



<b>Title</b>	Retaining wall behaviour in Dublin's estuarine deposits, Ireland
<b>Authors(s)</b>	Long, Michael (Michael M.), Menkiti, Christopher O., Skipper, Jacqueline, Brangan, Carl, Looby, Michael
<b>Publication date</b>	2012-01
<b>Publication information</b>	Long, Michael (Michael M.), Christopher O. Menkiti, Jacqueline Skipper, Carl Brangan, and Michael Looby. "Retaining Wall Behaviour in Dublin's Estuarine Deposits, Ireland." Institution of Civil Engineers, January 2012. <a href="https://doi.org/10.1680/geng.10.00037">https://doi.org/10.1680/geng.10.00037</a> .
<b>Publisher</b>	Institution of Civil Engineers
<b>Item record/more information</b>	<a href="http://hdl.handle.net/10197/4053">http://hdl.handle.net/10197/4053</a>
<b>Publisher's statement</b>	Permission is granted by ICE Publishing to print one copy for personal use. Any other use of these PDF files is subject to reprint fees.
<b>Publisher's version (DOI)</b>	<a href="https://doi.org/10.1680/geng.10.00037">10.1680/geng.10.00037</a>

Downloaded 2026-05-01 23:34:48

The UCD community has made this article openly available. Please share how this access benefits you. Your story matters! (@ucd\_oa)



© Some rights reserved. For more information

# Retaining wall behaviour in Dublin's estuarine deposits, Ireland

**1 Michael Long** MEngSc, PhD, CEng, MICE, MIEI  
Senior Lecturer, University College Dublin, Ireland

**2 Christopher Menkiti** MSc, PhD  
Director, Geotechnical Consulting Group, London, UK

**3 Jacqueline Skipper** DIC, FGS  
Senior Geologist, Geotechnical Consulting Group, London, UK

**4 Carl Brangan** PhD  
Geotechnical Engineer, Applied Ground Engineering Consultants  
(AGEC) Ltd, Bagenalstown, County Carlow, Ireland

**5 Michael Looby** MSc  
Director, Byrne Looby Partners, Dublin, Ireland



Practising engineers in the Dublin, Ireland, area have much experience in dealing with the boulder clay which underlies much of the city. However, significant deposits of estuarine soils, some of them soft, exist along the east side of the city and in particular in Dublin docklands. Some construction difficulties have previously been encountered in these materials and significant developments, including a large tunnel project, are planned in the area overlying these deposits. Few published data exist on retaining wall schemes in the deposits. Data from nine sites, including three detailed case histories, are presented which confirm that construction of deep propped excavations and cantilever walls up to 7.5 m are feasible in these deposits and can perform well. A key issue is the soil that is located at excavation level and competent deposits here are essential to prevent large displacements or possible instability. The resulting movement will also be sensitive to the overall system stiffness. There seems to be scope for more efficient future design, including more use of cantilever walls, particularly for temporary works purposes. Beam-on-spring type computer analyses tend to give conservative results for these deposits and more sophisticated finite-element analyses may be warranted for future schemes.

## 1. Introduction

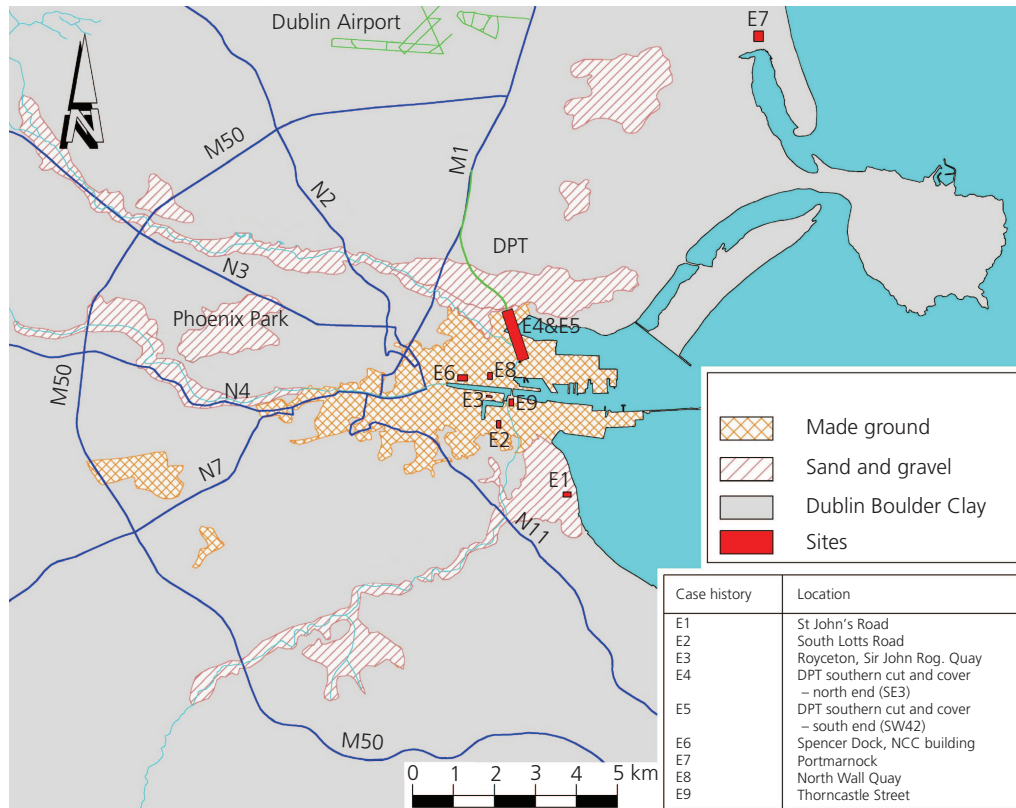
From a geotechnical engineer's viewpoint Dublin, Ireland is well known for its competent glacial lodgement till (Dublin Boulder Clay). Within this stratum, deep excavations are relatively easy to execute, little groundwater is encountered and lateral wall and associated ground movements are modest (Long *et al.*, 2012; Looby and Long, 2007). Similarly, steep slopes can be excavated in the material and can stand open and unsupported for some time (Long *et al.*, 2003; Menkiti *et al.*, 2004). However, significant deposits of soft estuarine soils exist along the east side of the city and in particular in Dublin docklands, see Figure 1(a). Figure 1(b) shows the typical stratigraphic juxtaposition of the estuarine deposits relative to the other geological formations. Much of the Dublin docklands area has been reclaimed from the sea in relatively recent times. Significant developments are planned in the area overlying these deposits especially around Dublin Port. Most importantly, the Dublin Interconnector rail project, which will link the two main railway stations at Connolly and Houston and the InterCity network will encounter these

materials. To date, little information has been published on the geotechnical properties of the deposits, and the main purpose of the present paper is to provide guidance for engineers designing and constructing future schemes in this part of Dublin and in similar materials elsewhere.

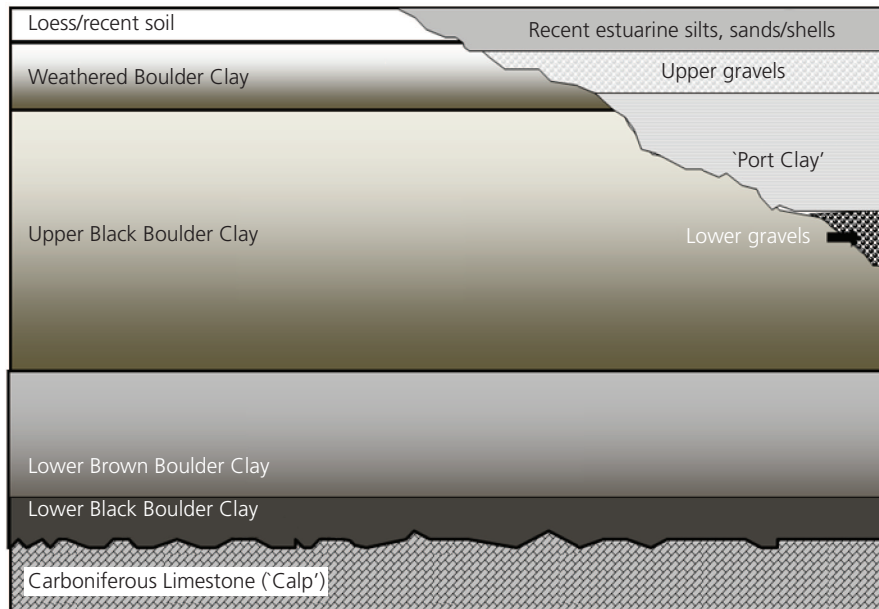
The paper first briefly summarises the background geology and ground conditions in the area and the design methods currently used in practice. The behaviour of nine embedded retaining walls in these materials will be summarised. Subsequently, detailed case histories will be outlined for three sites: two with propped walls and one with a cantilever. Finally some guidance will be given for design and construction of future similar works.

## 2. Background geology

Useful summaries of the ground conditions in the Dublin area are given by Farrell and Wall (1990) and Skipper *et al.* (2005). Recently Kearon (2009) has studied the relatively complex conditions in the Dublin docklands area. Perhaps the first



(a)



(b)

**Figure 1.** (a) Outline geology of area and site locations; (b) soil stratigraphy relative to other geological formations

comprehensive study of the ground conditions in the area was by Naylor (1965). His work involved an examination of 161 samples which were obtained from holes bored in an east–west direction along the North Quays (see E8, Figure 1(a)). A summary of the

succession of sediments he described is given in Table 1. From the fauna in the samples he suggested that most of the sediments on the succession were coastal or shallow marine in origin, with the exception being the laminated clay (i.e. Port Clay, layer 5).

Layer no.	Naylor (1965)	Skipper <i>et al.</i> (2005)
1	Black and grey mud	Recent estuarine silts
2	Mud and shelly sand with occasional pebbles and pebble beds	
3	Mud and sand with some pebbles and shells	(report that this layer has a fully marine modern fauna)
4	Coarse gravels with sand, shells absent	Upper gravels
5	Mid-grey silty, often laminated clay	'Port' Clay
6	Pebbles with cobbles and sand	Lower gravels
7	Stiff to hard gravelly or cobbly sandy clay with occasional sand or gravel lenses	Dublin Boulder Clay

**Table 1.** Summary of ground conditions in area

Skipper *et al.* (2005) summarised the ground conditions at the southern end of the Dublin Port Tunnel, near Fairview Park (see E4 and E5, Figure 1(a)), as detailed in Table 1. The sequence is similar to that of Naylor except the upper three layers of Naylor's model have been taken as a single layer by Skipper *et al.* The sequence of strata proposed by Skipper *et al.* (2005) has been utilised here and a brief description of each stratum follows. For further details, including some engineering properties and a discussion on how the thickness of each layer varies across the area, the reader is referred to Kearon (2009).

Overlying the strata described in Table 1 is made ground, which can be seen in virtually every borehole in the area. This may contain brick, rubble, ash, plastic, glass and ceramics in a mixed clay and gravel matrix. At least three different layers of made ground may be observed: Victorian, municipal (including hospital) waste from the mid twentieth century and dredged silts and sands used for land reclamation.

The recent estuarine deposits are usually less than 2 m thick and generally comprise soft grey organic, occasionally sandy, clayey silt with abundant marine/estuarine shells. According to Kearon (2009) the materials are very variable but have standard penetration test (SPT) *N* values in the range 1–20 with an average of about 10.

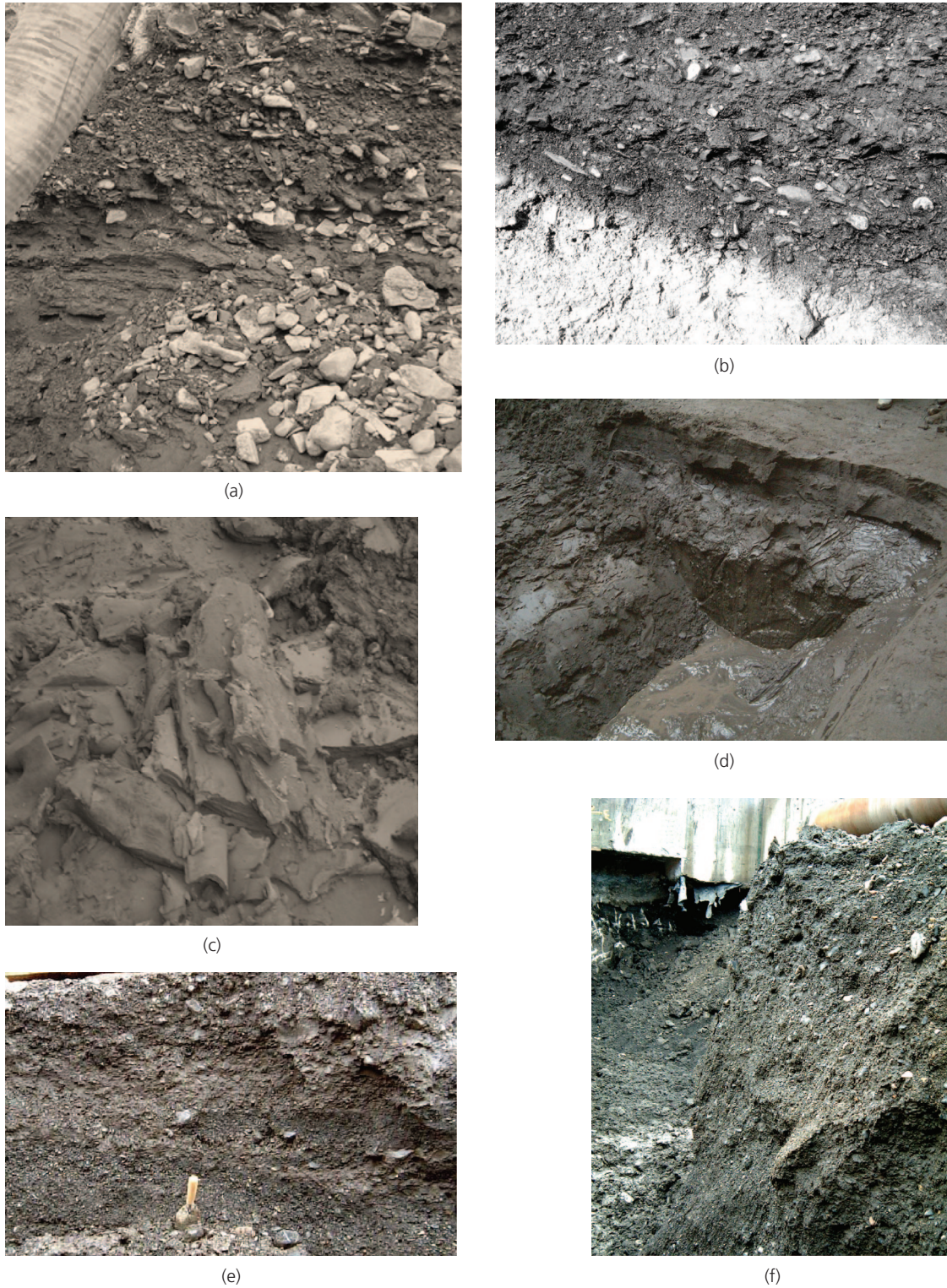
The upper gravels (Figure 2(a)) are a light to mid-grey sand to coarse gravel and cobbles. The material is poorly sorted and rounded, with angular and platy gravel clasts (Figure 2(b)) and appears to have been deposited in up to four fining-upwards cycles, each up to 2 m thick, in a fast-flowing river. These materials act as an aquifer in connection with adjacent surface water rivers. They are very permeable and drain quickly on exposure. SPT *N* values range between 20 and 60 with an average of about 40. Small exposures of the material up to 2 m thick have been seen to stand vertically, albeit precariously, for periods of several weeks.

Beneath the upper gravels is a firm to stiff slightly clayey silt with thinly interlaminated fine sand (Figure 2(a)). The material is

known locally and in historical literature as the 'Port Clay'. Kearon (2009) points out that the stratum is actually composed of three distinct layers: a lower stiff laminated clay, a middle medium dense sand and an upper firm silt. However, he points out that all three subdivisions are frequently not encountered. It is extensively burrowed, weathered and oxidised at the surface and has a light red/brown horizon up to 1 m thick, with the upper 5 cm being frequently dark orange.

The geological origin of the materials has been the subject of discussion and geologists have argued that they could be either marine deposits, which were subsequently overlain by ice, or glacial outwash deposits (Farrell and Wall, 1990). However, in the opinion of the present authors the laminated clays are estuarine in origin and are consistent with being deposited in a large protected estuary. The sand units are also consistent with this. Pollen of Younger Dryas age has been obtained from these sediments. The Younger Dryas was an extreme cold stage event at approximately 12 900 to 11 600 years before present, which was followed by rapid warming over approximately 50 years (personal communication 2005, Professor Pete Coxon, Trinity College, Dublin).

From an engineering point of view the material can be very variable. It can vary from fine sand that drains relatively rapidly, to a more plastic silt/clay that, when excavated, breaks up into layers that can be peeled apart along sandy or silty laminations (Figure 2(c)). As the material becomes even more plastic, the laminations become 'very sticky' and it becomes difficult or impossible to peel along them. Undrained, the more layered material (that is silt or fine sand) can display strong thixotropic behaviour (Figure 2(d)) and can liquefy or run on strong vibration. When excavated into, however, drainage can occur rapidly by way of the thin sandy lenses, horizons or laminations, and this process can quickly stabilise the surrounding material. Farrell and Wall (1990) described some geotechnical problems which have been encountered with this material. For example, it is difficult to assess the in situ strength from conventional unconsolidated undrained triaxial tests as the sand layers allow the material to take in water and thus cause softening.



**Figure 2.** Photographs of various strata: (a) basal contact between upper gravels and weathered laminated sandy silt (Port Clay); (b) upper gravels with very platy casts; (c) detail showing peel-apart behaviour of grey laminated silt/sand (Port Clay); (d) thixotropic behaviour of Port Clay; (e), (f) lower gravels below Port Clay

Isotropically consolidated tests give more representative results. Little *et al.* (1987) described the difficulties encountered during the construction of Ringsend pumping station caused by excavation base heave failure arising from high pore pressures in the sandy layers of the Port Clay. However, experience from the Dublin Port Tunnel indicates that the material can respond well to vacuum pumping, with some preservation of suctions and thus significantly enhanced stability. As the material can be variable, groundwater control solutions need to be specific to the local conditions. SPT tests also need to be treated with caution as they can be unreliable if water levels are not balanced in the borehole and borehole base instability is induced. Kearon (2009) reports a wide scatter in  $N$  values between 0 and 60 with an average over 20.

The lower gravels are typically a dense to very dense medium to coarse sandy gravel and cobbles (Figure 2(f)), the clasts being sub-angular to well rounded. The stratum appears to be cross bedded and to act as an aquifer, but does not conduct as much water as the upper gravels. These deposits can be very variable and may not always stand as a near vertical face when drained, as shown on Figure 2(f).

Beneath the lower gravels the Dublin Black Boulder Clay is encountered. This is a very competent glacial lodgement till, characterised by high strength and stiffness and low water content and permeability. Its properties have previously been described in detail by Long and Menkiti (2007a, 2007b). Bedrock in the area comprises strong to very strong locally moderately strong, thickly bedded argillaceous Carboniferous limestone, known locally as 'Calp'.

### 3. Design methods for retaining walls

Current geotechnical design procedures for retaining walls in Dublin estuarine deposits involve hand calculations or relatively simple computer analyses. To the authors' knowledge more sophisticated techniques, such as the finite-element method, have not been used.

For ultimate limit state, the general guidelines of Gaba *et al.* (2003) or BS 8002 (BSI, 1994), have been used to determine the required retaining wall toe penetration. Safety is typically introduced by applying a partial safety factor of 1.3 to the tangent of the effective constant volume friction angle ( $\phi'_{cv}$ ). Calculations are often performed by hand, using conventional Rankine active and passive earth pressure theory, or with the aid of a relatively simple piece of computer software such as Oasys–Stawal<sup>®</sup>, ReWard<sup>®</sup> or Support-It<sup>®</sup>.

Usually a beam-on-spring computer program, such as Oasys–Frew<sup>®</sup>, Wallap<sup>®</sup> or ReWard<sup>®</sup>, is used to determine wall bending moments, shears, prop/waler forces and serviceability limit state lateral wall movements. As beam-on-spring models are only able to determine wall movements and not the associated ground movements, these are often predicted using empirically based

methods, which were developed in the UK and USA (Clough and O'Rourke, 1990). Possible building damage is then assessed by comparing predicted building differential settlement, angular rotations or similar with empirically based tolerable limits, for example using Gaba *et al.* (2003) or Burland *et al.* (1977). Horizontal strains are sometimes accounted for by fitting a Gaussian curve to the predicted settlements and then adopting the horizontal strains from a tunnel that would generate such a Gaussian fit. 'Most probable' soil parameters considered representative of the in situ ground conditions would mostly be used for these serviceability limit state calculations, especially if the wall was intended for temporary work use. Ideally these parameters would be obtained from the back-analysis of similar walls. However, the risks involved in using these 'most probable' parameters need to be carefully assessed on a case-by-case basis. Some typical input parameters used in the analyses will be given below in the case histories (Section 5).

For the ultimate limit state calculations an onerous groundwater level (GWL) would be assumed in the calculations as suggested by Gaba *et al.* (2003) or BS 8002 (BSI, 1994). In the case of retaining walls in Dublin estuarine deposits this would normally correspond to an extreme high tide level. For serviceability limit state calculations the GWL would normally be taken as the highest level measured during the ground investigation or typically a mean tide level. Following excavation a simple steady-state seepage condition would be assumed to determine the altered pore pressures.

### 4. Database of retaining walls in the estuarine deposits

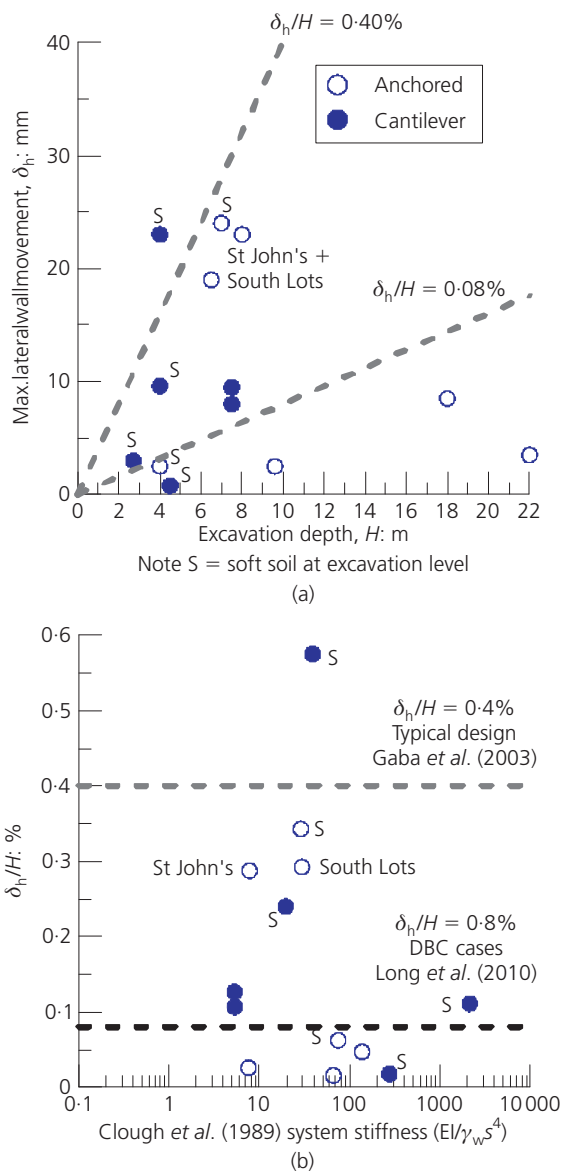
A summary of information pertaining to nine retaining walls in the Dublin estuarine deposits is given in Table 2 and the locations of the sites are shown in Figure 1(a). At four of the sites data are available for both propped and cantilever walls. Maximum excavation depth ranges between 4 m and 18 m for the propped walls and 2.7 m and 7.5 m for the cantilever cases. A plot of maximum measured lateral movement ( $\delta_h$ ) against retained height ( $H$ ) is shown in Figure 3(a). It can be seen that the data broadly fall into two groups. Many of the data points show  $\delta_h$  values less than 10 mm, that is very good wall behaviour. However, there is a group of points where more significant deflections of between 20 mm and 25 mm occurred. Long (2001) reviewed a large database of worldwide retaining wall behaviour and showed that the presence or otherwise of soft soils at excavation level was a key factor in the subsequent behaviour of the walls. This is consistent with these data in that soft deposits were present at excavation level at the two sites where greatest movements were recorded.

However, in order to obtain a complete understanding of the data, it is also necessary to take into account the retaining wall type and its stiffness or the prop/anchor configuration. In order to attempt to include these factors, the data are replotted on Figure 3(b) in the normalised form of  $\delta_h/H$  against Clough *et al.* (1989) system stiffness, which is defined as

Case history	Location	Ground conditions; Soil at dredge level	Soil strength, $s_u$ : kPa	H: m	h: m	B: m
E1	St Johns Road	Fill, alluvium, medium dense gravel and dense gravel	N = 40–60	8	1	15
E2	South Lotts Road	Fill, alluvium, dense gravel, DBC	N = 50 to ref	6.5	3.5	25
E3	Royceton, John Rog. Quay	Fill, loose gravel, soft silt, medium dense gravel and DBC	SPT N = 2–19	7	12.5	90
E4-P	DPT Southern C&C – North End (SE3) – propped	Made ground, soft silt, dense sand and gravel, DBC, Lmst	SPT N = 5–100	22	4.5	25
E5-P	DPT Southern C&C – South End (SW42) – propped	Made ground, soft silt, loose sand and gravel, port clay, dense sand and gravel, DBC	SPT N = 1–100	18	8.2	25
E6-P	Spencer Dock, NCC building – single anchor	Fill, soft silt, medium dense gravel and DBC		9.6	7.5	75
E7-P	Portmarnock – anchor	Soft sandy silt, DBC (1.5 m below)	N = 2–5	4	6	40
E8	North Wall Quay	Fill, soft silt, dense gravel and DBC	N = 35+	7.5	4.5	125
E9	Thorncastle St	Fill, alluvium, dense gravel, DBC	N approx. 10	4	12	30
E4-C	DPT Southern C&C – North End (SE3) – cantilever	Made ground, soft silt, dense sand and gravel, DBC, Lmst	N = 5–100	4.5	4.5	25
E5-C	DPT Southern C&C – South End (SW42) – cantilever	Made ground, soft silt, loose sand and gravel, port clay, dense sand and gravel, DBC	SPT N = 1–100	2.7	8.2	25
E6-C	Spencer Dock, NCC building – cantilever	Fill, soft silt, medium dense gravel and DBC		7.5	7.5	75
E7-C	Portmarnock – cantilever	Soft sandy silt, DBC (1.5 m below)	N = 2–6	4	6	40
Support configuration	s*: m	Wall type	Pile dia./spacing/length: m	Ei: kN/m <sup>2</sup>	$\delta_h$ : mm	Reference
Single prop	7.5	Contiguous	0.6/hard at 0.75/12.5	245 450	23	Brangan (2007)
Single prop	6	Secant	0.6/hard at 0.5/12.5	381 700	19	Brangan (2007)
Single anchor	7	Secant	0.9/hard at 1.4/14	690 135	24	BLP files – IC2
Two props	9	Diaphragm	1.2 thick/26.5 long	4 320 000	3.5	Curtis and Doran (2003)
Two props	7.5	Diaphragm	1.2 thick/24 long	4 320 000	8.5	Curtis and Doran (2003)
Single anchor	9.6	Secant	0.9/hard at 1.5/19	644 126	2.5	Looby and Long (2007) – Incls 5
Single anchor	4	Secant	0.6/hard at 1.0/8.5	191 000	2.5	BLP Files – IC 2 to 4
Cantilever	10.5	Secant	0.9/hard at 1.5/15.5	644 126	8	BLP files
Cantilever	5.6	Secant	0.6/hard at 0.5/13	381 700	23	Brangan (2007)
Cantilever	6.3	Diaphragm	1.2 thick/26.5 long	4 320 000	0.8	Curtis and Doran (2003)
Cantilever	3.78	Diaphragm	1.2 thick/24 long	4 320 000	3	Curtis and Doran (2003)
Cantilever	10.5	Secant	0.9/hard at 1.5/15.25	644 126	9.5	Looby and Long (2007) – Incls 4
Cantilever	5.6	Secant	0.6/hard at 1.0/7	191 000	9.6	BLP Files – IC 1

H = excavation depth, h = thickness of soft material, B = excavation width, s = support spacing = 1.4H for cantilever walls, E = Young's modulus, I = moment of inertia,  $\delta_h$  = maximum lateral wall movement

Table 2. Dublin estuarine deposits case histories



**Figure 3.** Database for retaining walls in Dublin estuarine deposits: (a) maximum movement plotted against excavation depth and (b) normalised movement plotted against Clough *et al.* (1989) system stiffness

$$1. \text{ System stiffness} = \frac{EI}{\gamma_w s^4}$$

where  $EI$  is the wall stiffness;  $\gamma_w$  is the unit weight of water (required to make the expression unit-less); and  $s$  is the support spacing (taken to be  $1.4H$  for cantilever walls).

Again the data fall into two groups. In many cases the normalised movements are small and are not dissimilar to the average  $\delta_h/H$  value of  $0.08\%$  for Dublin Boulder Clay sites from Long *et al.*

(2012). The two cantilever sites with system stiffness of about 5, and that plot with  $\delta_h/H \approx 0.1\%$ , are dominated by the relatively competent upper/lower gravels; this accounts for why they are so stiff and behave in a similar manner to the Dublin Boulder Clay.

Three of the group of five points showing larger normalised movement correspond to sites where soft soils were encountered at excavation level. At the other two sites (South Lotts and St Johns Road) the retaining wall system stiffness was relatively low. Also shown on Figure 3 are lines representing normalised movement of  $0.4\%$ , which represents a typical conservative design value as recommended by Gaba *et al.* (2003) in Ciria report C580. The data from the sites with soft soils at excavation level or where the system stiffness is low correspond relatively well with this relationship.

### 5. Case histories

In order to explore the behaviour of these walls more carefully, three representative case histories will be presented as follows

- the Dublin Port Tunnel (DPT) southern cut and cover (south end SW42), which is the deepest excavation carried out to date in these deposits and for which detailed measurements of the prop loads are available
- the 7.5 m cantilever retaining wall at North Wall Quay
- the 8 m propped wall at St John's Road, where the retaining wall system stiffness was relatively low.

#### 5.1 Results for deep excavation – Dublin Port Tunnel (DPT) southern cut and cover

##### 5.1.1 Scheme

The DPT central section comprises twin bored tunnels driven by tunnel boring machines launched from a central large-diameter shaft. Shallower sections of the tunnels at either end, where the invert level is less than about 25 m below ground level, were constructed using cut-and-cover methods. At the southern section of the works an approximate 400 m length of the tunnel was constructed in estuarine deposits within propped diaphragm walls. Data were obtained from four inclinometers along this section, two at the northern end (site E4 on Table 1 and Figure 1(a)) and two at the southern end (site E5 on Table 1 and Figure 1(a)). As the estuarine deposits were thicker at the southern end, and also as measurements of prop load were taken here, the results for this location (known as SE42/SW42 at chainage 4820 m) will be presented.

At this location the excavation depth was 18 m (ground level was about +8.1 m OD (above ordnance datum)) and the 1.2 m thick diaphragm wall was supported by two tubular steel props, see Figures 4 and 5. The upper prop was 1220 mm outside diameter (o.d.) with 14.2 mm wall thickness and the lower prop was 1620 mm  $\times$  19.0 mm. These props spanned approximately 25 m across the excavation. Props were connected directly to the diaphragm wall by way of reaction pads. This gave the advantage of having no waling beam. However, it meant prop positions were



(a)



(b)

**Figure 4.** DPT southern cut and cover section: (a) overview facing south with base of excavation at approximately 13.5 m in upper Black Boulder Clay and (b) detail at prop end

fixed and there was the risk of accidental prop removal which was mitigated by additional fixing chains. The props were not preloaded.

The diaphragm wall was constructed in 7.1 m panel lengths using a rope-suspended grab and a hydrofraise to construct the rock socket. The panel excavation was bentonite supported and excavation of each panel was typically carried out over a period of 2–5 days. Achieved verticality was significantly better than the 1% tolerance specified. A proprietary water stop sealing strip was also used. Typically T20 to T32 bars were used to provide vertical reinforcement, with T16 to T20 shear links. Additional reinforcement was required at prop levels to accommodate punching shear. Gaps were provided in the shear links to accommodate two 320 mm tremie pipes per panel. Panels were typically 24–27 m long embedded in the upper weathered limestone/mudstone. Overbreak during concreting of each panel was in the range 3–8%.

### 5.1.2 Ground conditions

Ground conditions were typical for the area, as shown on Figures 4(a) and 5(a). At the monitored cross-section (Figure 5(a)) there is a thick layer of made ground, overlying alluvium, upper gravels, Port Clay and lower gravels. At the retaining wall toe a thin layer of Dublin Boulder Clay overlies limestone bedrock at a depth of about 22 m. Of the 18 m retained height almost 12 m comprise estuarine deposits. SPT  $N$  values are shown in Figure 5(b) and confirm the made ground to be of loose consistency, with the alluvium and upper gravels being medium dense to dense or stiff. Groundwater was typically encountered towards the base of the made ground at a depth of about 7.5 m (0.6 m OD), with maximum measured values about +2 m OD.

Diaphragm walling through this stratigraphy was very successful and the bentonite slurry was sufficient to stabilise the excavation, even through the gravels with very little overbreak. Very occasional loss of bentonite slurry occurred in the made ground south of the railway crossing, where the made ground was largely municipal waste. The solution that was successfully adopted was to use cement–bentonite slurry to support the excavation in the poor ground. The slurry was left overnight to set in the surrounding ground, and the next day the panel was excavated. If necessary, the process was repeated.

### 5.1.3 Movement monitoring

Lateral wall movements recorded by inclinometer are shown on Figure 5(c) and show maximum movements of about 3.3 mm for the cantilever stage and 8.5 mm for the final excavation stage (Curtis and Doran, 2003).

### 5.1.4 Prop loads

At the DPT southern cut and cover section 106 steel props were used to support the diaphragm walls. These were mostly of 1550 mm outside diameter (o.d.) with a 10.5 mm wall thickness and had previously been used in the offshore industry. Up to three levels were used at the north end of this section, at Fairview, where the cut and cover section meets the bored tunnel and excavation depth is 24 m.

Vibrating wire strain gauges, type Geokon V4–4101, were installed in three upper props for the railway crossing reception shaft at a location just south of SE42/SW42. The shaft is 37 m by 18 m in plan, and ground conditions around it were very similar to that described in Figures 4(a) and 5, except that the diaphragm wall was 30 m deep with one layer of high-level propping in order to provide construction access beneath. Gauges were attached at the 3, 6, 9 and 12 o'clock positions at both the centre and end of the prop. Since the strain gauges were installed in a box excavation, three-dimensional effects would lead to prop loads that could be lower than those in a two-dimensional excavation. The results should be utilised with this in mind.

It is well known that the effect of temperature on prop loads can be very significant, see for example, Batten *et al.* (1999) and

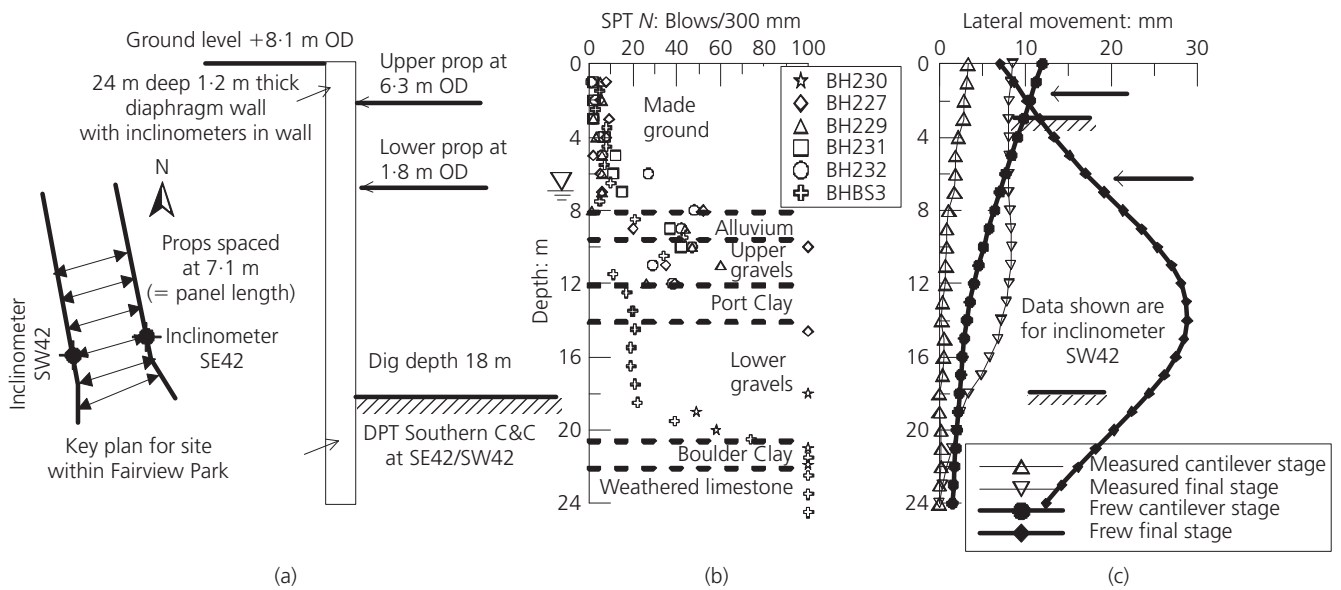


Figure 5. DPT southern cut and cover section: (a) retaining wall layout; (b) ground conditions; (c) lateral wall movements

Twine and Roscoe (1999), and hence thermistors were fitted with the strain gauges so that temperature correction of loads could be made. A plot of prop load against temperature for the period 22 August 2002 to 20 November 2002 is shown in Figure 6(a). During this time construction activity was minimal and the excavation depth was increased by up to 1 m only. Axial load increases approximately linearly with temperature. For a temperature increase from 8.5°C to 25.5°C, the corresponding increase in prop load is from 750 kN to 1500 kN. Using a coefficient of thermal expansion for the prop of  $11.3 \times 10^{-6}/^{\circ}\text{C}$ , the theoretical load increase should have been 960 kN. This suggests that end restraint has an effectiveness of 78%, which is significantly higher than the range of 40–60% suggested by Twine and Roscoe (1999).

Batten *et al.* (1999) suggest measured prop loads ( $P_M$ ) can be corrected for temperature effects ( $P_T$ ) as follows

$$2. \quad P_T = P_M - (T_M - T_D)f$$

where  $T_M - T_D$  is the average temperature rise above the datum reading and  $f = dP/dT$  is a factor determined from the load–temperature relationship.

Temperature-corrected prop loads are shown on Figure 6(b). The reception shaft diaphragm wall panels upon which the props bear were 6.5 m long and this dimension can be used to convert the data in Figure 6(b) to load/m excavation length. Most of the increase in prop load occurs during the bulk excavation up until 22 August 2002. After this time there is only a gradual increase in load corresponding to minor excavation activities until

20 November 2002 when soil was removed from behind the wall. Maximum prop loads are of the order of 1800 kN, which is significantly less than the prop capacity.

5.1.5 Measured compared with predicted performance

Measured lateral wall movements are compared to those predicted by Frew® in Figure 5(c) as an example of a typical beam-on-spring calculation. A summary of the input parameters used, for serviceability limit state calculations, is given in Table 3. These parameters are intended to be ‘most probable’ parameters and reasonably representative of in situ conditions. The GWL was assumed to be at 7.5 m depth, as measured during the ground investigation. It can be seen that although Frew® predicts the correct displacement profile, it significantly overpredicts the magnitude of the movement in both the cantilever mode and for the final stage excavation. In fact in the cantilever stage the predicted movements are already greater than the maximum measured at full excavation depth. In beam-on-spring computer programs such as Frew®, the pore pressures need to be specified by the user and here a steady-state seepage pore water pressure regime was input for the granular layers (the alluvium and Port Clay being assumed undrained with undrained strengths,  $s_u = 30$  kPa and 60 kPa respectively). The true groundwater regime is likely to have been much more complicated owing to the combined effects of reduction due to excavation and subsequent reconsolidation. It is also appears from the displacement profile that the single ‘most probable’ input stiffness values are too low and that in reality the materials are highly non-linear with high stiffness at low strains.

Frew® predicts prop loads of the order of 1300 kN (i.e. 200 kN/m for the 6.5 m panel length). This value is very similar to that

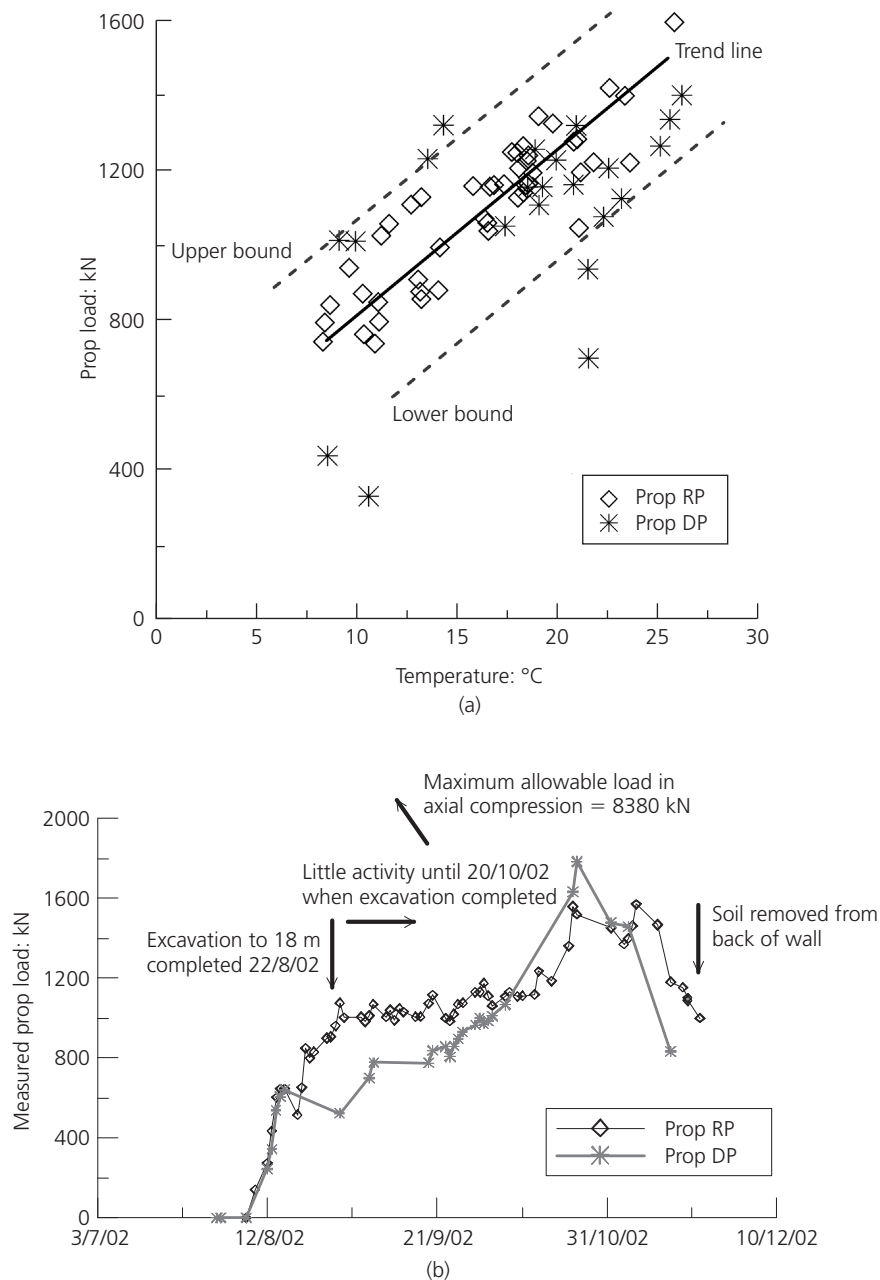


Figure 6. DPT southern cut and cover section: (a) temperature effects on props and (b) temperature-corrected prop loads

measured, suggesting again that the overpredicted displacements may be due to incorrect stiffness (which would govern predicted movements) rather than strength assumptions (which would determine forces).

#### 5.1.6 Comment

Despite the very significant depth of the excavation in relatively poor ground conditions, the measured movements were very modest in the two-dimensional cut. Bending moments were also

likely to have been lower than expected and it would seem there is scope for more efficient design. The reception shaft box excavation is an example of one such design, with a single prop supporting an 18 m deep excavation. This good performance in the estuarine deposits is likely to have been attributable to a combination of competent soils at excavation level and high retaining wall system stiffness. It seems that relatively simple beam-on-spring type computer codes are unable to capture the complexity of the real soil behaviour for situations such as this.

Material	$E$ : MPa	$\gamma$ : kN/m <sup>3</sup>	$K_0$	$K_a$	$K_p$	$\phi'$ : deg	$s_u$ : kPa
Made ground	10	18	0.5	0.29	4.0	30	n/a
Alluvium	5	16	0.53	1	1.0	n/a	30
Upper gravels	60	19	0.43	0.24	5.2	35	n/a
Port Clay	60	18	0.5	1	1.0	n/a	60–150
Lower gravels	80	20	0.36	0.17	7.4	40	n/a
Dublin Boulder Clay	100	22.5	1.0	1.0	1.0	n/a	400

$E$  = Young's modulus,  $\gamma$  = unit weight,  $K_0$ ,  $K_a$ ,  $K_p$  = at rest, active and passive earth pressure coefficients,  $\phi'$  = effective friction angle,  $s_u$  = undrained shear strength

**Table 3.** Summary of retaining wall analysis input parameters for serviceability limit state

## 5.2 Results for 7.5 m cantilever wall at North Wall Quay

### 5.2.1 Scheme

Details of the scheme are shown in the photograph of Figure 7 and in Figure 8. The site location is designated E8 on Figure 1(a). A 15.5 m long secant piled retaining wall was used to retain the 7.5 m deep dig. 'Hard' piles were at 1.5 m centres. The 'soft' piles comprised unreinforced lower grade concrete. The piles were constructed using the continuous flight auger (CFA) technique with three rigs, which had 60 kN pull down force and between 180 kN m and 350 kN m torque. About 10 and 20 revolutions per metre respectively were required in the materials above and within the Dublin Boulder Clay. Drilling time, for each pile, was typically 15–25 min, with a further 10 min required for concreting. Occasionally there was a delay during concrete truck changeover. Overall progress of the piling work varied between 10 and 15 piles per day. No issues with auger flighting arose. Overbreak was not significant and never greater than 5–10%. Excavation proceeded as rapidly as allowed by conventional plant. The formation was blinded immediately on exposure and



**Figure 7.** North Wall Quay site – 7.5 m cantilever

the base slab was constructed some 14 weeks after completion of the piling works.

### 5.2.2 Ground conditions

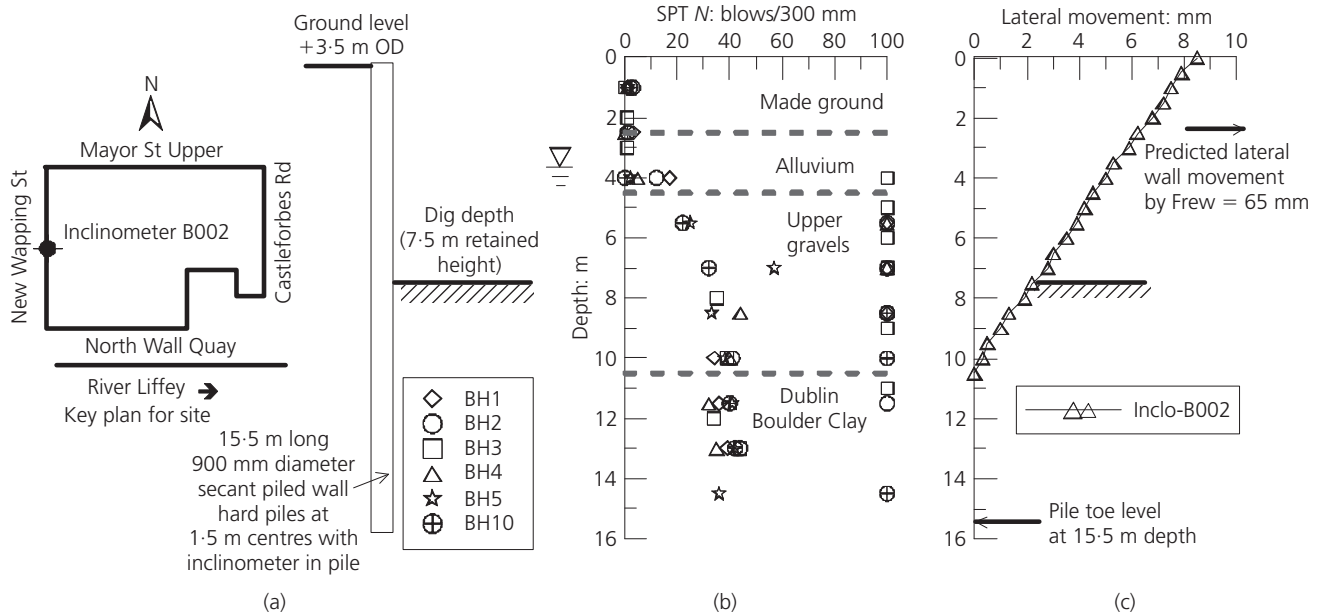
Ground conditions encountered were again typical for the area, as shown on Figure 8(a), with made ground, overlying alluvium, upper gravels and Dublin Boulder Clay. Here the Port Clay and lower gravels are absent. SPT  $N$  values are shown on Figure 8(b) and confirm the made ground and alluvium to be of loose/soft consistency, with the upper gravels and Dublin Boulder Clay being medium dense to dense/stiff to very stiff. There were frequent refusals (denoted by an  $N$  value of 100 on Figure 8(b)) in these two strata. The competent state of these two strata led to the decision to choose a cantilever wall solution. Groundwater was encountered in the upper gravels at a depth of about 3.5 m (0 m OD).

### 5.2.3 Movement monitoring

Lateral wall movements recorded by the inclinometer that showed most movement are shown in Figure 8(c). Maximum movement was about 8.5 mm ( $\delta_h/H = 0.11\%$ ) with the movement profiles being typical for a cantilever wall.

### 5.2.4 Measured compared with predicted performance

Lateral movements predicted by Frew<sup>®</sup> are very high, even with the assumption of 'most probable' soil input parameters and the actual measured GWL. Even if relatively high  $E'$  and  $\phi'$  values of 100 MPa and 40° are used, the resulting deflections are of the order of 65 mm, that is approximately seven times the measured value. The main reason for this is that, owing to the chosen seepage pore water pressures, the vertical effective stresses calculated by Frew<sup>®</sup> in the upper gravel layer on the passive side are very low, resulting in very low passive limit values. All of the nodes in this zone become 'passive' and the computer program then has to transfer load to the lower nodes in the Dublin Boulder Clay resulting in large movements. Although it is possible that some local yielding adjacent to the wall occurred in the field situation, it is unlikely that the entire stratum reached yield



**Figure 8.** North Wall Quay site: (a) retaining wall layout; (b) ground conditions; (c) lateral wall movements

pressure values and thus is able to provide considerable more resistance than calculated by the computer program.

### 5.2.5 Comment

This scheme was successfully completed with relatively modest movements being recorded. Maximum normalised movement ( $\delta_h/H$ ) was similar to the average value of 0.08% for Dublin Boulder Clay sites from Long *et al.* (2012). The main reason for this behaviour was the presence of competent strata at excavation level as the retaining wall system stiffness was relatively low (see Figure 3). Care needs to be taken when using beam-on-spring type computer programs in situations such as this, as the resulting predictions of displacements and bending moments can be very conservative.

## 5.3 Results for 8 m propped wall with low system stiffness at St John's Road

### 5.3.1 Scheme

Details of the scheme are shown in Figures 9 and 10. The site location is designated E1 on Figure 1(a). A 12.5 m long contiguous piled retaining wall was used to retain the 8 m deep dig. Piles were 0.6 m diameter at 0.75 m centres. Piles were constructed using the CFA technique with a rig which had 60 kN pull-down force and 150 kN m torque. Otherwise piling techniques and experience were very similar to that reported above for North Wall Quay. The site was 50 m long by 15 m wide and thus it was decided to restrain the wall using eight 254 mm × 254 mm universal column (UC) cross props. These were located at approximately 6 m centres and were connected to another vertical UC section embedded in a pile and supported at their centre by a bored pile (see Figure 9). Excavation at any one location on site

was completed within about 6 days and the formation was blinded with lean mix concrete immediately on exposure.

### 5.3.2 Ground conditions

Ground conditions encountered were somewhat different from the two sites described above. Geological maps of the area, such as that in Farrell and Wall (1990), show the area to be underlain by a thin layer of 'loam' over river terrace gravels. Investigations at the site revealed a thin layer of made ground overlying a sandy alluvial stratum, overlying upper gravels and then Dublin Boulder Clay, see Figure 10(b). Again the Port Clay and lower gravels are absent. SPT  $N$  values are shown in Figure 10(b) and confirm the upper gravels are of lower consistency than at the above two sites with an average  $N$  of about 28. Groundwater was encountered towards the base of the sandy alluvium at a depth of about 1.5 m.

### 5.3.3 Movement monitoring

Lateral wall movements recorded by the two inclinometers that showed most movement are shown in Figure 10(c) (Brangan, 2007). Maximum movements were about 19 mm and 23 mm ( $\delta_h/H = 0.25\text{--}0.29\%$ ), with the movement profiles being typical for a propped cantilever wall. As the tubes were installed within the piles and given the gradient of movement with depth below excavation level it would seem that total wall movement could have exceeded that shown by the inclinometers. The profile also suggests the props were taking a significant load.

### 5.3.4 Measured compared with predicted performance

Predicted lateral wall displacements from Frew<sup>®</sup>, using the serviceability limit state parameters listed on Table 3, and



Figure 9. St John's Road – 8 m propped wall with low system stiffness

assuming a GWL at 1.5 m, are shown on Figure 10(c). It can be seen that the analysis results are close to the measurements in both profile and magnitude. The difference between the measured and the computed result is due to Frew<sup>®</sup> calculating some movement below the toe of the wall which may also have occurred in the field. If  $E$  of the upper gravels was increased from 60 MPa to 80 MPa then the actual and predicted profiles would be very similar both in shape and magnitude.

### 5.3.5 Comment

Here somewhat larger movements were recorded when compared to the sites referred to above. This was due to a combination of relatively low system stiffness and the medium dense soil at excavation level. Beam-on-spring type models can give reliable predictions in cases such as this. Here, even though the upper gravels on the passive side of the wall reach yield stress in the analysis, the software can redistribute the out-of-balance load to the prop.

## 6. Guidance for future works

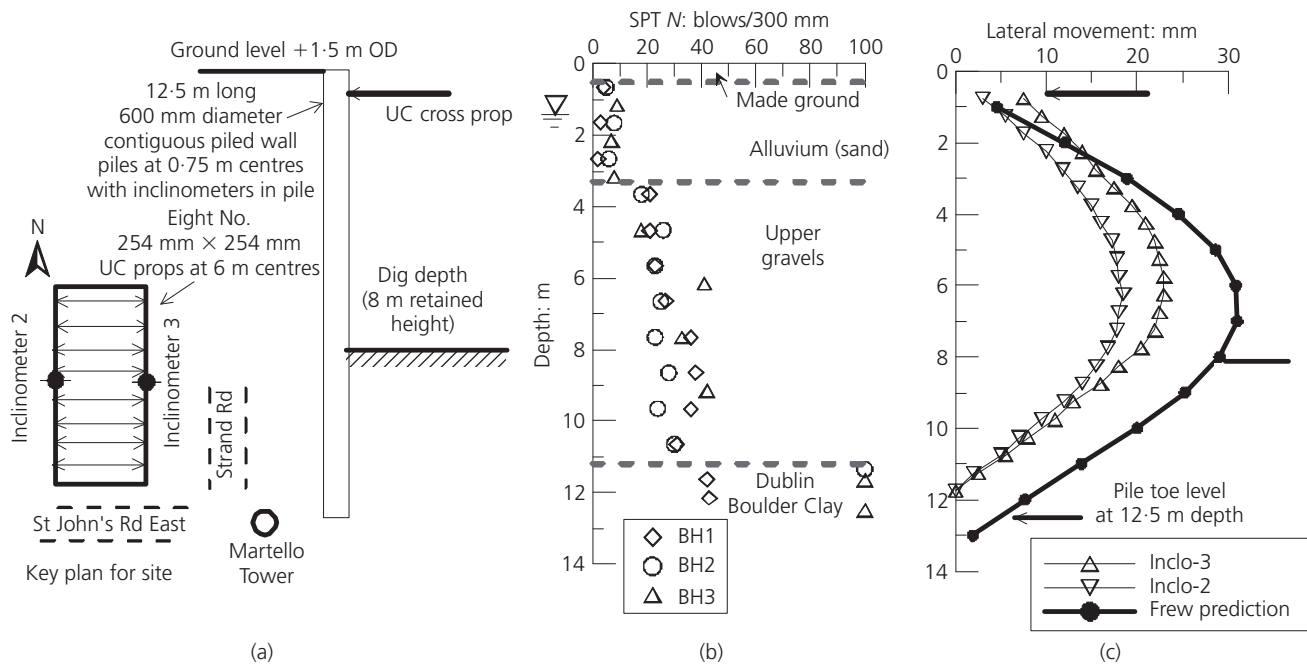
A key issue in the design and construction of future similar works is that the final excavation level should be located within competent deposits below the made ground and alluvium strata or below the Port Clay. Otherwise there is a significant risk of large lateral wall displacements. The data given in Figure 3 will allow a good first estimate to be made of the likely wall movements.

There seems to be scope for more use of cantilever walls in future schemes, particularly for temporary works purposes. Ambitious cantilever wall techniques should only be considered in conjunction with the observational approach. This requires reasonably accurate prediction of movements, agreed trigger/action levels and a contingency plan should actual movements exceed the action limits.

Currently used beam-on-spring type computer analyses for serviceability limit state calculations and determination of prop loads can yield conservative output and may lead to significant over-design. This is particularly the case for the cantilever mode where a free draining material occurs at excavation level. Owing to seepage pore water pressures the computer code will calculate and use very low passive resistance values for entire strata, whereas in the field situation only local yielding adjacent to the wall may occur. Beam-on-spring type computer programs are more suitable for propped walls where the excavation depth is modest. A particular issue with these methods is that they cannot capture important aspects of soil behaviour such as small strain stiffness and consolidation effects. Consideration therefore should be given to use of more sophisticated analysis techniques, such as the finite-element method; where the various strata are modelled fully as continua rather than nodes and where coupled consolidation calculations and non-linear stiffness parameters could be used to capture more realistic material and pore water behaviour.

## 7. Summary and conclusion

- (a) Data have been presented showing the successful completion of nine retaining walls in the complex Dublin estuarine deposits in an area where basements were not considered feasible until relatively recently.
- (b) Overall the behaviours of retaining walls fall into two groups
  - sites where there is stiff soil at excavation level and where the system stiffness is high show small movements not dissimilar to the average  $\delta_n/H$  value of 0.08% for Dublin Boulder Clay sites from Long *et al.* (2012)
  - sites where there is either soft soil at excavation level or where the system stiffness is relatively low; here normalised movement is typically 0.4%, similar to the design value recommended by Gaba *et al.* (2003) in Ciria report C580.
- (c) Beam-on-spring computer analyses may overpredict lateral wall movements in these deposits. Such analyses could result in conservative design and it is clear that the designer's inherent choice of pore pressure is of significant importance.



**Figure 10.** St John's Road: (a) retaining wall layout; (b) ground conditions; (c) lateral wall movements

More sophisticated analyses are warranted to capture more realistic soil and pore water behaviour.

- (d) There would seem to be scope for more efficient design and construction in future schemes, including more use being made of cantilever walls.

## Acknowledgement

Some of the data used in this paper were kindly provided by Mr Tony O'Dowd, PJ Edwards and Company, and his assistance is gratefully acknowledged.

## REFERENCES

- Batten M, Powrie W, Boorman R, Yu H-T and Leiper Q (1999) Use of vibrating wire strain gauges to measure loads in tubular steel props supporting deep retaining walls. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **137(1)**: 3–13.
- Brangan C (2007) *Retaining Walls in Dublin Boulder Clay*. PhD thesis, Civil Engineering Department, University College Dublin, Ireland.
- BSI (1994) BS 8002: Code of practice for earth retaining structures. BSI, London, UK.
- Burland JB, Broms BB and De Mello V (1977) Behaviour of foundations and structures. *Proceedings of the IXth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan*, vol. 2, pp. 195–546.
- Clough GW and O'Rourke TD (1990) Construction induced movements of in situ walls. *Proceedings of the ASCE Conference on Design and Performance of Earth Retaining Structures*, Cornell. ASCE, Reston, VA, USA, Publication No. 25, pp. 439–470.
- Clough GW, Smith EM and Sweeney BP (1989) Movement control of excavation support systems by iterative design. *Proceedings of the ASCE Foundation Engineering: Current Principles and Practices*. ASCE, Reston, VA, USA, vol. 2, pp. 869–884.
- Curtis P and Doran J (2003) *Retaining Wall Behaviour at the Dublin Port Tunnel*. BE Project Civil Engineering Department, University College Dublin, Ireland.
- Farrell E and Wall D (1990) Soils of Dublin. *Transactions of the Institution of Engineers of Ireland* **115**: 78–97.
- Gaba AR, Simpson B, Powrie W and Beadman DR (2003) *Embedded Retaining Walls – Guidance for Economic Design*. Ciria, London, UK, Report No. C580.
- Kearon B (2009) *The Use of LiDAR to Monitor Retaining Wall Movements and the Engineering Geology of the Dublin Docklands*. Civil Engineering Department, University College Dublin, Ireland.
- Little AL, Bond BL and Marshall WJ (1987) Groundwater control in the construction of a Dublin pumping station. *Proceedings IXth European Conference on Soil Mechanics and Foundation Engineering, Dublin* (Orr et al. (ed.)), vol. 1, 183–188.
- Long M (2001) A database for retaining wall and ground movements due to deep excavations. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* **127(3)**: 203–224.
- Long M and Menkiti CO (2007a) Characterisation and engineering properties of Dublin Boulder clay. *Proceedings of*

- the 2nd International Workshop on Characterisation and Engineering Properties of Natural Soils – Natural Soils 2006* (Tan TS *et al.* (eds)), Taylor and Francis, London, UK, vol. 3, pp. 2003–2045.
- Long M and Menkiti CO (2007b) Geotechnical properties of Dublin Boulder Clay. *Géotechnique* **57(7)**: 595–611.
- Long M, Menkiti CO, Kovacevic N *et al.* (2003) An observational approach to the design of steep sided excavations in Dublin glacial till. *Proceedings of Underground Construction 2003 – UC2003, ExCeL*. Hemming-Group Ltd, London, UK, pp. 443–454.
- Long M, Brangan C, Menkiti C, Looby M and Casey P (2012) Retaining walls in Dublin Boulder Clay, Ireland. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **165(4)**, <http://dx.doi.org/10.1680/geng.9.00091>.
- Looby M and Long M (2007) Deep excavations in Dublin – recent developments. *Transactions of Engineers – Ireland* **131**: 1–15.
- Menkiti CO, Long M, Kovacevic N *et al.* (2004) Trial excavation for cut and cover tunnel construction in glacial till – a case study from Dublin. *Proceedings of the Skempton Memorial Conference, Advances in Geotechnical Engineering* (Jardine RJ, Potts DM and Higgins KG (eds)). Thomas Telford, London, UK, vol. 2, pp. 1090–1144.
- Naylor D (1965) Pleistocene and Post Pleistocene sediments in Dublin Bay. *Scientific Proceedings of the Royal Dublin Society* **2A**: 175–188.
- Skipper J, Follett B, Menkiti C, Long M and Clark-Hughes J (2005) The engineering geology and characterisation of Dublin Boulder Clay. *Quarterly Journal of Engineering Geology and Hydrogeology (QJEGH)* **38(2)**: 171–187.
- Twine D and Roscoe H (1999) *Temporary Propping of Deep Excavations – Guidance on Design*. Ciria, London, UK, Report No. C517.

**WHAT DO YOU THINK?**

To discuss this paper, please email up to 500 words to the editor at [journals@ice.org.uk](mailto:journals@ice.org.uk). Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial panel, will be published as a discussion in a future issue of the journal.

*Proceedings* journals rely entirely on contributions sent in by civil engineering professionals, academics and students. Papers should be 2000–5000 words long (briefing papers should be 1000–2000 words long), with adequate illustrations and references. You can submit your paper online via [www.icevirtuallibrary.com/content/journals](http://www.icevirtuallibrary.com/content/journals), where you will also find detailed author guidelines.