The DMT - $\sigma_{hc}$ Method for Piles Driven in Clay.

S. Marchetti$^1$, G. Totani$^2$, R.G. Campanella$^3$, P.K. Robertson$^4$, B. Taddei$^5$

ABSTRACT: This paper presents a tentative method for evaluating the limiting skin friction $f_s$ for piles driven in clay based on $\sigma_{hc}$ (horizontal effective stress at the interface at the end of the reconsolidation) determined by DMT. The method may be considered a corollary of the following recent findings: a) $\sigma_{hc}$ against a driven pile is a dominant factor in determining $f_s$. b) The total horizontal stress against a pile driven in clay, and its evolution with time, is relatively independent from dimensions and (according to preliminary evidence) shape of the penetrating object. This paper illustrates determinations of $\sigma_{hc}$ against the dilatometer by using the standard DMT equipment and comments on a number of $\sigma_{hc}(t)$ "reconsolidation" curves obtained in various soil types. The paper also describes a comparison between $f_s$ predicted by the proposed method and $f_s$ determined from full scale pile load tests.

INTRODUCTION

Though empirical methods for evaluating the limiting skin friction $f_s$ for piles driven in clay are currently widely used, many research efforts have been recently directed towards developing more rational methods. Some of these methods are based on replacing the total stress approach with the more fundamental effective stress approach. Others are based on often sophisticated analytical models attempting to simulate the penetration process, such as cavity expansion methods (e.g. Randolph et al., 1979) and the strain path method (Balligh, 1985). However even the most recent models do not always provide estimates in agreement with observed behavior. One often mispredicted parameter is $\sigma_{hc}$, the effective horizontal stress against the pile after reconsolidation. Yet $\sigma_{hc}$ has been recognized as the dominant parameter in determining $f_s$ (Balligh, 1985; Karlarud and Haugen, 1985). Thus a direct determination of $\sigma_{hc}$ appears desirable, since its prediction is responsible for a large part of the uncertainty in the prediction of $f_s$.

This paper describes measurements of $\sigma_{hc}$ against the flat dilatometer (Marchetti, 1980) and presents a tentative method for predicting $f_s$ based on such determined value of $\sigma_{hc}$.

NOTATION

The horizontal effective stress $\sigma_h$ at the pile-clay interface varies considerably during the pile life. Therefore specific subscripts are added.

1 Prof. of Soil Mechanics, L'Aquila University, Italy
2 Res. Asst. of Soil Mechanics, L'Aquila University, Italy
3 Prof. of Civ. Engrg., Univ. of British Columbia, Vancouver, Canada
4 NSERC Univ. Res. Fellow, Univ. of British Columbia, Vancouver, Canada
5 Geotechnical Consultant, L'Aquila, Italy
to $\sigma^*_h$ to denote the phase to which $\sigma^*_h$ refers: a) $\sigma^*_h$ in the original pre-penetration conditions b) $\sigma^*_h$ during penetration c) $\sigma^*_h$ at the end of the reconsolidation (prior to shearing) d) $\sigma^*_h$ during quasi-static loading to failure, undrained. Moreover, the following symbols are adopted: $\kappa'_{00} = \sigma^*_h/\sigma^*_v$ and $K_{so} = \sigma^*_h/c_{vo}$, with $c_{vo}$ initial vertical effective overburden stress.

It should be noted that, in general, $\sigma^*_h \neq \sigma^*_h$, because, when the pile is loaded, excess pore pressure develops at the interface. Later this paper will focus on the relationship between $\sigma^*_h$ and $f_s$.

$f_s$ is defined herein as the axial load failing the pile in first-time shearing in a conventional pile load test, carried out at the end of the reconsolidation, i.e. without (ideally) possible strength gains due to thixotropy.

**BACKGROUND**

This section briefly recalls some findings particularly relevant to the method proposed in this paper.

1. When a pile penetrates into NC or moderately OC clay, high pore pressures are generated at the pile-clay interface, with a substantial drop of $\sigma^*_h$ against the pile shaft (Fig. 1). As the reconsolidation proceeds, $\sigma^*_h$ at the interface builds up again, up to a long term equilibrium value $\sigma^*_h$ generally higher (sometimes considerably) than $\sigma^*_h$ (see e.g. Assouz, 1985).

2. The observed total horizontal pressure $\sigma^*_h$ against the pile generally decreases significantly during the reconsolidation (see e.g. Balligh, 1985; Karlsrud and Haugen, 1985), somewhat in contrast with the prediction of a variety of cavity expansion-based methods, predicting $\sigma^*_h$ constant during reconsolidation.

3. Even the most recent theoretical models do not always predict correctly the effective stresses observed during pile installation and reconsolidation (e.g. Karlsrud and Haugen, 1985; Balligh, 1985).

4. A reliable prediction of $\sigma^*_h$ is a necessary step in predicting $f_s$, because $\sigma^*_h$ is a controlling factor on the ultimate shaft resistance (Balligh, 1985; Karlsrud and Haugen, 1985).

5. If an estimate of $\sigma^*_h$ is available, then $f_s$ may be evaluated by means of the "skin friction ratio" $\rho$ (Balligh, 1985). The parameter $\rho$ (analogous to the undrained strength ratio $\sigma^*_v/c_{vo}$) is defined as:

$$\rho = f_s / \sigma^*_h$$

where $f_s$ denotes the maximum shear stress at the soil-pile interface due to rapid (undrained) axial loading, from a condition involving no excess pore pressure, corresponding to first-time shearing.

7. Fig. 2 compares $f_s$ predicted by the $\alpha$-method and the $\lambda$-method (based on the field vane strength $S_{fv}$), with $f_s$ backfigured from load tests on piles in the moderately overconsolidated Haag clay (Norway). The predictions of $f_s$ were in contrast, even in shape, with $f_s$ "measured", which, on the other hand, were reasonably well predicted by $f_s = \rho \cdot \sigma^*_h$ with $\rho = 0.40$ (Karlsrud and Haugen, 1985).

8. The values of $\rho$ reported in the literature, backfigured from load tests on short piles (Karlsrud and Haugen, 1985) or on model piles (Assouz, 1985) or derived theoretically using the strain path method (Balligh, 1985) are still few, but vary in a relatively narrow range (0.25 to 0.40).

9. According to strain path models (Balligh, 1985) the pile diameter has

**Fig. 1. Schematic Diagram illustrating the variation of $\sigma^*_h$ at the Interface caused by the Pile Installation.**

**Fig. 2. Measured and predicted Limit Skin Friction $f_s$ (Data from Karlsrud and Haugen, 1985).**

**Fig. 3. Normalized Values of the Total Radial Stress as Measured with 3 Different Blades compared with Similar Measurements with the Penetrometer, in Boston Blue Clay (Balligh, 1984).**
no influence on $q_e$ at the pile-clay interface at the time of installation and during the subsequent reconsolidation. Experimental results obtained by Baligh (1964) show that, in lightly OC Boston Blue Clay, even the shape (cone, blade) of the penetrating object has minor influence on $q_e(t)$ (Fig. 3).

10. Another element supporting indirectly the low dependence of $q_e$ at the interface from the shape of the cross section is the similarity, in NC (or lightly OC) clays, of the values of the following ratios:

- $\frac{(u-u_0)/\gamma_0}{\gamma_0}$ for a circular cross section (where $u$ is the interface pore pressure during undrained penetration and $u_0$ = steady state equilibrium pore pressure) found by Baligh et al. (1985) to be typically, for the SBC they studied, between 2.3 and 2.4.

- $K_0 = \frac{p_{oc} - u_0}{\gamma_0}$ for the dilatometer blade (where $p_{oc}$ is the first dilatometer reading, mostly made by $u$) reported consistently in the DMT literature to be in the range 1.8 to 2.5, for NC (or lightly OC) clays.

PROPOSED (TENTATIVE) DMT-BASED METHOD

The following procedure is proposed:

1. The dilatometer is inserted into the ground and left for a time sufficient for reconsolidation (details of this operation are illustrated in the next section). Then the end-of-reconsolidation first dilatometer reading $p_{oc}$ is determined. $q_{e hc}$ is then obtained as:

$$ q_{e hc} = p_{oc} - u_0 $$

(2)

where $u_0$ is the steady state equilibrium pore pressure, assumed to be known.

2. $f_s$ is estimated by the Equation:

$$ f_s = p_{oc} - q_{e hc} $$

(3)

- For the friction ratio $f$ the value 0.20 is provisionally recommended, in view of the values reported in the literature and of the fact that end effects possibly existing in the dilatometer (the distance between the membrane center and the sharp bottom edge is 0.7 times the blade thickness) may tend to overestimate $q_{e hc}$. Many additional $p$ values, backfigured from pile load tests are necessary, however, for evaluating the method and for getting some information on the dependency of $p$ on factors such as soil type and stress history, upon end vs. closed ended piles, pile material, surface roughness, jacking vs. driving.

DETERMINATION OF $p_{oc}$ BY DMT

The value of $p_{oc}$ needed in Eq. 2 to obtain $q_{e hc}$ is the value of $p_{oc}$ against the dilatometer blade once the reconsolidation is ended. Since the time for reconsolidation varies considerably from soil to soil, the operator needs to be aware of the progress reached by the reconsolidation. This can be achieved in various different ways. This section describes the way that, after a few trials, was found to be the most convenient by the writers. Starting from the time the blade has been pushed to the desired depth, subsequent readings are taken (the second DMT reading is omitted to avoid the 1 mm deflection, i.e. the pressure is not zero as soon as the A-reading is attained). By plotting $A$ vs. log $t$, it can be easily recognized when equilibrium has been (substantially) reached. At this time:

1. A final reading $A_f$ is taken ($B_f$ is taken too) and converted into a corrected value by using the usual DMT correction formulae (Marchetti and Craparo, 1981).

2. By inspection of the $A(t)$ curve, the residual decrease of $A$ still to occur ($\Delta A_r$) is estimated.

3. Then $p_{oc}$ is obtained as:

$$ p_{oc} = p_{oc} - \Delta A $$

(4)

Measurements shown later in the paper indicate, if the observed $A(t)$ curve includes some part of the stabilized portion, then the errors associated with estimating $\Delta A_r$ are generally a low proportion of $p_{oc}$.

Experience has also shown that the null method of determination of $A$ permits to read $A$ with extremely high reproducibility (accuracy and reproducibility are essentially those of the pressure gauge of the control box, generally superabundant). This high reproducibility is of great help when one has to judge the tendency to stabilization of the A-readings.

The procedure described has been found to be quite convenient, because it can be executed with the standard "mechanical" DMT equipment, and the $p_{oc}$ determination is as simple as a normal DMT.

The possible utilization of the $q_e(t)$ reconsolidation curves for getting information, at least in a relative sense, of the consolidation characteristics of the soil is not discussed here. However it is noted:

- No theory is unfortunately available, at present, for interpreting the $q_e(t)$ curve relative to the geometry of the blade.

- On the other hand, the $q_e(t)$ reconsolidation curves against the dilatometer are considerably simpler to obtain than dissipation curves involving the measurement of pore water pressure.

RESULTS OF THE "RECONSOLIDATION" TESTS

Form of Presentation

The results of the "reconsolidation" tests are displayed in the form of individual $A(t)$ diagrams. Some information about the soil in which the reconsolidation tests were performed is given in Table 1. It is noted:

1. Most of the soil information in Table 1 consists in parameters interpreted from normal DMT performed just above and just below the depth of the reconsolidation test (normal DMT and reconsolidation tests cannot be performed, both, at one elevation). Thus, on top of other inaccuracies, mismatching errors, to some extent, are inevitable, especially in layered soils. Some information about the kind of approximation expectable from the DMT predictions may be found by the interested reader in Schmertmann (1984). For instance, the standard deviations reported in Schmertmann's comparative study for $K_0$ and OCR are 22% and 30% respectively.

2. Two forms of graphical presentation of the reconsolidation test results were considered, namely:

- $(q_e-u_0)/\gamma_0$ vs. time. If the normalized form confers some generality to the results and helps in comparing different curves. However, converting $A(t)$ to this ratio requires correcting $A(t)$ and evaluating $u_0$ and $q_e$, operations all involving, to some extent, subjective estimates.
**USE OF IN SITU TESTS**

### Tests at Rieti (Central Italy)

The main scope of these tests was to verify the feasibility of obtaining the consolidation curves using the standard DMT equipment and to gain some information with the times needed for consolidation. Reconsolidation tests were performed in two layers. The upper layer was predominantly silt, with some fine sand (see Table 1). The lower layer was predominantly silt with clay. The results, shown in Fig. 4, indicate:

1. In the upper leaner material, the stabilization typically requires 10 to 30 min. In one case (at the depth 4.9 m) the A-reading was stable since the beginning, presumably due to a more sandy layer (at this depth the "closing pressure", see Campanella et al., 1985, was found to be equal to \( \sigma_{10} \)).

2. In the lower more plastic material (Figs. 4c-d-e-f) stabilization times are considerably longer (a few hours).

3. The curve in Fig. 4g, though not fully stabilized, was the first one to justify some extrapolation guesswork. The residual decrease of A was estimated to be \( \Delta A_0 = 0.3 \text{ bar} \). Subsequent calculations, carried out according to Eqs. 2 and 4 and other common equations of soil mechanics indicate (see Table 1) that the estimated \( K_D \)-gain factor due to the penetration (defined as \( K_{DC}/K_{DO} \)) was in this material 1.47.

### Tests at Scoppito (Central Italy)

Reconsolidation tests were performed in an overconsolidated stiff silt clay layer (see Table 1) between depths 7.60 and 11 m. The results, shown in Fig. 5, indicate:

1. In the investigated layer, the stabilization typically requires 0.5 to 3 hours.

2. While most of the reconsolidation tests at this site were clearly incomplete, the curve relative to the depth 7.6 m shows signs of stabilization. The residual decrease of A was estimated to be \( \Delta A_0 = 0.25 \text{ bar} \). As shown in Table 1, the estimated \( K_D \)-gain factor caused by the penetration was in this material 2.53.

3. The tests whose results are shown in Figs. 5c and 5e were specifically aimed at investigating the possible perturbation of the A-reading operation on the reconsolidation curves. In fact even the A-reading operation involves a (very small) motion of the membrane (0.05 m at the center). In Figs. 5c and 5e the asterisks are the results of reconsolidation tests carried out 0.2 m below the main depth indicated in each figure, in which the intermediate readings, between the first one and the last one, were omitted. It can be seen that, though the results relative to each pair of tests do not coincide (because the starting A was somewhat different), the trends are in general agreement, suggesting that the A-reading operation does not cause important perturbations.

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### Table 1: Information about the Soil at the Test Sites (1 bar=100 kPa)

<table>
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<tr>
<th>Site</th>
<th>( n )</th>
<th>( \sigma_{10} )</th>
<th>( K_{DC} )</th>
<th>( K_{DO} )</th>
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<td>0.3</td>
<td>0.3</td>
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<tr>
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</tbody>
</table>

Fig. 4 Reconsolidation Test Results at Rieti (1 bar=100 kPa)

Fig. 5 Reconsolidation Test Results at Scoppito (1 bar=100 kPa)
Tests at Preturo (Central Italy)

At this site, long duration (1 to 2 weeks) reconsolidation tests were performed, in order to gain some information on long term behavior. The soil tested (see Table 1) was essentially heavily overconsolidated silty clay and clayey silt. The results, shown in Fig. 6, indicate:

1. In the sandy silt at 18 m depth (Fig. 6a), the initial fast drop of A after penetration (possibly not fully undrained) was followed by "creep" approximately linear vs log t.

2. The A-readings taken in the late stages in Figs. 6a and 6b were not isolated readings, but groups of readings taken in succession, over approximately 24 hour. The repetitions were performed for further investigating the influence of the A-reading operation on the readings themselves. It can be seen that repeated readings did cause a momentary, some reduction of the A value. However, the reduction was small and apparently "unremem-bered" when the first reading of a later succession was taken. While the A-reading repetitions in Figs. 6a and 6b were motivated by the reasons just explained, in general repeated readings in the late stages are of no use and rather should be avoided.

3. No special or unexpected feature was noted in the long term portion of the curves. Fig. 6 shows that, within practical limits of variation, once the slope of the curves starts to decrease, it will continue to decrease, or will remain constant, but will not increase again.

4. The long term portion of the curves in Fig. 6 would have been predicted satisfactorily from the initial portion, provided the initial portion was extended the time necessary to include some part of the stabilized portion. Based on the experience gained so far, the following practical rule for deciding when the reconsolidation can be arrested is suggested:

- By treating the A-log t curve as if it was an oedometer time-deformation curve, find "t 100" using the Casagrande's log t fitting method.

- Wait at least one log cycle, after "t 100", before arresting the test.

Another tentative suggestion, possibly helpful for evaluating the residual A, is still to occur, is to extrapolate the reconsolidation curve up to 100,000 min > 2.3 months (recommendation also taking into account experimental work by Todd and Charles, 1981).

5. Fig. 6c, showing results relative to a more clayey (lower H) material, exemplifies a frequently observed trend: in soils with a high clay fraction, the drop of q<sub>u</sub> during the reconsolidation is often considerable.

6. As shown in Table 1, the estimated penetration K<sub>e</sub>-gain was in this deposit particularly high (approximately in the range 2 to 4), especially in the stiff sandy silt layer at 18 m depth.

Comparison with Pile Load Tests Results

This section illustrates a case study comparing f<sub>p</sub> predicted by the proposed method with f<sub>p</sub> determined from full scale pile load tests.

Test Site (Latina, Central Italy)

Fig. 7 shows CPT profiles (obtained with the mechanical cone) and normal DMT results (3 superimposed profiles). Some classification results are reported in Table 1.

Fig. 6 Reconsolidation Test Results at Preturo (1 bar = 100 kPa)
From both CPT and DMT results it is apparent the presence of a harder layer between 15 and 16m depth (samples revealed the presence of stones and cemented seams in this layer).

Both the grain size distribution results and the DMT index I_d suggest similarity of the material (silty clay) below and above the hard layer. However CPT and DMT results indicate both that the upper material is normally consolidated, while the lower material is overconsolidated (according to DMT, in the lower clay OCR = 3.8).

Reconsolidation curves

Reconsolidation curves, obtained at several depths, proved to be unexpectedly long (given the then available experience). Due to practical constraints, only in one case (at 22.2 m depth) could sufficient time be allowed for reconsolidation (Fig.8). This reconsolidation curve leads (see Table 1) to \( \sigma'_u = \rho \cdot \omega - 3.8 \) bar. (Incidentally, it is noted that, in this case, as in many others, an error of even a few m of water in evaluating \( \omega \) would correspond to a relatively low error in the determined \( \sigma'_u \).) Thus, at 22.2 m depth, the proposed method predicts \( f = \rho \cdot 3.8 = 0.2 \cdot 3.8 = 0.76 \) bar.

Several other reconsolidation tests were performed in the lower clay. Though generally too short, the initial portion of these curves was in general agreement with Fig.8, suggesting that this curve may be taken as representative for the lower clay layer.
USE OF IN SITU TESTS

**Pile Load Tests**

Two test piles were driven (on Oct. 28, 1985) down to the depths 21.6m and 27m respectively, and tested to failure at 79 and 86 days later. The piles were 350 mm diameter driven "Lace" piles, consisting of a thin corrugated steel shell, mandrel driven into the ground then filled with concrete. The two pile lengths were selected in order to enable the determination, by difference, of the friction capacity in the lower clay, avoiding the uncertainty associated with evaluating $f_s$ in the hard layer.

During the load test, the load on the pile was increased monotonically in increments of $\approx 100$ kN, added every 10 min. The piles failed by punching (see e.g. Fig. 9) at approximately 700 kN and 1200 kN respectively, with a difference of 500 kN. Of this difference approximately 20 kN was attributed by using the conventional $f_s = 0.5$ formula, to the different tip resistance at 21.6 and 27m. The remaining 480 kN represent the friction capacity of the lower section, 5.4 m long of the 27m long pile. The lateral area of this section is 5.93 m². By division, one obtains $f_s = 0.81$ bar. This value, however, refers to the depth of 24.3 m (mid height of the section). By transposing 0.81 bar to the depth of 22.2 m (where $f_s$ had been determined), by assuming in this interval $f_s$ proportional to depth, one obtains $f_s = 0.73$ bar. This value agrees quite well with $f_s$ predicted by the proposed method with $\rho = 0.20$ ($f_s = 0.76$ bar).

The same results may be expressed in terms of the backfigured value of $\sigma$ which would have produced coincidence, equal to 0.19 in this case.

![Load-settlement Diagram for the 27 m long Test Pile](courtesy of Icels PalU Rome and Eng. M. Panini, Latina)

**Fig. 9** Load-settlement Diagram for the 27 m long Test Pile

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**REFERENCES**