Determination of undrained shear strength parameters for buried pipeline stability in deltaic soft clays

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ABSTRACT

Offshore pipelines are typically laid on the seabed and lowered into the seabed (trenched) over large sections to provide protection from shipping and fishing activities, or to stabilise the pipeline from hydrodynamic loads. Accurate estimates of the resistance to upward pipeline movement of the overlying clayey trench-backfill are important for design and analytical purposes. The undrained shear strength (c_u) of the upper layers of the seabed (particularly the initial 2 to 3 m below the mudline) is therefore a vital part of pipeline site investigations and is commonly found using the cone penetrometer (CPT). The limitations of standard CPTs may be overcome with the use of novel shaped penetrometers. The measured resistance can be easily used to calculate undrained shear strength using an unique bearing capacity or 'bar' factor. This paper describes the application of standard *in situ* cone penetrometer and T-bar tests from an ROV for the determination of undrained shear strength parameters for pipeline buried in deltaic soft clay. Comparisons with *in situ* plate bearing tests are also provided and the advantages of this novel form of *in situ* test are discussed.

KEYWORDS: clay, *in situ* test, penetrometer, pipeline, backfill.

INTRODUCTION

Offshore pipelines are typically laid on the seabed and lowered into the seabed (trenched) over large sections to provide protection from shipping and fishing activities, to stabilise the pipeline from hydrodynamic loads or for thermal stability. Since pipeline rupture can potentially cause permanent or long-term environmental damage, the risk of exposure of the pipeline due to upheaval buckling, wave-induced liquefaction or erosion must be carefully assessed.

Particularly at risk from upheaval buckling failures are pipelines in very soft clays, which are found towards the margins of estuaries and deltas. These may also contain fine sandy or silty laminations, and are commonly highly plastic. Accurate estimates of the resistance to upward pipeline movement of the overlying clayey trench-backfill are important for design and analytical purposes. The undrained shear strength (c_u) of the upper layers of the seabed (particularly the initial 2 to 3 m below the mudline) is therefore a vital part of pipeline site investigations and is commonly found using the cone penetrometer (CPT). This instrument provides a continuous measurement of undrained shear strength and also allows the stratigraphy of the profile to be identified. Unfortunately, the standard CPT (10 cm^2) is not very accurate in soft clay deposits due to the low tip resistances measured. To date, the empirical and theoretical solutions relating c_u to tip resistances are difficult to apply objectively and accurate estimates of c_u can therefore be difficult, and possibly misleading. These limitations may be overcome by varying the shape of the penetrometer, such that it causes symmetrical soil flow during penetration. This paper describes the application of standard *in situ* cone penetrometer and T-bar tests from a remotely operated vehicle (ROV) offshore for the determination of undrained shear strength parameters for pipeline buried in deltaic soft clay. Comparisons with *in situ* plate bearing tests are provided and the advantages of this novel form of *in situ* test for this type of application will be discussed.

PENETROMETER TESTING

Undrained shear strength profiling forms an important part of many site investigations involving soft clays. This is commonly achieved using a combination of laboratory shear strength tests on undisturbed samples and *in situ* testing methods. The most widely used of these *in situ* tests is the vane shear test (VST), which can be used to measure both the peak and residual undrained shear strength (c_u). However, despite its popularity, this test has a number of disadvantages: estimates of c_u can only be taken at discrete, well spaced, depths in the profile (hence the test is quite slow and a continuous profile cannot be obtained) and thin layers of stiffer material can affect the results.

In contrast, the cone penetrometer test (CPT) provides a continuous measurement of c_u and, in addition, allows the stratigraphy of the profile to be identified. Unfortunately, the CPT is not very accurate in soft clay deposits due to the low tip resistances measured. Also, the deformation mechanism around the cone during penetration is asymmetric in the vertical plane, hence correction for overburden pressure and pore pressure is also required. To date, the empirical and theoretical solutions relating c_u to tip resistance are difficult to apply objectively (they may require an estimation of an equivalent elastic stiffness, G) and estimates of c_u can therefore be in error.

Many of these limitations may be overcome by varying the shape of the penetrometer, such that it causes symmetrical soil flow during penetration. One such device is the T-bar penetrometer that was proposed by Stewart and Randolph (1994). The force required to push the bar laterally (but vertically) through the soil is measured and this is related to c_u using plasticity solutions for an infinitely long cylinder of varying roughness, α_r (Randolph and Houlsby, 1984). The soil deformation mechanism is symmetrical in the plane perpendicular to the axis of the bar, hence the *in situ* vertical stress is equilibriated across the T-bar and no correction for the overburden pressure is required. The measured resistance can be easily converted into a c_u value using a bearing capacity or 'bar' factor, N_t.

Previous testing has suggested that the T-bar gives reproducible profiles that are comparable with c_u measurements made using both the CPT and the VST (Stewart and Randolph, 1994). Since the plasticity solution is based on an infinitely long cylinder passing through the soil, the longer the bar length, the closer it approaches the idealisation. However, very large aspect ratio bars cause instability: non-uniform resistance against the bar along its length may provoke bending moments in the push rod, which can lead to spurious measurements of axial force and can damage the bar. In practice, bar lengths of L = 4 to 5 diameters (d) are used and end effects are reduced by polishing the ends smooth. The assumed soil deformation mechanisms that occur during penetration of these devices are shown in Figure 1. For the T-bar, the soil flow is essentially plane strain and closes fully behind the bar, apart from a small region around the vertical shaft. The cone penetrometer causes predominantly cavity expansion ahead of the penetrometer and the failure mechanism is asymmetric in the vertical plane.

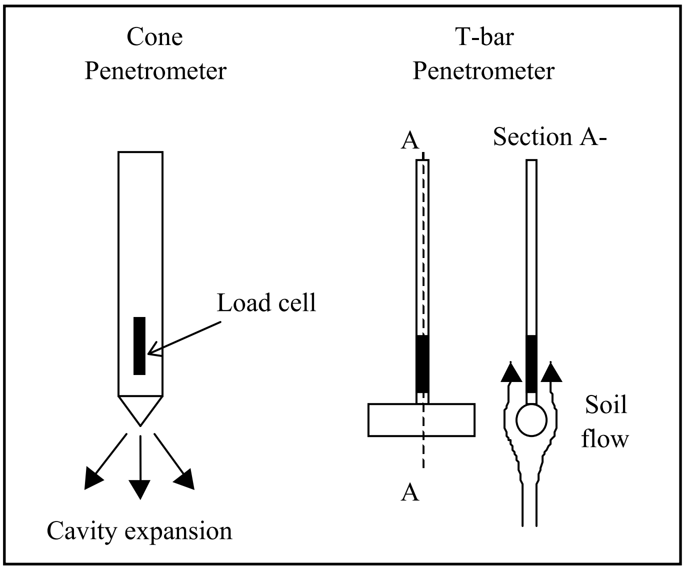

Figure 1: Soil failure mechanisms around different penetrometers

The undrained shear strength, c_u, for the aforementioned penetration devices can be related to the net bearing pressure q, using the following equation:

[1] $$c_u = q/N$$

where, N is a factor representing the relationship between shear strength and net bearing pressure for the separate devices. Subscripts of c and t will be used when referring to the cone and T-bar respectively.

When analysing the CPT results, a correction should be made to the measured bearing pressure, q_m, for both the pore pressure acting at the shoulder of the cone (pore pressure area correction) and the overburden pressure. Campanella and Robertson (1983) expressed this as:

[2] $$q_c = \frac{q_m - \sigma_v + u_o(1-\alpha_c)}{1+\alpha_c B_q - B_q}$$

where, σ_v is the total overburden pressure, u_o is the hydrostatic pore water pressure, α_c is the pore water area correction factor and B_q represents the build up of excess pore pressure due to loading as a ratio of the net bearing pressure.

The various corrections required to analyse the CPT represent a considerable reduction in q_m and have provided the impetus for testing alternative probe shapes, which require no correction. Because soil will largely flow around the T-bar probe during penetration a correction for overburden pressure is not needed and the measured bearing resistance (q_m) is equal to the net bearing resistance (q_t).

Penetrometer Factors

Theoretical and numerical study of the cone penetrometer factor N_c (e.g. Vesic, 1972; Baligh, 1975; Teh and Houlsby, 1991) suggests that it lies in the range 10-18. The work of Teh and Houlsby (1991) indicates that N_c is related to rigidity index (I_r=G/s_u), surface roughness and the *in situ* stress state. It is also thought to be dependent upon strength anisotropy and sensitivity. In practice, the value of N_c is usually based on empirical methods and can be significantly improved with previous experience of the site or material. (e.g. Aas et al., 1986). The T-bar factor N_t is found from an upper and lower bound plasticity solutions (Randolph and Houlsby, 1984) and is a function of the surface roughness, α_r only: N_c = 9.14 for a smooth (α_r = 0) T-bar and N_c =11.94 when α_r =1 (see Figure 2). An average value of 10.5 is used typically (Stewart and Randolph, 1994).

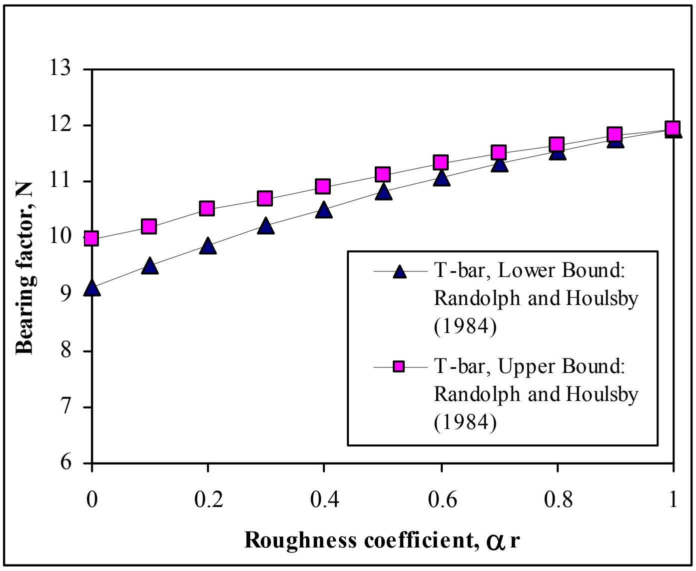

Figure 2: T-bar penetrometer factor with varying surface roughness

FIELD TESTING

Site and Material

The *in situ* penetrometer testing reported herein was conducted in the Nile delta in water depths of 25 to 160 m and was for the purposes of pipeline design. Estimates of sediment stratigraphy and undrained shear strength were required at locations along the length of the pipeline. A series of probes using standard and enlarged cones, and T-bar penetrometers were performed. The Nile delta region is a wave-dominated area and the sediments form a wide submarine fan that extends for 70 km into the Mediterranean. The near surface deposits have been laid down since the last ice-age and consist of marine and lagoonal silts/clays and sabkha clay layers (Rizzini et al., 1978). The materials in the top few metres of the seafloor are silty-clays (with 30 to 70% of the particles less than 2 μm). There is a thin veneer of coarser material at the surface and the sediment has high organic content and gassy pockets. This is a high plasticity material with liquid limits in the range of 70 to 80 %, plasticity indices of approximately 40 % and high liquidity indices, which are generally greater than unity. Assuming a normally consolidated, low sensitivity material, the Atterberg limit data would suggest very low near-surface undrained shear strengths of the order of 1 to 2 kPa.

Testing Programme

A combination of *in situ* probes (conducted from an ROV system) and plate loading tests were used to investigate the near surface (1 to 3 m) sediments at a range of locations. For the purposes of brevity, data from only one location will be reported in this paper, but this is generally representative of the site. Standard CPT (10 cm^2 projected area), enlarged head CPT (50 cm^2 projected area) and T-bar penetrometer tests (40 mm diameter, 250 mm long and a roughened sand blasted surface) were used in both intact sediments and backfill (from trench ploughing and jetting). A penetration rate of 2 cm/s was used as a standard for all of the tests; hence they were conducted in nominally undrained conditions. In addition, a series of four plate-load tests were conducted. Plates of different sizes (0.5 and 0.75 m breadth, and 10 mm thickness) were placed on the seabed and allowed to settle under self-weight and monitored during the process.

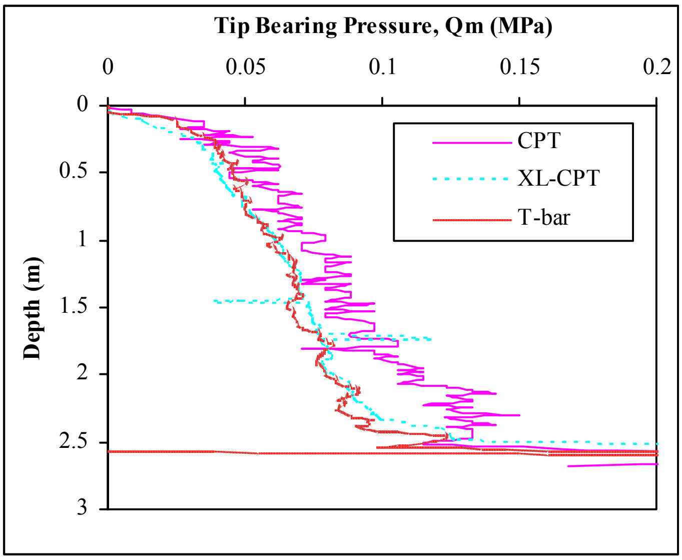

Figure 3: Tip bearing pressures in the intact material

IN SITU TESTING RESULTS

Intact Material

Of initial interest was the state of the intact or virgin material on the site, to provide information for trenching and as a benchmark for the behaviour of the subsequent backfill material. As mentioned previously, the measured bearing pressure (q_m) for the T-bar penetrometer is equivalent to the net bearing pressure (q). However, corrections for the overburden pressure and the pore water area correction imply significant differences between q_m and q_c profiles for the CPT. Figure 3 shows the measured tip bearing pressure profiles for the intact material for the T-bar, standard CPT and enlarged (XL) CPT. As can be seen, the measured resistance for the standard CPT is significantly higher than for the T-bar and enlarged CPT, highlighting the required correction for overburden pressure and pore pressure build up. When this correction is made, the deduced net bearing resistance profiles are more comparable. Interestingly, the enlarged CPT has a tip resistance very close to that of the T-bar. The standard CPT also displays a more variable profile than either the T-bar or the enlarged cone, suggesting differences in the soil deformation mechanisms around these devices or more introduced resolution errors. There also appears to be a stronger layer at approximately 2.5 m depth.

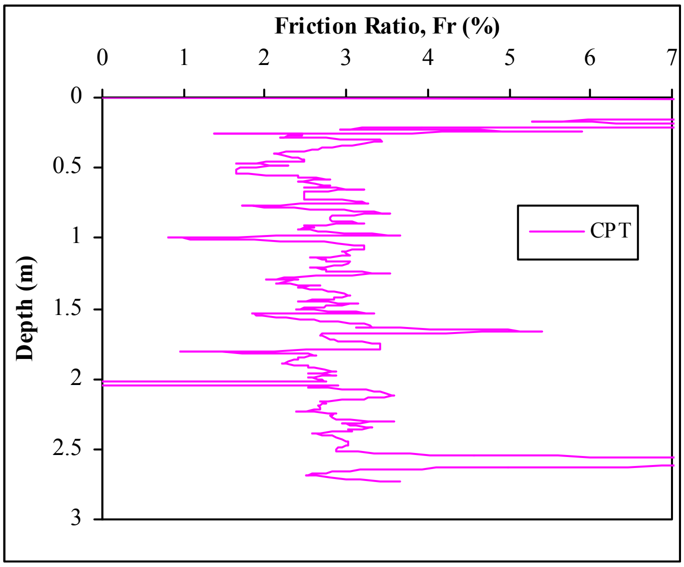

Figure 4: Friction ratio of the intact material

The friction ratio (F_r) from the standard cone penetrometer is shown in Figure 4. This shows a variation in friction ratio from 1 to 7 %, with an average between 2 and 3 %. Based on the friction ratio and the bearing pressure, the classification system of Campanella and Robertson (1983) would suggest that this material is generally a sensitive fine-grained material (clay), with organic material near the surface.

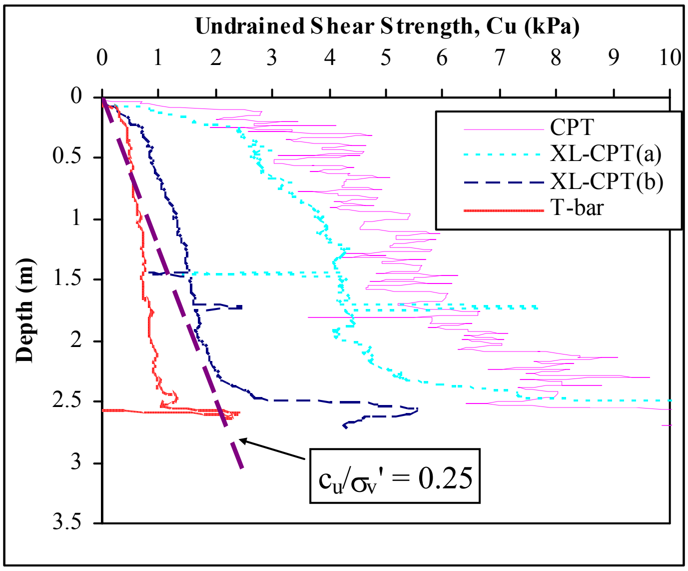

Figure 5: Undrained shear strength profiles of the intact material

The profiles shown in Figure 3 may also be used to deduce the undrained shear strength profiles for this soil. Comparisons of the undrained shear strength with depth using the CPT, T-bar and enlarged

CPT are shown in Figure 5. The undrained shear strength profiles show a very soft material that is predominantly normally consolidated, with an increase in undrained shear strength (c_u) of approximately 1 kPa per metre depth. The different devices show a variation in strength from 1 to 2 kPa near the surface, to 1 to 7 kPa at 2.5 m depth. A dashed 'theoretical' line is shown for comparison, where c_u has been estimated from $c_u = 0.23.\sigma_v'$ (Muir Wood, 1990), assuming $\gamma = 15$ kN/m^3.

All of the devices show a similar trend of strength increase with depth, with the standard cone giving comparatively high values. A cone bearing factor of $N_c = 12$ has been used to estimate c_u for the standard CPT and the enlarged cone (XL-CPT (a)). This value has been adopted from the work of Teh and Houlsby (1991), assuming $G/c_u = 100$ and $K_o = 0.7$. The T-bar estimate of the undrained shear strength (using the unique value of $N_t = 10.5$) is the lowest of the devices, but gives a similar trend that is more comparable with the 'theoretical' estimate. A second curve has also been plotted from the enlarged cone data (XL-CPT (b)). This has been estimated assuming that the device behaves in a similar manner to the 'ball' penetrometer (Watson et al., 1998), which also produces a symmetrical deformation mechanism around the penetrometer like the T-bar. The same bearing factor $N_{xc} = N_t = 10.5$ has been used to create this plot. Interestingly, this produces an estimate of undrained shear strength that is similar to the T-bar.

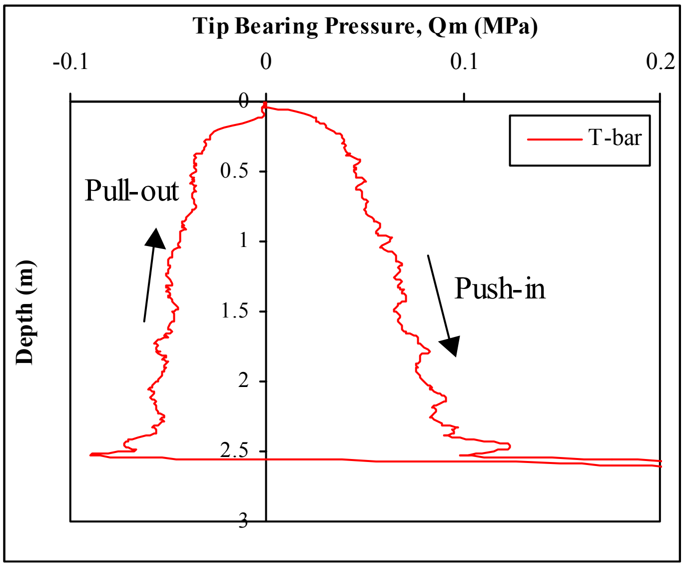

Figure 6: Push-in and pull-out phase of the T-bar test

A rough estimate of the sensitivity of the material may be made by comparing the push-in and pull-out phases of the T-bar test. This is shown in Figure 6 for the intact material and the pull-out phase (through presumably remoulded material) has a similar trend, but lower value than the push-in phase. A simple comparison between the two values of tip pressure at each depth suggests that this material has a sensitivity that ranges from 1 to 2, which is entirely consistent with the previous tests and the geological history of the deposit.

Backfill Material

The pipeline designer most interested in the state of backfill material after pipe-laying. In particular, clayey backfill will take time to consolidate, gaining strength as it settles and the proportion of the original strength regained is important to assess. Hence a series of T-bar tests were conducted in backfill material (approximately 3 and 8 months after backfilling respectively) for the trench jetted and ploughed material.

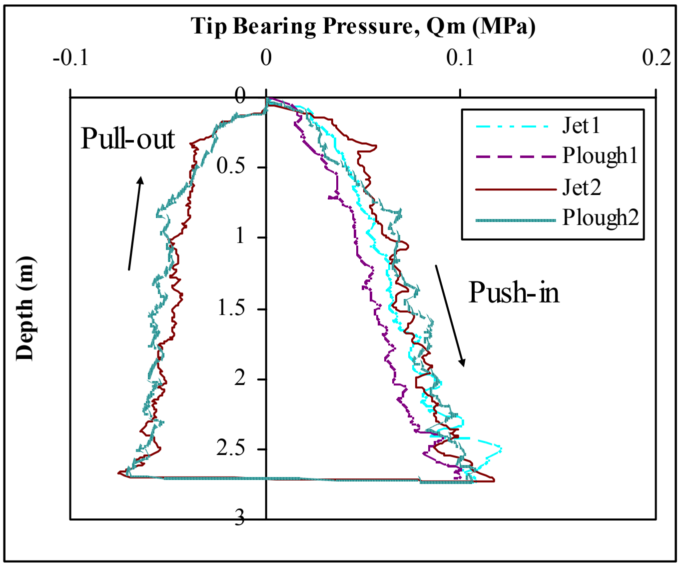

Figure 7: T-bar test results in the backfilled material

Bearing pressures from T-bar tests at four locations (two for each trenching method) close to where the intact material was tested are shown in Figure 7. These show very similar trends and values to those of the intact material (found with the T-bar, in Figure 1), and there is little difference between the resulting backfill from the two methods of trenching (given the very soft nature of the original material). No segregation of coarse material is obvious at the base of the trench, which may also occur with trench jetting methods during the 'fluidisation' stage. The range of sensitivity is also very similar.

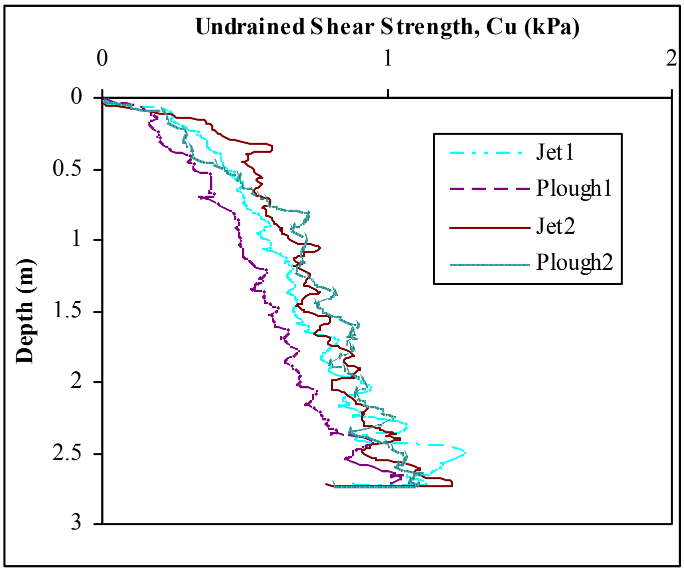

Figure 8: Undrained shear strength for backfilled material (from T-bars)

The estimated undrained shear strengths (c_u) from the above T-bar penetrometer tests are shown in Figure 8. Again these indicate a very similar range of shear strengths to those found in shown in Figure 5, for the intact material and little difference between the two methods of trenching.

Plate Loading Tests

Weighted steel plates were placed on the seabed at four locations to provide an independent estimate of undrained shear strength. Two of the plates were 0.5 m square in plan, another two were 0.75 m square. Their submerged weights were pre-selected to provide bearing pressures in the range 2 to 9 kPa. After placement, they were allowed to settle under self-weight. Measurements of initial settlement was made, followed by further measurements at regular time intervals.

Instantaneous settlement data

The results from the four plate loading tests are shown together in Figure 9. Measured footing settlement divided by footing breadth is plotted against the applied pressure (submerged weight/area) for each test to provide an indictative load-displacement graph. The small difference in plate sizes (0.5 m and 0.75 m) is likely to affect negligibly the results.

The results from the instantaneous results (dashed line in Fig. 9) suggest a bearing capacity, $V/A \approx 6$ kPa for the foundation sizes tested. A simple calculation using a footing bearing capacity factor, $N_c = 6$ (appropriate for a circular foundation in uniform soil) suggests that the average undrained shear strength in the deforming zone under the footings (perhaps to about 0.4 m depth) is about 1 kPa, Clearly, because of the soil non-uniformity beneath the footing, the bearing capacity factor under the footing will be increased (e.g. Davis & Booker, 1973) and the deforming zone will become shallower. However, this will make negligible difference to the postulated undrained shear strength for the small plates tested compared to the accuracy of the measured failure load.

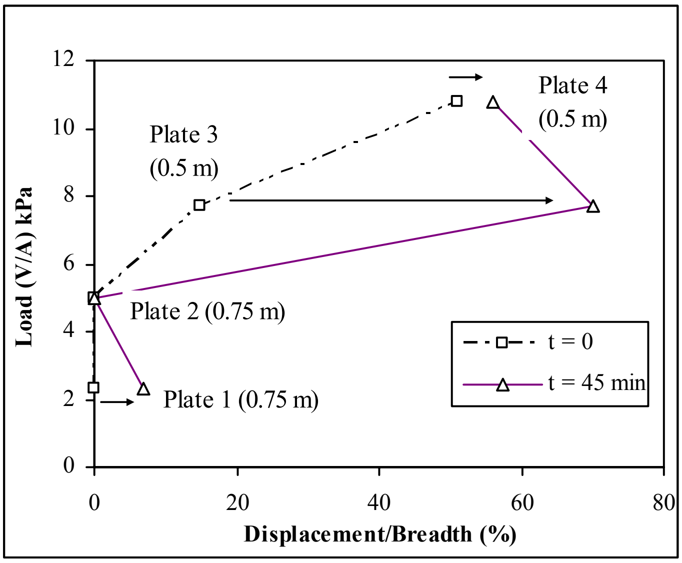

Figure 9: Results from instantaneous and 45 min settlement of plates

Interestingly, the postulated value of undrained shear strength, $s_u = 1$ kPa at 0.4 m depth is in best agreement with the measurement from the enlarged cone data when treated like a ball penetrometer (XL-CPT (b) on Fig. 5). It slightly over-predicts the strength compared to the T-bar data and the Muir Wood (1990) prediction assuming normally consolidated clay. This may be because the T-bar measures an average post-peak strength due to the large strain flow mechanism. However, the strengths measured by the CPT and XL-CPT (about 2 to 2.5 kPa at 0.5 m) predict that the heaviest plate would not cause bearing capacity failure. Clearly, this is at variance with the 250 mm settlement measured by for the 0.5 m breadth plate 4 (displacement/breadth = 50 %), which implies significant plastic penetration was required to stabilize the footing under the applied load. There was also significant settlement for the lighter plate 3. Both results suggest that the CPT tests are overestimating undrained shear strength and that the T-bar and enlarged ‘ball’ CPT are giving the best approximation to the actual strength..

Long-term settlements, variability and use in design:

Figure 9 also shows the measured settlement of each plate after it had rested on the soil surface for 45 minutes (solid line). The data indicate that Plate 2 may be on stronger sediment (perhaps a sandy lamination) and shows some variability of the amount of time-dependent settlement expected.

DISCUSSION

Pipeline burial design in clayey marine sediments requires accurate estimates of undrained shear strength parameters in the near surface layer (i.e. the first 2 to 3 m). This must be accomplished in an environment with high water pressures, from a remote position and with potentially very soft materials. Accurately determining the location of the mudline and ensuring minimal disturbance from a loading frame is important. ROV mounted systems such as that employed during this work are extremely useful in these respects. The cone penetrometer is often used to determine the undrained shear strength and accurate estimates of this parameter are difficult when a range of corrections are required for overburden and pore pressures, and determination of the bearing factor N_c is very subjective without prior knowledge or data from the site.

Novel shaped penetrometers are as easily used as the cone (and can be employed using standard cone equipment and the ROV) and remove much of the uncertainty in the estimation process. The *in situ* T-bar test results from the site suggest that $c_u \approx 1$-2 kPa at the surface, which increases very slowly to about 2-3 kPa at 2.5 m below the surface. This appears to correlate well with the observations from the plate load tests. Additionally, rough estimates of the sediment sensitivity can be made with the T-bar, which can be important for progressive failure modes when all shear planes are not mobilised simultaneously. Comparisons between the backfill and intact material have been made and for this particular site and material, there is very little difference between the pre and post (3-8 months) trenching shear strength behaviour. This may not necessarily be the case for other materials (or degrees of consolidation) and further sites need to be investigated to compile a suitable database for use in design. However, the described approach in this paper appears to provide an excellent methodology for this work.

Some interesting issues regarding the comparison between enlarged cones and the T-bar have been also found. Enlarged cones are often used to increase the sensitivity of measurement (Muromachi, 1981) and certainly as the strength reduces, a larger projected area is useful to ensure that the resolution of the cone load cell does not become a problem. However, comparison of the enlarged cone and standard cone for this site seems to suggest that some 'scaling' errors can occur, which has been observed before (e.g. Powell and Quaterman, 1988). This may be due to the area ratio of the cone and the rods, and the penetration rate leading to different deformation mechanisms and pore pressure generation around the penetrometers. The measurements of the stratigraphy using the two cones also appear to be different. The standard cone appears to more easily 'sense' the presence of thin layer boundaries ahead of the cone more easily because of its smaller diameter; in soft material the depth of influence has been estimated to be 2 to 3 diameters (Schmertmann, 1978). Again this suggests some scaling issues and differences in the deformation mechanism. The enlarged cone may also be compared to another device that behaves in a similar manner to the T-bar: the 'ball' penetrometer (Watson et al., 1998). Since there is the potential for flow around the larger cone (and closure behind towards the rods), the adoption of a similar analysis for estimation of the undrained shear strength has some merit. Other devices that cause this form of deformation mechanism have also been found to have similar 'N' factors (Randolph, 1998). This aspect needs to be investigated further.

CONCLUSIONS

This paper has described the application of *in situ* cone penetrometer and T-bar tests from an ROV for the determination of undrained shear strength parameters. Buried pipelines designed to employ the resistance of overlying clayey backfill against upward pipeline movement require good quality undrained shear strength data for much shallower depths than used in conventional offshore design. Some of the limitations of standard CPTs for estimating undrained shear strength have been demonstrated by the data shown. In particular, the choice of a suitable bearing factor N is quite difficult. Conversely, for the T-bar, the measured resistance can be easily converted into a c_u value using an unique (i.e. not a function of the soil state) bearing capacity or 'bar' factor. Comparisons with *in situ* plate bearing tests and enlarged cone tests have shown the advantages of these novel forms of *in situ* test for this type of application.

ACKNOWLEDGEMENTS

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