Back analysis of soil parameters from anchor tests

Paramètres à postériori du sol par essais sur ancrages

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SYNOPSIS Field measurements of the load-displacement distribution over the lengths of fixed anchors have been analysed for stress paths very close to the geostatic stresses . Load transfer curves of the fixed anchor in cohesionless and cohesive soils are derived from some case studies in the technical literature . Anchor interaction was modelled by an approximate closed-form non-linear solution, checked using numerical methods . Average shear modulus is deduced from the limit state design graphs, by means of three load-displacement states measured during the test , in particular at the first debonding of the anchor and reaching the shear strength of the fixed length at the head .

INTRODUCTION

The deduction of soil parameters from the full scale anchor test as a method lies between traditional investigation by in situ testing , and experimental large test , anticipating field measurements .
The measure uses the standard procedure employed for preliminary anchor design in many Countries. Disturbed soil around the shaft is limited by accurate installation at low injection pressure and by small diameter , maintaining geostatic stresses and drawing "global" near local soil modulus. The aims of this paper are :1) to present the essential background information on field anchor testing and to model tensile load transfer curves ; 2) to present a method for evaluating in situ the average shear modulus .

FIELD TEST CONSIDERATIONS

The anchors can be installed during the study in a manner following the pull-out test for concrete . In particular , the anchor test can be executed where the maximum design deviator stress is induced in a significant volume of the soil .
The instrumentation is simplified to a load cell and invar wires for measuring fixed-length load--displacements (see fig.1a) without measuring the load distribution .
During the test , the tensile load increases monotonically , each load increment being maintained for 60 min. , so as to analyze creep phenomena and the corresponding critical load . The ciclic procedure is reserved to analysing shear modulus degradation for slow phenomena ; corresponding load-displacement curves can be measured after the monotonic procedure .

THE LOAD TRANSFER MODEL

The interaction of the fixed anchor length with the surrounding soil may be analyzed as the shearing of concentric cylinders (Cooke, 1974; Randolph and Wroth, 1978; Scott, 1981; Frank, 1974 and 1983) : total stresses and model are shown in fig.1b .
Load transfer is assumed to be a non-linear function of the displacement s over an initial length :

$$\pi D \tau = \pi D \, \tau_f (1 - e^{Vs}) \qquad (1)$$

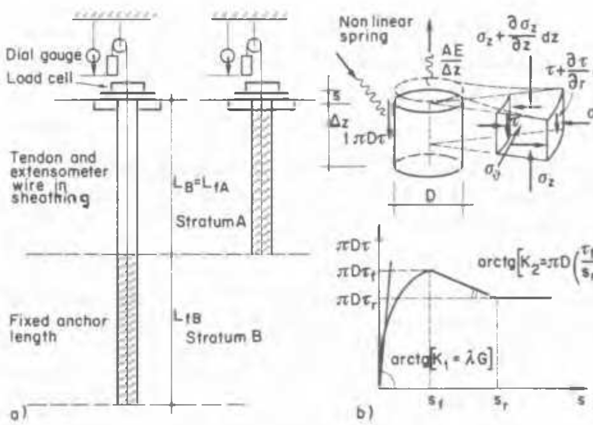

Fig.1- a) Schematic instrumentation of the anchor tests in selected strata
b) Non linear load transfer model

and over the remaining length ,to be a linear function :

$$\pi D \tau = \lambda G s \qquad (2)$$

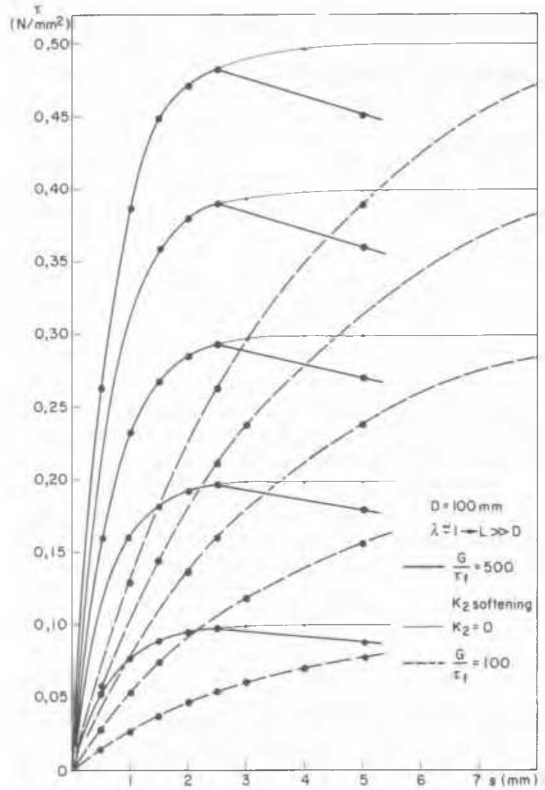

Fig.2- Load transfer curves modelled with or without softening (K_2=0) and characterized by 2 different failure indices (shear modulus/shear strength: G/τ_f) .

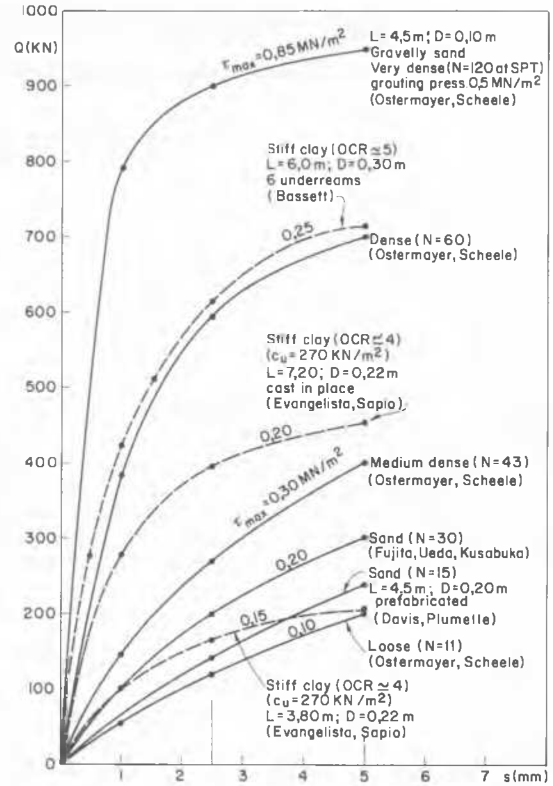

Fig.3- Load-displacement curves and corresponding maximum shear stress found in dense sands (continuous lines) and in stiff clays (dashed lines), by different Authors.

where , for τ_f and G constant (global average values) over the two lengths,

$$V = \lambda G / \pi D \tau_f \qquad (3)$$

thus being proportional to the soil's Vésic failure index G/τ_f , and :

$$\lambda = 2\pi / \ln\left[4(1-\nu)\, L/D\right] \qquad (4)$$

which is a function of the radius at which the shear stress becomes negligible in an elastic medium . Equation (1) gives $\tau = \tau_f$ for $Vs \gg 5$ and a linear solution $\tau \simeq \tau_f Vs$ for $Vs \leq 0{,}14$ (range of validity of the first term of the series development). For small displacement , the solution is also equivalent to the hyperbolic model :

$$\tau = \tau_f \frac{Vs}{1+Vs} \simeq \tau_f Vs - \tau_f V^2 s^2 \qquad (5)$$

Figure 2 shows ten theoretical load-transfer curves for λ=1 , or L>>D , and D=100 mm (typical for boring) and for two ratios G/τ_f=100 , or V==0,3 mm^{-1}, and G/τ_f=500 , or V=1,5 mm^{-1}.

In general , there is radial variation of the shear modulus , both after installation and after consolidation but for small displacements : $G_{ave} \simeq G_{\infty}$ of the undisturbed zone . The width of the disturbed zone around the anchor owing to low pressure injection is however small in relation to the length L, being proportional to the diameter D .

The effects of the negative pore water pressure, in comparison with hydrostatic values ("suction" effect), during consolidation and corresponding effective stress path have not been examined . On the other hand , a large portion of the displacement takes place immediately, especially for small s values . In particular, the limitation of the displacement to 1 mm permits the drained condition in 60 min. , for various soils , and for elastic isotropic model the undrained shear modulus coincides with the drained : G_u=G'.

BACKGROUND ON THE CASE STUDIES

In order to minimize the alteration of the geostatic stresses in the technical literature was considered only anchors executed at nil or low pressure . Fig. 3 shows typical load-displacement curves (continuous lines) for gravelly sand (Ostermayer et al., 1977) and for fine sand (Fujita et al., 1977; Davis et al., 1982) at different relative densities ; the dashed curves are for different stiff clays (Evangelista et al., 1977 and Basset, 1977) .

Displacement is limited to s_{max}=5 mm , this being generally higher than the critical displacement corresponding to local head anchor-soil debonding (start of plastic phenomena). The diameter and length of the anchor considered is generally D=0,1 m and L=4,5 m .
The load transfer curves $\pi D \tau_{max}$ versus s_{head} derivable from measured load distributions (indicated with τ_{max} in fig.3) are similar to the theoretical curves of fig.2. For example the transfer shear stress versus head displacement for the anchor in Ostermayer medium dense sand (see fig.3) corresponds to the dashed curve with τ_{max}=0,3MN/m² in fig.2 .
The softening shown in fig.2 is not found in the tests of fig.3 , but a small effect was introduced for a conservative numerical analysis .

IN SITU SHEAR MODULUS

A numerical program was developed combining a load transfer model with the vertical equilibrium equation : $dQ/dz = \pi D \tau$ and with vertical compatibility $s = 4\int_0^z Q dz / E \pi D^2$; no account was taken of radial compatibility . The boundary conditions are vertical anchor, $Q=Q_o$, $s=s_o$ for z=0 and Q=0 for z=L. In particular, a numerical method was developed for load transfer curves with softening . The anchor-soil stiffness matrix employed permits the analysis of : load, displacement, strain and shear-stress distributions , according to f.e.m. solution in elastic range (Esu, 1973; Banerjee 1978; Poulos and Davis, 1980) .
Load distributions at three different displacements s_o=0,1 , 2,5 and 5,0 mm , are plotted in fig. 4 and 5 respectively for L=3,0 m and L=7,0 m . Intermediate lengths have not been discussed here, but linear interpolation may be done .
Figures are shown for soils having G/τ_f=100 and G/τ_f=500 , to let the stifnesses be compared. The average shear modulus G is definable by the Q-L curves plotted for s_o=1 mm $[Q(1)=Q_{elast}]$. This displacement generally corresponds to an elastic soil and anchor behaviour ($\varepsilon_{tr} \leq$ 0,2‰), according to experimental load distributions. The maximum tensile strain ε_{tr}=0,2‰, allowable for anchor grouting ,and the corresponding "elastic" length are indicated in the Q-L curves at s_o=2,5 mm and 5,0 mm .
These curves are plotted when τ first reaches τ_f in the head of the fixed anchor length and, for examined soil, corresponding to the critical load $Q(2,5)=Q_{crit}$ for G/τ_f=500 or $Q(5,0)=Q_{crit}$ for G/τ_f=100. This Q_{crit} valuation at the "first plastic point" is very different from that based on the maximum experimental gradient of the creep phenome_

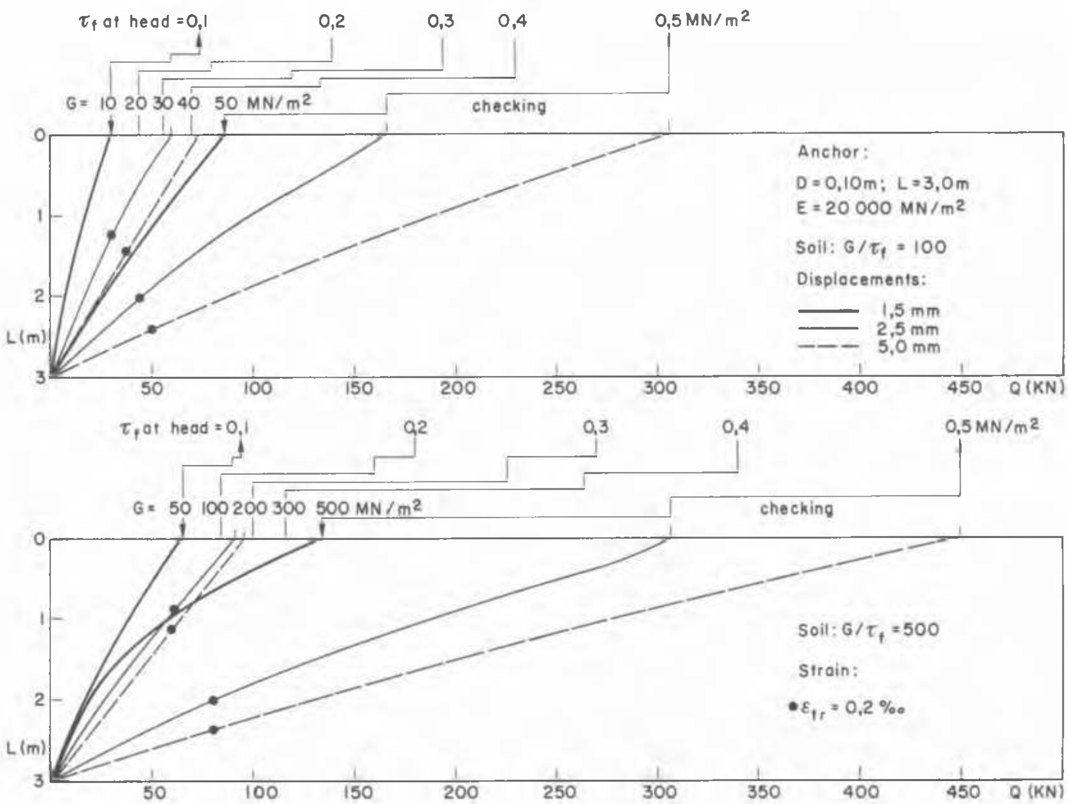

Fig.4- The average shear modulus G deduced from tensile load distributions at three limit head displacements of the anchor L=3,0 m, fixed in a soil of G/τ_f=100 and G/τ_f=500 respectively .

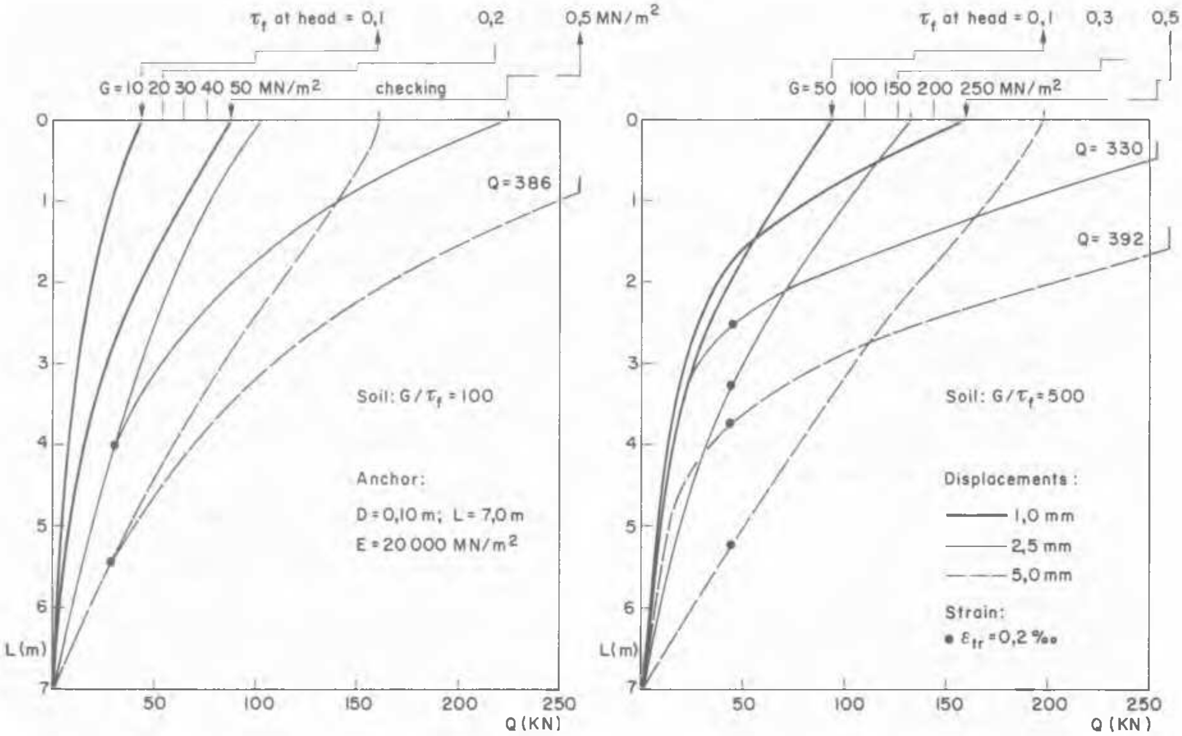

Fig.5- The average shear modulus G deduced from tensile load distributions at three limit head displacements of the anchor L=7,0 m, fixed in soil of G/τ_f=100 and G/τ_f=500 respectively .

na ,but it is still a reasonably close indicator for the soils examined . Figures 4 and 5 thus show the limit state design graphs for the evaluation of the G; better fitting is possible utilizing Q(1,0),Q(2,5) and Q(5,0) and corresponding head shear strengths . Checking the "spreads" between the 3 limit-state curves permits a more selective evaluation of the shear modulus . For example (see fig.4), for the anchor with L=3,0 m we find G=50 MN/m², for these measurements: Q(1,0)= =96 KN, Q(2,5)=166 KN and Q(5,0)=308 KN, for G/τ_f= =100; and for these measurements : Q(1,0)=66 KN, Q(2,5)=90 KN and Q(5,0)=94 KN, for G/τ_f =500 . But for two different soils, they are respectively τ_f=0,5 MN/m² and τ_f= 0,1 MN/m² definable by routine tests .

CONCLUSIONS

The evaluation outlined in this paper is based on a model for the transfer of load from the anchor to the soil that is in agreement with experimental results .A relatively simple and usual standard procedure may be employed during the investigation .
The measurement of tensile loads at different displacements permits the in situ evaluation of the global average shear modulus (figs 4-5),to improve the determination of the subsoil profiles .
A non-linear load transfer model was calibrated by numerical simulation to plot the design graphs. They may also be used to determine the fixed anchor length .

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