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Earth Pressure Measurements in a Trench Excavated in Stiff Marine Clay

Mesure des Poussées du sol dans une Tranchée Creusée dans une Argile Marine Rigide

by E. DiBiagio and L. Bjerrum, Norwegian Geotechnical Institute, Norway

Summary

This paper describes earth pressure measurements made in a braced experimental trench excavated in the stiff fissured crust of the Norwegian marine clay. The trench was approximately 14 m long, 0·9 m wide and 4 m deep. Each of the three measuring sections within the trench was 2 m wide and had 2 vertical rows of adjustable steel struts fitted with vibrating-wire type load gauges. Horizontal and vertical deformations of the soil mass were recorded. In addition, pore pressure piezometers were installed to measure changes in pore water pressures.

This article presents a description of the excavation, a summary of the soil properties, and a continuous record of measured strut loads between September 1955 and June 1956, with a comparison to calculated values.

The computed maximum total earth pressure, based on an effective stress analysis with the assumption that $c' = 0$, is in good agreement with the measured value. Peck's rule for distributing the total earth pressure appears to be suitable for application to this excavation.

Sommaire

La présente étude décrit les mesures de la poussée des terres dans une tranchée expérimentale étayée, qui fut creusée dans la croûte raide fissurée de l'argile marine norvégienne. La tranchée avait environ 14 m de long, 0·9 m de large et une profondeur de 4 m. Chacune des trois sections de mesure de la tranchée avait une largeur de 2 m et fut munie de 2 rangs verticaux d'étais d'acier ajustables pourvus d'appareils de mesure à corde vibrante. On a mesuré les déformations horizontales et verticales de la masse des terres. De plus, on fit installer des piézomètres permettant d'observer les modifications de la pression de l'eau interstitielle.

L'étude donne une description de l'excavation, un aperçu des propriétés du sol et la série continue des poussées mesurées sur les étais entre septembre 1955 et juin 1956 ainsi qu'une comparaison avec les valeurs calculées.

La poussée totale maximum des terres calculée en se basant sur une analyse des contraintes effectives en supposant $c' = 0$, concorde bien avec la valeur mesurée. La règle de Peck pour la répartition de la poussée totale des terres semble convenir pour application à cette excavation.

Introduction

The stability of trenches excavated in stiff but heavily fissured clays and the shear strength characteristics of the clay are still somewhat uncertain. Since the usual soil profile in Norway consists of from 2 to 6 m of dried or weathered crust, the Norwegian Geotechnical Institute, in an attempt to get a better knowledge of the stability of trenches excavated in fissured clay, began in September 1955 to carry out field measurements of the forces developed in the bracing of an experimental trench.

A detailed description of the trench and the results will be given in a later publication (E. DiBiagio: *Measurements of*

Strut Loads in an Experimental Trench in Stiff Fissured Clay— Internal Report, Norwegian Geotechnical Institute).

General Description of Excavation

A plan and section of the excavation are shown in Fig. 1. The main test section, in which the measurements were taken, was 14 m long, 0·9 m wide, and 4 m deep. One end of the trench slopes upwards to the surface, the other opened into a larger excavation, 5 m deep, composed of one vertical side and four others, each having different slopes. This section served
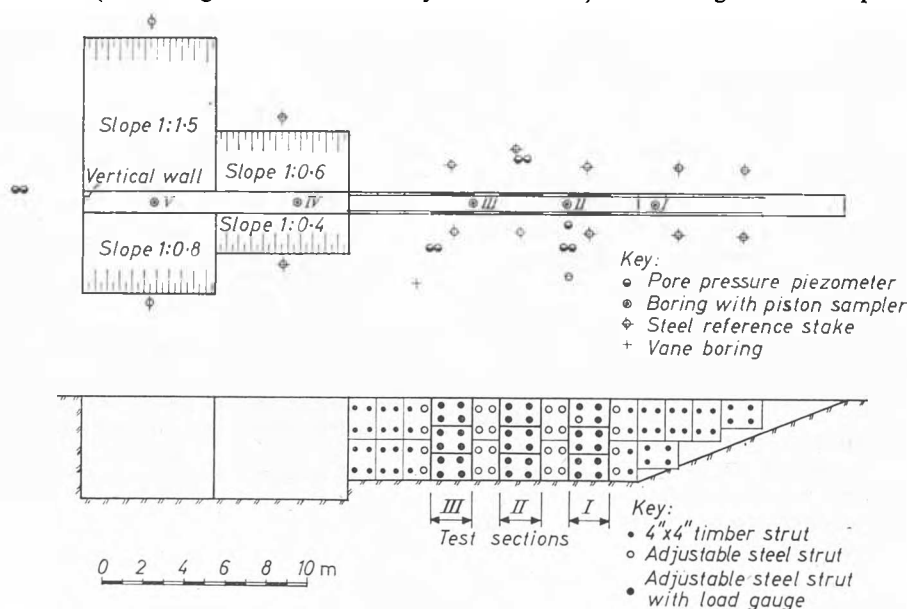

Fig. 1 Plan and section of excavation
Plan et coupe de l'excavation

both as a drainage sump and a visual comparison of how resistant the different slopes were to weathering processes.

Special steel reference stakes were placed in and around the excavation to record horizontal and vertical deformations of the soil mass. Thirteen pore pressure piezometers were installed, three of which were placed 17 m from the excavation to serve as a control.

The bracing scheme and location of the three test sections within the trench are shown in Fig. 1. Adjustable, screw-type steel struts were used in both the test and adjoining sections; hardwood wedges and 4 by 4 in. timber struts were used throughout the remainder of the trench. The sheeting consisted
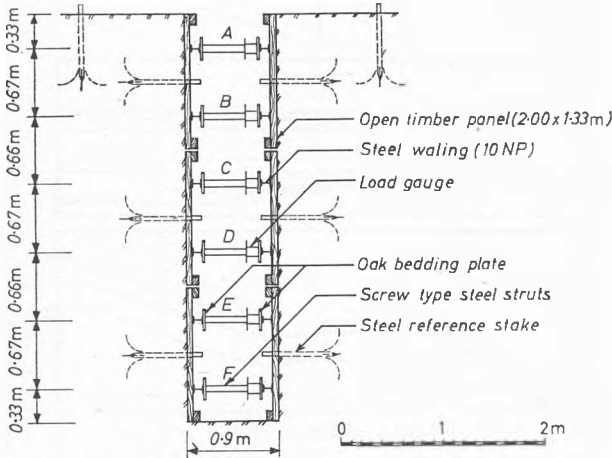
Fig. 2 Cross-section of trench
Coupe transversale de la tranchée

of prefabricated open panels, 2·00 by 1·33 m, made of 2 by 6 in. planks with an 11 cm gap between each plank. The panels were supported at the quarter points by walings (10 NP steel beams) 2·00 m long. Each wale, in turn, was supported at its quarter point by a strut. Each test section had two vertical rows of struts with six struts per row; since each section is 2 m wide, strut loads in tons are equal to the strut loads per meter run of trench. Because the sheeting in the test sections consists of three separate panels, one above the other, there is no possibility of the shear stresses transferring from one panel to another. Fig. 2 shows the bracing system and position of reference stakes driven into the clay through the slots in the panels of the test sections.

Each measuring strut was equipped with a cylindrical vibrating-wire type load gauge similar in construction to those described by COOLING and WARD (1953). However, because the load gauges were to be subjected to large fluctuations in temperature, a slight modification was made in the design to assure that the gauges would not be unreasonably sensitive to temperature. This was done by selecting materials having temperature coefficients of expansion approximately equal to the computed values needed to produce identical temperature expansions in the gauge cylinder and tensioned wire. Also the unit was constructed in such a way that shim washers could be used to vary the free length of the wire in relation to the gauge length of the cylinder. In this manner it was possible to adjust the thickness of washers in each gauge until a temperature variation less than $\pm$ 2 kg/° C. was attained. The gauges were designed for a maximum load of 10 ton. A Maihak instrument was used to take the measurements; readings, ordinarily, could be repeated corresponding to approximately $\pm$ 50 kg.

The load gauges have proved to operate quite satisfactorily under the adverse conditions of weather and temperature. During the interval from September to June, only one gauge ceased to operate properly.

Soil Sampling and Testing

Before the excavation was started, undisturbed samples were taken, with the Norwegian Geotechnical Institute 54 mm piston sampler, in five bore holes which extended to a depth of approximately 6·5 m. A total of 35 samples using 80 cm sampling tubes were taken. The location of these borings is shown in Fig. 1. In the laboratory the usual classification tests were carried out and the unconfined compression strengths were determined from samples trimmed to a cross-section of 3·6 cm by 3·6 cm and 10 cm in height. A summary of the results from a typical bore hole is given in Fig. 3.

As the trench was being excavated, chunk samples, 10 cm in diameter and 30 cm long, were taken to be used for conducting triaxial tests. Three samples were taken from a depth of 1·5 m, three from a depth of 2·5 m, and 3 from a depth of 3·5 m. Standard consolidated drained triaxial tests and consolidated undrained tests with pore pressure measurements were run to determine the shear strength parameters c and ϕ in terms of effective stresses. The samples were brought to failure by increasing σ_1; controlled strain loading was used, the rate being approximately 0·08 per cent/hour for the drained tests and 3 per cent/hour for the undrained tests. A summary of these tests is presented in Table 1.
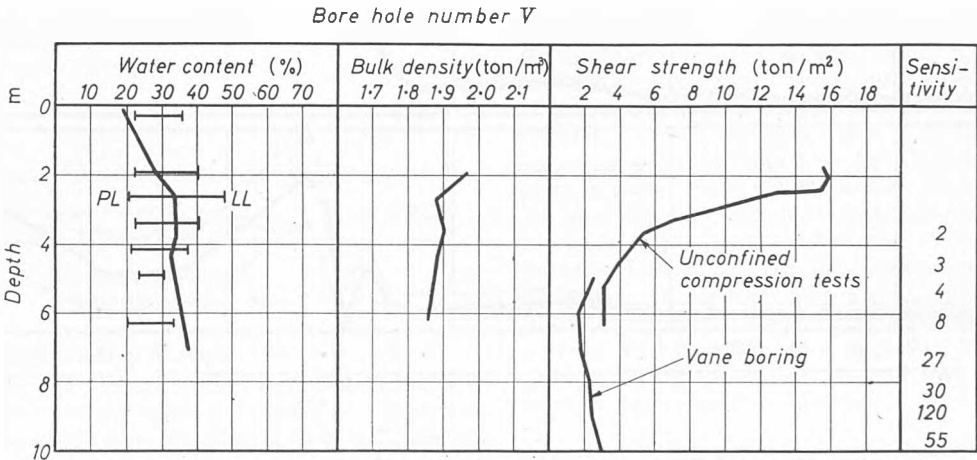
Fig. 3 Summary of tests from a typical bore hole
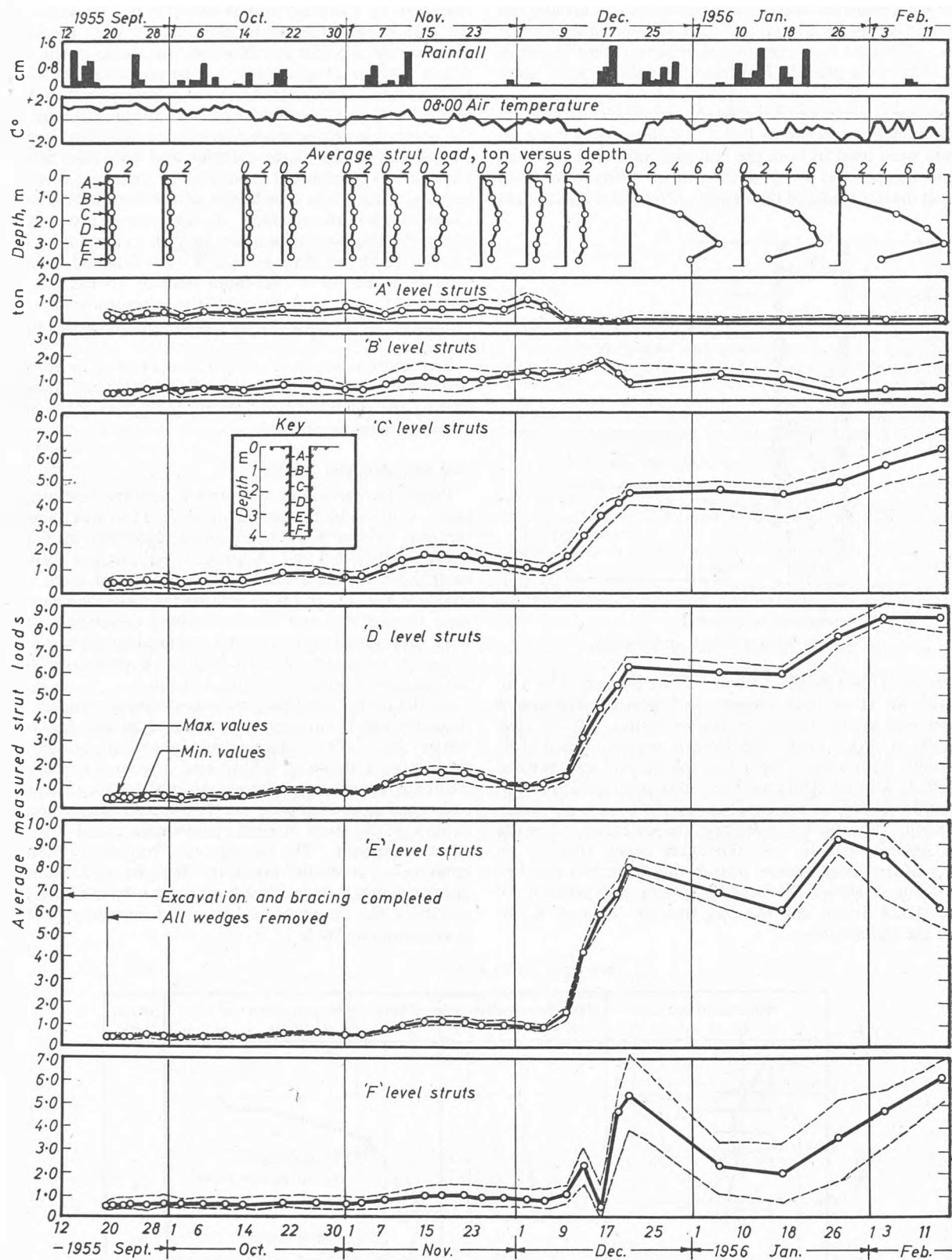Résumé des essais d'un trou de sondage typique

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Fig. 4   Measured strut loads
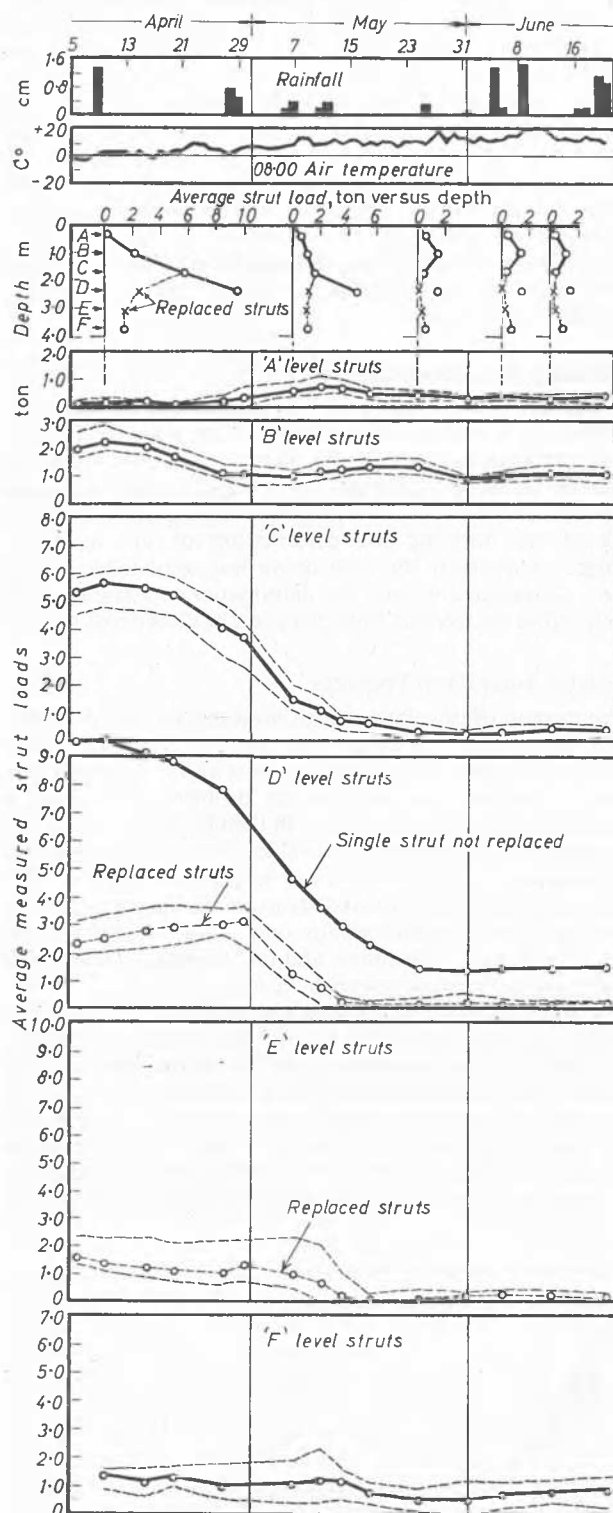Efforts mesurés sur étais

Fig. 5 Measured strut loads
Efforts mesurés sur étais

Table 1
Summary of triaxial tests
Rélevé des essais triaxiaux

Depth m	Consolidated drained		Consolidated undrained		Average	
	c' ton/m^2	ϕ' degrees	c' ton/m^2	ϕ' degrees	c' ton/m^2	ϕ' degrees
1·5	3·0	34	3·0	38	3·0	36
2·5	2·4	30	3·8	30	3·1	30
3·5	0·8	27	0·8	29	0·8	28

by driving hardwood wedges between them. The test sections were braced similarly and not until the entire trench was completed were the steel struts and load gauges installed. Whenever the load gauges had been placed, the steel strut was tightened to a load of approximately 200 to 300 kg and immediately afterwards all the wedges and temporary struts were removed from that section. The trench excavation was completed on 15 September; the installation of the load gauges and removal of the temporary struts in the test sections were completed on the following dates:

Test section III   September 12
Test section I   September 16
Test section II   September 20

During the excavating in sections I and II two small local failures occurred in the trench wall. Both of these took place at night or early morning in the relatively small unsupported section which was left unbraced so that the excavating machine could continue to operate. In both cases a chunk about 2 m high, 2·30 m wide and a maximum thickness of 0·40 m slid into the trench. The cavities created in the wall were very carefully filled with clay and cobblestones and compacted as much as possible. It is believed that this refilling was quite satisfactory.

Aluminium covers were placed over the load gauges to protect them from direct rays of the sun. Later, as winter was approaching, corrugated asbestos sheets were placed over the trench and its ends sealed off in order to prevent it filling with snow.

Measured Strut Loads (Autumn)

Average measured strut loads with maximum and minimum limits are plotted against time with corresponding daily temperature and precipitation in Figs. 4 and 5. In general, the strut loads increased gradually after the first measurements were taken. After each period of rainfall a sharp increase in the total earth pressure resulted, see Fig. 6, then the total earth pressure remained nearly constant for a short interval. Thereafter a characteristic decrease occurred until another interval of rainfall caused the whole cycle to be repeated again. The heaviest interval of rainfall, in the relatively dry autumn, took place between 5 and 12 November during which a total of 3·0 cm of rain were recorded. Following this, on 15 November, the loads in the struts reached their maximum values. Pore pressures likewise reached their maximum values at this time when the total earth pressure was greatest. Fig. 7 shows the distribution of strut loads for different times throughout the autumn with corresponding curves of zero pore pressure. Once again the same cyclic changes took place and the total earth pressure began to decrease.

Measured Strut Loads (Winter and Spring)

As the temperature started to drop below freezing, the immediate effect on the magnitude and distribution of the strut

Excavation and Installation of Bracing

The piezometers and reference stakes were installed early in the summer of 1955, and the trench was started on 1 September of that year. The panels were placed as soon as sufficient wall area was exposed to allow the panel to be positioned without it interrupting the excavating machine. As soon as the panels were set, the walings and timber struts were placed and tightened
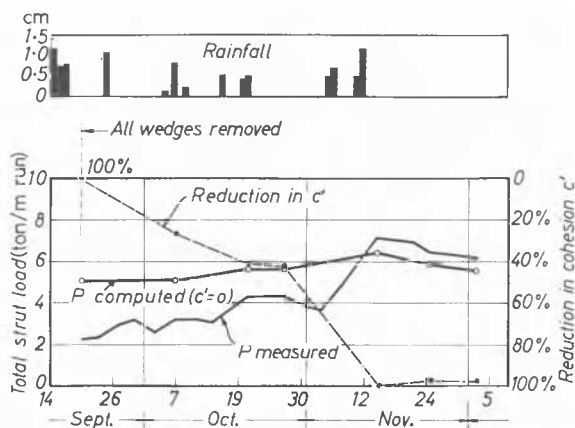
Fig. 6 Measured and calculated total earth pressure
Poussée totale des terres mesurée et calculée

of streaks in the clay that were formed as it slid by the rough edges of the planking on the panels. In addition, several horizontal cracks, about 80 cm long, 6 to 8 cm wide, and extending over 30 cm into the wall, were observed at depths between 2 and 3 m. These cracks indicate that the entire soil mass did not move downwards at the same rate and that the upper layer may have become partially wedged against the uppermost panel. This would offer a reasonable explanation for the unusual shape of the strut load *versus* depth diagram for June shown in Fig. 5.

As could be expected, the D and E level struts that were replaced had to be tightened during the thaw in order to maintain load in them.

Deformation Measurements

In Fig. 8 are shown the vertical and horizontal movements of the reference stakes at various times. Fig. 9 shows the horizontal deformations within the trench. On this figure the curves for December and April are obviously incorrect because the trench definitely became narrower during the winter as a result of the buckling and deformation of the struts and walings. Apparently the frost heave was responsible for this error. Consequently, only the deformations measured in the trench before the freezing took place can be considered as valid.

Calculated Total Earth Pressures

The method of calculating earth pressures by the $c\phi$-analysis (SEVALDSON, 1956) has never been used for calculating strut loads in excavations in clay but it seems in the present case to present a realistic approach to this problem. It should be mentioned that the pore pressures in the clay in the course of a few days changed from the initial values to values corresponding to a lowering of the ground water level by the trench. This means that, due to the fissured structure of the clay, the pure undrained conditions existed for only a very short time immediately after the excavation, and it is therefore more logical to calculate the earth pressures by a $c\phi$-analysis.

The application of the $c\phi$-analysis requires a knowledge of the shear strength parameters c' and ϕ' and of the pore pressures. The c' and ϕ' values were determined by triaxial tests and pore pressures were measured in the field at different times. As the line of zero pore water pressure is curved, the calculations were made graphically using the general slope stability equation developed by JANBU (1955). It is hereby assumed that the shear strength is fully mobilized along the sliding surface, which

loads was quite surprising. Freezing of the clay at the surface of the ground and within the trench resulted in a marked reduction in loads carried by the upper A and B level struts and an enormous increase in the forces transmitted to the lower struts. Fig. 7 shows the distribution of the maximum averaged strut loads recorded together with a curve denoting the limit of frost penetration.

As the soil continued to freeze the strut loads developed were sufficient to buckle some of the walings and struts. Finally, the damage became so severe that all measurements were stopped and five of the D level and all the F level struts and walings were removed after temporary timber braces were installed. The struts were straightened and the load gauges checked and recalibrated before being re-installed with new walings during the end of March.

The rather unexpected distribution of strut loads at the upper part of the trench during winter is thought to be a result of the upward expansion of the soil bordering the excavation. Unfortunately the bench mark used for levelling purposes was also subjected to an unknown displacement because of frost heave. Therefore the measured values cannot be considered to represent the total movement of the soil in the vicinity of the trench. However, visual observations through the open slots in the panels during the thaw gave conclusive evidence that the clay underwent a minimum downward movement of 10 cm at the mid-height of the trench. This was deduced from observations
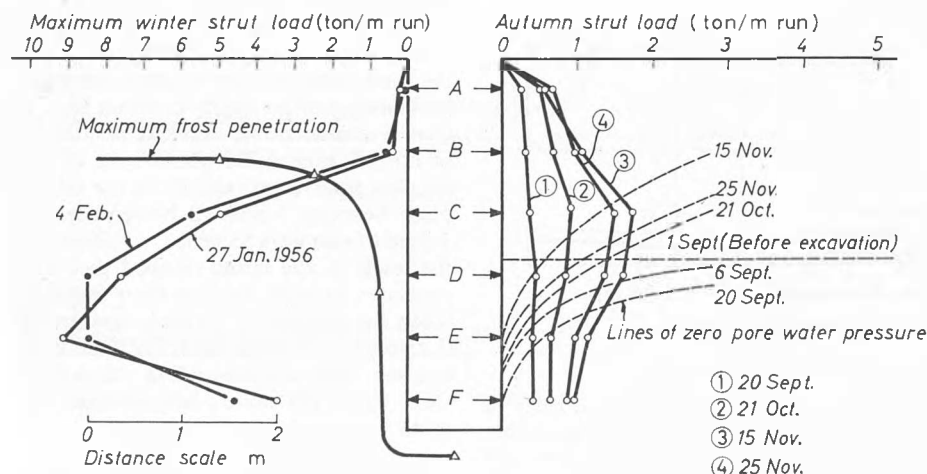

Fig. 7 Autumn and winter strut loads *versus* depth
Variations des efforts sur étais en automne et en hiver en fonction de la profondeur

moreover was assumed to be plane. The analysis is made by dividing the sliding wedge into vertical slices and the total earth pressure can then be calculated by the equation

$$P = \Sigma p \tan \alpha \varDelta x - \sum \frac{c' + (p - u) \tan \phi'}{\cos^2 \alpha (1 + \tan \alpha \tan \phi)} \varDelta x$$

where

P = horizontal force required to hold the failure wedge in equilibrium

$c'\phi'$ = shear strength parameters in terms of effective stresses

$p = \gamma h$, where h = average height of the slice

γ = bulk density

u = pore pressure

$\varDelta x$ = width of slice

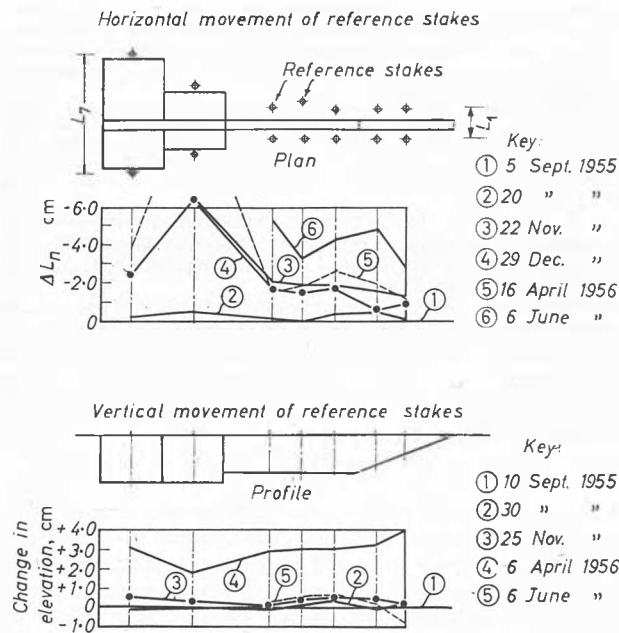α = angle the failure surface makes with the horizontal.

Fig. 8 Horizontal and vertical movements of the reference stakes
Mouvements horizontaux et verticaux des jalons de référence

The earth pressure was calculated by this method for various times using pore pressures corresponding to those measured in the field at the same date.

It can be mentioned at once that this method also resulted in much too small earth pressures compared to the measured values. Indeed, the $c\phi$-analysis indicated that the trench should be stable with no or very small strut loads. This means that the shear strength must be considerably smaller than the values introduced in the above formula.

From recent investigations of failures of cutting in stiff fissured clay (HENKEL and SKEMPTON, 1955; HENKEL, 1956) it is known that in this type of clay the cohesion intercept c' will decrease with time if the fissures are laterally unloaded. In the present case lateral movements were observed and it is therefore very probable that the over-estimate of the shear strength in the effective stress analysis is due to the same effect, i.e. a reduction of c' compared to the initial value measured by the triaxial tests.

Following this explanation the earth pressures were calculated at different times by a $c\phi$-analysis but now assuming that $c' = 0$. The results of the total earth pressures calculated in this way are shown in Fig. 6. It will be seen that the computed and measured value converge with time until, after the heavy rainfall in the beginning of November, they are within 10 per cent of being equal. Thus, there is reason to believe

that a decrease in c' took place until the value zero was reached on 15 November. This finding is strongly confirmed by the observation that after this time the changes in total earth pressure can be explained entirely by the variation in pore pressure.

In an attempt to evaluate the rate of change in cohesion, the value of c' required to obtain agreement between measured and calculated total earth pressure was calculated for different times. As the computed values with $c' = 0$ are constantly about 10 per cent lower than the values observed on 15 November and the following period, it is very likely that the value of ϕ' used in the calculation is not exactly correct. Therefore, for the computation of the rate of reduction in c' a minor adjustment was made until measured and computed values were equal at 15 November. From these calculations the cohesion required
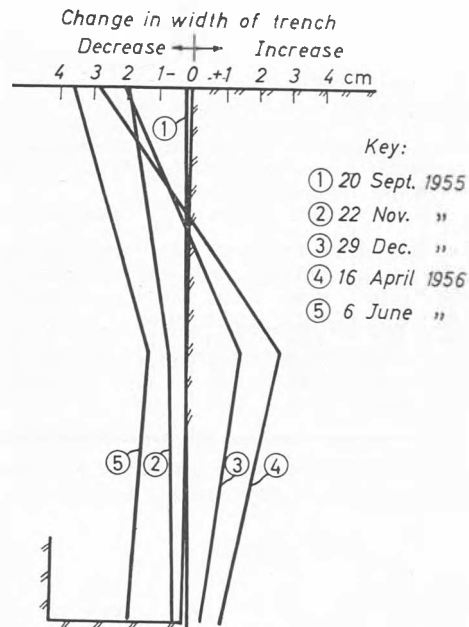

Fig. 9 Horizontal deformations in the trench
Déformations horizontales dans la tranchée

to equalize the measured and computed earth pressure on 20 September, when all the temporary struts were removed and measurements started, was found to be 0·87 ton/m², which is 28 per cent of the weighted c' derived from the triaxial tests. The required values of c' were also computed for the other observations and expressed as a percentage of the 0·87 ton/m². These values decrease continually until $c' = 0$ on 15 November. Fig. 6 shows the percentage reduction of the cohesion at the different times and for comparison the intervals of rainfall, which seem to contribute to the reduction, are indicated above the diagram.

A calculation has shown that a total thrust, similar to the measured value, could be predicted if the classical earth pressure theory is used assuming a horizontal ground water level to exist at a height above the bottom of the trench equal to 50 to 60 per cent of the distance from the bottom of the trench to the maximum ground water level. During the autumn, the maximum ground water level observed at the site was approximately 1·9 m above the water level at the start of the excavation.

Earth Pressure Distribution

Because of the lack of theoretical knowledge there is, consequently, a need for an empirical method of distributing the total earth pressure to the different struts in a trench. Based on the measurements from the subway in Chicago, PECK (1943) has

suggested an empirical rule assuming a trapezoidal pressure diagram with an area equal to $1 \cdot 55P$. Although Peck's design method is not intended to be used for shallow cuts or in stiff fissured clays, it was applied to the present case. The values obtained by distributing the total earth pressure calculated by the $c\phi$-analysis, with $c' = 0$ according to Peck's diagram, are compared in Table 2 with the maximum observed strut loads which occurred on 15 November. It can be seen from the figures that except for the under-estimation of the upper and lowermost struts the method appears to be quite satisfactory for application to the excavation described in this paper.

Table 2

Comparison of measured strut loads with those obtained using Peck's distribution rule

Comparaison des efforts mesurés sur étais avec la règle de répartition de Peck

Strut	Depth m	Strut loads—November 15					
		Measured			Computed and distributed by Peck's rule		
		Min. ton	Average ton	Max. ton	Strut load ton	Difference in %	
						Compared to average	Compared to maximum
A	0·33	0·26	0·56	0·94	0·39	− 30	− 141
B	1·00	0·52	1·07	1·72	1·97	+ 84	+ 13
C	1·66	1·32	1·74	2·20	2·14	+ 23	− 3
D	2·33	1·20	1·62	1·88	2·14	+ 32	+ 12
E	3·00	0·99	1·16	1·40	2·33	+ 101	+ 40
F	3·66	0·50	0·98	1·40	0·99	+ 1	− 42

References

BJERRUM, L. and KJÆRNSLI, B. (1956). *Analysis of Some Norwegian Natural Clay Slopes.* To be published.

COOLING, L. F. and WARD, W. H. (1953). Measurements of loads in earth supporting structures. *Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering,* Vol. 2, p. 162

HENKEL, D. J. (1956). Discussion of the paper Some comparisons between measured and calculated earth pressures by W. H. Ward. *Conf. on the Correlation between Calculated and Observed Stresses and Displacements in Structures,* Final vol., p. 491. Institution of Civil Engineers, London

— and SKEMPTON, A. W. (1955). A Landslide at Jackfield. *Proc. European Conf. Stability of Earth Slopes, Stockholm,* 1, 90

JANBU, N. (1955). Application of composite slip surfaces for stability analysis. *Proc. European Conf. Stability of Earth Slopes, Stockholm,* 3, 43

PECK, R. B. (1943). Earth pressure measurements in open cuts, Chicago (Ill.) Subways. *Trans. Amer. Soc. Civ. Engrs.* 108, 1008

SEVALDSON, R. A. (1956). Raset i Lodalen 6, oktober 1954. *Tekniske Meddelelser, Norges Statsbaner,* 2, 42

SKEMPTON, A. W. (1948). The rate of softening in stiff fissured clay with special reference to London clay. *Proc. 2nd International Conference on Soil Mechanics and Foundation Engineering,* Vol. 2, p. 50

TERZAGHI, K. (1936). Stability of slopes of natural clay. *Proc. 1st International Conference on Soil Mechanics and Foundation Engineering,* Vol. 1, p. 161