Title	Engineering characterisation of estuarine silts
Authors(s)	Long, Michael (Michael M.)
Publication date	2007-05
Publication information	Long, Michael (Michael M.). "Engineering Characterisation of Estuarine Silts" 40, no. 2 (May, 2007).
Publisher	Geological Society of London
Item record/more information	http://hdl.handle.net/10197/3093
Publisher's version (DOI)	10.1144/1470-9236/05-061

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Journal: Quarterly Journal of Engineering Geology and Hydrogeology

Title of paper: Engineering characterisation of estuarine silts

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Date paper originally written: December 2005

Date paper revised: February 2007

Key words: silt; compressibility, cone penetration tests, embankments, laboratory studies.

Title: Engineering characterisation of estuarine silts

by Michael Long, UCD.

Abstract:

Guidance is provided for geotechnical engineers designing civil engineering works in silty soils. A detailed characterisation of two estuarine silt sites in Ireland is performed and the soil properties are linked to their geological origin. It was found that these soils are susceptible to densification by conventional and high quality fixed piston tube sampling and care needs to be taken when using laboratory derived design parameters, particularly for consolidation and shear strength properties. One dimensional consolidation and creep of these silts can be modelled successfully by the well known Janbu formulation. Settlement predictions from laboratory derived parameters match reasonably with measured data but tend to underestimate primary consolidation, consistent with a sampling densification effect. Vane data should be used with caution as measured strength values may be high and it seems more reliable parameters can be derived from CPTU tests. Conventional techniques for determining soil strength from triaxial tests in silt are inappropriate due to the dilational nature of the material and more reliable and logical strength estimates can be made from a limiting strain criterion.

INTRODUCTION

Much geotechnical research has been conducted on clay and sand soils. Little information exists in the literature on the engineering properties of “intermediate” silty soils and how these are related to their geological origin. The reasons for this are probably related to the difficulties in sampling the material and subsequently preparing it for laboratory testing. As a result much of the work that has been carried out is on reconstituted material. Some examples of research on “undisturbed” material have been reported by Schultze and Horn (1965),

Börgesson (1981), Skúlason (1996), Høeg et al. (2000) and Sandven (2003) and these will be referred to in later sections.

In particular there is little guidance available for practicing engineers on how to choose design parameters, especially for problems related to 1D compression, creep and strength.

As a result of the recent economic boom in Ireland, significant funding is being invested in infrastructure development, particularly roads. Construction of new highways in coastal areas frequently involves crossing sites underlain by recent estuarine silts. For example silt deposits are regularly found in the coastal areas near the major cities of Dublin, Cork, Limerick and Waterford.

The purpose of the work described here was to characterize two typical Irish silt sites in detail, with a view to developing guidelines for practicing engineers working with these soils. A particular objective was to examine the influence of sampling disturbance on the measured soil properties. This was achieved by trying out a number of different soil samplers at each site. A separate focus was to examine the applicability of two well know in situ investigation techniques namely the vane test and cone penetration test to see if these formed a reliable alternative to sampling and laboratory testing.

THE SITES

Following a review of all available data (primarily from Farrell et al., 1996) two sites were chosen which spanned the range of silt types normally encountered in Ireland namely high plasticity organic silt and low plasticity inorganic silt. At both sites detailed information existed on the ground conditions and monitoring data from embankment construction was available. The location of the sites in Ireland is shown on Figure 1a.

The main site under consideration is located approximately 6.5 km south of Sligo along the Sligo / Collooney (N4) road (Figure 1b), in the north west of the Republic of Ireland. It lies approximately 1 km inland from Ballisodare bay, in an area known as Belladrehid, and

occupies is a low-lying stretch of flat land (1 mOD) between the higher elevations of Sligo and Ballisodare. Construction of a 15 km long highway, known as the Collooney Bypass, in the mid to late 1990's involved construction of embankments up to 8.5 m high and 65 m wide on peat and estuarine silts, up to 18 m in thickness.

The second site is located at Dunkettle, Co. Cork on the south coast of Ireland, see Figure 1c. Road embankments, up to 12.5 m height, were constructed on the site in the late 1980's and early 1990's. The site is located adjacent to the Jack Lynch tunnel and the N25 Cork – Waterford road. A full description of the site and the behaviour of the highway embankments are given by Flynn & Creed (1992).

BACKGROUND GEOLOGY

At the Sligo site the background geology comprises Carboniferous limestone laid down some 345 million years ago. Active karstic development occurred at times of lower sea level increasing the size of fractures in the rock. It is thought that the area was close to a glacier outlet to the sea and hence it was subjected to erosion and deposition of lodgement tills and fluvio-glacial deposits at the end of the last glaciation some 10,000 years ago (Nevill, 1969, Whittow, 1974, Mitchell, 1990, Mitchell and Ryan, 1997). These deposits are largely granular in nature.

Although there was no doubt some fluvial contribution, the soft deposits in the area are thought to be mainly of estuarine origin deposited in relatively shallow water. Occasionally, during periods when the land was dry, deposits of organic soils including peat formed in distinct lenses. The sequence at the site is completed by peat deposits which grew 3000 to 5000 years ago. In this material the vegetation that formed is still largely discernible.

The Dunkettle site is underlain by between 6 m and 10 m of estuarine silts. In some places the top 1 m to 2 m resulted from hydraulic filling, which took place some 6 years prior to the commencement of construction. The depositional environment at the site can be seen in the

photograph in Figure 2a. Little peat or organic soil was encountered. As occurred in Belladrehid, Dunkettle was also near a glacier outlet into the sea during the last glaciation, Reilly and Sleeman, 1977, Holland, 1981). Extensive, predominantly granular, fluvio-glacial deposits were deposited at the site and over the Cork area. Again the bedrock is Carboniferous limestone, shaley in parts, and has been subjected to karstic erosion.

DRILLING AND SAMPLING TECHNIQUES

During the original site investigation for the road construction at both sites “undisturbed” sampling was mostly by means of conventional open drive U100 tubes. Frequently the samples were lost during retrieval from the borehole. Efforts were subsequently made to retrieve samples by screwing two U100 tubes together, as shown on Figure 2, to retrieve “double U100” samples. (Note these sample tubes are sometimes referred to as U4 from the old imperial dimensions.) This technique is frequently used in Ireland in similar materials. Little was known at the time of the possible densification effects caused by drained (or at least partially drained) penetration of sampling tubes in sandy / silt material.

For this project it was decided to investigate these effects by sampling using several different techniques. In order to provide some baseline data samples were taken at both sites using the ELE 100 mm diameter fixed piston sampler. This is conventionally used in the UK and Ireland to obtain high quality samples of soft compressible material. At Sligo the tubes had the standard 30° cutting edge angle. At Dunkettle this was sharpened to 5°. In addition, again at both sites, sampling was carried out using the Norwegian Geotechnical Institute (NGI) 95 mm diameter sampler (Andresen, 1981) which was known to yield high quality samples of soft clays. Finally at Sligo samples were obtained using the “double U100” technique described above (Figure 2b) and at Dunkettle continuous samples were recovered using the MOSTAP[®] process (Weltman and Head, 1983, Long, 2002), see Figure 2a.

Except for the continuous samples, all of the samples were recovered by the same drilling crew from the base of a borehole, which was 200 mm in diameter. It was advanced using conventional shell and auger drilling and was maintained full of water. At Sligo the three sampling holes were located within 5 m of one another and the subsequent cone penetration tests (CPTU) tests. At Dunkettle the ELE and NGI samples were recovered adjacent to one another and some 1 km north of the MOSTAP[®] sampling.

A summary of the dimensions and features of samplers is given on Table 1. The CPTU tests were carried out according to ISSMGE (1999) using a standard 10 cm² cone.

LABORATORY TESTING

The principal means of studying the difference in behaviour of the material from the different samplers was by means of anisotropically consolidated undrained compression (CAUC) triaxial tests in which the specimens are anisotropically consolidated to the best estimate of the in situ stress. Maintained staged load (MSL) and constant rate of strain (CRS) oedometer tests were performed to study any sampling induced effects on compressibility parameters.

Triaxial tests

The procedures used were broadly those adopted as standard by the Norwegian Geotechnical Institute (NGI) as described by Berre (1982). Specimens of diameter 10.14 cm (as sampled) were trimmed to a height diameter ratio of about 1.8. Initially some isotropic consolidation was carried out at an effective cell pressure of $0.6 \sigma'_{h0}$ (in situ horizontal effective stress) before slowly applying the in situ stress. The coefficient of earth pressure at rest (K_0) was assumed to be equal to 0.6 from Brooker and Ireland (1965). The final consolidation stresses are kept constant until the rate of volumetric strain is less than 0.0001% per minute. Shearing was carried out at the slow rate of 4.5% per day. Corrections were applied for the restraining effects of the membrane and filter paper.

Oedometer tests

Both MSL and CRS oedometer tests were carried out. The purpose of the latter was to examine the effects of rate of loading on the material. In both cases specimens were extruded directly into 100 mm diameter lubricated oedometer rings. For the MSL tests, in order to attempt to accurately define the preconsolidation stress, “gentle” load increments to $0.25\sigma'_{v0}$ (the in situ vertical effective stress), $0.5\sigma'_{v0}$, $0.75\sigma'_{v0}$, and $1.0\sigma'_{v0}$ were initially used. Twenty-four hour maintained load stages were employed. For the CRS tests the rate of loading was between 0.5% and 5% per hour. Otherwise the procedures used were again broadly those adopted as standard by NGI (Sandbækken et al., 1986).

SITE CHARACTERISATION

Ground conditions - Sligo

Visual inspection of recovered samples confirms that three distinct soft strata were encountered as can be seen on Figure 3a. Thin layers of peat and a peaty silt mixture, about 2 m and 2.5 m thick respectively overlie about 9 m of sandy silt. The water table is located close to ground level.

The stratigraphy of the site is also confirmed by the results of two piezocone (CPTU) tests carried out adjacent to the sampling holes (Figure 4). The following measured and derived parameters were used to analyse the CPTU test results (Lunne et al., 1997):

q_t = cone resistance corrected for out of balance pore pressure effects

q_n = net cone resistance = $q_t - \sigma_{v0}$

f_s = measured sleeve friction

$$FR = \left(\frac{q_t}{f_s} \right) \times 100\%$$

u_0 = in situ or hydrostatic pore pressure with water table at 1.5 m

u_2 = pore pressure measured just behind conical tip

σ_{v0} = total overburden stress

$$B_q = \text{pore pressure parameter} = \frac{u_2 - u_0}{q_t - \sigma_{v0}}$$

$$N_m = \text{bearing capacity factor} = \frac{q_n}{\sigma_{v0}}$$

In the peat, above 2.0 m, the q_t and FR values are high indicating highly organic fibrous material. Below 2.0 m both these values decrease and u_2 values are lower or even negative indicating dilatancy in the more coarse organic material. In the sandy silt, below 4.5 m, q_t values are generally low increasing from about 0.4 MPa gradually to about 0.8 MPa at 10 m. These values are indicative of a soft or loose material. In this zone FR values are lower and u_2 are higher than the layers above, the latter being generally greater than the in situ pore pressure. The pore pressure parameter B_q profile clearly distinguishes the base of the more organic material and is generally positive (u_2 greater than u_0) between 4 m and 10 m. Below 10 m the u_2 profile plainly indicates the existence of sandy lenses with this layer with the pore pressure frequently dissipating rapidly back to the in situ value.

Ground conditions - Dunkettle

The silts found at Dunkettle are very similar in appearance to those found in Sligo both being grey to dark grey in colour and very soft to soft in consistency. At Dunkettle there are occasional bands of shelly debris and thin seams of fine sand. As can be seen from Figure 3b, a thin layer of peat and peaty silt overlies more homogenous silt material.

Basic material parameters - Sligo

Basic material properties for Sligo are shown on Fig. 3a. Data for the peat is scattered with moisture contents (w) up to 200% being measured with an average value of about 100%. Values are also scattered in the peat / silt but average at about 65%. In the sandy silt the average value is about 60%. These three layers have average bulk densities (ρ) of about 1.5

Mg/m³, 1.7 Mg/m³ and 1.65 Mg/m³ respectively. As would be expected from the depositional environment local organic zones occur in the sandy silt, for example at about 10 m, where higher moisture content and lower bulk densities are recorded.

Above 4.5 m in the particle size distribution data for the peat and peat /silt data are very scattered. However below 4.5 m, in the zone of the sampling comparative exercise, the material is relatively uniform with average clay, silt and sand content being 6%, 38% and 56% respectively. An average particle size distribution curve from this zone is also shown on Figure 5.

Organic content is highest in the peat being up to 38%. In the sandy silt values are variable and locally high but on average are about 6%.

It is not clear then whether the material should be classified as silt or sand. British Standard BS5930 (1999) contains no definitive guidance but says that such borderline material should be termed SILT /SAND unless other supporting laboratory tests (such as plasticity tests) are available. BS5930 (1999) suggests that the description of the material should be consistent with its engineering behaviour. In general, except in isolated sandy zones, it is possible to carry out a plasticity index test on the material. As can be seen from Figure 6, in the 4.5 m to 13.5 m zone the material has average liquid limit (w_L) and plasticity index (I_p) of about 68% and 17% respectively, with all data falling below the “A” line and within the classification “silt of high plasticity” on a standard plasticity chart. In the peat and peaty silt the average values are about 100% and 56% respectively.

Basic material parameters – Dunkettle

These are plotted on Fig. 3b and it can be seen that below a variable upper layer, data for the silt is relatively consistent and has w and ρ values similar to those measured at Sligo. Moisture content reduces from about 60% to 40% with depth and bulk density

correspondingly increases from about 1.6 Mg/m^3 to 1.8 Mg/m^3 . For the overlying peat and peaty silt average values are about 1.6 Mg/m^3 and 80% respectively.

In contrast with Sligo the silt content is much higher and the sand content is lower being on average 65% and 30% respectively. This can be seen on the grading curves plotted on Figure 5, with the Sligo average curve forming the “coarser” limit of all of the Dunkettle curves.

Plasticity data for Dunkettle, shown on Figure 6, fall below the “A” line as for Sligo but in this case the plasticity index is much lower and the material can be classified as “silt of low to intermediate plasticity”. Measured organic content for Dunkettle is small and generally less than 1%. Again this is contrast with the higher values measured at Sligo. These higher organic contents are, in part at least, the reason that the Sligo material displays high plasticity indices. Therefore care needs to be taken when using plasticity index in engineering correlations with estuarine silts as it may reflect the organic content and not the basic nature of the material.

Sampling induced densification

A comparison of average ρ and w values for each of the sampler types and from both sites is presented on Table 2. The objective here was to examine any sampling induced densification effects. Care was taken to choose values from the same depth range for each site. For Sligo values for the NGI 95 mm, double U100 and U100 (original site investigation) samplers are similar. The ELE 30° sampler gives ρ values on average 3% lower and w on average 7% higher than those of the other samplers. For Dunkettle the MOSTAP[®] data shows a clear difference from the others and ρ and w are on average 25% higher and 4% lower respectively.

Given the poor dimensions of the MOSTAP[®] sampler, in particular, these results are not unexpected and confirm the susceptibility of these materials to sampling induced densification.

Comparison of basic properties for both sites

It is likely that the principal difference between the two sites is that the tidal influence at Dunkettle, due to the greater proximity to the sea, is significantly more direct and energetic than that at Sligo. This probably accounts for its lack of organic content as well as the slightly greater density and lower moisture content.

In subsequent sections comparisons will be made between samples taken using different samplers. In any such exercise in a natural deposit, there is some concern that natural material variability may mask any sampling effects. In both cases the material is relatively uniform, in the pertinent zone, on a macroscopic scale, as evidenced by the CPTU tests. Nonetheless, in order to minimise any effects of natural material variability, laboratory test specimens were chosen to be as large as possible (e.g. 100 mm diameter for triaxial testing).

Behaviour in 1D compression – general note

In current practice in Ireland, the UK and elsewhere use is generally made of Terzaghi's theory of 1D consolidation. This involves obtaining primary consolidation properties from plots of oedometer log effective stress (σ'_v) versus strain (ϵ) or void ratio (e) and creep properties from plots of ϵ or e against log time. Whereas this theory may work well for uniform soft clays, in the author's experience it is difficult to apply to silty materials. This is due to the non linearity of the resulting curves and to the artificial separation of primary consolidation and creep effects.

Although conventional curves will also be presented here, use is made in general of Janbu's (1985) theory for primary and secondary settlements, in which the stress induced primary consolidation is calculated with an effective stress dependent tangent modulus, and the time dependent secondary consolidation is determined using the "time – resistance" concept. In this theory it is not necessary to separate primary and secondary consolidation

phases because in practice, especially with organic material, creep takes place in all parts of the process.

Behaviour in 1D compression – Sligo

A comparison of MSL oedometer test $\log \sigma'_v$ versus ε and σ'_v versus constrained modulus ($M = \Delta\sigma_v/\Delta\varepsilon$) curves, for the sandy silt at about 6 m are shown on Figure 7. The classical log stress versus strain curves are of rounded nature and it is impossible to determine the yield or preconsolidation stress. In particular the curve from the NGI 95 mm sampler is particularly flat indicating a higher stiffness than the others and suggesting that sampling has densified the material. This is consistent with observations of the driller who had difficulty handling the heavy sampler, which was formed of stainless steel rather than aluminium for the others.

According to Janbu (1985) silt or sand material will show a gradual increasing M with increasing σ'_v as the particles are compressed tighter together. He suggested the material could be characterised by a power function as follows:

$$M = mp_a \left(\frac{\sigma'_v}{p_a} \right)^{1-a}$$

where:

m = modulus number

p_a = reference stress = 100 kPa

a = exponent = 0.25 for silt

As can be seen from Figure 7 this model can be used successfully to characterise the behaviour of the Sligo silt. For the case shown at a depth of about 6 m, m is approximately equal to 16. Janbu's model has also been successfully applied elsewhere, for example to Icelandic silts by Skúlason (1996).

Results of CRS tests for double U100 samples from 12.75 m are shown on Figure 8, together with a MSL test for the same sampler and depth. Material behaviour is identical to

that of the MSL tests and again can be modelled successfully by the Janbu approach with m equal to 12. It appears that the effect of rate of loading, within the studied range, is negligible.

Behaviour in 1D compression – Dunkettle

A similar plot for the Dunkettle MSL tests is given on Figure 9. Material behaviour is very similar to that at Sligo and can be modelled using the Janbu approach with m equal to 25.

1D Compression parameters

Janbu (1985) found that there was a strong relationship between initial water content and modulus number for Norwegian marine clays. Janbu's relationship, for water content in the range 30% to 70%, is shown on Figure 10. Data for the two silt sites, together with some from a third site at Little Island, are also shown. Little Island is located some 2 km east of Dunkettle and ground conditions and depositional environment are essentially the same at both locations. It can be seen that a similar relationship to that found by Janbu exists with m decreasing (i.e. material more compressible) as water content increases. It is interesting to note that the estuarine silts show a similar m to the Norwegian marine clays. Janbu found that Norwegian inorganic silts had m approximately one order of magnitude higher than for clays. The lower values for the Irish material can be attributed to the organic content and to the loose state due to the energetic depositional environment.

From the oedometer testing M_0 (i.e. M at in situ vertical effective stress σ'_{v0}) for Sligo typically increases from about 1 MPa to 2 MPa with depth. M_0 can also be determined from CPTU data from the equation (Lunne et al., 1997a):

$$M_0 = \alpha_i q_{net}$$

Senneset et al. (1988) and Sandven (2003) suggest α_i is approximately equal to 2 for silty soils; yielding M_0 , between 0.8 MPa and 1.6 MPa, i.e. slightly lower than the oedometer moduli, again consistent with some sampling induced densification.

Values of Janbu's (1985) creep number r_s for the Irish silts vary between 250 and 350 with no clear relationship with stress. These are at the upper bound of those suggested by Janbu for clay with water content in the range 55% to 70%. It can be shown that this parameter is related to the more commonly used creep coefficient, C_{sec} , by the formula:

$$r_s \approx \frac{2.3}{C_{sec}}$$

From Figs. 7, 9 and 10 it can be seen that there is no strong relationship between the material behaviour and the sampler type.

BEHAVIOUR IN TRIAXIAL TESTS

Stress – strain behaviour

Some results of CAUC triaxial tests for Sligo and Dunkettle are given on Figures 11a and 11b respectively. In each case pairs of test results are presented, i.e. tests from two different samplers at the same depth. Data is presented in the form of deviator stress ($\sigma'_a - \sigma'_r$) versus axial strain (ϵ_a) and pore pressure (u) versus ϵ_a and as a mean stress / shear stress (s' , t') stress path plot [$s' = (\sigma'_a + \sigma'_r) / 2$ and $t' = (\sigma'_a - \sigma'_r) / 2$]. From the plots it can be noted:

- Except, perhaps, for the Sligo deviator stress versus ϵ_a plot, all of the results are very consistent.
- All tests show dilatant behaviour with deviator stress increasing with increasing ϵ_a .
- Up to ϵ_a of about 2%, pore water pressure increases but then decreases (dilatancy).

This dilatancy is particularly marked for the shallow Sligo samples.

- The stress paths show some small initial contraction but then dilate strongly and form a clear failure line.
- There is no clear difference between the results for the different sampler types.

Effective stress strength parameters

Effective stress strength parameters (ϕ' , c') are required for long term stability analyses. Current practice is to obtain these parameters from triaxial testing. Usually a generous safety factor is applied. As can be seen from Figures 11a and 11b both the Sligo and Dunkettle material show friction angle, ϕ' , values in excess of 40° . It may also be argued that both materials also possess some cohesion, c' , of the order of 2 kPa to 5 kPa. Although these ϕ' values appear high, others have reported similar results, e.g. Schultze and Horn (1965) for German silt ($\phi' = 36^\circ$), Börgesson (1981) for silt from northern Sweden (ϕ' up to 40°), Skúlason (1996) for Icelandic silt ($\phi' = 40^\circ$ and greater) and Høeg et al. (2000) who found ϕ' of about 37° also for Swedish silt. Nonetheless all of these values are much higher than would normally be expected for loose silty material and there must be some suspicion of sampling induced densification in each case.

Friction angle values can also be estimated from CPTU data (Senneset et al., 1988 reproduced in Lunne et al., 1997) by comparing the bearing capacity number (N_m) with pore pressure parameter B_q . Taking the chart corresponding to lightly overconsolidated silts ($\beta = 0^\circ$):

- At around 4 m, N_m is typically 7.5, $B_q = 0.1$, $\tan \phi' = 0.48$, $\phi' = 26^\circ$
- Below 4 m and above 10 m, N_m is typically 5, $B_q = 0.2$, $\tan \phi' = 0.45$, $\phi' = 24^\circ$

Although these values are somewhat low, they are much closer to those normally expected for loose silty material.

It should be noted that the triaxial results correspond to large strains. In practice, in order to provide sufficient safety factor and to minimise strains, a safety factor typically 1.3 on $\tan \phi'$ is applied. In this case this would result in a design value of about 35° , which corresponds

to strains of 0.5% to 1.0%, which are perhaps reasonable, if near the high side, of allowable working values.

It is concluded that sampling, by all the techniques used, has densified the material resulting in measured effective shear strength values larger than would be encountered in situ. These results support the necessity to apply a generous safety factor when using triaxial effective strength parameters in design.

Undrained shear strength (s_u)

Several researchers (e.g. Senneset et al., 1982 and Sandven, 2003) have noted that use of s_u values are inappropriate for soils where $B_q < 0.4$, i.e. for material coarser than clayey silt. As an alternative they suggest using an effective stress approach. Although B_q values are indeed low here, some discussion on s_u is necessary as this parameter is used frequently by practicing engineers both directly and in correlations.

Undrained shear strength data for both Sligo and Dunkettle is presented on Figure 12. Data from the original site investigation at both sites as well as triaxial test results carried out as part of this study are presented. For Sligo it can be seen that the in situ vane data is very scattered probably reflecting the influence of the organic material and the partially drained shearing process. Data from laboratory vane tests are less scattered and are on average close to the $0.3\sigma'_{v0}$ line (roughly representing the shear strength of a normally consolidated material). For Dunkettle the in situ vane data are less scattered due to the lower organic content and both these and data from unconsolidated undrained triaxial tests (UU) are on average close to the $0.3\sigma'_{v0}$ line.

For both sites CAUC s_u values are very high. These were interpreted using the conventional approach for clays, i.e. simple peak value. It is not clear how to interpret s_u from triaxial tests on silt which exhibits dilatant behaviour. As can be seen it is clearly not appropriate to adopt the same technique as for clays where the simple peak value is taken.

Here the applied shear stress can increase constantly with increasing strain. Some possibilities for the determination of s_u for silt are as follows:

1. Simple peak deviator stress regardless of strain (conventional approach)
2. Shear stress at some limiting strain
3. Pore pressure parameter $A = 0$ or $\Delta u = 0$
4. Reaching Mohr - Coulomb line
5. Peak principle stress ratio (σ'_1/σ'_3)
6. Peak pore pressure

There is little guidance in the literature as to which criterion is most appropriate.

Börgesson (1981) used criterion 2 with a limiting strain of 10%. Stark et al. (1992) used both criteria 1 and 6. In this case as can be seen from Fig 11a and 11b, criterion 3 does not apply.

For the purposes of this study criteria 1, 2 4 and 6 are used. For criterion 2 the strength will be taken at 2% strain. Taking the data from Sligo, the various results are presented on Figure 13.

Lines representing $0.3\sigma'_{v0}$ and an estimation from the CPTU tests are also shown for guidance. For the CPTU the following formula was used (Lunne et al., 1997) and N_{kt} was taken to be equal to 18, corresponding to B_q of 0.1 to 0.2.

$$s_u = \frac{q_{net}(= q_t - \sigma'_{v0})}{N_{kt}}$$

Criteria 2, 4 and 6 give similar results and are significantly lower than those from Criterion 1, with perhaps the result from Criterion 4 (reach Mohr – Coulomb line) being somewhat conservative. It is not surprising that Criteria 2 and 6 give almost identical results as on average peak pore pressure occurred at about 2.4% strain. From the point of view of engineering practice use of these 2 criteria is logical as it will result in strains limited to 2% and ensure no dilation (i.e. negative pore pressures) is permitted.

Data from the CPTU are also very encouraging especially in the sandy silt material where there is little influence for fibrous organic material.

FIELD BEHAVIOUR

Measured performance

It is possible to assess the reliability of the parameters obtained from laboratory testing by studying the in situ behaviour of the material beneath the highway embankments. Data is available from several locations at the Sligo site (Ruiz, 2003) and a typical example from C/S 405 m is given on Figure 14. On Figure 14a details of the embankment construction, ground conditions and instrumentation (namely settlement plates, piezometers, magnet extensometers and inclinometers) are given. Settlement data is given on Figure 14b. Vertical drains were installed at 1 m to 1.5 m centres resulting in very rapid dissipation of excess pore pressure. Less than 5 days after each filling stage all of the excess pore pressure due to the filling had dissipated and subsequently pore pressure varied only with tidal conditions.

It can be seen that, despite there being only about 7.5 m of compressible material, significant ground settlement of about 0.65 m, on average, occurred during the monitoring period of about 300 days. Of this 0.4 m occurred in the upper peat layer. It can also be seen from the slope of all the plots that creep settlement was significant and showed no signs of diminishing during the monitoring period.

Prediction from laboratory tests

Some results from 1D compression calculations for Sligo C/S 405 m are also shown on Fig. 14b. Calculations were performed using the computer program KRYKON (Svanø and Emdal, 1987, Svanø et al., 1991). KRYKON uses the Janbu (1985) 1D consolidation theory as discussed above. Some details of the input parameters are given on Table 3. No attempt was made to refine the input parameters in order to provide a good fit with the monitoring data. The results are relatively encouraging. In particular the creep component of the

settlement seems to be modelled very accurately given the similar slope of the measured and predicted lines. The computer predictions tend to underestimate primary consolidation at low stress (i.e. value of constrained modulus, M , used too high) and overestimate primary compression at higher stresses (i.e. value of modulus number, m , used too low). Both of these findings are consistent with sampling induced densification.

SUMMARY AND CONCLUSIONS

The main objective of this work was to provide guidance for geotechnical engineers designing civil engineering works in silty soils. It was achieved by detailed characterisation of two sites. Some findings are as follows:

1. The two sites investigated span the range of silty material normally encountered in Ireland.
2. Results of simple p and w measurements on specimens from the different sampler types confirm the susceptibility of these materials to sampling induced densification.
3. The well known Janbu model for 1D consolidation and creep can be used successfully to characterise the behaviour of the Sligo and Dunkettle silts. Specifically the value of a in the formula $M = mp_a \left(\frac{\sigma_v}{p_a} \right)^{1-a}$ was found to be 0.25 and m can be found reliably by correlation with water content.
4. Settlement predictions from laboratory derived parameters match reasonably with measured settlements but tend to underestimate primary compression at low stress. This is due to the sampling densification.
5. In silty soils in situ vane data should be used with caution as measured strength values may be high due to the partially drained shearing process and to reinforcement effect of fibres.

6. Useful parameters can be derived from CPTU tests in silty, particularly constrained modulus and undrained and effective stress shear strength. CPTU profiling is particularly useful for determining soil stratigraphy.
7. Triaxial test results yield high values of effective friction angle, again probably due to densification effects. Generous safety factors, higher than those used for clay, should be applied in design.
8. Similarly interpretation of triaxial tests for undrained shear strength (s_u) using the traditional approach gives unrealistically high values due to the dilatant nature of the material. A more reliable and logical approach is to take s_u at a limiting strain of about 2% or at peak pore pressure.
9. Future investigation is silty soils should use a combination of field measurements (particularly CPTU) and sampling using relatively a relatively large diameter sampler (ideally about 75 mm) with a sharp cutting edge angle.

ACKNOWLEDGEMENTS

The author is grateful to former MSc student at UCD, Chisco Ruiz, and to Senior Technician George Cosgrave for help with the laboratory tests. Tom Lunne of NGI provided assistance with regard to the NGI 95 mm sampler and Dr. Sean Raftery, formerly of McCarthy and Partners provided the Sligo monitoring data.

Table 1. Summary of dimensions and features of samplers.

Sampler	Length cm	D_e cm	D_w cm	D_s cm	Area ratio %	Cutting edge angle degrees	Inside clearance %
ELE 100	100	10.14	10.48	10.14	6.8	30 (Sligo) 5 (Dunkettle)	0
NGI 95	100	9.63	10.16	9.66	11	9	0.3
Double U100	90	10.14	11.44	10.14	27	60 and 15 (Figure 2b)	0
MOSTAP [®]	200	6.5	9.3	6.5	105	15	0

Notes: D_e = inside diameter of cutting shoe, D_w = outside diameter of cutting shoe, D_s = inside diameter of sampling tube, area ratio = $(D_w^2 - D_e^2)/D_e^2$, inside clearance = $(D_s - D_e)/D_e$.

Table 2. Study of sampling induced densification effects

	ELE 30°	ELE 5°	NGI 95	Double U100	U100	MOSTAP®
Sligo ρ (Mg/m ³)	1.627	n/a	1.695	1.683	1.648*	n/a
w (%)	63.1	n/a	58.8	59.5	56.9*	n/a
Dunkettle ρ (Mg/m ³)	50.6*	48.9	47.5	n/a	49.1*	36.9
w (%)	1.748*	1.772	1.777	n/a	1.735*	1.823

* Not UCD but is from original site investigation

Table 3. Summary of input parameters for 1D consolidation analysis

Material	ρ (Mg/m ³)	p'_c (kPa) ¹	M at p'_c	m	σ_{ref} (kPa) ²	c_v (m ² /yr) ³	r_s
Peat	1.5	$\sigma'_{v0}+5$	0.26 – 0.37	10	-25	200	350
Peat / silt	1.7	$\sigma'_{v0}+5$	0.56 – 0.66	14	-25	250	300
Sandy silt	1.65	$\sigma'_{v0}+5$	0.75 – 1.02	16 - 12	-25	350	250

Notes:

1. Preconsolidation stress. Water table assumed to be at 1 m.
2. Ordinate where back projection of M – σ' line crosses the x-axis
3. Coefficient of consolidation

LIST OF FIGURES

Fig. No.	Caption	File Name*
1	Site location (a) in Ireland (b)Sligo detail, (c) Dunkettle, Co. Cork	DELL/papers/QJEGH/Sligo/Irelandmap.jpg
2	(a) Mostap sampling at Dunkettle (b) “Double” U100 tubes	
3	Basic material properties (a) Sligo (b) Dunkettle	D(AH)/Sligo/Sligobasics.grf+sligotriaxsu1.xls D(E)/Dunkettle/Dunkettlebasics.grf+.xls
4	Sligo CPTU	D(AH)/Sligo/CPTU/cptu.xls+.grf
5	Particle size distribution	D(AH)/Sligo/Gradings.grf+Allpsd.xls+D(AH)/Dunkettle/Gradings.xls
6	Plasticity chart	D(AH)/Dunkettle/DunkettleAtterberg.grf+.xls+D(AH)Sligo/Atterberg.xls
7	1D compression behaviour - Sligo	D(AH)/Sligo/Sligooedcomp.grf+.xls
8	CRS test results for Sligo	D(AH)/Sligo/Sligoocrscomp.grf+Sligooedcomp.xls
9	1D compression behaviour - Dunkettle	D(AH)/Dunkettle/Dunkettleoedcomp.grf+.xls
10	Relationship between m and water content	D(AH)/Sligo/mandwatercontent.grf+oedsum1.xls
11	CAUC triaxial tests results (a) Sligo, (b) Dunkettle	D(AH)/Sligo/CAUCcomparison.xls+.grf D(AH)/Dunkettle/AlICAUC.xls+.grf
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14a	Details of Sligo C/S 405	D(AH)/Sligo/section405dimensions.grf+.xls
14b	Results of monitoring at Sligo C/S 405	D(AK)/Sligo/Settlement405m.grf+.xls

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Fig. 1. Site locations (a) Location of sites in Ireland, (b) Sligo detail (c) Dunkettle, Co. Cork

Fig. 2. (a) MOSTAP[®] sampling at Dunkettle (b) “double U4” sampling tubes (Standard UK / Ireland U100 or U4 sampling tubes. Crude cutting shoe shown on right hand tube. For sampling silts two tubes screwed together using connector as shown)

Fig. 3a. Basic material properties – Sligo

Fig. 3b. Basic material properties – Dunkettle

Fig. 4. CPTU - Sligo

Fig 5. Particle size distribution curves

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