma'_{pc} = 0.3 \cdot q_{net}$. This correlation may be used for all the clay till formations applying the rule $q_{net} \leq 8000$ kPa.

Reliable values of pre-consolidation pressure are necessary to establish the over-consolidation ratio of the soils. Therefore, given the issues discussed above, it has been difficult to establish meaningful OCRs for the Upper till. However, using the derived σ'_{pc} values, OCRs were found to range between 5.0 and 25.0 for the Upper Till and between 2.5 and 13.0 for the Lower till.

To establish a coefficient of earth pressure at rest, the results of the incremental loading oedometer tests were reviewed. Although this review concluded no apparent trends, GEO have determined equations to fit the upper and lower bound test results for both the Upper and the Chalk till. For the upper bound results, K_0 is approximately $0.42 \cdot OCR^{0.40}$ and for the lower bound results, K_0 is approximately $0.42\ OCR^{0.28}$.

Consolidation properties

A number of incremental loading oedometer tests have been undertaken on samples of the Glacial units in order to determine the constrained secant oedometer modulus, $E_{oed,sec}$, the coefficient of consolidation, c_k, the compression ratio, Q, the rate of secondary consolidation $C_{\alpha\varepsilon}$ (or ε_s) and the rate of secondary swell, C_{sw}.

It is apparent that the derived values of $E_{oed,sec}$ are markedly different between Glacial units and have been shown to increase with depth. The minimum derived values of $E_{oed,sec}$, with a stress increment not exceeding 500 kPa from the in-situ vertical effective stress, have been shown to be 600 MPa for Upper till, 1114 MPa for Chalk till, 83 MPa for Lower till and 31 MPa for Lowermost till. It should be noted that /28/ has devised a series of trend lines to predict lower bound values of $E_{oed,sec}$ for each Glacial unit. The adjusted trend lines predict $E_{oed,sec}$ values based on the plasticity index of the soil, provided that the unloading stress decrement is between 120 kPa and 500 kPa, cf. Table 11.5.2.3-3.

Table 11.5.2.3-3 Trend lines for lower bound values of oedometric secant modulus, where $E_{oed,sec} = A + B \cdot \sigma'_{unl}$ (kPa) and 120 kPa < σ'_{unl} < 500 kPa

Glacial unit	I_p [%]	A [kPa]	B
Upper till	< 10	$200 \cdot 10^3$	1000
Chalk till	-	$200 \cdot 10^3$	500
Lower till	< 10	$40 \cdot 10^3$	750
	10–14	$20 \cdot 10^3$	750
	14–18	0	750
	> 18	0	500
Lowermost till	-	0	250

The laboratory coefficient of consolidation, c_k, ranges between $2 \cdot 10^{-7}$ and $2 \cdot 10^{-5}$ m^2/sec for the Upper till and between $1 \cdot 10^{-7}$ and $1 \cdot 10^{-5}$ m^2/sec for the Lower till.

The Q values of each Glacial unit were found to be highly variable; with values in the Upper till ranging between 1.9 and 6.0 % and between 3.1 and 7.1 % in the Lower till. However, GEO have established an approximate relationship between Q and plasticity index, I_p. For Glacial till with an I_p less than 9 % the Q value was between 2 and 4 %, and, with an I_p between 10 and 16 % the Q value was between 4 and 7 %. Similarly, samples of till with an I_p of approximately 38 % were found to have a Q value in excess of 8 %.

Review of the creep properties noted a maximum value of $C_{\alpha\varepsilon}$ (or ε_s) along the initial loading curve of 0.26 %/lct and a maximum value of $C_{\alpha\varepsilon}$ (or ε_s) along the reloading curve of 0.09 %/lct. In both the loading and reloading testing, the rate of secondary consolidation was found to increase with increasing effective stress.

/28/ indicates that Glacial deposits with an I_p less than 20 % have no swell potential.

The coefficient of permeability, k (hydraulic conductivity), has been determined from CRS and IL testing at the corresponding vertical effective in-situ stress level, σ'_{vo}, at reloading. These tests have concluded average k values ranging from 4 to $48\cdot10^{-12}$ m/sec for the Upper Till, 2 to $22\cdot10^{-12}$ m/sec for the Chalk till, 6 to $55\cdot10^{-12}$ m/sec for the Lower till and 2 to $4\cdot10^{-12}$ m/sec for the Lowermost till.

Static shear strength

A number of laboratory tests were undertaken on samples of Glacial deposits that were recovered from borings drilled within 5 m of CPTUs. As a result, the geological logs of the type A-boring can be compared to the CPTU profile from the type B-boring and a correlation between the laboratory strength testing results and the q_{net} values made.

In the triaxial tests and direct simple shear tests the range of the undrained shear strength has been determined to 130 to 2300 kPa.

The net cone resistance q_{net} is often considered directly proportional to the undrained shear strength (cf. e.g. /16/). The undrained shear strength is, however, not unambiguously defined. Compared to c_u^C (CAUc-tests), /14/ indicates a cone factor N_{kt} (= q_{net}/c_u^C) ~ 10 for clay till. In ref. /15/ an average cone factor N_k (~ q_c/c_v) ~ 10 has been found for clay till of the Storebælt till type 0-1.

Despite a high variability in the cone factors calculated for the Glacial units in Fehmarnbelt, it is proposed that an N_{kt} (i.e. q_{net}/c_u^C) value of 10 is adopted when predicting c_u^C on basis of q_{net} for the Upper till and Chalk tills. For Lower and Lowermost tills, a value of 11.5 should be considered.

Based on the CPTU profiles and these cone factors both lower and higher undrained shear strengths than the range stated above for the advanced laboratory tests must be expected to be present in the Glacial deposits.

Effective shear strength parameters have been estimated from undrained triaxial compression and extension tests with pore water pressure measurements and from drained triaxial compression tests. The estimates are included in Table 11.5.2.3-4.

Table 11.5.2.3-4 Estimates of effective shear strength properties

Glacial unit	φ′ [°]	c′ [kPa]	No. tests
Upper till	33.4	54	5
Meltwater Sand	37.6	44	3
Chalk till	36.2	99	5
Lower till	36.2	0	8
Lowermost till	31.3	0	1

Danish experience notes that the undrained shear strength of clay till can be related to pre-consolidation pressure and in-situ vertical effective stress using the Shansep procedure. The Upper and Chalk tills have higher undrained shear strengths than the tills to which the Shansep procedure has previously been applied. For the Lower and the Lowermost tills, the undrained shear strength has been measured lower than what would be predicted by the Shansep approach using $\sigma'_{pc} = 0.3\cdot q_{net}$.

Given the limited number of compression, extension and direct simple shear measurements of undrained shear strength at close proximity to one another, and the large variability in the ratios between those values obtained, it is not possible to predict anisotropic factors for individual Glacial units but indeed the tills behave anisotropic. The shear strengths measured indicate that the $c_u^E = 0.65 \cdot c_u^C$, whereas a corresponding link towards the direct simple shear strength has not been found.

Small strain stiffness and damping

Small strain testing of the Glacial units proved difficult to achieve due of the inherent high strength and stiffness of these materials. Because of these constraints a limited amount of laboratory testing has been completed to provide small strain stiffness data for the Glacial deposits and to define how soil stiffness and damping vary with strain. Testing has been limited to specimens of Lower till and Chalk till; no small strain testing has been undertaken for the Upper till, Lowermost till and meltwater deposits.

The laboratory testing is complemented by small strain stiffness values derived from downhole logging using Vertical Seismic Profiling within boreholes through the Glacial deposits.

The test results indicate that small strain stiffness (G_0) derived from VSP logging varies between approximately 200 and 600 MPa. The lower values apply to the Lowermost Till and Lower Till at depths less than about 15 m, while the higher values were measured in Upper Till at depths less than about 15 m. Below 20 m depth the G_0 values generally converge to values generally between 300 and 400 MPa.

Laboratory testing and downhole logging results indicate G_{max}/q_{net} and G_0/q_{net} ratios in the range 20 to 100 with no distinct variation between the Glacial units.

Due to the size of the data set firm conclusions on how stiffness and damping vary with shear strain have not been determined. Normalised plots of G/G_{max} follow the expected 'S' shape profile when stiffness degradation profile is plotted against the log of cyclic shear strain, but the damping ratio values derived from cyclic triaxial tests appear to be high and do not show the expected trend of increasing damping ratio with increasing cyclic shear strain.

Cyclic undrained shear strength

To determine the number of cycles required to fail the Lower till with a combination of average and cyclic shear stresses, i.e. reach 15 % shear strain, six cyclic undrained direct shear tests as constant volume were undertaken.

Simplified diagrams have been used to assess the undrained cyclic shear strength. Such methods have indicated that the undrained cyclic shear strength equals approximately 70 % of the undrained static shear strength when number of cycles to failure N = 10.

11.5.3 Clays of Palaeogene origin

11.5.3.1 Geological description

All the Palaeogene formations from the youngest Søvind marl Formation over the Lillebælt clay Formation, the Røsnæs clay Formation, the Ølst Formation, the Holmehus Formation, the Æbelø Formation and maybe also (traces of) the Kerteminde Marl Formation are present below the Belt. Dating of 163 samples of Palaeogene clays (/6, appendix 1/) has formed the basis for the classification of the clay into Formations as shown on the longitudinal profile in Enclosure I (Drawing no. 070-02-09).

As part of the efforts to build up a reliable geological model for the area, it has been of significant importance to get detailed information on the exact stratigraphic position of the Palaeogene clays from boring to boring. As this cannot be gained solely from visual description, it was decided to perform micropalaeontologic investigation of a significant number of samples to support the visually based correlations.

In several borings it was observed that ash layers are (steeply) dipping. Furthermore, layers from some of the Palaeogene formations are extremely thick compared to what has been observed on other locations with Palaeogene deposits in Denmark. As an example, boring 09.A.002 passed through more than 80 m of Røsnæs clay even though this deposit is less that 30 m thick at all other investigated locations. Moreover, there are situations where two neighbouring borings from top to bottom of the Palaeogene clay layer has been drilled in different formations (boring 09.A.003 Røsnæs Formation and 09.A.010 Ølst Formation), and in boring 09.A.015 and 09.A.015A older layers are situated above younger layers. All those observations indicate that the uppermost part of the Palaeogene clay has been squeezed into giant, sometimes over-kipped anticlines by glacier pressure in the Quaternary period.

In almost all the borings that have been drilled deep into the Palaeogene clay, a marked increase in the CPT cone resistance below elevations between c. -70 m and c. -80 m has been observed. Dating of selected samples has shown that the level of the sudden increase in cone resistance is not related to a specific stratigraphic horizon/boundary but instead is located in different stratigraphic levels from boring to boring. Based on these observations, the boundary is interpreted as the sliding surface at the base for the glacial deformation of the upper part of the Palaeogene layers.

All of the Palaeogene deposits encountered, irrespective of the formation assigned to them, are very high plasticity clays. The clay content is typically 70-80 %, but may be less than 60 % in the Ølst Formation. The plasticity index is typically 50-140 % but slightly higher for the Ølst Formation, c. 90-150 %. Fissures abound, commonly associated with slickensides. A short description of the Palaeogene units is provided below. It is noted that the term “Tarras” often used in older German geological literature is supposed to be synonymous with “Palaeogene clay” and that it further is supposed that “Grüner Tarras” might be synonymous with Lillebælt clay and “Roter Tarras” with Røsnæs clay.

The distribution of the Palaeogene units below the area appears from the longitudinal profile in Enclosure I (Drawing no. 070-02-09). The result of the micropalaeontologic dating is described in /6, appendix 1/.

The *Lillebælt clay* is most often greenish or greenish grey in colour but can include layers with a more reddish tint. It is normally non-calcareous or sometimes slightly calcareous. It contains a few thin ash layers and also concretions orientated in bands are seen. The known thickness of the layer is 40 m in central Jutland (Viborg and Ølst), 70 m at the new Lillebælt Bridge and 100m in the southernmost part of Denmark.

In *Røsnæs clay* olive brown colours dominate but other shades of brown, olive grey and patches of grey and olive also appear. Due to a significant content of micro-shells, the clay is almost always calcareous. Most of the layer has no organic content at all, but two thin bands with a characteristic black colour and rich in organic matter are present in the uppermost part of the unit. The clay contains abundant nodules or concretions (siderite and others), and silty lilac or dark green ash layers appear. The Røsnæs clay layer is in A-boring located at Odder in Jutland 28m thick, which is considered to be the largest thickness measured in Denmark

Clay from the *Ølst Formation* is almost black, dark grey or very dark grey/olive grey, very high plasticity clay. It is almost always non-calcareous, and layers of volcanic ash are common. Burrows are observed and concretions of carbonate and pyrite occur. In boring 96.0.009 a 0.3 m thick carbonate cemented bed is observed at level -91m. The thickness of the Ølst clay layer is typically 10-15m in the Danish area.

Holmehus clay is very high plasticity clay, normally with clear bluish, reddish or, most commonly, greenish colour. It is non-calcareous and contains no ash layers. It is often heavily bioturbated. The thickness of the unit varies from c. 3m at Ølst to 40m in the Odder-area south of Aarhus..

The *Æbelø unit* consists of light grey to grey, non-calcareous, silty to very silty clay or even silt. It contains many silicified layers. The thickness varies between 15 and 60m, and it is often covered by rather thick transition layers to the above Holmehus Formation.

It must be noted that the floes of clays of Palaeogene origin in the Glacial deposits have been described under the geological description for Glacial deposits.

11.5.3.2 Geophysical properties

In borehole 09.A.701 there are three different Palaeogene clay units represented. Of those the Røsnæs unit has a high and increasing gamma radiation reading with depth. The transition to the underlying Ølst unit is marked by a gamma radiation peak and thereafter a saucer like depression trend. The transition from the Ølst unit to the Holmehus unit is more subtle and is mainly seen as an increase in gamma radiation values.

From the log trends it seems possible to distinguish between intervals of intact clays and folded successions. An example is seen in 09.A.002, where the interval below 60 m depth from the B-boring is evaluated to be intact. The folded intervals seem to stand out with more fluctuating natural gamma radiation, induction conductivity and gamma density log responses; the explanation for this is unknown.

The Palaeogene clay has been divided in the Lillebælt, Røsnæs, Ølst, Holmehus and Æbelø clay. The interpreted interval velocities and calculated physical soil properties from the VSP measurements have been correlated with the three Palaeogene clay units. The results are listed in Table 11.5.3.2-1. The change in measured velocities from the VSP does not always match the stratigraphy. This could be due to the limited precision of the VSP method and the difficulties with interpreting velocity intervals in grading lithologies.

Table 11.5.3.2-1 Derived Sonic and VSP interval velocities correlated with geology and age

Geology		Sonic		P-VSP		S-VSP	
		Int. Vel. [m/s]		Int. Vel. [m/s]		Int. Vel. [m/s]	
		Range	Average	Range	Average	Range	Average
Palaeogene clay	**Røsnæs**	1490-1675	1575	1675- 2025	1850	250-450	300
	Ølst 09.A.701	1570	1570	1700	1700	350	350
	Holmehus 09.A.701, 09.A.703	1560-1600	1580	1600	1600	250-350	300

The small strain shear modulus (G_0), Poisson's ratio and Young's modulus have been calculated using the results from the VSP measurements and the density log. The parameters have been calculated for the three Palaeogene clay formations; Røsnæs, Ølst and Holmehus Formations. The Røsnæs clay has been investigated in 4 borings, one on Lolland and three at Fehmarn. The high velocities at around 2000 m/s are only measured, where the boring penetrates less than 6 m of the Palaeogene clay unit and where the overlying sediment is the high velocity till unit. According to these observations it should be noted that the measured velocity could be a little too high. It should also be noted that the velocities measured with the Sonic-log are a bit lower than the measurements from the VSP.

Table 11.5.3.2-2 Calculated parameters in the Palaeogene clay by VSP

Geology		G_0		Poisson's ratio		Young's modulus	
		[MPa]		[-]		[MPa]	
		Range	Average	Range	Average	Range	Average
Palaeogene clay	**Røsnæs**	120-483	317	0.466-0.492	0.472	359-1423	932
	Ølst 09.A.701	157-224	184	0.479	0.479	465-664	545
	Holmehus 09.A.701, 09.A.703	90-155	120	0.483-0.492	0.486	265-455	360

It must be noted that the geophysical properties for the floes of clays of Palaeogene origin in the Glacial deposits have been described below the geophysical properties for Glacial deposits.

11.5.3.3 Geotechnical properties

General

The geotechnical properties in these deposits have been investigated through:

- Classification testing by Fugro on samples from the A-boreholes (/2/ and /23/).
- In-situ testing (CPTU) by Fugro in the B-boreholes (/2/ and /23/).

- Classification testing by GEO of selected samples from the A-boreholes for Advanced Laboratory Testing (/29/).
- Advanced geotechnical laboratory testing by GEO of selected samples from the A-boreholes (/29/).

Details of the geotechnical properties for clays of Palaeogene origin can be found in Appendix GDR 00.1-001-D.

Classification properties

An overview of the geotechnical classification properties for clays of Palaeogene origin appears from Table 11.5.3.3-1 and Table 11.5.3.3-2.

Table 11.5.3.3-1 Classification properties for the intact soil units.

		Røsnæs	Ølst	Holmehus
Water content, w	μ	32.5%	41.6%	32.5%
	σ	2.7%	6.9%	0.9%
	N	254	72	4
Total saturated unit weight, γ	μ	18.9 kN/m^3	18.2 kN/m^3	19.7 kN/m^3
	σ	0.9 kN/m^3	0.9 kN/m^3	2.4 kN/m^3
	N	277	69	4
Plastic limit, w_P	μ	31.7%	40.6%	30.5%
	σ	3.3%	8.0%	0.7%
	N	72	18	2
Liquid limit, w_L	μ	114%	154%	117%
	σ	22.2%	20.4%	0.7%
	N	72	18	2

μ: Arithmetic mean value, σ:Standard deviation, N: Number of data.

The plasticity chart for the clays of Palaeogene origin is included in Fig. 11.5.3.3-1. The A-line and division in dependency of plasticity is shown. It appears that the major part of the clays belongs to clay of very high plasticity.

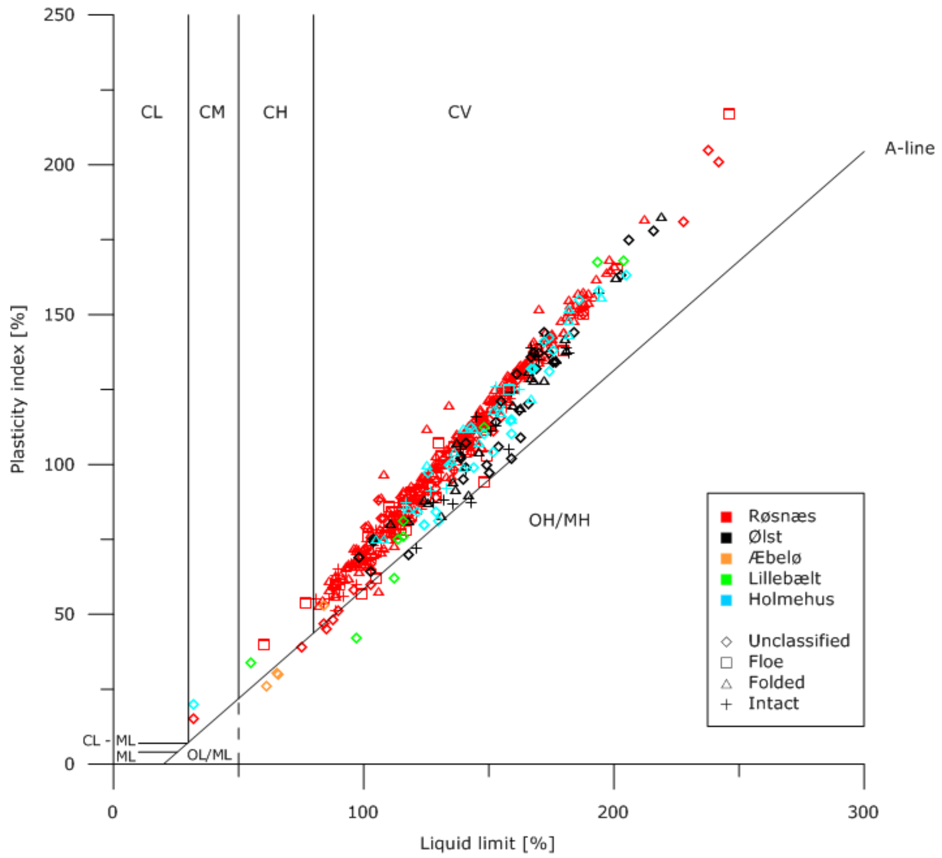

Fig. 11.5.3.3-1 Plasticity chart for clays of Palaeogene origin

Table 11.5.3.3-2 Classification properties for the folded soil units together with floes.

		Røsnæs	Ølst	Holmehus
Water contents, w	μ	36.5%	43.7%	37.4*/39.8%
	σ	4.6%	8.6%	5.4*/6.8%
	N	1151	153	107*/89
Total saturated unit weight, γ	μ	18.5 kN/m³	18.2 kN/m³	18.6*/18.1 kN/m³
	σ	0.8 kN/m³	0.9 kN/m³	0.9*/1.1 kN/m³
	N	1167	130	86*/73
Plastic limit, w_P	μ	31.7%	39.5%	35.3*/37.7%
	σ	4.9%	6.9%	5.0*/8.4%
	N	335	53	22*/26
Liquid limit, w_L	μ	136%	156%	151*/148%
	σ	32.0%	28.3%	21.8*/34.6%
	N	335	53	22*/26

μ: Arithmetic mean value, σ: Standard deviation, N: Number of data.
*: All results for borehole 10.A.058 have been classified as intact/folded Holmehus and belongs therefore (as by definition) to unclassified Holmehus. Due to the near surface location of unclassified Holmehus in borehole 10.A.058, this borehole is therefore presented separately.

CPTU

A considerable number of the type B-borings with CPTU have penetrated the clays of Palaeogene origin. Considering the net cone resistance, the following tendencies are found:

- The net cone resistance increases with depth for the folded Røsnæs formation, starting at 0.3-0.5 MPa at seabed level and increases to a value not exceeding 3.5 MPa at approximately 30m depth.
- A similar trend is seen for the folded Ølst formation and within borehole 10.A.058 (Holmehus), but for these two deposits, the net cone resistance is generally higher than found in the corresponding depth of the Røsnæs formation.

The net cone resistance has been used together with the undrained shear strength in compression to establish the cone factor $N_{kt} = q_{net}/c_u^C$, see /16/. The normalised cone resistance is defined by $Q_t = q_{net}/\sigma'_{v0}$ where σ'_{v0} is the estimated vertical effective in-situ stress. Assuming that the cone factor, N_{kt} is constant within one clay unit, e.g. folded Røsnæs clay, the normalised cone resistance may be expressed by $Q_t = N_{kt} \cdot c_u^C/\sigma'_{v0}$ and Q_t therefore links directly to the Shansep relation through the equations:

$$Q_t = N_{kt} \cdot A \cdot OCR^B = N_{kt} \cdot c_u^C/\sigma'_{v0}$$

where “A” and “B” are constants for a site-specific soil type.

The following tendencies are found for the folded and the intact Røsnæs-formation:

- For the folded Røsnæs clay Q_t decreases with depth, provided that the formation is NOT overlain by a different deposit, e.g. clay till.
- If the folded Røsnæs formation is overlain by a different soil unit, Q_t is more or less constant with depth, meaning that OCR is approximately constant.
- For the intact clay, Q_t is generally constant but the value may change slightly from borehole to borehole.

These tendencies have been illustrated on Fig. 11.5.3.3-2.

Stress and stress history

The pre-consolidation pressure can be estimated, but with the rate of secondary compression increasing with increasing stress, the virgin curve in a bilogarithmic plot will generally not be a straight line. An uncritical use of standard methods for estimating σ'_{pc} may therefore over-estimate the pre-consolidation pressure. The method of Janbu is the preferred method if applicable.

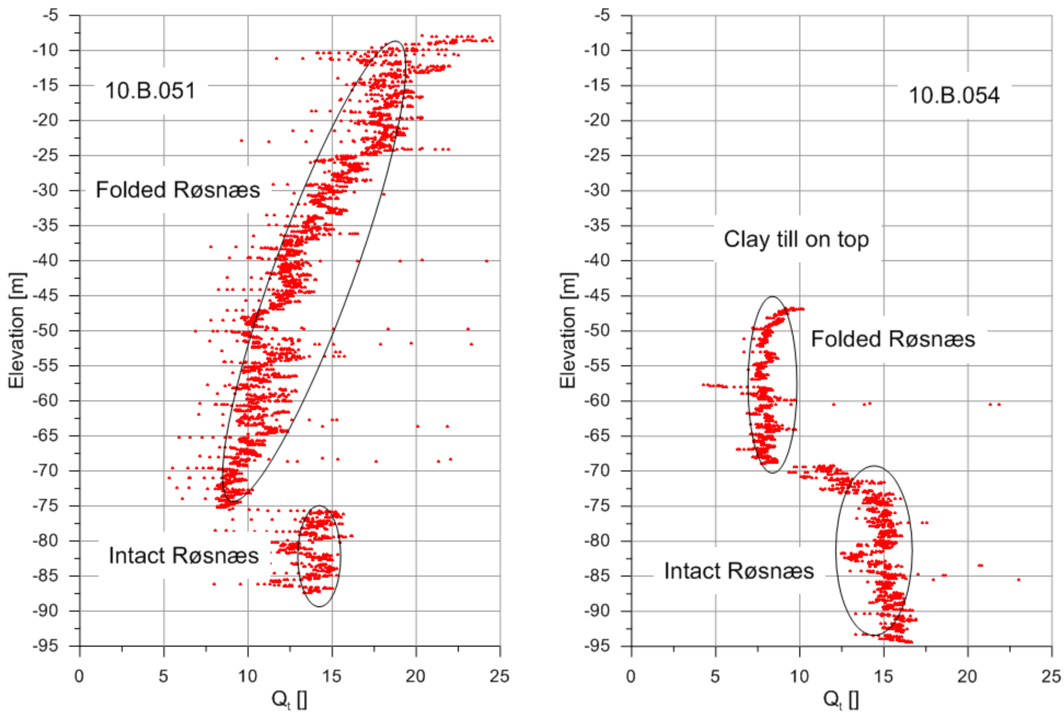

Figure 11.5.3.3-2 Normalised cone resistance, Q_t, versus depth for boreholes 10.B.051 and 10.B.054 where: 10.B.051: Folded Røsnæs from seabed and 10.B.054: Glacial and earlier deposits located between seabed to elevation -47 m.

The conclusion on folded and intact Røsnæs clay is that the mean value of the pre-consolidation pressure can be estimated by:

$$\sigma'_{pc} = \sigma'_{v0} \cdot [0.183 \cdot q_{net} / \sigma'_{v0}]^{1.287}$$

Laboratory test data from Holmehus, Ølst and floes of the different formations show that σ'_{pc} for these deposits can be estimated using the same equation.

Some soils exhibit a clear and well-defined “soil skeleton collapse” when σ'_{pc} is exceeded. The lack of a clear “soil-skeleton-collapse” mechanism during Oedometer testing on Palaeogene clay combined with variations in the estimated σ'_{pc} from tests having been performed on the same core indicates that the cores are disturbed. Studying the CPTU-measurements along different core runs reveals an unexpectedly high variation in the net cone resistance, within depth intervals as little as 0.05 m. This finding supports the variation in laboratory test results within one and the same core. In addition, sample disturbance of the individual specimens have been evaluated using the NGI method (X-ray). From this evaluation it appears that the majority of specimens tested are “very good to excellent”.

Based on the CPTU-evaluation and the investigation of sample disturbance on a specimen level, it is indicated that the material within the cores are well preserved and of comparably quality as the soil in the nature. It is, however, believed that the folded parts of the clay of Palaeogene origin do not react as a “typical soil”. It carries its own degree of disturbance due to its folded nature and as such, it adds an additional consideration within the discussion on sample disturbance.

It may be expected that the soil encountered in the A-boreholes do not have the same properties as the soil in the corresponding depth of the neighbouring B-borehole. This is supported by the ash-layers inclined with up to 60° as marked on some of the borehole profiles. Correlation between laboratory test results and the cone resistance has been performed using the same depth in the A and the B boreholes. Although this seems to be the only possible way through a correlation, the approach will call for a scatter in itself.

An estimate of the coefficient of earth pressure at rest is given. The estimate is based on a series of tests performed, coupling K_0 with OCR, which again is correlated with the net cone resistance. The following equation represents an approximate mean value of the K_0-measurements performed: $K_0 = 0.548 \cdot [OCR]^{0.515}$.

The principles addressed above are based on Oedometer testing. Extracting the normalised cone resistance from the CPTU’s on each location of the performed Oedometer tests implies that a normalised undrained shear strength c_u^C/σ'_{v0} may be attached to each Oedometer test, provided that a realistic cone factor is known. In this way a SHANSEP relation may therefore be established from the Oedometer tests directly. This SHANSEP relation from Oedometer tests fits well with the SHANSEP-relation, established by running undrained triaxial tests in a way, where the individual tests are sheared from a known over-consolidation ratio. Finally, the undrained triaxial tests consolidated for an effective in-situ stress state leads to a normalised shear strength that fits well with the normalised cone resistance, so the data are consistent.

Consolidation properties
Assuming that the in-situ pore pressure in the soil within the Fehmarnbelt is equal to the hydrostatic pore pressure, the in-situ state in the soil is difficult to model accurately when investigating consolidation properties. In nature there are no pore water gradients. In the laboratory, the soil will always exhibit some sort of volume change unless each load step is allowed to stay for two to three weeks. The difference between the laboratory behaviour and the behaviour in the nature is most likely small if a load is gradually increased during the test. However, first loading and then unloading (stress reversals), may imply that what is measured in the laboratory will not be fully representative for how the soil in the nature responds.

Approximate values of the constrained secant modulus for loading have been given for the folded formations. These values range from 20 to 60 MPa, but the values will vary depending on the loaded area, the load spread assumed and the load applied. The rate of secondary compression amounts to approximately 0.1-0.3% per log cycle of time.

During unloading of the soil, the constrained tangential Oedometer modulus has been addressed. When the specimens are sufficiently unloaded, the stress-strain relationship maps into a line for unloading, resembling the compression ratio, but with an inclination of 4-5% per log cycle of time. The rate of secondary swell is very dependent on the unloading stress relative to the estimated vertical effective in-situ stress; as the stress on the specimen decreases, the rate of secondary swell increases. Apparently the rate of secondary swell may exceed the rate of secondary compression by a factor of 2-4.

The range for the mean values of the coefficient of vertical consolidation extracted from the unloading step reaching the vertical effective in-situ stress is 0.1-0.2 m^2/year. The range for the mean value of the compression ratio for stress levels in excess of 2500 kPa is 10-20 %.

Static shear strength
The variability in the soil conditions is found when trying to correlate the undrained in-situ shear strength with other measurements/parameters.

The normalised undrained shear strength versus the laboratory controlled overconsolidation ratio OCR may be estimated as $c_u^C/\sigma'_v = 0.209 \cdot [OCR]^{0.777}$.

Effective shear strength properties have been extracted using CADc tests and CAUc tests. When doing a detailed check of the test enclosures for CAD-testing within e.g. folded Røsnæs Clay, some tests keeps dilating, others reveal a peak shear strength and some tests even show constant volume along the last part of the shearing phase (critical state). There appears to be no systematic trend in the results obtained. The mean value of the effective shear strength properties have thus been estimated by considering all CAD and CAUc tests allowing for an evaluation of effective shear strength properties as one population. The arithmetic mean values of the triaxial peak friction angle and the effective cohesion are 19.6° and 14 kPa, respectively.

Anisotropy ratios have been addressed, but it appears that the undrained shear strengths in compression, in extension and in direct simple shear are of the same average size.

The cone factor N_{kt} (i.e. q_{net}/c_u) has been established for the different formations, but there seems to be no significant difference. Comparing the estimated average N_{kt} for the different formations will show variations, but the magnitude of the variation is more or less directly linked to the number of tests performed within the relevant formations. The only exception is that the arithmetic mean value of $N_{kt, DSS}$ (23.9 for 31 numbers) in the folded Røsnæs formation is approximately 10 % lower than the arithmetic mean value of $N_{kt, CAUc}$ (26.1 for 45 numbers). This difference is believed to represent a strain rate in DSS exceeding the strain rate in undrained triaxial testing by a factor of 20. The strain rate has been investigated by separate DSS-testing, but these tests do not allow for clear conclusions to be drawn.

Until the results from Large Scale Testing become available, it is decided that:

- The soil exhibits a positive rate effect being 7 % shear strength increase per log cycle strain rate.
- The undrained direct simple shear strength, as measured in this test programme, shall be multiplied by 0.916 to comply with the undrained triaxial testing performed.
- Anisotropy factors are unity.

Treating all test types equally, implies that the undrained in-situ shear strength can be estimated using an arithmetic mean value N_{kt} = 26.9 (114 numbers) and a standard deviation of 9.1. The distribution is illustrated in Fig. 11.5.3.3-3. The geometrical mean value should give the best fit cf. /15/ and is N_{kt} = 25.5. These results cover Røsnæs, Ølst and Holmehus formations, whereas the one measurement in Æbelø is excluded.

The undrained in-situ shear strength will be reduced when the soil is unloaded. The established SHANSEP model seems to capture this effect, provided that OCR is defined using the reduced effective vertical stress.

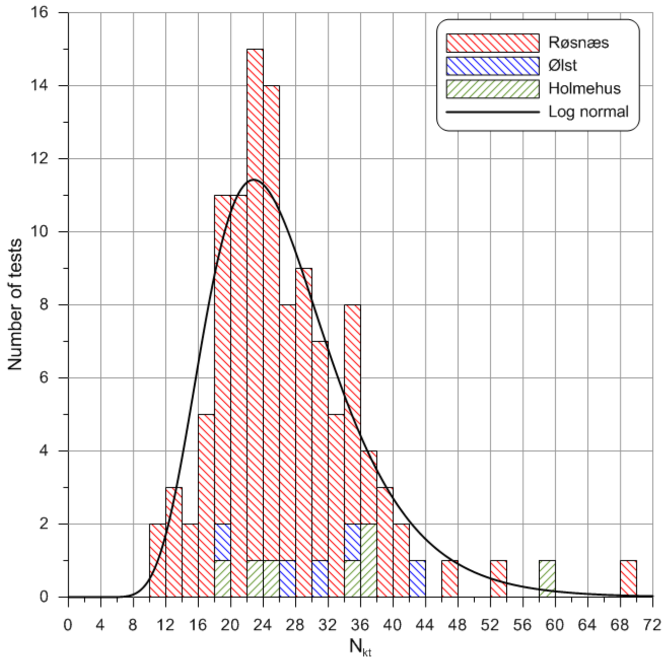

Figure 11.5.3.3-3 Correlation between CPTU-measurements and laboratory tests leading to an undrained in-situ shear strength (CAUc, CAUe and DSS). The number of tests is shown versus the cone factor estimated. Both floes, folds and intact deposits have been included.

Small strain stiffness and damping

The small strain shear modulus has been measured in laboratory using resonant column testing and by using bender elements installed in triaxial and resonant column test specimens. Bender elements have not provided reliable estimates of shear wave velocity in the 1:1 height to diameter test specimens used for CAU triaxial testing. Bender element measurements made on 2:1 height to diameter specimens used for cyclic triaxial and resonant column testing appear more reliable and provide estimates of shear modulus that are consistent with values derived from resonant column testing. Bender elements in 2:1 height to diameter specimens also provide shear wave velocity values consistent with those obtained from geophysical downhole logging using Vertical Seismic Profiling within boreholes through the same geological unit to that from which the specimens for testing were obtained.

The test results show that to a depth of about 30 m below ground level the small strain shear modulus (G_{max}) within the Palaeogene increases approximately linearly with depth to a value of about 100 MPa at 30 m depth. The downhole logging indicates that below 30 m G_0 appears to be essentially constant with depth at values ranging up to 400 MPa. G_{max} increases if the consolidation stresses increase, but the increase is relatively modest. Increasing the vertical effective stress by 50 %, for example, results in an approximate 10 % increase in G_{max}.

At depths less than about 30 m below ground level the test data indicates a ratio of G_{max} to net cone resistance, q_{net}, in the range 20 to 40. Downhole logging indicates G_0/q_{net} ratio at greater depth in the range 50 to 200. Under cyclic loading conditions the same non-dimensional stiffness degradation curve relating G/G_{max} to cyclic shear strain can be used for all Palaeogene materials.

Cyclic undrained shear strength

The folded Røsnæs and Ølst formations apparently have a cyclic shear strength higher than the static shear strength using the direct simple shear strength derived from q_{net} with N_{kt}=26 as a reference, provided that the equivalent number of load cycles do not exceed 10.

The Holmehus formation deviates in this respect, apparently being significantly weaker. Cyclic loading will imply a shear strength degradation. The cyclic shear strength is 40 % lower than the static shear strength, provided that the equivalent number of load cycles do not exceed 10.

If more detailed information is needed, the test results must be further addressed and supplementary testing may be initiated. It must finally be emphasised that Deltares has performed the cyclic DSS tests for the Røsnæs and Ølst formations using active height control, whereas the Holmehus tests are performed by Fugro/Houston, using passive height control.

The test results established when cycling a partly unloaded specimen indicates that moderate load amplitudes may not necessarily imply a shear strength reduction. In this case the shear strength reduction is relative to the shear strength as found after the soil has reached end of primary for the unloading stress in question.

Special evaluations and investigations
Special investigations and evaluations have been performed for clays of Palaeogene origin at the old Lillebælt Bridge and at the Fehmarnsund Bridge. The ground conditions at these two sites are comparable to the ground conditions in part of Fehmarnbelt Fixed Link alignment. Hence the borings performed serve as correlation boreholes for sites with similar ground conditions to part of Fehmarnbelt and documented behaviour during full scale and long term loadings.

Background, evaluations and details of the geotechnical properties for clays of Palaeogene origin at Lillebælt and Fehmarnsund can be found in Appendix GDR 00.1-001-F.

The geotechnical properties for the clays at these two sites have been investigated through:

- Classification testing by Fugro of samples from the type A-borings in Lillebælt (/17/).
- In-situ testing (CPTU) by Fugro in the type B-borings in Lillebælt /17/).
- Classification testing and field vane testing by GEO in the type A-boring below Pier 3 of the old Lillebælt Bridge in Lillebælt (Appendix GDR 00.1-001-F).
- Classification testing by GEO of samples from the type A-boring at Fehmarnsund (/34/).
- In-situ testing (CPTU) by GEO in the type B-boring at Fehmarnsund (/34/).
- Classification testing by GEO of selected samples from the type A-borings in Lillebælt and at Fehmarnsund for Advanced Laboratory Testing (/31/).
- Advanced geotechnical testing by GEO of selected samples extracted from the type A-borings in Lillebælt and at Fehmarnsund, cf. /31/.

The overall objective with the investigation in Lillebælt was to evaluate whether or not the bridge load has been fully or partly transferred to changed effective stresses within the clays of Palaeogene origin below the pier 3. The existing bridge piers have since c. 1932 undergone considerable settlements from c. 0.25 m for Pier 1 to more than 0.5 m for pier 3 and other piers (/41/).

The overall objective with the investigation at Fehmarnsund was to evaluate whether or not the load from the northern embankment has been fully or partly transferred to changed effective stresses within the floes of Palaeogene Røsnæs/Lillebælt clay beneath the embankment. The observed total settlement from 1961 to 1971 of the northern embankment is around 0.8 m. The settlements within the last 40 years have not been monitored (/41/). The model for estimating settlements, to be applied for the Fehmarnbelt Fixed Link, will be influenced by the soil response.

In conclusion the test results indicate that the effective stresses below Pier 3 of the old Lillebælt Bridge and below the northern embankment at the Fehmarnsund Bridge are affected and have been increased due to the weight of Pier 3 at the old Lillebælt Bridge respectively the weight of the northern embankment at the Fehmarnsund Bridge.

Another special investigation of the clays of Palaeogene origin included laboratory mixing, cement stabilisation and unconfined compression testing of this material (/32/).

11.5.3.4 Large Scale Testing

The Large Scale Testing at the site in shallow waters off the Fehmarn coast is ongoing and the results are being reviewed. The observations concern the behaviour of the folded Røsnæs clay. Three main topics are investigated:

- Monitoring the degree of sedimentation in the trial excavation and the slope stability through multibeam surveys as illustrated in Fig. 11.5.3.4-1.
- Monitoring displacements of and pore pressure developments in and below the bottom of the trial excavation by means of the installed extenso-piezometers as illustrated in the Fig. 11.5.3.4-2, Fig. 11.5.3.4-3 and Fig. 11.5.3.4-4.
- Monitoring the development in tension load capacity with time of the installed driven and bored piles as illustrated in the Fig. 11.5.3.4-5.

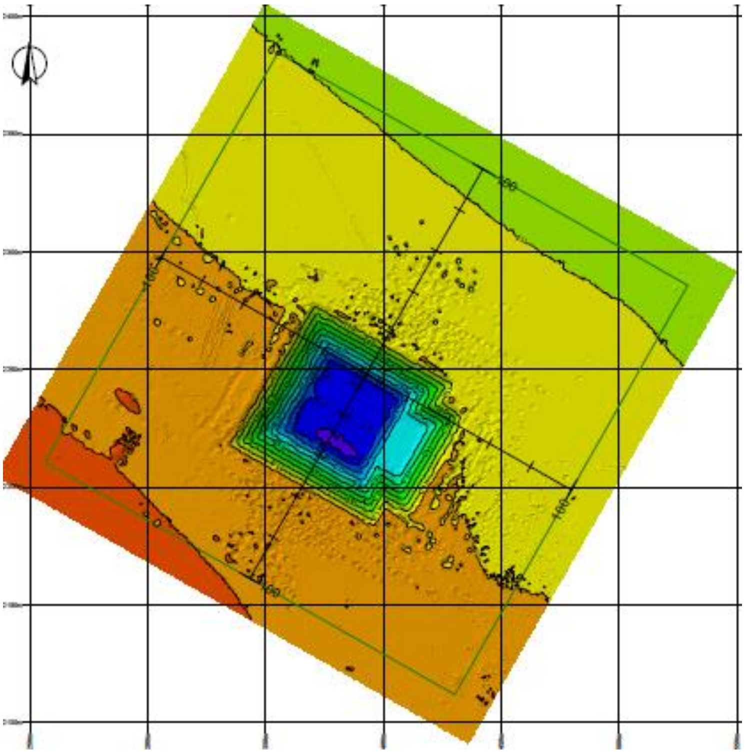

Figure 11.5.3.4-1 Multibeam survey 2010-10-14 of Large Scale Testing site

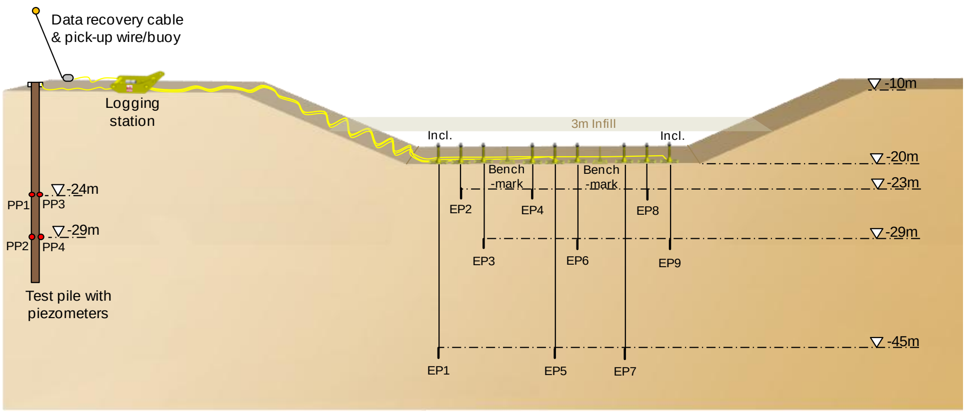

Figure 11.5.3.4-2 Overview of monitoring system with extenso-piezometers (EP1-EP9) and test pile piezometers (PP1-PP4)

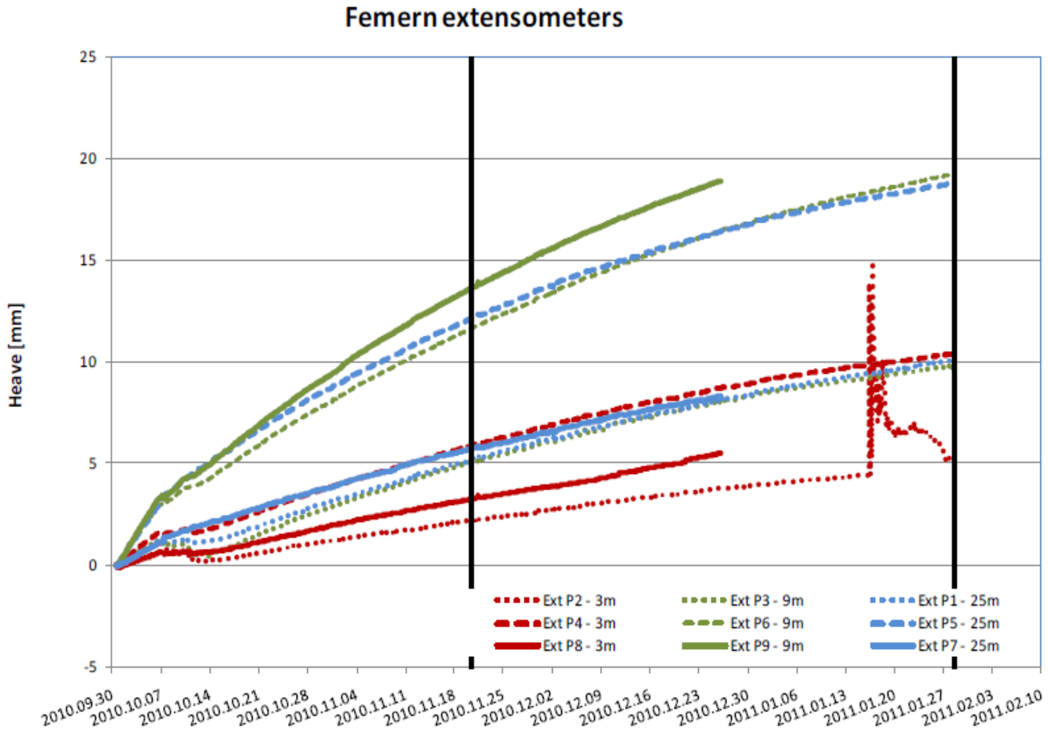

Figure 11.5.3.4-3 Extensometer data collected so far. All values are presented as relative to measurements on 2010-09-30 being the date of starting logging.

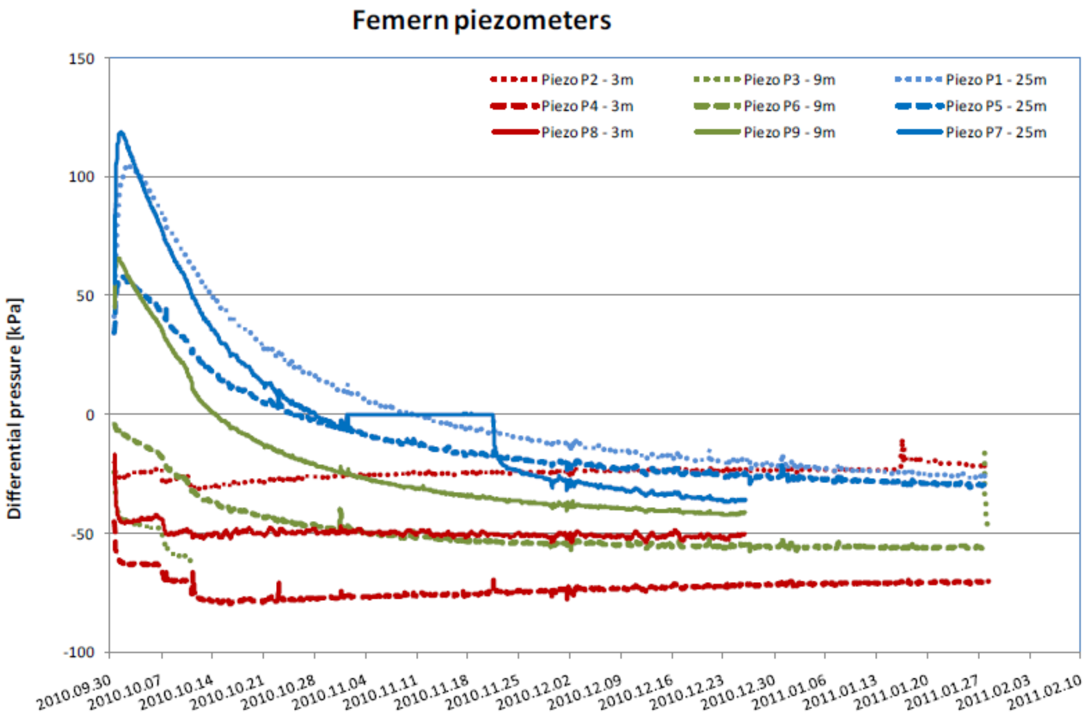

Figure 11.5.3.4-4 Piezometer data collected so far. All values are presented as relative to the hydrostatic pressure.

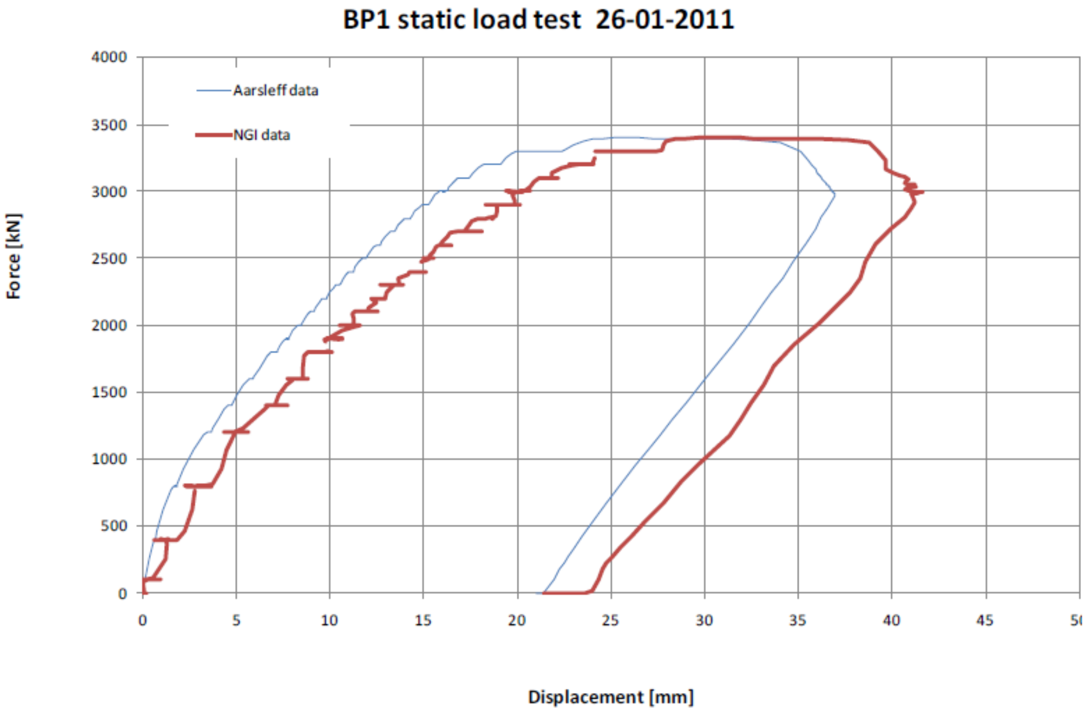

Figure 11.5.3.4-5 Load versus displacement during pile test of bored and in-situ cast concrete pile BP1 on 2011-01-26, with displacement measurements from Aarsleff and NGI

11.5.4 Cretaceous chalk

11.5.4.1 Geological description

Twelve of the 2009/2010 borings reached the chalk with borehole boring 09.A.008 taken almost 75 m into the deposit. As expected from the 1996-investigation, the chalk is typical Danish white chalk as known from Møns Cliff and a number of other locations. It has in all the borings been described as a muddy, slightly sandy white chalk with a minor content of flint nodules, distributed in layers. The degree of induration is almost consistently H2. Only in boring 09.A.008 a significant layer of H3 chalk has been described from the lowermost part of the borehole.

The chalk has typically average flint content less than 5 %. The flint appears to be present as nodules and not as plates, but the nodules are typically concentrated in layers. However, one of the borings has passed a flint body on 25 cm.

The age of the chalk on the boring profiles is written as Maastrichtien. A sample has been taken for dating from the uppermost chalk and from the bottom of boring 09.A.008. The upper sample was from middle Maastrichtien, the lowermost one was dated to the upper Campanian. Two further samples from the upper and lower part of the chalk in boring 09.A.019 have been analysed; they were respectively of upper and middle Maastrichtien age. As it is not possible by visual inspection with certainty to observe a difference between chalk of the two ages, and as it is not considered to be important from an engineering perspective to distinguish between chalk from the two periods, it has been decided to maintain the age description as Maastrichtien for all the chalk in the borings.

Observations of “black matter” in the chalk has been described and discussed in section 11.2.

11.5.4.2 Geophysical properties

On Lolland two onshore seismic lines have been recorded, one North–South (NS) and one West–East (WE). At the NS onshore seismic on Lolland, the top of Cretaceous chalk unit is a very distinct reflector in the northern part of the survey area; in the southern part, near to Rødby harbour, the reflector is less distinct. The reflector is southwards dipping from a depth of c. 60 ms Two-Way-Time (TWT) in the northern end of the line to c. 145 ms TWT in the southern end of the line. The unit is characterised by distinct parallel internal reflectors.

One seismic line on Fehmarn has been recorded in the NS direction and two lines in direction WE. The units on the NS line correlated with the marine seismic units and with borehole data offshore. On all three seismic lines the Cretaceous chalk surface is a very distinct reflector. On the NS line the reflector is dipping southwards from c. 235 ms TWT at the northern end of the line to c. 270 ms TWT at the southern end of the line. In the WE line close to the Fehmarn coast the reflector is gently dipping westwards from c. 265 ms TWT at the eastern end of the line to c. 255 ms TWT at the western end of the line. In the other WE line the reflector is semi horizontal at a depth of c. 250 ms TWT. The unit is characterised by distinct or rather distinct parallel internal reflectors.

The Chalk is very easy to recognize on the geophysical borehole logs; very low natural gamma radiation readings, high neutron porosities and high induction conductivities. The boundaries to the overlying units are very sharp, with an abrupt decrease in natural gamma radiations readings. The unit has normally very constant log-values, but in the boreholes 09.A.008 and 09.A.017 there are small intervals with higher natural gamma radiation readings.

11.5.4.3 Geotechnical properties

General

The geotechnical properties in the chalk deposit have been investigated through:

- Classification testing by Fugro of samples from the type A-borings (/2/ and /23/).
- In-situ testing (CPT) by Fugro in the type B-borings (/2/ and /23/).
- Classification testing by GEO of selected samples from the type A-borings for Advanced Laboratory Testing (/30/).
- Advanced geotechnical testing by GEO of selected samples from the type A-borings (/30/).

Details of the geotechnical properties for Cretaceous chalk can be found in Appendix GDR 00.1-001-E.

Classification properties

All chalk cores have been logged and described in accordance with the Danish Geotechnical Society Bulletin 1 with the degree of induration as detailed in Table 11.5.4.3-1. The classification in accordance with the International Society of Rock Mechanics (ISRM) has been included for comparison only. The distribution of the degree of induration for the recovered cores appears also from the table.

Table 11.5.4.3-1 Rock classification system and %of cores with varying levels of induration- Chalk

<table>
<tr><th colspan="2">ISRM Rock Classification</th><th colspan="2">DGS Bulletin 1 Classification</th><th rowspan="2">Distribution of degree of induration (%)</th></tr>
<tr><td>Rock Grade</td><td>Description</td><td>Degree of Induration</td><td>Description</td></tr>
<tr><td>R0</td><td>Extremely weak</td><td>H1</td><td>Unlithified</td><td>6.0</td></tr>
<tr><td>R1</td><td>Very weak</td><td>H2</td><td>Slightly indurated</td><td>87.8</td></tr>
<tr><td>R2</td><td>Weak</td><td>H3</td><td>Indurated</td><td>5.8</td></tr>
<tr><td>R3</td><td>Medium strong</td><td rowspan="2">H4</td><td rowspan="2">Strongly indurated</td><td rowspan="2">0.4</td></tr>
<tr><td>R4</td><td>Strong</td></tr>
<tr><td>R5</td><td>Very strong</td><td rowspan="2">H5</td><td rowspan="2">Flint</td><td rowspan="2">0</td></tr>
<tr><td>R6</td><td>Extremely strong</td></tr>
</table>

The chalk is predominantly slightly indurated H2 (88 % of the recovered cores) with a small percentage of H1 and H3 material. Results of all classification testing on chalk specimens are summarised in Table 11.5.4.3-2.

Table 11.5.4.3-2 Basic geotechnical classification properties for Chalk

Property	Depth range	No. of results	Arithmetic mean value	Standard deviation
Water content, w	0–30 m	327	33.3%	3.1%
	Below 30 m	61	28.8%	3.4%
Carbonate content, $CaCO_3$	-	62	95.0%	1.7%
Specific gravity of solids, d_s	-	126	2.69	0.04
Dry density, ρ_d	-	310	1.41 Mg/m^3	0.05 Mg/m^3
Void ratio, e	-	310	0.89	0.11
Porosity, n	-	310	0.47	0.03
Saturated unit weight, γ_{sat}	-	456	18.7 kN/m^3	0.8 kN/m^3
Effective unit weight, γ'	-	456	8.7 kN/m^3	0.8 kN/m^3

CPTU

Nine of the type B-borings with CPTU have penetrated into the chalk. The majority of the chalk material is generally categorised by a net cone resistance (q_{net}) between 10 and 15 MPa and a friction ratio (R_f) between 1.5 and 3.0%. Based on the borehole logging these parameters are assumed to represent the H2 material.

There are local layers with higher q_{net} values in excess of 30 MPa, and occasionally there are instances where refusal was met during testing. These are likely to be the stronger H3 and H4 layers and the flint bodies identified on the borehole logs.

Stress and stress history
Yield stress data derived from CRS oedometer testing is summarised in Table 11.5.4.3-3. Yield stress values σ_{yield} are high relative to the in-situ vertical effective stress σ'_0 and when correlated against a lower bound CPTU q_{net} profile the ratios of q_{net}/σ_{yield} are as given in Table 11.5.4.3-3. The minimum yield stress determined was 1900 kPa.

Table 11.5.4.3-3 Yield stress data for chalk

Property	No. of data points	Average value	Standard deviation
Yield stress (σ_{yield})	22	3905 kPa	1399 kPa
σ_{yield}/σ'_0	22	7.7	2.8
q_{net}/σ_{yield}	18	3.2	0.9

Consolidation properties
Oedometer modulus values derived from CRS tests on specimens of intact chalk are summarised in Table 11.5.4.3-4. There is a relatively large scatter in the measured values as indicated by the high standard deviations relative to the average values. This is most likely due to the variable nature of the chalk as evidenced by the CPTU data accentuated by the relatively small dimensions of the oedometer test specimens.

Correlations between oedometric modulus and net cone resistance (q_{net}) are also summarised in Table 11.5.4.3-4. These correlations are based on lower bound values to the q_{net} profiles.

Table 11.5.4.3-4 Constrained oedometric modulus values for chalk

Property	No. of data points	Arithmetic mean value	Standard deviation
$E_{oed,tan}$	21	370 MPa	172 MPa
$E_{1,reload,sec}$	22	2346 MPa	1545 MPa
$E_{1,reload,sec}/E_{oed,tan}$	21	6.6	3.1
$E_{oed,tan}/q_{net}$	17	34	16
$E_{1,reload,sec}/q_{net}$	18	210	141

Static shear strength
The compressive strength (σ_c) and tensile strength (σ_t) of intact specimens of chalk have been measured by uniaxial compressive tests and Brazil tests, respectively. Test results are summarised in Table 11.5.4.3-5 below.

Table 11.5.4.3-5 Summary of UCS and Brazil tensile strength test results

	UCS test	Brazil test	Strength ratio σ_c/σ_t
Number of results	19(*)	19	18(*)
Arithmetic mean value	σ_c = 1.73 MPa	σ_t = 0.22 MPa	8.4
Standard deviation	0.22 MPa	0.05 MPa	2.2

(*) Excludes low strength from 09.A.008 at 16.1 m below top of chalk

q_{net}/σ_c and q_{net}/σ_t ratios are summarised in Table 11.5.4.3-6. The q_{net} values used to derive these relationships are based on the lower bound envelope to the relevant CPTU data.

Table 11.5.4.3-6 q_{net}/σ_c and q_{net}/σ_t ratios

	q_{net}/σ_c	q_{net}/σ_t
Number of results	17	17
Arithmetic mean value	6.8	54.6
Standard deviation	2.0	15.6

Undrained shear strengths determined from the anisotropically consolidated undrained triaxial compression and extension tests are summarised in Table 11.5.4.3-7.

Table 11.5.4.3-7 Undrained shear strengths from CAU compression and extension tests

	Compression	Extension	Strength ratio c_u^C/c_u^E
Number of results	26	16	11
Arithmetic mean value	c_u^C = 1087 kPa	c_u^E = 781 kPa	1.4
Standard deviation	223 kPa	101 kPa	0.2

q_{net}/c_u values (equivalent to cone factor N_{kt}) derived from the triaxial compression and extension test results are summarised in Table 11.5.4.3-8. The average q_{net}/c_u for triaxial compression is twice the average q_{net}/σ_c ratio consistent with the definition of $\sigma_c = 2\ c_u^C$.

Table 11.5.4.3-8 q_{net}/c_u for triaxial compression and extension strength

	q_{net}/c_u ratios	
	Triaxial compression	Triaxial extension
Number of results	24	14
Arithmetic mean value	13.7	18.2
Standard deviation	3.4	4.6

Effective stress strength parameters derived from drained triaxial compression tests and undrained triaxial compression and extension tests with pore water pressure measurements are summarised in Table 11.5.4.3-9. In this instance upper and lower bound envelope parameters are provided.

Table 11.5.4.3-9 Effective stress strength parameters for triaxial compression and extension

	Triaxial compression		Triaxial extension	
	c′ [kPa]	φ′ [°]	c′ [kPa]	φ′ [°]
Upper bound	360	42	330	42
Lower bound	0	34	0	31

Modulus of elasticity values derived from UCS and CAU triaxial compression tests are summarised in Table 11.5.4.3-10.

Table 11.5.4.3-10 Modulus of elasticity values from UCS and CAU tests.

	E_{lvdt} from UCS tests	$E_{50,sec}$ from CAU tests
Number of results	20	9
Arithmetic mean value	970 MPa	1007 MPa
Standard deviation	525 MPa	263 MPa

Modulus of elasticity values normalised by the lower bound net cone resistance at the position of the test specimen are summarised in Table 11.5.4.3-11.

Table 11.5.4.3-11 Normalised modulus of elasticity values from UCS and CAU tests.

	E_{lvdt}/q_{net} **from UCS tests**	$E_{50,sec}/q_{net}$ **from CAU tests**
Number of results	18	9
Arithmetic mean value	85	103
Standard deviation	50	34

Small strain stiffness and damping
The results of the acoustic velocity measurements in 09.A.007 were:

- P-wave velocity: 2230-2464 m/s.
- S-wave velocity: 1748-1807 m/s.
- $E_{acoustic}$: 10847-11553 MPa.
- $G_{acoustic}$: 6063-6268 MPa.

The Young's moduli from acoustic measurements are ~10 times higher than the Young's moduli from the triaxial and UCS tests. This is in agreement with the general experience from testing on slightly indurated (H2) chalk of Maastrichtien age.

The P-wave velocity is also similar to the sonic P-wave velocity determined in the geophysical borehole logging in that boring (/5/).

11.6 Typical values for geotechnical parameters

11.6.1 General

The derived value is defined as the value of a geotechnical parameter obtained by theory, correlation or empiricism from test results and an assessment of these values has been included in the present report.

The characteristic values are based on results and derived values from laboratory and field tests, complemented by well-established experience. These values are established considering the possible solutions, including stress levels and applying the principle of a cautious estimate from the derived values. The characteristic values are presumed to be established by Femerns design groups for the bridge and tunnel solutions.

Derived and characteristic values should be obtained in accordance with ref. /10/ and /11/.

The basic key to the geotechnical parameters is:

1. The combined plan and longitudinal section in Enclosure I (Drawing no. 070-02-09).
2. The soil type.
3. The CPT value.

11.6.2 “Typical values”

The geotechnical data are presented in Enclosure II (Table with typical values of geotechnical properties). These data are to be understood as first assessment of derived values, correlated from the present investigations as well as general experience and empiricism.

The data may be adjusted and refined following completions of the ongoing Geotechnical Large Scale Testing.

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