Engineering characterization of estuarine silts

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Abstract

Guidance is provided for geotechnical engineers designing civil engineering works in silty soils. A detailed characterization 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. The 1D consolidation and creep of these silts can be modelled successfully by the well-known Janbu formulation. Settlement predictions from laboratory-derived parameters match measured data reasonably well, 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 that more reliable parameters can be derived from cone penetration tests. Conventional techniques for determining soil strength from triaxial tests in silt are inappropriate because of the dilatational nature of the material, and more reliable and logical strength estimates can be made from a limiting strain criterion.

The purpose of the work described here was to characterize two typical Irish silt sites in detail, with a view to developing guidelines for practising 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-known in situ investigation techniques, 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 that 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 were available. The location of the sites in Ireland is shown in Figure 1a.

The main site under consideration is located c. 6.5 km south of Sligo along the Sligo–Collooney (N4) road (Fig. 1b), in the NW of the Republic of Ireland. It lies c. 1 km inland from Ballisodare bay, in an area known as Belladrehid, and occupies a low-lying stretch of flat land (1 m OD) 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 1990s, 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 Fig. 1c). Road embankments, up to 12.5 m height, were constructed on the site in the late 1980s and early 1990s. 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 has been given by Flynn & Creed (1992).

Background geology

At the Sligo site the background geology comprises Carboniferous limestone laid down some 345 Ma ago. Active karstic development that occurred at times of lower sea level has increased 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 ka ago (Nevill 1969; Whittow 1974; Mitchell 1990; Mitchell & 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 that grew 3000–5000 years ago. In
this material the vegetation that formed is still largely
discernible.

The Dunkettle site is underlain by between 6 and 10 m
of estuarine silts. In some places the top 1–2 m resulted
from hydraulic filling, which took place some 6 years
prior to the commencement of construction. The depo-
sitional 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
near a glacier outlet into the sea during the last glacia-
tion (Reilly & 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, shaly in parts, and
has been subjected to karstic erosion.

**Drilling and sampling techniques**

During the original site investigation for the road con-
bstruction 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
in Figure 2, to retrieve ‘double U100’ samples. (It should
be noted that these sample tubes are sometimes referred
to as U4 from the old imperial dimensions.) This tech-
nique is frequently used in Ireland in similar materials.
Little was known at the time of the possible densifica-
tion effects caused by drained (or at least partially
drained) penetration of sampling tubes in sandy or silt
material.

For this project it was decided to investigate these
effects by sampling using several techniques. 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
dge 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 is
known to yield high-quality samples of soft clays.
Finally, at Sligo samples were obtained using the ‘double
U100’ technique described above (Fig. 2b) and at
Dunkettle continuous samples were recovered using the
**MOSTAP** process (Weltman & Head 1983; Long
2002; see Fig. 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 (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 sam-
plers is given in Table 1. The CPTU tests were carried
out according to **ISSMGE** (1999) using a standard
10 cm² cone.
The principal means of studying the difference in behaviour of the material from the various 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 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 0.6σ'\(_{v0}\) (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 & Ireland (1965). The final consolidation stresses were kept constant until the rate of volumetric strain was less than 0.0001% min\(^{-1}\). Shearing was carried out at the slow rate of 4.5% day\(^{-1}\). 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, to attempt to accurately define the preconsolidation stress, ‘gentle’ load increments to 0.25σ'\(_{v0}\) (the in situ vertical effective stress), 0.5σ'\(_{v0}\), 0.75σ'\(_{v0}\) and 1.0σ'\(_{v0}\) 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% h\(^{-1}\). Otherwise the procedures used were again broadly those adopted as standard by NGI (Sandbekken et al. 1986).

![Table 1. Summary of dimensions and features of samplers](image)

| Sampler      | Length (cm) | \(D_e\) (cm) | \(D_{ew}\) (cm) | \(D_s\) (cm) | Area ratio (%) | Cutting edge angle (deg) | Inside clearance (%) |
|--------------|-------------|--------------|----------------|--------------|----------------|------------------------|---------------------|
| ELE 100      | 100         | 10.14        | 10.48          | 10.14        | 6.8            | 30 (Sligo)             | 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 (Fig. 2b)    | 0                   |
| MOSTAP\(^\text{®}\) | 200   | 6.5          | 9.3            | 6.5          | 105            | 15                     | 0                   |

\(D_s\) inside diameter of cutting shoe; \(D_{ew}\) outside diameter of cutting shoe; \(D_e\) inside diameter of sampling tube; area ratio = \((D_{ew}^2 - D_e^2)/D_e^2\); inside clearance = \((D_s - D_e)/D_e\).

Laboratory testing

Site characterization

Ground conditions, Sligo

Visual inspection of recovered samples confirms that three distinct soft strata were encountered, as can be seen in 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 (Fig. 4). The following measured and derived parameters were used to analyse the CPTU test results (Lunne et al. 1997): \(q_c\), cone resistance corrected for out-of-balance pore pressure effects; \(q_n\), net cone resistance (= \(q_t - \sigma_v\)); \(f_s\), measured sleeve friction; \(N_m\), bearing capacity factor (= \(q_t/\sigma_v\)).

In the peat, above 2.0 m, the \(q_t\) and \(f_s\) values are generally low, 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 \(F\) values are lower and \(u_2\) values are higher than in 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 \((u_2 > u_0)\) between 4 and 10 m. Below 10 m the \(u_2\) profile plainly indicates the existence of sandy lenses within 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 homogeneous silt material.

**Basic material parameters, Sligo**

Basic material properties for Sligo are shown in Figure 3a. Data for the peat are 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 m\(^{-3}\), 1.7 Mg m\(^{-3}\) and 1.65 Mg m\(^{-3}\), respectively. As would be expected from the depositional environment, local organic zones occur in the sandy silt (e.g. at about 10 m) where higher moisture content and lower bulk densities are recorded.

Above 4.5 m the particle size distribution data for the peat and peat–silt 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 in 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 (BSI 1999) contains no definitive guidance but states that such borderline material should be termed SILT/SAND unless other supporting laboratory tests (such as plasticity tests) are available. BS5930 (BSI 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 4.5–13.5 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 in Figure 3b and it can be seen that, below a variable upper layer, data for the silt are
relatively consistent, and the \( w \) and \( \rho \) values are 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 to 1.8 \( \text{Mg m}^{-3} \). For the overlying peat and peaty silt average values are about 80% and 1.6 \( \text{Mg m}^{-3} \), respectively.

In contrast to 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 in Figure 5, with the Sligo average curve forming the ‘coarser’ limit of all of the Dunkettle curves.

Plasticity data for Dunkettle, shown in 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 in contrast to the higher values measured at Sligo. These higher organic contents are, in part at least, the reason for the Sligo material displaying 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 \( \rho \) and \( w \) values for each of the sampler types and from both sites is presented in 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 \( \rho \) values on average 3% lower and \( w \) on average 7% higher than those of the other samplers. For Dunkettle, the MOSTAP\(^*\) data show a clear difference from the others, and \( \rho \) 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 the two sites

It is likely that the principal difference between the two sites is that the tidal influence at Dunkettle, because of the greater proximity to the sea, is significantly more

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**Table 2. Study of sampling-induced densification effects**

|                | ELE 30\(^*\) | ELE 5\(^*\) | NGI 95 | Double U100 | U100 | MOSTAP\(^*\) |
|----------------|--------------|------------|--------|-------------|------|--------------|
| Sligo \( \rho \) (\( \text{Mg m}^{-3} \)) | 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 \( \rho \) (\( \text{Mg m}^{-3} \)) | 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.*

n.a., not applicable.
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.

Below, 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. None the less, to minimize 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 ($\sigma'_v$) v. strain ($\varepsilon$) or void ratio ($e$) and creep properties from plots of $\varepsilon$ 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, because of the nonlinearity of the resulting curves and the artificial separation of primary consolidation and creep effects.

Although conventional curves will also be presented here, use is made in general of the theory of Janbu (1985) 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 $\sigma'_v$ v. $\varepsilon$ and $\sigma'_v$ v. constrained modulus ($M = \Delta \sigma_v/\Delta \varepsilon$) curves, for the sandy silt at about 6 m, are shown in Figure 7. The classical log stress v. strain curves are of rounded nature and it is impossible to determine the yield or preconsolidation stress. 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 as for the others.

According to Janbu (1985), silt or sand material will show a gradually increasing $M$ with increasing $\sigma'_v$ as the particles are compressed tighter together. He suggested that the material could be characterized by a power function as follows:

$$M = m \sigma'_v \left(\frac{\rho_a}{\rho'_p}\right)^{1-a}$$

where $m$ is the modulus number, $\rho'_p$ is the reference stress ($\approx 100$ kPa), and exponent $a$ is 0.25 for silt.

As can be seen from Figure 7, this model can be used successfully to characterize 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 in Figure 8, together with an 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 in 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 30–70%, is shown in 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 owing to the energetic depositional environment.

From the oedometer testing $M_0$ (i.e. $M$ at in situ vertical effective stress $\sigma'_v$) 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)

$$M_0 = a_q \phi_{int}.$$  

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

Values of the Janbu (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 55–70%. 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 Figures 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 in Figure 11a and b. In each case,
pairs of test results are presented (i.e. tests from two different samplers at the same depth). Data are presented in the form of deviator stress ($\sigma_\text{d}' = \sigma_\text{d} - \sigma_\text{a}$) v. axial strain ($\varepsilon_\text{a}$) and pore pressure ($u$) v. $\varepsilon_\text{a}$ and as a mean stress/shear stress ($\sigma_* = (\sigma_\text{d} + \sigma_\text{r})/2$ and $\tau_* = (\sigma_\text{d}' - \sigma_\text{r}')/2$). From the plots the following can be noted: (1) except, perhaps, for the Sligo deviator stress v. $\varepsilon_\text{a}$ plot, all the results are very consistent; (2) all tests show dilatant behaviour, with deviator stress increasing with increasing $\varepsilon_\text{a}$; (3) up to $\varepsilon_\text{a}$ of about 2%, pore water pressure increases but then decreases (dilatancy); this dilatancy is particularly marked for the shallow Sligo samples; (4) the stress paths show some small initial contraction but then dilate strongly and form a clear failure line; (5) there is no clear difference between the results for the different sampler types.

**Effective stress strength parameters**

Effective stress strength parameters ($\varphi'$, $c'$) 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 Figure 11a and b, both the Sligo and Dunkettle material show friction angle, $\varphi'$, values in excess of 40°. It may also be argued that both materials also possess some cohesion, $c'$, of the order of 2–5 kPa. Although these $\varphi'$ values appear high, others have reported similar results; for example, Schultz & Horn (1965) for German silt ($\varphi' = 36^\circ$), Börgesson (1981) for silt from northern Sweden ($\varphi' = 40^\circ$ and greater) and Hoeg et al. (2000), who found $\varphi'$ 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 by 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 ($\beta = 0^\circ$): (1) at around 4 m, $N_m$ is typically 7.5, $B_q = 0.1$, tan $\varphi' = 0.48$ and $\varphi' = 26^\circ$; (2) below 4 m and above 10 m, $N_m$ is typically five, $B_q = 0.2$, tan $\varphi' = 0.45$ and $\varphi' = 24^\circ$. 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, to provide sufficient safety...
factor and to minimize strains, a safety factor of 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 0.5–1.0%, which are perhaps reasonable, if rather high, for 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; Sandven 2003) have noted that use of $s_u$ values is inappropriate for soils where $B_q$ is <0.4; that is, for material coarser than clayey silt. As an alternative, they suggested 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 practising engineers both directly and in correlations.

Undrained shear strength data for both Sligo and Dunkettle are presented in 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 are 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 $0.3\sigma'_{vo}$ line (roughly representing the shear strength of a normally consolidated material). For Dunkettle the in situ vane data are less scattered because of the lower organic content, and both these and data from unconsolidated undrained triaxial tests (UU) are on average close to the $0.3\sigma'_{vo}$ 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 that 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 the Mohr–Coulomb line; (5) peak principal stress ratio ($\sigma'_{1}/\sigma'_{3}$); (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 Figure 11a and b, 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.

Data from the CPTU test are also very encouraging, especially in the sandy silt material where there is little influence of 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 are available from several locations at the Sligo site (Ruiz 2003) and a typical example from C/S 405 m is given in Figure 14. In Figure 14a details of the embankment construction, ground conditions and instrumentation (namely, settlement plates, piezometers, magnet extensometers and inclinometers) are given. Settlement data are given on Figure 14b. Vertical drains were installed at 1–1.5 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 caused by 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 in Figure 14b. Calculations were performed using the program KRYKON (Svanø & Emdal 1987; Svanø et al. 1991), which uses the Janbu (1985) 1D consolidation theory as discussed above. Some details of the input parameters are given in Table 3. No attempt was made to refine the input parameters 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. the value of constrained modulus, \( M \), used was too high) and overestimate primary compression at higher stresses (i.e. the value of modulus number, \( m \), used was 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 characterization 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 various 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 characterize
the behaviour of the Sligo and Dunkettle silts. Specifically, the value of $a$ in the formula $M = \rho p_a (\sigma'_v / \rho_a)^{1.9}$ 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 measured settlements reasonably well, 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 because of the partially drained shearing process and the reinforcement effect of fibres.

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

| Material       | $\rho$ (Mg m$^{-3}$) | $p'_v$ (kPa)$^a$ | $M$ at $p'_v$ (MPa) | $m$ | $\sigma_{ref}$ (kPa)$^b$ | $c_v$ (m$^2$ a$^{-1}$)$^c$ | $r_s$ |
|----------------|--------------------|------------------|---------------------|-----|---------------------|---------------------|------|
| Peat           | 1.5                | $\sigma'_v + 5$  | 0.26–0.37           | 10  | $-25$               | 200                 | 350  |
| Peat–silt      | 1.7                | $\sigma'_v + 5$  | 0.56–0.66           | 14  | $-25$               | 250                 | 300  |
| Sandy silt     | 1.65               | $\sigma'_v + 5$  | 0.75–1.02           | 16–12| $-25$               | 350                 | 250  |

$^a$Preconsolidation stress. Water table assumed to be at 1 m. $^b$Ordinate where back-projection of $M-\sigma'$ line crosses the $x$-axis. $^c$Coefficient of consolidation.
(6) Useful parameters can be derived from CPTU tests in silty soils, 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 as a result of 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 because of 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 investigations on silty soils should use a combination of field measurements (particularly CPTU) and sampling using a relatively large diameter sampler (ideally about 75 mm) with a sharp cutting edge angle.

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