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**IN-SITU STRENGTH CHARACTERISATION OF PEAT AND ORGANIC SOIL
USING FULL FLOW PENETROMETERS**

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ABSTRACT

Full flow penetrometers have been shown to overcome problems experienced with the cone penetrometer measuring resistance in very soft peat and organic soil, and give a much more uniform measure of resistance than the cone in fibrous peat. However, at present there is no guidance on the interpretation of strength parameters in these soils using the T-bar and ball. This paper examines the results of tests using these devices at two research sites in the Netherlands in conjunction with high quality Sherbrooke sampling for laboratory testing. In fibrous peat, the T-bar and ball provided a more uniform measure of resistance with a lower degree of scatter than the cone. The in-situ testing results have been compared to the laboratory tests to assess the range of resistance factors relating penetration resistance to the undrained shear strength (s_u) and have been shown to occupy a lower range of values than the cone penetrometer. However, penetration tests in these soils are likely to be influenced by partial drainage effects and this should be considered during testing and the subsequent interpretation of results. Recommendations are made for the use of full flow penetrometers to obtain strength parameters in these soils.

Keywords: Peat, organic soil, in-situ testing, shear strength, full flow penetrometers

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INTRODUCTION

Peat, or bog as it is commonly referred to in Ireland, is a soil that consists of partly decomposed or undecomposed organic material derived from plants and micro bacteria. Different names are used to refer to this material throughout the world, for instance the term ‘Muskeg’ is used in Canada and ‘Veen’ is used in the Netherlands. While different terms are used to describe this soil, they all share the common characteristics of high water content and compressibility, low bulk density and stiffness but a significant level of strength for a soil which has such a low solid fraction. The presence of peat and organic soils is often problematic to engineering projects which encounter them, often resulting in large settlements under load, long term creep and resistance issues, while adequate material models for these soils in particular are lacking.

While it would be preferable to avoid the problems posed by these difficult soils, this is not always feasible. In the Netherlands, organic and peat subsoils coincide with the most densely populated areas and these materials must be dealt with during the construction of dykes and waterways, road and railway embankments, excavations and tunnels (Den Haan and Kruse 2007). In Ireland, the gradual shift to renewable sources of energy and the need to exploit the high wind speeds available has led to increasing numbers of wind farm developments in upland settings, often in mountainous peatland environments. The problem of maintaining stability during development on this very

weak soil was emphasised by a devastating landslide which occurred during one such development at Derrybrien, Co. Galway in 2003 (Boylan et al. 2008a). To prevent the occurrence of future landslides and allow for the development in similar peatland settings, stability assessments are often carried out assuming fully undrained conditions using the undrained shear strength (s_u). Assuming fully undrained conditions is a cautious approach for assessing the stability of these deposits as many of the causal factors prior to and during instability are poorly understood (Boylan et al. 2008a; Warbuton et al. 2004). In Ireland, the field vane test (FVT) is widely used to obtain undrained strength parameters, with the cone penetration test being employed to a lesser extent. While these tests are routinely used to obtain strength parameters, these tests are problematic in peat and organic soils due to uncertain failure conditions around the vane circumference, which may yield misleading results (Landva 1980).

IN-SITU TESTING OF PEAT AND ORGANIC SOIL

Owing to difficulties obtaining high quality samples as well as problems with sample preparation and handling in the laboratory, in-situ methods are often preferred for assessing the strength of peat and organic soils. The field vane test (FVT) is widely used to obtain strength parameters, despite known deficiencies of this test in peat. In a comprehensive review of the vane test in peat, Landva (1980) observed that a void was generated behind the blade into which the compressed peat in front of the blade drained resulting in a modified peat. Landva (1980) and Helenelund (1967) also reported that a cylindrical shear surface occurred at a diameter 7 mm to 10 mm outside the edge of the blade and the length of the vane shear face was shorter due to the compression / void mechanism described above. Therefore the assumed failure surface, from which the mobilised strength is calculated, is quite different to the actual failure surface. Landva

(1980) concluded that the field vane test is “of little engineering value in fibrous material” and is also not suitable for organic soils. Helenlund (1967) similarly concluded that the “test is not reliable in fibrous peat”.

Cone penetration testing with pore pressure measurements (CPTU), which provides a profile of resistance and pore pressure generation with depth, is widely used in these soils particularly for material identification purposes and to a lesser extent for interpreting strength parameters. Measured tip resistances (q_c) are low, typically between 0.1MPa to 0.3MPa and some standard cones (10cm² bearing area) have difficulty measuring resistances at this level, which can be explained by the sensitivity of cones and temperature effects (Boylan et al. 2008b). In these soft soils, corrections to the measured resistance (q_c) for pore water pressure effects and overburden resistance can be a large percentage of the measured resistance (Long and Gudjonsson 2004). In addition to this, the measured resistance can be very scattered due to the variable interaction with fibres which adds uncertainty to the value of mobilised resistance that should be used to determine the representative strength at a particular depth.

Lunne et al. (1997) point out that in peat and organic soil, penetration is likely to occur under partially drained conditions, which would result in a measured resistance higher than that from undrained penetration. Viergever (1985) performed cone penetration tests in organic soil using cones with large projected areas (50 and 100 cm²) and the traditional cone (10 cm² projected area). The results of tests using the largest cone measured resistances that were 30- 45% lower than the 10 cm² cone. The larger cones measured a more homogenised soil volume resulting in a substantial reduction in the standard deviation of the penetration resistance. Dimensional analysis shows that the degree of partial drainage during continuous penetration is controlled by the normalised velocity V , defined as (Finnie and Randolph 1994):

$$[1] \quad V = \frac{vd}{c_v}$$

where v is the penetration rate, d is the diameter of the penetrometer, and c_v is the coefficient of consolidation of the soil. The tests conducted with the larger cones would therefore have a greater normalised velocity (V) and reduced effects of partial drainage on the mobilised resistance. Therefore, the use of larger probes appears beneficial in increasing the sensitivity of measurements and reducing the effects of partial drainage when consolidation is dominant.

FULL FLOW PENETROMETERS

Full flow penetrometers (see Figure 1) have developed initially from the miniature T-bar used in the centrifuge to improve the measured resistance over cone penetrometers (Stewart and Randolph 1991) and was later scaled up for use in offshore site investigations (Randolph et al. 1998). The ball penetrometer was then developed to reduce the chance of the load cell being subjected to bending moments induced from non-symmetric resistances along the T-bar (Watson et al. 1998). These penetrometers have several advantages over the standard cone. Improved accuracy is obtained in soft soils due to the larger projected area – typically between 28 - 100 cm² compared to the standard 10 - 15 cm² of the cone. This results in improved resolution and reduced sensitivity to any load cell drift and temperature effects. As the soil is able to flow around these probes, the overburden pressure is equilibrated above and below the probe except for the area of the shaft. This means that the measured resistance requires minimal adjustment to provide the net resistance compared to possibly large corrections to the measured resistance of a cone penetration test. The lower level of corrections to be applied to data obtained by the T-bar and ball gives a higher degree of confidence in

its net resistance compared to the cone in soft soils. One of the advantages of these probes is the availability of plasticity solutions based on an ideal perfectly plastic Tresca soil model, which provide upper and lower bound resistance factors relating to undrained shear strength (s_u) (Martin and Randolph 2006; Randolph and Houlsby 1984; Randolph et al. 2000). While these resistance factors are for perfect soil conditions (i.e. strain softening, soil anisotropy and viscous rate effects are not accounted for), they provide a sound theoretical basis to convert penetration resistance to undrained shear strength. Theoretically, the T-bar and Ball occupy much narrower ranges of resistance factors than the cone as the latter are influenced by soil stiffness (or rigidity index, I_r) and in-situ stress ratio ($\Delta\sigma = (\sigma_{v0} - \sigma_{h0})/2s_u$, where σ_{v0} and σ_{h0} are the in-situ total vertical and horizontal stresses respectively) and as a result vary widely (Teh and Houlsby 1991). Analytical expressions for the T-bar and ball, that adjust the ideal resistance factor from plasticity solutions to account for the opposing effects of strain softening and viscous strain rate effects during penetration have also been developed (Einav and Randolph 2005). Numerous experimental studies (e.g. (Boylan et al. 2007; Long and Gudjonsson 2004; Lunne et al. 2005; Randolph 2004) have been carried out, predominantly in clayey soils which have confirmed that the T-bar and Ball occupy a narrower range of resistance factors than the cone and are more uniform with depth. Long et al. (2010) also found this to be the case for organic soils along the Thames estuary in London. The influence of soil sensitivity and viscous strain rate effects on penetration resistance, and the effect of these factors on the appropriate resistance factor relating to undrained shear strength (s_u) has also been demonstrated from field studies (Boylan et al. 2007; Randolph and Andersen 2006; Yafrate and DeJong 2006). A number of studies have also shown that the T-bar and ball can overcome problems with the cone measuring resistance in soft peat and organic soils, providing a much more

uniform measure of resistance than that obtained with the cone (Boylan and Long 2006; Long and Gudjonsson 2004; Oung et al. 2004). This can be explained by the larger failure surface mobilised during penetration, thus averaging the local effects by which the smaller cone can be more influenced. However, there have been no extensive studies carried out on the required resistance factors to relate to undrained shear strengths (s_u) measured in the laboratory, or guidance for choosing an appropriate resistance factor for these soils. The use of cones larger than the standard 10 cm² cone penetrometer has already been demonstrated to reduce the effects of partial drainage on penetration resistance. The larger bearing area of the T-bar and ball penetrometer have comparable effects, while the rounded shape of the full flow penetrometers allows for analytical solutions for simplified soil conditions.

This paper examines the use of full flow penetrometers (T-bar and ball) to obtain strength parameters in peat and organic soils. Testing was carried out at two sites in the Netherlands using a T-bar, ball and cone penetrometer, in conjunction with high quality sampling of the subsoils involved. The results of these in-situ tests were compared to shear strengths obtained from laboratory tests and the range of resistance factors to relate the two has been examined. The use of variable rate penetration tests to identify if penetration is taking place under undrained or partially drained conditions is also examined, drawing on the results of a preliminary study carried out at a site in Western Ireland. The relative flow mechanism of peat and organic soils from the front to the back of these probes is very complex (e.g. partial drainage, fibre reinforcing effects) and requires further research. The presented experimental resistance factors incorporate these effects, although it is not yet possible to distinguish and model the various phenomena. Lastly, recommendations have been made for the use of full flow penetrometers to obtain strength properties of these soils.

SITES

The main investigations described in this paper were carried out at two research sites located near Vinkeveen and Bodegraven in the Netherlands (See Figure 2). A preliminary study on the use of variable rate penetration tests in peat was also carried out at a site near Loughrea in Western Ireland. The locations at these sites where this research was undertaken, were chosen for their uniformity, evaluated using multiple penetration tests to assess the repeatability of the penetration profile.

Loughrea Site

The Loughrea site is located in the west of Ireland, approximately 125 km west of Dublin. The site is located in the Slieve Aughty Mountains roughly 11km south of the town of Loughrea. The site is a blanket bog peat complex that covers a vast area of the mountains in the locality. The site itself has a gentle slope of 2° and is located at an altitude of 345 m OD (Malin).

Figure 3 shows the basic geotechnical properties of this site with depth. The state of peat decomposition was determined using the Von Post scale (von Post and Granlund 1926) which is based on visual classification and a hand squeeze test where the nature of the expelled material is used to determine the decomposition level. Hobbs (1986) provides a description of the methodology used to determine the level of decomposition using this method. While the full methodology includes a description of several properties (e.g. degree of wetness, smell, plasticity etc.), this classification scheme is often used to classify the level of decomposition alone. The scale classifies the level of decomposition by determining a humification number (H) between 1 and 10, where H1 refers to a soil which has undergone no decomposition and H10 is a soil which is completely decomposed. While the term humification strictly means the content of

humic substances in the soil, it is usually taken to mean in this context, the same thing as decomposition. Although this method is rather qualitative, it provides useful insight into the extent of decomposition and the relative variability of peat at a particular location. Engineering properties such as permeability also decrease with increasing decomposition (Ingram 1983). Peat decomposition increases from H4/H5 above 1 m depth to a peak of H8 at 2 m before reducing again to H6/H7. Significant amounts of wood fragments were noted at 2 m and at 3.5 m. Water contents range generally from about 1000% to 1500% with some local peaks around 1700%. Bulk densities are relatively uniform with depth with an average value of 1060 kg/m^3 . Organic contents determined from the loss on ignition at 440°C are on average 95%, which according to the scheme of Landva et al. (1983), classifies the soil as 'peat'.

Vinkeveen Site

The Vinkeveen site, which is approximately 18 km north of Utrecht, is located on land adjacent to the A2 motorway between Amsterdam and Maastricht at an elevation of 1.6 m NAP (Amsterdam Ordnance Datum). The Vinkeveen site is part of the Rhine delta system and the geomorphological formation of this deltaic plain was highly influenced by climate changes and changes in fluvial deposition during the Late Weichselian and Holocene period (Berendsen 2005). The largest formation of peat occurred during the early Atlantic (5000-7000 yr B.P.) with straight river patterns and large-scale crevassing as a result of Holocene sea level rise. This was later followed by a meandering fluvial formation. The erosive influence of the sea was prevented by closure of the coastline, resulting in a continued gradual formation of peat.

Figure 4(a) summarises the basic geotechnical properties of the peat at the Vinkeveen site. For the purpose of this research, sampling focused on the regions between 2 - 2.5 m

and 4 - 4.5 m. The site consists of 5.1 m of peat and peaty organic soil which overlies silty clay. In the region between 2 – 2.5 m, the level of decomposition is H6/H7 and H5/H6 in the region from 4 - 4.5 m. This indicates that the soil at this site is moderately to strongly decomposed. Water contents are scattered between 500% and 750% in the shallower zone, while values in the deeper zone range from 750% - 1000%. Bulk densities are relatively uniform with depth with an average value of 1022 kg/m³. Organic contents are an average of 75% between 2 – 2.5 m, and an average of 86% in the region from 4 – 4.5 m. According to the classification scheme suggested by Landva et al. (1983) for geotechnical purposes, the soil in the shallower region would be classified as a ‘peaty organic soil’ (Organic content between 60 – 80 %) while the soil in the deeper region would be classified as ‘peat’ (Organic content greater than 80%).

Bodegraven Site

The Bodegraven site is located approximately 35 km east of Den Haag and 30 km south-west of the Vinkeveen site. The site itself is located on land adjacent to the N11 motorway between Bodegraven and Leiden at an elevation of 1.7 m NAP. This site is also part of the Rhine delta system and formed under similar conditions to the Vinkeveen site.

Figure 4(b) shows the basic geotechnical properties of the Bodegraven site. At this site, sampling focussed on the regions between 0.8 – 2 m and 4 – 5 m. The top 0.8 m of the profile consists of stiff clay, which is underlain by 6 m of peaty organic soil. The state of decomposition of the material between 1 – 1.5 m is H5/H6 on the Von Post scale while the material between 4 - 4.5 m is H6/H7 before transforming into organic clay. Below 0.8 m, wood fragments are irregularly encountered in the soil profile. Water contents are scattered from 270% to 330% in the region between 1 – 1.5 m, while in the

deeper region, water contents range from 200% to 300%. Bulk densities are relatively uniform with an average value of 1153 kg/m^3 . Organic contents are an average of 46% in the region between 1 – 1.5 m, and an average of 39% in the deeper region. According to Landva et al. (1983), the soil is classified as an ‘organic soil’ as the organic contents are in the range between 4 – 59%.

IN-SITU TESTING AND SOIL SAMPLING

Cone penetration tests

Cone penetration tests with pore pressure measurements (CPTU) were carried out using a standard 10 cm^2 (35.7 mm diameter), 60° apex cone with a pore pressure filter at the u_2 position at the shoulder of the cone. The cone has an unequal area ratio (a) which was measured in a pressure chamber, of 0.75. The cone penetration tests were carried out in accordance with the ISSMGE reference test procedure (ISSMGE 1999). To avoid the effects of temperature differentials between the surface and the ground, the cone was equilibrated to the ground temperature by immersing the cone in a bucket of water at the same temperature as the ground prior to each test (Boylan et al. 2008b). Testing was conducted at the standard penetration rate of 2 cm/sec.

The measured resistance, q_c is corrected for pore pressure effects using Equation 2;

$$[2] \quad q_t = q_c + (1 - a)u_2$$

where q_t is the corrected cone resistance. The net resistance (q_{net}) is then calculated by subtracting the total vertical stress (σ_{v0}) due to the overburden as shown in Equation 3;

$$[3] \quad q_{\text{net}} = q_t - \sigma_{v0}$$

The friction ratio (R_f) is calculated as a percentage using Equation 4;

$$[4] \quad R_f = \frac{f_s}{q_t} \times 100$$

where f_s is the measured sleeve resistance. The pore pressure parameter (B_q) is calculated using Equation 5;

$$[5] \quad B_q = \frac{\Delta u}{q_{net}} = \frac{u_2 - u_0}{q_{net}}$$

where u_0 is the hydrostatic pore pressure.

T-bar and ball penetrometer testing

The T-bar used in this research is 250 mm long and 40 mm diameter giving a 100 cm² bearing area and a shaft to penetrometer area ratio (A_s/A_p) of 0.1. The surface of the T-bar was lightly sand blasted to give a slightly roughened surface. T-bar tests were carried out by unscrewing the cone of the piezocone and replacing it with the T-bar. Testing was conducted in the same manner as a standard piezocone test.

The ball penetrometer has a 113 mm diameter, giving a bearing area of 100 cm² and a shaft to penetrometer area ratio (A_s/A_p) of 0.1. The surface of the ball was also lightly sand blasted similar to the T-bar. Ball penetrometer tests were conducted by unscrewing the cone of the piezocone and replacing it with the ball.

The net penetration resistance ($q_{T\text{-bar}}$ or q_{ball}) is calculated in the same manner as the cone using Equation 6 except for the addition of an area ratio (A_s/A_p), which is the ratio of shaft area (A_s) to the bearing area of the probe (A_p). Also, as the pore pressure generated during penetration is often not measured on full flow penetrometers, the

hydrostatic pore pressure (u_0) is used to correct q_c for pore pressure effects. In peat and organic soil where the excess pore pressure generated during penetration (Δu) is low, the error due to the use of u_0 would typically be less than 2% of the net resistance ($q_{T\text{-bar}}$ or q_{ball}). Therefore, the effect of using u_0 is deemed insignificant.

$$[6] \quad q_{T\text{-bar}} \text{ or } q_{\text{ball}} = q_c - [\sigma_{v0} - (1-a)u_0] \frac{A_s}{A_p}$$

Testing at the sites in the Netherlands was carried out at penetration rates of 0.2 cm/sec, 2 cm/sec and 10 cm/sec to assist in identifying the drainage condition under which penetration is taking place at different penetration rates. This range of penetration rates represents the minimum and maximum rates that were available on the penetration rig and is similar to the range of rates which most commercial penetration rigs are capable of.

A preliminary study (Boylan 2008) using the T-bar was conducted at the Loughrea site in western Ireland to assess if penetration in peat at various rates would result in undrained penetration. Tests were conducted at rates of 0.5, 2 and 5 cm/sec to a depth of 4 m. Two tests were carried out at 2 cm/sec, one at the beginning and one at the end of testing to confirm that differences in measured resistance were not due to natural variation of the peat. Figure 5(a) show the results of all the tests. Above 2 m, all the T-bar tests have similar resistances with no particular trends apparent. Below 2 m, there is a clear distinction with the tests conducted at 0.5 cm/sec and 5 cm/sec having higher resistances than the tests conducted at 2 cm/sec. Figure 5(b) shows the variation of the normalized net resistance ($q_{T\text{-bar}}/\sigma'_{v0}$) with penetration rate (v) at various depths in the profile. It is accepted that it would be more appropriate to present the data in terms of the normalised velocity (V), however coefficient of consolidation (c_v) data are not

available for this site. At 1 m, $q_{T\text{-bar}}/\sigma'_{v0}$ is constant with penetration rate except for a slight reduction at a rate of 5 cm/s. At the other two depths, there is a distinct drop in $q_{T\text{-bar}}/\sigma'_{v0}$ at 2 cm/sec.

For the lower two depths (2.5 m & 3 m), the reduction in $q_{T\text{-bar}}/\sigma'_{v0}$ for penetration of the T-bar at 2 cm/sec, suggests that the mobilised resistance may be less effected by partial drainage than for penetration at 0.5 cm/sec. The increase in $q_{T\text{-bar}}/\sigma'_{v0}$ for penetration at 5 cm/sec may be an indication of the dominance of viscous strain rate effects over consolidation effects (Chung et al. 2006) at this penetration rate. The difference between the upper depth level (1 m) and the lower depths (2.5 m & 3 m) appears to be due to the transition in the peat at 2 m, with the peat below this depth being more strongly decomposed and less permeable than the peat above this transition. In the shallow peat (1 m) the slight reduction of $q_{T\text{-bar}}/\sigma'_{v0}$ at 5 cm/sec may indicate that partial drainage effects are beginning to decay at this rate. Insight into the mechanisms controlling the mobilised resistance at various penetration rates would be greatly enhanced with measurement of the pore pressure regime around the penetrometer to identify the drainage characteristics during penetration. However, these tests demonstrate that the mobilised resistance in peat is influenced by the penetration rate and thus the drainage conditions and this should be considered when interpreting the results of penetration tests in peat and organic soils.

Soil sampling

High quality block samples were obtained using the Sherbrooke block sampler (See Figure 6a). The sampler, described by Lefebvre and Poulin (1979), is a specially designed open cage carving tool which carves a small annulus with an outside diameter of 410 mm using three cutters positioned 120° apart, at a low rotation speed while

applying a small volume of water through the jets at the bottom to flush the carved material. For sampling at the Vinkeveen site, the rotation speed of the sampler was kept at ~2-3 rotations/min and the water pressure of the jets kept at 1 bar to prevent clogging. At the Bodegraven site, a rotation rate of ~5 rotations/min and water pressure between 2-5 bar was used. When the sampler has carved a block sample, the three horizontal knives at the bottom of the sampler, held in open position during sampling by pins and springs, are closed by releasing the torque spring and the block sample is cut loose from the soil by 5-10 rotations. The block sample, which rests on the three knives, is carefully brought to the surface while slowly rotating. The sample is then placed on a bottom plate, wrapped in cling film, aluminium foil, and again cling film and completely waxed with 90% paraffin and 10% bee wax by weight (See Figure 6b). Mathijssen et al. (2008) provides further details about the sampling procedures adopted to obtain these samples.

LABORATORY TESTING

Triaxial compression tests

Anisotropically consolidated undrained triaxial compression (CAUC) tests were carried out using an advanced stress path system developed by GDS Instruments Ltd, employing a Bishop & Wesley hydraulic triaxial cell (Bishop and Wesley 1975). Tests were conducted on 70 mm diameter samples with height to diameter (h/D) ratio of 1.7. To enable consolidation to low effective stresses (typically between 2 – 10 kPa with a resolution of 0.5 kPa), a differential pressure transducer (DPT) was installed between the cell pressure and back pressure lines to control the differential pressure. Other measures such as thin membranes (0.2 mm thickness) and lubricated end platens were used to reduce the levels of correction to be applied to data and the effects of end

restraint. Test specimens were anisotropically consolidated in two stages and ratios of horizontal to vertical effective stress (K_0) were estimated from the relationship with bulk density suggested by Den Haan and Kruse (2007). Therefore, K_0 values between 0.3 – 0.5 were used for specimens from the Vinkeveen site and between 0.3 – 0.4 for specimens from the Bodegraven site. The exact K_0 values used in each test depended on the vertical and horizontal effective stresses applied, which were accurate to a resolution of 0.5 kPa. Following consolidation, specimens were sheared at a rate of 10% axial strain per day. The undrained shear strength (s_u) was assigned at the peak shear stress or the mobilised shear stress at 15% axial strain (ϵ_a) in tests where no peak occurred. This cut-off is slightly higher than the 10% axial strain cut-off that is often used for clays (Lunne et al. 2006) but reflects the larger strains required to mobilise strength in these soils.

Direct simple shear tests

Direct simple shear (DSS) tests were carried out using the UCD-DSS device, developed at University College Dublin, which was specially developed for this research (Boylan and Long 2009). This device was designed to enable testing of peat specimens at the required low effective stresses and allow visual monitoring of the specimen during shearing. Tests were conducted on 70 mm square specimens with a typical height of 20 mm. Two sides of the specimen are enclosed by transparent polymethyl methacrylate (PMMA) side walls which allow the deformation of the specimen during shear to be monitored using a digital camera. The digital images were analysed using the Particle Image Velocimetry (PIV) (White et al. 2003) image analysis technique, which allows identification of yielding in the specimen or slippage of the specimen during shear. Tests were carried out as K_0 consolidated constant volume DSS (CVDSS) tests,

whereby the test is run at a slow rate to prevent the development of pore pressures and the vertical load on the specimen is continually varied during shearing to maintain a constant height and in turn a constant volume. The change in vertical stress which occurs during the test is assumed equal to the change in pore water pressure which would have occurred if the test was truly undrained (Dyvik et al. 1987).

For these tests, specimens were sheared at a rate of 4% shear strain per hour. The undrained shear strength (s_u) was assigned at the peak shear stress or at the mobilised shear stress at 30% shear strain (γ) in tests where no peak occurred. Similar to the triaxial tests, a larger cut-off than the 15% shear strain used for clays (Lunne et al. 2006) has been employed here due to the larger strains required to mobilise strength in these soils.

RESULTS

Laboratory testing

Figure 7(a) and (b) show the results of both triaxial compression and direct simple shear tests for the Vinkeveen and Bodegraven sites respectively. Table 1 and Table 2 summarise basic properties and the consolidation stresses for each of the tests and the subsequent mobilised shear strengths. For the Vinkeveen site, the undrained shear strength (s_u) obtained from triaxial tests ranges from 4.4 - 8.2 kPa while the s_u from DSS tests ranges from 3.5 - 7.6 kPa. At the Bodegraven site, the s_u from triaxial compression tests ranges from 8.9 – 12 kPa while for DSS tests, s_u ranges from 7.2 – 8.7 kPa. The ratio of strengths between DSS and triaxial compression ($s_{u-DSS}/s_{u-triaxial}$) is in the range of 0.69 – 0.86 for the Vinkeveen site and between 0.7 – 0.79 for the

Bodegraven site which reflect the anisotropic nature of the strength and the different stress paths followed in both tests.

In-situ testing – standard tests

Cone, T-bar and ball penetrometer tests were carried out at both research sites at the standard rate of 2 cm/sec to examine the relationship between penetration resistances for the various penetrometers. Figure 8 shows the results of two cone penetration tests carried out at the Vinkeveen site. For the net resistance (q_{net}), the individual results are shown as well as the average result. Except for the initial 0.5 m, net resistances (q_{net}) are between 0.1 to 0.2 MPa and are uniform with depth. Friction ratios (R_f) reduce from about 4% close to the surface to approximately 2% at 6 m. The resistances that were measured on the sleeve are close to the accuracy of the strain gauges and may not be accurate. The pore pressure readings are close to hydrostatic throughout, resulting in B_q of 0 except for some minor deviations, indicating that penetration is taking place under partially drained to drained conditions (Schnaid et al. 2004). Figure 9(a) shows the net resistances ($q_{\text{T-bar}}$ & q_{ball}) measured during penetration for the T-bar and ball tests conducted at the Vinkeveen site. Individual tests are shown using dashed lines, while the average for each penetrometer is shown using bold lines. The trends with depth between both penetrometers are variable. In the most scattered region, from 1 to 2 m, the average T-bar resistance is higher, while below 3 m, the average ball resistance is generally greater than the T-bar. The coefficient of variation (COV), which is the ratio of the standard deviation to the mean, between individual T-bar and ball tests is 15% and 20% respectively while the COV of cone tests is 35%. The higher COV for the cone is primarily due to the more variable interaction of the relatively small cone (10 cm²) with the frequent fibres in the peat compared to the larger area being measured by the

full flow penetrometers (100 cm^2). Comparing net resistances from the cone with the T-bar and ball (Figure 9(b)), it is clear that the cone data are much more scattered. The average net resistance of the cone is broadly similar to the other penetrometers, except for the region between 1 – 3 m where the cone resistance is up to 25% higher.

Figure 10 shows the results of cone penetration tests conducted at the Bodegraven site. Except for the initial 1 m through the clay layer, net resistances (q_{net}) are typically 0.2 MPa and are uniform with depth. Some localised increases between 2 m and 4 m are due to wood layers present in the organic soil. Pore pressure readings show an excess pore pressure throughout with a B_q generally between 0 and 0.2 in the peaty organic soil above 4.5 m and increasing slightly in the organic clay below this. Similar to the Vinkeveen site, penetration is considered to take place under partially drained to drained conditions. Due to a problem with the friction sleeve during testing at this site, no friction ratio results were measured. Figure 11(a) shows the net resistances ($q_{\text{T-bar}}$ & q_{ball}) for the T-bar and ball tests conducted at the Bodegraven site. The measured resistance is similar for the two penetrometers except in the region between 2 m and 4.5 m. The ball resistance is seen to be higher close to 2.5 m while at depths between 3.5 m and 4.5 m, the T-bar resistance is higher. The variability among individual T-bar and ball tests at this site is greater than that observed at the Vinkeveen site, with a COV of 25% for both penetrometers. This is compared to a slightly lower COV of 20% for the cone at this site. This increase in scatter for the T-bar and ball is likely due to the irregular amount of wood fragments present in the soil at this site. Comparing net resistances from the cone with the T-bar and ball (Figure 11(b)), it can be seen that the cone measures a net resistance approximately 30% higher than the full flow penetrometers, except for the scattered region between 2m and 3.5m. This higher resistance is believed to reflect the differences in the failure mechanisms around the

different penetrometers. For the full flow penetrometers, the soil is able to flow around the penetrometer, albeit possibly influenced by the presence of fibres, while the cone has to displace the soil, resulting in a higher measure of resistance.

In-situ testing – variable rate tests

Variable rate penetrometer tests were carried out at both sites using the ball to assess the influence of penetration rate on the measured resistance. For these tests, the ball was penetrated constantly through the profile at rates of 0.2 cm/sec, 2 cm/sec and 10 cm/sec. Figure 12(a) shows the net resistances from these tests conducted at the Vinkeveen site. The 2 cm/sec test is the average of the tests shown in Figure 9(a). Except for the points where there is a rod change (approx. 1 m intervals) and below 2 m, the measured resistance from the test conducted at 0.2 cm/sec is greater than the standard 2 cm/sec profile. The net resistance from the test conducted at 10 cm/sec is often close to and lower than the 2 cm/sec profile. Figure 12(b) compares the net resistance normalized by the vertical effective stress ($q_{\text{ball}}/\sigma'_{v0}$) at each penetration rate (v) for three depths. It can be seen that at each depth $q_{\text{ball}}/\sigma'_{v0}$ decreases with increasing penetration rate. The decrease in normalized resistance with increasing penetration rate illustrates a decay in partial drainage effects on penetration resistance. It cannot be concluded whether the test conducted at 10 cm/sec is taking place under undrained conditions or if the penetration resistance is still partially drained. Direct comparisons can only be made to the data in Figure 5 using the non-dimensional normalized velocity (V). However, since no reliable data are available on the coefficient of consolidation (c_v) for these sites, no comparison can be made as yet.

Figure 13 shows the net resistances from the variable rate tests conducted at the Bodegraven site. With the exception of the clay layer above 1 m, there is no clear trend

with penetration rate among the three tests. Given the variability of penetration resistances experienced at this site (See Figure 11(a)) and the irregular presence of wood fragments in the soil profile, it appears that the level of variability is greater than any changes in the mobilised resistance due changes in the penetration rate. Therefore, no conclusions can be made about the effect of penetration rate on the measured resistance at this site.

Correlation between in-situ and laboratory strength parameters

For the cone penetration test, the undrained shear strength (s_u) can be calculated by dividing the net resistance (q_{net}) by an appropriate resistance factor (N_{kt}) as shown in Equation 7;

$$[7] \quad s_u = \frac{q_{net}}{N_{kt}}$$

There are few published studies which look at the range of N_{kt} factors for peat soils. Hanzawa et al. (1994) suggested the use of an N_{kt} of 10, based on an empirical relationship with s_u results from the direct shear test. However, the cone penetration profiles used to develop this relationship were not corrected for pore pressure effects using Equation 2. Den Haan and Kruse (2007) describe how measured resistances are divided by 15, without any correction for pore pressure and overburden resistance to obtain s_u . While a value of 15 is in line with some of the data presented in this paper, the presented data show that the resistance factor is not a constant value. It should be noted that disregarding of the corrections in the correlations in this reference (Den Haan & Kuse, 2007) hinders the comparability with the data in this paper.

Values of N_{kt} calculated for the Vinkeveen and Bodegraven sites are shown in Figure 14. Resistance factors have been calculated separately based on shear strengths from triaxial compression (s_{u-TC}) and direct simple shear (s_{u-DSS}). For these calculations, the average net resistances (q_{net}) over a distance of one probe diameter (35.7mm), centred on the reference depth have been used. Table 3 summarises the range of values, averages, standard deviations and the coefficients of variation (COV) obtained at both sites. Overall, N_{kt} values range from 11.1 to 44.9, with standard deviations ranging from 5.2 to 17.5 and COV's between 18 – 60.1%. The standard deviations and COV's are lowest for both strength cases at the Bodegraven site. This is surprising given the more variable nature of the soil at this site.

The net resistance measured by the T-bar and ball (q_{T-bar} or q_{ball}) is related to the undrained shear strength (s_u) using a resistance factor (N_{T-bar} or N_{ball}) as shown in Equations 8 and 9.

$$[8] \quad s_u = \frac{q_{T-bar}}{N_{T-bar}}$$

$$[9] \quad s_u = \frac{q_{ball}}{N_{ball}}$$

Resistance factors for the T-bar and ball (N_{T-bar} and N_{ball}) have been derived from plasticity solutions using a perfectly plastic Tresca soil model, assuming fully undrained, non viscous and non softening soil behaviour (Martin and Randolph 2006; Randolph and Houlsby 1984; Randolph et al. 2000). These solutions show N_{T-bar} to range from 9.1 to 11.9 and N_{ball} to range from 11 to 15.3 depending on the interface roughness of the penetrometer. The T-bar and ball used for this research were lightly sandblasted and not fully smooth and therefore the average resistance factors for an

interface roughness value of 0.4 will be used for comparisons in this paper, giving a $N_{T\text{-bar-Ideal}}$ of 10.5 and a $N_{\text{ball-Ideal}}$ of 13.

Values of $N_{T\text{-bar}}$ and N_{ball} calculated for the Vinkeveen and Bodegraven site are shown in Figure 15. For these calculations, the average net resistances ($q_{T\text{-bar}}$ and q_{ball}), from tests conducted at a rate of 2 cm/sec, over a distance of one probe diameter (40mm and 113mm respectively) centred on the reference depth have been used. Table 4 summarises the range of values, averages, standard deviations and the coefficients of variation (COV) obtained at both sites. Values of $N_{T\text{-bar}}$ range from 12.5 to 26.5 while N_{ball} values range from 10.1 to 34.7. The standard deviations about average values are lowest for resistance factors based on strengths from triaxial compression ($s_{u\text{-TC}}$) and are lowest for the Bodegraven site. Compared to the values of N_{kt} from cone penetration tests presented earlier, the standard deviation of $N_{T\text{-bar}}$ and N_{ball} values are lower, with the T-bar having the lowest values. The narrower range of resistance factors for the T-bar and ball compared to the cone and the lower COV's would make these devices more advantageous to use in site investigations in peat and organic soil.

Except for some N_{ball} values in the deepest section of the Bodegraven sites, resistance factors are generally higher than the ideal values from plasticity solutions ($N_{T\text{-bar-Ideal}}$ and $N_{\text{ball-Ideal}}$). Referring back to the trends observed at the Vinkeveen site in Figure 12, penetration at 2 cm/sec using the ball penetrometer appears to have taken place under partially drained conditions, yielding a penetration resistance higher than the undrained penetration resistance. As the T-bar has a smaller diameter than the ball, penetration of the T-bar at 2 cm/sec at this site would also have taken place under partially drained conditions. The range of resistance factors higher than the ideal undrained values from plasticity solutions can therefore be explained somewhat by partial drainage during penetration. While the variable rate penetration tests at the Bodegraven site were not

conclusive, the pore pressure parameter (B_q) for the cone penetration tests indicated that they took place under partially drained conditions and it is likely that penetration of the larger T-bar and ball penetrometers took place under similar conditions.

As the level of drainage that takes place during penetration is related to the permeability (k) which is in turn related to the void ratio (e_0) (Hanrahan 1954; Mesri and Ajlouni 2007), examination of the relationship between $N_{T\text{-bar}}$ and N_{ball} with e_0 at zero load, gives further insight into the trends of the resistance factors. Figure 16 shows this relationship for (a) $N_{T\text{-bar}}$ and (b) N_{ball} . In both cases, it can be seen that the respective resistance factors tend to increase with increasing e_0 , reflecting the higher permeability and thus high degree of drainage taking place in these soils. Therefore, the void ratio (e_0) may be a useful parameter to examine when choosing an appropriate resistance factor for tests in peat and organics soils where partial drainage may occur.

CONCLUSIONS AND RECOMMENDATIONS

This paper has examined the use of the T-bar and ball penetrometer to obtain in-situ strength parameters in peat and organic soils. The purpose of this examination is to provide guidance on the interpretation of the results of full flow penetrometer tests in these soils. In-situ testing has been carried out at two research sites in the Netherlands in conjunction with high quality Sherbrooke sampling for laboratory testing. Preliminary in-situ tests were also carried out at a peat site located near Loughrea in western Ireland. The in-situ testing results have been compared to laboratory tests to assess the range of resistance factors relating penetration resistance to undrained shear strengths obtained from triaxial compression and direct simple shear tests.

The results of in-situ testing at the two sites have shown the T-bar and ball to give a more uniform measure of resistance than the cone, with a lower degree of scatter at the Vinkeveen site. The resistance profile is more uniform than that of the cone, which may be explained by the larger volume of material mobilised during penetration and reduced sensitivity to small fibres. At the Bodegraven site, the T-bar and ball had a slightly higher degree of scatter than the cone, which appears to be due to the irregular presence of wood in the soil profile. Resistance factors relating penetration resistance to undrained shear strength for the T-bar and ball ($N_{T\text{-bar}}$ & N_{ball}) occupied a lower range of values than the comparable resistance factor for the cone (N_{kt}), coupled with lower standard deviations and coefficients of variation. In particular, the greatest correlation was found for $N_{T\text{-bar}}$ based on undrained shear strengths measured in triaxial compression ($s_{u\text{-TC}}$).

However, examination of the penetration resistance from variable rate penetration tests at the Vinkeveen site suggests that penetration took place under partially drained conditions at the standard penetration rate of 2 cm/sec and it is likely that this was also the case for the Bodegraven site. The insight that could be gained from variable rate tests in these soils is limited by the range of penetration rates available on most penetration rigs. The effects of partial drainage on measured resistance contributed to the range of resistance factors obtained for the T-bar and ball, resulting in values greater than those indicated from theoretical plasticity solutions assuming ideal undrained conditions. Examination of the empirical resistance factors with void ratio (e_0) showed the values of resistance factors to increase with increasing void ratio and thus increasing permeability, reflecting the influence of partial drainage.

Given the likely influence of partial drainage on penetration resistance in these deposits, care needs to be taken when interpreting penetration tests in these soils. Variable rate

penetration tests should be undertaken, over the widest range of penetration rates available to provide insight into the drainage conditions during penetration, and possibly the penetration rate required for undrained penetration. Theoretical resistance factors which assume undrained behaviour during penetration should not be used to interpret results of penetration tests when partial drainage takes place. Laboratory tests on high quality samples, conducted at the appropriate in-situ stresses should be undertaken to provide guidance on an appropriate resistance factor to be used in these situations. Thorough soil classification (e.g. Landva et al. (1983)) should also be undertaken to assist in the interpretation of in-situ and laboratory test results. Expanding the rates of penetration of site investigation rigs and measurement of the pore pressure during penetration will also enhance the potential of variable rate tests to provide further insight into the drainage conditions during penetration. While the effects of partial drainage during penetration and the presence of fibres in the soil both influence the measured penetration resistance and therefore the range of resistance factors, further research on their effects on developing failure mechanisms during penetration is highly recommended.

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TABLES

Table 1. Summary of Triaxial Compression (TC) Tests

Site	Test Specimen Depth	Bulk Density ρ	Water Content w	Consolidation Parameters			Undrained Shear Strength s_{u-TC}
				Vertical Effective Stress σ'_{vc}	Horizontal Effective Stress σ'_{hc}	$K_0 = \sigma'_{hc}/\sigma'_{vc}$	
	(m)	(Mg/m ³)	(%)	(kPa)	(kPa)	(-)	(kPa)
Vinkeveen	2.09	1.005	707	5	2	0.40	4.4
	2.1	1.03	679	5	2	0.40	6.3
	4.61	1.008	879	6	3	0.50	8.0
	4.61	1.017	980	6	3	0.50	8.2
Bodegraven	1.13	1.106	298	7	3	0.43	8.9
	1.14	1.173	303	7	3	0.43	11.2
	4.15	1.174	244	10	4	0.40	10.4
	4.15	1.15	251	10	4	0.40	12.0

Table 2 – Summary of Direct Simple Shear (DSS) Tests

Site	Test Specimen Depth	Bulk Density ρ	Water Content w	Consolidated Vertical Effective Stress σ'_{vc}	Undrained Shear Strength s_{u-DSS}
	(m)	(Mg/m ³)	(%)	(kPa)	(kPa)
Vinkeveen	2.02	1.021	655	5	3.9
	2.15	1.034	706	5	3.5
	4.55	1.037	924	6	7.6
	4.68	1.018	914	6	6.4
Bodegraven	1.13	1.352	297	7	8.7
	1.22	1.298	287	7	7.2
	4.15	1.233	222	10	7.8

Table 3 – Summary of N_{kt} Factors

Site		N_{kt}	
		S_{u-TC}	S_{u-DSS}
Vinkeveen	Range	11.1-31.2	13.7-44.9
	Avg.	19.6	29.1
	Std. Dev.	10	17.5
	COV (%)	51.2	60.1
Bodegraven	Range	14.9-31	22.9-32.3
	Avg.	22.2	29
	Std. Dev.	7.5	5.2
	COV (%)	33.6	18

Table 4 - Summary of N Factors for T-bar & ball

Site		N_{T-bar}		N_{ball}	
		S_{u-TC}	S_{u-DSS}	S_{u-TC}	S_{u-DSS}
Vinkeveen	Range	12.5-19.8	13.1-26.5	13.2-28.1	14.1-34.7
	Avg.	14.7	19.5	18.6	23.6
	Std. Dev.	3.4	6.1	7.0	9.8
	COV (%)	23.4	31.1	37.5	41.3
Bodegraven	Range	12.6-16.8	16.3-22.3	10.1-19	15.5-21.7
	Avg.	15.0	18.9	13.9	18.9
	Std. Dev.	1.9	3.1	3.9	3.1
	COV (%)	12.4	16.3	28.3	16.6

FIGURES

Figure 1. Cone penetrometer with full flow penetrometers, T-bar and ball	37
Figure 2. Map showing sites in The Netherlands and Ireland	37
Figure 3. Basic soil properties at Loughrea site	38
Figure 4. Basic soil properties (a) Vinkeveen site (b) Bodegraven site.....	39
Figure 5. Variable rate T-bar penetrometer tests at Loughrea site (a) Net resistances (b) Normalized resistances at various depths (from Boylan (2008))	40
Figure 6. (a) Sherbrooke block sampling (b) Waxed block.....	41
Figure 7. Laboratory strength parameters (a) Vinkeveen site (b) Bodegraven site	41
Figure 8. Cone penetration tests at Vinkeveen site.....	42
Figure 9. Vinkeveen site (a) T-bar & ball net resistances (b) Comparison of net resistance from T-bar, ball and cone	42
Figure 10. Cone penetration tests at Bodegraven site.....	43
Figure 11. Bodegraven site (a) T-bar & ball net resistances (b) Comparison of net resistance from T-bar, ball and cone	43
Figure 12. Variable rate ball penetrometer tests at Vinkeveen site (a) Net resistances (b) Normalized resistances at various depths	44
Figure 13. Net resistances from variable rate penetrometer tests at Bodegraven site.....	45
Figure 14. Cone resistance factor (N_{kt}) for (a) Vinkeveen site (b) Bodegraven site.....	45
Figure 15. T-bar and ball resistance factors (N_{T-bar} & N_{ball}) for (a) Vinkeveen and (b) Bodegraven sites..	46
Figure 16. Relationship between empirical resistance factors and void ratio (a) N_{T-bar} (b) N_{ball}	47

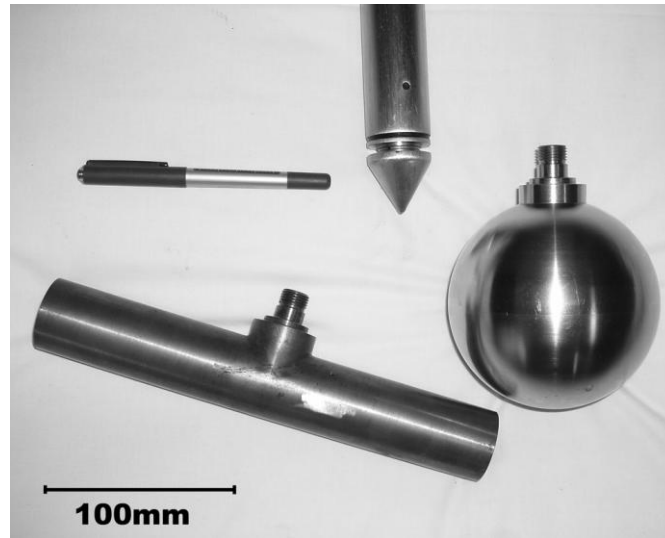


Figure 1. Cone penetrometer with full flow penetrometers, T-bar and ball



Figure 2. Map showing sites in The Netherlands and Ireland

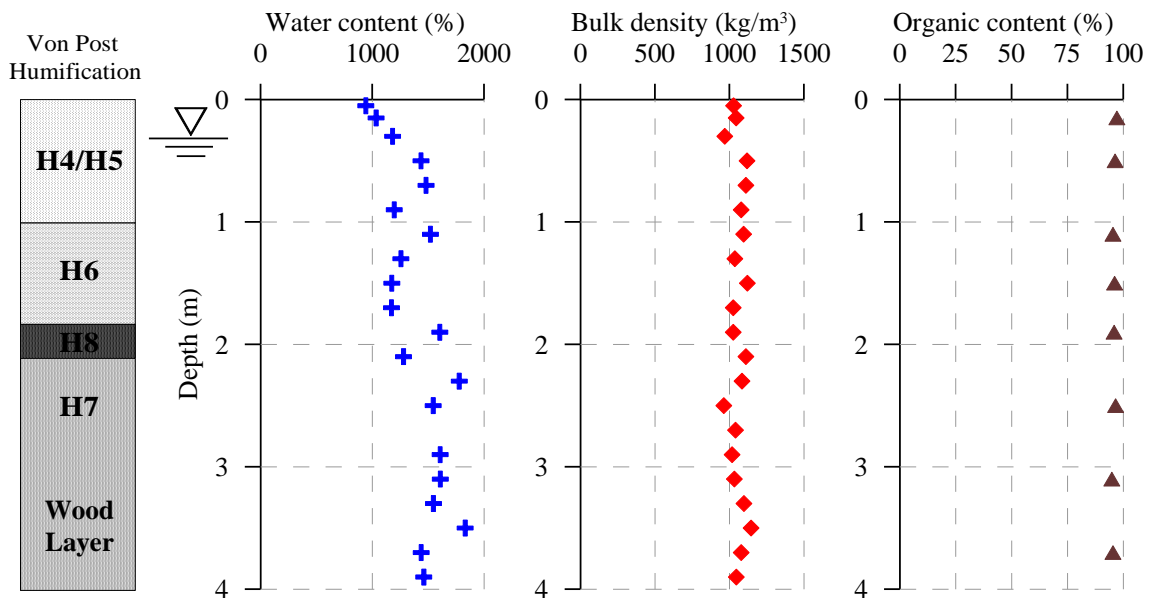


Figure 3. Basic soil properties at Loughrea site

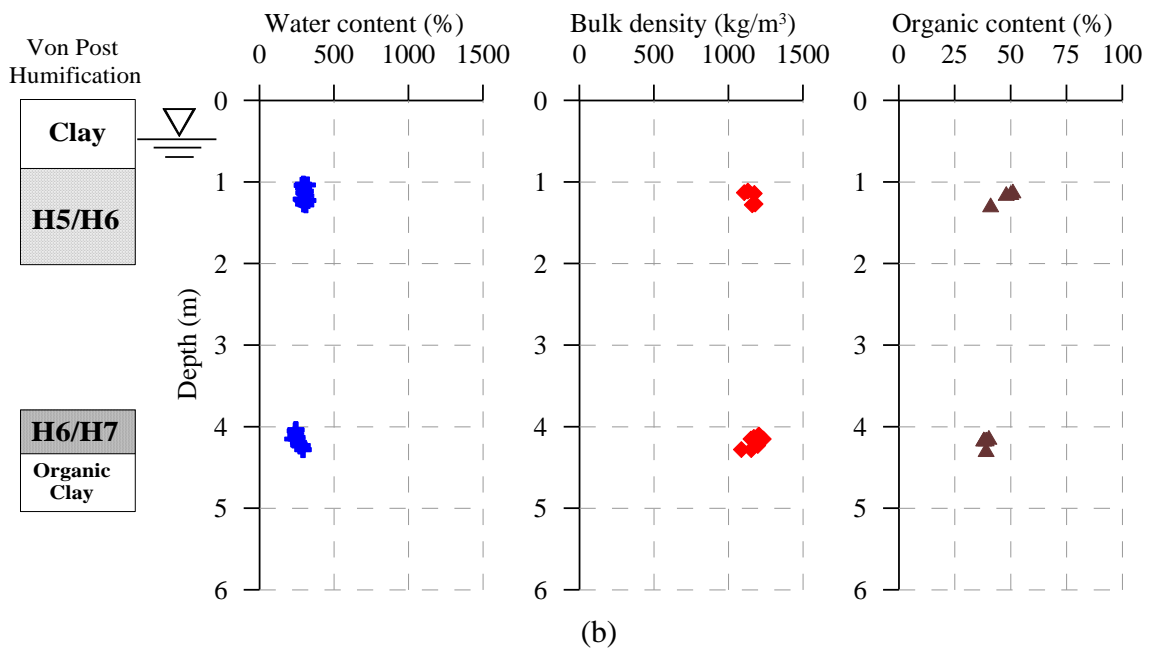
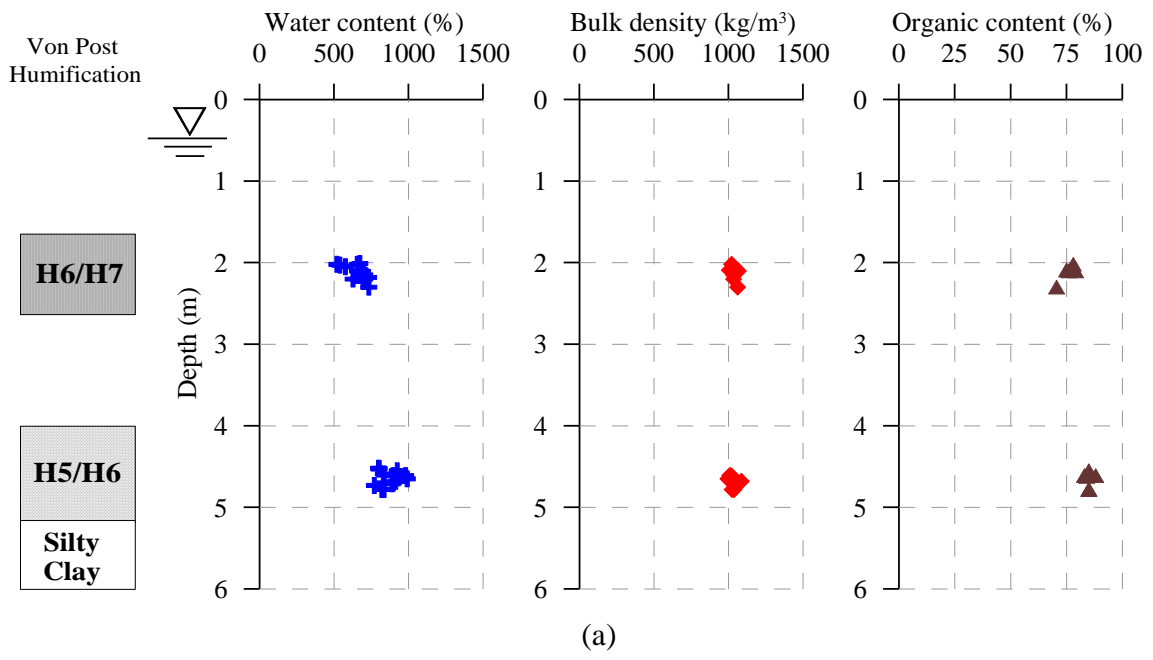


Figure 4. Basic soil properties (a) Vinkeveen site (b) Bodegraven site

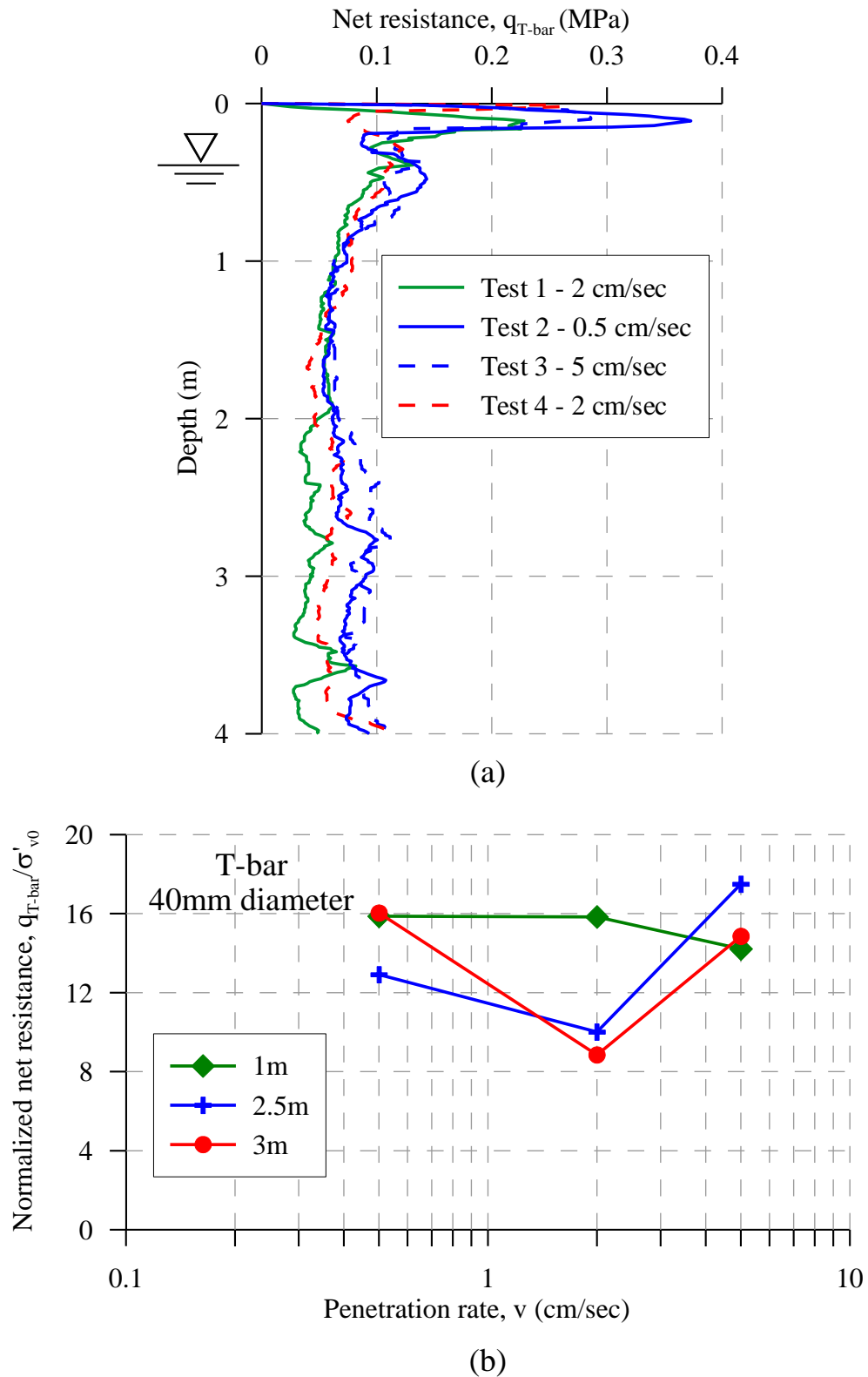


Figure 5. Variable rate T-bar penetrometer tests at Loughrea site (a) Net resistances (b) Normalized resistances at various depths (from Boylan (2008))



Figure 6. (a) Sherbrooke block sampling (b) Waxed block

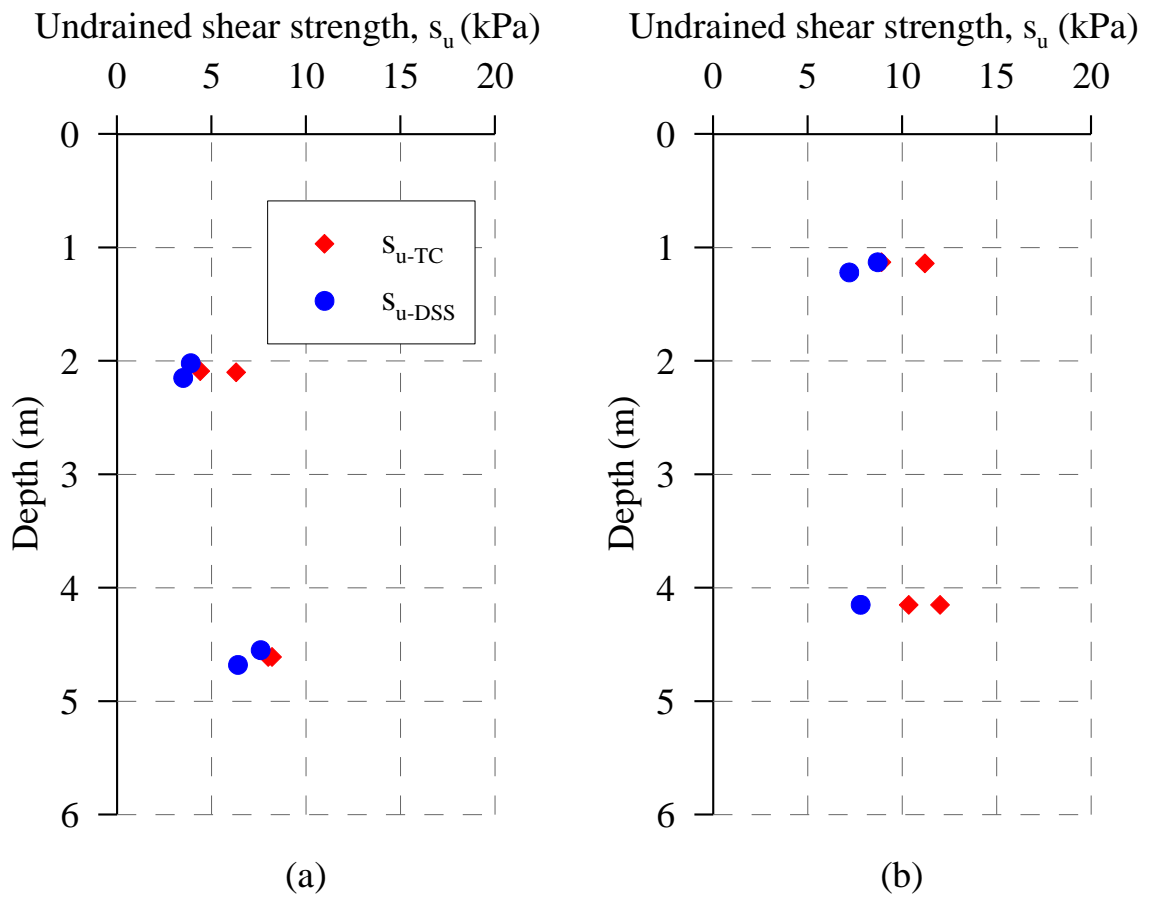


Figure 7. Laboratory strength parameters (a) Vinkeveen site (b) Bodegraven site

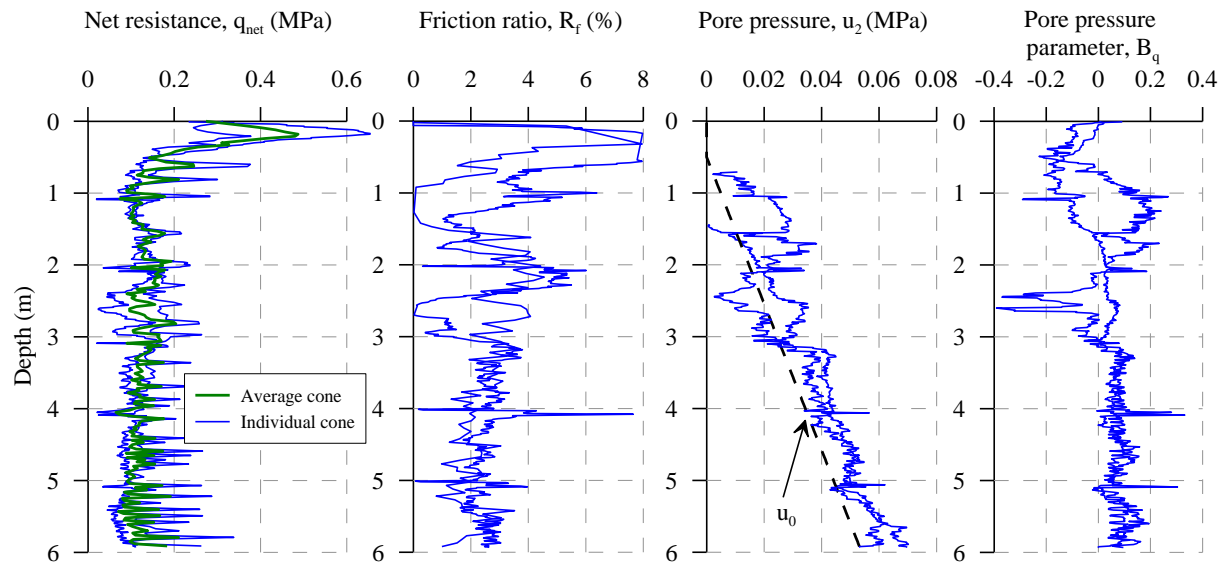


Figure 8. Cone penetration tests at Vinkeveen site

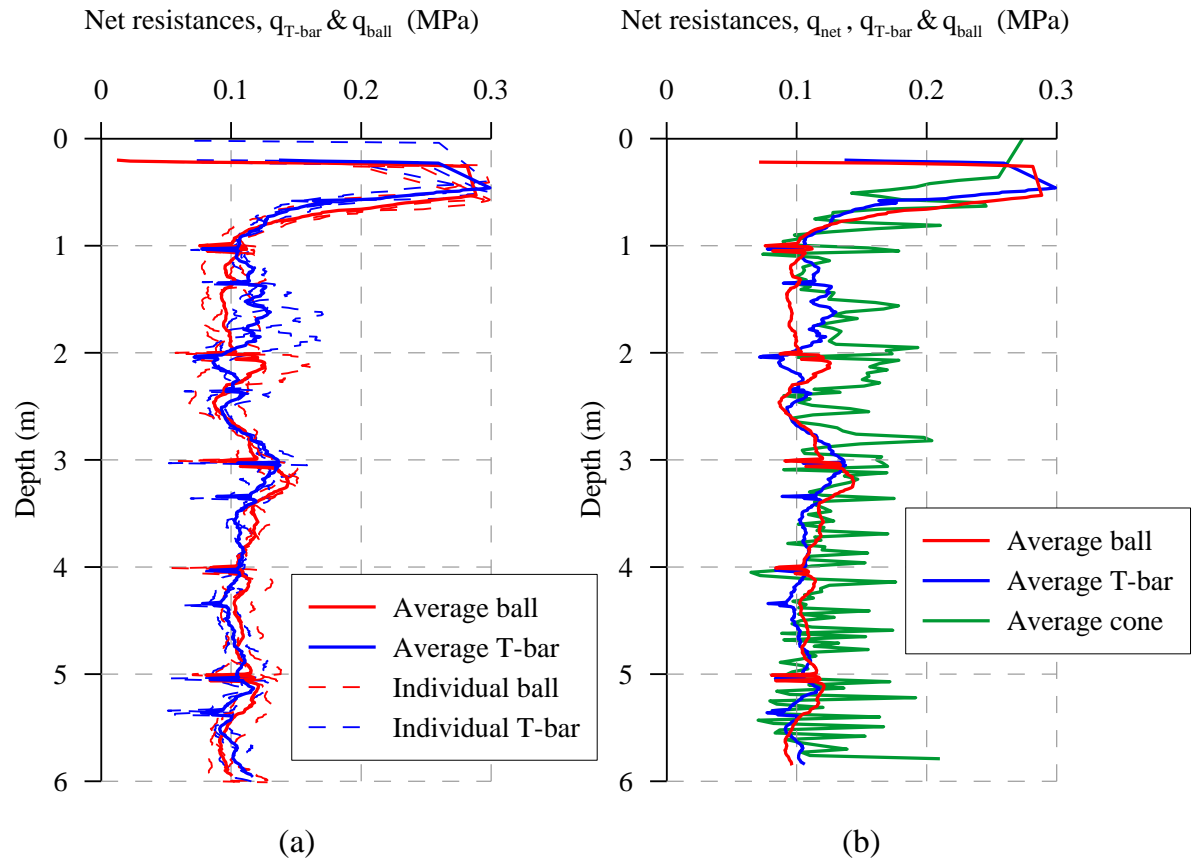


Figure 9. Vinkeveen site (a) T-bar & ball net resistances (b) Comparison of net resistance from T-bar, ball and cone

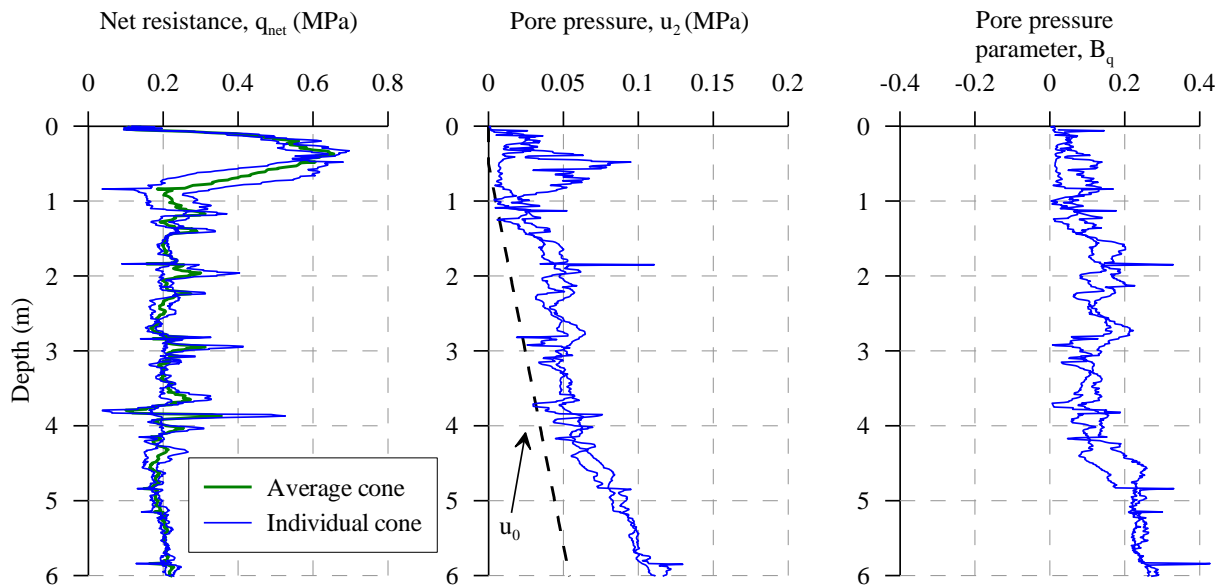


Figure 10. Cone penetration tests at Bodegraven site

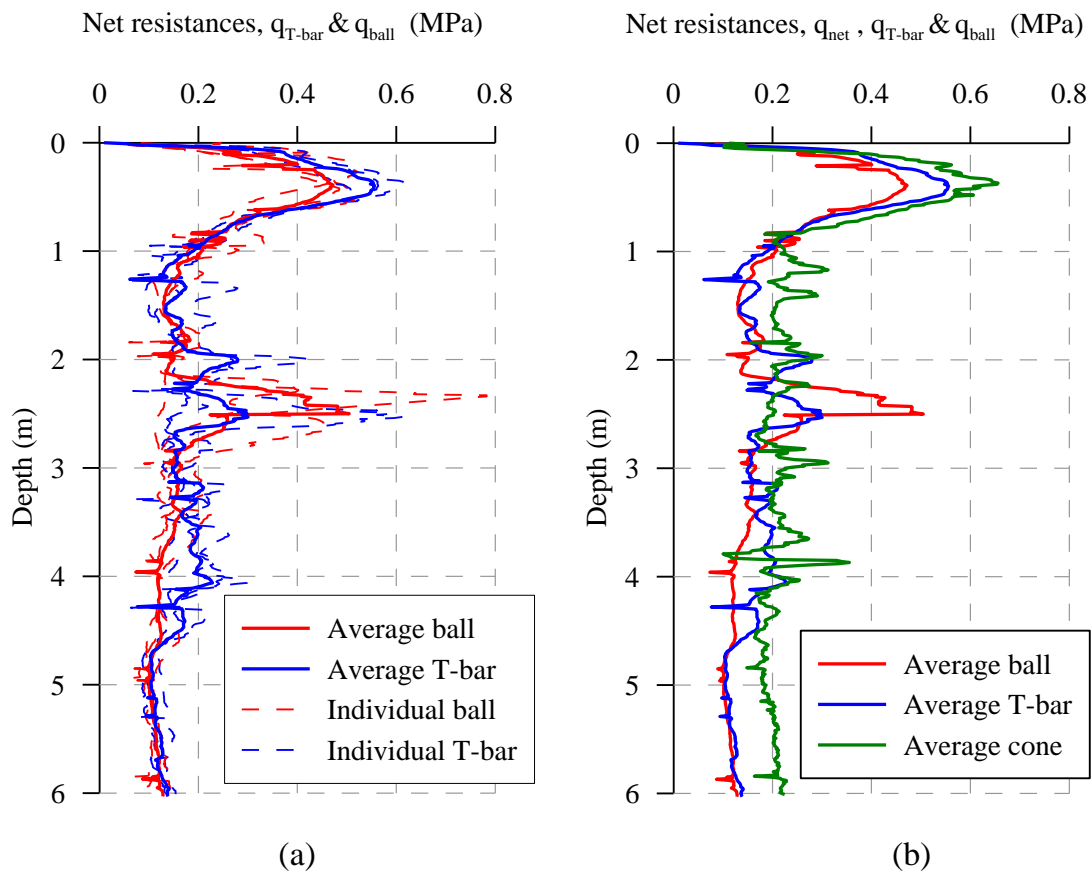


Figure 11. Bodegraven site (a) T-bar & ball net resistances (b) Comparison of net resistance from T-bar, ball and cone

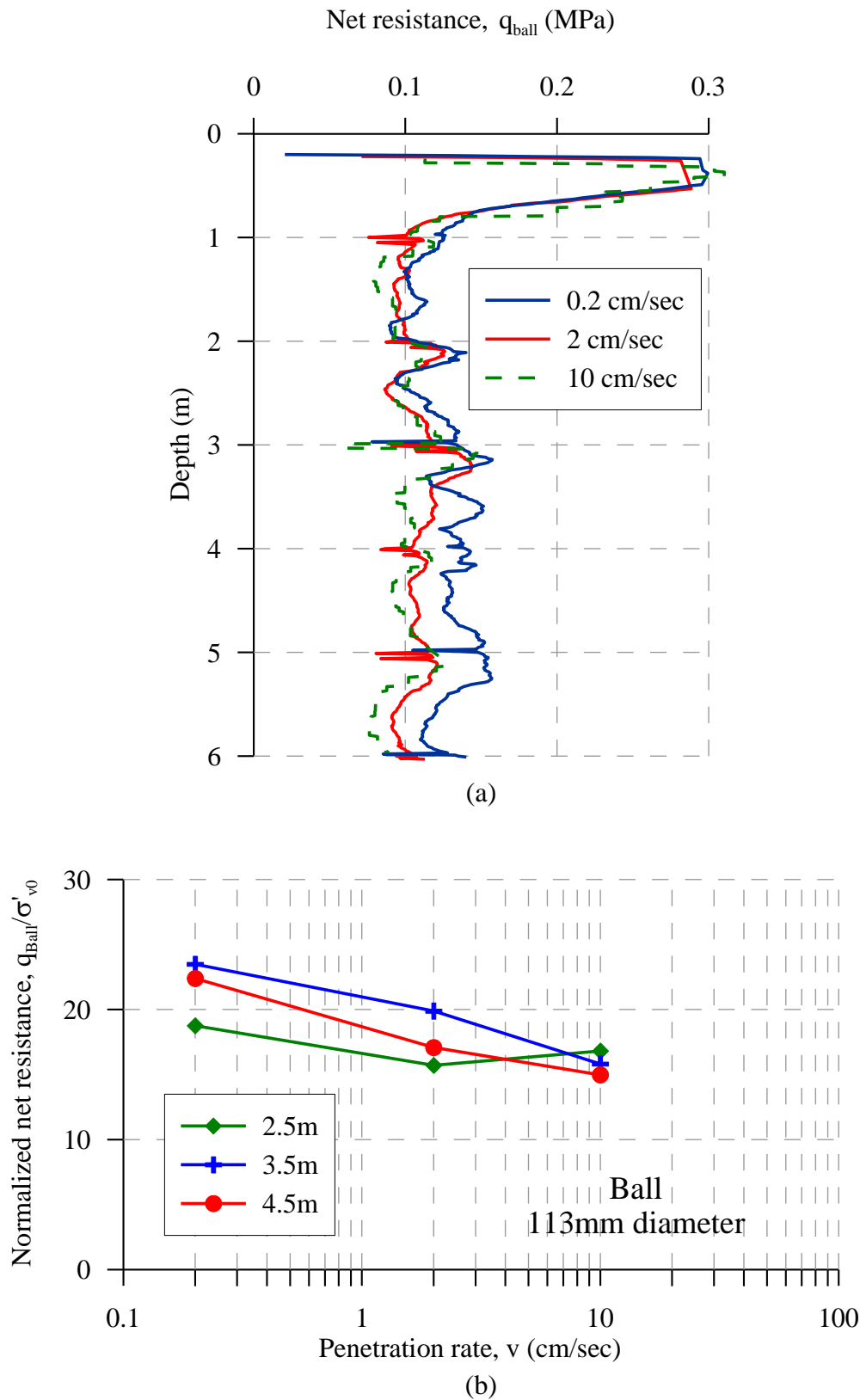


Figure 12. Variable rate ball penetrometer tests at Vinkeveen site (a) Net resistances (b) Normalized resistances at various depths

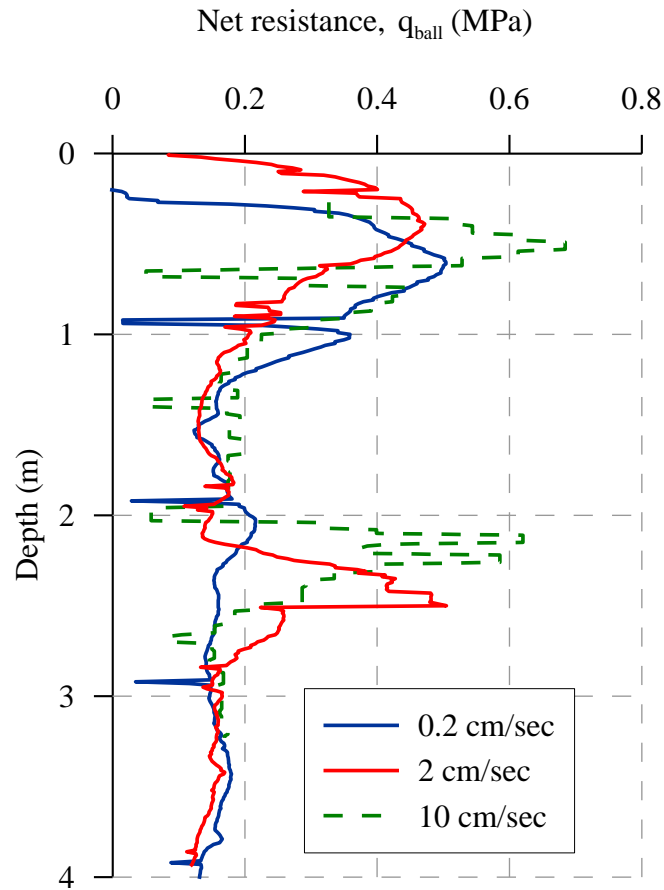


Figure 13. Net resistances from variable rate penetrometer tests at Bodegraven site

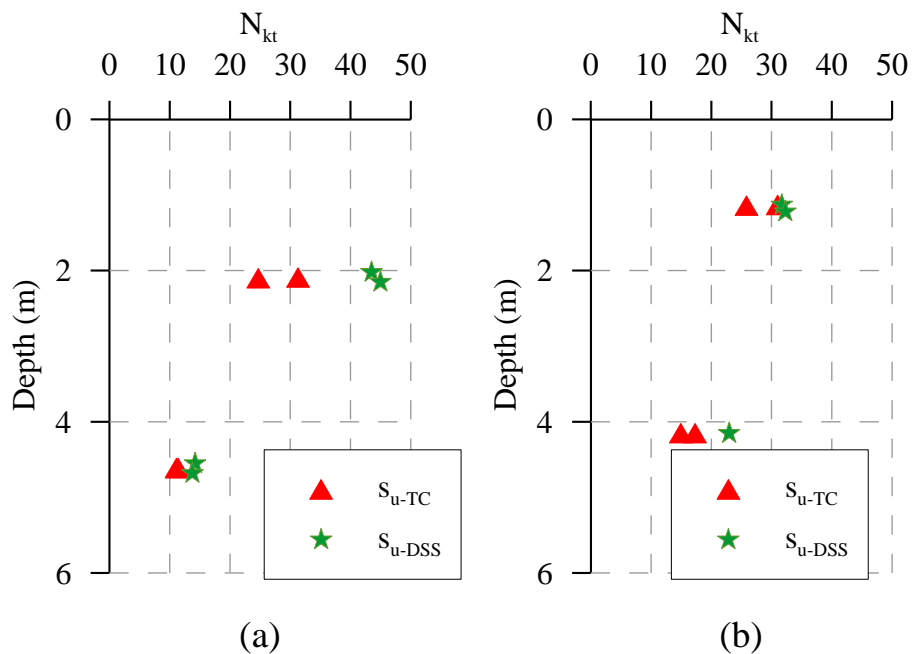
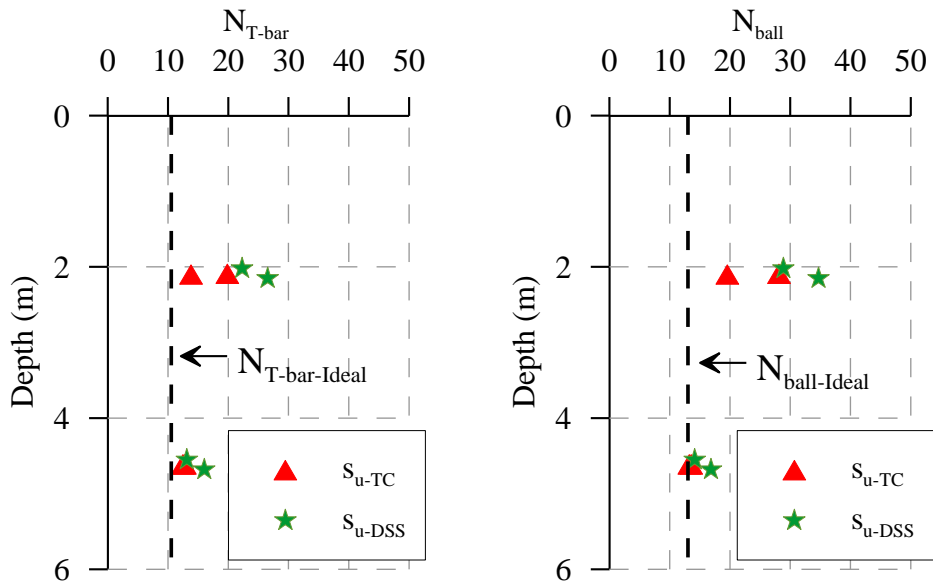
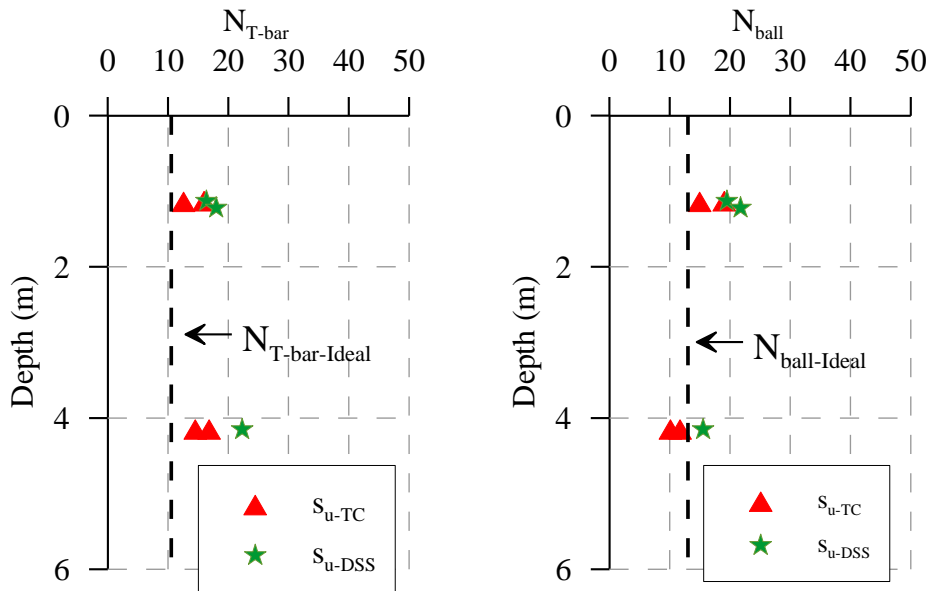


Figure 14. Cone resistance factor (N_{kt}) for (a) Vinkeveen site (b) Bodegraven site

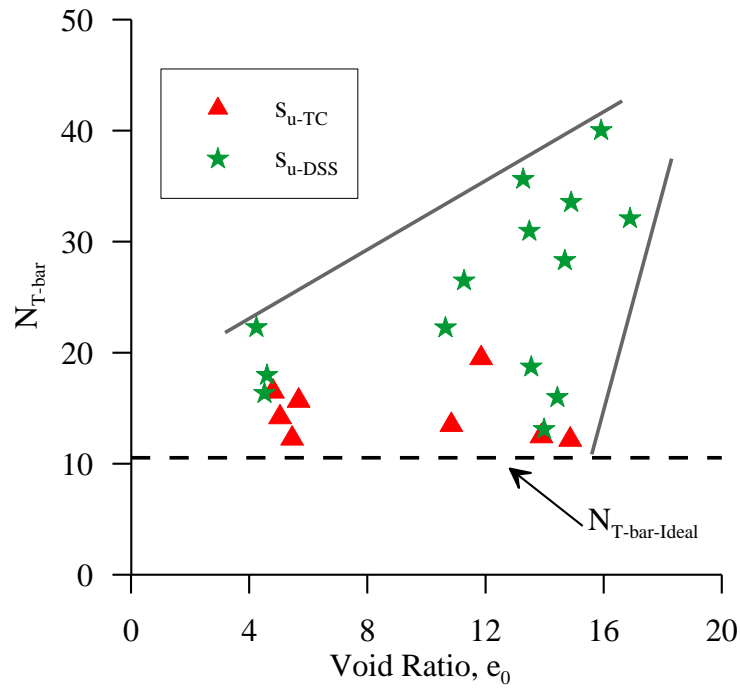


(a)

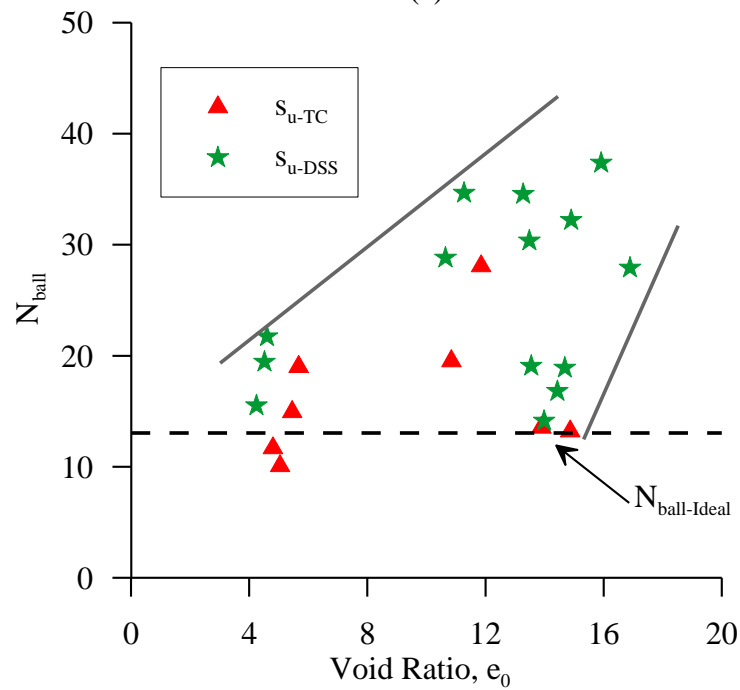


(b)

Figure 15. T-bar and ball resistance factors ($N_{T\text{-bar}}$ & N_{ball}) for (a) Vinkeveen and (b) Bodegraven sites



(a)



(b)

Figure 16. Relationship between empirical resistance factors and void ratio (a) $N_{T\text{-bar}}$ (b) N_{ball}