# Strut Loads and Related Measurements on Contract 63a of the Oslo Subway

## Mesures des contraintes dans les étresillons de rideaux de palplanches (Contrat 63a, Métropolitain d'Oslo)

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**Summary**

The authors present some results of strut load measurement and related observations on an open cut in clay for the extension of the Oslo subway system. This excavation, 30 metres long, 11 metres wide and braced at three levels was excavated to a depth of 11,5 metres below ground surface. In order to maintain the desired factor of safety against a bottom heave failure, the final 5,7 metres were excavated under water.

This article gives relevant soil data, a description of the excavation and measurement programme, and a continuous record of measured strut loads. Settlements, deflections of the sheeting, and maximum bending moments calculated from slope measurements are given for a few important stages. Measured strut loads are compared to calculated values using the empirical rules of R. Peck and J. Brinch Hansen. The suitability of these rules is discussed.

**Sommaire**

L'article rend compte de certains résultats obtenus lors de la mesure des forces développées dans les étrésillons des rideaux de palplanches, et des flèches prises par ces dernières, dans un batardeau creusé dans de l'argile pour la construction d'un chemin de fer souterrain à Oslo.

Ce batardeau — mesurant 30 mètres de long, 11 mètres de large, et 11,5 mètres de profondeur au-dessous du niveau du sol — a été étrésillonné à trois hauteurs différentes. Pour obtenir une sécurité suffisante contre un soulèvement du fond du batardeau, les derniers 5,7 mètres ont été creusés sous l'eau.

L'article rend compte des caractéristiques du sol, ainsi que des détails de construction et des procédés selon lesquels le travail a été effectué. Les résultats des mesures sont donnés par des figures et des tableaux. Pour plusieurs profondeurs d'excavation les forces mesurées dans les étrésillons sont comparées aux forces correspondantes calculées selon les règles empiriques établies par R. Peck et J. Brinch-Hansen. La validité de ces règles empiriques est commentée dans la conclusion.

### Introduction

Construction of the eastward extension of the Oslo subway system has provided an opportunity for the Norwegian Geotechnical Institute to carry out a series of field measurements on strutted excavations in soft clay. Measurements at one of the tunnel sections in a deep clay deposit, Contract 63 (a), are described in this paper. The location of this contract together with a plan and section of the excavation are shown on Fig. 1.

The authors confine themselves mainly to a presentation of the measured strut loads and some of the deflection and settlement observations for the most important construction stages.

### Soil conditions

The considerable number of borings carried out in this area clearly indicate reasonably uniform soil conditions along the tunnal route. On Fig. 1 (c) shear strength values, obtained from vane borings and unconfined compression tests on undisturbed samples taken with a 54 mm piston sampler, are plotted for five borings nearest the excavation. Variations in moisture content, liquid and plastic limits and soil classification for one of the borings are also shown. These normally consolidated, marine clay deposits have an average sensitivity value of 4. The curve drawn through the plotted shear strength values, Fig. 1 (c), represents the values used for earth pressure calculations.

### Cofferdam details and construction method

On the basis of stability calculations including a factor of safety of 1·3 against a bottom heave failure it was found that the excavation should not be continued below elevation — 3·0 unless additional measures were taken to ensure stability of the cutting. This difficulty was overcome by filling the excavation with water after reaching elevation — 3·0 and excavating to final depth under water; the weight of water being adequate to hold the bottom down and to ensure the desired factor of safety. For this reason this excavation is known locally as "vanngröft" which means water trench.

The sheet piling (section modulus 2 050 cu. cm/m), 16 m in length, was driven after first excavating through the fill material to elevation + 0·65. When the piling was completed, the uppermost or *A* level bracing system was installed and excavation inside the cofferdam began using grab equipment. After excavating to elevation — 1·75 the *B* level struts and waling were positioned. The next excavation stage was to elevation — 3·00 with the grabs and then hand excavation of trenches for the installation of the *C* level struts and waling.

All waling joints were welded; space between the waling and sheeting was wedged up with steel plates and/or concreted. The struts were wedged up, by hand, with steel wedges; in no case was the strut load due to wedging greater than 1 or 2 tons.

Upon completion of this bracing system, the cofferdam

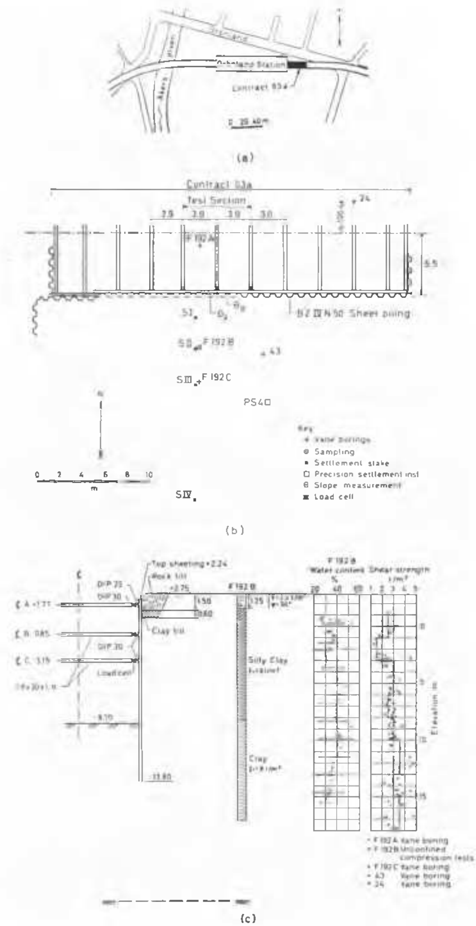



Fig. 1 Location, plan and section of Contract 63a.
Situation, plan et coupe relatifs au lot 63a.

was filled with water and excavation continued using grabs and divers working with jetting nozzles to clean along the sheeting and beneath the struts. The next excavation stage was to elevation — 7·00. Based on control measurements of the width of the excavation at this level, a decision was made that excavating could proceed to final depth without additional strutting. While excavating from — 3·0 to — 8·70 the water level was maintained at approximately elevation + 1·30. The method of constructing the tunnel (in progress at the time of writing) after the final excavation depth was reached has no bearing on the contents of this paper, so it will be omitted except to point out that at a later stage, after part of the tunnel is concreted, the water level in the excavation will be lowered about 5 metres which will create considerable net uplift pressures on the bottom of the tunnel. Part of the necessary anchorage will be provided by the sheet piling; it is for this reason that the sheeting extends so far below the bottom of the elevation.

**Measurement programme**

The measurement programme at "vanngröft" was quite elaborate and a complete discussion of the different observations is out of the question here, but nevertheless it may be of interest to mention the type of measurements undertaken.

All measurements were concentrated on the south side of the excavation. Equipment mounted on several of the piles before driving enabled the measurement of unit earth pressures and pore pressures on both sides of the sheeting to be done. Temperatures in the clay behind the sheeting were measured. Piezometers were installed at sixteen points in the clay mass. Strut loads were measured in three adjacent struts at each bracing level. Inclinometer or slope measurements were taken at two profiles in order to determine deflections of the sheeting.

A base line was used to measure horizontal movement of the top of the sheet pile wall. Settlements of the sheeting and of the top of the clay surface at various distances from the excavation were determined to within ± 1 mm by optical leveling. The special settlement stakes were cased along their full length to eliminate movements due to freezing and thawing. One of the settlement measuring points, *PS* 4, a precision settlement measuring unit, accurate to ± 0·01 mm, was installed in 1955. The locations of the measuring points are shown on Fig. 1 (b).

Specially designed, watertight, load cells, each containing three vibrating wire strain gauges were placed as shown on Fig. 1 to determine strut loads. These cells, calibrated in the laboratory before installation are considered reliable to ± 2 tons. A check on the zero drift of the three *A* level load cells made 160 days after installation showed a zero drift for each cell corresponding to a load of less than 1·0 ton.

The two installations for slope measurements are shown on Fig. 1 (b). An inclinometer fitted with a special set of wheels was lowered in a 6 cm × 6 cm tube that had been welded to the sheeting before driving. These measurements were made for the Institute by Consulting Engineer P. A. Madshus, using his inclinometer, essentially a pendulum and Wheatstone bridge arrangement in which the pendulum functions as a variable contact on a resistance spool. Slope angles to the nearest 0·0003 radian can be determined with this device. Slope readings were taken before and after each important excavation stage; usually several runs or sets of measurements were taken and the number of instrument readings at each point normally ranged from 4 to 6 for each run. Averaged values were used for deflection calculations. The determination of the instrument's zero point or reading when exactly in a vertical position was done with special care before and after each run. Each set of observations started at the same point and readings were taken every 0·25 m along the length of the piling. Horizontal movement of the starting point was measured from a fixed base time.

The slope measurements were used to compute the deflection of the sheeting and to obtain an estimate of maximum bending moment in the piling. An example illustrating how the bending movements were determined is shown on Fig. 2. On Fig. 2 (a) are plotted the slope values for three different runs; each point represents the average of 4 to 6 readings. The points plotted on Fig. 2 (b) are the initial slope readings taken after the piles were driven and before excavation started. These initial values clearly indicate local slope variations because of irregularities in the guide tube, warping due to welding etc. If curve (*a*) is corrected for these initial variations, the curve Fig. 2 (c) is obtained which presents no severe difficulty in drawing a smooth slope

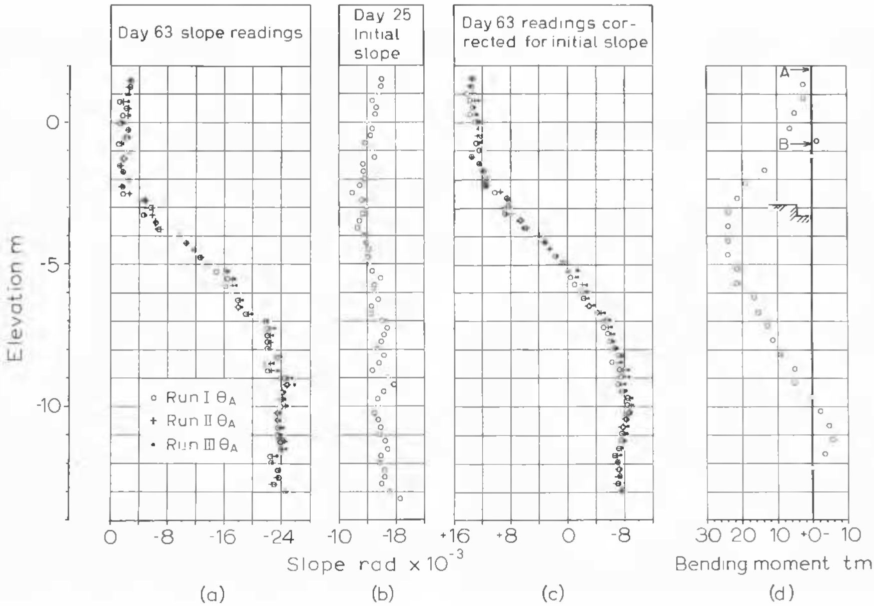



Fig. 2 Example of slope curves and corresponding bending moments.
Exemples de courbes d'inclinaison, et de moments de flexion correspondants.

curve through the points, in particular those points below the lowest strut. This smooth curve, when differentiated yields the bending moments shown on Fig. 2 (d).

**Observations during the major excavation stages**

The progress chart on Fig. 3 (a) shows the various excavation stages. For convenience in discussing the results, instead of using dates to denote the time of a particular event, the term "day number", which refers to the number of days that have elapsed after the instrument sheet piles were driven, is used. Day number "zero" is January 11, 1960.

The time-settlement curve for *PS* 4 is shown on Fig. 3 (b). The curves of average strut load per metre run are plotted against time on Fig. 3 (c). A few of the settlement observations are summarized in Table 1 and on Fig. 4 an attempt is made to compare the amount of settlement to the amount of deflection of the sheeting.

Table 1

| Day No. | Settlement cm | | | | | |
|---|---|---|---|---|---|---|
| | Sheeting | S I | S II | S III | PS 4 | S IV |
| 39 | 0.5 | 4.0 | 3.7 | 2.1 | 1.25 | 0.1 |
| 63 | 0.9 | 8.4 | 9.1 | 6.2 | 3.59 | 0.5 |
| 84 | 1.4 | 10.1 | 10.5 | 7.7 | 4.57 | 0.8 |
| 109 | 1.6 | 10.8 | 11.2 | 8.6 | 4.88 | 0.8 |
| 137 | 2.1 | 11.9 | 12.6 | 10.0 | 5.92 | 1.2 |

*Excavation from elevation* + 0·65 *to* — 1·75 :

The sheet piles were driven during January, 1960, with the bottom of the excavation then at + 0·65. On day 19, the *A* level bracing was placed but not blocked up until day 26. During the interim, the waling was not bearing against each and every section of sheeting, but was only in occasional contact. On day 26, steel plates and concrete were inserted to take up the remaining free space between the sheet piles and walings. Excavation in the test section started on day 28 and finished on day 29. On Fig. 3 (c) it is seen that the *A* level struts had increased to ≈ 4 tons/m before excavation started and deflection measurements showed 2 mm northward displacement of the south sheet pile wall (these horizontal movements were measured at points located 13 cm above the centre of the *A* struts). Both events were presumably associated with freezing around the top of the excavation. As a result of excavating to — 1·75 the *A* strut loads increased, the ground surface began to settle and northward movement of the south wall was observed. On Fig. 5 (a) for day 40, the calculated classical net earth pressure is shown as well as bending moments and deflection curves derived from slope measurements, horizontal displacement of the top of sheeting and measured strut loads.

*Excavation from elevation* — 1·75 to — 3·00 :

The *B* level struts were placed on day 40 and wedged up on day 42. Excavation started on day 53 and completed on day 59. Immediately the *B* strut loads increased and the *A* strut loads began to decrease indicating rotation of the sheeting about the *B* level struts; however, at the same time, the top of the south wall moved northward. On Fig. 5 (b) the calculated pressure, strut loads, moments and deflections are shown for day 64. Two sets of deflection curves

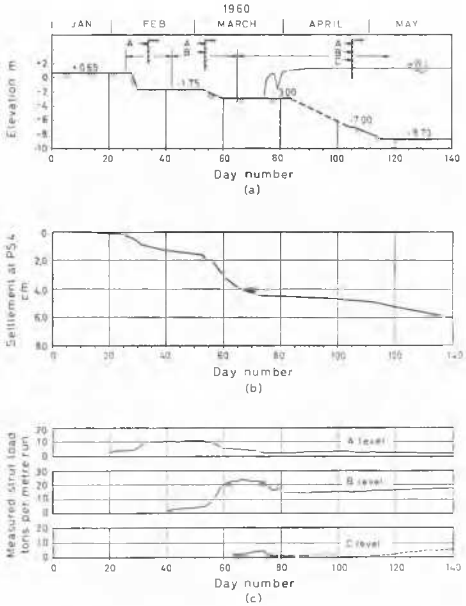



Fig. 3 (a) Progress chart ; (b) Time-settlement curve for $PS4$ ; (c) Measured average strut loads.

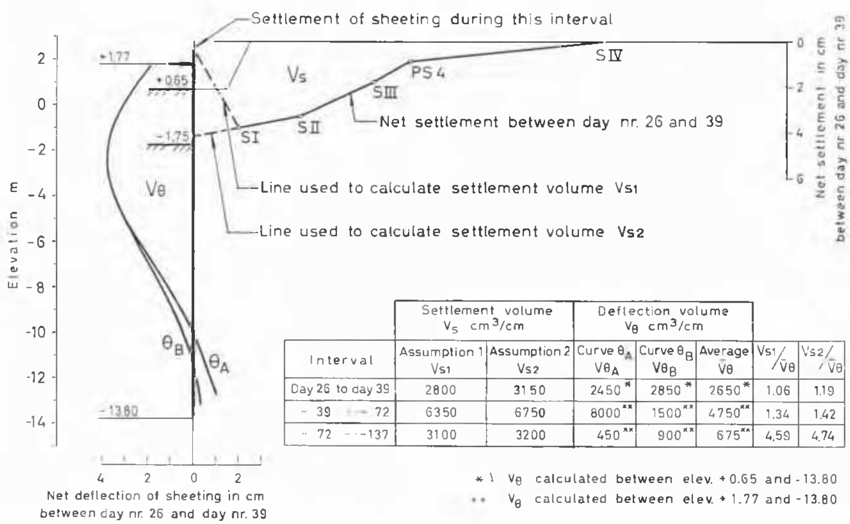
(a) Diagramme de l'avancement des travaux ; (b) Courbe temps-tassement pour $PS4$ ; (c) Forces moyennes mesurées dans les étrésillons.

are given, one showing the total deflection since the start of excavation and the other the increment in deflection since the last set of slope measurements. It is apparent that the deflections of the sheeting at locations $\theta_A$ and $\theta_B$ were different during this stage. The bottom of the $\theta_A$ pile moved northward, however, the bottom of $\theta_B$ moved in the opposite direction. In order to examine this event more thoroughly it is necessary to look at the relative position of the two piles after driving, as measured on day 25 and 26, shown on Fig. 6. It is seen that although both piles were relatively straight, they did not lie in the same plane indicating that the sheet pile was warped after driving and held in this position by the surrounding soil mass. (Both of these piles had wedged shaped shoes welded below the bottom of the inclinometer guide tubes. The position and form of these was such that a deflection force could have developed during driving that would have acted in the direction necessary to deflect each pile as shown on Fig. 6). Removal of the clay in the excavation would eventually allow the sheeting to spring back into a more favourable position. On Fig. 6 the deflected position of the piles are also shown for day 64 and it is seen that after this springback the positions of the two piles more nearly resembled one another than before.

At the time of writing, there was some uncertainty concerning the zero point inclinometer reading for day 64 corresponding to a possible error in deflection at the bottom of the piles of approximately $\pm$ 6 mm.

The $C$ struts were wedged up on day 65 and on day 72 an extra set of slope measurements were taken ; the results are shown on Fig. 5 (c).

One factor presumably having considerable influence on the measured strut loads and northward displacement of the top of the cofferdam is freezing of the clay behind the sheeting, particularly during the period of low temperatures before the beginning of the excavation to elevation — 3.0. The north wall because of its exposure to solar radiation




| Interval | Settlement volume $V_s$ cm³/cm: Assumption 1 $V_{s1}$ | Settlement volume $V_s$ cm³/cm: Assumption 2 $V_{s2}$ | Deflection volume $V_\theta$ cm³/cm: Curve $\theta_A$ $V\theta_A$ | Deflection volume $V_\theta$ cm³/cm: Curve $\theta_B$ $V\theta_B$ | Deflection volume $V_\theta$ cm³/cm: Average $\bar{V}\theta$ | $V_{s1}/\bar{V}\theta$ | $V_{s2}/\bar{V}\theta$ |
|---|---|---|---|---|---|---|---|
| Day 26 to day 39 | 2800 | 3150 | 2450 * | 2850 * | 2650 * | 1.06 | 1.19 |
| " 39 — 72 | 6350 | 6750 | 8000 ** | 1500 ** | 4750 ** | 1.34 | 1.42 |
| " 72 — 137 | 3100 | 3200 | 450 ** | 900 ** | 675 ** | 4.59 | 4.74 |

* ) $V_\theta$ calculated between elev. + 0.65 and - 13.80

** $V_\theta$ calculated between elev. + 1.77 and - 13.80

Fig. 4 Comparison between ground surface settlements and deflections of the sheeting.
Comparaison entre le tassement du terrain et la flexion du rideau de palplanches.

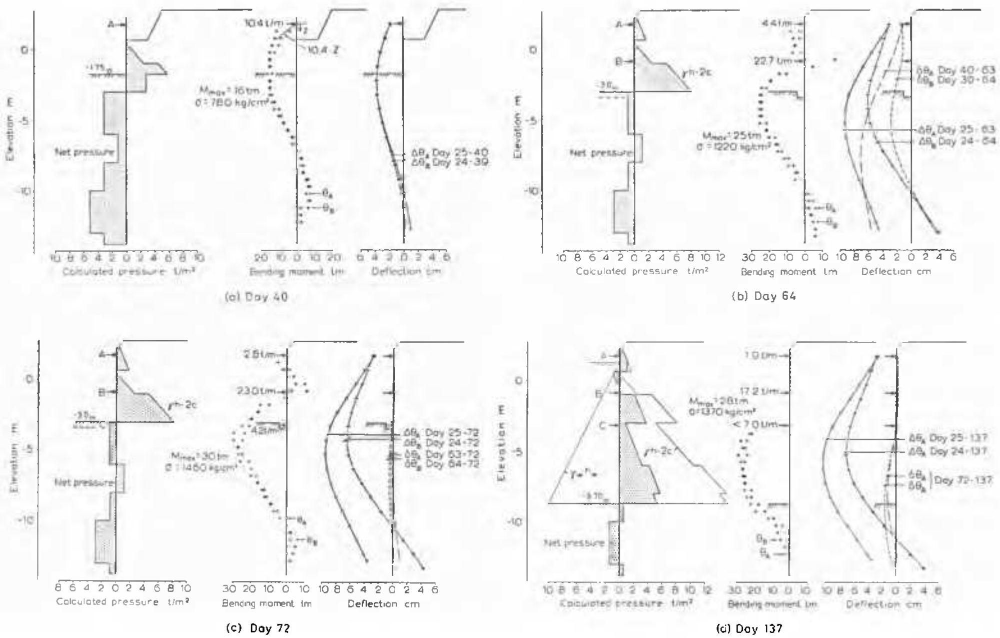



Fig. 5 Calculated pressure, measured strut loads, deflections and bending moments.
Pressions calculées, forces mesurées dans les étrésillons, flèches, et moments de flexion.

during daylight hours would have been warmer than the south wall which at this time of year is completely unexposed to the sun. Thereby temperature differences would have existed in the clay behind the sheeting on both sides of the excavation enabling the clay to freeze sooner and to a greater degree behind the south wall than behind the north wall. This would have resulted in a force tending to push the top of the sheeting in a northerly direction. In order to check this possibility, holes were drilled through the sheeting at two elevations on both sides of the excavation and temperature measurements in the clay were taken at 5 cm intervals with a thermocouple. Results of one series of temperature measurements are shown on Fig. 7.

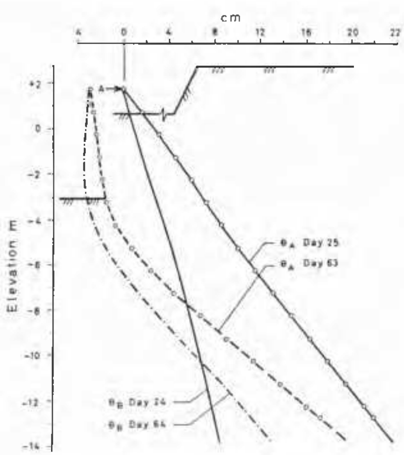



Fig. 6 Initial inclination of the two measurement piles and their position after excavating to elevation — 3·0.
Inclinaison initiale des deux palplanches de mesure, et leur position après excavation jusqu'au niveau — 3,0.

Before filling the excavation with water, the thermocouple was inserted in the upper hole at the south wall where the last remaining zone of temperatures below freezing point existed. Subsequent observations indicated a complete thaw on day 83 and from then on the excavation can be considered completely free from frost.

*Final excavation to elevation* — 8·70 :

The excavation was filled with water on day 74. During days 84-89 some excavating was done in the test section and between days 101 to 105 there was a concentrated effort to excavate to elevation — 7·0 within the bounds of the test section. Excavation began once more on day 106 and by day 116 the final elevation of — 8·70 was reached. On day 137 slope measurements were taken and the strut loads, calculated moments, deflection and classical earth pressure are shown on Fig. 5 (d).

The *C* strut and load cell on the west side of the test section was accidentally pulled up by the grab on day 86 by which time the *C* strut loads were very small. The strut was immediately replaced and wedged up by the divers. On day 115 the load cell on the *C* strut on the east side became inoperative and remained so. Later the middle strut load gradually increased over a considerable time interval before additional load was observed in the west strut indicating

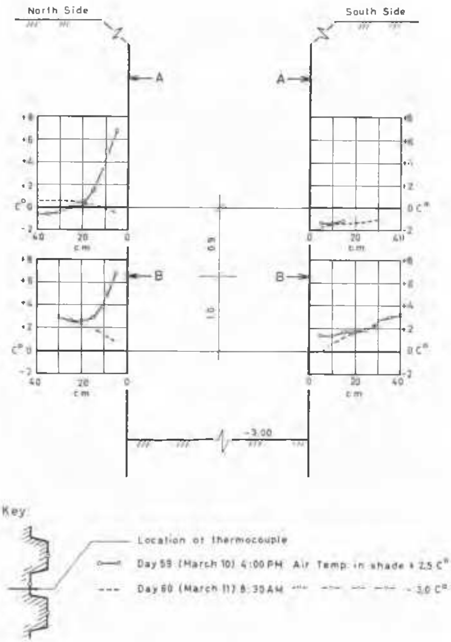



Fig. 7 Temperature measurements in the clay mass behind the sheeting.
Températures mesurées dans l'argile derrière le rideau de palplanches.

that the west strut had not been satisfactorily wedged up (confirmed by reports from the divers). Obviously the middle strut was carrying part of the load which should have gone to the west strut. The middle *C* strut loading on day 136 was 7·0 tons/m; therefore the average strut load must have been less than 7 tons per metre, as given by Fig. 5 (d).

**Discussion of results**

The classical active earth pressure was calculated for each depth of excavation. Strut loads were computed according to Peck's empirical rules (Ralph B. Peck, 1943) and the method proposed by Brinch Hansen for braced cuttings (J. Brinch Hansen, 1953). The values of shear strength and density used are shown on Fig. 1 (c). A summary of these calculations and a comparison with the measured strut loads are given in Table 2. No calculations were performed for day 40 since the presence of the open trench behind a limited length of the sheeting at this stage is relatively large in comparison to the deep excavation, and therefore the computations are influenced by questionable assumptions.

When applying empirical rules, the material above the top of the sheeting (+ 2·24) was considered as a surcharge. For days 84, 106 and 137 with water in the excavation, the pressure diagram obtained by subtracting the water pressure from Peck's or Brinch Hansen's pressure diagram was distributed over the struts in the usual manner. Calculations using Brinch Hansen's method include the recommended factor of safety (1·4) which is applied to the undrained shear strength.

A comparison of the calculated and measured strut loads considering each excavation stage as a separate cutting without consideration of strut loads developed before this level was reached is shown in columns 1 to 4, table 2. The *B* strut loading on days 64 and 84 is severly under estimated by Peck's rule. For the last two excavation stages this rule gives safe results, but overestimating the *C* strut. Other field measurements have also shown that this rule is followed only for certain excavation depths and for smaller depths it gives unsafe design values (Tschebotarioff, 1951). Brinch Hansen's method, column 4, gives conservative results for days 64 and 84; with increasing depth, i.e., for the last two stages, it underestimates the *B* strut and overestimates *A* and *C*.

The information given in Table 2, columns 5 to 7, is of more interest with respect to the application of these empirical methods for design purposes. These columns summarize the maximum calculated and measured strut loads existing at or prior to each excavation stage. All the *A* strut loads calculated using Peck's rule for the various depths of excavation are smaller than the maximum measured value of

Table 2

| Day No. | Excavation stage | Strut | Strut load t/m | | | | Max strut load to date | | | Thrust above excavated level t/m | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Measured Ave. | Measured Max. | After R.Peck | After B.Hansen | Measured | After R.Peck | After B Hansen | Classical | $\frac{1}{2}\gamma h^2$ | $\frac{1}{2}\gamma_w h_w^2$ | ①+⑩ | ⑪/⑧ | ⑪/⑨ | $n_a$ |
| | | | ① | ② | ③ | ④ | ⑤ | ⑥ | ⑦ | ⑧ | ⑨ | ⑩ | ⑪ | ⑫ | ⑬ | ⑭ |
| 40 | | A | 10.4 | 11.3 | | | | | | | | | | | | |
| 64 | | A | 4.4 | 4.7 | 6.2 | 11.6 | 11.3 | 6.2 | 11.6 | 15.4 | 32.3 | 0 | 27.1 | 1.76 | 0.84 | 0.49 |
| | | B | 22.7 | 25.8 | 13.4 | 33.0 | 25.8 | 13.4 | 33.0 | | | | | | | |
| | | Σ | 27.1 | | | | | | | | | | | | | |
| 84 | | A | 1.9 | 2.1 | 5.9 | 11.5 | 11.3 | 6.2 | 11.6 | 15.4 | 32.3 | 7.7 | 25.2 | 1.64 | 0.78 | 0.40 |
| | | B | 14.5 | 16.9 | 9.5 | 17.4 | 25.8 | 13.4 | 33.0 | | | | | | | |
| | | C | 1.1 | 2.1 | 0.8 | 8.0 | 1.1 | 0.8 | 8.0 | | | | | | | |
| | | Σ | 17.5 | | | | | | | | | | | | | |
| 106 | | A | 2.4 | 2.8 | 6.7 | 12.5 | 11.3 | 6.7 | 12.5 | 50.2 | 90.0 | 35.7 | 53.9 | 1.07 | 0.60 | 0.43 |
| | | B | 14.9 | 16.9 | 18.7 | 13.9 | 25.8 | 18.7 | 33.0 | | | | | | | |
| | | C | 0.9 | 1.7 | 16.5 | 13.7 | 1.1 | 16.5 | 13.7 | | | | | | | |
| | | Σ | 18.2 | | | | | | | | | | | | | |
| 137 | | A | 1.9 | 2.3 | 7.1 | 11.1 | 11.3 | 7.1 | 12.5 | 74.2 | 123.5 | 52.9 | ~79.0 | ~1.07 | ~0.64 | ~0.43 |
| | | B | 17.2 | 19.3 | 22.0 | 13.4 | 25.8 | 22.0 | 33.0 | | | | | | | |
| | | C | <7.0 | 7.0 | 29.0 | 20.9 | 7.0 | 29.3 | 20.9 | | | | | | | |
| | | Σ | <26.1 | | | | | | | | | | | | | |

11·3 tons/m. This is not true for Brinch Hansen's method. It should be stressed, however, that this measured value probably includes unknown freezing effects. Likewise, Peck's rule never yields a *B* strut load equal to the maximum measured value; however, Brinch Hansen's method does. Both of these methods give uneconomical design values for the *C* strut.

In column 12, table 2, the ratio of the strut loads and water thrust to the classical active thrust is about 1·7 for days 64 and 84. This ratio decreases to approximately 1·1 for the deeper excavation stages, indicating a state more nearly resembling the classical active state. Column 13, table 2, shows earth pressure coefficients ranging from 0·84 to 0·60.

Fig. 4 shows that for the interval day 72 to 137 the settlement volume was nearly 5 times the deflection volume of the sheeting. No significant reductions in pore pressure were observed during this period as the water in the excavation prevented drainage of the clay. This implies that the bottom of the excavation must have been pressed upwards with simultaneous movements within the clay mass resulting possibly in the mobilization of larger shearing stresses corresponding to the change in ratios given in columns 12 and 13, table 2.

The centre of pressure $n_a$, column 14, determined from the measured strut loads and water pressure varies between 0·40 and 0·49 which compares favourably with those observed elsewhere (Peck, 1943).

The location of the maximum bending moments can be seen on Fig. 5; the maximum observed bending moment corresponds to a maximum steel stress of 1 460 kg per sq. cm.

## Conclusions

The following conclusions can be stated on the basis of the measurements at this site.

These observations verify earlier findings that R. Peck's empirical rule for calculating strut loads underestimates the loads for small excavation depths. When applied to the final depth of excavation, which in this case corresponds approximately to the critical depth $4c/\gamma$, this rule gives reasonable strut loads, although slightly underestimating the maximum observed loads in the upper two struts and overestimating the lower one.

J. Brinch Hansen's method of calculating strut loads gives safe design values for all the excavation stages providing the strut loads are calculated for each excavation level. This method also gives an uneconomical value for the lower strut.

Temperature is a factor to be considered in the design of strutted excavations.

Ground surface settlements adjacent to the excavation are a result of deflections of the sheeting and heave of the bottom of the excavation and not due to a lowering of the ground-water table.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