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The DMT - σ_{hc} Method for Piles Driven In Clay.

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ABSTRACT: This paper presents a tentative method for evaluating the limiting skin friction f_s for piles driven in clay based on σ'_{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) σ'_{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 σ'_{hc} against the dilatometer by using the standard DMT equipment and comments on a number of $\sigma_h(t)$ "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 limit 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 (Baligh,1985). However even the most recent models do not always provide estimates in agreement with observed behavior. One often mispredicted parameter is σ'_{hc} ,the effective horizontal stress against the pile after reconsolidation. Yet σ'_{hc} has been recognized as the dominant parameter in determining f_s (Baligh,1985;Karlsrud and Haugen, 1985). Thus a direct determination of σ'_{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 σ'_{hc} against the flat dilatometer (Marchetti,1980) and presents a tentative method for predicting f_s based on such determined value of σ'_{hc}.

NOTATION

The horizontal effective stress σ'_h at the pile-clay interface varies considerably during the pile life. Therefore specific subscripts are added

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to σ_h' to denote the phase to which σ_h' refers: a) σ_{ho}' in the original pre-penetration conditions b) σ_{hp}' during penetration c) σ_{hc}' at the end of the reconsolidation (prior to shearing) d) σ_{hf}' during quasi-static loading to failure, undrained. Moreover the following symbols are adopted : $K_{oo} = \sigma_{ho}'/\sigma_{vo}'$ and $K_{oc} = \sigma_{hc}'/\sigma_{vo}'$, with σ_{vo}'= initial vertical effective overburden stress.

It should be noted that, in general, $\sigma_{hc}' \neq \sigma_{hf}'$, because, when the pile is loaded, excess pore pressure develops at the interface. Later this paper will focus on the relationship between σ_{hc}' and f_s.

Q_{ult} 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 σ_{hc}' against the pile shaft (Fig.1).
2. As the reconsolidation proceeds, σ_h' at the interface builds up again, up to a long term equilibrium value σ_{hc}' generally higher (sometimes considerably) than σ_{ho}' (see e.g. Azzouz, 1985).
3. The observed total horizontal pressure σ_h against the pile generally decreases significantly during the reconsolidation (see e.g. Baligh, 1985; Karlsrud and Haugen, 1985), somewhat in contrast with the prediction of a variety of cavity expansion-based methods, predicting σ_h constant during reconsolidation.
4. 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; Baligh, 1985).
5. A reliable prediction of σ_{hc}' is a necessary step in predicting f_s', because σ_{hc}' is a controlling factor on the ultimate shaft resistance (Baligh, 1985; Karlsrud and Haugen, 1985).
6. If an estimate of σ_{hc}' is available, then f_s may be evaluated by means of the "skin friction ratio" ρ (Baligh, 1985). The parameter ρ (analogous to the undrained strength ratio S_u/σ_{vo}') is defined as:

$$\rho = f_s \;/\; \sigma_{hc}' \quad \ldots\ldots\ldots\ldots \quad (1)$$

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 α-method and the λ-method (based on the field vane strength $S_{u,FV}$) with f_s backfigured from load tests on piles jacked in the moderately overconsolidated Haga clay (Norway). The predicted f_s were in contrast, even in shape, with f_s "measured", which, on the other hand, were reasonably well predicted by $f_s = \rho \cdot \sigma_{hc}'$ with ρ =0.40 (Karlsrud and Haugen, 1985).
8. The values of ρ reported in the literature, backfigured from load tests on short piles (Karlsrud and Haugen, 1985) or on model piles (Azzouz, 1985) or derived theoretically using the strain path method (Baligh, 1985) are still few, but vary in a relatively narrow range (0.25 to 0.40).
9. According to strain path models (Baligh, 1985) the pile diameter has

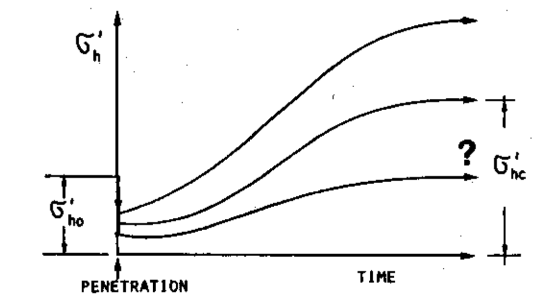

Fig.1. Schematic Diagram illustrating the variation of σ_h' at the Interface caused by the Pile Installation.

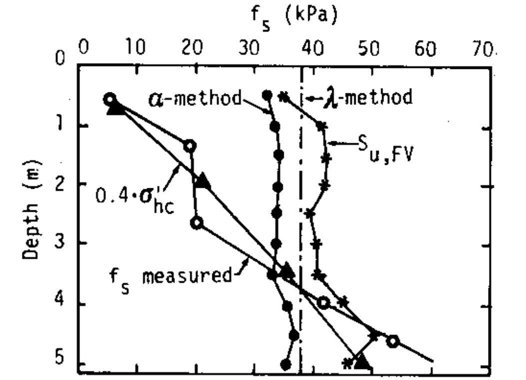

Fig.2. Measured and predicted Limit Skin Friction f_s (Data from Karlsrud and Haugen, 1985).

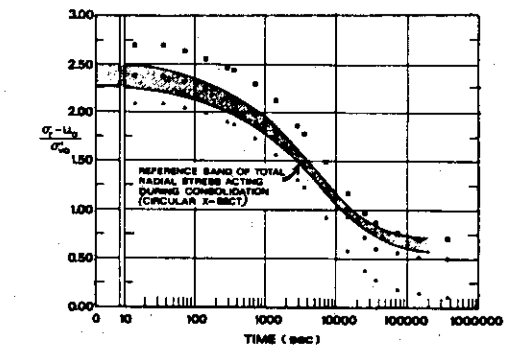

Fig.3. Normalized Values of the Total Radial Stress as Measured with 3 Different Blades compared with Similar Measurements with the Penetrometer, in Boston Blue Clay (Baligh, 1984).

no influence on σ_h at the pile-clay interface at the time of installation and during the subsequent reconsolidation. Experimental results obtained by Baligh (1984) show that, in lightly OC Boston Blue Clay, even the shape (cone, blade) of the penetrating object has minor influence on $\sigma_h(t)$ (Fig.3).

10. Another element supporting indirectly the low dependence of σ_h 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:

- $(u-u_o)/\sigma'_{vo}$ for a circular cross section (where u is the interface pore pressure during undrained penetration and u_o=steady state equilibrium pore pressure) found by Baligh et al.(1985) to be typically, for the BBC they studied, between 2.3 and 2.4.
- $K_D = (p_o-u_o)/\sigma'_{vo}$ for the dilatometer blade (where p_o 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:

Step 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. σ'_{hc} is then obtained as:

$$\sigma'_{hc} = p_{oc} - u_o \quad (2)$$

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

Step 2 . f_s is estimated by the Equation:

$$f_s = \rho \cdot \sigma'_{hc} \quad (3)$$

For the skin friction ratio ρ the value 0.20 is provisorily 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 ~7 times the blade thickness) may tend to overestimate σ'_{hc}. Many additional ρ values, backfigured from pile load tests are necessary, however, for evaluating the method and for getting some information on the dependency of ρ on factors such as soil type and stress history, open ended 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 σ'_{hc} is the value of p_o 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 DMT A-readings are taken (the second DMT B-reading is omitted to avoid the 1 mm deflection, i.e. the pressure is deflated to 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 p_{of} value by using the usual DMT correction formulae (Marchetti and Crapps, 1981).
2. By inspection of the A(t) curve, the residual decrease of A still to occur (ΔA_c) is estimated.
3. Then p_{oc} is obtained as:

$$p_{oc} = p_{of} - \Delta A_c \quad (4)$$

Measurements shown later in the paper indicate that, if the observed A(t) curve includes some part of the stabilized portion, then the errors associated with estimating ΔA_c are generally a low proportion of σ'_{hc}. 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 gage 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 $\sigma_h(t)$ 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 $\sigma_h(t)$ curve relative to the geometry of the blade.
- On the other hand, the $\sigma_h(t)$ 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_o and OCR are 22% and 30% respectively.

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

- $(\sigma_h-u_o)/\sigma'_{vo}$ vs time. This 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 σ'_{vo} and u_o, operations all involving, to some extent, subjective estimates.

SITE	z (m)	σ'_{vo} (bar)	u_o (bar)	PART 1: I_D	OCR	K_{oo}	S_u (bar)	M (bar)	PART 2: p_{of} (bar)	ΔA_c (bar)	p_{oc} (bar)	σ'_{hc} (bar)	K_{oc}	K_{oc}/K_{oo}	PART 3: z (m)	Sand %	Silt %	Clay %	LL	PI
RIETI	4.30	0.49	0.2	0.5	1.5										3.5	26	56	18	25	3
	4.90	0.53	0.3	1	1.5										18	10	56	34	48	24
	17.40	1.39	~1.6	~0.4	~2	~0.8	~0.5	~80												
	17.80	1.41	"	"	"	"	"	"												
	18.20	1.43	"	"	"	"	"	"												
	18.60	1.48	1.7	0.55	1.7	0.7	0.5	70	3.52	0.3	3.22	1.52	1.03	1.47						
SCOPPITO	7.6	0.95	0.42	0.9	4	1.2	0.6	250							11.2	/	62	38	58	27
	7.8	0.96	0.44	0.7	5	1.3	1	250	3.85	0.25	3.6	3.16	3.29	2.53						
	10	1.16	0.66	0.6	7.5	1.4	1.2	280												
	10.4	1.20	0.70	0.6	7.5	1.4	1.2	350												
	10.6	1.21	0.72	0.4	9	1.6	1.6	350												
	11	1.25	0.76	0.4	9	1.8	1.6	350												
LATINA	22.2	1.64	2.1	0.35	3.8	1.15	1.1	160	6.08	0.2	5.9	3.8	2.32	2.01	15	5	60	35	46	24
															22	10	57	33	41	20
															29.5	/	51	49	45	25
PRETURO	18	1.59	1.7	1	5	1.3	1.7	700	10.51	0.4	10.1	8.4	5.3	4.1	18	50	45	5		
	25.6	2.36	2.4	0.45	5.2	1.3	1.9	400	10.96	0.2	10.8	8.4	3.6	2.8	25.6	32	35	33	35	14
	25.9	2.38	2.4	0.35	6.5	1.4	2.3	400	9.7	0	9.7	7.3	3.1	2.2	25.9	8	48	44	41	20

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

Part 1 : Geotechnical Parameters interpreted from "normal" DMT by using the Marchetti (1980) Correlations.

Part 2 : Final Calculations performed at the Conclusion of the Reconsolidation Tests.

Part 3 : Classification Test Results

- A(t) vs time. If the main scope of the test is the determination of σ'_{hc} (for the f_s prediction), then the normalization to σ'_{vo} is not particularly useful, and this more direct form of presentation seems generally preferable.

Tests at Rieti (Central Italy)

The main scope of these tests was to verify the feasibility of obtaining the reconsolidation curves $\sigma_h(t)$ using the standard DMT equipment and to gain some information with the times needed for reconsolidation.

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 u_o).

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.4f, though not fully stabilized, was the first one to justify some extrapolation guesswork. The residual decrease of A was estimated to be ΔA_c=0.3 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_o-gain factor due to the penetration (defined as K_{oc}/K_{oo}) was in this material 1.47.

Tests at Scoppito (Central Italy)

Reconsolidation tests were performed in an overconsolidated stiff silty 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 7.8m shows signs of stabilization. The residual decrease of A was estimated to be ΔA_c=0.25 bar. As shown in Table 1, the estimated K_o-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 mm at the center). In Figs.5c and 5e the asterisks are the results of reconsolidation tests carried out 0.20m 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.

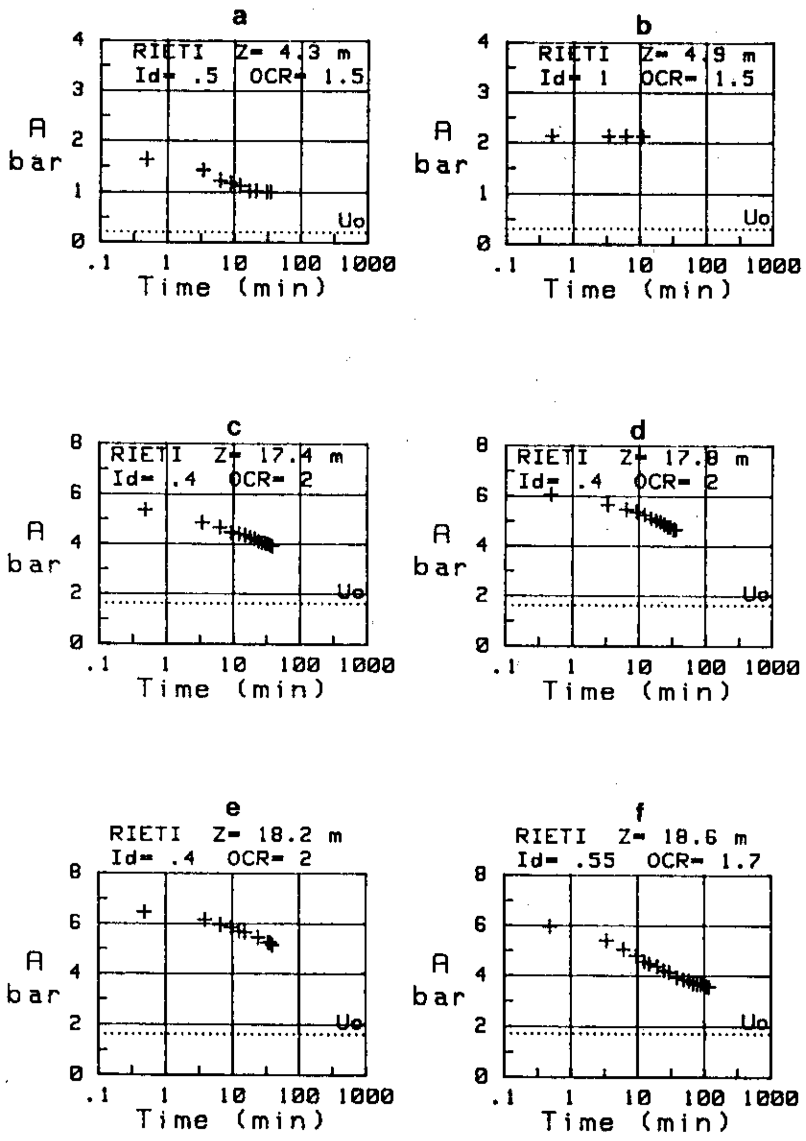

Fig.4 Reconsolidation Test Results at Rieti (1 bar=100 kPa)

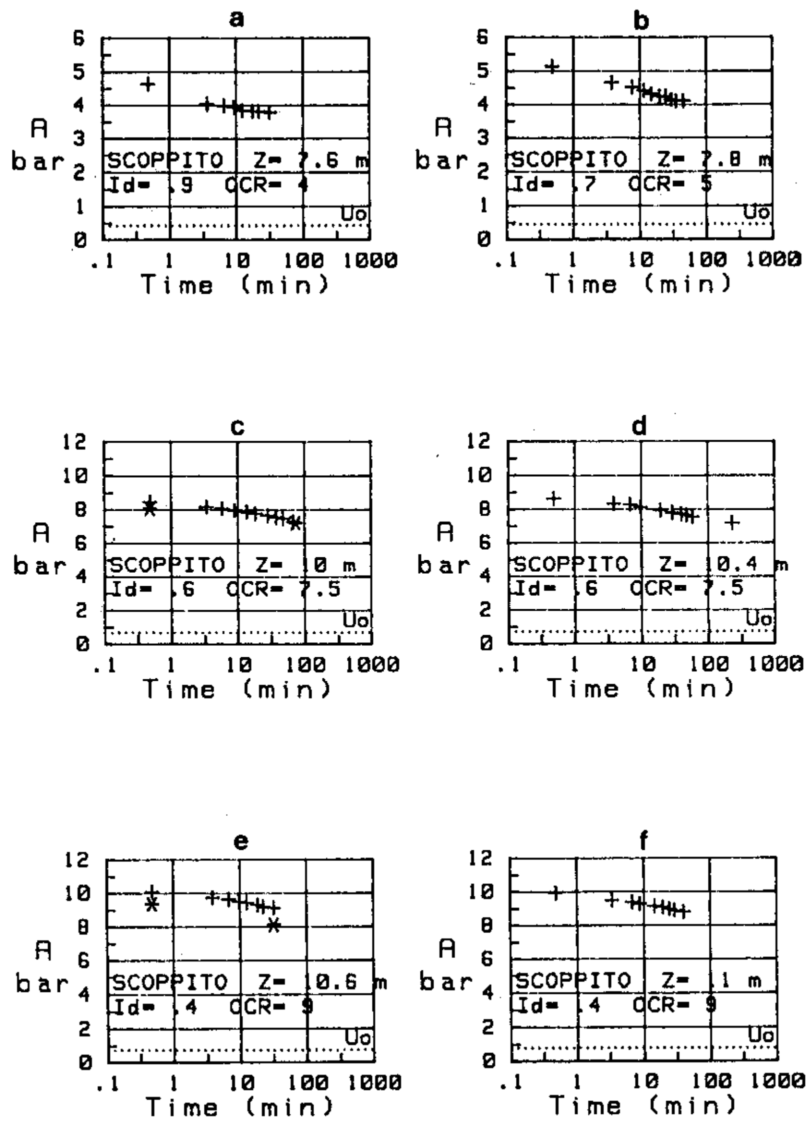

Fig.5 Reconsolidation Test Results at Scoppito (1 bar=100 kPa)

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 ½hour. 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, momentarily, some reduction of the A value. However this reduction was small and apparently "unremembered" 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 satisfacorily 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 sofar, 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 ΔA_c 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 Tedd and Charles, 1981).

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

6. As shown in Table 1, the estimated penetration K_o-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_s predicted by the proposed method with f_s 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.

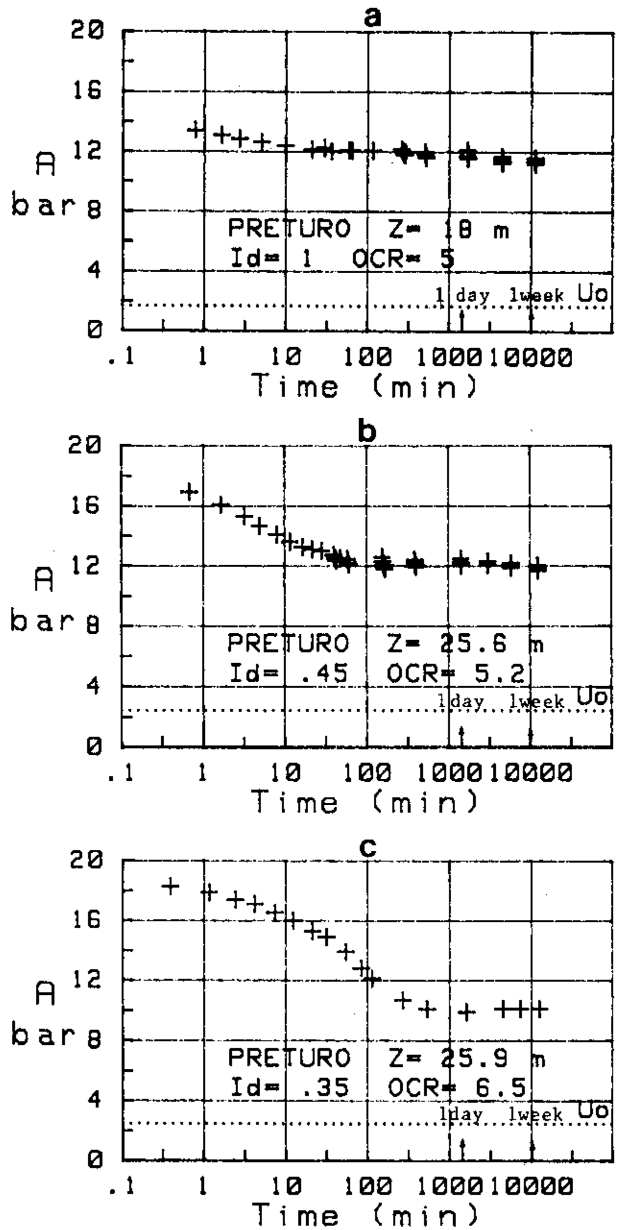

Fig.6 Reconsolidation Test Results at Preturo (1 bar = 100 kPa)

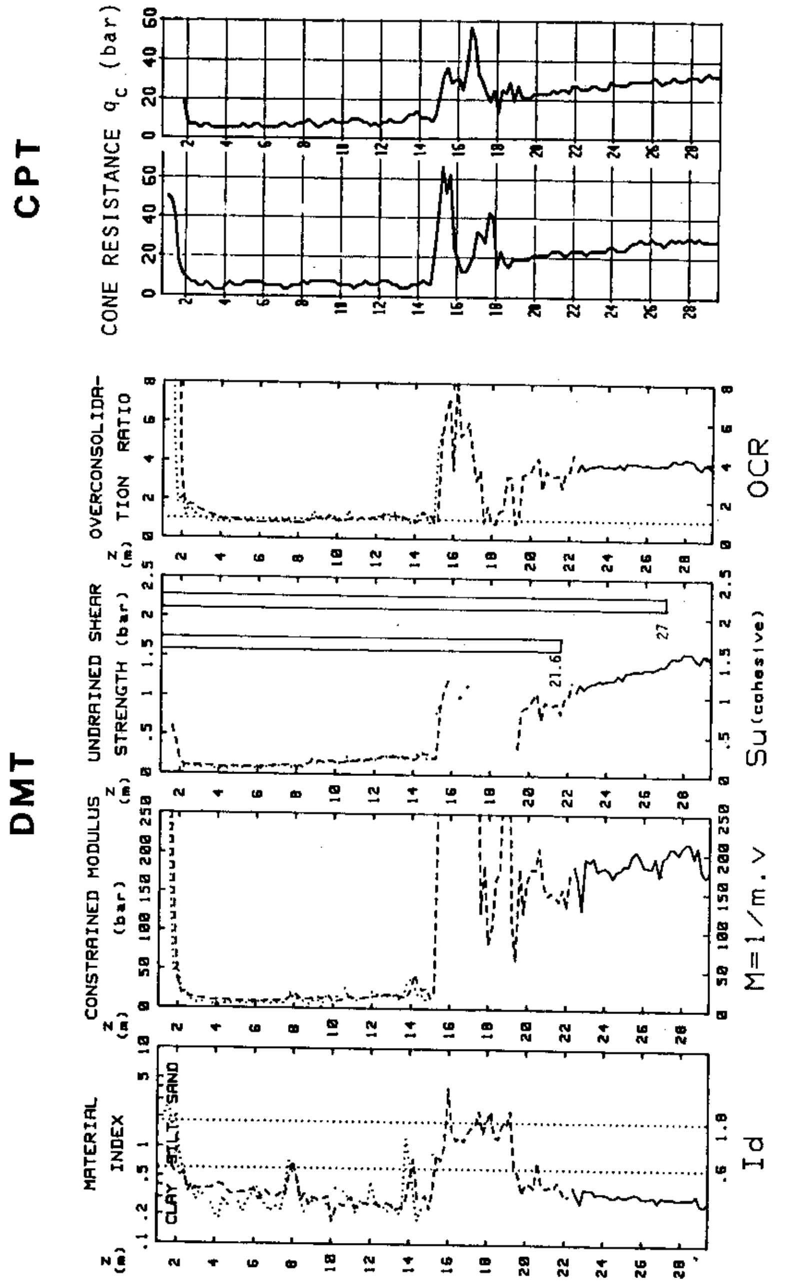

Fig.7. DMT and CPT results at Latina

From both CPT and DMT results is apparent the presence of a harder layer between 15 and 18m 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 $\sim$ 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 $\sigma'_{hc} = p_{oc} - u_o$ = 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 u_o would correspond to a relatively low error in the determined σ'_{hc}). Thus, at 22.2 m depth, the proposed method predicts $f_s = \rho \cdot 3.8$ = $0.2 \cdot 3.8$ = 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.

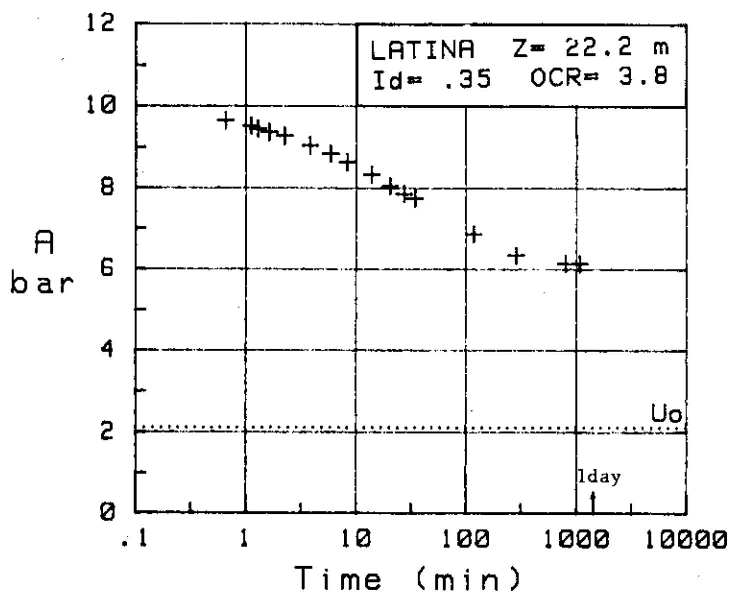

Fig.8 Reconsolidation Test Results at Latina (1 bar=100 kPa)

Pile Load Tests

Two test piles were driven (on Oct.28.1985)down to the depths 21.6m and 27m respectively and test loaded to failure 79 and 86 days later. The piles were 350 mm diameter driven "Lacor" 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 piles was increased monotonically, in increments of ∿ 100 kN,added every 10 min. The piles failed by plunging (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 9 · Su 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 m^2. By division,one obtains f_s=0.81 bar. This value,however,refers to the depth 24.3 m (mid height of the section). By transposing 0.81 bar to the depth of 22.2 m (where σ'_{hc} 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 ρ = 0.20 (f_s=0.76 bar).

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

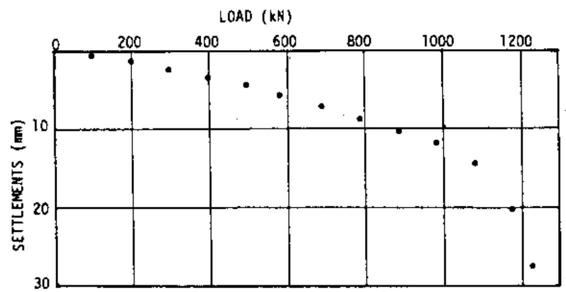

Fig.9 Load-settlement Diagram for the 27 m long Test Pile (courtesy of Icels Pali,Rome and Eng.M.Panini,Latina)

CONCLUSIONS

1. This paper presents a tentative method for evaluating the limit skin friction f_s for piles driven in clay,based on σ'_{hc} (horizontal effective stress at the end of the reconsolidation) determined by DMT. The method rests on the following findings of recent research : (a) σ'_{hc} against a driven pile is a dominant parameter in determining f_s. (b) The horizontal stress σ_h against a pile driven in clay,and its evolution with time,is relatively independent on dimensions and (according to preliminary evidence) on shape of the penetrating object.

2. The standard DMT equipment permits an easy determination of σ'_{hc} against the dilatometer blade,the measurement at the base of the method.

3. The reconsolidation time needed to evaluate σ'_{hc} varied,for the soils tested,from a few minutes to one day,depending on soil type. Tentative recommendations are given in the paper to help deciding when the reconsolidation test may be ended.

4. According to the proposed method, σ'_{hc} is converted into f_s by the equation $f_s = \sigma'_{hc} \cdot \rho$. The few values of ρ reported in the literature vary in a relatively narrow range. The value ρ = 0.20 has been provisorily recommended,until more data become available. In one case study illustrated in the paper,where piles were test loaded to failure,f_s predicted using ρ = 0.20 proved reasonable.

5. In view of the simplicity of the method,a fast accumulation of experimentally determined ρ values should be possible. Indeed many additional ρ determinations relative to various soil and pile types are necessary for a thorough evaluation of the method.

6. The experience gained sofar suggests that the dilatometer may be a practical tool for getting,in a simple way,information about the penetration-induced K_o-gain properties of cohesive soils. In the soils tested,the K_o gain factor ($=K_{oc}/K_{oo}$) was found to vary between 1.5 and 4 . The data obtained,however,do not seem to indicate evident correlations between the K_o-gain factor and OCR or other parameters.

7. The $\sigma_h(t)$ reconsolidation curves may provide some information,at least in a relative sense,on the consolidation characteristics of the soil.

REFERENCES

Azzouz,A.S.(1985),"Lecture 9 of MIT Special Summer Course: Recent Developments in Measuring and Modeling of Clay Behavior for Foundation Design", Cambridge,Mass.,Aug.5-9,1985.

Baligh M.M.(1984),"Personal Communication to M.Jamiolkowski",also reproduced as Fig.39 in Jamiolkowski,M.,Ladd C.C.,Germaine,J.T. and Lancellotta,R.,New Developments In Field and Laboratory Testing,SOA Report,XI ICSMFE,Vol.1,pp. 57-153,S.Francisco,Aug.1985.

Baligh,M.M.(1985),"Lectures 11 and 13 of MIT Special Summer Course : Recent Developments in Measuring and Modeling of Clay Behavior for Foundation Design",Cambridge,Mass.,Aug.5-9,1985.

Baligh,M.M.,Martin,R.T.,Azzouz,A.S.,and Morrison,M.J.(1985),"The Piezo-Lateral Stress Cell",XI ICSMFE,Vol.2,pp.841-844,S.Francisco,Aug.1985.

Campanella,R.G.,Robertson,P.K.,Gillespie,D.G.,and Grieg J.(1985),"Recent Developments in In-Situ Testing of Soils,XI ICSMFE,Vol.2,pp.849-854, S.Francisco,Aug.1985.

Karlsrud ,K. and Haugen,T.(1985),"Axial Static Capacity of Steel Model Piles in Overconsolidated Clay",XI ICSMFE,Vol.3,pp.1401-1406,S.Francisco.

Marchetti,S.(1980),"In Situ Tests by Flat Dilatometer",ASCE Jnl.GED,Vol. 106,No.GT3,pp.299-321.

Marchetti,S. and Crapps D.K.(1981),"Flat Dilatometer Manual",Unpublished Report,Gainesville,Florida,June 1981.

Randolph,M.F.,Carter,J.P. and Wroth C.P. (1979),"Driven Piles in Clay - The Effects of Installation and Subsequent Consolidation",Geotechnique, Vol.29,No.4,pp.361-393.

Schmertmann J.H.(1984),"The New In-Situ Marchetti Dilatometer Test",Geotecnical News,Vol.2,No.3,pp.34-35.