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Consolidation and Permeability Behavior of High Porosity Baltic Seabed Sediments

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CONSOLIDATION AND PERMEABILITY BEHAVIOR
OF HIGH POROSITY BALTIC
SEABED SEDIMENTS

BY
AJOYKUMAR AG

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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MASTER OF SCIENCE THESIS
OF
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UNIVERSITY OF RHODE ISLAND

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ABSTRACT

The consolidation and permeability characteristics of high porosity surficial sediments from the Coastal Benthic Boundary Layer (Naval Research Laboratory) Special Research Program test site in the Eckernförde Bay Baltic Sea were studied using a backpressured, constant rate of deformation (CRD) consolidation and permeability testing system driven by flow pumps

The silty clay sediments from the Baltic test site are characterized by high void ratios (above 6), large organic content and dissolved or free methane gas bubbles. The high organic content (up to 17%) influences several of the unique features of these sediments including high water contents (up to 500%; porosities up to 93%), high consistency limits and high compressibility. The sediments are dominated by clay and silt size fractions (42% and 55% respectively) with a small percentage (3%) of sands. Organic content determinations performed on selected samples indicated a variation from 9% to 17% in the organic fraction (dry weight basis). Atterberg Limits were determined for several samples from different locations within the site and appear to be high (w_L between 190% and 280%, I_p from 130% to 195%) in comparison with available data on other marine sediments. The Atterberg Limits were used in conjunction with the grain size and organic content data to classify the sediment as an organic clayey silt, OH in accordance with Unified Soil Classification System (USCS).

Results from CRD consolidation tests show apparent overconsolidation of sediments at the surface layers with a gradual transition to normally consolidated state at about 300 cm. The highly compressible nature of the material is reflected in the large values of compression index (3.2 to 6.8 for box core samples; 2.7 to 4.3 for gravity core samples). These high values are consistent with the large values of Plastic and Liquid Limits. Recompression indices range from 0.3 to 0.5. The coefficient of

consolidation for the samples tested range from 10^{-3} cm²/s to 10^{-5} cm²/s which are in the typical range for marine sediments.

In situ permeability ranges from 9.8×10^{-7} cm/s to 3.2×10^{-4} cm/s with the larger values corresponding to deeper samples. The permeability values plot into two fairly distinct zones depending on whether the samples were from box cores upper 40 cm) or gravity cores (deeper zones). The permeability of the box core samples ranges from 2.5×10^{-7} cm/s to 3.0×10^{-6} cm/s for void ratios ranging from 4.5 to 7.0 and the permeability of gravity cores ranges from 1.4×10^{-7} cm/s to 3.2×10^{-5} cm/s for void ratios ranging from 3.0 to 6.2. Within each group of samples there is a linear relationship between void ratio and log of permeability. For a given void ratio, the gravity cores show larger permeabilities than the box core samples due to the presence of fissures and cracks caused by gas migration.

Comparison of CRD consolidation test results for undisturbed and remolded samples at same depth interval indicate a reduction in the compression index (20-35%) due to remolding. No significant change was observed in permeability due to remolding for the box core samples compared. Permeability determinations made in vertical and horizontal orientations indicated larger (by a factor of about 1.7) permeabilities in the horizontal direction than in the vertical direction presumably due to favorable orientation of grains. No significant stress state anisotropy exists between vertical and horizontal directions at the shallower depths as observed from consolidation curves of samples oriented in the two directions.

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PREFACE

The main body of this thesis has been written in a manuscript format suitable for publication in a scholarly journal. The intent is to submit the manuscript to the ASTM Geotechnical Testing Journal for publication. The contents of the thesis are the analysis of the consolidation and permeability behavior of the high porosity seabed sediments from the Eckernförde Bay, Baltic Sea, which is one of the test sites of the Coastal Benthic Boundary Layer, Special Research Program sponsored by the Naval Research Laboratory.

The constant rate of deformation consolidation (CRD) and permeability test system used in the study is briefly introduced in the manuscript. Appendix A contains a comparison of this system and the solution scheme adopted, with other incremental loading and continuous loading test methods. Details of the consolidation and flow pump permeability test procedures are given in Appendix B.

Consolidation and permeability test results from the undisturbed samples are given in Appendix C. Results from remolded samples are placed at appendix D. Appendix E contains plots of coefficient of consolidation against void ratio. Summary tables and figures generated from these data are presented and discussed in the manuscript.

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MANUSCRIPT

**CONSOLIDATION AND PERMEABILITY BEHAVIOR OF HIGH
POROSITY BALTIC SEABED SEDIMENTS**

Abstract

The consolidation and permeability characteristics of the high porosity surficial seabed sediments from the Coastal Benthic Boundary Layer (Naval Research Laboratory) Special Research Program test site in the Eckernförde bay, Baltic Sea, were studied using a constant rate of deformation (CRD) consolidation and permeability testing system driven by flow pumps. These silty clay sediments are characterized by high void ratios (above 6), high organic content (up to 17%) and dissolved or free methane gas bubbles. The high organic content influences several unique features of the sediments including high water contents (up to 500%; porosities around 90%), high consistency limits and high compressibility. Grain size distribution and other physical properties including organic contents were determined and the material classified in accordance with Unified Soil Classification System (USCS) as organic clayey silt, OH.

Constant rate of deformation consolidation tests were performed to define the stress history and in situ stress state of the sediments from the test site. Consolidation test results from twenty four samples show apparent overconsolidation of the sediments at the surface layers with a gradual transition to normally consolidated state below 150 cm depth. The highly compressible nature of the material is reflected in the large values of compression index (3.2 to 6.8 for box core samples from the upper 40 cm; 2.7 to 4.3 for gravity core samples from deeper zones between 60 and 380 cm). These high values are consistent with the large values of Atterberg limits (w_L between 190% and 305%, I_P from 128% to 208%) and organic content (9% to 17%).

Permeability measurements were made during various stages of consolidation using a flow pump. Coefficient of permeability at in situ void ratios ranges from 9.8×10^{-7} cm/s to 3.2×10^{-4} cm/s with the larger values corresponding to deeper samples. The permeability values plot into two fairly distinct zones depending on whether the samples are from near the sediment surface (upper 40 cm) or from deeper locations.

The permeability of the box core samples ranges from 2.5×10^{-7} cm/s to 3.0×10^{-6} cm/s for void ratios ranging from 4.5 to 7.0 and the permeability of gravity cores ranges from 1.4×10^{-7} cm/s to 3.2×10^{-5} cm/s for void ratios ranging from 3.0 to 6.2. There is a linear correlation between void ratio and log of permeability for each class of sediments.

Introduction and Background

During the last several years there has been increased interest in nearshore ocean sediments and their relationship to various environmental processes. The need to develop improved systems such as those for the detection of mines on or embedded in the seabed in nearshore waters is one of the factors motivating a concentrated research effort to study these interactions. The work reported here is an investigation into the variability of consolidation and permeability behavior of the high porosity surficial seabed sediments from the Coastal Benthic Boundary Layer/ Special Research Program (CBBL/SRP) test site, in the Eckernförde Bay, Baltic Sea. The CBBL/SRP is sponsored by the Office of Naval Research and "addresses physical characterization and modeling of benthic boundary layer processes and the impact these processes have on seafloor properties" (Richardson, 1994). The focus of the CBBL/Special Research Program is the study of acoustic characteristics and stress wave propagation in seabed sediments with the objective of improving technology for detecting objects such as mines on or embedded in the seabed.

The investigation into the variability of seabed sediment microstructure and stress-strain behavior in relation to acoustic characteristics being carried out at the University of Rhode Island, Marine Geomechanics Laboratory (URI/MGL) forms part of the CBBL Special Research Program. The three major aspects of this work are: a) field work to obtain sediment cores for laboratory experiments as well as to perform in-

situ characterization and testing, b) triaxial compression, consolidation and permeability experiments to determine stress-strain-time behavior and geoacoustic properties, and c) microstructural numerical modeling of geoacoustic wave propagation (Silva et al., 1993). The research work carried out under this study is intended to establish the consolidation and permeability characteristics of the Baltic Sea sediments and to derive the geotechnical parameters necessary for numerical acoustic and stress-strain behavior models.

The Baltic test site

The Baltic sea study site is a 1 km x 2.3 km area between approximately 9° 58.5' and 10° 0.5' East longitudes and 54° 29.2' and 54° 30.2' North latitudes in the Eckernförde Bay, Germany (Figure 1). This area was chosen because of the interest in the site by the Forschungsanstalt für Wasserschall und Geophysik (FWG), Germany and the relatively uniform and undisturbed condition of the seabed sediments. The water depth within the main study site is approximately 25 m. The prime study site is located within a zone of mud surrounded by zones dominated by sands. The sediments from the study site are soft clayey silts characterized by high void ratios (exceeding 6), high organic content (ranging from 10% to 20%) and methane gas. This soft, black, natural sediment with a distinct organic odor is preserved in a relatively undisturbed state below the zone of biological mixing and oxidation which is restricted to the upper 2 to 3 cm (Richardson, 1994). The origin of the sediment is attributed mainly to abrasion of the glacial substrate which forms the site of the Baltic Sea (Emelyanov, 1992). Average sediment accumulation rates for the Baltic Sea are estimated to be 1 to 2 mm per year (Brüggmann et al., 1992) though higher rates exist at the study site. Methane gas bubbles have been found to be scattered from 20 to 100 cm depth, below which there are clusters of small bubbles (Richardson, 1994).

Ten box cores, six gravity cores and one diver core were obtained from the site during two research cruises conducted in February and May 1993 (Figure 2). The box cores were obtained using a 50 cm x 50 cm corer. The gravity cores in February, 1993 were taken with a corer supplied by FWG with 12.7 cm nominal diameter and those in May, 1993 were taken with the URI Large-diameter Gravity Corer (LGC) consisting of a PVC core pipe (10.2 cm ID., 0.6 cm thick and up to 3 m long, special nose cone/catcher assembly and adjustable weight stand). From these cores thirty 5.1 cm diameter, undisturbed consolidation and permeability samples were obtained at site. In addition four 7.0 cm diameter Shelby tube samples were taken from the box cores which were subsampled in the laboratory to obtain an additional ten consolidation samples. Twenty four samples were selected from among these for analyzing consolidation and permeability characteristics. The samples were selected with a view to facilitate the analysis of the variation in consolidation and permeability characteristics laterally as well as with depth from the sediment surface. These samples were located either within the test site or close to the boundary (Figure 2).

Literature Review

Compressional wave velocity through elastic solids is governed by the well known equation

$$V_p = \sqrt{\frac{M}{\rho}} \quad (1)$$

(Mitchell, 1993) where V_p is the velocity, M is the constrained modulus and ρ is the mass density. The constrained modulus M is the reciprocal of coefficient of volume change m_v and is evaluated for soils or sediments typically through one dimensional consolidation tests from the relation

$$M = \frac{1}{m_v} = \frac{(1 + e_0)}{a_v} \quad (2)$$

where e_0 is the initial void ratio and a_v is the coefficient of compressibility defined as

$$a_v = -\frac{de}{d\sigma'_v} \quad (3)$$

where e is the void ratio and σ'_v is the effective vertical stress (Holtz and Kovacs, 1981). The velocities of compressional and shear wave propagation through a given sediment depends on the density, the effective confining stress and the fabric (Mitchell, 1993). The stress history and in situ stress state of marine sediment deposits can be analyzed through one dimensional consolidation tests.

The compression index (C_c) is directly related to constrained modulus through the equation

$$C_c = \frac{(1 + e_0)\sigma'_v}{0.435M} \quad (4)$$

(Lambe and Whitman, 1969) and thus has a direct bearing on acoustic behavior of sediments. Consolidation properties are in turn significantly influenced by permeability characteristics. Therefore, accurate measurement of permeability characteristics is very important in studies related to wave propagation.

Relatively little data exist on the compressibility, permeability, and other geotechnical engineering properties of the Baltic seabed sediments. Their unique characteristics make it difficult to extrapolate properties from the existing data on other sediments. The clayey silts from the Eckernförde Bay tested in this study contain large amounts of organic matter (9% to 17%) which can have a significant effect on the properties. There have been only a limited number of studies on the effect of organic content on geotechnical properties of soils and in particular on marine sediments.

In a study of the geotechnical properties of highly organic continental slope deposits off the coast of Peru, Keller (1982) showed the overwhelming influence of organic carbon on natural water content and bulk density. Water contents as high as 853% were observed in these organic sediments. The organics, especially after decomposition, assume colloidal properties and have a very high absorptive capacity for water. On a dry weight basis humus has water holding capacity of several hundred percent (Millar et al., 1969).

Index properties of several offshore clays including sediments from the Baltic were summarized by Meyerhoff (1979). The Baltic sediments plotted slightly below the A-line which is consistent with their high organic content. The effects of organic carbon on plasticity properties have been studied on terrestrial soils. Tests on seventy samples from Illinois (Odell et al., 1960) showed that Liquid Limit, w_L , Plastic Limit, w_p and Plasticity Index, w_L increase linearly with increasing organic content. A linear increase of Plastic and Liquid Limits with increasing organic carbon content was also demonstrated by Bush et al. (1981) from tests on coastal sediments from Chile.

It has been observed that the upper few meters of essentially all deep sea clays exhibit appreciable amounts of "apparent" overconsolidation (Silva, 1990). This effect causes the sediments to behave like overconsolidated clays even when there has been no actual overburden removal and is attributed to the presence of inherent interparticle bonding and/or cementation. Both the effective overburden stress and the overconsolidation ratio (real or apparent) play an important role in determining the engineering behavior of marine sediments.

Smectite rich inorganic sediments from the North Central Pacific have been shown to have high compression indices between 2.5 and 3.0 (Silva et al., 1984). Organic matter has also been found to have a large influence on compressibility. A study of the effect of organic matter content in an organic silty clay from Brazil showed

that compressibility increased as a linear function of percentage of organic content (Coutinho et al., 1987). Holtz and Kovacs (1981) reports compression indices of 4.0 and above for organic clays.

There have been several studies of the permeability properties of marine sediments. Silva et al. (1981) report permeability values ranging from 10^{-4} cm/s to 10^{-8} cm/s for deep sea sediments ranging from coarse grained oozes to fine grained smectites. A linear relationship between void ratio and log of permeability was shown in this study for each class of sediment.

Sediment Characterization

The Baltic seafloor has been significantly influenced by alternating advances and retreats of ice through the glacial periods and transgression processes (Brüggmann et al., 1992). The movement of ice masses dug valleys and troughs in the ground and filled them with glacial debris. The major source of the present day sediments is abrasion of shores and bottom with smaller amounts of riverine input (Emelyanov, 1992). These processes have resulted in the sediments of the study site within Eckernförde Bay being dominantly clayey muds.

Physical property tests were performed on samples selected from different regions of the study site. Liquid and Plastic Limits, and specific gravity determinations were performed in general accordance with ASTM (1990) standards D-4318, and D-854, respectively. The grain size analyses were performed in accordance with the pippette method of Folk (1974) based on Stoke's law of settling velocity. These properties have been used to classify the sediment samples in accordance with the Unified Soil Classification System (USCS) as adapted by ASTM.

Index properties and specific gravities of 25 sediment samples were determined in accordance with the above ASTM standards (Table 2). The water contents were

corrected for a salinity of 25 parts per thousand. This value of salinity was arrived at on the basis of salinity determinations performed using a hand held refractometer on samples of the pore fluid (Silva et al., 1994). The samples are assumed to be fully saturated for evaluating phase relations.

The water contents are in the very high range of 400% to 500% in the upper 10 cm of the sediment and decrease to somewhere between 200% and 300% below 10 cm. The corresponding void ratios are in the 5 to 7 range (porosities of 86% to 83%). The water content values in the upper 10 cm are much higher than reported for typical marine sediments. Below the 10 cm depth the reduction in water content with depth is very small (Figure 3) with correspondingly small increase in bulk density. The high water contents and void ratios are attributed to the influence of mineralogy and organic matter. The natural sediments are in general at higher water contents than the Liquid Limit (Liquidity Index, I_L , less than 1.0) near the surface and at same or slightly lower water content than Liquid Limit (Liquidity Index greater than 1.0) at deeper levels (about 300 cm) (Table 1).

The Liquid Limits, of most of the samples range between 190% and 305% (Table 2). The Plastic Limit ranges from 44% to 103% and the Plasticity Index varies between 128% to 208% with an average value of 161%. These ranges appear to remain fairly constant with depth. In comparison, the typical values of w_L , w_P and I_p for a silty clay from the Gulf of Maine continental shelf were 124%, 47% and 48%, respectively (Poulos, 1988). The Atterberg Limits have been plotted on the Casagrande classification chart (Figure 4). The w_L axis has been extended to 400% and the I_p axis to 250% to represent all samples for which consistency limits are determined. All samples plot fairly close to the extended A-line on the plasticity chart suggesting the use of dual classification symbols.

The grain size distribution curves from 24 samples summarized in Figure 5 and the grain size data (Table 3) indicate that the average values of percentage (by weight) of the clay, silt and sand fractions (excluding sample 210-BS-DC) are 42%, 55% and 3% respectively. In comparison the grain size data of clayey silt sediments from the shelf and slope compiled by Hamilton (1974) are 34%, 60% and 6% respectively. Comparison of the grain size curves shows that the sediment samples from across the CBBL study site are very similar in terms of texture.

Bulk densities were determined by weighing plug samples taken with a constant volume, thin-walled plug sampler. Bulk densities range from 1.12 g/cm³ to 1.21 g/cm³ in the upper 300 cm. Bulk densities evaluated from water contents assuming 100% saturation indicated slightly larger values of 1.16 g/cm³ to 1.23 g/cm³ which is probably due to the fact that some gases have come out of solution during core recovery. The bulk density remains fairly constant at 1.20 g/cm³ with depth below the upper 10 cm in most box cores and gradually increases to 1.23 g/cm³ at greater depths (380 cm). Specific gravity values (corrected for salt content) range from 2.48 to 2.70 in the upper 50 cm with the majority of the samples between 2.50 and 2.65 (Table 2). The lower values of specific gravity can be attributed to the high percentage of organic matter present.

Organic content determinations were performed on 15 samples from across the study site in accordance with ASTM D-2974. The organic content based on dry weight at a temperature of 440° C ranges between 9% and 17% with an average value of 12% (Table 3). This is considered a very high organic content for marine sediments. The criterion for classifying a sediment as organic is however not the absolute percentage of organics present but whether the organics have a significant influence on the properties of the sediment (ASTM D-2487). This is attributed to the reversible swelling mechanism induced by the organic fibers which ceases beyond a critical stage during

drying (Mitchell, 1993). According to the USCS as adapted by ASTM, a fine grained soil is termed an organic silt or organic clay if the Liquid Limit after oven drying at 105° C is less than 75% of the value for the original specimen before oven drying. Liquid Limit determinations on samples 252-BS-BC, 11 cm and 238-BS-BC, 27 cm indicated Liquid Limits of 193% and 200%, respectively before oven drying and 75% and 82% after oven drying. This reduction to approximately 40% of the Liquid Limit before oven drying confirms the large influence of organics on these sediments. In view of the grain size distribution being dominated by clay fraction and the location of samples close to the A-line on the plasticity chart, these sediments may be classified as OH with the descriptive notation organic clayey-silt (ASTM adopts the same notation OH for organic silts and organic clays).

Further analysis shows that the w_L , w_p and I_p increase linearly with increasing organic content. For a given increase in organic content the increase in w_L is larger than for w_p . Similar behavior has been previously observed in organic marine sediments (Bush and Keller, 1981). Again this relationship indicates that the high values of w_L and w_p are a consequence of the large organic content of the samples.

Another unique feature of the Baltic sediments is the presence of methane gas, especially below the box core depths. The presence of gas in sediments can have significant influence on the in situ effective stresses and can cause disturbance during sampling. Evidence of gas migration and expansion was observed in some of the box cores taken during the February, 1993 cruise. The gravity cores taken in February, 1993 showed many cracks and pockets throughout caused by degassing when the cores were brought to the surface. No significant gas expansion was observed in the box cores taken in May, 1993 but there was ample evidence of gas in the deeper zones of the gravity cores. The tests performed for this study however concentrate on samples from the upper 100 cm and therefore the assumption of 100% saturation for estimating phase

relations is considered reasonable. Also, the CRD consolidation samples were backpressured to 414 kPa to redissolve any gases that emerged out of solution during the sampling processes.

Previous studies by Lisitzin and Emelyanov, 1981 indicate that the silty clays of the Baltic Sea consist in general of clay minerals (30% to 45%), quartz and feldspars (15% to 30%) and calcite and dolomite (3% to 10%). To estimate the relative abundance of smectites in the clay fraction (in view of their known influence on Atterberg Limits), X-ray diffraction analyses were conducted on the smaller than two micron fraction (clay size) of two samples from box core 238-BC and three samples from gravity core 029-GC. Diffraction patterns from oriented aggregates of these samples showed an apparently weak presence of smectite. The low percentage of smectite is consistent with the glacial origin of the Baltic sediments and therefore it is concluded that the influence of this mineral on the engineering properties of these sediments is not dominant. Evidence exists to suggest that the overwhelming effect of organics can mask the influence of clay mineralogy even in samples that have a significant amount of smectite (Keller, 1982). For the Baltic samples this is manifested in the substantial reduction in Liquid Limit as a result of oven drying which is attributed primarily to the influence of organics and not clay mineralogy.

Test system and Procedures

1) CRD consolidation and permeability testing system

The consolidation and permeability tests performed in this study have been conducted using a constant rate of deformation consolidometer driven by flow pumps (Gavrisheff, 1992) similar to a system developed by H. Olsen (Olsen et. al, 1989). The system has backpressuring capability and automated data acquisition equipment. The major components of the system are: 1) the consolidometer, 2) the flow pumps, 3)

differential and absolute pressure transducers, 4) air pressure regulators, 5) bladder accumulators, and 6) the data acquisition system (Figure 6).

The central part of the URI/MGL system is an SBEL Model C-400 backpressured consolidometer. The consolidometer houses the sample within the teflon confining ring of dimensions 5.1 cm diameter and 2.85 cm thick. Porous stones are placed at the top and the bottom of the sample. The chamber surrounding the sample is filled with sea water and back pressure can be applied by one of the pressure regulators through a bladder accumulator which forms the air-sea water interface. The loading piston, made of lightweight acrylic, is attached to a frictionless rolling diaphragm which separates the load chamber from the back pressure chamber. The load stress can be applied to the sample by the load pressure regulator through an oil column reservoir during the saturation phase, or by the consolidation flow pump directly during the consolidation phase. The deformation of the sample is monitored by a displacement transducer (LVDT) attached to the loading piston.

Two Harvard Apparatus Model 909 flow pumps provide means of consolidating the sample and performing permeability tests. The consolidation flow pump can deliver constant flow rates of $4.61 \times 10^{-5} \text{ cm}^3/\text{s}$ at 50% speed of the slowest gear (gear 12) and the permeability flow pump can deliver a flow rate of $1.83 \times 10^{-5} \text{ cm}^3/\text{s}$ at 100% speed of gear 10 which are typical flow rate settings.

A set of differential pressure transducers are used to measure the total stress acting on the sample and the pore pressure induced at the bottom of the sample, with reference to the backpressure. These transducers consist of a stainless steel pressure diaphragm housed inside a pressure cavity connected to the pressure sources to be measured. The diaphragm flexes in response to the differential pressure and induces a magnetic reluctance proportional to the applied pressure differential. The diaphragm used in the total stress transducer for tests on the Baltic samples has a 56 kPa full scale

pressure and an accuracy of ± 0.14 kPa and the diaphragm used for pore pressure measurements has a 35 kPa full scale pressure and ± 0.09 kPa accuracy.

2) Testing Procedures

The CRD tests were performed in general accordance with ASTM D-4186. All the samples were 5.1 cm in diameter by 2.85 cm. thick. Since the samples have very low shear strength (0.4 kPa to 1.6 kPa) and high water contents, special care was necessary to minimize disturbance during sampling and set-up. Samples were extruded from the sample tubes directly into the consolidometer ring with acrylic pistons and the ends trimmed to correct dimensions with a wire saw.

The saturation phase is initiated by connecting the consolidometer ports to the appropriate valves and opening the load chamber to the oil reservoir. Saturation pressure is increased in steps of 34.5 kPa (5 psi) every half hour up to 414 kPa (60 psi). A small seating load of 0.2 kPa (0.03 psi) is maintained throughout saturation. The sample is kept at 414 kPa overnight to ensure complete saturation after which a B-parameter test is performed. The B-parameter values ranged from 0.95 to 0.99 and were achieved within one minute indicating acceptable levels of saturation.

Constant rate of deformation consolidation is initiated by closing off the connection to the oil reservoir and starting the load flow pump (Figure 6). The Norwegian Geotechnical Institute's recommendation which specifies that the strain rates should be such that the pore pressures induced do not exceed 10% of the applied effective stress (Crawford, 1988) was adopted for determining the strain rates. A strain rate of 4.9×10^{-7} /s corresponding to gear 12, with 50% speed rating of the flow pump was found to be appropriate to meet this criterion. This strain rate was employed in most of the tests to ensure that all samples were subjected to the same strain rate effects. The effective stress was increased beyond the preconsolidation stress and then rebound was initiated. In some tests the sample was unloaded and reloaded again to determine

the recompression index, C_r . The sample deformation Δh , total stress σ and pore pressure at the base of the sample u_b are recorded during the CRD test using the computerized data acquisition system. A back up record is maintained using the strip chart recorder.

Three to fourteen permeability measurements were made during the course of each test. The first permeability test was started after the sample was well seated to avoid flow around samples (channeling). All permeability tests were done by withdrawing pore fluid from the bottom of the sample during the loading phase of the consolidation test. A strip chart recorder was used to judge whether the pressure gradient induced by this flow had reached a steady state condition. Different flow rates were selected at different stages of consolidation to maintain the gradients below five. At the end of test the consolidated specimen was extracted and the entire sample used for water content determination.

3) Solution Scheme for CRD Tests

In the constant rate of strain (deformation) consolidation (CRD) test, the rate of deformation of the sample is kept constant. A theoretical solution for this type of test has been developed by Smith and Wahls (1969) which gives the average effective stress on the sample, $\bar{\sigma}'$, as

$$\bar{\sigma}' = \sigma - \alpha u_b \quad (5)$$

where σ is the total stress and u_b is the pore pressure at the base of the sample. It has been shown that the value of the constant, α , a constant relating the average pore pressure across the height of the sample to u_b can be taken to be approximately equal to $2/3$ for all practical values of strain rate.

During the initial stages of the test, data sampling is done at 10 to 15 minutes intervals. During later stages data are recorded at half hour intervals. This results in 300 to 600 data points on the $e - \log \sigma'$ curve which provides a nearly continuous curve. In CRD tests the sample deformation is typically evaluated from the known constant rate of deformation, r and the elapsed time, t . The CRD system used in this study however incorporates an LVDT to monitor the sample deformation directly. Therefore the instantaneous void ratio e , is obtained from the relation

$$e = e_0 - \Delta h / H_s \quad (6)$$

where e_0 is the initial void ratio and H_s is the height of solids evaluated from phase relations. The initial void ratio of the sample is obtained from phase relations using the water content. The coefficients of consolidation, c_v , are evaluated from the values of coefficient of volume change, a_v , at values of void ratio corresponding to the permeability measurements.

Results and Discussion

a) Test Program

Constant rate of deformation consolidation and permeability tests were performed on 22 undisturbed and 2 remolded samples taken from seven different box cores and six gravity cores for this study (Table 4). The majority of samples (fourteen) were taken from the upper 50 cm of sediment as this was the zone of most interest to the CBBL/CRP project. The test matrix was developed to investigate the following major aspects:

- (1) Variability of consolidation and permeability characteristics with depth.

(2) Variability of consolidation and permeability characteristics laterally across the study site.

3) The effect of remolding on permeability and consolidation characteristics.

(4) Differences in permeability characteristics in the vertical and horizontal directions.

A set of comparisons was made between consolidation tests performed using a Soiltest Evanston Model standard consolidometer and the CRD system on samples from 225-BS-BC at 33-38 cm depth. The comparison of results (Table 3) shows that there is good agreement between the preconsolidation stress, compression index and permeability parameters obtained from the two types of tests.

b) Consolidation Results and Stress State

The preconsolidation stresses were evaluated from the e -log σ' curves in accordance with the Casagrande procedure (Casagrande, 1936). The nearly continuous e -log σ' curves obtained from the CRD tests (Figure 7) allowed fairly accurate evaluation of the preconsolidation stress (Table 4). It should be noted that these sediments had extremely high porosities/void ratios and therefore the stresses in the initial phases of the tests are very low (less than 1 kPa). Based on the shapes of the compression curves most of the samples were of good quality and considered to be a fair representation of the in situ condition. The comparison of e -log σ' curves for the box core 264-BC and gravity core 52-GC (Figure 7) indicates that there is good continuity of the compression behavior with depth. Tests that were run at low and intermediate strain rates (Table 4) resulted in pore pressures of less than 10% of the applied effective stress. The tests run at higher strain rates resulted in higher pore pressure buildup and have been excluded from analysis of in situ stress state and overconsolidation ratio, OCR . The effective overburden stresses were determined from the average bulk density profile for the site which was developed from water content

profiles of individual cores. Plug samples taken for bulk density measurements include the gases that came out of solution during core recovery and are therefore lower than calculated from water content measurements. It is felt that calculated densities are a better indication of the in situ conditions. The preconsolidation stress tends to increase with depth as does the effective overburden stress (Figure 8). The values of preconsolidation stress from the lower strain rate tests converge to the overburden stress at deeper levels (around 300 cm). This behavior is typical of most surficial seabed sediments that exhibit apparent overconsolidation (Silva, 1981).

The variation of the overconsolidation ratio, OCR , defined as the ratio of preconsolidation stress σ_c' , to effective overburden stress σ_o' ,

$$OCR = \sigma_c' / \sigma_o' \quad (7)$$

with depth (Figure 9) is a measure of the in situ stress state. OCR values decrease from over 6 near the surface to 1.2 at around 150 cm and approach unity beyond that depth. The apparent preconsolidation stresses indicated by the samples in the absence of any actual overburden stress removal is attributed to interparticle bonding and is typical of surficial marine sediments (Silva, 1981). In organic soils it has been suggested that such bonding could develop as a result of the interaction of gel complexes that exist in the humic fraction with the clay particles (Pusch, 1973). At greater depths the higher overburden stresses overcome these bonding effects and the OCR approaches unity, indicating normally consolidated state.

The compression indices estimated from the e - $\log \sigma'$ curves are plotted against the sample depth in Figure 10 and shows that there is significant variability in the C_c values within the upper 50 cm ranging from 3.2 to 6.8. Below that depth the C_c is between 2.7 and 4.3. These results clearly show the highly compressible behavior of

the Baltic sediments and uniformity with depth. Values of C_c as obtained from these tests compare well with published results for organic clays which typically exceed 4.0 (Holtz and Kovacs, 1981). The effect of organic matter on the strength and stiffness of soils depends on whether the organic matter is decomposed or consists of fibers which can act as reinforcement (Mitchell, 1993). In the former case, as in the case of the Baltic sediments, usually the compressibility increases and strength decreases as a result of the higher water content and plasticity contributed by the organic matter. Recompression indices were evaluated for some of the samples for which a unloading-reloading cycle was performed and range from 0.3 to 0.5. Average value of recompression index to compression index is 0.12 which compares well with the typical value of 0.1 (Holtz and Kovacs, 1981).

c) Permeability Results

Permeability determinations were made at different stages of consolidation during each test to define the variation of coefficient of permeability with void ratio. Flow pumps were used to induce flow from the top to the bottom of the sample during the loading phase of consolidation. The coefficients of permeability, k , determined from the individual tests are presented as void ratio versus log permeability (e -log k) curves together with the e -log σ plots (Figure 7). A linear relationship between e and log of k is observed in all the samples tested. Least square regression lines have been fitted to individual samples and the correlation coefficients range between 0.85 to 0.98 with most values falling above 0.90. The permeability values extrapolated to in situ void ratios range from 9.8×10^{-7} cm/s to 3.2×10^{-4} cm/s (Table 4). Given the highly flocculated microstructure of these sediments (reflected in the high void ratios), the permeability is lower than would be predicted by extrapolation of data from inorganic marine sediments. This has been previously observed in Baltic Sea sediments and has been attributed to the microbiogenic and organic constituents that wrap around the clay

particles to form an "aqueous plastic framework" that retard pore fluid flow (Brüggmann et al., 1992) .

Most of the coefficients of permeability fall into two fairly distinct zones depending on whether the samples are from shallow depths (box core samples) or from deeper locations (gravity cores) as seen from the combined plot of permeability against void ratio (Figure 11). The permeability of box cores range from 2.5×10^{-7} cm/s to 3×10^{-6} cm/s for void ratios from 4.5 to 7.2, whereas for the samples from the gravity cores it ranges from 1.3×10^{-7} cm/s to 3.2×10^{-5} cm/s for void ratios from 3.0 to 6.2. Least square regression lines have been fitted to the two groups separately. The slope of the regression line for the gravity core samples is smaller than for box cores. Consequently, at void ratios closer to in situ values the gravity core samples have a higher permeability than samples from the shallow depths. In general, for a given change in void ratio, the corresponding change in permeability is larger for the gravity cores compared to box core samples.

However it should be noted that permeability results from some of the cores fall into intermediate zones on the combined e - $\log k$ plot. Samples from the gravity core 333-GC plots along with the group of box core samples instead of the group of gravity cores. It is however noted that this sample was taken at a depth of 60 cm and consequently has permeabilities closer to box core samples than gravity cores. Samples from box cores 040-BC and 038-BC plot between the two groups. These two cores are from the western part of the study site whereas all other box cores are from the eastern part or the center of the study site.

When considering the permeability of fine grained soils three levels of soil fabric are important (Mitchell, 1993). "The micro-fabric consisting of regular aggregations of particles and very small pores, the mini-fabric that consists of these assemblages and the inter-assemblage pores between them and the large scale macro-fabric that contains

cracks, fissures, laminations, root holes and trans-assemblage pores". The flow at the macro fabric level if present can be much larger than at the smaller levels. Gravity core samples from the Baltic Site did present evidence of macro-fabric level pores or cracks possibly due to degassing and abundance of organic matter. At shallower depths these effects were relatively less evident. Such differences in fabric can explain the differences in the magnitudes of permeability exhibited by samples from gravity cores and box cores at comparable void ratios. As consolidation progresses these differences become reduced causing the permeability values to converge. In the gravity cores there was a more rapid reduction in permeability due to closing of fissures whereas in box cores the reduction is only due to reduction in void ratio with consolidation.

The coefficient of consolidation for the samples have been evaluated from the e - $\log \sigma'$ and e - $\log k$ relationships. Since direct permeability measurements are made using the permeability flow during various stages of consolidation, c_v can be evaluated directly rather than by inverse solution as in the case of standard consolidometer tests. Typical values for c_v range from 10^{-3} cm²/s to 10^{-5} cm²/s. As expected, plots of c_v versus e from individual tests show a reduction in the magnitude of the coefficient of consolidation with decreasing void ratios during consolidation (Figure 12).

The permeability values corresponding to a reference value of 5.5 were obtained by interpolating from the regression lines of e - $\log k$ plots for individual samples and are plotted against respective depths in Figure 13. This value of void ratio was chosen since it is close to the average value of in situ void ratio and is within the range of void ratios at which permeability measurements were made. Most samples fall in a range of 4.0×10^{-7} cm/s to 2.0×10^{-5} cm/s. The average permeability appears to increase with depth from approximately 1.0×10^{-6} cm/s at about 25 cm, to 6.0×10^{-6} cm/s at 150 cm. A similar trend is visible from the plot of permeability at in situ void ratios versus depth. In view of the fact that no major variations with depth exist in the grain size

distribution, mineralogy, or organic content within the upper 400 cm, these trends in permeability can be attributed to changes in macro-fabric that contains fissures, cracks, laminations etc. (Mitchell, 1993). While these structures dominate and control permeability at deeper levels, nearer to the surface these features are less evident, and the permeability is significantly lower.

The in situ stress profile (Figure 8) indicates that the preconsolidation stresses of samples at the same depth within the sediment column are roughly equal irrespective of the relative location within the test site. The *OCR* profiles (Figure 9) likewise follows a fairly unique curve with depth irrespective of the relative lateral locations of the samples. These trends suggest that the sediments across the site have been subjected to a uniform stress history resulting in uniform in situ stress state.

Duplicate samples were available from core 225-BS-BC at 28-33 cm and from 252-BS-BC at 27-31 cm. These samples were tested after remolding under vacuum and the results compared against test results from the undisturbed samples. Vacuum was applied to ensure that void ratio was not altered during remolding. As expected the preconsolidation stress of 2.0 kPa of the undisturbed specimen sample 225-BC has disappeared due to the remolding action. The compression index before remolding was 3.2 which reduced to 2.0 after remolding. For sample 252-BC the undisturbed compression index was 3.7 which reduced to 3.0 as a result of remolding. Remolding clearly has a significant effect on the consolidation characteristics of these sediments. The comparison between undisturbed and remolded permeabilities for samples 225-BS-BC indicates that the permeability in the remolded state is very nearly equal to that in the undisturbed state (Figure 14). This was not unexpected since the sediments exist in a highly flocculated state in situ (as evidenced by the high void ratios) and the void ratio was kept constant during remolding action.

A consolidation subsample was extruded in the horizontal orientation from the box core 264-BS-BC at 33 cm depth and a vertically oriented sample was obtained from the Shelby tube taken from the same box core at a depth range of 34 to 38 cm. These samples were tested to analyze the variations in permeability and other characteristics between the vertical and horizontal directions. There is very good agreement of the preconsolidation stresses and compression indices obtained in both cases (Figure 15). The ratio of preconsolidation stress from the horizontal sample to that of the vertical sample is approximately unity. For an initial isotropic state the constrained modulus varies as a function of the average confining stress (Lambe and Whitman, 1969) and thus controls the acoustic characteristics.

The permeability plots indicate that permeability in the horizontal direction is slightly larger (about 1.7 times) than in the vertical direction (Figure 15) which can be attributed to the lower resistance to flow due to more favorable grain orientation in the horizontal direction as compared with the vertical direction. Previous studies on terrestrial soils have measured ratios of vertical to horizontal permeability from less than one to more than seven (Mitchell, 1993). Large differences in permeability can result in significantly different rates of consolidation in the two directions. The differences measured here are not very large but these results suggest that even for these very high porosity/water content sediments there may be some anisotropic microstructure in the upper 50 cm. However, the measured differences in permeabilities for these sediments are too small to substantially influence the consolidation properties as seen from the comparison of consolidation curves in the two directions (Figure 15).

Summary and Conclusions

The silty clay sediments from the Baltic site are characterized by high void ratios (up to 7.5) and high organic content (9% to 17%). It appears that the high organic

content controls several of their unique features including high water contents (up to 500%) high consistency limits and high compressibility (compression indices of 2.7 to 6.8). Constant rate of strain consolidation (CRD) tests performed using a special flow pump system have been used to determine the consolidation and permeability characteristics of these soft sediments.

Grain size determinations performed using the pipette method show that these sediments have a large silt fraction (average 55%), dominant clay size fraction (average 42%) and a small percentage (3%) of sand. Grain size distributions indicate that the percentage grain size fractions of samples from different locations deviate only slightly from the mean values. Mineralogy analyses performed on selected samples indicate that the clay size fraction consists mainly of illite, kaolinite, plagioclase, quartz, and small amounts of smectite. No significant changes are observed in the amounts of these fractions with depth from the surface as judged from the talc internal standard analysis done on samples from different depths. The amount of organic material varied from 9% to 17% and there appears to be a linear correlation between percentage of organics and the Atterberg Limits. The dependence of consistency limits on organic fraction seems to indicate that the organic fraction has the dominant influence on the engineering properties of these materials.

Atterberg Limits are higher (w_L ranges between 190% and 305%, w_p ranges from 44% to 103%) in comparison with available data on other marine sediments. The natural Baltic sediments are in general at higher water contents than the Liquid Limit near the surface and at same or slightly lower water content than Liquid Limit at deeper levels (about 300 cm). This is reflected in the apparent increase in strength of the sediments with depth.

The in situ stress state of sediments have been characterized on the basis of consolidation and permeability results from 22 undisturbed sediment samples.

Consolidation results show apparent overconsolidation of the sediments at the surface layers with a gradual transition to normally consolidated state at about 150 cm depth. In highly organic soils, the interparticle bonding effect which produces apparent overconsolidation is attributed mainly to the interaction of gel complexes present in the humic fraction with the soil grains. The highly compressible nature of the Baltic Sediments is reflected in the large values of compression index (3.2 to 6.8 for box core samples; 2.7 to 4.8 for gravity core samples). Recompression indices ranged from 0.3 to 0.5. The coefficient of consolidation for the samples tested ranged from 10^{-3} cm²/s to 10^{-5} cm²/s which are in the typical range for marine sediments.

In general, the coefficients of permeability fall into two fairly distinct zones depending on whether the samples are from shallow depths (box core samples) or from deeper locations (gravity cores). The permeability of box core samples range from 2.5×10^{-7} cm/s to 3×10^{-6} cm/s for void ratios from 4.5 to 7.2, whereas for the samples from the gravity cores it ranges from 1.3×10^{-7} cm/s to 3.2×10^{-5} cm/s for void ratios from 3.0 to 6.2. For a given void ratio the gravity core samples show a larger permeability compared to box core samples which can be attributed to macro-fabric level features such as fissures and cracks in gravity core samples formed due to gas migration during core recovery. Permeability values of the gravity core 333-GC plots closer to the group of box core samples instead of the group of gravity cores (Figure 11). The sample from 333-GC was taken at a depth of 60 cm and consequently has coefficients of permeability closer to box core samples than gravity cores. Samples from box cores 040-BC and 038-BC from the western part of the study site plot between the two groups.

Tests done on samples from identical depth interval in undisturbed and remolded conditions indicate the break-down of the preconsolidation stress due to remolding and also indicate a reduction (20% to 35%) in the compression index from remolding.

Comparison of permeabilities in the remolded and undisturbed samples indicate that at least for the shallow sediments there is no significant change in permeability due the remolding. Permeability in the horizontal direction is found to be larger (by a factor of approximately 1.7) than in the vertical orientation presumably due to favorable orientation of grains. No significant stress anisotropy exists between vertical and horizontal directions at the shallower depths (less than 50 cm) as is observed from consolidation curves of samples oriented in the two directions.

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**Table 1. Physical Properties of Sediments from the
CBBL/NRL Test Site, Baltic Sea.**

Sample Designation	Depth (cm)	Water content (%)*	Specific Gravity*	Liquid Limit (%)*	Plastic Limit (%)*	Liquidity Index
029-GC	340	236	2.77	277	82	0.79
029-GC	60	250	2.54	250	109	1.00
034-GC	30	297	2.48	246	69	1.29
034-GC	120	249	2.57	268	88	0.89
035-BC	40	246	2.55	229	66	1.10
035-BC	4	317	2.70	218	60	1.63
038-BC	21	211	2.53	219	51	0.95
040-BC	3	233	2.59	190	44	1.29
040-BC	31	191	2.56	223	56	0.81
041-BC	13	250	2.58	214	84	1.28
048BC	0.25	366	2.71	---	---	---
048-BC	25.5	219	2.59	200	53	1.13
048-BC	49	321	2.54	274	113	1.29
210-BC	8	204	2.63	215	64	0.93
225-BC	0.5	336	2.52	246	86	1.56
225-BC	16.5	229	2.56	237	74	0.95
225-BC	29	246	2.60	220	62	1.16
238-BC	0.25	373	2.50	305	97	1.33
238-BC	10.5	233	2.61	225	80	1.06
238-BC	27	228	2.64	235	79	0.96
238-BC	47	270	2.50	280	94	0.95
252-BC	5	236	2.71	224	84	1.09
252-BC	16.5	243	2.58	225	64	1.11
252-BC	28	271	2.57	242	64	1.16
264-BC	27	232	2.59	---	---	---

* Corrected for 25 ppt salinity
GC= Gravity Core

BC= Box Core

Table 2. Texture and Organic Fractions, Sediments from the CBBL/NRL Test Site, Baltic Sea.

Sample Designation	Depth (cm)	Sand (%)*	Silt (%)*	Clay (%)*	Organics (%)
029-GC	60	3	51	46	--
029-BC	340	5	65	30	--
034-GC	30	1	53	46	--
034-GC	120	5	50	45	--
035-BC	40	5	49	46	11
035-BC	4	2	55	43	10
038-BC	21	2	59	39	10
040-BC	3	2	56	42	9
040-BC	31	1	54	45	11
041-BC	13	2	58	40	11
048-BC	0.3	2	59	39	--
048-BC	25	5	62	33	11
048-BC	49	2	60	38	--
238-BC	0.3	4	56	40	--
238-BC	11	3	59	38	10
238-BC	27	2	54	44	11
238-BC	47	3	52	45	12
225-BC	0.3	4	51	45	--
225-BC	17	3	56	41	--
225-BC	29	4	52	44	--
252-BC	5	3	57	40	--
252-BC	17	3	54	43	--
252-BC	28	2	54	44	--
264-BC	27	3	53	44	--
Average		3.0	55	42	
252-BS-BC	0.3	--	--	--	12
029-BS-GC	210	--	--	--	17
029-BS-GC	460	--	--	--	15
029-BS-GC	280	--	--	--	17
029-BS-GC	90	--	--	--	14

*Sand > 0.062 mm

Silt 0.002-0.062 mm

Clay <.002 mm

**Table 3. Comparison Between CRD and Standard Test Results;
Sample 225-BS-BC, 33-38 cm.**

	<u>CRD Test</u>	<u>Standard</u>
Sample depth (cm)	33-38	33-38
Initial water content (%)	244	254
Initial void ratio	6.2	6.5
Preconsolidation stress (kPa)	2.4	3.5
Compression Index	3.1	3.8
OCR	3.1	4.7
Range of permeability (cm/s)*	3×10^{-7} to 8×10^{-7}	2×10^{-7} to 8×10^{-7}

*corresponding to void ratio, $e = 4$ to 5.5

Table 4. Summary of CRD Consolidation and Permeability Test Results; Baltic Sediments.

Sample Designation	Depth (cm)	Water Content (%)**	Initial Void Ratio	Precons. Stress (σ'_c)(kPa)	Overb. Stress (σ'_o)(kPa)	OCR	Compr. Index (C_c)	Recomp Index (C_r)	Permeability (k)*** (cm/s)
29-GC*	73	243	6.02	4.6	1.2	3.9	3.5	-	2.2×10^{-5}
29-GC*	377	236	6.39	11.1	6.6	1.7	4.3	-	5.0×10^{-1}
34-GC*	142	250	6.27	7.0	2.4	2.9	3.1	-	1.0×10^{-5}
35-BC	16	285	7.10	2.7	0.3	9.3	3.9	-	4.1×10^{-6}
35-BC*	26	254	6.59	2.8	0.4	6.7	4.1	-	2.6×10^{-6}
35-BC*	33	274	6.83	3.5	0.5	6.6	4.9	-	1.8×10^{-6}
38-BC*	19	249	6.30	3.8	0.3	12.4	6.8	-	5.3×10^{-6}
40-BC (1)	32	238	5.93	3.5	0.5	6.8	6.3	0.3	8.0×10^{-6}
40-BC (2)	32	236	5.90	2.9	0.5	5.6	3.4	0.3	1.2×10^{-5}
52-GC	256	179	4.67	5.1	4.4	1.2	3.9	-	3.2×10^{-5}
225-BC	16	289	7.22	1.9	0.3	7.3	3.2	-	1.1×10^{-5}
225-BC	36	244	6.20	2.3	0.6	4.0	3.2	0.5	9.8×10^{-7}
225-BC (R)	37	218	5.52	-	-	-	2.0	-	1.6×10^{-6}
238-BC	31	284	7.36	3.4	0.5	6.8	4.3	0.4	2.6×10^{-6}
252-BC (1)	29	284	7.12	2.5	0.5	5.3	3.7	-	3.0×10^{-6}
252-BC (2)	29	276	7.17	-	0.5	6.8	4.3	-	3.3×10^{-6}
252-BC (R)	29	282	7.08	3.2	-	-	3.0	-	3.7×10^{-6}
264-BC (H)	33	275	6.95	2.3	0.5	4.3	3.5	-	7.6×10^{-6}
264-BC (V)	36	274	6.93	2.5	0.6	4.3	3.5	0.5	3.4×10^{-6}
304-GC	144	280	7.03	4.6	2.4	1.9	3.5	0.3	4.3×10^{-5}
304-GC	150	259	6.55	3.6	2.5	1.4	2.9	-	2.8×10^{-4}
318-GC	144	221	5.61	3.3	2.4	1.4	2.7	0.5	3.2×10^{-4}
318-GC	157	245	6.19	3.7	2.7	1.4	4.1	-	7.5×10^{-5}
333-GC	60	266	6.75	4.3	1.0	4.5	3.4	-	5.5×10^{-6}

* CRD Tests at High Strain Rate
R= Remolded

**Corrected for 25ppt. salt content
V= Vertical

*** At In Situ Void Ratio
H= Horizontal

Figure 1. CBBL/NRL Study Site in Eckernförde Bay, Baltic Sea.

Figure 2. Location of Cores and Acoustic Towers, 1993.

Water Content (%)*

Density (g/cm^3)

Shear Strength (kPa)

*corrected for 25ppt. salt content

Figure 3. Profile of Water Content, Bulk Density and Shear Strength with Depth; 238-BC and 252-BC Baltic Sea.

Figure 4. Location of Baltic Sediments on the Casagrande Classification Chart.

Figure 5. Grain Size Distribution of Sediments from CBBL Test Site; Baltic Sea

Figure 6. Schematic of Constant Rate of Deformation Consolidation and Permeability Testing System.

Figure 7. Consolidation and Permeability Results from Samples 264-BC, 34-38 cm and 052-GC, 253-258 cm.

Figure 8. In Situ Profiles of Preconsolidation Stress And Overburden Stress, CBBL Test Site, Baltic Sea.

Figure 9. Profile of Overconsolidation Ratio with Depth, CBBL Test Site, Baltic Sea.

Figure 10. Profile of Compression Index with Depth, CBBL Test Site, Baltic Sea.

**Figure 11. Variation of Permeability with Void Ratio;
Gravity and Box Core Samples.
(Sediments from CBBL Test Site, Baltic Sea).**

Figure 12. Variation of Coefficient of Consolidation with Void Ratio (Typical); Sample 333-BS-GC, 58-62 cm.

Figure 14. Sample 252-BC, 27-31 cm; Comparison of Permeabilities in the Undisturbed and Remolded States.

Figure 15. Comparison of Consolidation and Permeability Properties in Vertically and Horizontally Extruded Samples; 264-BC, 33 cm.

APPENDIX A

Review of Consolidation and Permeability Test Methods

APPENDIX A

Review of Consolidation and Permeability Test Methods

Accurate evaluation of hydraulic conductivity (permeability) and consolidation properties of seabed sediments is extremely important in assessing seabed processes and engineering systems. Geotechnical problems such as geoacoustic modeling, settlement calculations, slope stability analysis, hazardous waste disposal, etc. require a detailed understanding of the permeability and compressibility behavior of the sediments. The need for increasingly more accurate measurement of these parameters has assumed greater importance due to the trend towards more optimal engineering design as well as due to the need to better predict pore fluid migration through sediments. Recently there has been an increased interest in coastal environmental problems which usually require a better understanding of seabed processes and behavior.

1. Review of Consolidation Test Methods

Compressibility of clays are commonly evaluated in the laboratory by means of the one dimensional (oedometer) consolidation test. Variations of this test can be classified into the following categories (Gill, 1989):

- Incremental loading tests
 - Multistage loading (24 hour schedule)
 - Multistage loading (primary)
- Continuous loading tests
 - Controlled gradient
 - Constant pore pressure-load ratio
 - Restricted flow
 - Constant rate of deformation

The incremental load tests typically need 10 to 14 days for completion depending on the stress levels to be applied during testing and the load increment ratio. In recent

years several continuous loading test methods as listed above were devised in an effort to perform consolidation testing in shorter periods of time.

In all of these tests the sample is placed in a vertical cylinder of relatively rigid material with a smooth internal surface which confines it laterally while the load is applied vertically. Drainage is permitted from one or both ends through filter papers and porous stones. The deformation of the sample under the applied load is measured to derive the effective stress-void ratio relationship. This test approximates the compression of the soil in the field reasonably well. The theoretical basis for evaluation of the consolidation characteristics in the incremental load tests is the Terzaghi formulation (Terzaghi, 1943) for one dimensional consolidation which is a one dimensional diffusion equation given by

$$\frac{\partial \sigma}{\partial t} - \frac{\partial u}{\partial t} = -c_v \frac{\partial^2 u}{\partial z^2} \quad (\text{A-1})$$

where

$$c_v = \frac{k}{\gamma_w m_v} \quad (\text{A-2})$$

σ	=	applied stress
u	=	pore pressure
k	=	permeability
γ_w	=	unit weight of permeant
m_v	=	coefficient of volume change
t	=	time
z	=	depth

c_v is assumed to be a constant (for a load increment) called coefficient of consolidation.

This equation is modified suitably for the different test methods and different solutions are obtained by taking into account the appropriate boundary conditions for each test.

The consolidation tests performed in this study employed the CRD testing technique. The theoretical formulations for the different test methods are briefly reviewed here vis - a - vis the CRD technique.

a) Incremental Load Tests:

In incremental load tests consecutive loads are applied at the end of every 24 hours (24 hour schedule). Each load beginning with the second is usually equal to the sum of the previous loads thus giving a load increment ratio of one (1). In order to better define the e -log σ' curve and thus the preconsolidation stress, sometimes a smaller load increment ratio of 0.5 is employed with consequent increase in test duration. In the multistage (primary) loading type of tests, the successive loads are applied at the end of primary consolidation at each load step with a view to decreasing the testing time. End of primary consolidation can be decided by monitoring the dissipation of pore pressure at the undrained surface.

In this type of test, the total stress remains a constant during each increment. i.e.

$$\frac{\partial \sigma}{\partial t} = 0 \quad (\text{A-3})$$

and the Terzaghi equation reduces to the form

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad (\text{A-4})$$

The solution for this equation is obtained as a Fourier series describing the variation of the pore pressure u with depth z and time t . Since theoretically, all pore pressures have dissipated at the end of each load step, the effective stress σ' is given by

$$\sigma' = \sigma \quad (\text{A-5})$$

where σ is the total stress and is reported against the corresponding void ratio obtained from the deformation data. For each load the coefficient of consolidation is estimated by an inverse solution technique using either the Taylor's square root of time method (Taylor, 1948) or from logarithm of time t versus dial gage reading plots.

b) Continuous Loading Tests:

1) Constant gradient test

The constant gradient test was introduced by Lowe et al. (1969) with the basic objectives of: "1) Having the stress conditions throughout the specimen as uniform as possible, 2) having a uniform rate of compression throughout the test and 3) being able to run tests at different slow rates of compression , so that extrapolation of data to actual even slower rates of compression typically occurring in the field can be made". In this type of test a small constant excess pore pressure is maintained at the undrained bottom boundary of the sample by adjusting the loading rate suitably. In this test the pore pressure at the base remains a constant so that

$$\frac{\partial u}{\partial t} = 0 \quad (\text{A-6})$$

The consolidation equation in this case is given by

$$\frac{\partial \sigma}{\partial t} = -c_v \frac{\partial^2 u}{\partial z^2} \quad (\text{A-7})$$

The solution for pore pressure for the boundary conditions existing in this test can be shown to be a parabola given by

$$u = \Delta u \left(1 - \frac{z^2}{H^2} \right) \quad (\text{A-8})$$

where Δu is the pore pressure difference across the sample depth z and H is the sample height. The effective stress therefore is found to be

$$\sigma' = \sigma - \frac{2}{3}u_b \quad (\text{A-9})$$

where u_b is the pore pressure at the base. The coefficient of consolidation is given by

$$c_v = \frac{\partial \sigma}{\partial t} \frac{H^2}{2\Delta u} \quad (\text{A-10})$$

From observations of applied stress σ with time t , the pore pressure at the base u_b and increments of σ for corresponding increments of t during the controlled gradient test, the effective stress and coefficient of consolidation can be determined using the above equations.

2) Constant Pore Pressure-Load Ratio Test

In the constant pore pressure-load ratio test, the ratio of base pressure to the applied stress is maintained at a constant value by continuously adjusting the loading rate, which simplifies the analytical solution. During the test continuous records of the applied stress σ , sample deformation δ and base pore pressure u_b are maintained so that the time derivatives $\dot{\sigma}, \dot{\delta}, \dot{u}_b$ of these variables are known. To evaluate compressibility, the following equations were derived by Janbu et al., 1981.

$$\dot{\sigma}_{avg} = \sigma - \bar{f} u_b \quad (\text{A-11})$$

where,

$$\bar{f} = \frac{a \cosh(a) - \sinh(a)}{a \cosh(a) - 1} \quad (\text{A-12})$$

$$a = \cosh^{-1}(1 - \lambda)^{-1} \quad (\text{A-13})$$

and

$$\lambda = \frac{\dot{u}_b}{\dot{\sigma}} \quad (\text{A-14})$$

If λ is a constant then $\bar{f}$ is a constant thus simplifying the expression for effective stress.

3) Restricted Flow Test

The restricted flow consolidation test was developed by Sills et al. (1986). The applied stress is built up to a predetermined level and held constant thereafter. Drainage is allowed at a small constant flow rate from one end of the sample such that the pressure gradient across the sample is only a small percentage of the applied stress (which keeps the effective stress and the void ratio nearly uniform across the sample). The total stress, deformation of the sample and the pore pressure at both ends of the sample are monitored. The effective stress is reported as total stress minus the mean pore pressure which is given by

$$\sigma'_{ave} = \sigma - u_{avg} \quad (\text{A-15})$$

where

$$u_{ave} = \left(\frac{1}{3}\right)u_{drained} + \left(\frac{2}{3}\right)u_{undrained} \quad (\text{A-16})$$

where $u_{drained}$ and $u_{undrained}$ are the pore pressures at the drained and undrained ends of the sample respectively.

4) Constant Rate of Deformation Test (CRD)

In the constant rate of strain (deformation) consolidation, the rate of vertical deformation of the top surface of the sample is kept constant. The total applied stress and pore pressure developed at the base of the sample are monitored. A theoretical solution for this type of test has been developed by Smith and Wahls (1969). The basic consolidation equation is written as

$$\frac{\partial}{\partial z} \left(\frac{k}{\gamma_w} \frac{\partial u}{\partial z} \right) = \frac{1}{1+e} \frac{\partial e}{\partial t} \quad (\text{A-17})$$

Assuming that permeability k is independent of position (and is a function of time only) this equation is reduced to

$$\left(\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} \right) = \frac{1}{1+e} \frac{\partial e}{\partial t} \quad (\text{A-18})$$

The void ratio e is assumed to be a linear function of position z and time t given by

$$e = e_0 - rt \left[1 - \frac{b}{r} \left(\frac{z - 0.5H}{H} \right) \right] \quad (\text{A-19})$$

where e_0 is the initial void ratio, r is the strain rate, H is the sample height and b is a parameter depending on variation in void ratio with depth and time. Using this expression for void ratio the consolidation equation can be solved for the variation of pore pressure. However due to the complexity of this solution it is further assumed that the term $(1+e)$ can be replaced by $(1+\bar{e})$ where $\bar{e}$ is the average void ratio across the sample. The resulting solution is given by

$$u = \frac{\gamma_w r}{k(1+\bar{e})} \left[(Hz - \frac{z^2}{2}) - \frac{b}{r} \left(\frac{z^2}{4} - \frac{z^3}{6H} \right) \right] \quad (\text{A-20})$$

The average effective stress is obtained as

$$\bar{\sigma} = \sigma - \alpha u_b \quad (\text{A-21})$$

where u_b is the base pore pressure and α , a constant can be shown to be

$$\alpha = \frac{\bar{u}}{u_b} = \frac{\frac{1}{3} - \frac{b}{r} \frac{1}{24}}{\frac{1}{2} - \frac{b}{r} \frac{1}{12}} \quad (\text{A-22})$$

where $\bar{u}$ is the average pore pressure. It has been shown that the value of α can be taken to be approximately equal to $2/3$ for all practical values of b/r . The coefficient of consolidation c_v can be given in terms of the strain rate r as

$$c_v = \frac{rH^2}{a_v u_b} \left[\frac{1}{2} - \frac{b}{r} \left(\frac{1}{12} \right) \right] \quad (\text{A-23})$$

where a_v is the coefficient of compressibility defined by

$$a_v = -\frac{de}{d\sigma' v} \quad (\text{A-24})$$

2. Laboratory Determination of Permeability

In the laboratory, permeability of soils is determined by performing one of the following types of tests:

- 1) Constant Head Method
- 2) Falling Head Method
- 3) Flow Pump Test.

Permeability can also be evaluated from the oedometer tests by the application of consolidation theory.

Traditionally the first two of the above methods have been generally employed in laboratories. In the constant head method a constant gradient is imposed on the soil sample by means of a column of water and the resulting flow is measured. The permeability is evaluated from the Darcy's (1856) law given by

$$q = k i A \quad (A-25)$$

where,

q = is the flow rate

k = is the coefficient of permeability

i = is the gradient

A = is the cross sectional area of the sample

In the falling head test the gradient is allowed to vary. The initial head, final head and the time of flow are measured from which the permeability is calculated. The falling head test is usually used for fine grained, low permeability materials whereas the constant head test is employed with highly permeable soils like sand.

The conventional test methods have several disadvantages. Fine grained sediments are typically subjected to very low hydraulic gradients in the field. Flow rates under these gradients are consequently very small. Since the resolution of volume measurement devices are typically larger than 10^{-3} cm^3 , permeability tests of fine grained sediments can be very time consuming. Subjecting the samples to higher gradients can result in rearrangement of the soil fabric, non uniform void ratios and consequently incorrect values of permeability (Olsen, 1966). Flow of the permeant fluid around the soil sample known as "channeling" and the so called seepage induced

consolidation may also occur producing incorrect results. Use of capillary tubes for flow measurement has associated problems such as contamination of capillary tubes and flow around the marker bubbles (Olsen, 1966).

Olsen (1966) introduced the flow pump technique for permeability measurement of fine grained sediments. In this method a known steady flow rate is imposed on the sample using the flow pumps and the gradient induced across the sample due to this flow is measured with pressure transducers. The permeability is computed using the Darcy's law. The advantage of this technique is derived from the fact that in the laboratory it is easier to monitor low pressure gradients than correspondingly low flow rates. Flow pumps that can easily deliver flow rates as low as 10^{-5} cm³/s have been adapted from the medical industry for these tests.

Determination of permeability from oedometer tests is accomplished by applying consolidation theory. These calculations are subject to error since graphical procedures are employed in the analysis. Another disadvantage all of the traditional methods have in common is that in order to determine the permeability at different void ratios at various stages of consolidation it is necessary to interrupt the consolidation tests which increases the time required for completion of these tests excessively. The flow pump testing technique as adopted in this testing program is a convenient way to perform permeability tests without interrupting the consolidation process.

APPENDIX B

Equipment and Test Procedures

APPENDIX B

Equipment and Procedures

System components

The consolidation and permeability tests performed in this study have been conducted using a constant rate of deformation consolidometer driven by constant-flow pumps (Gavrisheff, 1992). The system has backpressuring capability and automated data acquisition equipment. The major components of the system are: 1) the consolidometer, 2) the flow pumps, 3) the differential and absolute pressure transducers, 4) the air pressure regulators, 5) bladder accumulators and 6) the data acquisition system. A schematic diagram of the system is given in Figure 6.

The central part of the system is a SBEL Model C-400 backpressured consolidometer. The consolidometer houses the sample within the teflon confining ring of dimensions 5.1 cm diameter and 2.85 cm height. Porous stones are placed at the top and the bottom of the sample. The chamber surrounding the sample is filled with sea water and back pressure can be applied by one of the pressure regulators through a bladder accumulator which forms the air-sea water interface. The loading piston made of lightweight acrylic is attached to a frictionless rolling diaphragm which separates the load chamber from the back pressure chamber. The load stress can be applied to the sample by the load pressure regulator through an oil reservoir during saturation phase or by the consolidation flow pump directly during the consolidation phase. The deformation of the sample is monitored by the Linear Variable Displacement Transducer (LVDT) attached to the loading piston. The LVDT has a resolution of 2.54×10^{-5} cm. The permeability port provided at the bottom of the consolidometer allows withdrawal of pore fluid through the bottom of the sample during permeability testing.

Two Harvard Apparatus Model 909 flow pumps provide means of consolidating the sample and performing permeability tests. The Model 909 is a single carriage, infusion/ withdrawal pump. A twelve speed gear box in conjunction with a variable speed motor permits an infinite number of flow rates. The consolidation and permeability flow pumps have syringes of inside diameter 2.54 cm and 1.04 cm respectively attached to them. Figure B-1 illustrates the working principle of the flow pumps. The flow is induced by driving the piston at a constant rate within the syringe. The consolidation flow pump can deliver constant flow rates of $4.61 \times 10^{-5} \text{ cm}^3/\text{s}$ at 50% speed of the slowest gear and the permeability flow pump can deliver a flow rate of $1.83 \times 10^{-5} \text{ cm}^3/\text{s}$ at 100% speed of gear 10 which are typical flow rate settings. A figure of eight configuration of valves and tubing attached to the syringes make it possible to continue infusion of fluid in the same direction even after the piston reaches the end of its stroke and the travel direction is reversed.

A set of three Validyne Engineering Corporation Model P-305D differential pressure transducers are used to measure the total stress acting on the sample and pore pressure induced at the bottom of the sample, with reference to the backpressure. These transducers consist of a stainless steel housing and a central diaphragm partitioning the housing into two chambers connected independently to the pressure sources (Figure B-2). The diaphragm flexes in response to the pressure differential and thereby induces a voltage proportional to the magnitude of the pressure differential. The pore pressure is measured either by the low or high pore pressure transducer depending on the magnitude of the induced pore pressure. The differential mode of operation enables the transducers to detect small pressure changes while operating at very high pressure levels. The diaphragms can be replaced to suit the operating pressure ranges. The diaphragm used in the effective stress transducer for tests on the Baltic samples was of 56 kPa full scale pressure and an accuracy of $\pm 0.14 \text{ kPa}$ and the diaphragm used for pore pressure measurements was of 35 kPa full scale pressure and $\pm 0.09 \text{ kPa}$ accuracy.

The absolute pressure transducer connected to the back pressure chamber of the consolidometer is employed during B-parameter test. This transducer measures the increase in porepressure in response to a known increase in total stress applied to the sample.

The air pressure supply to the system was provided through two Geotest Model S-5430-SMS pressure regulators. The regulators can be set to work in a differential mode to maintain a constant seating load during saturation phase. i.e.. The regulator feeding the load chamber of the consolidometer can be set at say 2.1 kPa above the regulator supplying to the back pressure chamber. However this differential of 2.1 kPa (which was found to be the minimum that can be maintained reliably) proved to be excessive for the soft Baltic sediments.

Therefore, an alternate arrangement consisting of an oil reservoir with a height adjustment mechanism was incorporated into the system. With this arrangement the pressure is supplied to both the load chamber and the backpressure chamber by the same regulator. The load pressure is applied through the oil reservoir and therefore the net seating load acting on the sample is the head due to the oil column plus the stress due to the buoyant weight of the stone and the piston. This arrangement enabled seating loads as low as 0.3 kPa to be applied to the sample during backpressuring.

The three bladder accumulators (Geostore Model S-470) employed in the system essentially provide interfaces between the various fluids. The load bladder accumulator provides the interface between the air pressure supply and sea water preventing dissolution of air into the permeant fluid. The backpressure bladder accumulator separates the backpressure regulator from the backpressure chamber containing sea water. The sea water/oil bladder accumulator forms the interface between mineral oil filling the load chamber of the consolidometer and the sea water contained in the flow pumps thus protecting the sensitive components of the consolidometer from corrosive

effects of sea water. Stainless tubing, valves and fittings have been used throughout the system to prevent corrosion problems.

The data acquisition system consists of a Macintosh II ci computer having an 80 MB hard drive and 8 MB RAM, a Mac ADIOS II multifunction analog to digital (A/D) board, a SBEL signal conditioning and power supply device and a Soltec model 1234 strip chart recorder. A custom built signal conditioning and power supply device with forty data input channels interfaced with the A/D converter board provides power to the DPTs. The data acquisition program residing on the host computer (Macintosh II ci) is capable of continuously recording up to eight channels of input from a single system. The strip chart recorder functions as a backup and helps to monitor the progress of permeability tests.

Calibrations

The LVDT which measures the deformation of the sample during consolidation was calibrated using a dial gage with 0.0025 cm (0.001 inch) divisions. The LVDT has a maximum stroke of 0.635 cm (± 0.25 inch) which corresponds to ± 2048 counts as output. The top half of the consolidometer with the LVDT and loading piston assembled in place was mounted on a vertically sliding mechanism. The consolidometer was advanced downwards from its top position in steps of approximately 0.058 cm (0.02 inches) while the LVDT was held stationary using fixed support. The relative movement of the piston (and the attached LVDT) was monitored using the dial gage while the LVDT output was recorded in terms of counts, SBEL signal conditioning device output as well as strip chart output. This information is used to derive the LVDT calibration graph as shown Figure B-3.

The effective stress transducer was calibrated using the pressure regulator. The applied stress on one side of the transducer was increased in steps of 6.895 kPa (one psi) up to the full scale pressure while the other side was exposed to atmospheric

pressure. The response of the diaphragm was recorded in terms of counts and strip chart reading and was used to obtain the effective stress calibration graph (Figure B-4). The calibration was verified before each individual test.

The pore pressure transducer was calibrated to measure a maximum of 13.79 kPa (2 psi) and hence the pressure regulators (which have a minimum pressure level of approximately 1.5 psi) could not be employed for the purpose. A water column with ruler attached to it was used to apply known pressures to the transducer in this case. The counts and chart readings were recorded as before for developing the calibration graph (Figure B-5). The absolute pressure transducer was calibrated using the Pressure regulator to a full scale pressure of 483 kPa (70 psi).

The flow rates delivered by the permeability flow pump were calibrated by measuring the piston travel velocities. This was accomplished by mounting a dial gage with 0.0025 cm (0.001 inch) divisions against the flow pump piston and monitoring the distance traveled at a specified speed rating of the pump using a timer. The flow velocities are then calculated from the piston velocity and the known area of cross section of the syringe. The flow rates obtained are tabulated in Table B-1. The rates selected are the ones at which permeability tests were normally conducted. These flow rates are within 2% to 5% of the flow rates as furnished by the manufacturer. The larger deviations were observed at less than 100% speed ratings of a given gear.

Testing Procedures

The CRD tests were performed in accordance with ASTM D-4186. All the samples tested under this study were of 5.1 cm. diameter by 2.85 cm height. The samples were of low shear strength and consistency and special care was necessary to ensure that they were not disturbed during sampling. The samples were extruded from the sample tubes into the consolidometer ring with acrylic pistons and trimmed to correct dimensions with a wire saw. Water content samples were taken from bottom and top of

the sample. The consolidometer was closed with the loading piston in a raised position. The piston is then lowered slowly to the correct position by filling the load chamber of the consolidometer under gravity and secured in that position by closing off the load port valve.

The back pressure chamber is now filled with deaerated sea water from a reservoir through the back pressure port. The consolidometer is tilted slightly to allow all the air to escape through the bleed port of the backpressure chamber. When sea water fills the backpressure chamber and starts to flow out through the bleed port, both ports are closed. The initial position of the LVDT is recorded.

The saturation phase is initiated by connecting the consolidometer ports to the appropriate valves and opening the load chamber to the oil reservoir. Saturation pressure is increased in steps of 5 psi (34.5 kPa) every half hour up to 60 psi (413.7 kPa). A small seating load of 0.2 kPa is maintained throughout saturation. The sample is kept at 413.7 kPa (60 psi) overnight to ensure complete saturation after which a B-parameter test is performed. This involves applying a known load increase (usually $\Delta\sigma = 6.9$ kPa) to the load pressure chamber (with the back pressure valve closed) and measuring the resulting increase in porepressure Δu with the absolute pressure transducer. The B-parameter is calculated as

$$B = \Delta u / \Delta\sigma \quad (B-1)$$

Values of B obtained during tests on the Baltic samples ranged from 0.96 to 0.99 indicating satisfactory level of saturation.

Constant Rate of Deformation of consolidation is initiated by closing off connection to the oil reservoir and starting the load flow pump. ASTM guidelines for determination of strain rates was found unsuitable for the Baltic samples as the liquid limit values were far beyond the range of ASTM specifications. Extrapolation from

these values would result in inordinately long testing time. The guideline adopted for determining the strain rate was the Norwegian Geotechnical Institute's recommendation which specifies that the strain rates should be such that the pore pressures induced should not exceed 10% of the applied effective stress (Crawford, 1988). A strain rate of 4.9×10^{-7} /s corresponding to gear 12, 50% speed rating of the flow pump was found to be sufficient to maintain the pore pressure well below 10%. This strain rate was employed in most of the tests to ensure that all samples were subjected to the same strain rate effects. The effective stress was built up to well beyond the preconsolidation stress and then rebound was initiated. In some tests the sample was reloaded and unloaded again to enable determination of recompression index.

Continuous records of total stress, base pore pressure and sample deformation were maintained during the test through the data acquisition software. The strip chart recorder was used to maintain a back up record of the data.

Three to fourteen permeability determinations were performed during the course of each test. The first test was started after the sample was well seated to avoid flow around samples (channeling). All permeability tests were done by withdrawing pore fluid from the bottom of the sample during the loading phases of the consolidation test. The strip chart recorder was used to judge whether the differential pressure had reached steady state. Different flow rates were selected at different stages of consolidation to maintain the gradients below five (5). Temperatures were recorded during the permeability tests to enable temperature corrections for permeability to be made. At the end of consolidation the consolidated specimen was extracted and the whole sample used for water content determination.

Strain Rates

The issue of the appropriate strain rate at which to run a constant rate of strain test is still an open question (Crawford, 1988). There are two aspects to the problem.

The first is the effect of strain rate on the primary (hydraulic consolidation) and the second, its effect on secondary consolidation (creep). A higher strain rate induces higher pore pressures at the base of the sample resulting in a large variation of effective stress across the sample and consequently large (nonlinear) variation of void ratio. Since most solutions to the consolidation equation assumes a uniform (or linear) void ratio distribution across the sample, higher strain rates can result in inaccurate evaluation of preconsolidation stress and compression index.

The second influence of strain rate is on the secondary consolidation. The secondary consolidation is a creep phenomenon. As defined by ASTM it is "due principally to the adjustment of the internal structure of the soil mass". There is no reason to believe that the adjustment of the soil grains is not taking place during all stages of consolidation (Crawford, 1988). However in interpreting the results of standard one dimensional consolidation test results it is assumed that there is no secondary consolidation until the pore pressures have been fully dissipated (Holtz and Kovacs, 1981). The basis for this assumption is that for natural soils, the secondary component of settlement is a very small percentage of the primary settlement. The studies by Mesri and Godlewski (1977) on 22 different natural soils showed that the ratio of the coefficient of secondary settlement to compression index C_{α}/C_c varies from 0.025 to 0.10. Therefore the influence of strain rates on secondary compression is only a much smaller percentage of the total consolidation characteristics.

The solution by Smith & Wahls (1969) as adopted for these tests initially assumes a linear void ratio given by

$$e = e_0 - rt \left[1 - \frac{b}{r} \left(\frac{z - 0.5H}{H} \right) \right] \quad (\text{B-2})$$

where e_0 is the initial void ratio, r is the strain rate, H is the sample height and b is a parameter depending on variation in void ratio with depth and time. During the test,

deviations from linearity will increase with increase in pore pressures induced at the base. Therefore development of large pore pressures at the base of the sample as during a high strain rate test leads to inaccuracies in the test result. Further in the formulation by Smith and Wahls, the solution for pore pressure obtained by solving consolidation equation for the boundary conditions of the CRD test is simplified by replacing the term $(1 + e)$ which is a function of time t and depth z , by $(1 + \bar{e})$ (which is a function of time t only, where $\bar{e}$ is the average void ratio across the sample. This is somewhat equivalent to assuming a uniform void ratio across the sample at all times. Therefore best results can be obtained by maintaining strain rates slow enough to keep low values of pore pressure across the sample.

The rate of strain in the field is estimated to be about $10^{-9}/s$. It is obviously impractical to conduct consolidation tests at such low strain rates. The strain rates during Standard 24 hour loading consolidation tests range from about $10^{-2}/s$ to $10^{-7}/s$ during a single load (Kabbaj et al., 1986). Standard tests of Baltic sea samples (load increment ratio 0.5) performed in conjunction with this study indicated that the strain rates varied from $1.7 \times 10^{-4}/s$ to $3.3 \times 10^{-8}/s$. It is known that at least for some soils, these differences in strain rate have a great influence on the interpretation of parameters such as the preconsolidation stress which cannot be ignored (Crawford, 1988).

The ASTM D-4186 method for the selection of strain rates for CRD tests is based on the liquid limit of the soil with the intention of keeping the excess pore water pressures between 2 and 20% of the applied vertical stress. These cannot be applied satisfactorily to the Baltic samples since the range of liquid limit covered by ASTM guidelines is only up to 140 whereas typical liquid limits for the Baltic samples are from 200% to 250%. extrapolation of the ASTM rates to these liquid limits result in strain rates lower than $1 \times 10^{-9}/s$. the ASTM rates were found unsuitable by earlier researchers (Armour and Drnevich, 1986; Crawford, 1988) as well.

Larsson and Sallfors (1986) reported that for constant rate of strain tests conducted in Sweden a rate of strain of $2 \times 10^{-6}/s$ was adopted which allowed pore pressures of less than 10% of the applied stress to be developed at the sample base. These rates were selected on the basis of a study of various strain rates in comparison with multiple stage loading tests. It was assumed that no further reduction in preconsolidation pressure occurred below this rate. The criterion adopted by the Norwegian Geotechnical Institute for selection of strain rates is such that pore pressures do not exceed 10% of the applied effective stress (Crawford, 1988). This criterion of strain rate selection was adopted for the tests performed on the Baltic samples.

The strain rate required to maintain a specified u_b/σ_v is found to be a function of permeability, liquidity index and compressibility. Therefore different samples could be tested at different strain rates as long as the pore pressure did not exceed the 10% limit. However in order to preclude any possible strain rate effects in making comparisons between different samples it was decided to select a common strain rate for all tests on Baltic sediments. Based on initial tests a strain rate of $4.9 \times 10^{-7}/s$ was found to be sufficient to maintain the base pore pressure below 10% level for all samples. Some tests were run at intermediate strain rates of $8 \times 10^{-7}/s$ and in these tests u_b reached close to, but below 10% . Tests that were run at strain rates higher than $8 \times 10^{-7}/s$ caused the pore pressure to increase well above the 10% limit. Results from these tests are eliminated from the analysis of stress state.

Data Reduction

Continuous records of sample deformation Δh , total stress σ and pore pressure at the base of the sample u_b are maintained during the CRD test. During the initial stages of the test, data sampling is done at 10 to 15 minutes intervals. During later stages data is recorded at half hour intervals. This results in 300 to 600 data points on the e -log σ' curve which provides a nearly continuous curve. Typically, in CRD tests

the sample deformation is evaluated from the known constant rate of deformation r and the time elapsed, t . The CRD system employed for tests in this study however incorporates an LVDT for monitoring the sample deformation directly. Therefore the instantaneous void ratio e is obtained from the relation

$$e = e_o - \Delta h / H_s \quad (B-3)$$

where e_o = is the initial void ratio

H_s = is the height of solids evaluated from phase relations

ΔH = is the deformation

The initial void ratio of the sample is obtained from phase relations using the water content.

The average effective stress $\bar{\sigma}$ acting on the sample is evaluated from the relation

$$\bar{\sigma} = \sigma - \alpha u_b \quad (B-4)$$

where α is assumed to be 2/3 in accordance with the solution by Smith and Wahls (1969) and the total stress σ and pore pressure u_b are measured directly. Typically plots of variation of e , σ' and u_b with time are developed from the above relations (Figure B-6) and serve as a record of the progress of the test. The narrow peaks in the pore pressure curve represent the permeability determinations. These basic parameters are used to obtain the e -log σ curve. The data points recorded during permeability tests are eliminated from the e log σ curve since pore pressure gradients resulting from the flow are superimposed on the pore pressure developed due to consolidation. The pore pressure level returns to that due to consolidation alone at the end of permeability tests. The large number of data points obtained during the CRD test enable the

preconsolidation stress to be defined more precisely compared to standard consolidometer tests. Typical e -log σ curves (Figure 7) however indicate that the system is highly sensitive to the level of disturbance to the specimens.

Coefficient of Consolidation

The coefficient of consolidation c_v for the tested samples have been evaluated from the e -log σ' and e -log k relationships. Since direct permeability measurements are made using the permeability flow during various stages of consolidation, c_v can be evaluated directly rather by inverse solution as in the case of standard consolidometer tests. The coefficient of compressibility a_v is evaluated from the slope of the e -log σ curve.

$$a_v = -\frac{e_2 - e_1}{\sigma_2 - \sigma_1} \quad (\text{B-5})$$

where e_1 and e_2 are the initial and final void ratios for a given interval and σ_1^1 and σ_2^1 are corresponding effective stresses. The specific storage s_s is calculated from the relation

$$s_s = \frac{a_v \gamma_w}{1 + e} \quad (\text{B-6})$$

where a_v is the coefficient of compressibility and γ_w is the unit weight of the pore fluid.

The coefficient of consolidation is given by

$$c_v = \frac{k}{s_s} \quad (\text{B-7})$$

The permeability values, k are obtained from the e -log k plots. The c_v values are plotted against the average void ratio for each interval. As expected, c_v values from

individual tests show a reduction in the magnitude of coefficient of consolidation with decreasing void ratios during consolidation (Figure 12). Typical values for c_v range from 10^{-3} cm²/s to 10^{-5} cm /s.

Comparisons with Standard consolidation tests

Two consolidation tests were performed on standard incremental loading consolidometers to enable comparisons between results from the CRD tests and 24 hr multiple stage loading tests. The Baltic samples exhibit significant variability of geotechnical properties even between samples from the same cores and same depth. Therefore one set of comparisons were made by testing remolded pacific illite which ensured uniformity between samples. The other comparison were made between samples taken from the same Baltic cores at the same depth.

The CRD test on the Pacific illite was done at a strain rate of 8.6×10^{-7} /s. A B-parameter of 0.96 was achieved at the end of back pressuring sequence. The test was completed in 6 days and five permeability measurements were made. The MSL 24 hr loading test was done on a Soil Test consolidometer and a load increment ratio of 0.5 was adopted. The test took 14 days to complete. When accounted for the slight difference in initial void ratios in the two tests due to the effect of backpressuring in the CRD test, The e-log σ curves obtained from both tests (Figure B-7) show very good correlation. Even though the sample was remolded both tests indicate a small preconsolidation stress; 1.3 kPa from the CRD test and 2.7 kPa from the standard test. The compression index as evaluated from the CRD test is found to be 0.53 and 0.66 from the standard test.

The second set of comparisons were made between two samples from the Baltic sea core 0225-BS-BC at 33-38 cm depth. A strain rate of 4.3×10^{-7} /s was adopted for the CRD tests (loading phase). During the test, pore pressures were maintained at less than 10% of the effective stress. The effective stress was built up to 10 kPa and then the

sample was unloaded. The load increment ratio adopted for the standard consolidometer test was 0.5. The strain rates during standard test were monitored and varied between $0.3 \times 10^{-8}/s$ to $1.7 \times 10^{-4}/s$ during the loading phase. Permeability and coefficient of consolidation were evaluated from the standard tests as well as the CRD test for comparison.

The comparison between results summarized in Table 4 show that there is reasonable agreement between the preconsolidation stress, compression index and permeability parameters obtained from the two types of tests. The CRD test appears more sensitive to the effects of sample disturbance compared to the standard test as judged from the e - $\log \sigma$ curves (Figure B-8). This is attributed to the inherent friction in the lever mechanisms of the standard consolidometers which becomes important at the very low stress levels applied to the Baltic samples. The permeability results and coefficient of consolidation are also compared in Figures 8. In view of the inherent variability of properties exhibited by the Baltic samples it is difficult to judge the impact of the test methods on the differences in results.

This series of tests indicate that reasonable agreement exist between the geotechnical parameters as derived from the two types of tests for the Baltic sediments for the rate of strains adopted. Even though there were large variations in the strain rate during the standard consolidometer tests, the strain rates stabilized after dissipation of excess pore pressure. Therefore it is concluded that the strain rate effects are small on these materials if the rates are slow enough to ensure that excess pore pressure dissipation is nearly complete during the CRD test.

Mineralogy tests

In order to determine the mineral composition of samples from the study site, X-ray diffraction was performed on five samples (from core 0238-BS-BC at depths of 0.0 -0.5 cm, 4-7 cm and 46-48 cm and from core 029-BS-GC at depths of 60 cm and 400

cm). A Scintag XDS-2000 theta-theta diffractometer was used to scan the samples with unfiltered Cu-K α radiation at 45kV and 40 mA. The first set of scans were done on bulk powder samples from 1° to 32° Bragg angles at 1° per minute scan rate. The diffraction patterns from these scans showed clear peaks of quartz and relatively weak peaks of illite and plagioclase. In order to better identify the clay minerals a second set of scans were done on the smaller than 2 mm fractions of the same samples. These tests clearly showed the presence of smectite and also indicated kaolinite (Figure B-9). The smectite fraction was confirmed (in view of possible interference from chlorite peak) by glycolating the samples at 50° C and scanning them again up to 10° angle.

Table B-1. Permeability Flow Pump Rate Calibrations.

Gear	Speed setting	Velocity cm/s	Area (cm ²)	Flow (cm ³ /s)
5	100%	5.53E-03	0.897	7.68E-04
6	100%	2.20E-03	0.897	3.06E-03
7	100%	1.10E-03	0.897	1.53E-04
8	100%	5.55E-04	0.897	7.70E-05
8	50%	2.91E-04	0.897	4.04E-04
8	25%	1.57E-04	0.897	2.18E-04
8	70%	3.92E-04	0.897	5.44E-04
9	100%	2.22E-04	0.897	3.09E-05
10	100%	1.06E-04	0.897	1.83E-05
11	100%	5.29E-05	0.897	7.35E-06
12	100%	2.12E-05	0.897	2.95E-06

**Figure B-1. Cross Section of Load Flow Pump Syringe
(Modified from Gavrisheff, 1992).**

Figure B-2. Cross-Section of Differential Pressure Transducer (Modified from Gavrisheff, 1992).

Figure B-3. Typical LVDT Calibration.

Figure B-4. Typical Effective Stress Calibration.

Figure B-5. Typical Pore Pressure Calibration.

Figure B-6. Variation of (a) Void Ratio , (b) Effective Stress and (c) Pore Pressure with time (Typical); Sample 052-BS-GC 253-258 cm.

Figure B-7 Remolded Illite (North Central Pacific); Comparison Between Standard and CRD Consolidation Tests

Figure B-8. Comparison of Consolidation, Permeability and Coefficient of Consolidation Results from CRD and Standard Consolidometer Tests; Sample 225-BS-BC, 33-38 cm.

Figure B-9. X-Ray Diffraction Pattern of Smaller than 2 micron Fraction;
Sample 238-BS-BC.

APPENDIX C

Consolidation and Permeability Results

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Consolidation and Permeability Results

The results from CRD consolidation and permeability tests on the twenty two (Table 4) undisturbed sediment samples are presented in figures C-1 to C-22. Part (a) of the figures show the compression curve. The preconsolidation stress evaluated by the Casagrande procedure is indicated on the respective figures. Part (b) of the figures show the permeability results. Regression lines have been fitted to the e -log k plots.

Figure C-1. Compression Curve and Permeability Measurements; Sample 029-BS-BC, 71-75 cm.

(a) Compression Curve

(b) Permeability Measurements

Figure C-2. Compression Curve and Permeability Measurements; Sample 029-BS-BC, 375-379 cm.

Figure C-3. Compression Curve and Permeability measurements; Sample 034-BS-GC, 142 cm Depth.

Figure C-4. Compression Curve and Permeability Measurements; Sample 035-BS-BC, 13-19 cm.

(a) Compression curve

(b) Permeability Measurements

Figure C-5. Compression Curve and Permeability Measurements; Sample 035-BS-BC, 23-28 cm Depth.

Figure C-6. Compression Curve and Permeability Measurements; Sample 035-BS-BC, 30-35 cm.

Figure C-7. Compression Curve and Permeability Measurements; Sample 038-BS-BC, 17-21 cm Depth.

Figure C-8. Compression Curve and Permeability Measurements; Sample 040-BS-BC (1), 29-34 cm Depth.

**Figure C-9. Compression Curve and Permeability Measurements;
Sample 040-BS-BC(2), 29-34 cm.**

Figure C-10. Compression Curve and Permeability Measurements;
Sample 052-BS-GC, 253-258 cm.

Figure C-11. Compression Curve and Permeability Measurements; Sample 225-BS-BC, 13-18 cm.

(a) Compression Curve

(b) Permeability Measurements

**Figure C-12. Compression Curve and Permeability Measurements;
Sample 225-BS-BC, 33-38 cm.**

Figure C-13. Compression Curve and Permeability Measurements; Sample 238-BS-BC, 28-33 cm.

Figure C-14. Compression Curve and Permeability Measurements;
Sample 252-BS-BC(1), 27-39 cm.

(a) Compression Curve

(b) Permeability Measurements

Figure C-15. Compression Curve and Permeability Measurements; Sample 252-BS-BC (2), 27-31 cm.

Figure C-16. Compression Curve and Permeability Measurements; Sample 264-BS-BC, 33 cm (Horizontal).

Figure C-17. Compression Curve and Permeability Measurements; Sample 264-BS-BC, 34-38 cm (Shelby).

Figure C-18. Compression Curve and Permeability Measurements; Sample 304-BS-GC, 138-143 cm.

Figure C-19. Compression Curve and Permeability Measurements; 304-BS-GC, 147-152 cm.

Figure C-20. Compression Curve and Permeability Measurements;
Sample 318-BS-GC, 141-146 cm.

Figure C-21. Compression Curve and Permeability Measurements; Sample 318-BS-GC, 154-159 cm.

Figure C-22. Compression Curve and Permeability Measurements;
Sample 333-BS-BC, 58-62 cm.

APPENDIX D

Consolidation and Permeability Results from Remolded Samples

APPENDIX D

Consolidation and Permeability Results from Remolded Samples

Samples 225-BS-BC, 37 cm and 252-BS-BC, 29 cm were remolded under vacuum and tested using the CRD system. The consolidation and permeability results from these two (Table 4) sediment samples are presented in figures D-1 and D-2. Part (a) of the figures show the consolidation results and part (b) shows the permeability results. Regression lines have been fitted to the e -log k plots.

Figure D-1. Compression Curve and Permeability Measurements; Sample 225-BS-BC, 34-39 cm (Remolded).

Figure D-2. Compression Curve and Permeability Measurements; Sample 252-BS-BC, 27-31 cm (Remolded) .

APPENDIX E

Coefficients of Consolidation

APPENDIX E

Coefficients of Consolidation

The coefficients of consolidation were evaluated at void ratios corresponding to the permeability measurements from the e -log σ and e -log k relationships. The c_v values are plotted against the average value of the void ratio for the interval for which c_v is evaluated in figures E-1 to E-14. An increase in c_v with void ratio is evident from most plots. The coefficients of consolidation range from 10^{-3} cm²/s to 10^{-5} cm²/s.

Figure E-1. Variation of Coefficient of Consolidation with Void Ratio; Sample 029-BS-GC, 71-75 cm.

Figure E-2. Variation of Coefficient of Consolidation with Void Ratio; Sample 034-BS-GC, 140-144 cm.

Figure E-3. Variation of Coefficient of Consolidation with Void Ratio; Sample 035-BS-BC, 13-19 cm.

Figure E-4. Variation of Coefficient of Consolidation with Void Ratio; Sample 040-BS-BC, 29-34 cm.

Figure E-5. Variation of Coefficient of Consolidation with Void Ratio; Sample 040-BS-GC (2), 29-34 cm.

Figure E-6. Variation of Coefficient of Consolidation with Void Ratio; Sample 052-BS-GC, 253-258 cm.

Figure E-7. Variation of Coefficient of Consolidation with Void Ratio; Sample 225-BS-BC, 13-19 cm.

Figure E-8. Variation of Coefficient of Consolidation with Void Ratio; Sample 225-BS-BC, 33-38 cm.

Figure E-9. Variation of Coefficient of Consolidation with Void Ratio; Sample 225-BS-GC, 34-39 cm (Remolded).

Figure E-10. Variation of Coefficient of Consolidation with Void Ratio; Sample 238-BS-BC, 28-33 cm.

Figure E-11. Variation of Coefficient of Consolidation with Void Ratio; Sample 252-BS-BC, 27-31 (1) cm.

**Figure E-12. Variation of Coefficient of Consolidation with Void Ratio;
Sample 304-BS-GC, 138-143 cm.**

Figure E-13. Variation of Coefficient of Consolidation with Void Ratio;
Sample 304-BS-GC, 147-152 cm.

**Figure E-14. Variation of Coefficient of Consolidation with Void Ratio ;
Sample 333-BS-GC, 58-62 cm.**

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