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Title of paper: Retaining Wall Behaviour in Dublin's Fluvio-glacial Gravel

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Retaining Wall Behaviour in Dublin's Fluvio-glacial Gravel by Long et al.

Synopsis:

Practicing engineers in the Dublin area have a significant experience in dealing with the boulder clay which underlies much of the city. However the 45 m deep buried pre-glacial channel north of River Liffey is infilled with fluvio-glacial deposits which behave very differently from an engineering point of view. Case history data from eight sites and a detailed examination of the retaining wall behaviour at two of the sites show retaining wall movements appear to be governed by system stiffness (i.e. a combination of wall stiffness and support configuration). It seems relatively simple beam – spring type computer programs will provide data for reasonably accurate designs of retaining walls for basements of up to two levels. Input parameters such as K_0 , ϕ' and soil stiffness need to be carefully specified. Ground water inflows can be significant but can be dealt with by providing a good cut-off into the underlying glacial till or bedrock and by conventional pumping techniques. Geophysical techniques such as MASW, S/P waves and resistivity can be very useful for determination of soil properties, such as degree of saturation, density and stiffness and for material characterisation (i.e. distinguishing the presence of these materials in contrast to the boulder clay).

Keywords: Retaining walls, Granular materials, Excavation, Piles and piling, diaphragm walls

1. INTRODUCTION

From a geotechnical engineer's viewpoint Dublin is well known for its competent glacial lodgement till (colloquially known as 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; Dougan et al. (1996), Long (1997), Brangan and Long (2001), Long (2002), Looby and Long (2007), Kearon (Kearon, 2009), Looby and Long (2010) and Long et al (2011). Similarly steep slopes can be excavated in the material and can stand open unsupported for some time; Long et al (2003) and Menkiti et al. (2004). However a significant geological feature of the city is the buried pre-glacial channel north of the River Liffey. This channel is up to 45 m deep and is infilled mostly with water bearing fluvio-glacial gravels of variable consistency. Significant developments are planned in the area overlying these deposits around Dublin Port and the north inner city. Most importantly, the Dublin Metro North project, which will link the city centre to the airport, via underground railway lines, will encounter these materials. To date, little information has been published on the geotechnical properties of the deposits and the main purpose of this paper is to provide guidance for engineers designing and constructing future schemes in this part of Dublin and in similar materials elsewhere.

This paper firstly summarises the behaviour of eight embedded retaining walls in these materials. Details of the construction at two sites in this zone are outlined. The sites were located at two contrasting positions within the buried channel and monitoring during construction recorded different levels of lateral wall movement and groundwater inflows.

Finally in order to provide design parameters for the material, and as it is not always possible to easily distinguish these materials from the boulder clay till using

conventional shell and auger drilling and logging, some experience of detecting the presence of these materials using geophysical S/P-wave and resistivity techniques are outlined.

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). It is reasonable to conclude, from an engineering point of view, that the geology of the Dublin area is dominated by the stiff glacial lodgement till, known locally as Dublin Boulder Clay; Long and Menkiti (2007a) and Long and Menkiti (2007b).

Farrington (1929) identified the existence of a pre-glacial channel just north of the River Liffey, see Figure 1. It is most dominant towards the north of the present channel of the River Liffey near the city centre and returns near the current mouth of the river. Near Houston Station on the west side of the city the channel crosses over the present river course to the south forming a more localised deeper zone. It is unclear whether widely varying sea levels, tectonic movements or some weakness in the underlying rock gave rise to the channel. However from an engineering point of view, it has significant importance in that it is generally filled with deposits of glacial and fluvio-glacial gravels. Locally the gravels are underlain by a thin layer of boulder clay over the limestone bedrock. Depth to bedrock reaches a maximum of about 45 m below ground level at the mouth of the river. Along the length of the city's main street (O'Connell St.) the bedrock surface falls from about 10 m to 25 m over a distance of some 0.5 km.

The mode of formation of the gravel deposits within the channel has not been studied in detail. It is likely however that the deeper deposits were formed by drainage channels either beneath or within the Pleistocene glacial ice. More recent deposits closer to the surface may be river gravels or river terrace gravels. To the authors' knowledge

little research work on this topic has been done since the original publication of Farrington (1929). This paper attempts to at least begin to address this lack of information.

3. DATABASE OF RETAINING WALLS IN DUBLIN FLUVIO-GLACIAL GRAVELS

A summary of information pertaining to eight retaining walls in the Dublin fluvio-glacial deposits is given on Table 1 and the location of the sites are shown on Figure 1. Seven of the eight sites lie on the southern flank of the north city centre channel (i.e. the most important area economically). Site G4, Railway St. is close to the base of the channel. Site G5, Clancy Barracks is situated in the south - western deep zone again on the flank of the channel.

The retaining walls include both propped and cantilever walls. Maximum excavation depth ranges between 4 m and 11 m. A plot of maximum measured lateral movement (δ_h) versus retained height (H) is shown on Figure 2a. It can be seen that δ_h values vary between 1.5 mm and 15.5 mm. There does seem to be some tendency for δ_h to increase with increasing H but there is no apparent relationship with the support type.

The data shown on Figure 2a takes no account of the retaining wall type, its stiffness or the prop / anchor configuration. In order to attempt to include these factors, the data are replotted on Figure 2b in the normalised form of δ_h/H against Clough et al.(1989) system stiffness, which is defined as:

$$\text{System stiffness} = \frac{EI}{\gamma_w s^4} \quad (1)$$

where:

EI = wall stiffness

γ_w = unit weight of water (required to make expression unit-less)

s = support spacing (taken to be 1.4H for cantilever walls)

Normalised movements appear to decrease significantly with increasing stiffness. Also shown on Figure 2 are lines representing normalised movement (δ_h/H) of 0.08%, [which is the average for Dublin Boulder Clay sites from Long et al. (2011)], 0.18% [average of 169 case histories worldwide where there was stiff soil at dredge level, Long (2001)] and 0.4% [representing a typical design value as recommended by Gaba et al. (2003) in CIRIA report C580]. It seems this system stiffness is much more important than the position of the site in the buried channel.

Overall it can be seen that the behaviour of the walls is very good. The reasons for this finding will be explored in the following sections where two case histories from Smithfield (case history G3) and Railway St (G4) are outlined in detail. These sites are at different locations in the buried channel and data from these two sites fall in contrasting positions on the plots on Figure 2.

4. SMITHFIELD SITE

4.1 Location and history

The site location is shown on Figures 1a and 1b and some details of the ground conditions and support system are given on Table 1 and Figure 3 and also by Brangan (2007) and Long and Dhouwt (2003). The site is approximately 210 m by 80 m and is surrounded mostly by public highways though an important large diameter cast iron water main is present on the Queen St side of the site. Originally the topography of the site comprised a gentle slope from a level of about +8 mOD in the north to about 5.5 mOD in the south. The basement excavation was required for three levels of car parking and was 10 m to 11 m from existing ground level (formation level from -3 mOD in the north to -5.2 mOD in the south).

4.2 Site Investigations and ground conditions

A summary of the ground conditions encountered in the four site investigations is given on Table 2 and on Figure 3. The made ground material comprises a variable mixture of brick, masonry and domestic refuse in a clayey gravel matrix. Underlying the made ground the gravels are medium dense to very dense coarse, with occasional cobbles and boulders. A detailed discussion on the engineering parameters of this stratum will be given in the next section. A compact silt / stiff clay was observed above the gravel in southern section of the site possibly associated with recent alluvial deposits of the River Liffey.

Groundwater level across the site varied between -0.9 mOD to +2 mOD with an average level of +0.5 mOD. There was no evidence that it was influenced by tidal effects.

4.3 Properties of gravels

Particle size distribution analyses for the gravels are shown on Figure 4a. On average the material contains 10% to 15% sand with the remaining constituents consisting of roughly equal amounts of fine, medium and coarse gravel. The gravels are dominated by limestone with occasional red sandstone lithologies.

There appears to be no clear pattern in the variation in sand content with depth. Test results for the Smithfield site and for two immediately adjacent sites at Irish Distillers and on Friary Avenue show the material is relatively uniform. It should be noted that these tests may not accurately reflect the gravel content as the boreholes were either 200 mm or 250 mm in diameter and particles greater than 75 mm were removed before sieving.

The possibility of fine material being washed out during the shell and auger drilling or sampling processes was examined by recovering additional samples from the bulk excavation on site. Curves for samples taken from -2mOD, -3mOD and -4.5 mOD

(denoted UCD tests), fall on the left (fines) side of the plot and confirm that the more representative sand content may be about 25%. This suggests approximately half of the sand content may have been lost during the drilling process.

Standard penetration test results for the gravels at the same three sites and also for Collins Barracks (see Section 6 on Geophysics) are shown on Figure 5a. There is significant scatter in the data particularly for Smithfield. Refusal is denoted by a value of 100 blows / 300 mm. The different investigations appear to yield different results, reflecting different practices as to when the test should be terminated. Of the approximately 200 SPT values reported from Smithfield, 130 were recorded as “refusal”. Although some of these tests may have encountered resistance from cobbles they nonetheless indicate the gravels are in a very dense to dense state [density categories after Table 13 BS EN ISO 14688-2:2004 (2006)]. The Collins Barracks results are very similar to those at Smithfield, albeit with no refusals recorded.

In the Irish Distillers and Friary Ave. investigations, there appears to be some tendency for an increase in SPT N with depth. In both cases the values suggest that, like at Smithfield and Collins Barracks the material is in a dense to very dense state. Three of the 23 SPT tests recorded refusal.

Further evidence of the density (and the cobble content) of the material can be obtained from the chiselling times recorded (see Table 2). These were on average 7 hours per borehole and the chiselling was required throughout the depth of boreholes.

Some falling head permeability tests confirmed the gravels had very high permeability.

4.4 Properties of bedrock

Bedrock comprises strong to very strong locally moderately strong thickly bedded argillaceous limestone, known locally as “calp”. Total core recovery was high and the

rock quality designation (RQD) and average unconfined compressive strength was about 45% and 80 MPa respectively.

Pumping from a test well constructed into the bedrock showed that inflow rates should be modest. It also confirmed that there was a definite hydraulic connection between the bedrock and the gravels and that the latter had high permeability conservatively of the order of 1×10^{-3} m/s.

4.5 Analysis and design

The retaining wall design was carried out using the GEOSOLVE-WALLAP[®] computer program. The design toe level of the wall was determined using a factor of safety on moments following the Burland et al. (1981) methodology. The Burland et al. (1981) technique gave slightly more conservative ($\approx 9\%$) design moment and thus was utilised in preference to the strength factor method detailed in BS8002 (1994).

These calculations used moderately conservative soil parameters, which are summarised on Table 3. Moderately conservative parameters were adopted as little previous experience of the material behaviour was available at the time of design and a robust solution was required. The three key input parameters are the co-efficient of earth pressure at rest (K_0), gravel stiffness (E') and effective friction angle (ϕ'). E' was determined from the geophysical testing (having corrected for strain effects), see Section 6. BS8002 (1994) gives guidance for the determination of ϕ' and suggests taking peak and critical state friction angles as:

$$\phi'_{\max} = 30^\circ + A + B + C \quad (2)$$

$$\phi'_{\text{crit}} = 30^\circ + A + B \quad (3)$$

where: A, B, and C are the contributions from angularity, grading and density of the material. Here the $A+B+C$ contribution was taken to be 6° to 7° . Again this chosen

value is consistent with the geophysical data. K_0 was assumed to equal $1 - \sin\phi'$. Groundwater level was assumed to be at 1.25 mOD.

4.6 Construction

A diaphragm wall solution, rather than a secant pile wall, was adopted by the retaining wall contractor, Cementation – Skanska, so as to minimise risk of water ingress through joints in the piles and at rock head. Full details of the construction at Smithfield are presented by Daynes and McCann (2003). The diaphragm wall was constructed using crane mounted rope suspended clam shell grabs. The wall thickness was 800 mm, with 7.2 m long panels and with a proprietary water stop sealing strip at each panel joint to improve water tightness. Guidewalls were 1 m to 2m deep. A bentonite based support fluid was used rather than the alternative of polymer based solution. This decision was based upon concerns regarding the coarseness of the gravels so as to ensure an adequate bentonite filter cake. A 5% by volume bentonite concentration was used, rather than the more typical 4%.

All wall panels were excavated to rock head to prevent water ingress below the toe of the wall (see Figure 3). Rock head was easy to identify by means of the strong resistance to penetration and the fragments of weathered limestone recovered. Reinforcement was provided to the toe level calculated for an adequate factor of safety on toe lateral stability (typically 15 m to 16 m), with the remaining depth of panel unreinforced. Reinforcement density was approximately 100 kg/m^3 and cages were spliced on site during placing. Where the design depth of the panel was not achieved, due to high rock level, rock dowels were used to ensure lateral stability. Concrete overbreak was typically 15% above the theoretical value. Typically one panel was constructed per rig every two days. Two rigs were used on the site, thus simplifying logistics as one panel was constructed every day.

Generally one level of tie back ground anchorages was used for retaining wall support (Figure 6a), except near the Queen St water main where two levels were used (Figure 6b) so as to minimise the risk of damage from wall deflection. A capping beam was only constructed along the section of diaphragm wall adjacent to the water main. The tie-back anchorages comprised 4 or 5 strand anchorages with design working loads of 640 kN to 750kN.

In the temporary condition during construction, dewatering was by means of 22 wells via submersible pumps each of which had a capacity of 5 l/s.

4.7 Performance

A comparison between the design stage analyses using GEOSOLVE-WALLAP[®] V5.01 (as originally done by Cementation – Skanska) and subsequently using OASYS-FREW[®] V18.2 is shown on Figure 7b. Input parameters are summarised on Table 3. A particular feature of the analyses is the surcharge loading of 40 kPa assumed so as to mimic high adjacent ground level at the northern site end in addition to traffic loading. Both programs give similar predicted maximum displacements of about 20 mm and also bending moments, and shear and anchor forces. WALLAP[®] predicts a wall displacement profile similar to that of a propped cantilever whereas FREW[®] predicts a cantilever wall type shape.

Typical inclinometer displacements profiles are shown on Figure 7c. Maximum recorded movements were small, always less than 4 mm and most of the profiles resembled the movement of a cantilever, see data for IC33, Panel 33, on Figure 7. The inclinometer locations are shown on Figure 3. These recorded movements were close to the accuracy of the inclinometer system and were much less than the design predicted displacement as shown on Figure 7b.

Some displacement, usually under 2 mm, was recorded away from the excavation at the top of the wall, due to the effects of pre-stressing the ground anchors. When the

anchors were de-tensioned the top of the retaining wall moved towards the excavation by as much as 5 mm.

Back-analyses were subsequently performed using FREW[®], as shown on Figure 7d. It was found that predicted movements were sensitive to surcharge loading, K_0 , gravel stiffness (E') and were relatively insensitive to effective friction angle (ϕ'). The output with unchanged K_0 and ϕ' but with surcharge of 10 kPa and E' values of 80 MPa at the surface and increasing at a rate of 50 MPa / m with depth gives a reasonable comparison with the measured displacement. These E' values are derived from the shear wave velocity (V_s) values detailed in Section 6 having made a reduction to account for the decrease in stiffness with strain.

A total station survey was also performed, especially close to the water main along Queen Street, in an attempt to measure lateral movements and settlement but movements were too small to be detected.

Details of the pumping effort during Phases 1 to 3 are shown on Figure 8 a. It can be seen that the pumping effort required was small. Flow rates never exceeded 23 l/s with average flow rates in the range 5 l/s to 10 l/s. At any time only 5 to 6 of the available 22 wells were in use. Locations of piezometers located outside the site are shown on Figure 3 and water levels (Figure 8b) were little affected by the dewatering. Maximum drawdown was of the order of 0.4 m and it seems that the water levels were much more influenced by rainfall and tidal effects than by site dewatering. These data indicate that a very successful seal was achieved at the diaphragm wall toe.

5. RAILWAY St. SITE

5.1 Location and history

This site is located at the corner of Railway Street and James Joyce Street (formerly Corporation Street), east of Smithfield, see Figures 1 and 11. Mabott Lane forms the

western boundary. The site is 80 m long by 30 to 45 m in width, with ground level at approximately 3.5 mOD.

5.2 Summary of ground conditions

The soil comprised 2.2 m of fill over sands and gravels (see also Figure 11). Bedrock was intact limestone similar to that at Smithfield. In the ground investigation it was proven to fall from -16.3 mOD (20 m depth) in the north west of the site to -23.6 mOD (27 m depth) towards the south of the site. The top 0.6 m of the rock on average was reported to be weathered. Groundwater was around 0.0 mOD (3.5 m depth).

5.3 Properties of gravels

This material was predominately dense gravels with cobbles and boulders, though deposits of medium dense sands were also recorded in the upper 12 m. No particle size distribution analyses are reported for the Railway St. site. However particle size distribution analyses for two sites adjacent to the site (at Store St. Garda station and Eason's Gardiner St.) are shown on Figure 4b. These data indicate the material is very similar to that at Smithfield. It contains about 20% sand with the remaining constituents consisting of roughly equal amounts of fine, medium and coarse gravel. Like at Smithfield there is no clear pattern in the sand content with depth. Again the gravels are dominated by limestone with occasional red sandstone lithologies.

Standard penetration test N values for sites in the Railway St area are shown on Figure 9. Again the N values suggest the materials are generally in a dense to very dense state similar but perhaps a little lower than those at Smithfield. SPT N values were usually between 30 and 70 in the gravels and 20 to 40 in the sand. Compared to Smithfield relatively few refusals were recorded. Of the 150 or so tests reported only 9 refusals were noted. Significant chiselling times were also measured but these were usually less than 5 hours per borehole.

A pumping test was carried out in a 220 mm diameter well 7.8 m deep. Although water was pumped at a rate of up to 16.3 l/s, resulting in a drop in water level in the well of about 2.9 m the influence on the adjacent monitoring wells was very small indicating the material has very high permeability of the order of 10^{-2} to 10^{-3} m/s.

5.4 Analysis and design

The retaining wall design was carried out in a very similar manner to that at Smithfield, using the “moderately conservative” parameters listed on Table 3 and as has been discussed above in Section 4.5. Here the computer program FREW[®] was used instead of WALLAP[®] but, as has been shown in Section 4.7, both give similar output.

5.5 Construction

At Railway St. a final excavation depth of 8 m was required. A secant wall was preferred to a diaphragm wall on cost grounds and also because the cobble content of the material was lower than at Smithfield.

Piles were 600 mm in diameter at 500 mm centres. Full length reinforcement, comprising 8T32 bars, was placed in male piles only and the “soft” piles comprised unreinforced lower grade concrete. The retaining wall was constructed using the continuous flight auger (CFA) technique with an 85 tonne, 250 kNm torque rig which has a pull down force of 100 kN. About 10 revolutions per minute were required to progress the drilling and it took typically 20 minutes to drill the full pile length. As concrete volumes of up to 21 m³ per pile were required, two concrete trucks discharged simultaneously into a hopper to prevent any stoppages. Overall progress of the piling work varied between 10 and 15 piles per day and no difficulties were posed by cobbles and boulders. Overbreak was typically 5% to 10% but occasionally 15% to 20%, especially where the gravel stiffness increased with depth.

The top of the capping beam varied from 3.1 to 3.45 mOD. As constructed pile toe levels were measured at between 20 m depth in the north and 27 m depth in the south, consistent with the rock head values revealed by the boreholes, see Section 5.2. As there was no specific rock socket constructed there was some concern regarding inflow beneath the base of the piles or through gaps caused by the loss of secant action below dig level. A guide wall was used and a tight verticality of 1:150 was specified in order to minimise these effects.

Temporary propping was provided across the excavation by the Groundforce™ proprietary system of hydraulic struts, see Figures 10a and 11. These extended across the width of the site, supported at the centre by a pile used as a column. Shorter struts were used as cross-bracing at the corners.

Prior to the commencement of construction, two wells were drilled to bedrock using the piling rig. These wells were lined with slotted plastic pipes and submersible pumps installed. Following secant wall construction, the area within the site was dewatered and subsequently pumping was required throughout construction.

5.6 Performance

Inclinometers 1 and 2 at the southern end of the site (see Figures 10a and 11) and beside the adjoining buildings, and where the bedrock was deepest, recorded maximum movements of 9 to 11 mm at the capping beam (see example on Figure 11b). The displacement profile was typical of a propped excavation. The other locations were less onerously loaded and had shallower bedrock. Inclinometer 3 was adjacent to Mabbot Lane on the western side of the site (see Figures 10a and 11) and showed movement of 7 mm by this stage. Inclinometer 4 beside James Joyce Street (opposite Inclinometer 3 on eastern side) recorded movement of 5 mm. The contractor also recorded movements of 5 mm into the site at Location 4 by surveying with a total station (which from

experience had an accuracy of ± 2 mm). Inclinator 3 was monitored for more than one month after the props were removed by which time displacements had increased to 12 mm, taking a profile typical of a cantilever (Figure 11b). No ground settlement readings were taken external to the secant pile wall. However no cracking or damage to adjacent structures were observed.

Details of some back-analysis using FREW[®] are shown on Figure 11c. A surcharge of 10 kPa was used to model the adjacent buildings. Input parameters are the same as those in Table 3 except, following the Smithfield back-analysis, E' for the gravels has been taken to be 80 MPa at top of the gravels and then to increase at a rate of 50 MPa / m with depth. Predicted and measured movements at the wall top (i.e. prop) are similar but below this level FREW overpredicts the measured movements considerably. Parametric studies show that, in contrast to Smithfield, the output is relatively insensitive to K_0 and stiffness values but is very sensitive to ϕ' and if a value of 40° is used there is a reasonable fit between the measured and predicted values.

Moderate inflows of groundwater, of the order of 10 l/s to 12 l/s, were recorded. This can be seen on Figure 10a, where water inflow is occurring into an excavation for foundation bases and Figure 10b which indicates the pumping effort required and the nature of the gravels. It is considered that these relatively low flow levels confirm that a reasonable cut off was developed at the pile base and that there was minimal loss of secant action below dig level.

5.7 Comparison of performance at Smithfield and Railway St.

At both sites the gravel materials have similar particle size distributions, SPT values and permeability. It is possible that the gravels in the Smithfield site have higher cobble content based on the number of SPT refusals recorded and evidence from construction such as that shown on Figure 5b.

Much more significant wall movements were recorded at Railway St. compared to Smithfield. This could have been due to a combination of factors such as the greater depth to bedrock at Railway St., the slightly lower SPT N values at Railway St. and the higher cobble content in the gravels at Smithfield. However as shown on Figure 2 it would seem that the system stiffness (i.e. the combination of retaining wall stiffness and support configuration) was the primary reason for the lower moments recorded at Smithfield.

Groundwater inflows were relatively low at both sites and presented no practical difficulties. However, although the area of the Smithfield site is some 5.5 times greater than that at Railway St. pumping effort was of a similar magnitude at both sites. At both sites no seepage was noted through the retaining wall above dig level. Therefore inflow must have been either around the wall toe at its interface with the bedrock or else by upward flow through discontinuities in the rock. It would seem most likely that the diaphragm wall construction technique used at Smithfield provided a more effective cut off than the secant wall at Railway St.

6. INVESTIGATION OF FLUVIO-GLACIAL GRAVELS USING GEOPHYSICAL TECHNIQUES

6.1 Introduction to techniques used

Geotechnical engineers have several objectives when utilising geophysical techniques in the Dublin fluvio-glacial gravels as follows:

- to help distinguish these materials from the lodgement till, found extensively in the Dublin area, which can be difficult using conventional shell and auger drilling and logging techniques,
- for detecting the extent of these materials,

- to provide information on the engineering properties of the gravels (e.g. degree of saturation and density) to complement the questionable SPT data.

These techniques also have the advantage over more conventional investigative approaches of being non-intrusive and as a result may be used to investigate a large amount of ground relatively quickly and cost-effectively.

The most common geophysical approach for distinguishing gravel deposits is using resistivity measurements e.g. O'Connor et al (2009). Unfortunately, the effectiveness of resistivity investigations in urban environments may be severely limited as a result of interference from electrical noise and underground services and the difficulties in planting electrodes into the ground. As a result this approach has not been used extensively in Dublin city centre. Although not tested in this study, recently developed continuous capacitive resistivity (CCR) systems are more suited for urban resistivity investigations as electrodes do not need to be planted and can instead be towed along the ground.

Measurements of seismic velocities may also be indirectly used to indicate the presence of gravels, due to their dependence of density and small strain elastic stiffness. P-wave seismic refraction testing, in particular, has generally been used in the past as an indicator for gravel density. As P waves are able to travel through water, this approach also has the advantage of being able to distinguish between saturated and unsaturated gravels and thereby provide an indication of the depth to the water table.

Although it has become popular in recent years as a tool for characterising the shear wave velocity (V_s) and corresponding small strain stiffness (E_{max} , Eq. 3) of the shallow subsurface [Donohue et al. (2003) and Donohue and Long (2008)], the Multichannel Analysis of Surface Wave (MASW) approach has not been tested previously for distinguishing gravel deposits in Dublin. Young's modulus is a key input parameter into

retaining wall analyses. The large amplitude nature of surface waves relative to body waves makes them more suitable for investigations in urban areas, such as Dublin, where first arrivals may be difficult to detect. According to elastic theory E_{\max} may be calculated from the shear wave velocity using the following equation:

$$E_{\max} = 2\rho(1 + \nu)V_s^2 \quad (4)$$

where E_{\max} = (Pa), V_s = (m/s), ρ = density (kg/m^3) and ν = Poisson's ratio.

Knowledge of V_p and V_s also provides a measure of ν :

$$\nu = \left(\frac{(V_p / V_s)^2 - 2}{2 * (V_p / V_s)^2 - 2} \right) \quad (5)$$

Poisson's ratio for unsaturated gravels is generally significantly less than that for boulder clay and therefore combining these two measurements should improve our ability to distinguish these materials non-intrusively at small strains.

Typical ranges of resistivity and seismic measurements for materials commonly found in central Dublin are provided in Table 4. This table has been produced from this work, general experience of the authors and the limited available previous publications, e.g. O'Connor (2001). Note the contrast in geophysical properties between Dublin Boulder Clay and the gravel deposits. Resistivity measurements are generally found to be higher, and seismic measurements (V_s and V_p) generally found to be lower in Dublin's gravel deposits than in Dublin Boulder Clay. The geophysical techniques used are also complementary as, for example, on the basis of resistivity alone we cannot usually distinguish between saturated silty gravel and unsaturated clayey gravel. P waves travel at a velocity of approximately 1500 m/s in water and therefore may be used to distinguish between saturated and unsaturated gravel deposits. Also, in very dense, saturated gravels the V_p contrast between gravels and Dublin Boulder Clay can

occasionally be unclear (Table 4). It is therefore recommended, wherever possible, to perform all three techniques in order to assist interpretation.

6.2 Investigation of sites

Two sites, Foley St. Park and Blackhall Green (see Figures 1 and 12), with confirmed gravel deposits were investigated using a combination of electrical resistivity tomography (ERT), P-wave seismic refraction and MASW. Greenfield sites were selected in order to limit the effect of electrical noise on the resistivity measurements, thereby providing an appropriate comparison between the seismic (refraction and MASW) and resistivity results.

Foley St. Park

Foley St. Park (see details on Figure 12) is located immediately to the east of the Railway St site. Shell and auger borehole logs and SPT N tests from Railway St., Store St. Garda Station (which is 125 m to the south), the adjacent Liberty House development on James Joyce St., and an office development on Foley St. all indicate generally similar medium dense to dense, slightly silty, fine to coarse sandy gravel. SPT N values, presented on Fig. 9, are generally less than 30 in the zone of the geophysical investigation.

Resistivity values for the central and north-eastern area of the Foley St investigation, shown in Figure 13a are suggestive of either unsaturated clayey silty gravel or saturated sandy gravel (Table 4). They also appear to be lower in the west of the profile possibly indicating finer material. Above 5 m to 6 m P wave velocities (Figure 13b) are quite low for this gravel layer ($V_p = 600$ m/s) indicating an unsaturated loose to medium dense gravel, consistent with the SPT test results as discussed above. Below 6 m the P wave velocities are higher ($V_p = 1650$ m/s) and are suggestive of saturated medium dense to dense gravels again consistent with the SPT values and confirming the location of the

water table. An increase in shear wave velocity (V_s), see Figure 13c, is observed below approx. 4 m depth indicating an increase in gravel density again consistent with the SPT and P wave velocity data. V_s values also appear to be higher for the finer gravels in the southwest (200-250 m/s) than for the northeast (125-200 m/s).

Blackhall Green

Boreholes located near to Blackhall Green at Collins Barracks (approx. 75m west) and at a residential development on Benburb St. (approx. 60m south), see Figure 12, generally indicate fine to coarse sandy gravel, with increasing fines content (26%) towards the north end of Collins Barracks. SPT N values for the Collins Barracks site are shown on Figure 5a and are generally between 25 and 60 (average 46) indicating a dense gravel and are similar to those reported for the Smithfield site above.

Resistivity values for Blackhall Green (Figure 14a) indicate a relatively high resistivity gravel layer (green-purple) overlying a layer of much lower resistivity (shown in blue). i.e. clearly distinguishing the gravels from the boulder clay. Interestingly, resistivity values for the upper layer are higher in the south (up to 600 Ωm) indicating unsaturated silty gravel. Towards the north the resistivity values reduce to less than 200 Ωm , indicating an increase in fines content as observed in the borehole logs. Unsaturated gravels, however, generally have resistivity values in excess of 250 Ωm (Table 4) in Ireland which suggests that the resistivity of these gravels may have been affected by urban contamination. The low resistivity layer at a depth of greater than 5 m exhibits characteristic resistivity values expected of Dublin Boulder Clay (40-100 Ωm).

P-wave velocities measured from seismic refraction tests, shown in Figure 14b also indicates three layers. These include an upper layer of made ground ($V_p = 250$ m/s), a middle layer of dense unsaturated gravel, with a $V_p = 900$ m/s and a deeper layer of

glacial till, with a V_p of 2000 m/s. The value for the gravels is higher than at Foley St and is thus consistent with the SPT values being higher at this site. The 2D shear wave velocity profile shown in Figure 14c, indicate lower velocities for the dense gravel layer (250- 400 m/s) than for the underlying glacial till (400-700 m/s).

Summary

In summary the geophysical techniques can help delineate the presence and extent of these materials in contrast to the boulder clay. However, from the point of view of geotechnical engineers designing retaining walls, they provide information for the choice of input parameter such as E' (having corrected the in situ measured values for strain effects).

7. CONCLUSIONS / RECOMMENDATIONS FOR FUTURE

WORKS

1. From an engineering point of view the fluvio-glacial deposits in the 45 m deep buried pre-glacial channel north of River Liffey in Central Dublin are considerably different from the boulder clay found over most of the city.
2. Case history data from eight sites and a detailed examination of the retaining wall behaviour at two of the sites indicates that retaining wall movements are governed by system stiffness (i.e. a combination of wall stiffness and support configuration).
3. Relatively simple beam – spring type computer programs will provide data for reasonably accurate designs, provided that input parameters such as applied surcharge, K_0 , ϕ' and soil stiffness are carefully specified. To date these programs have only been justified for basements of about two levels and further work is required to assess their accuracy for multi-propped and deeper basements.

4. Ground water inflows can be significant but can be dealt with by providing as good a cut off as is practical and economical into the underlying glacial till or bedrock and by dealing with the remaining inflow by pumping from wells / sumps within the site.
5. The use of the standard penetration test (SPT) in these materials may not be appropriate given the large particle size. At the very least the tool must be calibrated to the requirements of an appropriate standard such as pr EN ISO 22476-3 (2002). Consideration could also be given to using large penetration tests (LPT), which have been used successfully in similar materials in the US, Japan and Italy, see Daniel et al. (2003).
6. Geophysical techniques, such as MASW, S/P wave and resistivity can provide very useful information to the geotechnical engineer designing retaining walls in these materials. Data can be provided on the nature of the material (e.g. distinguish it in contrast to the boulder clay), its degree of saturation, density and stiffness.

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Tables for:

Retaining Wall Behaviour in Dublin's Fluvio-glacial Gravel by Long et al.

Table 1. Dublin gravel case histories

Case history	Location	Ground conditions <i>Soil at excavation level</i>	SPT N (blows / 300 mm) range (average)	H (m)	h (m)	B (m)
G1	Kings Inn St	Made ground, dense gravel , DBC, Imst.	N = 30 - 45 (37)	5.7	3.9	30
G2	Jervis St. Shopping Cen.	Dense gravel over DBC	SPT N = 50+	9.7	3	100
G3	Smithfield	Fill, dense gravel , Imst	22 - 98 (52)	11	3	80
G4	Railway St.	Made ground, dense gravel , Imst.	18 - 80 (44)	8	2.2	37
G5-P	Clancy Barracks - Anchored	Fill / silt, dense gravel , DBC	5 - 80 (42)	7.2	4	70
G5-C	Clancy Barracks - Cantilever	Fill / silt, dense gravel , DBC	5 - 80 (42)	7.9	4	150
G6	Hammond Lane	Made ground, dense gravel , DBC, Imst.	20 - 80 (42)	4	3	40
G7	Church Street	Fill, dense gravels	20 - 80 (45)	6	4.5	30
G8	Parnell St./ Granby Place	Fill, dense gravel	Dense	4	3	62

Support configuration	s (m)	Wall type	Pile dia. / spacing / length (m)	EI (kN/m ²)	δ _h (mm)	Reference
Single prop	5.7	Secant	0.6 / 0.5 / 12	381700	5.5	Brangan (2007)
Single prop	8.5	Secant	0.9 (0.81 in rock) / 1.54 / 6.5-11.9	1254800	3	Long (1997) / Dougan et al. (1996)
Single anchor	7	Diaphragm	0.82 thick / 14-18 long	1130300	4	Brangan (2007), Long and Dhouwt (2003)
Single horizontal prop	8	Secant	0.6 / 0.5 / 12	381700	11	Brangan (2007)
Single anchor	7.2	Secant	0.9 / 1.5 / 11.5	644126	12	Looby and Long (2010) / Kearon (2009)
Cantilever	11.1	Secant	0.9 / 1.5 / 15.5	644126	15.5	Looby and Long (2010) / Kearon (2009)
Cantilever	5.6	Secant	0.6 / 0.5 / 9.5	381700	1.5	Brangan (2007)
Cantilever	8.4	Secant	0.6 / 0.5 / 12.5	381700	4	Looby and Long (2010)
Cantilever	5.6	Secant	0.6 / 1 / 9.5	190850	3.2	Looby and Long (2010)

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 and δ_h = maximum horizontal movement

Table 2. Summary of ground conditions at Smithfield site.

Stratum	Range	Average	Comment
	Thickness (m)	(m)	
Made ground	1.8 – 7.5	3.0	Only one location > 4.3m
Alluvium / stiff clay	Absent to 7.5	0.0	Noted along southern site
Medium dense sand	Absent to 12.1	0.0	Noted in NE corner only. Verified by construction records
Dense gravels	9.0 to 13.0	10.8	
Boulder Clay	Absent to 1.9	0.0	Noted along southern site boundary
Limestone bedrock	At 13.8 m to 21.0 m depth	Proven to 7.2 m	Average depth to rock was 16.2m
Chiselling	1.5 to 19 hours	7 hours	Mostly in dense gravels
Groundwater	-0.9m to +2.04mOD	At +0.5 mOD	Street level slopes from +8mOD to +5.5mOD is

Table 3. Summary of design input parameters

Stratum	Young's modulus E (MPa)	Bulk density γ (kN/m ³)	Effective friction angle ϕ' (deg.)	Coefficient of earth pressure at rest K_0
Made ground	4.0 + 2.5z*	18	28	0.53
Stiff clay#	100	20	34	1.3
Gravels	150	20	36 - 37	1-Sin ϕ'

* z = depth

Local presence only

Table 4. Typical resistivity, V_p and V_s values for a range of glacial materials found in the Dublin area.

Material Description	Resistivity (Ω -m)	P-wave velocity (m/s)	S-wave velocity (m/s)
Sand & Gravel	>1000	300-500 (loose)	100-200 (loose)
<i>(Unsaturated)</i>		500-800 (medium)	200-300 (medium)
		800-1200 (dense)	300-500 (dense)
Silty Gravel	500-1000	300-500 (loose)	100-200 (loose)
<i>(Unsaturated)</i>		500-800 (medium)	200-300 (medium)
		800-1200 (dense)	300-500 (dense)
Silty Clayey Gravel	250-500	300-500 (loose)	100-200 (loose)
<i>(Unsaturated)</i>		500-800 (medium)	200-300 (medium)
		800-1200 (dense)	300-500 (dense)
Silty Gravel	200-350	1450-1600 (loose)	100-200 (loose)
<i>(Saturated)</i>		1600-1700 (medium)	200-300 (medium)
		1700-1900 (dense)	300-500 (dense)
Silty Clayey Gravel	100-225	1450-1600 (loose)	100-200 (loose)
<i>(Saturated)</i>		1600-1700 (medium)	200-300 (medium)
		1700-1900 (dense)	300-500 (dense)

Boulder Clay	40-100	500-1800 (firm-stiff)	200-400 (firm-stiff)
		1800-2300 (stiff-hard)	400-700 (stiff-hard)
Dublin Black Boulder Clay	40-80	2000 - 2300	500-700

Figures for:

Retaining Wall Behaviour in Dublin's Fluvio-glacial Gravel by Long et al.

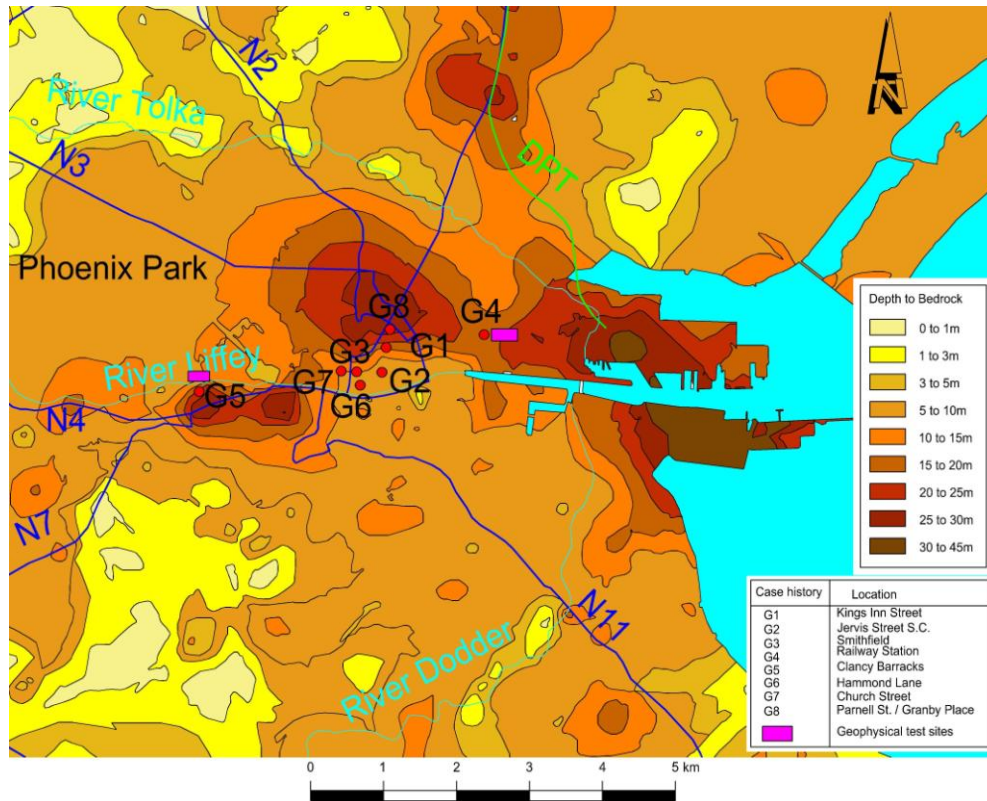


Figure 1. Depth to bedrock contours showing retaining wall sites and geophysical test locations (depth to bedrock courtesy Geological Survey of Ireland, www.gsi.ie)

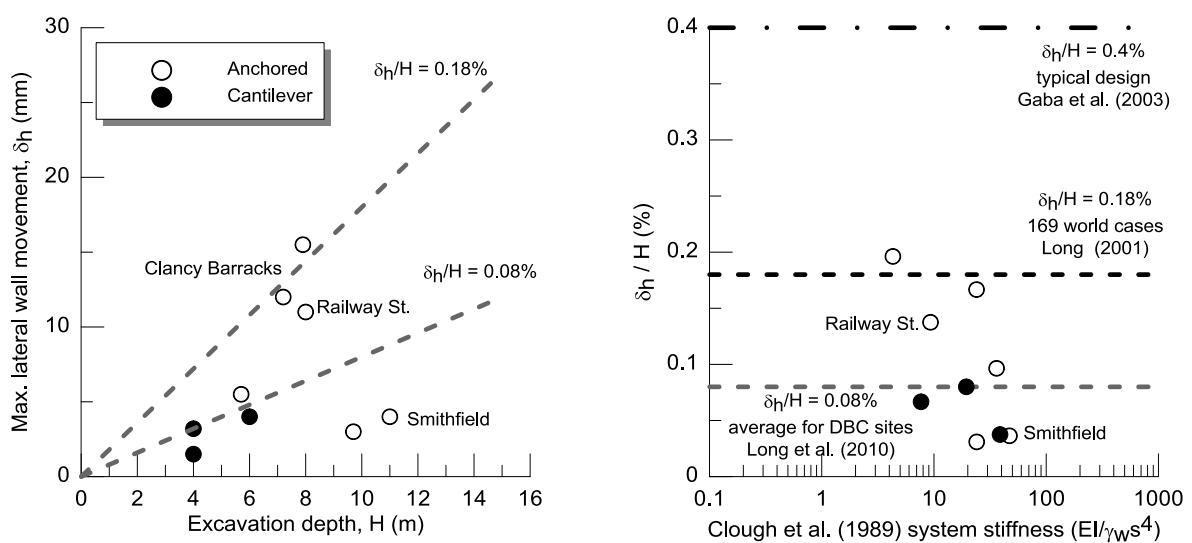


Figure 2. Dublin fluvio-gravel database (a) δ_h versus H (b) δ_h / H versus $EI/\gamma_w s^4$

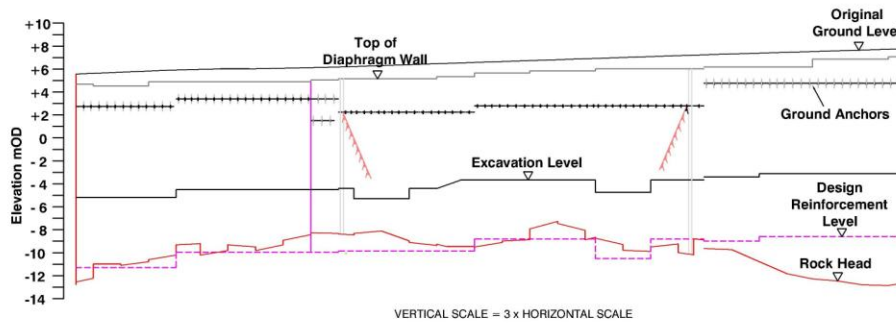
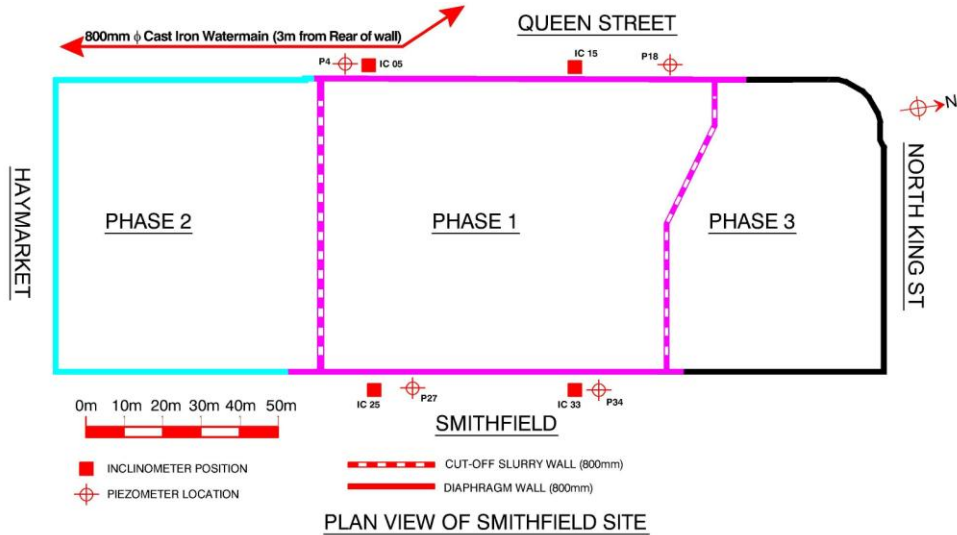


Figure 3. Plan view and section of Smithfield site

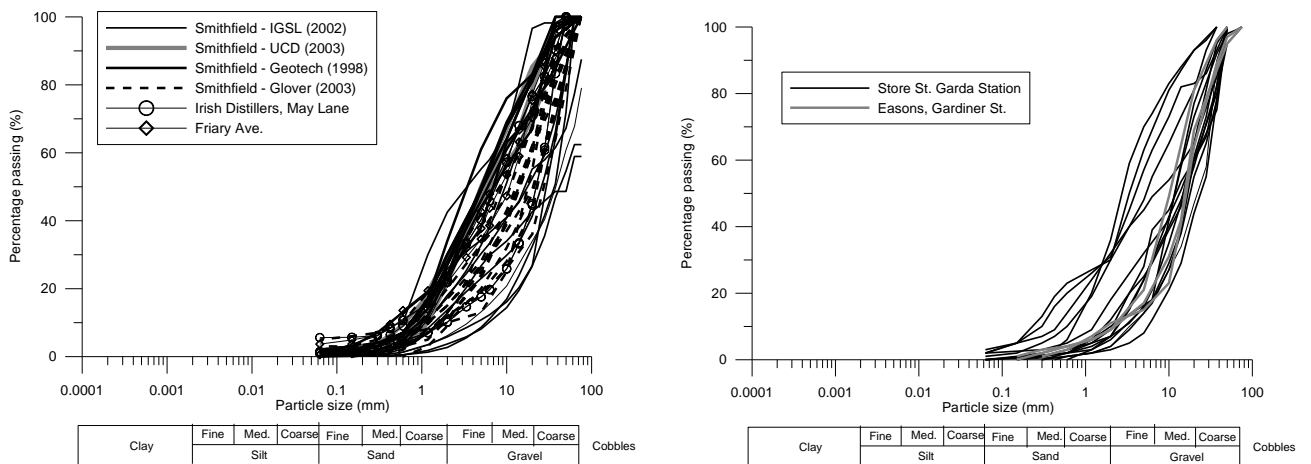


Figure 4. Particle size distribution curves for (a) Smithfield and (b) Railway St.

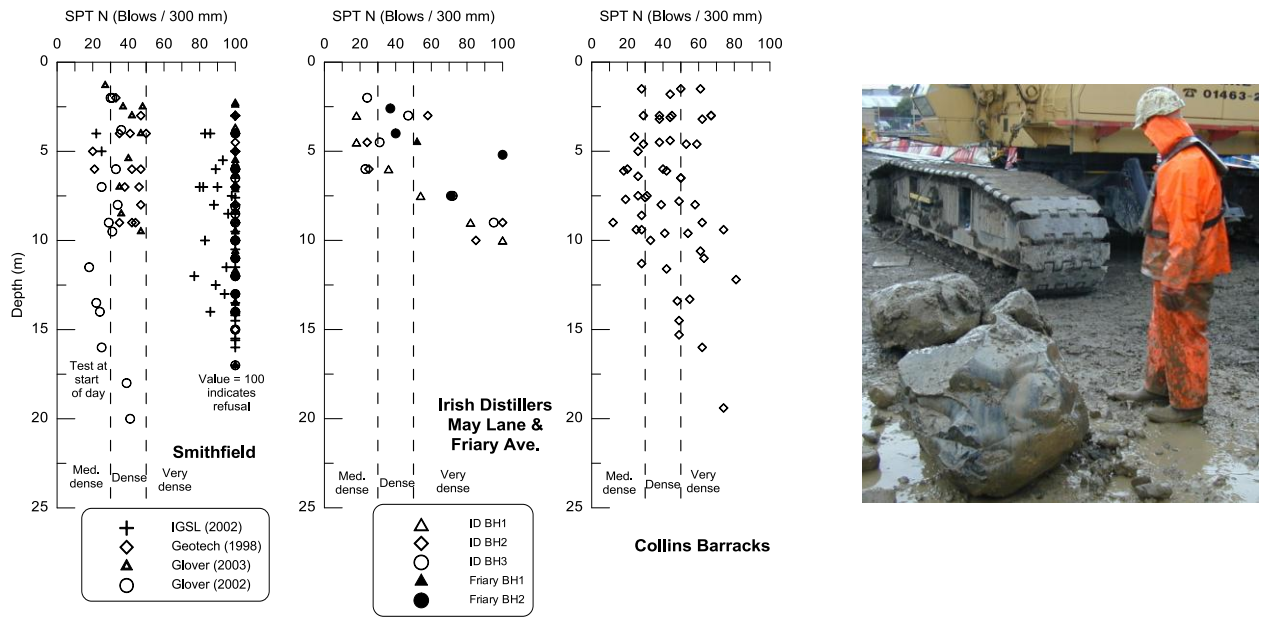


Figure 5. (a) Standard penetration tests for Smithfield area (density categories after BS EN ISO 14688-2:2004) (b) boulder recovered during diaphragm wall construction



Figure 6. Construction at Smithfield (a) Typical approach and (b) near Queen St watermain

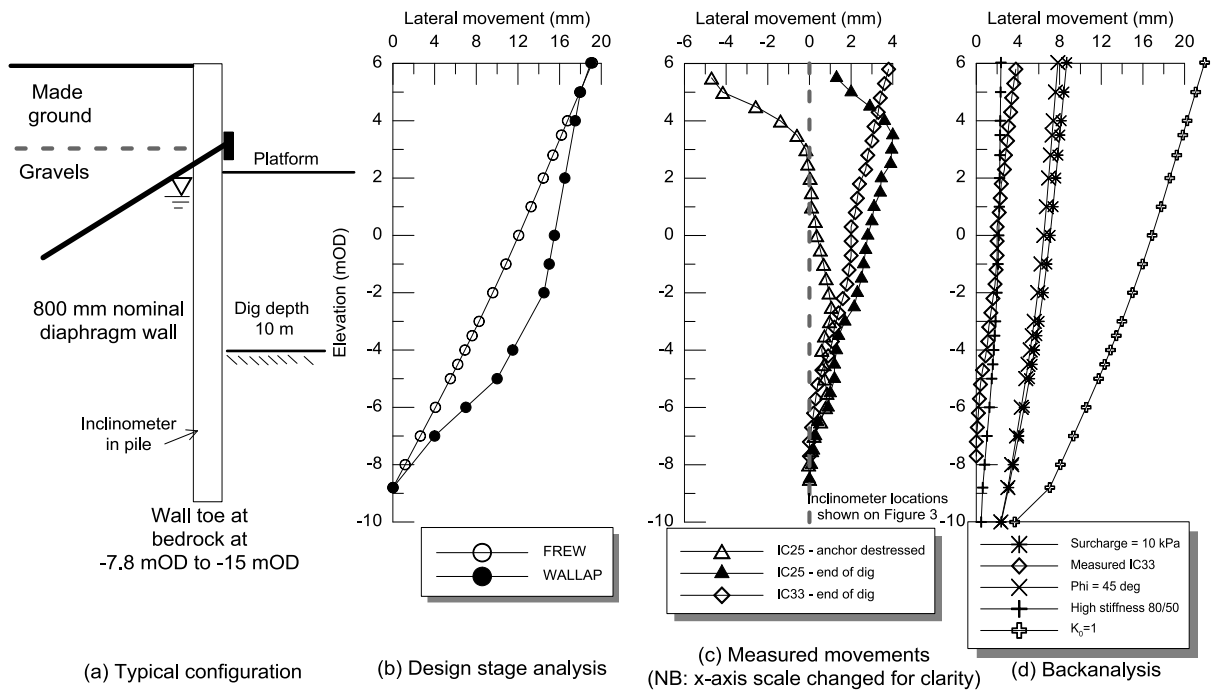


Figure 7. (a) Typical retaining wall configuration, (b) design stage analyses, (c) measured movements and (d) back-analysis for Smithfield

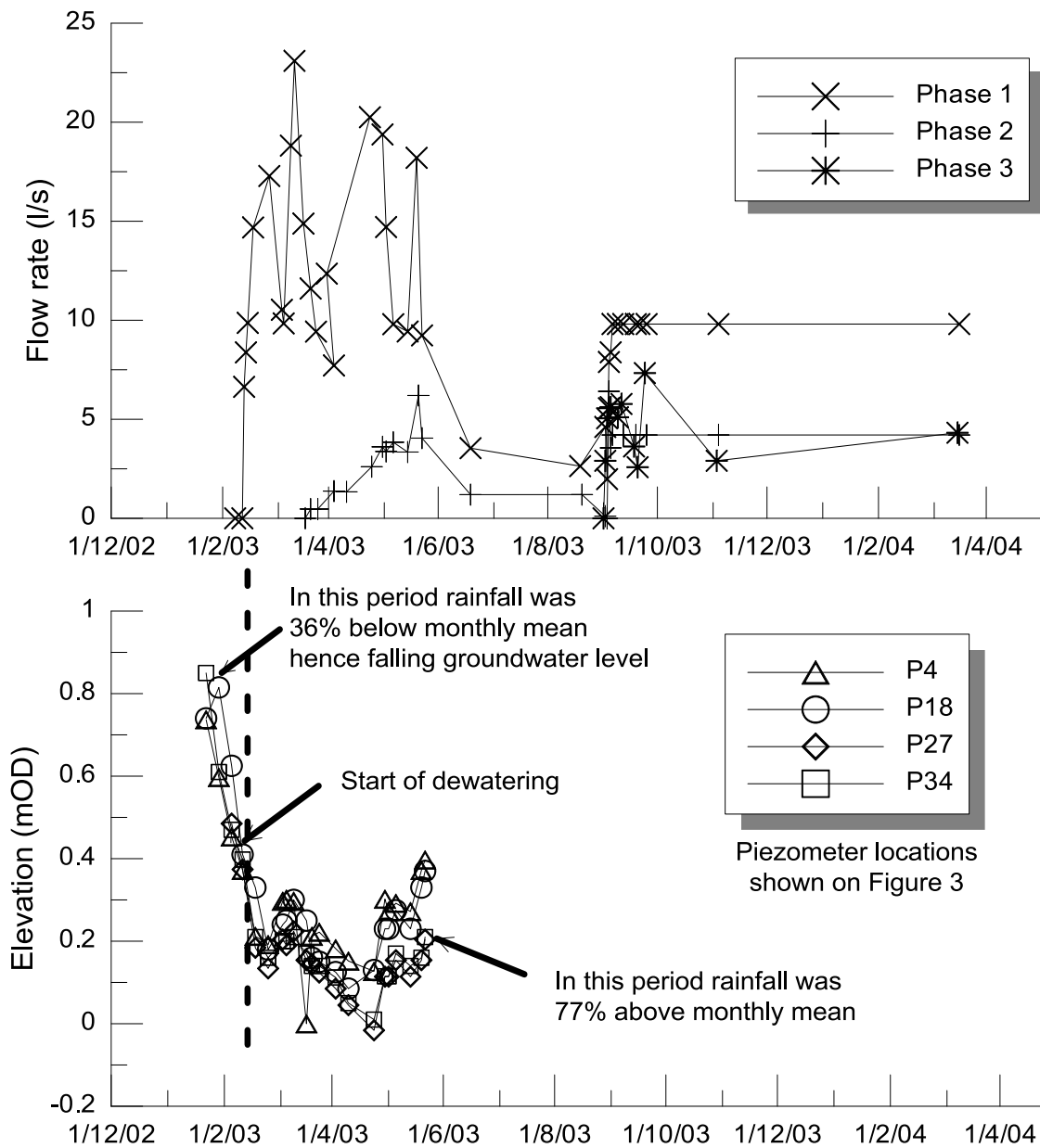


Figure 8. Details of (a) pumping and (b) groundwater levels outside site

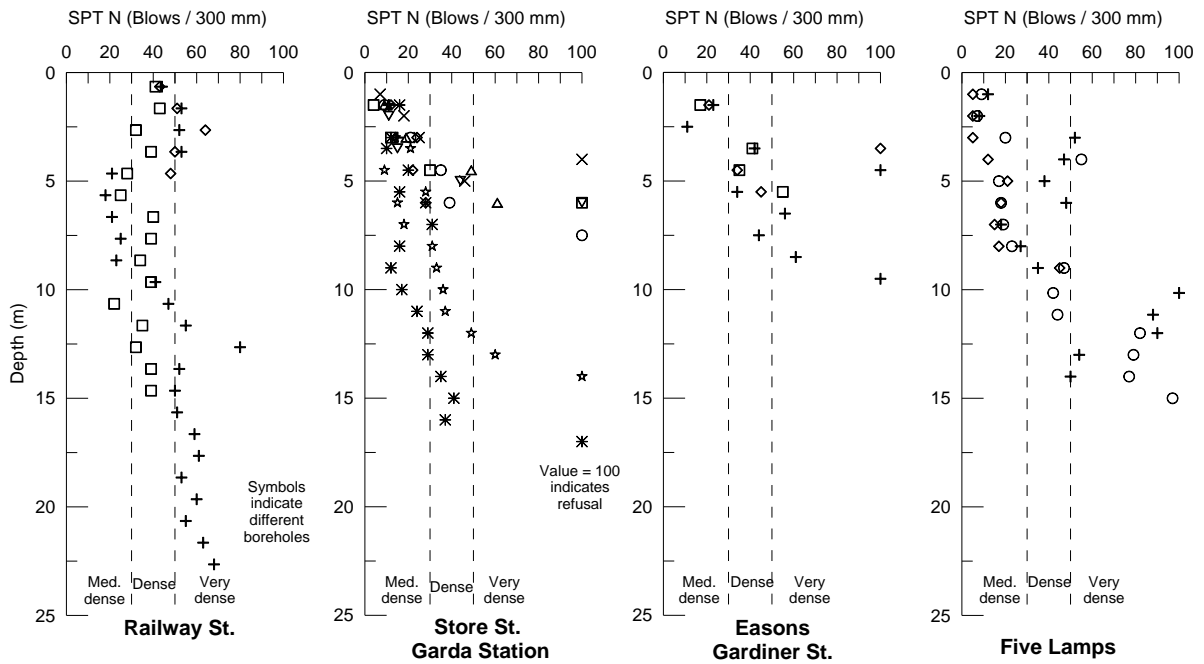


Figure 9. Standard penetration tests for Railway St. area (density categories after BS EN ISO 14688-2:2004)

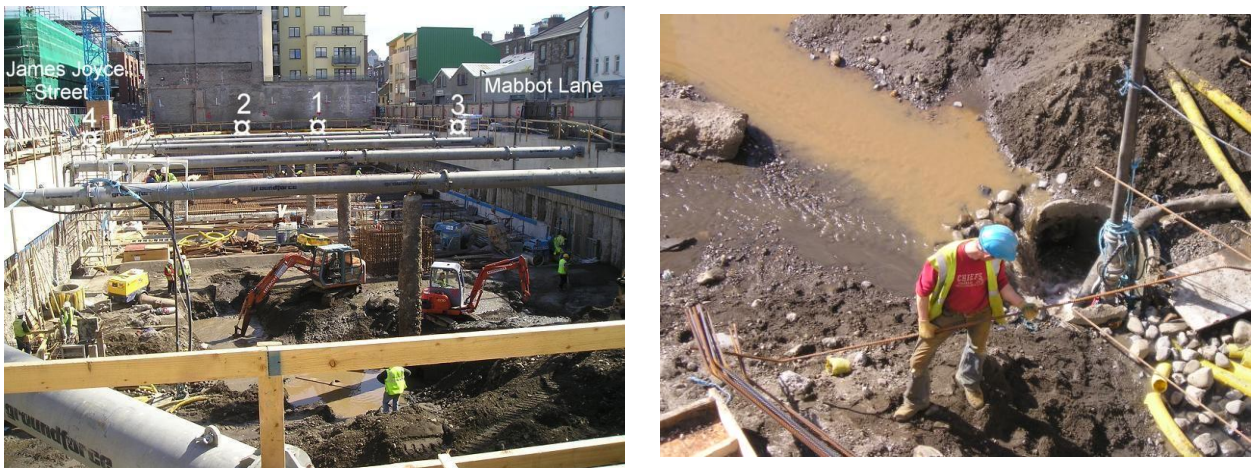


Figure 10. (a) Excavation in progress at Railway St. View is north to south. Locations of inclinometers 1 to 3 are also indicated (b) indication of dewatering effort required and nature of gravels.

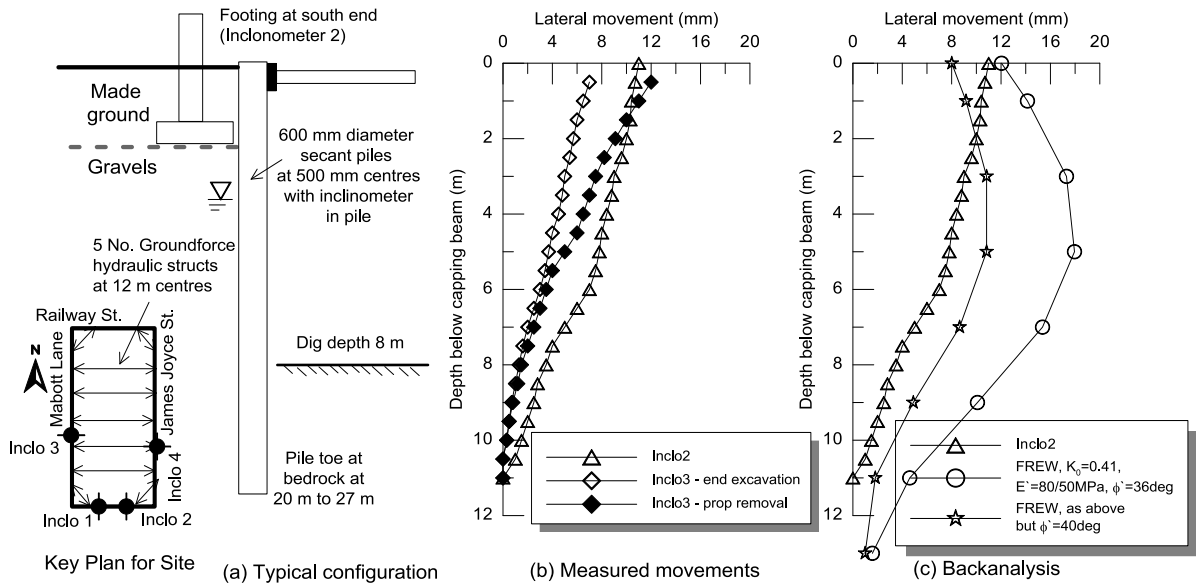


Figure 11. (a) Typical retaining wall configuration, (b) measured movements and (c) back-analysis for Railway St.

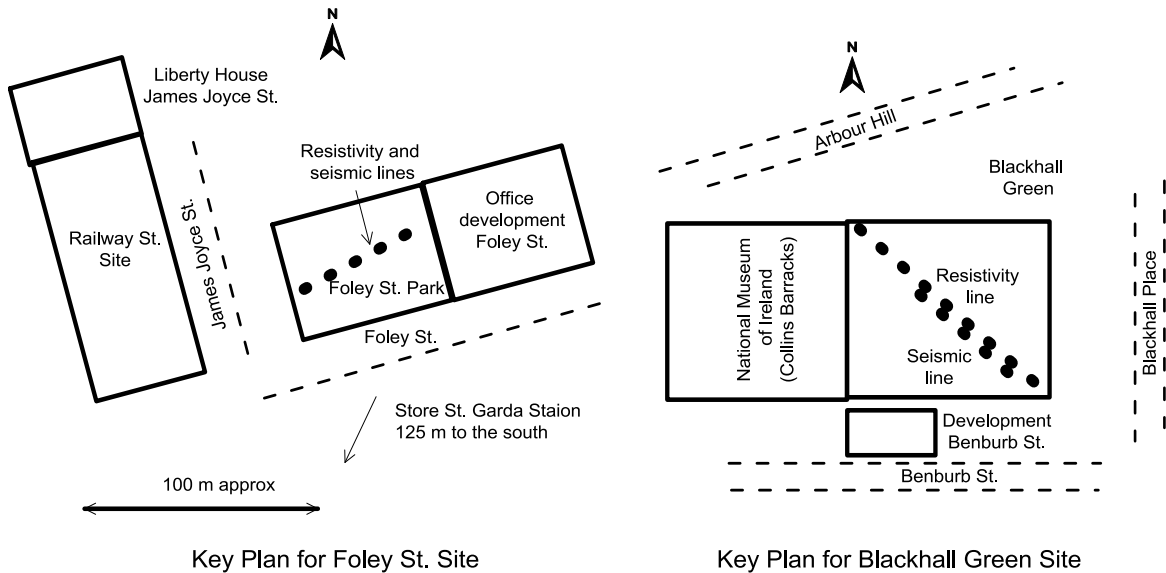


Figure 12. Location and key plans for Foley St and Blackhall Place Sites

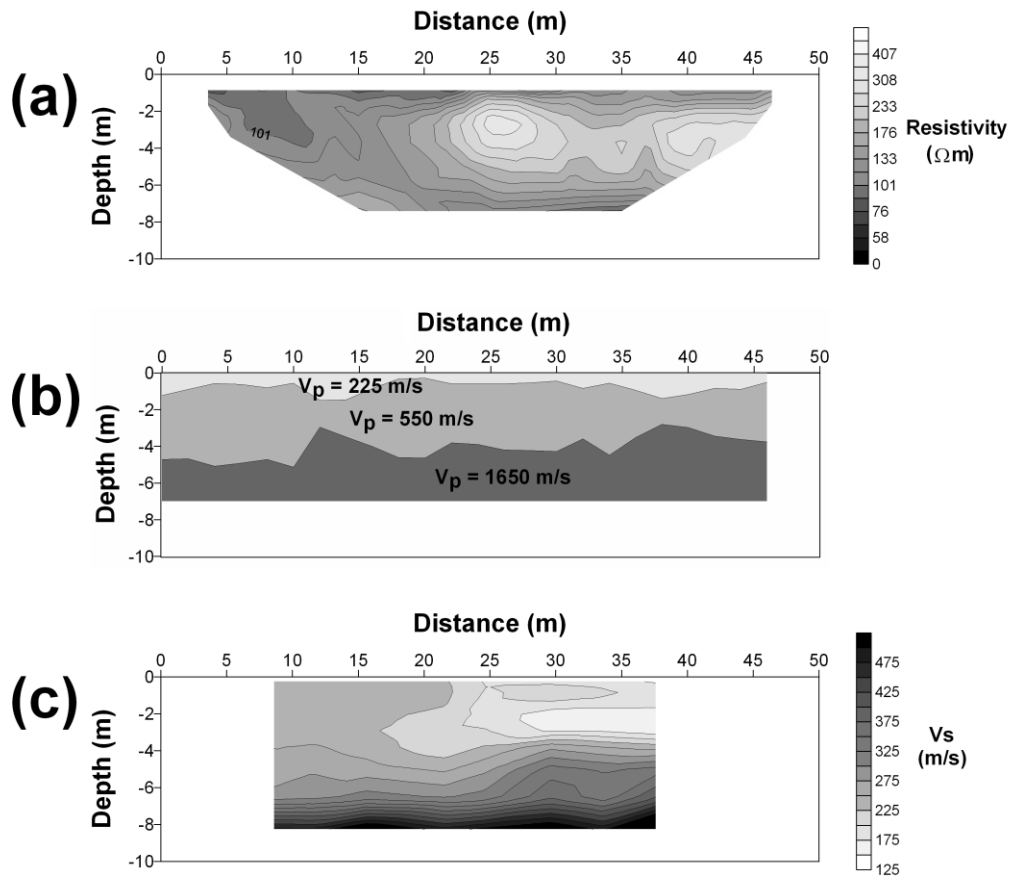


Figure 13. (a) Electrical Resistivity Tomography, (b) P wave seismic refraction and (c) Multichannel analysis of surface waves results from Foley St.

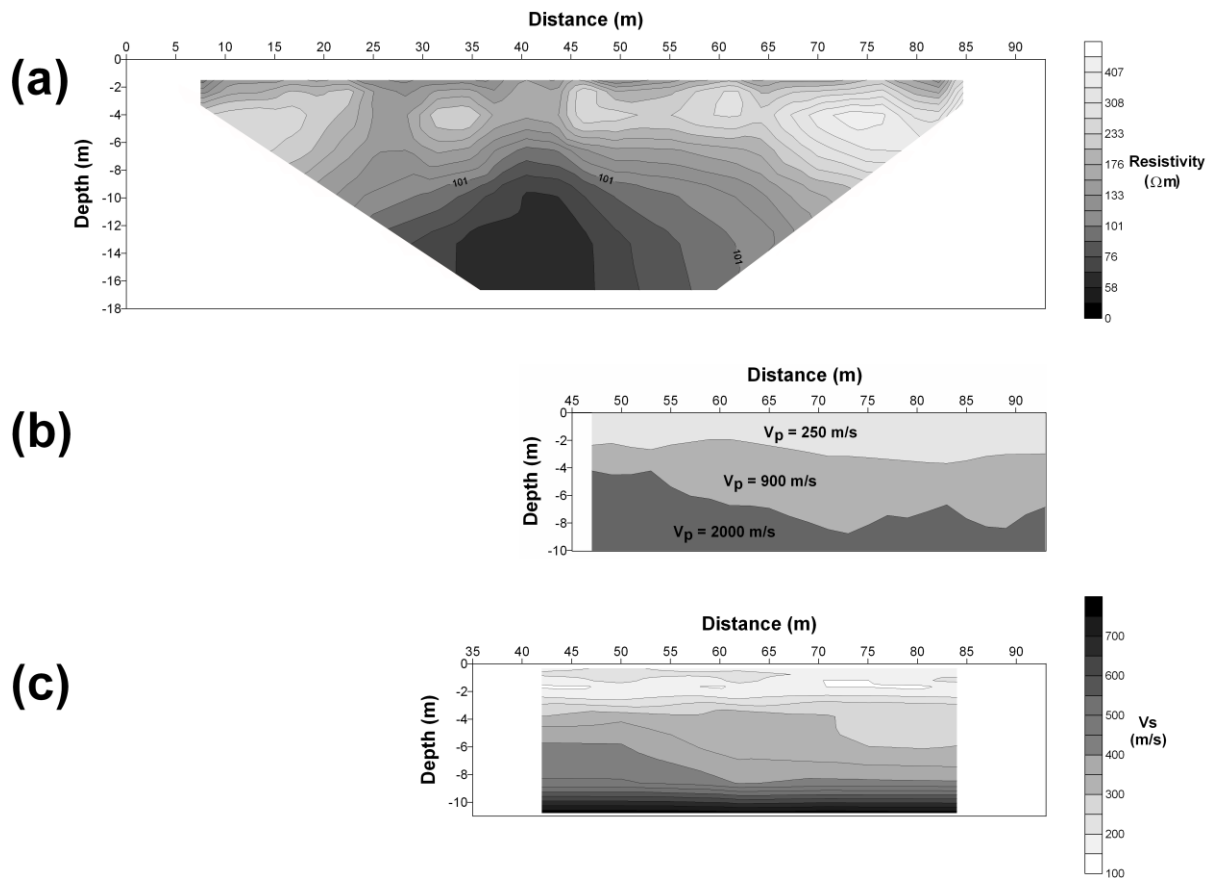


Figure 14. (a) Electrical Resistivity Tomography, (b) P wave seismic refraction and (c) Multichannel analysis of surface waves results from Blackhall Green.

End