Some Geotechnical Properties of Carlingford Clay

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ABSTRACT: This paper describes some of the challenges and difficulties of drilling, sampling and *in situ* testing through heterogenous fill, estuarine deposits and clay in Greenore Port, Ireland. The paper focuses on the characterization of the geotechnical properties of the clay, which is a laminated intermediate plasticity silty clay indigenous to Carlingford Lough. Little construction experience and no published data is available for this material. Geotechnical properties are derived from factual ground investigation information which consisted of cable percussion boreholes, UT100 samples, in situ CPTU, SPT tests and laboratory testing. Correlations from both published case history data from around the world and widely used in local industry are employed. Their applicability to Carlingford Clay is evaluated and recommendations for further studies are provided.

Keywords: UT100 sampling; CPTU; dissipation tests; CIUC triaxial testing; soil strength and stiffness

1. Introduction

This paper presents and describes some of the challenges and difficulties encountered during ground investigation (GI) for a proposed development at Greenore Port, Ireland, specifically in relation to a laminated silty Clay (Carlingford Clay) deposit at depth. The geotechnical properties of the silty Clay deposit are presented, based on available factual GI information and cross-referenced with common industry correlations.
The proposed development consists of a new quay wall, dredging operations and a new quay apron, see Fig 1.
Only GI data made available to authors is used in this paper.

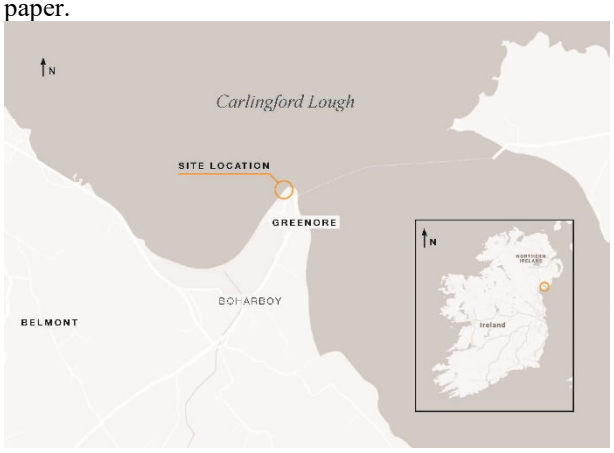

Figure 1. Site location plan.

2. Ground and groundwater conditions

2.1. Geology

The Quaternary geology in the Carlingford Lough area has been heavily influenced by glacial processes. The onset of glaciation in Ireland at the start of the Quaternary was approximately 2 million years ago, with as many as six glacial periods of glacial advance and retreat until about 14,000 years ago (ya) Baxter [1]. Carlingford Lough is a fjord (a long narrow deep inlet of the sea) which is formed when a glacial valley floods due to a rise in sea level. The majority of the sediments which cover the area formed during the Midlandian glaciation in Ireland. This occurred 19-13Kya. This represents the last advance and retreat of the ice sheet. The range of glacial sediment is highly variable and deposits of fine clays – boulders can be observed in the area. However, glaciation was not the only process which affected the area during the Quaternary period. As the ice melted there was an influx of water released into the oceans, resulting in rapid sea level rise. The sea level rose faster than the isostatic rebound of the landmass, as a result sea level at this time was much higher than present day sea level. Evidence for this can be seen in the area due to the presence of raised beach deposits. It is not clear if Carlingford Clay is a product of glacial melt out or of marine origin and further work in this area is merited.

2.2. Ground investigation

A number of historical GI's were made available to the authors at the outset of the project. These typically included cable percussion drilling techniques in the overburden soil and rotary coring techniques in the underlying bedrock. No *in situ* testing was carried out in the Carlingford Clay deposit under consideration as part of these historic GI's. Only classification information in the form of field descriptions of composition and consistency, bulk density measurements and Atterberg limit test results were available for preliminary design.

Project specific GI was undertaken under the technical direction of the authors, with a key objective to characterize the Carlingford Clay. The GI relied upon cable percussion (CP) boreholes and piezocone penetration testing (CPTU) to establish the site stratigraphy. Discrete soil sampling, laboratory testing (Triaxial CIUC and Oedometer tests) Standard Penetration Testing (SPT) and CPTU *in situ* testing were performed to access geotechnical properties.

CPTU were performed using a 20-ton truck mounted rig. The CPTU holes were pre-augered to depths of 3m below commencement level using a window sample rig to reduce the risk of encountering obstructions in the Made Ground. The electric CPTU cone (Type TE2) was employed and saturation of the cone was witnessed by Arup using glycerin fluid prior to use. Testing was performed in accordance with EN ISO 22476-1 [2].

Of the seven CPT holes only two were able to overcome the overlying Made Ground, Estuarine and Terrace deposits and penetrate into the Carlingford Clay stratum.

2.3. Stratigraphy

The geological profile on site is typically a downward sequence of Made Ground, Estuarine Deposits, Laminated silty Clay and Limestone. A summary of the range of levels and thicknesses of encountered stratigraphy is provided in Table 1.

Table 1. Summary of Ground Profile

Strata	Top of strata		Thickness (m)
	mOD	mbgl	
Made Ground	4.2 to 4.7	0	2.1 to 9.2
Estuarine and Terrace Deposits	2.3 to -5.0	2.1 to 9.2	11.5 to 14.0
Laminated silty Clay	-11.4 to -12.8	15.6 to 19.0	2.1 to 4.4
Limestone	-13.9 to -17.5	18.1 to 19.1	Unproven

The Made Ground is heterogenous in nature. Log descriptions vary from predominately coarse grained (hardcore, gravel) to fine grained (clay) with many man made objects noted (red brick and concrete). This material would most likely have been placed behind the existing quay wall initially during construction of the port in the late 1890's and during subsequent redevelopment over the intervening years.

The descriptions of the Estuarine and Terrace Deposit vary from grey brown silty slightly sandy Gravel to slightly gravelly Sand with occasional shells thin layers of organic clay, cobbles and boulders. The gravel is fine to coarse sub-rounded to sub-angular and the sand is fine to coarse. Due to the heterogeneous nature of the Made ground it is not possible to determine the interface between the Made Ground and the underlying natural soils.

The Carlingford Clay is situated beneath the Estuarine and Terrace deposits. Descriptions vary from firm to stiff greenish grey to grey to greyish brown slightly sandy silty Clay with laminations of fine sand and occasional gravel, cobbles, boulders and seams of grey silt and clayey silt.

2.4. Drilling disturbance

As part of the project specific GI cable percussion boreholes were sunk using 250mm diameter casing reducing to 200mm to achieve the target depths. To reduce the potential for 'blowing' conditions, due to differential water pressure between the bore and surrounding soil, the boreholes were topped up with water during advancement. In addition, measurements of casing depth relative to base of borehole was recorded during borehole advancement to ensure *in situ* testing was performed below the casing toe level. The addition of water to the boreholes did not take place overnight or between shifts.

The variable nature of the Made Ground combined with the tidal effects of the sea resulted in difficult drilling and slow progress. In some cases, it took over six days to reach the top of the Carlingford Clay (16.5 to 17m below ground level, bgl).

SPT's were performed at 1m intervals in the Made Ground / estuarine deposits. However, implausibly low values (N values <5) were recorded at depths of 10.5m bgl. Fig 2 shows a comparison of both measured SPT N results, split by historical GI results, project specific GI results and SPT N values correlated from measured CPTU data. The process followed to convert measured CPT results to SPT N values is defined by Kulhawy and Mayne [3] and is a function of the fines content of the material under consideration. Refer to [3] for details.

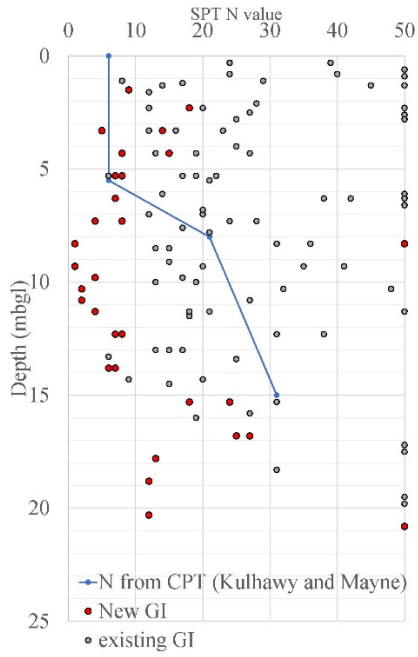

Figure 2. Comparison of CPT and SPT insitu testing

The historical GI results appear to match well with the correlated CPT results. The project specific GI results appear lower. This contrast is likely the result of significant soil disturbance at the base of the borehole resulting is a reduction in mean effective stress (p′) of the soil which was exasperated by the length of time taken to advance the boreholes and the effects of tidal groundwater flow. Previous movements of the existing dock wall are likely to account for some of the lower SPT N values close to this structure.

In some cases, double drives of SPT were performed but the resultant N values did not show significant increase in blow counts with depth.

It is difficult to prevent these issues without using a heavy drilling fluid [4] to prevent loosening of the material at the base of the borehole. The use of drilling fluid during borehole advancement is not commonplace within the Irish GI industry. These issues encountered during borehole advancement highlight the importance of having several different pre-planned investigations approaches. An example of this, was being able to use CPT results to demonstrate that the measured project specific SPT results were not representative of *in situ* conditions. Establishing this enabled justification of more representative design parameters in the upper coarse-grained soils.

2.5. Clay sampling

100mm diameter thin walled driven open tube samplers (UT100) were used to recover samples of the silty clay within the boreholes.

UT100 was developed in the UK, Gosling *et al*, [5], to meet the criteria for Class 1 sampling in accordance with EN ISO 22475-1 [1] and the 'UT' refers to undisturbed thin walled version of the original U100 which is unsuitable for taking undisturbed samples. The typical geometry of a UT1000 is as follows; taper angle of 5°, Area ratio (C_a) of 14.97% and inside clearance (C_i) of 0.19%

The cutting shoe was inspected on site by Arup prior to use and samples were waxed immediately upon retrieval from the borehole.

Good sample recovery of between 70 and 100%, of a total sample length of approximately 450mm, was achieved using the UT100 tubes.

A method of evaluating sample quality, Lunne *et al* [6], is by assessing the normalised change in void ratio ($\Delta e/e_o$) when consolidating the sample back to its estimated *in situ* effective stress in both triaxial and Oedometer tests. As this approach was developed for soft clays, volumetric strain (ε_{vol}) is also included. Soil suction prior to testing was not measured during laboratory testing.

The results are presented in Table 2 and show that poor to very poor sample quality was achieved. Fig 3 indicates possible shearing on the periphery of the sample tube. This shearing is most likely the result of driving the UT100 sample tube into the material.

Figure 3. UT100 sample of laminated clay at BH101 at 19.25m (following CIUC test)

Such shearing effects most likely equate to the soil being disturbed during sampling and is reflected in the ε_{vol} measurements reported in Table 2. In addition to these induced disturbance during sampling, the sample most likely softened (swelling due to stress relief) *in situ* prior to sampling due to slow progression of borehole prior to sampling. Due to the high level of ε_{vol} recorded, the Oedometer test results are not considered further.

Table 2. Assessment of sample quality

Sample number	Test	$\Delta e/e_o$	ε_{vol} (%)	Rating
102_U35_18.25m	CIUC	0.13	6	poor
101_UT3_19.25m	CIUC	0.10	4.9	poor
201_UT42_18.75m	CIUC	0.24	10.1	very poor
101_UT3_19.25m	Oed	0.20	-	very poor
101_UT1_17.25m	Oed	0.19	-	very poor
201_UT24_17.75m	Oed	0.22	-	very poor

2.6. Groundwater

Groundwater monitoring, via data loggers installed in solid 50mm diameter standpipes with targeted response zone, was carried out as part of the GI. Three locations were monitored, a tidal gauge attached to the front of the existing quay wall, a standpipe installation with a response zone in the Estuarine and Terrace deposits 5 m behind the existing quay wall and a standpipe installation with a response zone in the Estuarine and Terrace deposits 25m behind the existing quay wall. The results are displayed in Fig 4 and demonstrate the groundwater is tidal with the influence of the tidal range diminishing with distance from the quay wall.

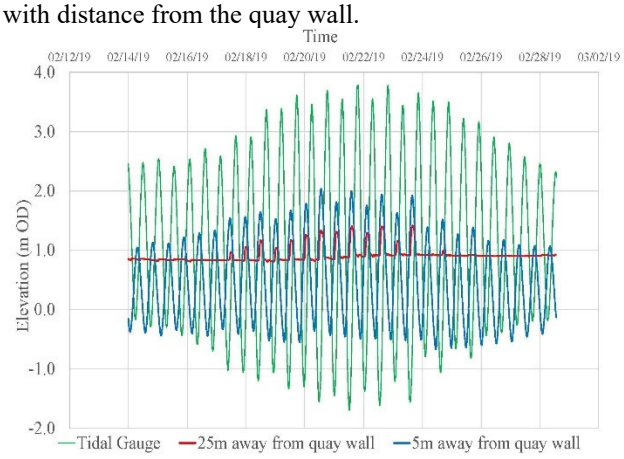

Figure 4. Groundwater data logger monitoring results

3. Geotechnical properties

3.1. Classification

The clay was described, in the borehole logs as stiff light grey slightly sandy silty CLAY with laminations of fine sand. Sand is fine to coarse. Three grading tests were undertaken on the material, Fig 5, display a high fines content between 89 and 99% however, the one sedimentation analysis (via hydrometer) displays a clay content of 16% which indicates the material is predominately Silt. A photograph of the sample post testing is shown in Fig 3.

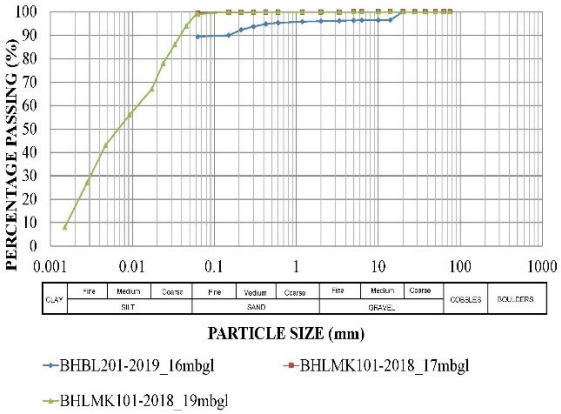

Figure 5. Particle size distribution curves

There is a possibility the sedimentation analysis results suffered from theoretical limitations of applying Stokes' law during the analysis.

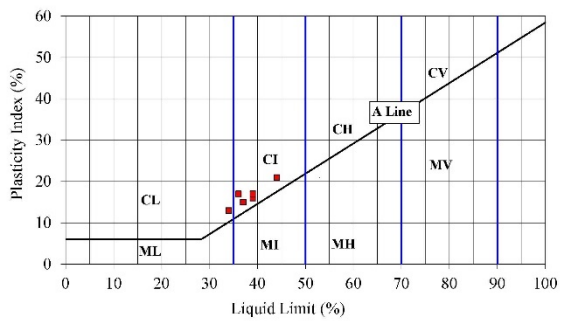

Figure 6. Plasticity chart

In contrast to the hydrometer analysis the Atterberg limit results plot above and trend parallel to the A line. The results indicate a clay of intermediate plasticity.

A summary of the available classification testing is presented in Table 3 and Fig 6.

Table 3. Summary of classification results

Parameter	Range	Characteristic value
γ	19.6 to 19.9 kN/m^3	19.75 kN/m^3
w	20 to 30 %	26%
w_L	34 to 44 %	37%
w_P	19 to 23%	22%
I_P	13 to 21 %	16%
I_L	0.05 to 0.65	

CPTU measurements are provided on Fig 8. Raw measurements, corrected cone resistance (q_t), pore pressure (u) and sleeve friction (f_s) are all presented against depth. In addition, two second order properties are plotted with depth, friction ratio (R_f) and pore pressure ratio (B_q).

CPTU tests allow prediction of soil behaviour type based on measurements taken during shearing.

A number of approaches are available to derives soil behaviour type. All methods are a function of q_t and contrasting these values against a function of either f_s or u. By inspection the classification based on u should be more accurate than classification based on f_s due to accuracy tolerance of pore pressure sensor relative to sleeve friction sensor. In the authors experience it is local industry practice to classify onshore materials using a function of f_s due to lack of repeatability of u readings due to de-saturation of pore pressure sensor during CPTU testing. Inspection of the measured u data indicates u measurements at this site did not suffer from de-saturation. Therefore, the approach by Schneider *et al* [7] is applied to derive soil behaviour type as the approach is based on functions of normalised cone resistance and normalised excess pore pressure measurements. While some of the material plots as transitional soils and silts and low I_r clays the majority of measurements plot as clay. Fig 7.

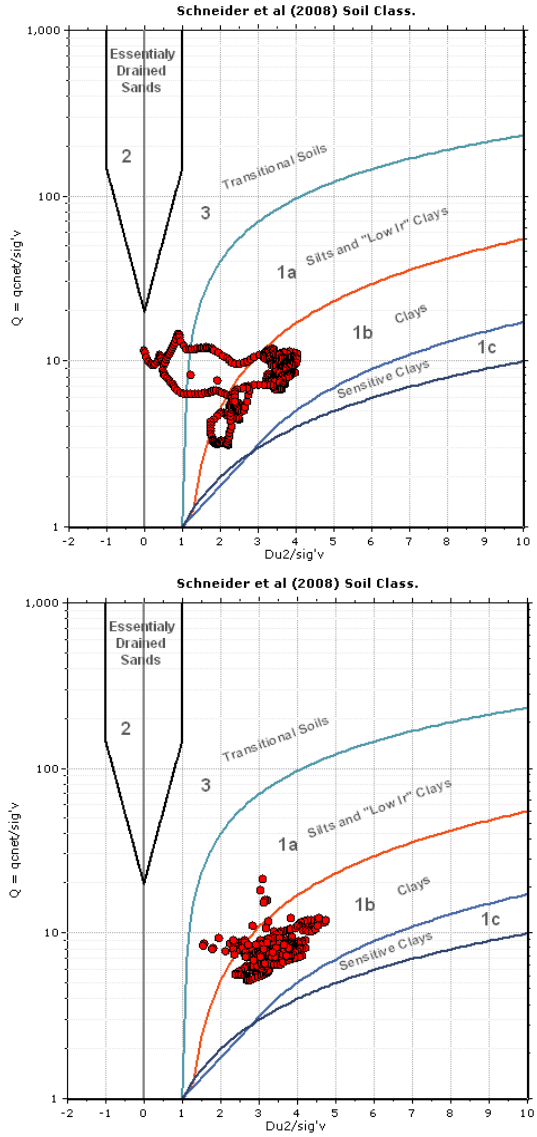

Figure 7. CPTU soil classification chart (Q_t vs. $\Delta u_2/\sigma_v'$)

In summary:

- The material plots above Casagrande A-Line, indicating a Clay
- Has a high fines content (>89%). Only one hydrometer was undertaken which indicates a clayey Silt.
- The classification of the soil using the CPTU data indicates clay type behaviour.
- The shearing behavior (Section 3.2) in triaxial compression is dominated by its Silt content

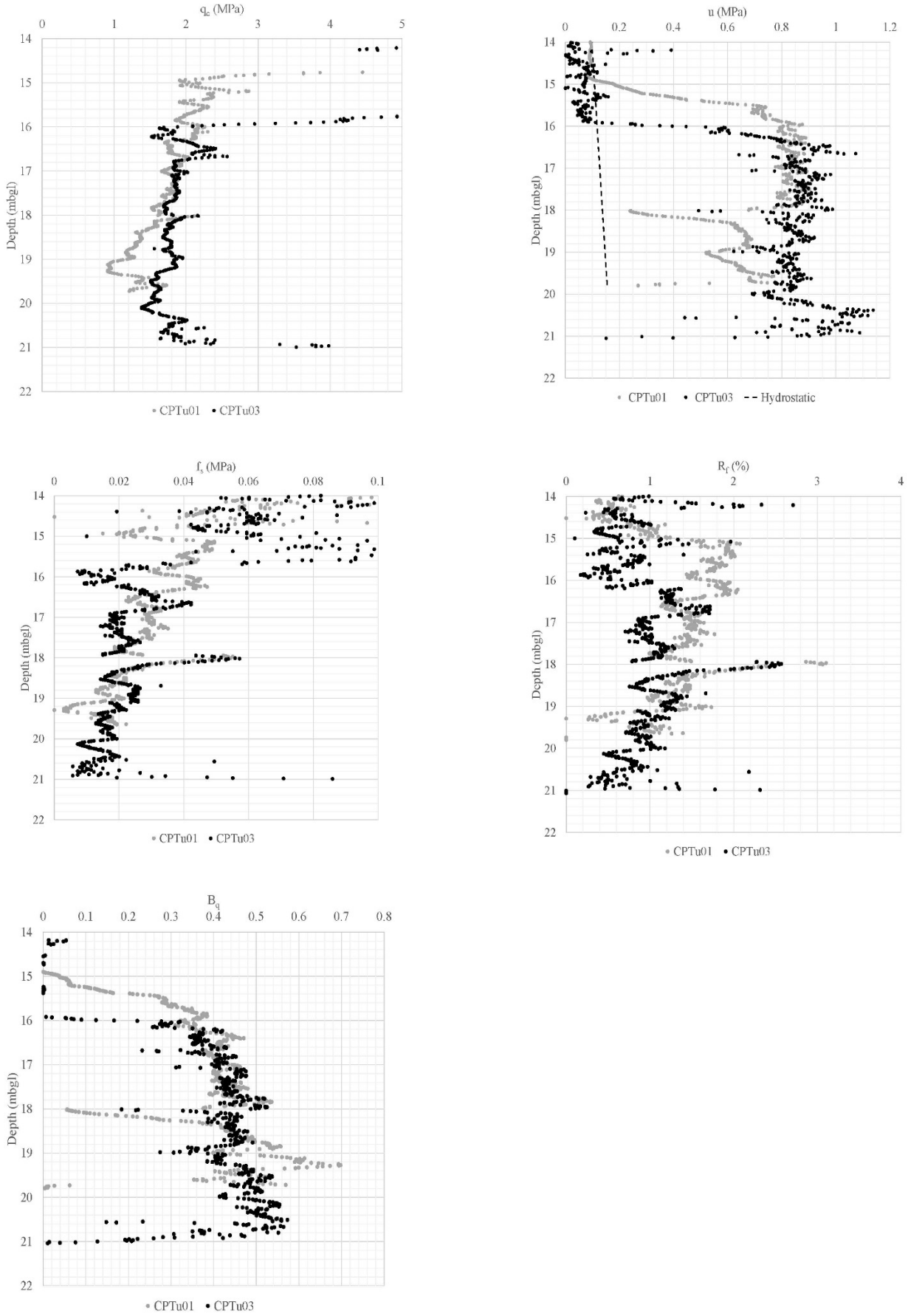

Figure 8. Cone penetration test results with depth in clay: a) corrected cone resistance, b) pore pressure, c) sleeve friction, d) Friction Ratio, e) Pore Pressure Ratio

3.2. Behavior during shearing

To appraise the material behavior during shearing both the CPTU and triaxial test results are examined. For the CPTU tests (see Fig 8) it is clear the material displays undrained behaviour during shearing as excess pore pressure develops upon encountering the material and maintains as an excess while the probe progresses through the material.

For the consolidated isotropically undrained compression (CIUC) triaxial tests the results are less clear cut. Triaxial tests were undertaken at depths of between 18 and 19m bgl on UT100 samples. Each sample was consolidated back to its estimated *in situ* stress and then sheared undrained. A back pressure of 300kPa was applied in each test.

Two of the samples tested (denoted by green and blue, Fig 9a, 9b, 9c) display initial contractive behaviour (ε_a<3%), followed by a phase transformation then followed by dilatant behaviour to large strains (ε_a>18%) and results in strain hardening upon shearing. This behavior is typical of a:

- low plasticity post glacial silt, Blaker et al [8];
- low plasticity silts, Georgiannou *et al* [9]; and
- low plasticity clay sheared from the normally consolidated line or as a result of cementation, Hight *et al* [10].

Similar behavior was also noted by the authors experience of CIUC testing of Dublin Port Clay, Arup [11]. Swelling effects may have been caused during borehole advancement or sampling or may have been due to cavitation and pore pressure equalisation within the sample within the time frame between sampling and testing.

The remaining sample (UT3-19.25m) displays behaviour more typical of a light to medium overconsolidated higher plasticity Clay, *i.e.,* less pronounced contractant behaviour prior to reaching a peak strength and displaying limited brittle post-peak behaviour. Test in BH101 (UT3-19.25m) was stopped at a axial strain of 5% compared with over 18% for the other tests. Fig 3 suggests a failure surface developed within the sample and therefore the sample could not sustain any further application of deviatoric stress. It is worth noting that this sample (BH101) displayed the lowest volumetric strain during consolidation, *i.e.,* $\Delta e/e_o \leq 0.1$ and thus the lowest sample disturbance recorded out of all samples tested.

Given the variation in behavior during shearing displayed during CPTU testing relative to triaxial testing it is concluded the coarser grained laminations are having a more pronounced effect during the triaxial shearing relative to CPTU progression. This is most likely due to scale effects, possibly the smearing effect of the insitu material by the progression of the CPTU cone tip and the effect this smearing has on the pore pressure sensor behind the cone tip. The coarser grained laminations appear to dominate the behaviour during shearing on two of the triaxial samples.

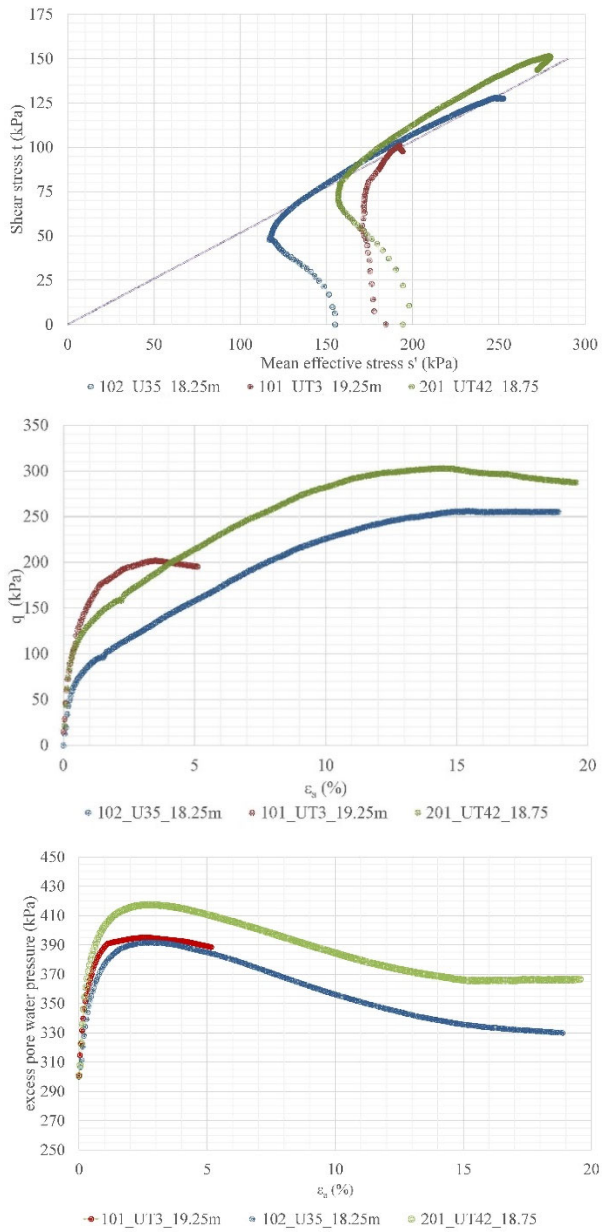

Figure 9. CIUC triaxial tests a) undrained effective stress paths b) deviator stress versus axial strain c) excess pore pressure versus axial strain

3.3. Undrained strength

The total stress or undrained shear strength (c_u) of the material is considered based on a number of different approaches, namely correlation with SPT and CPT results, and measured results from triaxial tests.

It is important to note that there is no single unique value of undrained shear strength of a clay and it depends on, stress path, strain rate, orientation of the stresses and volume of material under stress. As such, a design c_u value needs to consider the mode of failure that might be developed in the design scenario under consideration.

3.3.1. SPT

Six SPT tests were taken in the clay using an SPT hammer with energy ratio, E_R=64%. Uncorrected N values were measured between 12 and 27 blow counts with

a characteristic N value of 15 (See Fig 2 between 16 and 20 mbgl). A widely used approach in local industry is to correlate c_u from N by employing the Stroud [12] relationship

$$c_u = f_1 N. \quad (1)$$

The factor f_1 is a function of I_p and assuming a characteristic value of I_p = 16% gives an f_1 value of 6. Therefore c_u = 90kPa.

It is noted that the original work undertaken by Stroud predates the current standard I.S. EN 22476-3 [13] where rod size has increased. As such this correlation is likely to be conservative, White *et al* [14].

3.3.2. CPTU

The most common approach in the local industry is to correlate CPTU measurements to c_u by factoring the q_t values using the following relationship

$$N_{kt} = (q_t - \sigma_{v0}) / c_u. \quad (2)$$

N_{kt} is an empirical cone factor and typically a value of 15 is employed [15]. Using these values gives a range of results as displayed in Fig 10a. A characteristic c_u value of 100kPa is chosen from this plot.

In cases where the pore pressure sensor does not suffer from de-saturation during shearing c_u as a function of B_q may be a more reliable method of deriving c_u in the authors' opinion using the following relationship

$$c_u = \Delta u / N_{\Delta u}. \quad (3)$$

Where Δu is excess pore pressure and $N_{\Delta u}$ is pore pressure cone factor.

This is due to the fact that c_u is a function of measured excess pore pressure rather than a function of a measured resistance at the cone tip as measured by the q_t value. The range of values of $N_{\Delta u}$ applied are taken from the relationship with B_q (between 0.4 and 0.5, Fig 8e) defined by Karlsrud [16]. A range of $N_{\Delta u}$ of 5-8 is applied. This gives a range of c_u as defined in Fig 10b and 10c. The c_u values typically range from 85kPa to 130kPa, which gives an average of 105kPa.

3.3.3. CIUC triaxial

The shearing behavior described in section 3.2 results in no unique peak undrained shear strength. This makes the choice of failure criterion difficult.

Brandon et al [17] summaries a number of different failure criterion for defining c_u, (i) maximum deviator stress (ii) maximum stress ratio (σ_1'/σ_3'), (iii) pore pressure parameter $\bar{A}$ =0 or Δu=0, (iv) maximum pore water pressure and (v) limiting strain. They found, for low plasticity dilatant silts, that basing failure on pore pressure parameter $\bar{A}$ =0 was the most effective approach.

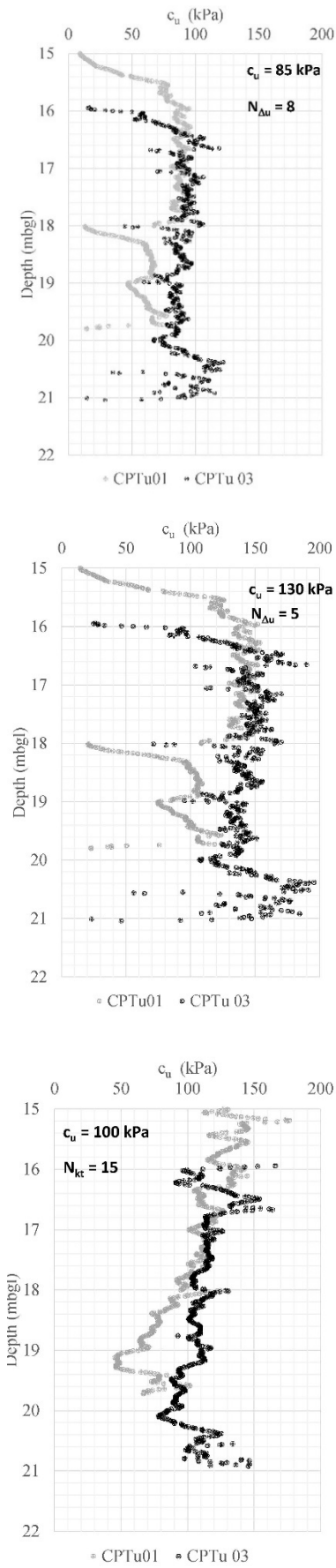

Figure 10. Undrained strength with depth: a) lower bound c_u from excess pore pressure, b) upper bound c_u from excess pore pressure, c) c_u from measured q_t values

The following is a review of the testing using each of these failure criterion: approach (i) and (ii) resulted in excessive strain (ε_a >7%) at failure. Criterion (iii) was not applicable as the pore water pressure remaining positive during testing (Fig. 9c). Undrained strength at maximum pore pressure was 62, 84 and 97kPa (U35, UT42, UT3) and all occurred at axial strains close to 2.5%. These are deemed conservative as the stress path where the point of maximum pore presssure has occurred had not reached failure. While limiting strain (ε_a) to 5%, resulted in c_u values of 82, 108 and 94kPa.

All samples were taken within 1mbgl of each other, and the variability may in part be as a result of sample disturbance. It is important that an appropriate design value of c_u for serviceability limit state is carefully considered.

3.4. Effective stress strength parameters

The CIUC triaxial tests provide the only direct measurements of effective stress strength available at time of writing. The behavior of the stress paths at the end of each test indicates, in some cases, a marked reduction in strength, which is likely the effects of the membrane on the area correction.

Plotting a characteristic line through the s′-t stress path data displayed on Fig 9a gives c'=0kPa and an ultimate angle of shearing resistance of φ′ = 31°.

3.5. Consolidation

Two dissipation tests were carried out as part of the CPTU testing. Both were carried out at a depth of 18mbgl. The results are displayed in Fig 11 where measured pore pressure (u_2) is plotted against time (logarithmic scale). An initial increase in pore pressure is noted which is indicative of a non standard dissipation curve, this is followed by a typical monotonic reduction of pore pressure with time.

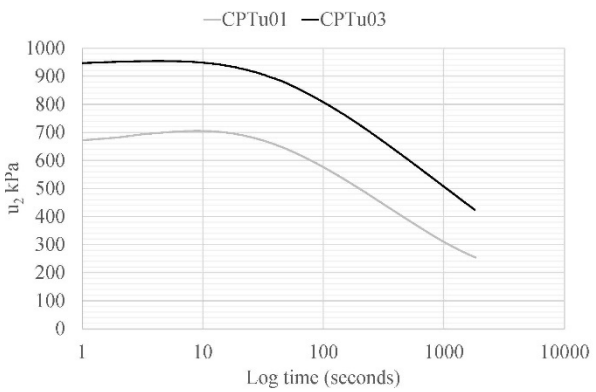

Figure 11. Pore pressure dissipation with time

The Teh and Houlsby [18] method is used to determine the coefficient of horizontal consolidation (c_h) with the square-root time curve correction applied to account for the non standard dissipation curve, Sully et al [19]. The corrected normalised pore pressure (U) is presented in Fig 12.

The rigidity index (I_r=G/c_u) is a key parameter for calculating c_h and is defined as the ratio of shear stiffness to undrained strength. An I_r of between 20 and 90 was estimated from the CIUC tests, with an average value of 57. The stiffness and strength were selected at 50% of the stress at failure, which is considered to represent an average response of soil to an advancing cone, Konrad & Law [20], Schnaid et al. [21]) and Krage et al [22]. As a result, the range of c_h values derived from CPTU tests is between 32 and 68 m^2/year. The results from the triaxial tests are significantly lower, with c_v between 0.2 and 1.6 m^2/year, where c_v typically reduces with stress. This is consistent with Long [4], however, these laboratory values may have been affected by poor sample quality.

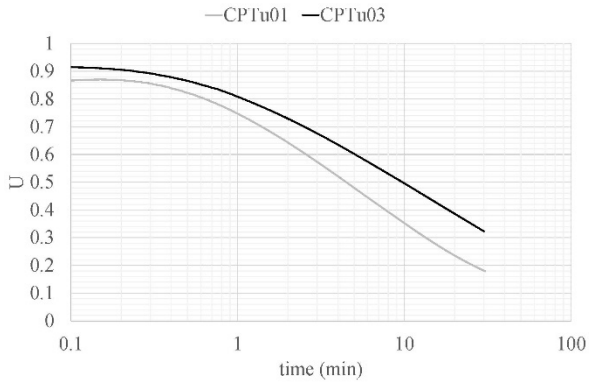

Figure 12. Normalized pore pressure dissipation with time.

3.6. Stiffness

Undrained stiffness ($E_{u\ secant}$) of the clay has been determined from the CIUC tests. Normalised stiffness characteristics are shown in Fig 13 and the results display a typical non linear response with increasing axial strain.

A characteristic E_u/p' = 1750 (kPa) for axial strain at 0.1% (typical for retaining walls applications) reducing to 650 for 1% strain is interpreted from Fig 13.

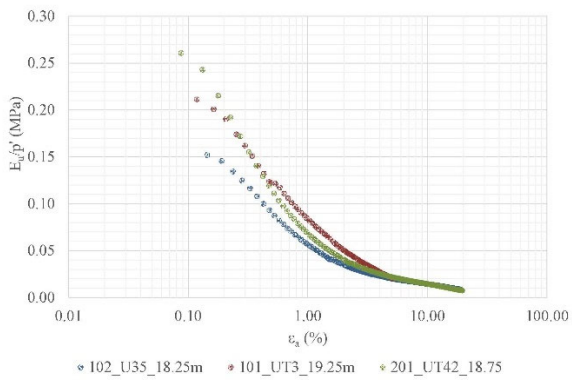

Figure 13. Nonlinear normalized undrained stiffness characteristics

4. Conclusions

The following is a summary of the key conclusions:

1. Ground investigations in port environments can be challenging due to the tidal groundwater and heterogeneous nature of material placed behind old quay walls. The CPTU has been used successfully and has proven invaluable over the more traditional SPT test in the upper coarse grained soils. This highlighting the importance of having a number of different ground investigation techniques available for soil characterization;

2. While UT100 sample tubes are designed to meet EN ISO 22475-1 Class 1 sampler geometry specification, they produced poor to very poor quality samples in stiff laminated clay. This poor quality return was most likely due to:
    a. Composition of material, presence of more granular laminations allowing pore pressure equalisation and cavitation occur within sample post sampling and prior to testing;
    b. Driven sampling effects; and
    c. Material softening *in situ* (swelling due to stress relief) due to time effects, i.e. slow progression of borehole prior to encountering Carlingford Clay.
3. An initial geotechnical characterization has been undertaken on the Carlingford Clay and the following is noted;
    a. It is a complex material with a matrix of finer grained particles with intermittent coarser sand and silt laminations.
    b. It is an intermediate plasticity Clay, I_p= 15% ±3%
    c. The undrained strength of this material lies between 65 and 110 kPa, equating to a stiff Clay.
    d. The ultimate angle of shearing resistance is in excess of $\varphi' = 31°$.
4. Suggested further research:
    a. Investigate clay sample disturbance and time effects using Geobore S techniques as an alternative to UT100 sample tubes and undertaking measurements of soil suction;
    b. Carry out additional triaxial testing on undisturbed samples immediately post sampling to better calibrate c_u against SPT tests and CPTU. Careful consideration of the type of sampling is important
    c. Shear box and further triaxial testing to calibrate φ'.
    d. An investigation of the geological original could include undertaken scanning electron microscope to understand its micro structure and X-ray defraction to understand the clay mineralogy.
    e. Investigate the effect of soil structure and fabric by undertaking reconstituted testing of the clay to establish intrinsic properties.

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