

## Geotechnical properties of Dublin Boulder Clay

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Geotechnical characteristics of Dublin Boulder Clay (DBC), based on detailed site investigation and site experience from some recent large projects in Dublin, are presented. This paper attempts to synthesise available information in parallel with recent work by Skipper *et al.*, who provide an updated understanding of the geology of the DBC. Having assessed the effects of sampling disturbance, the paper characterises the various formations and sub-units of the DBC. The interpreted material behaviour is related to observed engineering performance. It was found from the behaviour of earth retained structures that intact, clayey, DBC formations are two to three times stiffer than assessed from high-quality laboratory tests on block samples. DBC is shown to be significantly stiffer than other well-characterised tills. Relatively inexpensive multichannel surface wave techniques can give very reliable estimates of in situ small-strain stiffness. High undrained triaxial compression strengths were measured, and it appears that simple UU tests on high-quality specimens give good results. Significant strength anisotropy was suggested by undrained triaxial extension strengths that were only 30–50% of the triaxial compression strengths. Field horizontal permeability values of the intact clayey till units have a representative mean of about  $10^{-9}$  m/s, and when compared with laboratory values suggest that the material may exhibit some degree of anisotropy of permeability. Overall, the measured engineering parameters for the DBC are favourable for many construction projects. Further work is required in order to understand the in situ horizontal stress profile and the stiffness anisotropy of the till.

**KEYWORDS:** glacial soils; in situ testing; laboratory tests; site investigation; strength; stiffness

### INTRODUCTION

This paper gives geotechnical characteristics of the various formations constituting the Dublin Boulder Clay (DBC), based on experience of some large projects in Dublin, the capital city of the Republic of Ireland. DBC is the primary superficial deposit overlying bedrock. As a result of the recent boom in the Irish economy, several large-scale civil engineering projects have been completed or substantially

On présente ici les caractéristiques géotechniques de l'argile à blocs de Dublin (DBC), basées sur une étude détaillée du site et l'expérience du terrain acquise sur de larges projets récents réalisés à Dublin. Cet article tente de synthétiser les informations disponibles en parallèle avec les travaux récents de Skipper *et al.*, qui ont permis d'apporter de nouvelles informations contribuant à la compréhension de la géologie de la DBC. Il présente une caractérisation des diverses formations et sous-unités de la DBC, une fois déterminés les effets du remaniement causé par le prélèvement. L'interprétation du comportement du matériau est liée aux performances techniques observées. D'après le comportement des structures retenues par la terre, on a pu déterminer que les formations intactes et argileuses de DBC sont deux à trois fois plus rigides que celles étudiées par des essais en laboratoire de haute qualité sur des échantillons de blocs. On a pu montrer que la DBC est significativement plus rigide que les autres tills aux caractéristiques connues. Des techniques de MASW (Multichannel Analysis of Surface Wave) relativement peu coûteuses permettent d'obtenir des estimations fiables de la rigidité à petites déformations in situ. Des résistances élevées en compression triaxiale non drainée ont été mesurées, et il apparaît que de simples essais non consolidés-non drainés (UU) réalisés sur des échantillons de haute qualité donnent de bons résultats. L'observation de résistances en extension triaxiale non drainée représentant uniquement 30 à 50 % des résistances en compression triaxiale suggèrent une forte anisotropie. Les valeurs de perméabilité horizontales in situ pour les unités argileuses intactes de till ont une moyenne représentative d'environ  $10^{-9}$  m/s, ce qui, par comparaison aux valeurs en laboratoire, suggère que le matériau peut présenter un certain degré d'anisotropie de la perméabilité. Dans l'ensemble, les paramètres techniques mesurés pour la DBC sont favorables pour de nombreux projets de construction. Il est nécessaire de poursuivre les travaux de recherche pour mieux comprendre le profil de contrainte horizontale in situ et l'anisotropie de rigidité du till.

advanced. From an associated wealth of borehole investigations, in situ tests, laboratory tests, excavation exposures and monitored field performance, these projects have provided much more detailed information on the superficial deposits in Dublin.

Technical data are now available from a concentration of such projects in Dublin, but these are not in the public domain, and have not been interpreted or correlated on a (geological) formation-level scale. There are significant economic benefits in understanding the characteristics of the DBC, as it underlies much of the city: see Fig. 1. A parallel study is reported by Skipper *et al.* (2005) that provides a more detailed understanding of the geology of the DBC, and serves as a backdrop to this paper.

Many of the data are from the Dublin Port Tunnel Project (DPT, Site 1), and have been augmented by information

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from other important developments, as listed below and shown in Fig. 1.

#### Site 1: Dublin Port Tunnel

The central part of the DPT project comprises 12 m external diameter twin bored tubes, with lengths of cut-and-cover tunnel at either end. The project involves the excavation of about 1.5 million m<sup>3</sup> of soil and rock. Although project-wide information is considered, the focus in this paper is on experience gained at the northern cut-and-cover section and shaft WA2, where excavations were carried out in the superficial deposits to about 25 m depth: see Fig. 1. Further details of the project and a review of case histories associated with it are given by Long *et al.* (2003), Menkiti *et al.* (2004) and Milligan *et al.* (2006).

#### Other sites

A summary of the other sites considered and the data obtained is given in Table 1, and the site locations are shown in Fig. 1. Note that the sites span an area approximately 10 km east–west and 12 km north–south, confirming the geographical dominance of DBC in the area.

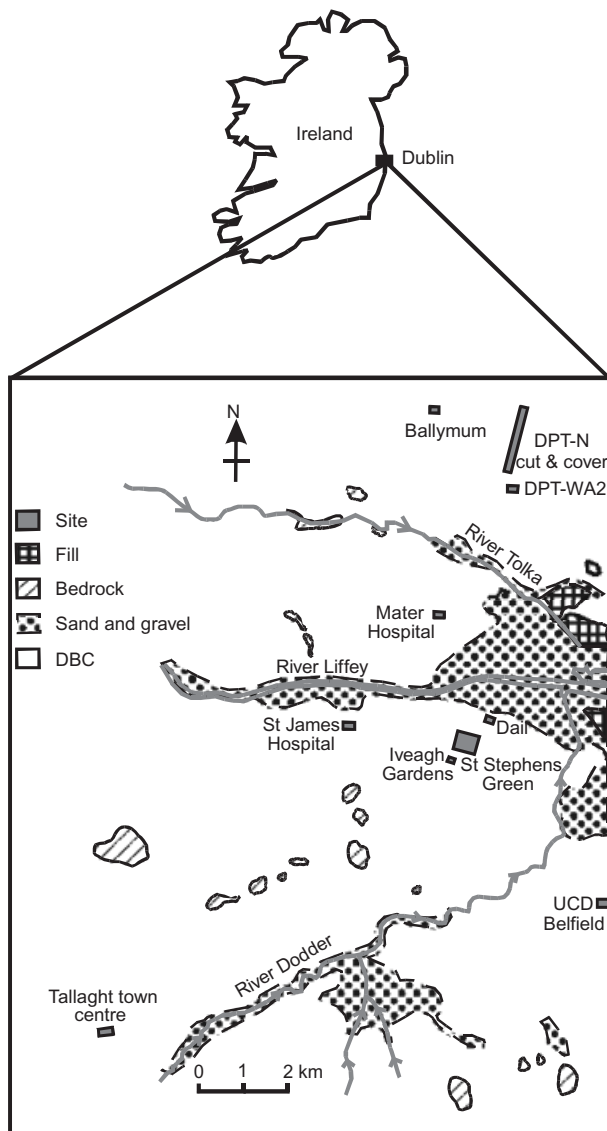


Fig. 1. Site locations showing outcrop of DBC

## SPECIAL SAMPLING METHODS

### Undisturbed block samples

Undisturbed pushed-in block samples were recovered from the DPT project. The block samples were taken using 300 mm or 350 mm cubical, thin-walled steel samplers, with a 20° or 45° outside angle cutting edge respectively, and 9 mm thick sidewalls. These sampler dimensions give an area ratio (AR) of 10–12%, where  $AR = (A_e - A_c) / A_c$ , and  $A_e$  and  $A_c$  are the external and internal areas of the cutting shoe. Only the central portions of the block samples were tested.

### Geobore 'S' rotary coring

High-quality samples were recovered, at several of the study sites, using this wire-line, triple tube rotary-coring technique with polymer flush to optimise sample recovery. The wire-line assembly allows the sample to be retrieved immediately after coring, and minimises the time for which the sample is in contact with the flushing medium. In general, 95% core recovery was achieved with experienced drilling crews. Samples for laboratory tests were cut from the recovered cores and sealed with wax and clingfilm immediately after core recovery. At the DPT site those obtained in 2000 and 2002/2003 were by Geobore-S boring, and the 1996 sampling was by a mixture of conventional double- and triple-tube core barrels.

## GEOLOGICAL SETTING

Since a detailed description of the geology is given by Skipper *et al.* (2005), only a brief summary is provided here. The area of outcrop of the DBC is indicated in Fig. 1, based on the Geological Survey of Ireland 1:50 000 scale soils map of Dublin. Note that this sketch map is not intended to be a definite geological map of Dublin, but merely aims to show the location of the study sites and the prevalence of DBC in the area. In this zone, generally a thin layer of recent deposits of made ground or loess overlies DBC—a lodgement till deposited under ice sheets more than 1 km thick. The following distinct formations of the DBC have been identified by Skipper and co-workers, based on work for the DPT project. As well as at the DPT site all four units have been identified at the Mater Hospital (Fig. 1). South of the River Liffey the third unit (LBrBC) appears to be absent. Further work is required to define distribution of the various units throughout the city.

### Upper Brown Boulder Clay (UBrBC)

This is the weathered uppermost formation of the DBC. It is a 2–3 m thick, stiff to very stiff, brown, slightly sandy clay, with rare silt/gravel lenses and some rootlets, particularly in the upper metre. Farrell *et al.* (1995a) have confirmed that this material is the weathered zone of the underlying Upper Black Boulder Clay (UBkBC), and that oxidation has produced the brown colour. The UBrBC represents a complex pedogenic horizon, which developed during a period of climate warming and glacial retreat after the deposition of the UBkBC.

### Upper Black Dublin Boulder Clay (UBkBC)

This is a very stiff, dark grey, slightly sandy clay, with some gravel and cobbles. It is typically 4–12 m thick. Rare, sub-vertical, rough and very tightly closed fissures spaced at 0.5–0.75 m were observed at some locations. Considerable mechanical effort was required to excavate the material, which tended to break into pedes of similar dimensions. In

**Table 1. Summary of sites considered other than Dublin Port Tunnel**

Site	Project	Fieldwork	Laboratory data	Reference
Ballymun Northern Gateway	Raft foundation for a large tower block	High-quality Geobore-S cores	$u_r$ , $K_0$ , stiffness from local strain gauge transducers.	Brangan (2007)
Dáil Eireann	Retaining structures for a deep excavation adjacent to the Irish Parliament buildings	High-quality Geobore-S cores. Data from performance of the retaining walls	$s_u$ , various tests Isotropic compression, oedometer, $s_u$	
Mater Hospital	Large hospital structure	High-quality Geobore-S cores. MASW geophysics	$s_u$	
St James Hospital (LUAS)	Platform structures and underground excavation for light rail system	MASW and cross-hole geophysics		
Iveagh Gardens Tallaght	Portal for Dublin Metro Commercial development with deep excavation	MASW geophysics MASW geophysics. Performance of retaining walls		

the DPT project, these fissures were observed to be so tightly closed in the in situ material that they could not be seen in borehole cores, and were obvious only in excavated material and slopes owing to the manner in which the material broke apart. Thin horizontal cobble lines of striated, faceted cobbles are frequently seen, which persist laterally for tens of metres before terminating abruptly. These are associated with the sub-horizontal discontinuities and possibly with shear planes within the lodgement till. Occasional small gravel lenses occur along the cobble lines, and these may be water bearing. It was observed during the DPT project that the gravel lenses and cobble lines constitute a network that appears to be hydraulically interconnected in some areas and hydraulically isolated elsewhere.

*Lower Brown Dublin Boulder Clay (LBrBC)*

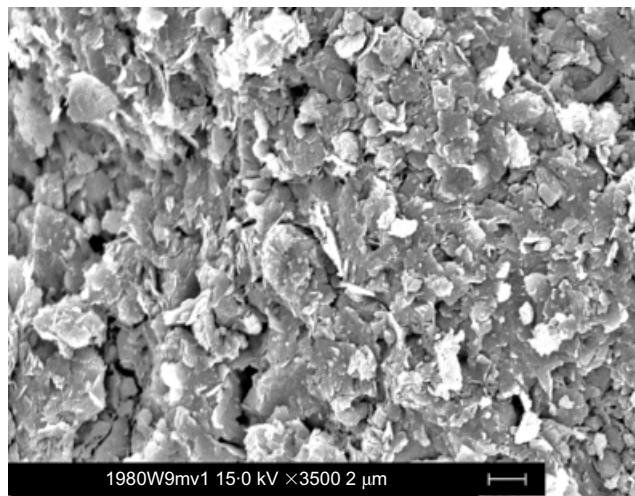
The LBrBC exists as a 5–9 m thick, hard, brown, silty clay, with gravel, cobbles and boulders. It has previously been called the ‘sandy boulder clay’ as it is similar to but siltier than the UBkBC above. The unit contains more frequent, larger and more complex silt/gravel lenses and cobble lines than the UBkBC. At the DPT site a continuous 2 m thick layer of silty sand/fine gravel exists within the unit at 10–16 m depth.

*Lower Black Dublin Boulder Clay (LBkBC)*

The LBkBC is a patchy layer of hard, slightly sandy, gravelly clay with an abundance of boulders. It is generally more plastic to the touch than the LBrBC. Its thickness does not exceed 4 m and is typically less than 2 m.

*Soil fabric*

In general, the intact clayey DBC formations comprise a dense packing of well-graded material, leading to a high bulk unit weight, low pore sizes and hence low permeability. This is consistent with the in situ development of suctions following stress relief, as reported by Long *et al.* (2004). Fig. 2 shows a typical view of the UBkBC, as observed under a scanning electron microscope. The view is typical of the clayey formations of DBC. It can be seen that the clay and silt sized fractions comprise both platy and rotund particles.



**Fig. 2. Typical view of the fabric of the UBkBC**

**BASIC MATERIAL PARAMETERS**

*Stratigraphy and groundwater*

At the Port Tunnel site all four units of the DBC were present, as shown in Fig. 3. Also shown on this plot are piezometric pressures measured in piezometers and standpipes, whose tips were located at various depths. It can be seen that the measured pressures correspond to approximately hydrostatic conditions, with a groundwater table about 2 m below ground level.

*Moisture content and bulk density*

The distribution of moisture content and bulk density with depth for the DPT site is also shown in Fig. 3, and average values of all the data points are summarised in Table 2. Overall, both moisture content and bulk density values are uniform with depth. A detailed assessment of the data reveals that average moisture content in the UBrBC is somewhat larger than for the other layers, with a corresponding slightly smaller bulk density. The UBkBC has the lowest moisture content and highest bulk density. Values for the LBrBC and the LBkBC are very similar. The water content of the stone fraction alone was found to be negligible, indicating that all the soil moisture is retained in the soil matrix.

The formation most commonly encountered in normal excavations or at foundation level in Dublin is the UBkBC,

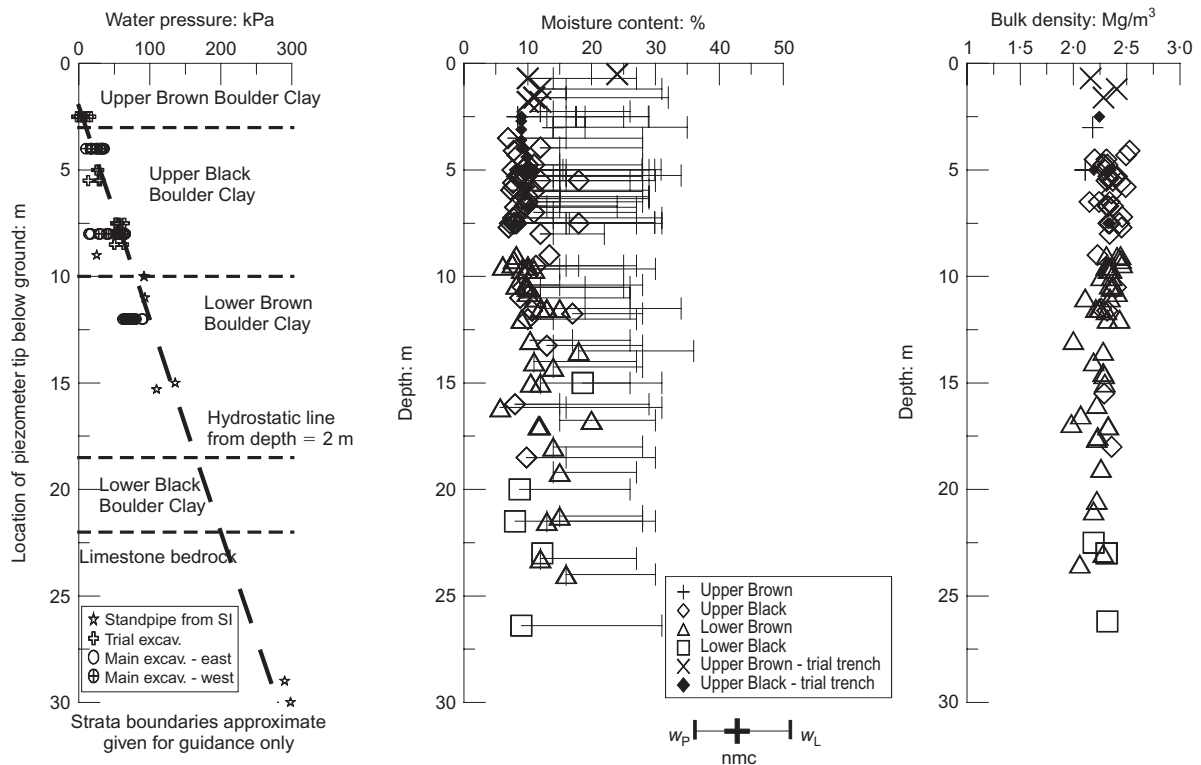


Fig. 3. Soil stratigraphy, groundwater and basic parameters: DPT site

and the values given in brackets in Table 2 for this material are typical values reported by Lehane & Simpson (2000). It can be seen that the DPT site data fall within the reported general range.

#### SOIL COMPOSITION

##### Particle size distribution

Particle size distribution curves for the four formations at the DPT sites are shown in Fig. 4. Perhaps the most uniform material is the UBkBC. Data for this site compare very well with the range of values for black DBC presented by Orr & Farrell (1996) (see Fig. 4), and with the typical values given by Lehane & Simpson (2000) (see Table 2). The curves represent well-graded material, and are typical of those of a lodgement till (Hanrahan, 1977; Trenter, 1999). The single available test for the UBrBC confirms that its composition is more or less identical to that of the UBkBC. The composition of both the LBrBC and LBkBC is more variable. For details of the sub-layers within the LBrBC, the reader is referred to Skipper *et al.* (2005).

Also shown in Fig. 4 are some curves for various layers and lenses within the main strata at the DPT site, but which are typical of those encountered elsewhere. These include lenses of relatively gravelly material in the UBkBC. These are often composed of distinct gravel/silt units, and may fine up or down. Deltaic sequences of sands, silts and clays are found in the LBrBC.

In general, the quantities of the various constituents are relatively constant with depth. Based on the average values quoted in Table 2, it can be seen that there is some tendency for the quantity of clay and silt to increase with depth. Except for the LBkBC, the sand content remains relatively constant with depth, and the gravel content reduces with depth.

##### Mineralogy

X-ray diffraction studies were carried out at the National History Museum, London, on samples of each of the four formations. Specimens tested had particle size less than 2  $\mu\text{m}$ . On average, 76% of these specimens were composed

Table 2. Summary of average basic material properties: DPT northern cut-and-cover site

	UBrBC	UBkBC	LBrBC	LBkBC
Moisture content: %	13.1	9.7 (11 $\pm$ 3)	11.5	11.3
Bulk density: Mg/m <sup>3</sup>	2.228	2.337	2.283	2.284
Liquid limit: %	29.3	28.3 (25 $\pm$ 4)	30.0	29.5
Plastic limit: %	15.9	15.1	14.9	17.8
Plasticity index: %	13.4	13.2 (11 $\pm$ 2)	15.1	11.8
Clay content: %	11.7	14.8 (15 $\pm$ 5)	17.8	17.5
Silt content: %	17.0	24.7	28.3	30.5
Sand content: %	25.0	24.7	25.7	34.0
Gravel content: %	46.3	35.8 (30 $\pm$ 5)	28.0	35.5

Note: The formation most commonly encountered in Dublin is the UBkBC. Values given in brackets are from general experience in Dublin as reported by Lehane & Simpson (2000).

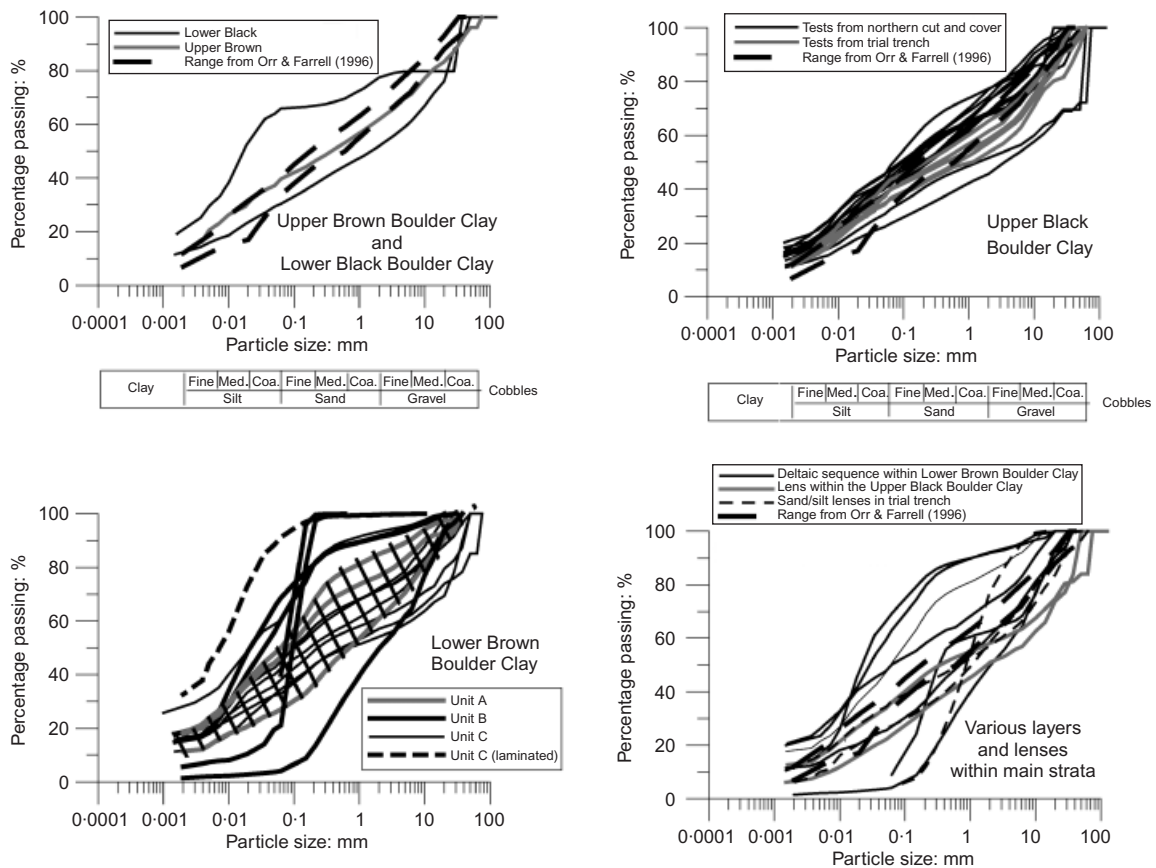


Fig. 4. Particle size distribution curves: DPT site

of clay minerals. Quartz, calcite and traces of amorphous iron made up the remainder. The clay minerals comprise a small fraction of kaolinite (i.e. 4–14% of the clay minerals), with the balance being split between the illite (28–43% of the clay minerals) and interstratified illite/smectite (48–57% of the clay minerals). There were no clear differences between the specimens from each of the four formations, except that there was some evidence of expandable clays found in the LBkBC specimens below 25 m depth. The presence of illite/smectite and kaolinite is suggestive of some surficial weathering.

*Organic content*

There is no evidence of organic material in the DBC. Loss-on-ignition values are generally close to zero.

*Specific gravity*

The specific gravity of DBC is typically 2.70. There is no clear difference in the results from the various formations.

SOIL PLASTICITY

Values of liquid limit and plastic limit for the DPT site are plotted against depth in Fig. 3 and on the ‘A-line’ plasticity chart in Fig. 5. Except for the presence of some clayey lenses in the LBrBC, the values are remarkably uniform, and data for all four materials fall in the classification ‘clay of low plasticity’ on the plasticity chart. Natural moisture content values fall very close to the plastic limit. Detailed assessment of the data in Table 2 confirms that the average values of index properties for all four materials are very similar. In the field, the LBkBC clay appeared to be the most plastic unit. This is in conflict with the limited

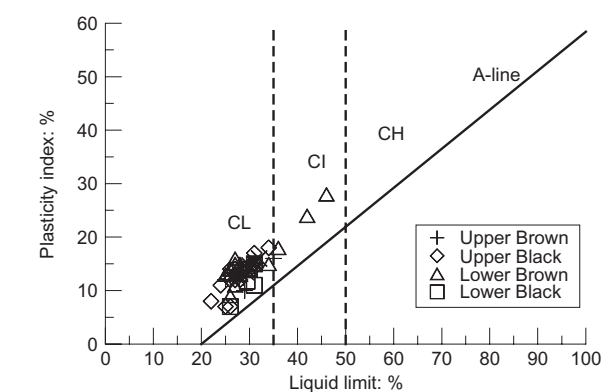


Fig. 5. Plasticity chart: DPT site

laboratory data (four results), where it appears to have a slightly higher average plastic limit with a correspondingly lower average plasticity index than the other units. Again, data from the DPT site are very similar to those reported by others: see Table 2.

IN SITU STRESS

As many of the more recent developments in Dublin involve two to three levels of basement car parking, engineers regularly face the problem of attempting to estimate the degree of overconsolidation and the coefficient of earth pressure at rest ( $K_0$ ). It is not possible to estimate these parameters directly from the geological history of the Dublin sites. This is because the depositional environment of these lodgement tills was complex, and there is a high degree of uncertainty regarding the stress conditions and pore water pressures imposed by past glaciations. Certainly it was

unlike that of marine or lakebed deposits, which were sedimented, consolidated and possibly also unloaded under one-dimensional conditions.

For tills it seems likely that the one of the most important mechanisms resulting in the heavily loaded state of the material is shearing during glacial advance rather than compression under the weight of glacier ice. This has been confirmed by Garneau *et al.* (2004), who carried out tests on a Dutch glacial till in a lateral stress oedometer and confirmed that material to have anisotropic horizontal stresses. The higher stress corresponded to the direction of ice advance.

In the absence of data, so far engineers in Dublin have based their estimate of  $K_0$  on published measurements by others, as summarised in Table 3. Values of  $K_0$  in the range 1.0–1.5 have been assumed in design.

For the DPT and Ballymun projects a variety of techniques were used to estimate  $K_0$ , as summarised in Table 4. For a variety of reasons all of the tests proved problematic. High-pressure dilatometer tests values show considerable scatter, range between 0.2 and 2.5 with an average of about 1.5, and show a tendency to a decrease with depth as would be expected. Powell & Butcher (2003) point out that estimates of in situ stress from pressuremeter tests in tills are difficult, particularly because of the stony nature of the material. As cores had to be cut using a heavy-duty rotary saw, it is likely that any residual suction close to the ends was removed during this cutting process, thus giving low values. In the special triaxial tests the samples were loaded axially at slow rates, and the horizontal stress was increased

in order to maintain horizontal strain close to zero. Horizontal strain was measured by a local (Hall effect) gauge. Three series of tests were carried out, with relatively low values being recorded in each case. In these tests it is not clear whether a measurement is being made of the stress condition required to maintain zero lateral strain on a disturbed sample, or of the in situ  $K_0$  value.

Current practice in Dublin (e.g. Dougan *et al.*, 1996; Long, 1997) is to use  $K_0$  values for DBC in the range 1.0–1.5. In the absence of further data from in situ testing (e.g. hydraulic fracture) and laboratory testing (e.g. lateral stress oedometer), it is recommended that designers continue to use these values and assess the sensitivity of the output to the assumptions.

#### PERMEABILITY

In the absence of permeable zones, the in situ mass permeability ( $k$ ) of the DBC is a key factor in controlling the behaviour in excavations. For the DPT site use was made of the natural ability of the DBC to stand unsupported at steep slopes (Long *et al.*, 2003; Menkiti *et al.*, 2004). It was therefore very important to characterise the permeability of the intact cohesive clay till and the more permeable zones. A variety of techniques were used to determine  $k$ , as summarised in Table 5.

It can be concluded from the measurements that typical permeability values for the intact cohesive till in the UBkBC and UBrBC formations are in the range  $10^{-9}$  to  $10^{-11}$  m/s. For design and finite element analyses at the DPT site, a

**Table 3. Summary of  $K_0$  measurements in till from literature**

Till	Technique	$K_0$	Reference
Iowa, USA	Stepped blade dilatometer	1.3	Handy <i>et al.</i> (1982)
Cheshire, UK	Hydraulic fracture and packer	0.7–1	Al-Shaikh-Ali <i>et al.</i> (1981)
Iowa, USA	Dilatometer, spade cell, pressuremeter	3–4 to 4 m depth then decreases to 1	Lutenegger (1990)
Cowden, UK	As above	As above	Powell & Butcher (2003)

**Table 4. Summary of  $K_0$  measurements on DBC**

Site	Technique	$K_0$	Reference used
DPT	High-pressure dilatometer (Cambridge Insitu device)	0.2–2.5 with average of 1.5	Mair & Wood (1987) Marsland & Randolph (1977) Hawkins <i>et al.</i> (1990) Chandler & Gutierrez (1986)
DPT	Filter paper	Low	Chandler & Gutierrez (1986)
DPT	Special triaxial: no preconsolidation	0.2–0.3	
	Triaxial: isotropic consolidation to $\sigma'_{v0}$	0.5–0.64	
	Triaxial: anisotropic consolidation with $K_0 = 0.6$	0.45–0.53	
Ballymun	Special triaxial: no preconsolidation	0.8	Chandler & Gutierrez (1986)
Ballymun	Filter paper	Low	

**Table 5. Summary of permeability measurements on DBC for DPT site**

Technique	Measured value: m/s	Reference
Variable head through conventional piezometer tips ( $k_h$ )	Intact cohesive till: $10^{-9}$ Silty/granular zones: $10^{-5}$ to $10^{-6}$ Mixed cohesive and granular zones: $10^{-7}$ to $10^{-9}$	BS 5930 (BSI, 1999)
Variable head through vibrating-wire piezometer tips with fine bore ( $k_h$ )	Range: $10^{-12}$ to $10^{-8}$ Representative mean: $10^{-9}$	Boylan & Carolan (2004)
Laboratory triaxial permeability tests ( $k_v$ )	Average value: $5 \times 10^{-11}$ (low scatter)	BS 1377 (BSI, 1990)
Consolidation stage of triaxial tests ( $k_v/k_h$ )	$10^{-9}$ to $10^{-10}$	BS 1377 (BSI, 1990)
Rowe cell ( $k_v$ )	$10^{-9}$ to $10^{-10}$ : little dependence on stress	BS 1377 (BSI, 1990)

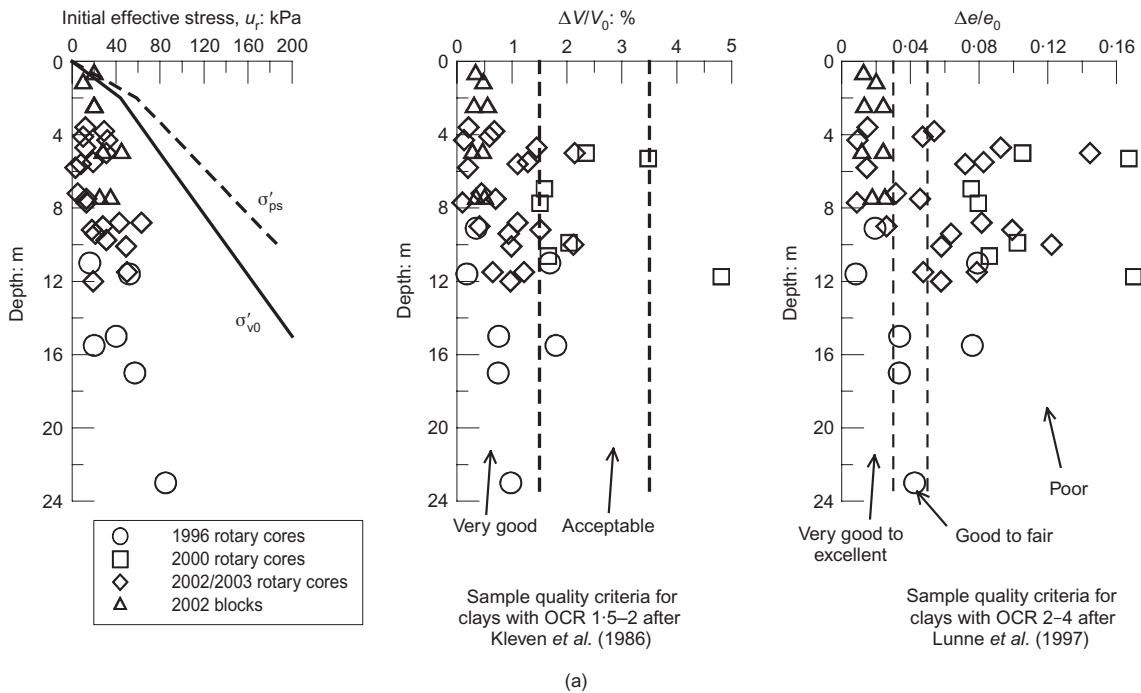
moderately conservative value for the mass permeability of cohesive till of  $10^{-9}$  m/s was used (Menkiti *et al.*, 2004). A higher value of  $10^{-8}$  m/s was adopted for the UBrBC. The mass permeability for hydraulically continuous silty/granular zones can be taken as  $10^{-6}$  m/s. A similar range of  $10^{-11}$  to  $10^{-8}$  m/s for the UBkBC was suggested by Lehane & Simpson (2000).

*Anisotropy of permeability*

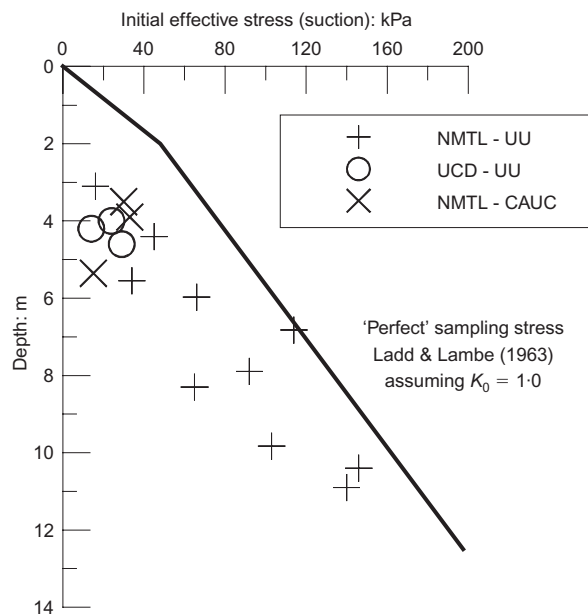
Permeability values as summarised above are thought to be appropriate values for horizontal flow in the till. Comparison of in situ and laboratory data (Table 5) suggests some degree of anisotropy, with perhaps vertical permeability likely to be an order of magnitude lower. This is consistent with the fabric and geology of the DBC units (Skipper *et al.*, 2005).

SAMPLE QUALITY

Three methods have been used to assess sample quality: see Fig. 6(a) and Table 6. Initial effective stress (or suction) in the samples,  $u_r$ , is compared with Ladd & Lambe's (1963) 'perfect sampling' stress ( $\sigma'_{ps}$ ). ( $K_0$  assumed = 1.5 for DPT. If  $K_0 = 1$ ,  $\sigma'_{ps} = \sigma'_{v0}$ , where  $\sigma'_{v0}$  is the in situ vertical effective stress.) Suction was measured in a triaxial cell by applying an initial cell pressure to the samples under undrained conditions, and recording the equilibrium pore pressure. Also plotted are (a) the normalised volume change ( $\Delta V/V_0$ ), and (b) the normalised void ratio change ( $\Delta e/e_0$ )—both measured during consolidation to the best estimate of in situ stress following the work of Lunne *et al.* (1997) and others. For the DPT site, it appears that the residual effective stresses ( $u_r$ ) are low, and come close to the  $\sigma'_{v0}$  or  $\sigma'_{ps}$  values only near the top of the sequence. However, these low values could be due to the testing technique used, as



(a)



(b)

Fig. 6. Sample quality: (a) DPT site; (b) Ballymun site

**Table 6. Sample quality assessment: DPT and Ballymun sites**

Site	Samples	Avg. $u_r$ : kPa	Avg. $\Delta V/V_0$ : %	Avg. $\Delta e/e_0$
DPT	1996 cores	45.0	0.93	0.042
DPT	2000 cores	n/a	2.49	0.112
DPT	2002/2003 cores	24.2	0.9	0.06
DPT	2002 blocks	25.4	0.41	0.019
Ballymun	2003 cores	77	0.68	0.041

filter paper was attached to the samples to accelerate consolidation.

For the DPT block samples  $\Delta V/V_0$  and  $\Delta e/e_0$  values are low, and the specimens can be categorised as ‘very good to excellent’. The 1996 and 2002/2003 cores are also generally of high quality, with more scatter in the data for the 1996 cores. These 1996 cores were recovered with a mixture of conventional double- and triple-tube core barrels. The quality of the 2000 cores is generally poorer than that of the other two sets. This increase in quality of the GeoBore-S cores with time possibly reflects improved experience in operating this technique in DBC, reorientation of the core head face discharge to minimise disturbance, and also a switch from air mist to polymer gel as a flushing medium.

Initial sample suctions for Ballymun (Fig. 6(b)) are relatively high, and close to  $\sigma'_{ps}$ . Suction values also increase with depth, and the trend is for little deviation from the ‘perfect’ sampling line, indicating that sample quality is consistently high. Measured values are generally higher than for the DPT site. This is likely to be due to the absence of filter drains on the specimens in this case. Average  $\Delta V/V_0$  and  $\Delta e/e_0$  values are also low (Table 6).

From the point of view of the practising engineer, these results confirm that triple-tube coring techniques and block sampling both provide high-quality samples, from which the results of the strength and stiffness tests are more representative of the in situ conditions.

**ONE-DIMENSIONAL COMPRESSION BEHAVIOUR**

Typical maintained load oedometer tests and triaxial isotropic compression tests for UBkBC from the Dáil Eireann site are shown in Fig. 7 (Brangan, 2007). Based on the rounded nature of the conventional plots of log stress ( $\sigma'_v$ ) against void ratio ( $e$ ), and following the discussion above on the mode of formation of the material, there is no clear evidence of a ‘preconsolidation’ stress ( $p'_c$ ). It is probable that the concept of a  $p'_c$ , as applied to normal sedimented clay, is not relevant for this material. Any numerical value for  $p'_c$  that may be obtained could simply be an artefact of the way the data are plotted.

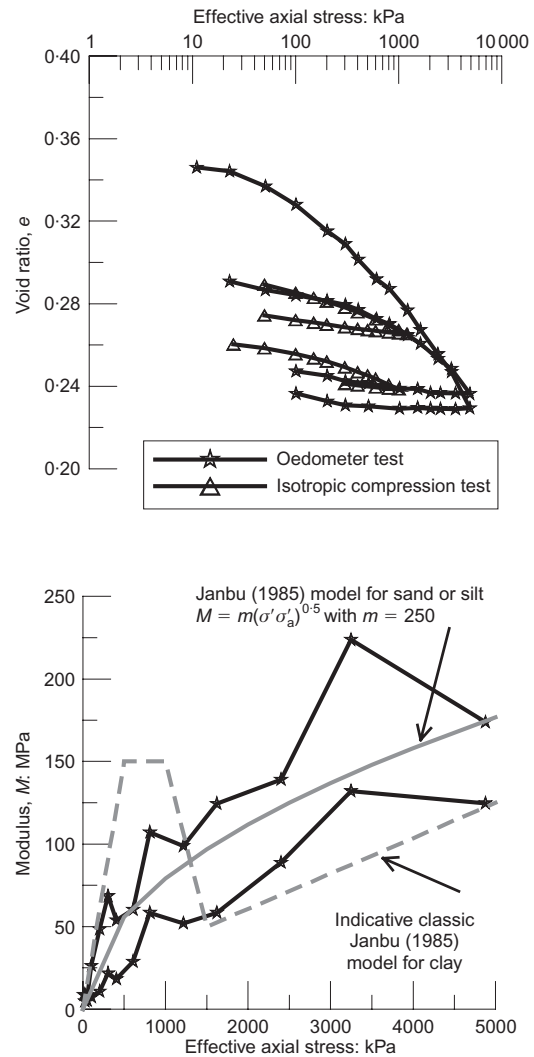
Inspection of the plot of  $\sigma'_v$  against constrained modulus  $M$  ( $M = 1/m_v$ ) shows a gradual increase in  $M$  with applied stress, and suggests the material behaves much more like sand or a silt than clay (Janbu, 1985): see Fig. 7. For sand or silt Janbu (1985) suggested

$$M = m(\sigma' \sigma_a)^{0.5} \tag{1}$$

where  $\sigma_a = 100$  kPa. These plots imply that the material has a modulus number ( $m$ ) of about 250, which is typical for a natural sand or silts with water content of about 10%.

*Compressibility and expansibility*

With due regard to the comments made above on the applicability of conventional one-dimensional consolidation theory to DBC, the curves presented in Fig. 7 suggest that



**Fig. 7. One-dimensional oedometer and isotropic compression tests from Dáil Eireann site**

the material has a compression index ( $C_c$ ) in the range 0.05–0.08. These are equivalent to critical state  $\lambda$  values of between 0.022 and 0.034, which are more or less identical to those suggested by Lehane & Faulkner (1998) for DBC. Similarly the swelling index ( $C_s$ ) lies in the range 0.004–0.007 (equivalent to  $\kappa$  in the range 0.002–0.003)—again very similar to values suggested by others for DBC.

*Coefficient of consolidation*

Owing to the small specimen size generally used in the tests (76–100 mm diameter and 19 mm height), a wide variety of results has been reported for the coefficient of consolidation ( $c_v$ ). Hanrahan (1977), Lehane & Simpson

(2000) and Brangan (2007) suggest that values may fall in the range  $40 \pm 20 \text{ m}^2/\text{year}$ . Similar large uncertainty in the assessment of field  $c_v$  of glacial tills has been reported by others (e.g. Trenter, 1999). Consequently, it is recommended that calculations be performed using  $M$  and  $k$  as an alternative to  $c_v$ .

**SMALL-STRAIN STIFFNESS**

In order to provide input for finite element analyses of the retaining walls and deep shafts at the DPT site, significant efforts were made to determine the small-strain shear stiffness of the various units, and its variation with strain. Techniques used at the DPT site included

- (a) cross-hole seismic
- (b) seismic refraction
- (c) multichannel analysis of surface waves (MASW)
- (d) shear wave velocity measurements using bender elements on triaxial specimens
- (e) measurements from Hall-effect gauges mounted on triaxial samples
- (f) values measured using unload–reload loops in the high-pressure dilatometer tests.

In addition, at the St James Hospital site both cross-hole seismic and MASW surveys were carried out. At Ballymun, Dáil Eireann, and Mater Hospital triaxial tests were conducted using sample-mounted transducers.

In situ measurements of small-strain shear modulus ( $G_{max}$ ) at the DPT site are plotted against depth in Fig. 8, together with the stratigraphy at the test location. It can be observed that  $G_{max}$  increases rapidly with depth from about 250 MPa in the UBrBC to about 1000 MPa at a depth of 8 m (in the UBkBC) and more slowly below this level. However, it increases sharply to about 2000 MPa in a dense sand layer at Shaft WA2, which is reflected in the cross-hole seismic data. Slightly higher values of 1000–1500 MPa are measured in the LBkBC, reflecting the high boulder content observed in this unit during excavation of Shaft WA2.

A summary of the results of laboratory bender element tests is also superimposed in Fig. 8, and the data seem to lie towards the lower bound of the in situ data. This is likely to be due to sample disturbance effects in the 1996 rotary

cores. It may also be due, in part, to specimen preparation difficulties associated with inserting the benders into the very stiff and stony samples. It was first necessary to excavate a small hole in the specimen and then seal the bender element in this hole using a mixture of epoxy resin and reworked till.

In general there is very good agreement between the results from the three in situ geophysical techniques. Clearly, the resolution of the seismic refraction data is poorer than that from either the MASW or cross-hole tests.

Further evidence of the reliability and accuracy of the MASW technique is provided by Donohue *et al.* (2003) for work at the St James Hospital site, as shown in Fig. 9(a). Agreement between the two in situ techniques is excellent. The practical implication of this work is that the relatively inexpensive MASW technique can be reliably used for determining in situ shear wave velocity (and hence  $G_{max}$ ) in DBC.

Shear wave velocity ( $V_s$ ) measurements for various DBC sites obtained using the MASW technique are shown in Fig. 9(b) (Donohue *et al.*, 2003; Donohue, 2005). It can be seen that, except perhaps for the St James hospital site, the values are very similar, suggesting the material is relatively uniform across the city.

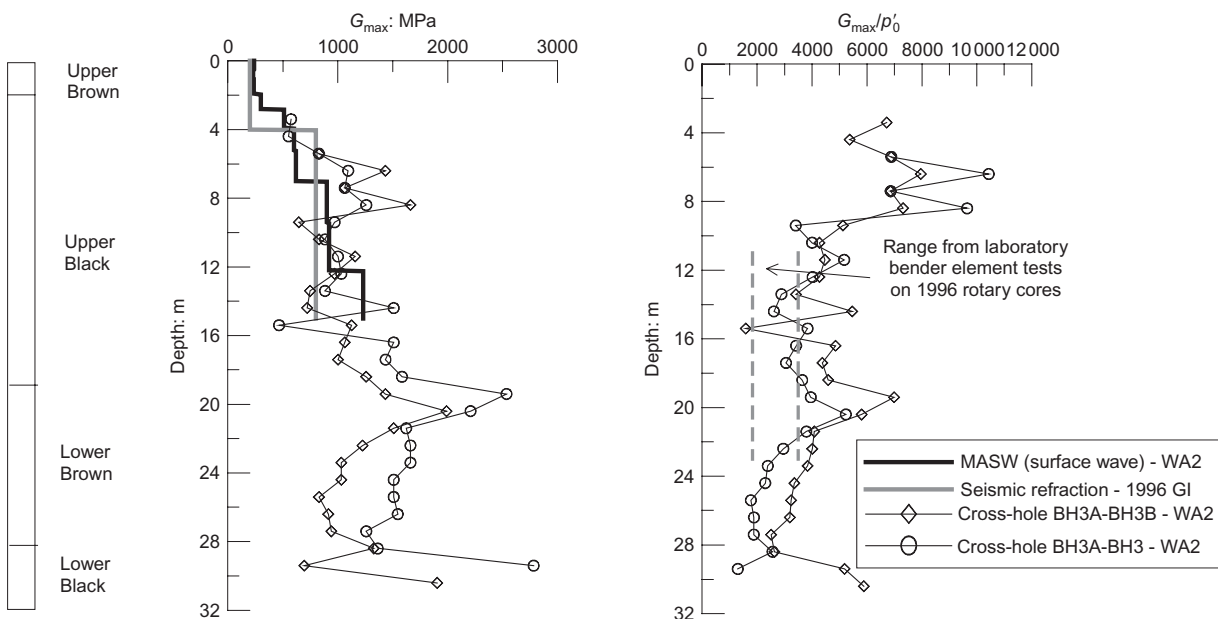
**STRESS–STRAIN BEHAVIOUR**

*Triaxial deviator stress–strain and stress paths*

Following measurement of  $u_r$  as described previously, specimens were saturated then consolidated in two stages, first isotropically to  $0.5\sigma'_{v0}$ , and subsequently anisotropically to  $(\sigma'_{v0}, \sigma'_{h0})$ , where  $\sigma'_{h0}$  is the in situ horizontal effective stress, based on the assumed  $K_0$  value. This final stress state was maintained until the samples stabilised (i.e. rate of volume change  $< 0.001\%/min$ ). Thereafter, the samples were sheared in undrained compression (CAUC) or extension (CAUE) at a strain rate of 4.5% per day. Some drained tests were also undertaken.

Plots showing some typical stress–strain behaviour and stress paths [in  $(\sigma'_a + \sigma'_t)/2, (\sigma'_a - \sigma'_t)/2$  space] for the UBrBC, UBkBC and LBrBC are shown in Fig. 10, and the following conclusions can be drawn.

- (a) In general, the behaviour of all three materials is



**Fig. 8. Small-strain stiffness: DPT site**

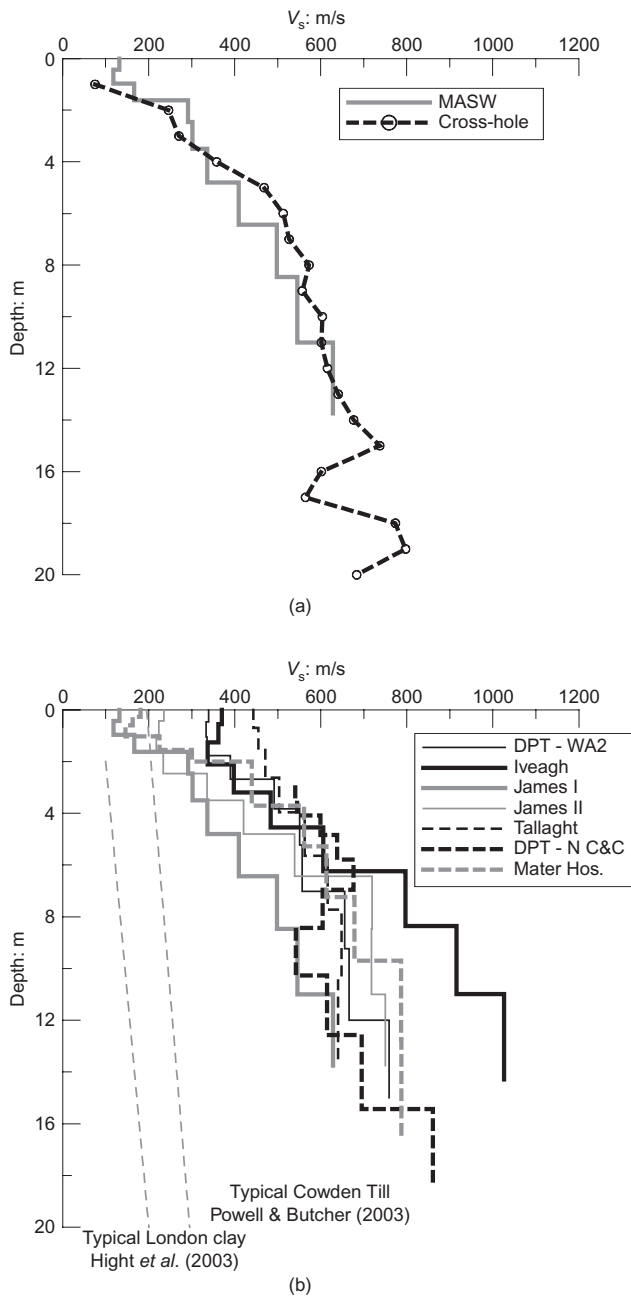


Fig. 9. In situ shear wave velocity from MASW technique: (a) St James Hospital Location 1; (b) various DBC sites

similar, with the UBrBC being weaker than the other two (see also Fig. 13).

- (b) The materials exhibit a strong dilatant behaviour.
- (c) The materials possess a strong anisotropy of undrained shear strength, with  $s_u$  in compression being three to four times that in extension.
- (d) There is not strong evidence for anisotropy of effective strength parameters.
- (e) There is some evidence that the quality of the block samples is marginally better than that of the rotary core (2002/2003) samples.

#### Pore pressure response

Pore pressure response for typical compression tests is shown in Fig. 11. The behaviour of the UBkBC and the LBrBC is similar. Initially, positive pore pressure changes are generated up to strain of about 1.5%, followed by strong dilatant behaviour, with net negative pore pressures being

developed after about 3.5% strain. The response of the UBrBC follows a similar pattern, but is much more muted.

The strong dilatant behaviour of the UBkBC and the LBrBC has significant implications for the observational approach as used in the construction of steep cuts in the material, such as on the DPT site. Such ductile behaviour will mean that, with careful monitoring, a warning of an impending failure will be apparent some time before the event.

#### Variation in stiffness with strain

Plots of undrained secant Young's modulus ( $E_u$ ) against strain, for the same triaxial tests as for Fig. 10, are shown in Fig. 12. From the plots the following observations can be made.

- (a) All three materials exhibit strong non-linearity of stiffness.
- (b) Stiffness increases with depth.
- (c) Stiffness in triaxial compression and extension are more or less the same.
- (d) There is no significant difference between the stiffness of block samples and the 2002/2003 rotary cored samples.
- (e) The projected small-strain stiffness from triaxial test results is similar to that derived from the in situ MASW testing.

From the point of view of practising engineers, the implication of this latter point is that a combination of good-quality triaxial tests on rotary cored samples and the relatively inexpensive in situ MASW technique can be used to estimate the full stiffness–strain relationship.

At present in Ireland, practising engineers usually adopt a simple linear elastic perfectly plastic constitutive model. A single 'operational' value of  $E_u = 100$  MPa is used, as derived from field observations by Farrell *et al.* (1995b). From the DPT tests, presented in Fig. 12, this value coincides with axial strains of the order of 0.01–0.1%, which seems reasonable for the normal level of strain encountered in engineering works.

Stiffness values from unload–reload loops in the high-pressure dilatometer tests are also shown as insets in Fig. 12. Shear modulus from the dilatometer tests has been converted to undrained modulus of elasticity and cavity strain has been converted to shear strain, after Jardine (1992). These stiffness values correspond to shear strains of the order of 0.05–0.7%, and can be seen to be significantly higher than the equivalent values from the triaxial tests. This possibly suggests some degree of stiffness anisotropy, consistent with the geological origin of the material, with horizontal values being perhaps four times greater than vertical ones. It may also reflect the presence of cobbles at the test pocket. The implied level of stiffness anisotropy is inconsistent with the triaxial data, for which the major principal stress is vertical (compression) or horizontal (extension), although the triaxial data also include changes to the relative magnitude of the intermediate principal stress. Further work is required on stiffness anisotropy of the DBC, as insufficient data are available to make definitive conclusions.

#### Comparison with other well-characterised materials

Figures 9(b) and 12 show a comparison between typical in situ shear wave velocity and normalised stiffness values ( $E_u/p'$ ) for UBkBC and LBrBC and data for London clay (Hight *et al.*, 2003) and Cowden till from the east coast of the UK (Powell & Butcher, 2003) respectively. DBC stiffness values are extremely high when compared with other stiff clays. It

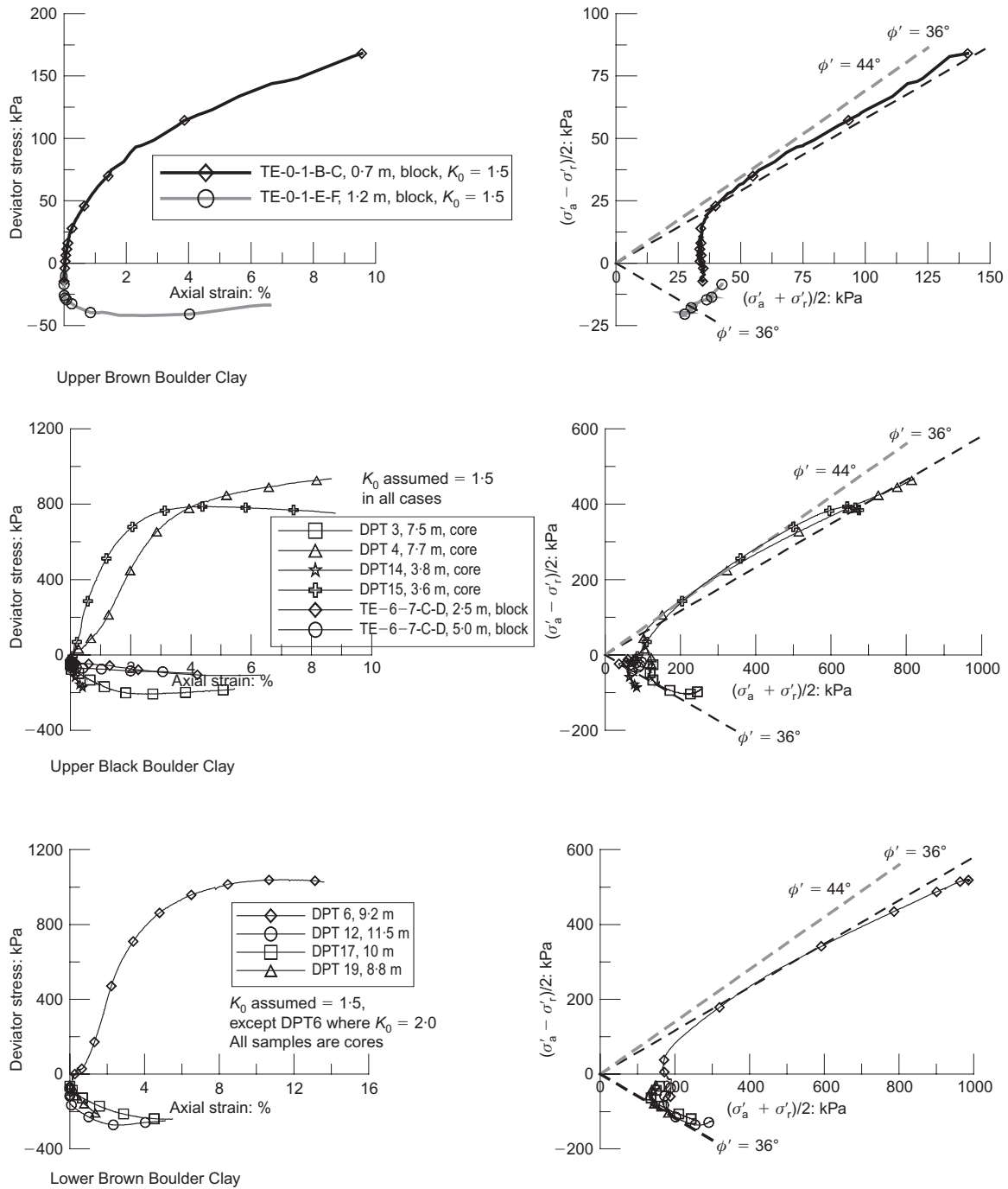


Fig. 10. Stress-strain behaviour and stress paths

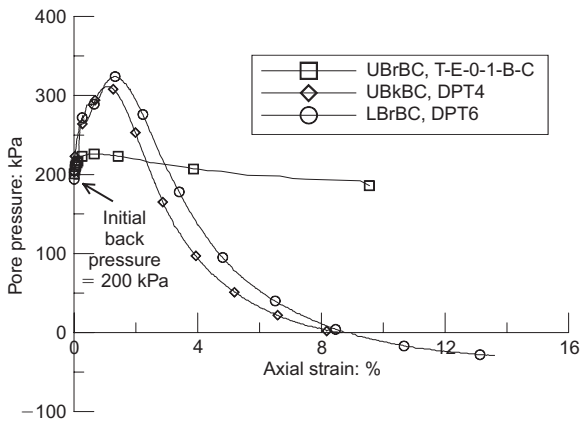


Fig. 11. Pore pressure response

is about six to eight times stiffer than London clay, and about five times stiffer than Cowden till.

*Stiffness from FE back-analysis*

For finite element analyses of various works on the DPT site, the DBC was assumed to have isotropic stiffness properties. The stiffness model for UBkBC and LBrBC used in the original finite element models is shown in Fig. 12. It can be seen that it was based mainly on the results of triaxial testing and laboratory Bender element testing. As has been discussed previously, it is now recognised that the effects of sampling disturbance and perhaps stiffness anisotropy mean that these original values were conservative. Subsequent back-analyses of an unsupported cut trial excavation (Menkiti *et al.*, 2004) and the lateral movements of the diaphragm walls supporting shaft WA2 (Cabarkapa *et al.*,

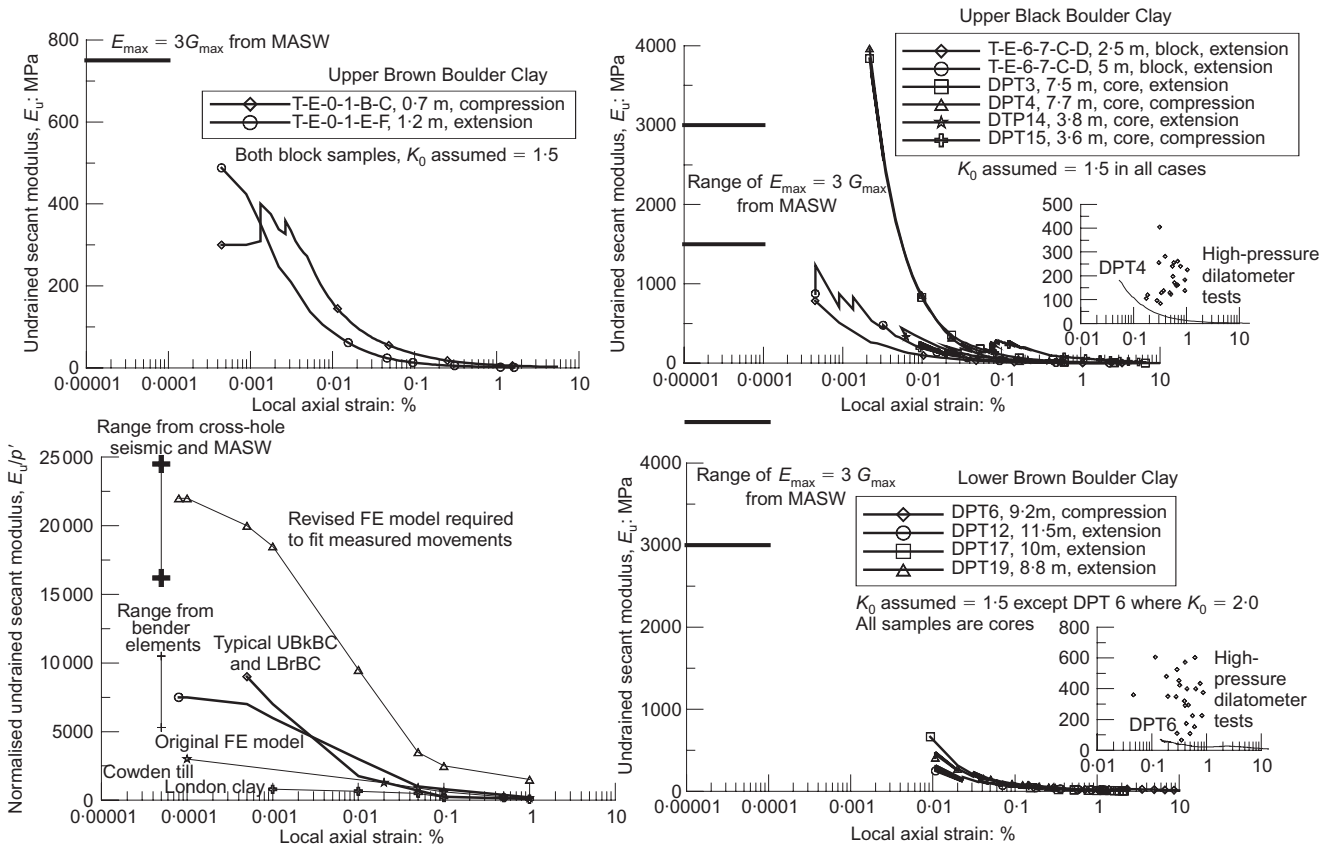


Fig. 12. Variation in stiffness with strain

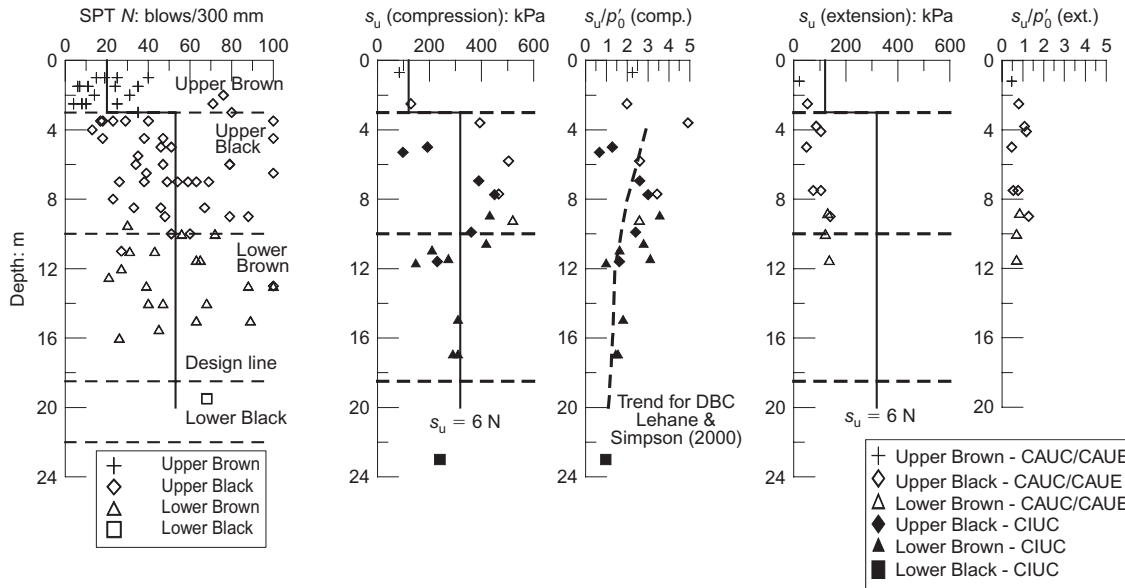


Fig. 13. Undrained shear strength from SPT and CAUC, CIUC and CAUE triaxial tests

2003) have confirmed this to be the case. A revised stiffness model required to fit the measured movements is shown in Fig. 12.

UNDRAINED SHEAR STRENGTH  
CAUC/CIUC/CAUE triaxial testing

The undrained shear strengths ( $s_u$ ) measured in the CAUC, CIUC or CAUE tests are presented in Fig. 13 and Table 7, where these tests are defined. Although the choice of  $s_u$  for a dilatant/strain-hardening material is not straightforward, peak values have been used here to be consistent

with current Irish practice. Values normalised by the initial mean effective stress,  $p'_0$ , are also given, and average values are presented in Table 7. Other researchers have taken  $s_u$  at a limiting strain (e.g. 2%), at the point where the stress path comes close to the Mohr–Coulomb failure line, or when pore pressure becomes negative. In this case the first two techniques would give significantly lower  $s_u$  values, whereas the third approach would result in values similar to those obtained from the simple peak. For engineering works control of deformation may be the dominant design factor, and design stress must be consistent with tolerable strain.

Despite some scatter in the data, there is a trend for

Table 7. Average laboratory  $s_u$  values: DPT site

	CIUC*		CAUC*		CAUE*	
	$s_u$	$s_u/p'_0$	$s_u$	$s_u/p'_0$	$s_u$	$s_u/p'_0$
Upper Brown	n/a	n/a	84	2.25	21	0.46
Upper Black	287	1.93	373	3.23	87	0.87
Lower Brown	297	2.11	520	2.58	129	0.75
Lower Black	240	0.98	n/a	n/a	n/a	n/a

\* CIUC, consolidated isotropically undrained compression test; CAUC, consolidated anisotropically undrained compression test; CAUE, consolidated anisotropically undrained extension test.

increasing strength with depth in the UBrBC and UBkBC from a value of about 80 kPa in the UBrBC to about 500 kPa at the boundary of the UBkBC and LBrBC clays at about 10 m depth. A similar trend can be seen in the normalised data, and the values of  $s_u/p'_0$  are indicative of material previously subjected to high load. The trend in  $s_u/p'_0$ , particularly that for the CIUC tests, is similar to the relationship suggested by Lehane & Simpson (2000): see Fig. 13. Lehane & Simpson's data were from CIUC tests on 100 mm diameter rotary-cored samples, similar to the 1996 cores under study in this paper. Lehane & Faulkner (1998) report  $s_u/p'_0$  values of about 0.45 for normally consolidated reconstituted DBC. This value represents the lower bound for the data presented in Fig. 13.

It is not thought that these high values are strongly influenced by the stone content. This was not so high as to mean the pieces were touching. They were generally surrounded by a matrix of 'clayey' material. These high strength values are consistent with field observations. Significant mechanical effort is required to excavate the material, and the perceived strength and stiffness increase with depth.

Extension strength (CAUE) data are also shown in Fig. 13 and Table 7 ( $K_0$  was assumed equal to 1.5). These tests give surprisingly low values, which increase approximately linearly from about 20 kPa near the surface to 130 kPa at 12 m depth. This linear increase with depth is not as apparent for

the normalised data. Strength anisotropy ratio  $s_{u-CAUE}/s_{u-CAUC}$  is about 0.25. Other researchers (e.g. Karlsrud *et al.*, 2005) suggest that this ratio should be of the order of 0.4–0.5 for material with an  $I_p$  of 10–15%. It was observed that most of the samples failed by forming shallow dipping failure planes (necks), which often engaged pre-existing discontinuities. It seems likely that the low values are due to the granular nature of the material. In contrast, compression failure was consistently of the classical 'barrel' type.

Further study is warranted to investigate the implied anisotropy of undrained shear strength, and the practical implications for engineering works.

UU triaxial testing

Unconsolidated undrained (UU) triaxial tests are frequently used in practice, given the time and cost constraints of carrying out more sophisticated tests. These tests are used as classification tests as well as tests to produce design parameters, so a good number of tests are usually carried out. Results from a series of UU tests on the DPT 2002/2003 rotary-cored specimens from a single borehole are presented in Fig. 14(a) and summarised in Table 8. For all the units,  $s_u$  values from UU and CIUC tests are very similar. Average CIUC  $s_u$  values for the UBkBC and LBrBC are 287 kPa and 298 kPa respectively, compared with

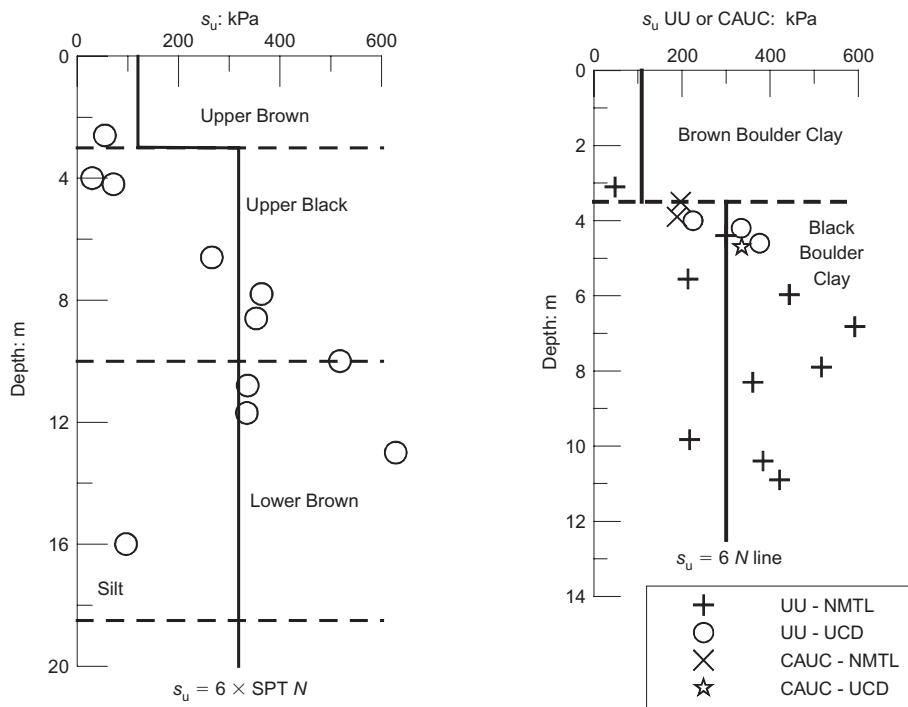


Fig. 14. Undrained shear strength from UU tests: (a) DPT, (b) Ballymun

**Table 8. Average SPT  $N$  and laboratory  $s_u$  values: DPT site**

	Avg. SPT $N$ : blows/300 mm	Avg. CAUC $s_u$ : kPa	CAUC $s_u/N_{avg}$	Avg. CIUC $s_u$ : kPa	CIUC $s_u/N_{avg}$	Avg. UU $s_u$ : kPa	UU $s_u/N_{avg}$
Upper Brown	19	84	4.4	n/a	—	n/a	—
Upper Black	53	373	7.0	287	5.4	217	4.1
Lower Brown	53	520	9.8	297	5.6	383	7.2
Lower Black	68	n/a	—	240	3.5	n/a	—

217 kPa and 383 kPa from the UU tests. A similar finding was made at the Ballymun site (Fig. 14(b) and Table 9). Sample quality assessment for these tests was presented in Fig. 6(b). Data from two laboratories, UCD and a commercial laboratory, are reported. This result has practical implications in that, once high-quality rotary core samples are available, it may be adequate to carry out cheaper UU rather than more expensive and time-consuming CIUC tests.

#### *In situ testing*

Standard penetration test  $N$  values for the DPT site are also shown in Fig. 13 (all data with equivalent  $N > 100$  have been omitted owing to the probable influence of cobbles). Although, like UU tests, the problems with SPT testing have been thoroughly discussed in the literature, the test is performed routinely in ground investigations in Ireland. The observed scatter in the data is considerable and typical, reflecting the large and variable stone content of the till. Stroud & Butler (1975) and Stroud (1989) suggest that, for a glacial till with average plasticity index of about 12%, the ratio  $f_1 = s_u/N_{avg}$  is about 6.0. Farrell & Wall (1990) report a similar value for the UBrBC and UBkBC, again based on CIUC or UU triaxial tests. Data for the DPT and Ballymun sites are summarised in Figs 13 and 14 and Tables 8 and 9, and suggest that  $f_1 = 6$  may be appropriate for UU and CIUC tests, but that a higher value of  $f_1$  could apply to CAUC tests.

Some in situ  $s_u$  data are also available from high-pressure dilatometer tests at the DPT site. Data were interpreted according to the limit pressure approach advocated by Powell & Butcher (2003) for Cowden till ( $s_u = \text{limit pressure}/6.18$ ). This analysis resulted in very high values of  $s_u$ , being on average about 835 kPa for the UBkBC and LBrBC. These authors show that this approach gives results consistent with large-diameter plate tests. Other available

interpretation techniques (see review by Mair & Wood, 1987) often give much higher values of  $s_u$ . This is likely to be due to fact that most of these techniques were developed for either stone-free contractant sedimentary clays or weak rocks, rather than the highly dilatant material under consideration here. It is also possible that in the DBC the tests were at least partially drained, hence resulting in the higher strength values.

#### *Undrained strength values in till reported by others*

Undrained shear strength values reported for other tills are summarised in Table 10.  $s_u$  values for the Dublin till are often considerably higher. The reasons for this are not clear, but DBC seems to have higher stone content than that encountered elsewhere.

### EFFECTIVE STRESS SHEAR STRENGTH

#### *Intact till*

From the stress path plots presented in Fig. 10, the following inferences can be drawn.

- The materials have a peak effective friction angle ( $\phi'$ ), in compression, of about 44°. This somewhat high value may be consistent with the nature of the clay fraction and the relatively high sand content in the till.
- Large deformation strength corresponds to  $\phi'$  of about 36°. This is similar to the critical state friction angle ( $\phi'_{cv}$ ) of  $34 \pm 1^\circ$  suggested by Lehane & Faulkner (1998).
- This divergence of the stress paths from the linear Mohr–Coulomb envelope at high strains may be indicative of the development of failure along some fissures or joints within the specimen rather than a

**Table 9. Average SPT  $N$  and laboratory  $s_u$  values: Ballymun site**

	Avg. SPT $N$ : blows/300 mm	Avg. CAUC $s_u$ : kPa	CAUC $s_u/N_{avg}$	Avg. UU $s_u$ : kPa	CIUC $s_u/N_{avg}$
Upper Brown	18	n/a	—	n/a	—
Upper Black	50	240.8	4.8	340.2	6.8

**Table 10. Summary of  $s_u$  measurements on till from literature**

Site	$s_u$ : kPa	Technique	Reference
Cowden, England	90–150	High-quality pushed samples	Powell & Butcher (2003)
Cheshire, England	As above	UU tests on U100 samples	Dobie (1989)
Cheshire, England	As above	As above	Al-Shaikh-Ali <i>et al.</i> (1981)
Chapelcross, Scotland	Lower than DBC	UU tests on U100 samples	Chegini & Trenter (1996)
Glasgow, Scotland	100–200	UU and UC tests on U100 samples. Large dia. plate tests	McKinlay <i>et al.</i> (1974)
North East England	50–410	UU, CIUC tests	Robertson <i>et al.</i> (1994)
Iowa, USA	100–150	CIUC tests on Shelby tube samples	Lutenegger (1990)

mass failure of the material, similar to that reported by Vaughan & Walbancke (1975) for Cow Green till.

- (d) In triaxial extension  $\phi'$  appears to be about  $36^\circ$ , that is, similar to the critical state or large-strain compression value. This again may be due to failure along some microstructural feature in the material.
- (e) Effective cohesion ( $c'$ ) is close to zero. This is consistent with the small and often rotund clay-sized fraction, the mode of formation, involving a high degree of shearing and lack of evidence of cementation bonding between the particles in fabric studies.

Peak friction angles (assuming  $c' = 0$ ) have been plotted against the mean consolidation stress ( $p'_0$ ) in Fig. 15. There is a clear tendency for decreasing friction angle with increasing effective stress, particularly for the extension tests. Thus the material has a curved rather than linear failure surface, similar to the conclusions of Atkinson & Little (1988) for St Albans till and Robertson *et al.* (1994) for Northumberland till.

*Reconstituted till*

Lehane & Faulkner (1998) report results of triaxial tests on reconstituted DBC. All drained and undrained stress paths terminate on straight lines passing through the origin, when  $p'$  at failure exceeds 200 kPa. These lines correspond to  $\phi' = 34 \pm 1^\circ$  in both compression and extension. This value is identical to  $\phi'_{cv}$ , and confirms that the behaviour of intact DBC in shear compares well with the behaviour of reconstituted material. Gens & Hight (1979) and others have found a similar result.

*Large-strain (residual) strength*

Loughman (1979) studied the residual shear strength of DBC and found that, in general, the  $\phi'_{res}$  was high ( $>30^\circ$ ) owing to the high silt content of the material. This confirms that, as would be expected from the mode of formation of the material, it has low sensitivity.

CONCLUSIONS

There are significant economic benefits in understanding the characteristics of DBC, as it underlies much of the city.

This paper gives geotechnical characteristics of the various formations constituting the DBC, and some of the main findings, which may be of interest to practising engineers, are as follows.

- (a) There are clear differences in the engineering parameters between the four units of DBC as defined by Skipper *et al.* (2005).
- (b) The two uppermost units are the most important from the point of view of engineering projects, and these seem to be relatively homogeneous with depth and with location throughout the city.
- (c) Localised granular lenses can be encountered. These can have very significant practical implications, for example for pile construction and temporary slope stability.
- (d) High-quality samples of DBC can be obtained using triple-tube rotary techniques with polymer flush or block sampling where access permits.
- (e) For such high-quality samples, triaxial CIUC or CAUC tests on these high-quality cores give similar  $s_u$  values to simple UU tests.
- (f) DBC possesses a high degree of anisotropy of undrained shear strength.
- (g) DBC is considerably stiffer and stronger than other tills documented in the literature.
- (h) Inexpensive MASW surface wave studies can give reliable in situ values of shear wave velocity (and thus small-strain stiffness). These data, combined with those from triaxial tests, which include sample-mounted transducers, can give the full stiffness–strain response of the material for design purposes.
- (i) DBC appears to have a curved effective stress failure envelope with negligible effective cohesion.
- (j) Comparison of field (horizontal) permeability values and laboratory (vertical) values suggests some degree of permeability anisotropy, with the horizontal permeability being greater than the vertical permeability.
- (k) The material may exhibit some degree of anisotropy of undrained shear strength.

Owing to the complicated depositional process involved in situ, horizontal stress in DBC is poorly understood. Similarly anisotropy of stiffness has not been fully explored. Both of these parameters have important implications for future work in Dublin, which will involve deep excavations (e.g. the

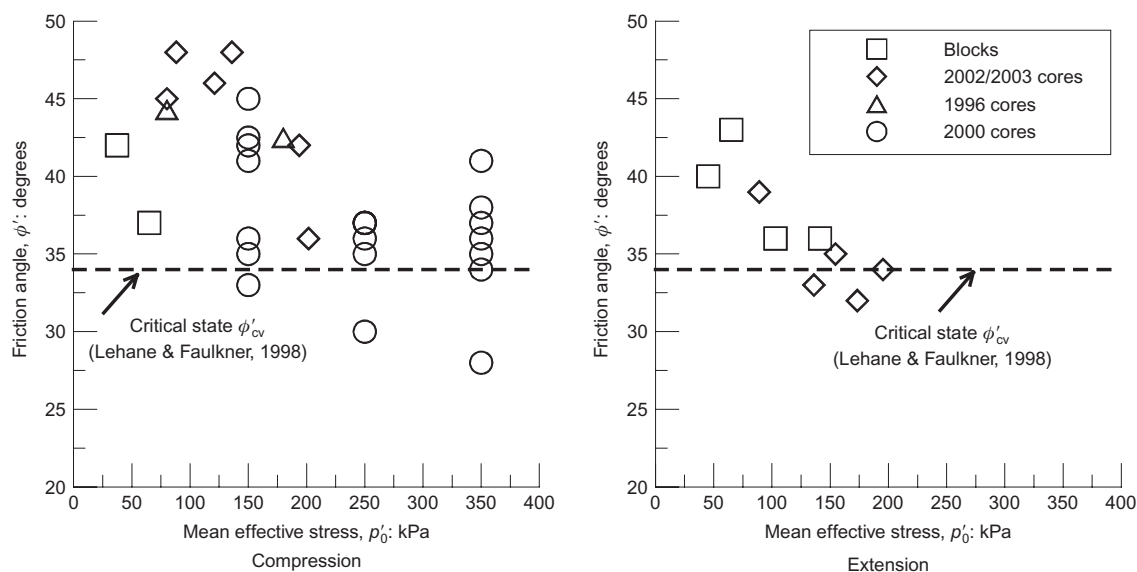


Fig. 15. Effective peak friction angle

central Dublin section of the Metro project). Work should be carried out in the laboratory using recently developed techniques such as lateral stress oedometers and bender element testing with multiple shear wave directions. Future field investigations should include hydraulic fracture testing and various complementary geophysical techniques.

#### ACKNOWLEDGEMENTS

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#### NOTATION

AR	sampler area ratio = $(A_e - A_c)/A_c$ , where $A_e$ and $A_c$ are external and internal areas of cutting shoe
$c'$	effective cohesion
$C_c$	compression index
$C_s$	swelling index
$c_v$	coefficient of consolidation
$e$	void ratio
$\Delta e/e_0$	normalised void ratio change during consolidation
$E_u$	undrained secant Young's modulus
$f_i$	$s_u/N_{avg}$
$G_{max}$	small-strain shear modulus
$k$	coefficient of permeability (h and v correspond to horizontal and vertical)
$K_0$	coefficient of earth pressure at rest = $\sigma'_{h0}/\sigma'_{v0}$
$M$	constrained modulus
$m$	modulus number (i.e. slope of normally consolidated part of $M-\sigma'_v$ curve)
$m_v$	coefficient of unit volume change
$N$	standard penetration test value
$p'$	mean effective stress = $(\sigma'_a + 2\sigma'_r)/3$
$p'_0$	in situ mean effective stress = $(\sigma'_{v0} + 2\sigma'_{h0})/3$
$p'_c$	preconsolidation stress
$s'$	mean effective stress = $(\sigma'_a + \sigma'_r)/2$
$s_u$	undrained shear strength
$t'$	shear stress = $(\sigma'_a - \sigma'_r)/2$
$u_r$	initial sample suction (or residual effective stress)
$\Delta V/V_0$	normalised volume change during consolidation
$\phi'$	effective friction angle
$\phi'_{cv}$	critical state friction angle
$\phi'_{res}$	residual (high strain) friction angle
$\kappa$	slope of swelling line
$\lambda$	slope of isotropic compression line
$\sigma'_a$	axial effective stress
$\sigma'_r$	radial effective stress
$\sigma'_{h0}$	in situ horizontal effective stress
$\sigma'_{v0}$	in situ vertical effective stress

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