NGI

CONTRACT REPORT

DET NORSKE VERITAS

BLOCK 58, WEST DELTA AREA GULF OF MEXICO

SEGMENT PILE TEST. PREDICTIONS OF RESPONSE DURING PILE INSTALLATION AND MONOTONIC STATIC AXIAL LOADING.

81222-3 1st SEPTEMBER, 1982

Norges Geotekniske Institutt

CONTRACT REPORT

DET NORSKE VERITAS

BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

SEGMENT PILE TEST.
PREDICTIONS OF RESPONSE DURING PILE INSTALLATION AND MONOTONIC STATIC AXIAL LOADING

81222-3 1st SEPTEMBER, 1982

THIS REPORT CONTAINS ESTIMATES OF:

- SKIN FRICTION DURING INSERTION
- PRESSURE CHANGES DURING INSERTION
- DISSIPATION OF PORE PRESSURES AFTER INSTALLATION
- STRESS CONDITIONS AFTER CONSOLIDATION
- LOAD-DEFORMATION CHARACTERISTICS VALID FOR MONOTONIC STATIC LOADING

ALSO A FEW IDEAS FOR A LOADING PROGRAM HAVE BEEN PRESENTED.

cont...

Norwegian Geotechnical Institute NGI

Address:	Telephone:	Telex:	Telegrams:
P.O.Box 40 Tåsen Oslo 8 Norway	(02) 23 03 88	19787 ngi n	GEOTEKNIKK

NGI

RESUME

The estimated immediate increase of horizontal total and effective stresses as well as the increase of pore pressure are given in Table 5, as a function of the wall thickness of the cutting shoe. All values are normalized with respect to the in situ overburden effective stress σ_{vo}', which is an unknown quantity in the soil profile in question. Typical range of values estimated are:

Increase of horizontal total stress: $\frac{\Delta\sigma_h}{\sigma_{vo}'} = 1.43\text{-}0.88$

Increase of pore pressure: $\frac{\Delta u_o}{\sigma_{vo}'} = 1.21\text{-}0.66$

Increase of horizontal effect stress: $\frac{\Delta\sigma_h'}{\sigma_{vo}'} = 0.22$

The first number is valid for closed-ended segment pile while the latter is valid for a cutting shoe wall thickness of t = 1.5 mm. These predictions were based on very simple elasto-plastic expansion theory. The predicted effective stress *increase* is probably not correct as we would expect a horizontal effective stress *decrease* to occur immediately after pile installation.

Based on estimated remoulded undrained shear strength, the predicted skin friction during pile insertion is

$$\tau_{oi} = 0.44\ Z\ \text{kPa}$$

where Z = penetration depth in metres.

The predicted necessary time to reach 90% consolidation may vary between 1 day for a cutting shoe wall thickness of 1.5 mm to 7 days for a closed end pile.

NGI

Predictions of the load-displacement behaviour of the pile segment are limited to monotonic static loading only. Maximum skin friction is estimated to be mobilized at a displacement of the order of 0.7 mm in soil stratum II and 1.6 mm in soil stratum III. These estimates were based on a theoretical model where NGI's experience with shear modulus variation as a function of shear stress level was incorporated.

On the following pages are presented a more detailed description of the analyses and estimates applied.

for the
NORWEGIAN GEOTECHNICAL INSTITUTE

Ove Eide

Fritz Nowacki

FN/mjs

NGI

CONTENTS: PAGE:

LIST OF DRAWINGS

NGI

1. INTRODUCTION

The Norwegian Geotechnical Institute (NGI) is serving as consultants to Det norske Veritas, Norway, in connection with a Tension Pile Study carried out for Conoco Norway Inc.

This report presents predictions of pile and soil response during pile segment installation and subsequent static (monotonic) loading. The predictions are made using available theoretical and empirical analyses.

Closed-ended as well as open-ended pile segments have been considered.

2. SOIL CONDITIONS

Soil conditions are described in Ref. 1.

3. SMALL DIAMETER SEGMENT PILES

The steel pipe pile analysed has dimensions

diameter	ϕ = 3" = 0.0762 m
assumed total length	$L_t \cong$ 90" = 3.3 m*
length of instrumental section	L_s = 36" = 0.9144 m*

Further known details are shown in Drawing 040.

* Assumed prior to receipt of Ertec Report Volume II, dated 19th August, 1982.

4. IN SITU STRESS CONDITIONS

Due to the uncertainty with respect to the in situ pore pressure conditions, an upper and lower bound of in situ effective stresses were considered in these predictions.

The horizontal in situ stress conditions were evaluated on the basis of a relationship between K_o', I_p and OCR given in Ref. 4. The following values for K_o' have been applied:

Stratum	Assumed OCR	Average I_p %	K_o'
I	1.0	40	0.65
II	1.0	30	0.60
III	1.0	50	0.70

Table 1. Assumed earth pressure coefficients at rest.

The "interpreted maximum past pressure" as given in Ref. 1 was suggested as the lower bound of effective stress. The stresses based on this assumption are shown in Table 2, while the stresses assuming hydrostatic pore pressure conditions are shown in Table 3.

Depth below mudline m	Total vertical stress	Assumed pore pressure	Vertical effective stress	Horizontal effective stress
	kPa			
0.0	162	162	0	0
15.2	401	349	52	34
24.4	552	486	66	43 40
48.8	977	872	105	63 74
77.1	1451	1142	309	216

Table 2. Assumed minimum in situ effective stress conditions. Data from Ref. 1, Plates 12A, 31, 32.

Depth below mudline m	Total vertical stress	Assumed pore pressure	Vertical effective stress	Horizontal effective stress
	kPa			
0.0	162	162	0	0
15.2	401	323	78	51
24.4	552	415	137	{89 82
48.8	977	660	317	{190 222
77.1	1451	944	507	355

Table 3. Assumed maximum in situ effective stress conditions (Hydrostatic in situ pore pressure). Data from Ref. 1, Plates 12A, 31, 32.

5. PILE INSTALLATION

5.1 *PRESSURE CHANGES AGAINST SEGMENT PILE IMMEDIATELY AFTER INSTALLATION*

Simple elasto-plastic cylindrical cavity expansion theory predicts the following maximum excess pore pressure u_o at the pile surface immediately after installation (Refs. 6 and 7):

$$\Delta u_o = s_u \ln \left(\beta \frac{\overline{G}}{s_u}\right) \qquad (1)$$

where

s_u = undrained shear strength, chosen to be 0.22 σ_{vo}' in the present predictions.

$\overline{G}$ = average secant shear modulus of the soil in the elastic zone.

β = $1 - \left(\frac{r_i}{r_o}\right)^2$

r_i = inner radius of pile segment

r_o = pile segment radius.

Total pressure changes at the pile surface can also be estimated on the basis of cavity expansion theory (Ref. 6):

$$\Delta\sigma_h = s_u \; (1 + \ln \; (\beta \frac{\overline{G}}{s_u})) \qquad (2)$$

The immediate increase of effective stresses can be estimated by subtracting Eq. 1 from Eq. 2:

$$\Delta\sigma_h' = \Delta\sigma_h - u_o = s_u \qquad (3)$$

These formulae indicate an increase of the effective stress immediately after pile installation. We do not believe this to be correct for the soil type in question. According to our experience the immediate increase in pore pressure will be larger than the increase in total stress in this type of soil. This means that the theory either overpredicts the total stress increase or underpredicts the increase in pore pressure.

The major problem in connection with practical application of this simple elasto-plastic theory is to select an appropriate average shear modulus (or ratio $\overline{G}/s_u$).

Based on the available laboratory data and our experience, the following initial shear modulus were estimated (G_o = shear modulus for very small shear strain levels):

Stratum	Assumed OCR	Assumed G_o/s_u
I	1.0	700
II	1.0	800
III	1.0	500

Table 4. Estimated G_o/s_u-values.

Further on, it was assumed that an appropriate average $\overline{G}/s_u$-value is equal to half the assumed initial value, i.e.

$$\overline{G}/s_u \cong \tfrac{1}{2} \; G_o/s_u \qquad (4)$$

NGI

A summary of the predicted ranges of excess pore pressure and total horizontal stress increase immediately after installation is shown in Table 5. The values were normalized with respect to the in situ vertical effective stress. The range given reflects the range of G_o/s_u given in Table 4.

Wall thickness of cutting shoe mm	r_i mm	$\beta = 1 - (\frac{r_i}{r_o})^2$ -	$\Delta\sigma_h/\sigma_{vo}'$ -	$\Delta u_o/\sigma_{vo}'$ -	$\Delta\sigma_h'/\sigma_{vo}'$ -
38.1*	0.0	1.0	1.43-1.54	1.21-1.32	0.22
9	29.1	0.42	1.24-1.35	1.02-1.13	0.22
6	32.1	0.29	1.16-1.26	0.94-1.04	0.22
3	35.1	0.15	1.02-1.12	0.80-0.90	0.22
1.5	36.6	0.08	0.88-0.98	0.66-0.76	0.22

* Closed-end segment pile

Table 5. Summary of estimated stress changes at the pile surface immediately after segment pile installation.

5.2 SKIN FRICTION DURING INSERTION

The estimated penetration resistance has been based on estimates of the remoulded shear strength of the clay. Only a very few measurements of the sensitivity has been made during the laboratory investigations. Two tests reported in Ref. 3 may indicate the following sensitivities:

Stratum	S_t
II	~1.5
III	~2.0

A method for calculation of remoulded strengths based on index properties has been suggested in Ref. 8. Remoulded strengths calculated from this method have been plotted on Drawing 042, where also other shear strength estimates and results have been plotted.

The predicted minimum skin friction (τ_{oi}) during insertion can be described by the equation

$$\tau_{oi} = 0.44\ Z\ \text{kPa} \qquad (5)$$

where Z = penetration depth in *metres*. The estimate is valid for full displacement pile and a rate of penetration of the order of 2 cm/min. The predicted minimum is slightly lower than the measured sleeve friction from CPT results.

Application of a higher rate of penetration and an open-ended cutting shoe may increase the skin friction.

6. DISSIPATION OF EXCESS PORE WATER PRESSURE

The solution given in Ref. 9 of the dissipation problem was applied in these predictions. The solution has been, however, modified somewhat based on pore pressure dissipation test results described in Ref. 10.

The following modification of the theory was carried out:

$$T^* = \frac{1}{\eta} T \qquad (6)$$

where

$T^* = \frac{c_r t}{r_o^2}$ = empirical adjusted time factor

η = empirical coefficient suggested as

$\frac{\Delta u}{\Delta u_o}$	η
0.2	1
0.4	2
0.6	3
0.8	4

T = theoretical time factor for cylindrical drainage given in Ref. 9.

c_r = radial coefficient of consolidation.

t = time after pile insertion

r_o = outer radius of pile segment.

The solution, which also is based on expansion cavity theory, require an estimate of the soil stiffness ratio $\overline{G}/s_u$ analogous to the solutions applied in Section 5 of this report.

For the case of using a hollow cutting shoe the consolidation problem is not yet correctly solved to our knowledge. As a preliminary suggestion we have applied the factor β described in Section 5.1 in combination with the stiffness ratio $\overline{G}/s_u$.

Based on the laboratory results carried out on the soil material in question, we estimated to use the following coefficient of consolidation

$$c_r = 1 \text{ m}^2/\text{year}$$

in the calculations. This value was assumed to be representative for Structures II and III in the normally consolidated stress range.

The results of the analyses are shown in Drawing 043. It is seen that the necessary time for 80-90% dissipation may be 4-7 days in case of a close-ended pile segment. Depending on the wall thickness of the cutting show applied, this time may be reduced to 1-2 days or less. The theory used does not provide solutions for higher or lower ratios of $\beta\ \overline{G}/s_u$ than the curves shown.

NGI

7. STRESS CONDITIONS AFTER CONSOLIDATION

Based on linear consolidation theory one would predict that the total horizontal stress would remain constant. This means that the final effective horizontal stress would be $\sigma'_{ho} + \Delta\sigma_h$. However, based on practical experience we suggest that the stress conditions along the segment pile will be

$$\sigma_v' \rightarrow \sigma'_{vo}$$
$$\sigma_h' \rightarrow 1.1\ \sigma'_{ho}$$
$$\Delta u \rightarrow 0$$

after complete consolidation.

8. STATIC LOADING

8.1 SKIN FRICTION

The following maximum skin friction was predicted

$$f = \alpha\, s_u \qquad (7)$$

where

α = 0.9 provided that at least 90% pore pressure dissipation is waited for
s_u = 0.22 σ_{vo}'
σ_{vo}' = in situ vertical effective stress.

The estimated static skin friction is consequently

$$f \cong 0.2\ \sigma_{vo}'.$$

NGI

8.2 LOAD-DISPLACEMENT CHARACTERISTICS UNDER STATIC LOADING

A theoretical calculation of a t-z curve for static, monotonic increasing loading was carried out.

The calculations were based on the following assumptions:

Shear stress distribution

$$\tau = \frac{\tau_o \, r_o}{r} \tag{8}$$

where

τ_o = shear stress at the pile surface
r_o = outer pile radius
r = radial distance from the pile centre line.

Secant soil shear modulus

$$G_s = G_o \, e^{-c \left(\frac{\tau}{s_u}\right)^n} \tag{9}$$

where

G_o = initial shear modulus
c = coefficient (parameter)
n = coefficient (parameter)

Integration of shear strain over a soil slice

$$Z_s = r_o \int_1^{\xi_m} \frac{\tau}{G_s} \, d\xi \tag{10}$$

where

$\xi = r/r_o$

NGI

$\xi_m = r_m/r_o$

r_m = "magic" distance from the pile where the shear strain is assumed negligible. The solution is not sensitive to changes of r_m.

Soil data obtained on Drammen clay was used in order to assess values for the soil parameters c and n.

Predictions for the soil and pile conditions in question have been presented in Drawing 044 as normalized curves.

9. COMMENTS ON THE TEST PROGRAM

We recommend that a major part of the tests should be carried out applying a static and cyclic stress level relevant to a foundation pile.

Tentatively, we suggest the following test sequence:

a) After installation of pile segment to the scheduled depth, await 90% consolidation .
b) Perform a static test which is terminated at the initiation of failure.
c) Reduce the load immediately to $x\sigma_{vo}'$ as a static shear stress along the pile segment.
d) Apply a cyclic shear stress of $\pm y\sigma_{vo}'$ with a load period of 5 seconds (i.e. the shear stress along the segment is varied between $(x + y)\sigma_{vo}'$ and $(x - y)\sigma_{vo}'$). The cyclic loading should be terminated at a certain number of load cycles (1500) or at a certain level of large deformations.
e) Perform a static test immediately after termination of cyclic loading.

The factors x and y should be varied in the programs within the following ranges:

x = 0-0.15

y = 0-0.22

The values should not be changed during a cyclic test.

Creep data at various stress levels as well as effects from static and cyclic preshearing at various stress levels prior to a severe cyclic loading will also be of great interest if time permits.

10. REFERENCES

1. Ertec (1982)
   Tension pile study. Volume I.
   Site investigation and soil characterization study at Block 58, West Delta Area, Gulf of Mexico.
   Report No. 82-200-1. April 1982.

2. Institutt for geoteknikk, NTH (1982)
   Conoco Tension Pile
   Triaxial and oedometer tests on Cyl. No. 71 and 119 from McClelland's Boring 5.
   Report 0.82-02-1. May 1982.

3. Norwegian Geotechnical Institute (1982)
   Block 58, West Delta Area, Gulf of Mexico.
   Laboratory Report.
   Contract Report 81222-2, dated 18th June, 1982.

4. Brooker, E.N. and H.D. Ireland (1965)
   Earth pressure at rest related to stress history.
   Canadian Geotechnical Journal. Vol. 2, No. 1, pp. 1-15.

5. Ertec (1981)
   Final Technical Report
   Subproject CNRD 13-1.
   Project No. 81-204. Dated 28th August, 1981.

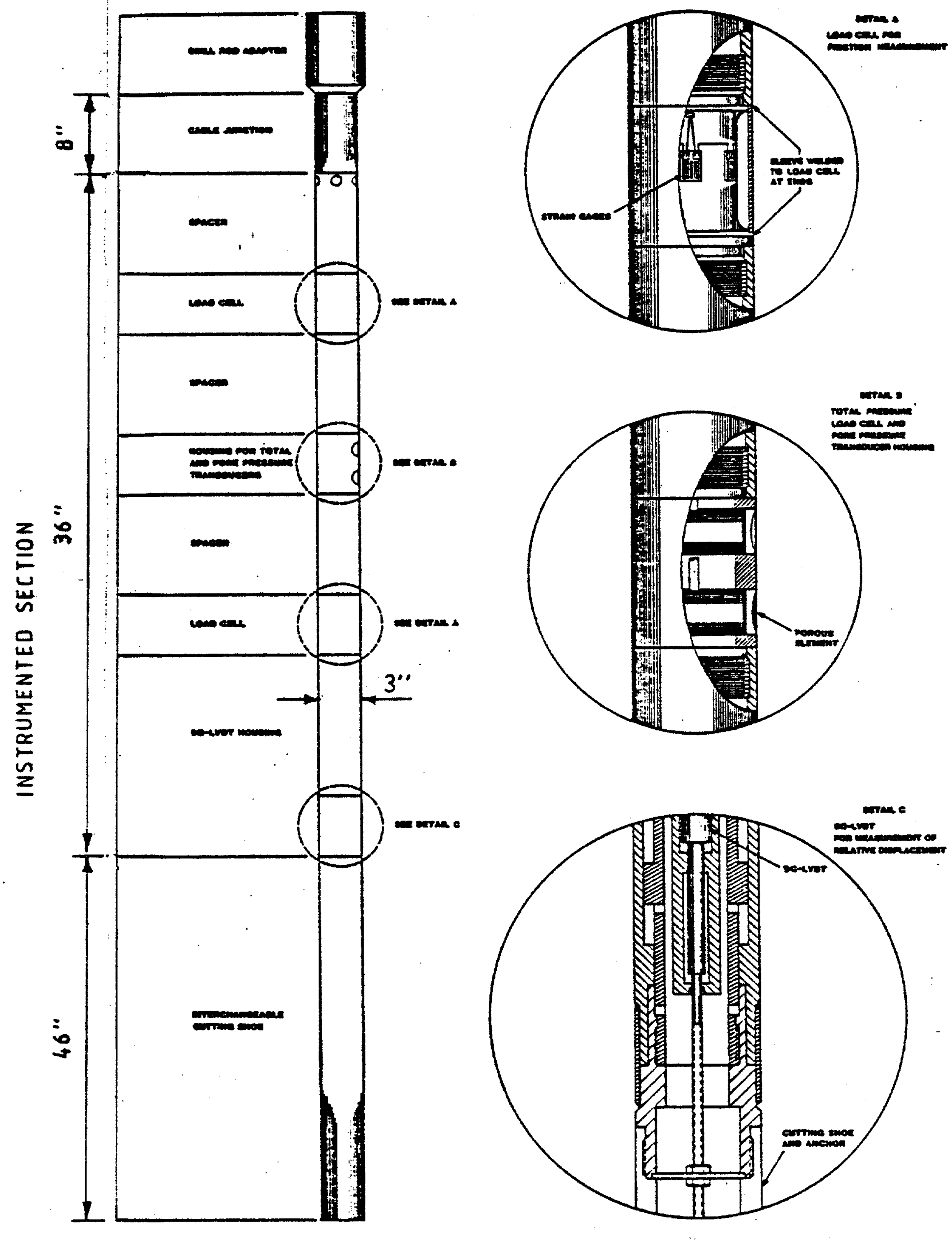
6. Randolph, M.F., J.P. Carter and C.P. Wroth (1979)
   Driven piles in clay - the effect of installation and subsequent consolidation.
   Geotechnique 29, No. 4, pp. 361-393.

7. Randolph, M.F. and C.P. Wroth (1979)
   An analytical solution for the consolidation around a driven pile.
   Int. J. Numer. Anal. Methods Geomech.
   Vol. 3, No. 3 July-September 1979, pp. 217-230.

8. Wroth, C.P. and D.M. Wood (1978)
The correlation of index properties with some basic engineering properties of soil.
Canadian Geotechnical Journal
Vol. 15, No. 2, May 1978.

9. Torstensson, B.-A. (1977)
The pore pressure probe.
Geoteknikkdagen 1977, Oslo.
Paper No. 34, pp. 34.1-34.15.

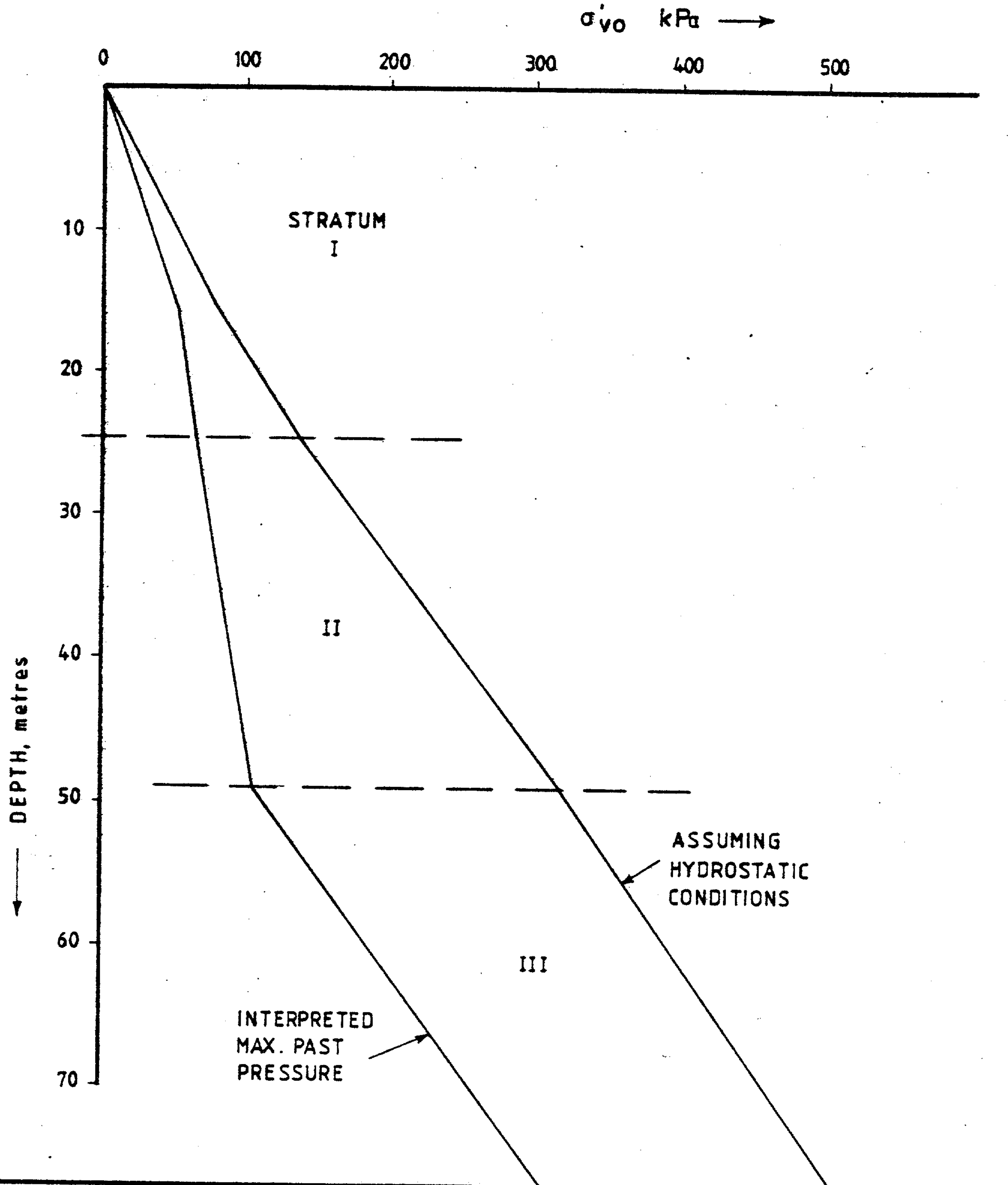
10. Gillespie, D. and R.G. Campanella
Consolidation characteristics from pore pressure dissipation after piezometer cone penetration.
Soil Mechanics Series No. 47.
Department of Civil Eng., University of British Columbia, Vancouver, Canada.



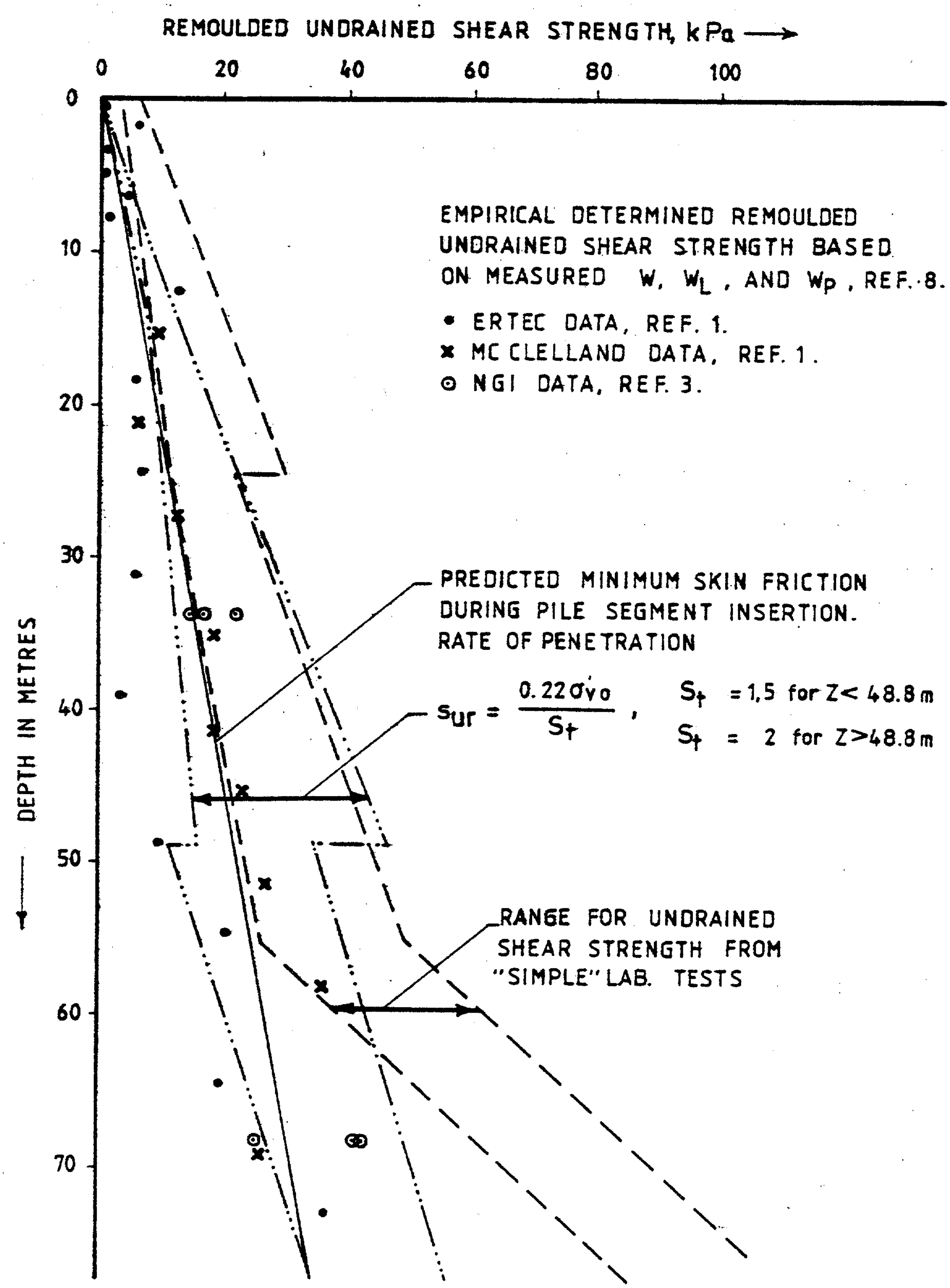
INSTRUMENTED PILE SEGMENT MODEL

SEGMENT LENGTH DATA TAKEN FROM REF. 5, PLATE 19.

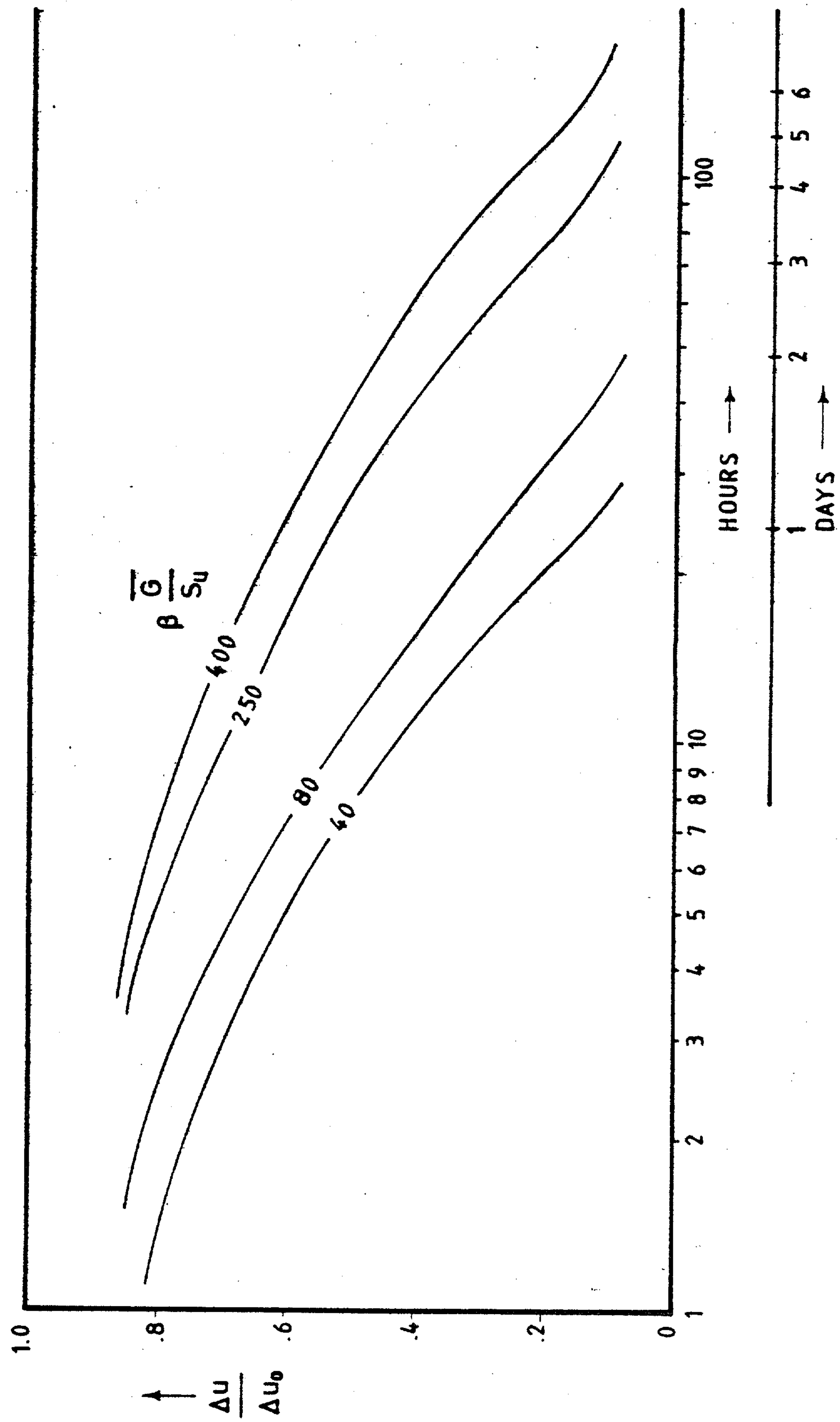
CONOCO TENSION PILE STUDY	Date	Drawn by
INSTRUMENTED PILE SEGMENT MODEL	Approved	
	Project no. 81222-3	
Norwegian Geotechnical Institute	Drawing no. 40	



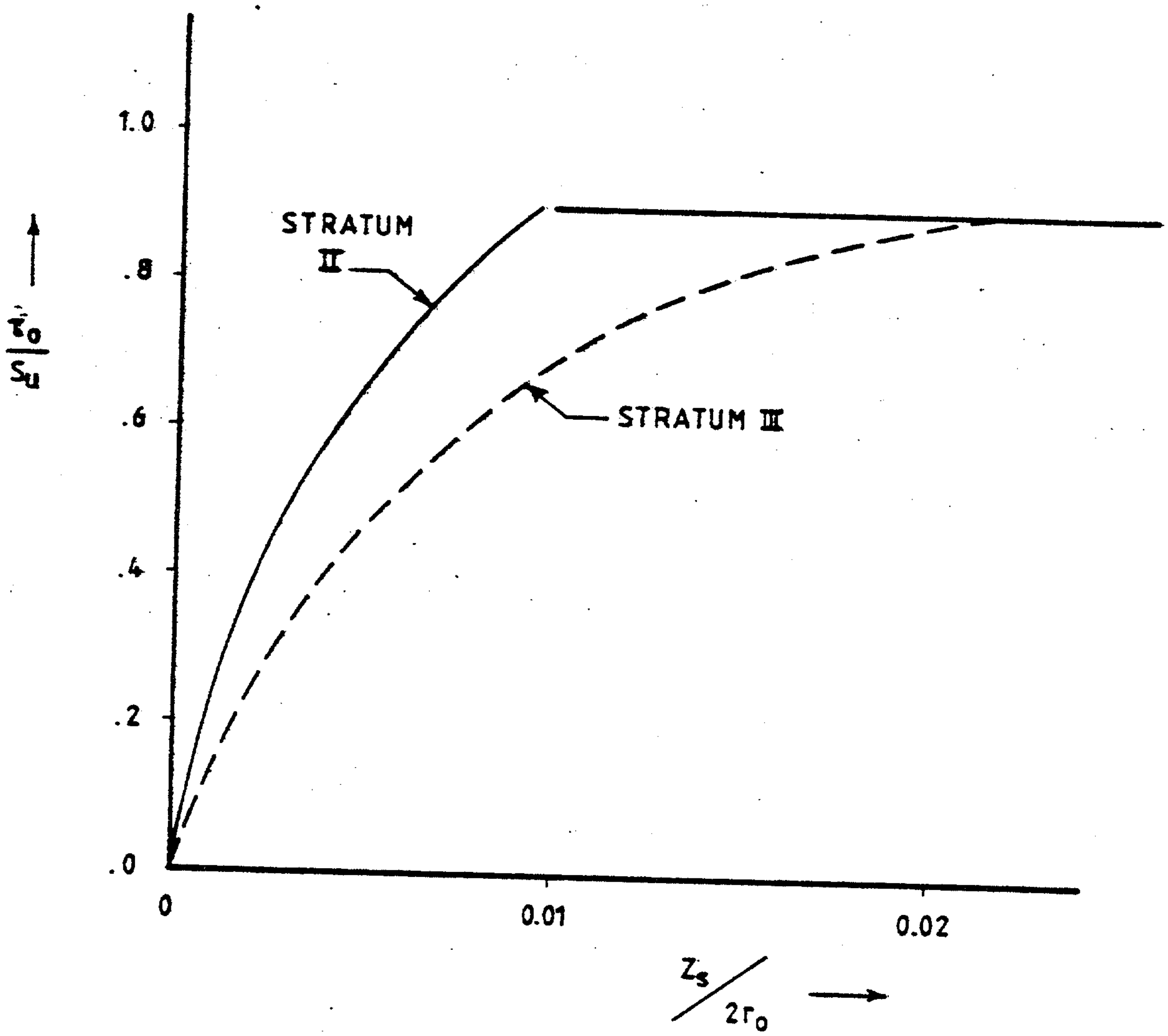
CONOCO TENSION PILE STUDY	Date 26.08.82	Drawn by H.Ö.
ASSUMED RANGE OF EFFECTIVE STRESSES WITH DEPTH. DATA FROM REF. 1	Approved	
	Project no. 81222-3	
Norwegian Geotechnical Institute	Drawing no. 41	



CONOCO TENSION PILE STUDY	Date 26. 08. 82 · Drawn by H. Ö.
ESTIMATED REMOULDED UNDRAINED SHEAR STRENGTHS AND PREDICTED SKIN FRICTION DURING PILE SEGMENT INSERTION	Approved
	Project no. 81222-3
Norwegian Geotechnical Institute	Drawing no. 42



CONOCO TENSION PILE STUDY	Date 26.08.82	Drawn by H.Ö.
ESTIMATED PORE PRESSURE DISSIPATION AT THE PILE SURFACE	Approved	
	Project no. 81222 - 3	
Norwegian Geotechnical Institute	Drawing no. 43	



τ_o = shear stress at segment pile surface

S_u = 0,22 σ'_{vo} =assumed undrained shear strength

Z_s = segment pile displacement

r_o = outer radius of segment pile

CONOCO TENSION PILE STUDY	Date 26.08.82	Drawn by H.Ö.
NORMALIZED τ-Z CURVES PREDICTED FOR SEGMENT TESTS MONOTONIC STATIC LOADING CONDITIONS	Approved	
	Project no. 81222-7	
Norwegian Geotechnical Institute	Drawing no. 44	