Influence of Shear Rate on Undrained Vane Shear Strength of Organic Harbor Mud

Benjamin Friedrich Schlue, Ph.D.[1]; Tobias Moerz, Ph.D.[2]; and Stefan Kreiter, Ph.D.[3]

Abstract: Dredging operations in European harbors for maintenance of navigable water depth produce vast amounts of harbor mud. Between 2005 and 2007, the second largest harbor construction project in Germany was designed as a pilot study to use dredged harbor mud as backfill material to avoid expensive disposal or ex situ treatment. During this project, a partial collapse of the backfill highlighted the need for an improved assessment of undrained shear strength of naturally occurring liquid harbor mud. Using vane shear testing, this study evaluates the effect of shear rate on the undrained shear strength of harbor mud. It is shown that measured values for both peak and residual shear strength are significantly influenced by shear rate effects. Furthermore, the influence of shear rate on the peak shear strength is found to be independent of water content while the influence of the shear rate on the residual shear strength strongly depends on water content. New shear rate dependent correction factors μ are proposed using the test results and the observed time to failure in the harbor basin. The proposed correction leads to significant lower design undrained shear strengths than the classical Bjerrum correction and would have predicted the failure during the construction.

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Author keywords: Dredged mud; Soft clay; Vane shear test; Time effects; Correction factor; Shear rate effects.

Introduction

The major North Sea tidal estuaries of the Ems, Weser, and Elbe Rivers are characterized by vast accumulations of organic-rich mud derived from marine and river suspension transport. High rates of mud accumulation in these dynamic systems are driven by a complex interaction of variable river discharge and tide and wind induced processes. Increased mud accumulation is promoted by tidal asymmetry and particle flocculation in the freshwater/saltwater mixing zone (Schrottke et al. 2006). The highly engineered estuaries of the Ems, Weser, and Elbe Rivers belong to the most frequented waterways worldwide and host a number of important ports. Deepening the navigation channel in these rivers causes an artificial rise in the tidal range and an increase in the suspension load. The small-grained material is carried through the locks into the basins of the harbors in northern Germany where it settles as organic harbor mud. This mud must be regularly dredged out of the harbor basins in order to maintain the navigational depth. To avoid costly disposal of this often contaminated material, the dredged harbor mud is increasingly reused on-site as backfill and construction material, a procedure used during the backfilling of the harbor basin in the East Harbor of the international port area of the Free Hanseatic City of Bremen in Bremerhaven, Germany (Metzen et al. 2006). During this excavation and construction project, a total of about $180,000\ \mathrm{m}^3$ of soft to liquid harbor mud was relocated and used as backfill behind a heavy sheet pile structure that will serve as a new wharf (Fig. 1), creating 14 acres of new harbor area (Schlue et al. 2007, 2009). To accelerate the consolidation of the mud, a layer of geotextile and several thin sand layers were placed on top of the mud backfill. During the placement of the sand layers, larger parts of the backfilling area underwent vertical displacements indicating a partial collapse of the mud layer; nevertheless, the project was finished successfully in 2007. The described difficulties over the course of this project raised several questions regarding the determination of the undrained shear strength in harbor mud, which this study would like to address.

Vane shear testing played an important role during the planning and construction phase of the East Harbor extension project. It is widely used for its simplicity, speed and relative cost, and it is the only method commonly used in both laboratory and field settings in contrast to other soft soil test methods such as fall cone penetrometers or full-flow penetrometers (Zreik et al. 1995; Stewart and Randolph 1994). However, the results of vane shear tests are affected by many factors such as shear rate, strength anisotropy or rod friction effects (Aas 1965; Flaate 1966; Wiesel 1973; Menzies and Mailey 1976; Torstensson 1977; Biscontin and Pestana 2001; Schlue et al. 2007). Shear rate or peripheral velocity is among the most important factors affecting vane shear test results but there is currently no common international standard for peripheral velocity or rotation rate in vane shear tests (Leroueil and Marques 1996).

The problems in deriving the actual soil resistance from labo-

[1]Project Manager, ARCADIS Deutschland GmbH, Europaplatz 3, 64293 Darmstadt, Germany (corresponding author). E-mail: b.schlue@arcadis.de

[2]Professor, Dept. of Marine Engineering Geology, MARUM-Center for Marine Environmental Sciences, Univ. of Bremen, Leobener Str., MARUM-Building, 28359 Bremen, Germany.

[3]Postdoc, Dept. of Marine Engineering Geology, MARUM-Center for Marine Environmental Sciences, Univ. of Bremen, Leobener Str., MARUM-Building, 28359 Bremen, Germany.

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Fig. 1. Air photo of the East Harbor construction site, where 180,000 m^3 of harbor mud were relocated behind a sheet pile wall (photo used with permission of bremenports GmbH & Co. KG)

Table 1. Natural Soil Parameters and Mineral Composition

Parameter	Value	Unit
Particle density ρ_s	2.57	g/cm^3
Dry density ρ_d	0.46	g/cm^3
Bulk density ρ	1.55	g/cm^3
Initial water content w_0	1.83	—
Initial void ratio e_0	4.64	—
Initial water permeability k	10^{-8}	m/s
Content of smectite	36	w%
Content of illite	36	w%
Content of kaolinite/chlorite	28	w%
Loss on ignition V_{Gl}	12.0	w%
C_{tot}	4.602	w%
TOC	2.932	w%
S_{tot}	0.723	w%
Liquid limit w_L	1.410	—
Plastic limit w_P	0.484	—
Plasticity index I_P	0.926	—
Pore-water salinity	10.85	g/L

ratory and field vane shear tests in harbor mud motivated this study and raised the following questions:

- What influence does shear rate have on vane shear test results in liquid to soft harbor mud?
- Is the influence of shear rate on the measured undrained shear strength of harbor mud comparable to relationships established for other cohesive sediments?
- Is the correction factor μ proposed by Bjerrum for shear rate effects applicable to dredged harbor mud?

Below, a brief description of the dredged harbor mud and test procedures is provided. The test results are then presented and evaluated with respect to the questions raised.

Soil Characterization and Testing Procedures

Soil Characterization

The examined sediment is harbor mud from the East Harbor in Bremerhaven on the banks of the Weser River and has been described in detail by Schlue et al. (2007, 2009). The harbor mud settles under brackish conditions in the tide independent quiet waters of the harbor basin. The suspended solids originate from the Weser River, which is connected to the East Harbor basin by a lock.

The representative sample material was obtained by piston coring after dredging and redeposition of the mud; characteristic soil parameters were determined by preliminary laboratory tests. The harbor mud is an extremely plastic, organogenic clay, with a liquid to soft consistency (Table 1; German Standards Organisation 1996, 2000, 2002a). The grain size distribution is shown on Fig. 2. The brackish environment is confirmed by the pore-water conductivity of 18.35 mS/cm corresponding to a salinity of 10.85 g/L (Unesco 1981).

The clay mineral composition was determined by X-ray diffraction and shows no predominance of any clay mineral group (Table 1). The clay mineralogy of the harbor mud is similar to clays in tidal flats in the southeastern North Sea in having a high content of expandable smectites, considered to originate from Pleistocene subsurface strata (Zöllmer and Irion 1996). A comparison of the amount of clastic material supplied by the Ems, Weser, and Elbe Rivers during the past 7,500 years showed that about 90% of the North Sea coastal deposits are of marine origin, while only about 10% are of fluvial sources (Hoselmann and Streif 2004). The main organic substances encountered in marine mud are polysaccharides and proteins composed of peptides and amino acid, lipids, hydrocarbons like cellulose, lignin composed of aliphatic and aromatic hydrocarbons as well as humic acids (Berner 1980). The extreme, and geotechnically problematic properties of this harbor mud are probably promoted by these organic substances.

Test Procedures

At first, a statistically valid sample material from several barrels was homogenized using a handheld soil mixer, where a high water content of the mud allowed for viscous behavior and effective mixing. Subsequently, the homogenized harbor mud was dried in a drying oven at 60°C until the liquid consistency was transformed into a soft consistency. This temperature was chosen to ensure a gentle drying process without decomposing organic matter or removing chemically bound water from the sample material. Since no salt was removed, the drying led to salinity varia-

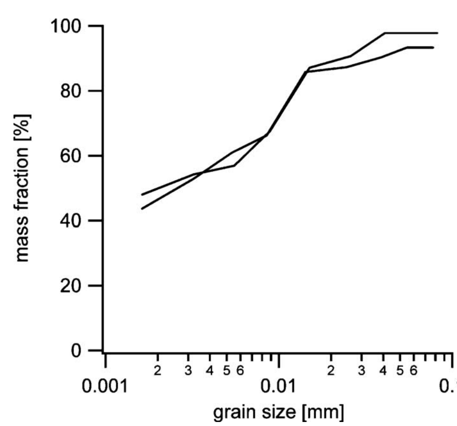

Fig. 2. Grain-size distribution examined by two separate analyses according to German Standards Organisation (1996)

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tions in the sample suite between 12 and 26 g/L. These salinity changes cause variation in the undrained shear strength of no more than 10% and correspond to the daily variation in the freshwater/saltwater mixing zone of the Weser estuary (Kay et al. 2005; Spaethe 2002). To maintain the same overall salt content and minimize the geochemical variations, three subsamples with water contents w of 0.71 (LI=0.25, soft consistency), 1.12 (LI =0.69, very soft consistency), and 1.77 (LI=1.39, liquid consistency) were produced by adding small quantities of deionized water. These water contents represent the observed range during the construction project in the East Harbor (Schlue et al. 2009). Eighteen laboratory vane shear measurements were then carried out on each soil consistency, varying the rate of rotation ω between 18 and 1,800°/min. These applied constant rates of rotation correspond to peripheral velocities v_{pV} $(=d \cdot \pi \cdot \omega / 360)$ from 1.57 to 157 mm/min for the vane used ($d|h|t=10|8.8|1\ \text{mm}^3$), assuring a shearing process under undrained conditions (Blight 1968; Matsui and Abe 1981). All measurements were taken with a high-precision laboratory vane shear test apparatus (Haake Rotovisco RV20, Haake, Karlsruhe, Germany) which provided a theoretical shear strength resolution of 0.00257 kPa for the vane used. All vane shear measurements were conducted in a container with a volume of $1{,}700\ \text{cm}^3$, preventing boundary effects and assuring sufficient clearance ($>3d$ from circumference of the shearing) from the edge and bottom of the container, and between individual measurements (ASTM 1987; Flaate 1966; v. Bloh 1995). The vane was inserted very slowly into the soil sample (<1 mm/s) to minimize the accumulation of excess pore-water pressure during insertion.

Although constant concentrations of free gas as small as <10 vol% can affect the absolute shear strength of a soil, these effects are not believed to have any significant influence on its shear rate dependence (Wichman et al. 2000; van Kessel and van Kesteren 2002; Schlue et al. 2009). Furthermore, the short time period between soil homogenization and the vane shear measurements is believed not to have allowed for the creation of any gas by biogeochemical degradation of organic carbon as would be expected in the field.

Results and Discussion

Undrained Peak and Residual Shear Strength Data from Vane Shear Tests

The measured undrained vane shear strength was evaluated and corrected for rod friction effects according to Schlue et al. (2007). The method accounts for shear stresses transferred where the rod connects to the vane and assumes a uniform shear stress distribution around the vane. The term residual shear strength is assigned to the measured shear stress after two complete vane revolutions (Biscontin and Pestana 2001). The measured shear strength for the three soil consistencies and peripheral velocities between 1.57 and 157 mm/min are illustrated on Fig. 3. At a liquid consistency (w=1.77, LI=1.39) the shear rate dependent peak shear strengths s_{uV} vary between 0.25 and 0.43 kPa and the residual shear strengths s_{urV} between 0.06 and 0.30 kPa. At a very soft consistency (w=1.12, LI=0.69), the s_{uV} increased to values between 1.26 and 2.14 kPa while s_{urV} increased to values between 0.43 and 1.30 kPa. At a soft consistency (w=0.71, LI=0.25) peak shear strengths increased substantially to 7.20 to 11.80 kPa and residual shear strengths of 2.90 to 6.50 kPa were measured.

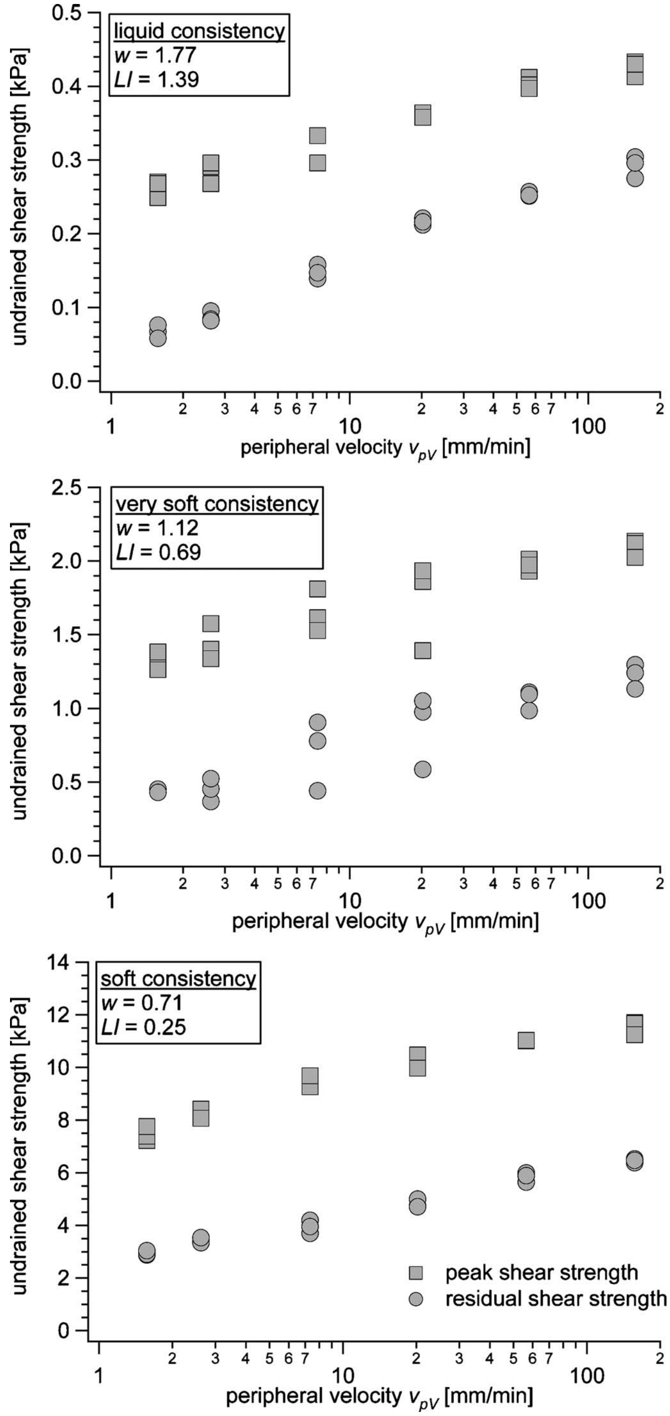

Fig. 3. Measured peak and residual shear strength versus peripheral velocity for the tested water contents

Influence of Shear Rate on Vane Shear Test Results in Organic Harbor Mud

To better illustrate the influence of shear rate on vane shear test results, the measured shear strengths are normalized to a reference shear strength. In this study, the reference peak shear strength s_{u0} and reference residual shear strength s_{ur0} are defined as the measured values at a peripheral velocity v_{p0} of 1.57 mm/min, which is the slowest possible velocity for the used test setup. The normalized vane shear strengths, s_{uV}/s_{u0} and s_{urV}/s_{ur0}, are shown on Fig. 4 for all consistencies tested. It can be seen that

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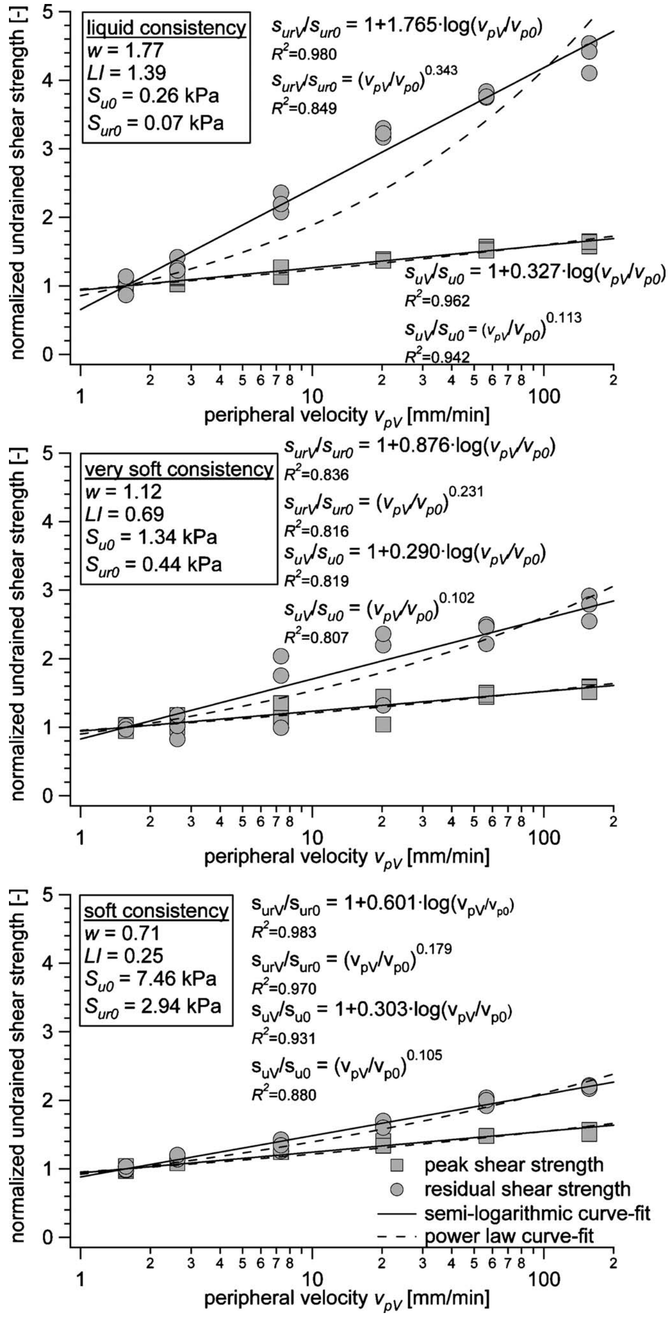

Fig. 4. Normalized peak and residual shear strength versus peripheral velocity for the tested water contents; curve fits according to Eqs. (1) and (2)

both the undrained peak and residual shear strength increase significantly with increasing peripheral velocity. The measured s_{uV} for the highest peripheral velocity of 157 mm/min is 55 to 65% higher than that determined at the reference velocity v_{p0}, which is consistent for the three consistencies tested. In contrast, the increase in s_{urV} varies between 120% at a soft consistency and 350% at a liquid consistency (Fig. 4) showing that for liquid to soft consistencies, the residual shear strength is even more sensitive to changes in peripheral velocity than the peak shear strength. Our findings are consistent with those of other studies, e.g., Biscontin and Pestana (2001). However, Biscontin and Pestana (2001) did not observe any shear rate influence on the residual shear strength of slightly cemented model soil consisting of kaolinite, bentonite, and fly ash. The influence of shear rate on the residual shear strength of harbor mud observed in this study can be explained by the increasingly fluid like soil behavior of decreasing viscosity with rising water content. The harbor mud behaves more and more like a sediment suspension following rheological models rather than exhibiting a unique coefficient of friction in the residual state. It is known that transferable shear stresses within sediment suspensions as well as their viscosity are strongly dependent on shear rate and water content (Verreet and Berlamont 1988; Coussot 1997; Whitehouse et al. 2000; Winterwerp and v. Kersteren 2004).

The dependence between the shear rate and measured shear strength has been traditionally interpreted according to a power or semilogarithmic law

$$\text{Power law}\quad s_u/s_{u0} = (v_p/v_{p0})^{\beta} \tag{1}$$

$$\text{Semilogarithmic law}\quad s_u/s_{u0} = 1 + \alpha \log(v_p/v_{p0}) \tag{2}$$

where α, β = characteristic soil parameters.

Fig. 4 shows both power-law and semilogarithmic curve fits according to Eqs. (1) and (2) for the measured normalized shear strengths. It can be observed that the undrained peak shear strength can be accurately described by both curve fits though the semilogarithmic fit is slightly better. Only at a liquid consistency is the undrained residual shear strength significantly better described by the semi logarithmic curve fit.

For peak shear strength, the evaluated parameter α from the semi logarithmic curve fit varies between 0.28 and 0.33, while under the power-law curve fit, the evaluated parameter β varies between 0.10 and 0.12. These are comparatively high values and correspond to an increase in measured undrained peak shear strength of approximately 30% per 10-fold increase in peripheral velocity. The harbor mud soil parameters α and β are listed in Table 2 for comparison with those found in other studies. From this it can be seen that the influence of shear rate on the peak shear strength of the tested harbor mud is comparable to that from the Gulf of Mexico clay studied by Schapery and Dunlap (1978) and to Exuma Sound clay from the Bahamas studied by Smith and Richards (1975).

In contrast to the peak shear strength, the measured parameters α and β for residual shear strength are also dependent on consistency because of the above mentioned viscosity effects dominating the behavior after mobilization of the peak shear strength at higher water contents and liquidity indices. Linear correlations between liquidity index LI and the shear rate dependence of the residual shear strength were tested for the dredged harbor mud, leading to the following equations:

$$\beta = 0.144\text{LI} + 0.14 \quad (-) \tag{3}$$

$$\alpha = 1.04\text{LI} + 0.277 \quad (-) \tag{4}$$

For $0.25 \leq \text{LI} \leq 1.39$ these equations yield values between $0.53 \leq \alpha \leq 1.72$ and $0.18 \leq \beta \leq 0.34$, corresponding to an increase in measured residual shear strength of 50 to 170% per 10-fold increase in peripheral velocity. As can be seen from Fig. 4, with decreasing w and LI, the parameters α and β approach the values found for the peak shear strength.

Derivation of a Correction Factor to account for Shear Rate Effects on Measured Vane Shear Strength

A significant shear rate dependence of undrained shear strength has been demonstrated by this study. This dependence underlines

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Table 2. Comparison of Derived Parameters α and β for the Peak Shear Strength (cf., Biscontin and Pestana 2001)

Clay	I_p (%)	α (%)	β	Reference
Grangemouth	22	5–6	0.025	Skempton (1948)
Bromma	31	20	0.086	Cadling and Odenstad (1950)
Aserum, Drammen, Lierstranda, Manglerud	9	1–2	0.006	Aas (1965)
Different silts and clays	—	16	0.05	Migliori and Lee (1971)
		5	0.01	Halwachs (1972)
Ska Edeby	50–100	3–6	0.02	Wiesel (1973)
Bäckebol	50–65	12	0.05	Torstensson (1977)
Askim	80–90			
Gulf of Maine and Mexico	—	13	0.05	Smith and Richards (1975)
Exuma Sound clay, Bahamas		27	0.1	
San Diego silt I	64	36	0.13	Perlow and Richards (1977)
San Diego silt II		21	0.08	
Gulf of Maine clay	78	60	0.2	
Gulf of Mexico clay	—	33	0.107	Schapery and Dunlap (1978)
Pierre shale	103	21	0.05	Sharifounnasab and Ullrich (1985)
Saint-Louis de Beaucours	13–19	2–3	0.01	Roy and LeBlanc (1988)
Saint-Alban clay	6–18	1	0.004	
Bentonite-Kaolinite mix	75	15	0.055	Biscontin and Pestana (2001)
East Harbor mud	93	30	0.11	This study

the need for a correction of vane shear test results to account for the different shear rates observed in large-scale field-failure processes and applied in vane tests (Bjerrum 1972, 1973). A correction factor μ can be derived from the ratio of undrained shear strength mobilized in the field at low shear rates and the shear strength measured by vane shear tests at higher shear rates. Thus, using Eqs. (1) and (2), μ can be derived as

$$\text{Power law} \quad \mu = \frac{(v_{pF}/v_{p0})^{\beta}}{(v_{pV}/v_{p0})^{\beta}} = \left(\frac{v_{pF}}{v_{pV}}\right)^{\beta} \tag{5}$$

$$\text{Semilogarithmic law} \quad \mu = \frac{1+\alpha \log(v_{pF}/v_{p0})}{1+\alpha \log(v_{pV}/v_{p0})} \tag{6}$$

where v_{pF}=peripheral velocity that would correspond to the shear rate in the field and v_{pV}=peripheral velocity used in vane shear tests.

Eq. (6) has a limited domain because decreasing peripheral velocities in the semilogarithmic curve-fit Eq. (2) return unrealistic, small or even negative shear strengths and cannot be extrapolated to small peripheral velocities. Eq. (6) is therefore undefined for a wide range of v_{pF} and α which was found to be relevant for this study. Since the power-law derived correction factor from Eq. (5) has no such problems in the following part of this study, the power-law correction is used, even though it is more appropriately used for the semilogarithmic approximation (Hinchberger 1996; Soga and Mitchell 1996).

Application of the Introduced Correction Factor on Organic Harbor Mud

After deposition of the harbor mud behind the sheet piling during the construction project in Bremerhaven, a geotextile layer was placed to prevent mixture between the mud and sand layers. These sand layers were meant to accelerate the consolidation of the mud and to achieve the required installation height of the backfill. The deposited mud layer in the backfill area collapsed after addition of the fourth sand layer, increasing the existing 40-cm sand layer by another 20 cm (Fig. 5). According to eye witnesses, strong deformations were noticeable within the first 10 h after completion of sand layer four.

Although the geometry and loading mechanisms are fundamentally different, this paper makes the assumption that the shear zone in the field and in the lab evolve in a similar way. Specifically, we assume that the shear zones are comparable in terms of relative displacement at failure. This assumption is central for the derivation of the typical shear rate in the field used in the following analysis. Two observations support this assumption: First, the harbor mud exhibits a clear peak in shear strength followed by strain softening. This soil behavior suggests that the shear strain localizes in a narrow zone after failure, because the first onset of failure softens the material. In granular material, localization of shear strain is observed for strain softening materials (Roscoe 1970; Vardoulakis and Graf 1985). Second, such localized shear zones were observed during the lab vane tests. The peripheral velocity dependence found by Biscontin and Pestana (2001) for different vane geometries also point to comparable shear zones for different geometries. If this assumption is accepted, the correction factor μ according to Eq. (5), can be expressed in terms of time to failure. The failure scenario in the East Harbor corresponds to a time to failure in the field t_{fF} of about 600 min (10 h). From the observed displacements along the shear plane at failure x_f and the applied peripheral velocities v_{pV} in the laboratory vane shear tests carried out for this study, the time to failure t_{fV} in the vane shear tests can be determined. Fig. 6 shows the observed displacements at failure x_f in the vane shear tests carried out for this study. No significant dependence on the shear velocity is observed

$$x_f \approx 2.1 \ \text{(mm)} \tag{7}$$

According to Eq. (7) the time to failure t_{fV} can now be estimated

$$t_{fV} = \frac{x_f}{v_{pV}} \approx \frac{2.1}{v_{pV}} \tag{8}$$

Using the assumption of comparable displacements along the shear plane prior to failure in the field situation and vane shear

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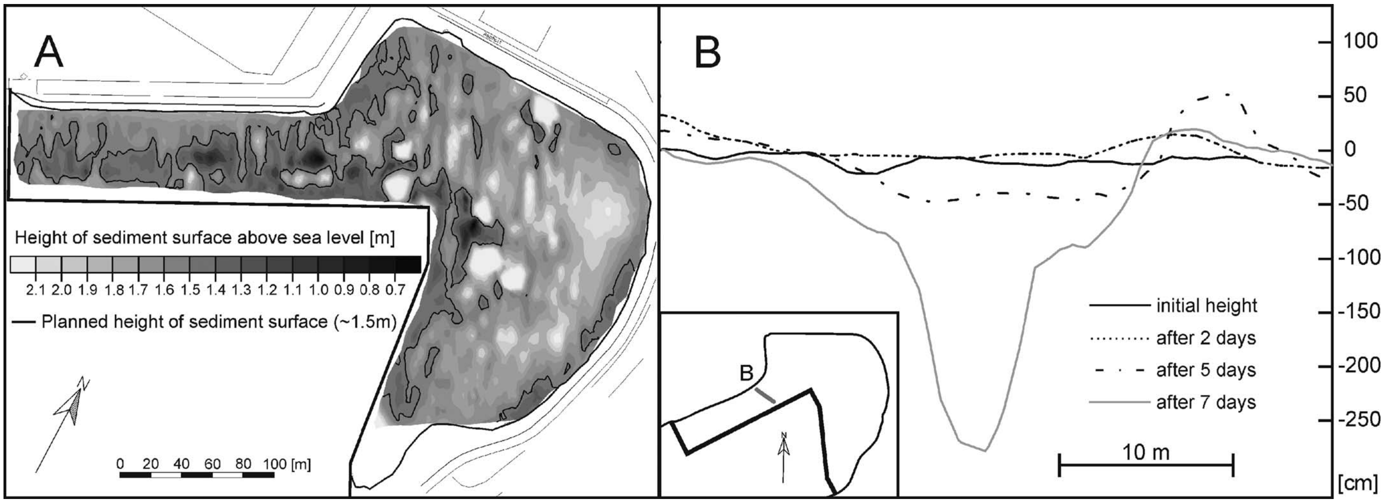

Fig. 5. (a) Strongly deformed backfill area 20 days after sand installation with ground motion amplitudes of up to 2 m; (b) height of the geotextile surface before and during sand installation obtained by soundings. Note the strong deformation that occurred between Days 5 and 7 after start of installation with amplitudes of 3 m (inset: location of the sounding Profile B).

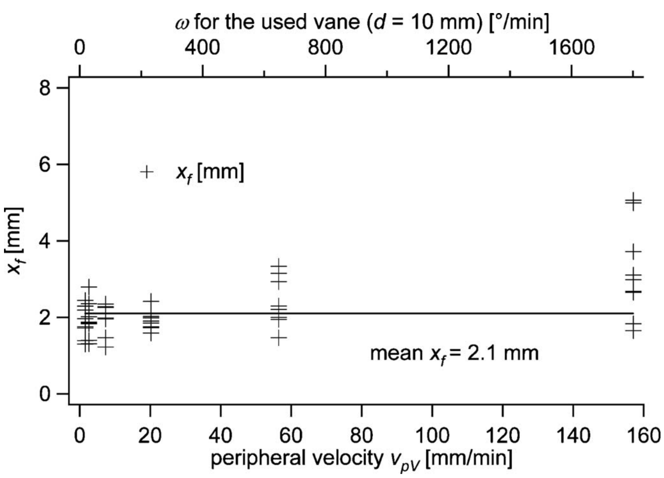

Fig. 6. Displacements along shear plane prior to failure versus peripheral velocity in laboratory vane shear tests in harbor mud

tests, the correction factor μ according to Eq. (5) is only dependent on the ratio t_{fF}/t_{fV} and the equation for α can be simplified using Eq. (8) and the observed t_{fF}=600 min

$$\mu = \left(\frac{v_{pF}}{v_{pV}}\right)^{\beta} \approx \left(\frac{t_{fV}}{t_{fF}}\right)^{\beta} = \left(\frac{2.1/v_{pV}}{600}\right)^{\beta} \approx (300 \cdot v_{pV})^{-\beta} \qquad (9)$$

Eq. (9) is based on the observed time to failure in the field of 600 min and is therefore based on the particular shear rate observed during the partial collapse of the extension area in the East Harbor. Considering the measured specific parameter μ and v_{pV}, Eq. (9) yields

$$\text{Peak shear strength } \mu \approx (300 \cdot v_{pV})^{-0.11} \qquad (10)$$

with β=0.11 for the peak shear strength.

The results of Eq. (10) for the peak shear strength, which are the main interest with regard to geotechnical design, are illustrated on Fig. 7 versus peripheral velocities and additionally as rates of rotation using different vane sizes commonly recommended by international standards. For the rates of rotation between 1 and 100°/min typically suggested by these international

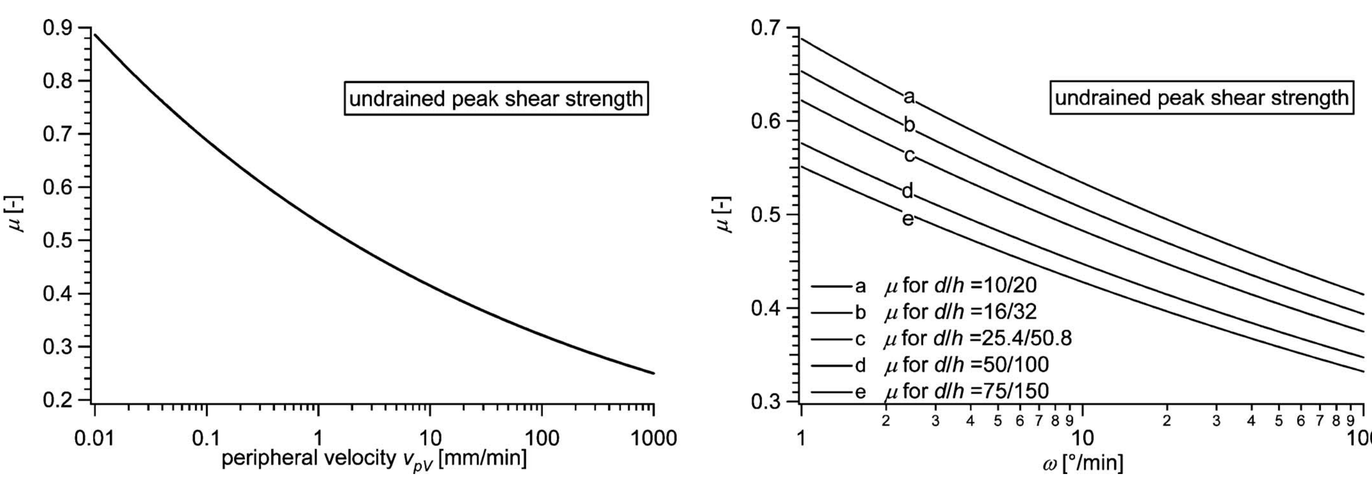

Fig. 7. Correction factors μ for the undrained peak shear strength of organic harbor mud versus peripheral velocity v_{pV} and rate of rotation ω using exemplary vanes geometries d/h (mm/mm) recommended by common international standards

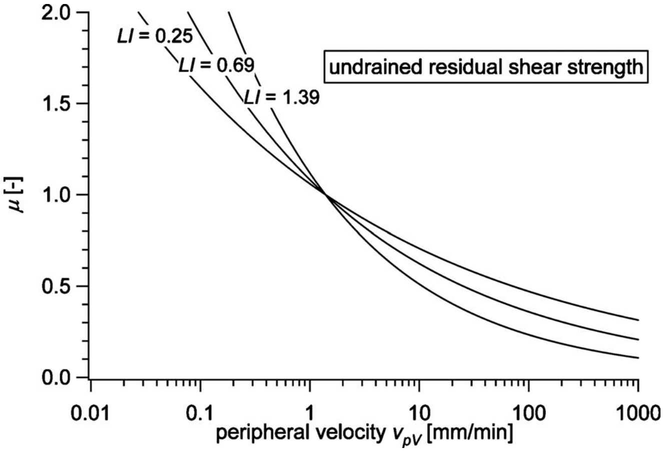

Fig. 8. Correction factors μ for the undrained residual shear strength of organic harbor mud versus peripheral velocity v_{pV} for the three consistencies tested

standards such as, ASTM (1987), British Standards Institution (1990), or German Standards Organisation (2002b), the proposed correction factor μ varies between 0.3 and 0.7 for the peak vane shear strength s_{uV}.

In a field-failure situation the shear rate will accelerate after mobilization of the peak shear strength (Fig. 5) and afterward decelerate until a stable state is reached because of the change of driving and resisting forces during and after failure. Therefore, Eq. (9) cannot be used to correct the residual shear strength of East Harbor mud. In the residual state of the collapse of the extension area in the East Harbor, large deformations with faster deformation rates were observed, as illustrated on Fig. 5(b). From the deformation between Day 5 and Day 7, it can be seen that the deformation rate was at least about 4 m within 2 days. This deformation corresponds to a minimum velocity of 1.39 mm/min, which is, according to Cruden and Varnes (1996), a moderate shear velocity often observed in natural landslides and ground failures in clayey soils. Using this minimum shear velocity in the residual state of 1.39 mm/min observed during the ground failure in the East Harbor, a conservative expression for a correction factor for the undrained residual shear strength of East Harbor mud is derived as

$$\text{Residual shear strength}\ \mu = \left(\frac{v_{pF}}{v_{pV}}\right)^{\beta} \approx \left(\frac{1.39}{v_{pV}}\right)^{\beta} \approx (0.72 \cdot v_{pV})^{-0.144 \cdot \mathrm{LI} - 0.14} \quad \text{for}\ 0.25 \leq \mathrm{LI} \leq 1.39 \tag{11}$$

with β according to Eq. (3) for the residual shear strength.

The results of Eq. (11) for the residual shear strength are illustrated on Fig. 8 versus peripheral velocities, which can be calculated for any given vane size and used at a rate of rotation according to $v_{pV} \approx d\pi\omega/360$.

With the derived correction factors to account for the shear rate effects on vane shear test results in organic harbor mud, the corrected undrained peak shear strength s_u and residual shear strength s_{ur} can now be estimated as

$$s_{u(r)} = \mu \cdot s_{u(r)V} \tag{12}$$

The application of Eqs. (10)–(12) allows for the correction of laboratory and field vane test data taken at arbitrary rates of rotation in harbor mud or other soft, extremely plastic, organogenic clays.

Comparison of the Derived Correction Factor with Those Proposed by Other Studies and International Standards for the Peak Vane Shear Strength

Based on numerous case studies on Scandinavian clays, Bjerrum proposed a correction factor μ for the undrained shear strength measured by vane shear tests to account for the longer time to failure in the field t_{fF}, which has typically been assumed to be several weeks or months (Bjerrum 1972, 1973). The short time to failure t_{fF} of about 600 min observed in the East Harbor is assumed to be due to the liquid to soft consistency of the harbor mud, so t_{fF} is assumed to be dependent on the plasticity and consistency of the soil. For clays with lower water contents, the time to failure may be longer, as observed by Bjerrum. Differences in t_{fF} may have a serious influence on the introduced correction factor μ compared to those recommended by Bjerrum and other writers.

Bjerrum proposes a correction factor μ to account for time and anisotropy effects in relation to the index of plasticity I_p based on a rate of rotation of 6°/min. For the organic harbor mud tested in this study with $I_p = 0.926$, the correction factor μ would be 0.63, independent of the actual used rate of rotation in vane shear tests and the specific time to failure of the sediment (Bjerrum 1973). This factor has been confirmed by a number of other studies (Azzouz et al. 1983; Larsson et al. 1984) and is also recommended by the German Standards Organisation (2002b) for soft sediments. Since there is no internationally accepted standard for the rate of rotation in vane shear tests, the recommended ω varies between 6 to 12°/min (British Standards Institution 1990), 30°/min (German Standards Organisation 2002b; for soft sediments) and 60 to 90°/min (ASTM 1987). In reality, measurements for large field monitoring campaigns are often carried out at higher rates of rotation in order to save time or to be able to do more single measurements. The results of Eq. (10) for the peak shear strength for these proposed rates of rotation are listed in Table 3 for common vane geometries represented by the ratio d/h, where $\Delta\mu$ is the relative difference to $\mu = 0.63$ as proposed by Bjerrum. Table 3 shows that for rates of rotation proposed by common international standards, the introduced shear rate dependent correction factor μ differs significantly from that found by Bjerrum. The larger the used vane at a given rate of rotation, the more important the correction of shear rate effects and therefore, the smaller the necessary correction factor μ. This is due to the larger peripheral velocities at a given rate of rotation for larger vanes. It can be seen on Fig. 7 that, for the vane used, the correction factor for the peak shear strength proposed in this study would be 0.63 for a rate of rotation of less than 3°/min. Consequently, the application of the correction factor proposed by Bjerrum in combination with the rates of rotation recommended after the standards British Standards Institution (1990), German Standards Organisation (2002b), and ASTM (1987) overestimates the undrained shear strength between 11 and 85%. This underlines the importance of applying a correction factor that takes into account the peripheral velocity of the vane of the actual measurements and soil specific shear rates during field-failure processes. This study presents a complete set of correction factors.

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Table 3. Comparison of Correction Factors μ Determined according to Eq. (10) for Rates of Rotation and Vane Geometries Proposed by Common International Standards; $\Delta\mu$ Is the Relative Differences to $\mu=0.63$ Proposed by Bjerrum

Standard	Proposed ω (degrees/min)	Vane geometry d/h (mm/mm)	μ according to Eq. (10) (-)	$\Delta\mu$ (%)
BS 1377-9 (British Standards Institution 1990)	6–12	10/20	0.57–0.52	11–21
		16/32	0.50–0.54	17–26
		25/50	0.47–0.51	24–34
		50/100	0.44–0.47	34–43
		75/150	0.42–0.45	40–50
DIN 4094-4 (German Standards Organisation 2002b)	30	10/20	0.47	34
		16/32	0.45	40
		25/50	0.43	47
		50/100	0.40	58
		75/150	0.38	66
ASTM D4648 (ASTM 1987)	60–90	10/20	0.44–0.42	43–50
		16/32	0.40–0.42	50–58
		25/50	0.38–0.40	58–66
		50/100	0.35–0.37	70–80
		75/150	0.34–0.35	80–85

Back-Analysis of the Ground Failure in the East Harbor and Validation of the Introduced Correction Factor

To check if the introduced new shear rate dependent correction factor would have predicted the ground failure during the East Harbor extension project, a simplified back-analysis was conducted. The sand layers that caused the failure were installed by spraying and settling the sand through the water column from a floating platform in 10-m wide strips. The sand layer that led to the ground failure has a thickness of about 20 cm, corresponding to a total stress increment of 2 kPa. Assuming a classic ground failure approach for flexible footing on cohesive soil, the minimum necessary undrained shear strength to avoid failure and the factor of safety for the geostatic system can be assessed following Prandtl (1920) and German Standards Organisation (2005). Fig. 9 shows the geostatic system and a hypothetic failure plane, which is the basis for the back-analysis. This system setup is simplistic, but reflects the observed failure geometries in the East Harbor with multiple ground failure events covering most of the backfill area with common diameters between 5 and 20 m (Fig. 5).

According to Prandtl (1920) and German Standards Organisation (2005), the critical undrained shear strength is

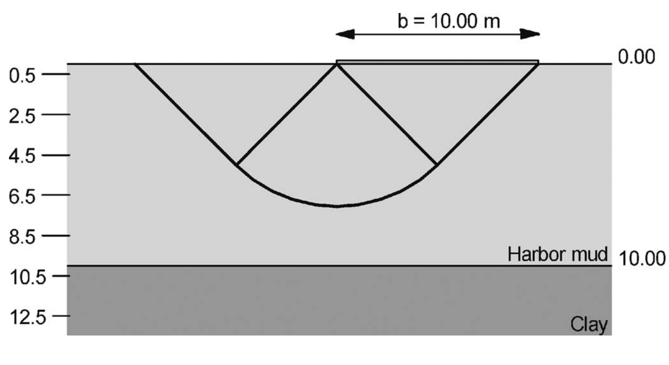

Fig. 9. Geostatic system and hypothetic failure plain for the simplified back-analysis according to Prandtl 1920 and DIN 1054

$$s_{u,\min} = \frac{p}{2+\pi} \tag{13}$$

where p=applied vertical stress.

To avoid ground failure under the assumed conditions, Eq. (13) indicates that a minimum undrained peak shear strength $s_{u,\min}$ of 0.39 kPa is necessary to avoid failure. Shortly after the observed surface deformations Metzen (2006) carried out laboratory measurements on East Harbor mud sediment cores obtained by piston coring. Fig. 10 shows the measured water contents and bulk densities as well as raw data of the undrained peak vane shear strength of one representative sediment core. The short time between redeposition of the East Harbor mud behind the sheet piling and the occurred failure of the backfill in connection with very unfavorable drainage and consolidation properties of the harbor mud (Schlue et al. 2009) did not allow for significant changes in water content, density or undrained shear strength with depth as a consequence of soil consolidation. Therefore, the effective stress within the relocated harbor mud is assumed to be constant and the soil body more or less homogeneous. This supports the applicability of the very simple, homogeneity assuming ground failure approach of Prandtl (1920) and German Standards Organisation (2005) for the presented back-analysis.

For the laboratory vane ($d/h=10/20$ mm/mm) used by Metzen (2006) and the used rate of rotation of 30°/min (German Standards Organisation 2002b) the determined uncorrected mean undrained peak vane shear strength was 0.75 kPa at that time (Fig. 10). The mean value is used because the failure planes had to pass through the whole harbor mud layer. In contrast to the μ of 0.63 derived by Bjerrum, the correction factor according to Eq. (10) is 0.47 for $\omega=30°/\text{min}$. Table 4 shows the uncorrected as well as corrected shear strength after Bjerrum (0.47 kPa) and after Eq. (10) proposed in this study (0.35 kPa). Furthermore, the calculated factor of safety for the geostatic system is listed assuming the different corrected shear strengths and $s_{u,\min}=0.39$ kPa. The correction according to Bjerrum predicts no ground failure and results in a factor of safety of 1.21. Applying the correction factor proposed by this study, the system cannot be verified, leading to a factor of safety of 0.90.

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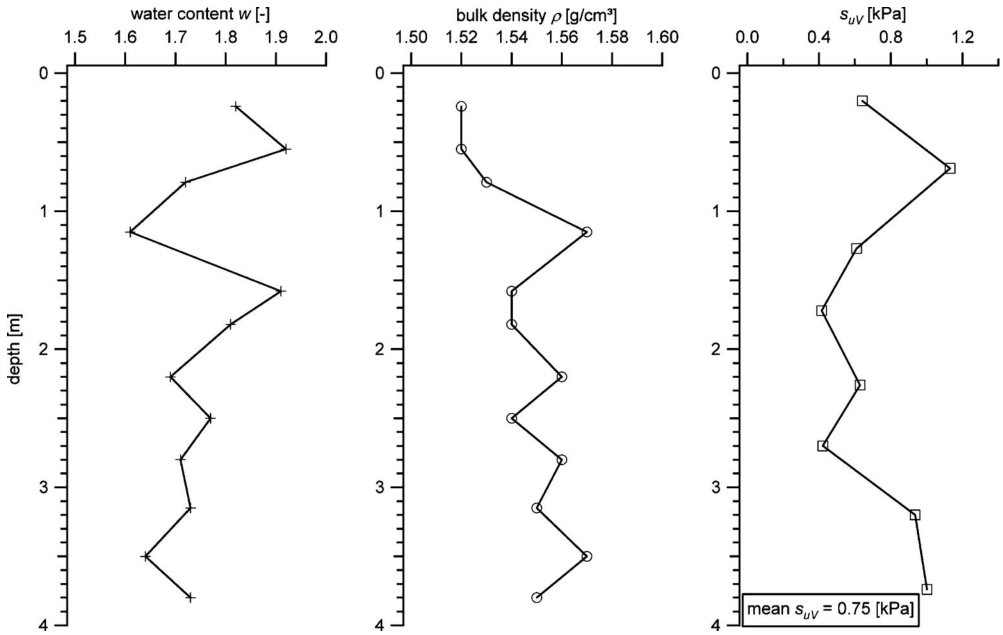

Fig. 10. Water contents, bulk densities, and vane shear strengths gathered from piston cores shortly after failure of the extension area (Metzen 2006)

This back-analysis indicates that the correction according to Bjerrum is not leading to a prediction of the observed failure during the East Harbor extension project. In the future, ground failures in similar liquid to soft harbor mud might be better predicted and avoided using the proposed shear rate dependent correction factors.

Conclusions

Extremely plastic, organogenic clay, with a liquid to soft consistency like harbor mud is an extremely difficult foundation soil and in addition to the known unfavorable properties, the low undrained shear strength of such a material is more rate dependent than previously known.

The shear rate has a significant influence on both undrained peak and residual shear strength of organic harbor mud as increasing shear strength can be measured with increasing shear rate. In this study, the measured undrained peak shear strength varied between 55 and 65% with peripheral velocities between 1.57 and 157 mm/min. The undrained peak shear strength of harbor mud increases on the order of 30% per 10-fold increase in peripheral velocity; a relationship nearly constant for all harbor mud consistencies tested. In contrast, the increase in residual shear strength varies between 120% at a soft consistency and 350% at a liquid consistency, resulting in a 50 to 170% increase in the measured residual shear strength per 10-fold increase in peripheral velocity.

The relationship between peripheral velocity in vane shear tests and measured shear strength of harbor mud can be described by both a semilogarithmic law and a power law. Since the semilogarithmic expression cannot be extrapolated to small peripheral velocities, the more robust power-law description was used to develop shear rate dependent correction factors. The derived correction factors are solely dependent on the ratio between the shear rate observed in large-scale field-failure processes and the shear rate actually used in the vane shear tests. The typical shear rate in the field for the following analyses was taken from the East Harbor project using the time from loading until the first strong deformations were observed in the field. Furthermore, it was assumed that the localized shear zone in harbor mud during laboratory vane tests and in field failures are comparable in terms of relative displacements. The effect of varying water contents on μ for peak shear strength is negligible. In contrast, a change in water content has a significant influence on μ for the residual shear strength.

The correction factor for rate effects proposed by Bjerrum is shown to overestimate the undrained shear strength of organic harbor mud. For ω between 6 and 90°/min as recommended by the common international standards British Standards Institution (1990), German Standards Organisation (2002b), and ASTM (1987) this overestimation is in the range of 11 to 85% dependent on the used vane geometry with vane diameters between 10 and 75 mm. This underlines the importance of the shear rate dependent correction method proposed in this study. Generally, the larger the used peripheral velocity in a vane shear test, the more important the correction of vane shear test results for shear rate effects and the smaller the necessary correction factor μ. Therefore, this correction becomes even more important when larger field vanes are used rather than miniature vanes.

We show that, among many other factors that might have led to the observed deformations during the East Harbor extension project, the shear rate dependent soil strength behavior plays an

Table 4. Corrected Undrained Shear Strength according to Bjerrum and Eq. (10) and Resulting Factors of Safety for the Assumed Geostatic System

Correction factor	s_u (kPa)	Factor of safety
No correction	0.75	1.92
According to Bjerrum	0.47	1.21
According to this study	0.35	**0.90**

Note: Boldface font represents the safety factor according to the study is below one.

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important role and needs to be addressed in future projects using similar backfill material and rapid loading. A simplified back-analysis indicates that the correction according Bjerrum does not predict the observed failure during the East Harbor project. In the future, ground failures in liquid or soft harbor mud might be better predicted and therefore avoided using the shear rate dependent correction factor proposed in this study.

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Notation

The following symbols are used in this paper:

C_{tot} = total carbon content;
C_v = coefficient of consolidation;
d = vane diameter;
e_0 = initial void ratio;
h = vane height;
I_p = index of plasticity;
k = water permeability;
LI = liquidity index;
p = applied vertical stress;
R^2 = coefficient of correlation;
S_{tot} = total sulfur content;
s_u = undrained peak shear strength;
s_{ur} = undrained residual shear strength;
s_{urV} = undrained residual vane shear strength;
s_{ur0} = reference undrained residual shear strength;
s_{uV} = undrained peak vane shear strength;
s_{u0} = reference undrained peak shear strength;
TOC = total organic carbon;
t = vane blade thickness;
t_{fF} = time to failure in the field;
t_{fV} = time to failure in vane shear test;
V_{Gl} = loss on ignition;
v_p = peripheral velocity;
v_{pF} = peripheral velocity corresponding to field shear rate;
v_{pV} = peripheral velocity in vane shear tests;
v_{p0} = reference peripheral velocity;
w = water content;
w_0 = initial water content;
w_L = liquid limit;
w_P = plastic limit;
x_f = displacement along shear plane at failure;
α = semilogarithmic law parameter;
β = power-law parameter;
μ = correction factor for shear rate effects;
ρ = bulk density;
ρ_d = dry density;
ρ_s = particle density; and
ω = rate of rotation in vane shear tests.

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