Strut Loads in a Braced Excavation in Soft Clay

by

P. J. MOORE, B.E., M.S., Sc.D., M.I.E.Aust.
Reader in Civil Engineering, University of Melbourne

and

M. C. ERVIN, B.E., M.Eng.Sc., Stud.I.E.Aust.
Engineer, Country Roads Board of Victoria

SUMMARY. A description is given of a series of model tests in soft clay involving excavation through the clay with the vertical sheeting being supported by spring loaded struts. A method is proposed for calculating the total thrust carried by the struts and comparisons with observations are made. Comparisons are also made between individual observed and calculated strut loads, the latter being determined from a trapezoidal apparent earth pressure distribution.

1 NOTATION

- c_u undrained cohesion
- c_w wall adhesion
- D depth of sheeting
- H excavation depth
- K equivalent earth pressure coefficient
- K_A Rankine earth pressure coefficient
- m reduction factor
- N stability number ($=\rho gH/c_u$)
- N_c bearing capacity factor
- N_D dimensionless depth of sheeting ($=\rho gD/c_u$)
- z depth below top of backfill
- ρ saturated density of soil

2 INTRODUCTION

For the determination of strut loads in braced excavations in soft clay no satisfactory theory has yet been developed. A number of observations for braced excavations has been made and these observations have formed the bases of various empirical design rules. During the construction of the Chicago subway, Peck (1943) found that the magnitude of the total lateral thrust on the excavation was in approximate agreement with that determined from Rankine earth pressure theory. The distribution of lateral pressure however, was non-hydrostatic. Peck also found that almost all of the deformation at a given elevation occurred before excavation reached that depth so that yielding of the sides of the excavation could not be prevented entirely by the use of struts. The Rankine earth pressure distribution is illustrated in Fig. 1.

To facilitate comparison of the total thrusts derived from various lateral pressure distributions over the depth of excavation H, the lateral thrust has been equated to that obtained from a triangular lateral pressure distribution with an equivalent earth pressure coefficient K.

$$\text{total lateral thrust} = \tfrac{1}{2}\rho gH^2K \qquad (1)$$

Hence, for the Rankine pressure distribution

$$\tfrac{1}{2}\rho gH^2 - 2c_uH = \tfrac{1}{2}\rho gH^2K \qquad (2)$$

$$\therefore \quad K = 1 - \frac{4c_u}{\rho gH} = 1 - \frac{4}{N} \qquad (3)$$

Because of longitudinal variations in the soil and variations in construction procedure, Peck proposed that a trapezoidal distribution of lateral pressures drawn to envelope the observed apparent

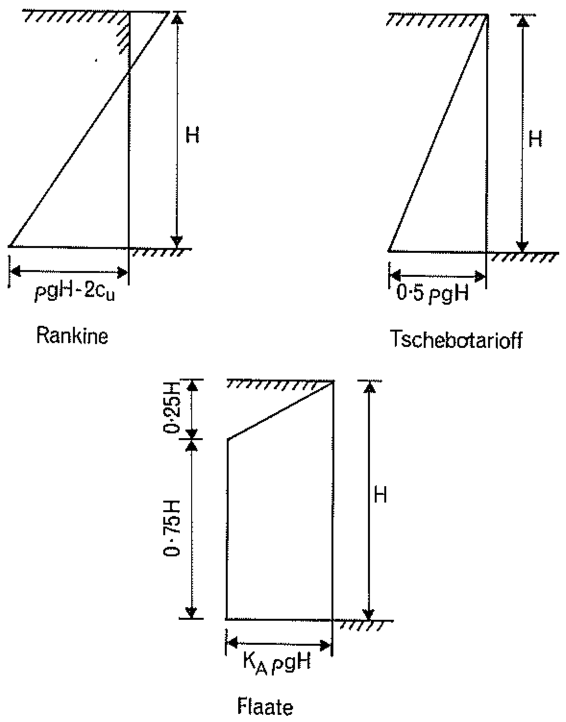

Fig. 1 Pressure Distributions for Braced Excavations

pressures obtained from measurements of strut loads, should be used for design purposes. This trapezoidal distribution originally proposed by Peck has since been superseded by the distribution proposed by Flaate (1966) as shown in Fig. 1. This new distribution has been supported by Terzaghi and Peck (1967), and has been discussed more recently by Peck (1969). With this distribution the value of the earth pressure coefficient K_A, is the Rankine active value given by equation (3). Tschebotarioff (1951) reviewed the literature and proposed the triangular pressure distribution shown in Fig. 1.

The magnitudes of the equivalent earth pressure coefficients K for a triangular distribution of lateral pressure, having total thrusts equal to each of the pressure distributions shown in Fig. 1 have been related to the stability number N in Fig. 2. For comparative purposes some additional lines have been included in this figure. Earth pressure coefficients K have also been calculated

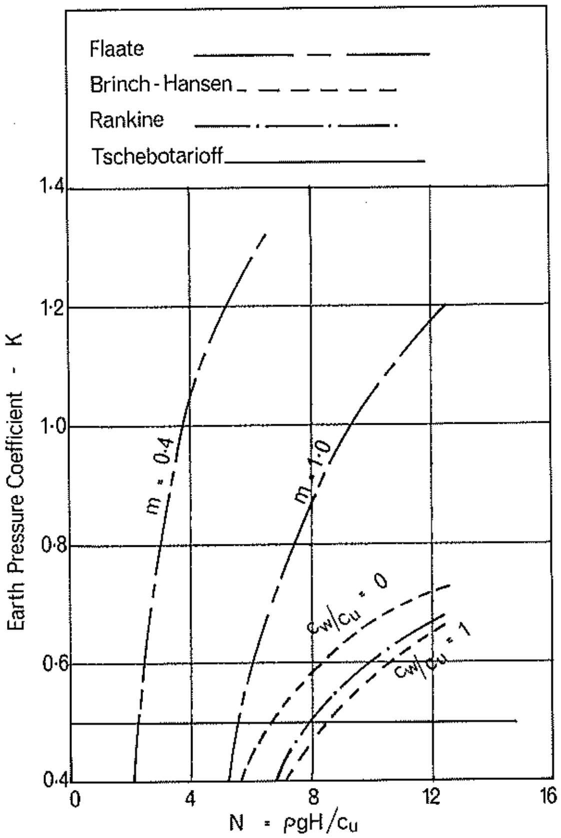

Fig. 2 Equivalent Earth Pressure Coefficients for Various Pressure Distributions

from the method of Brinch Hansen (1953) in which consideration is given to the fact the centre of rotation for the sides of a strutted excavation is located near the ground surface. It was stated above that the coefficient K_A, in the distribution proposed by Flaate in Fig. 1 was equal to the active Rankine value given by equation (3). However, to allow for the unexpectedly high strut loads observed during excavations for the Oslo subway Flaate proposed that the coefficient K_A in his distribution should be modified to the followin expression

$$K_A = 1 - \frac{4m}{N} \qquad (4)$$

where m = a reduction factor (less than unity) to be applied when N exceeds 3 to 4 in the case of clays characterised by those encountered in Oslo.

Terzaghi and Peck (1967) proposed a value of the reduction factor of about 0.4 for these clays. For the two values of m of 1.0 and 0.4 the earth pressure coefficient K has been calculated and is shown in Fig. 2 as a function of N. The wide level of disagreement exhibited in Fig. 2 presents a confusing picture for the design engineer and indicates the need for a much clearer understanding of the behaviour of braced excavations.

3 EXPERIMENTAL INVESTIGATION

To facilitate observation of strut loads for which the excavation may be carried out under closely controlled conditions an experimental arrangement was devised as shown in Fig. 3. With an investigation on a small scale braced excavation, the construction procedure could be standardized. It was hoped that the effects of construction and other variables, which in the field apparently contribute to the wide scatter in reported

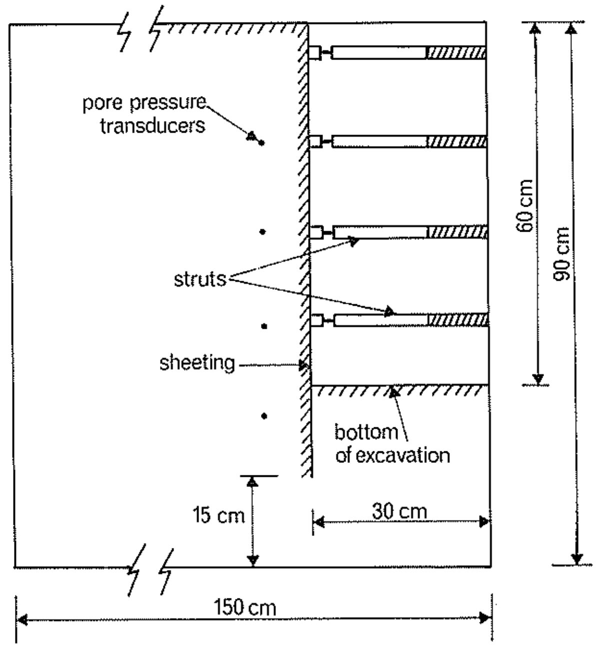

Fig. 3 Experimental Arrangement

observations of strut loads, couldbe minimized. The 23 cm wide bin in which the tests were carried out, was made of marine plywood inside a steel frame. Frictional effects inside the bin were minimized by means of the provision of double thicknesses of polythene sheet separated by silicone grease. In the vicinity of the sheeting an 8 cm wide strip of teflon was attached to the sides to minimize edge friction in the region of movement. The sheeting which consisted of 22 gauge galvanized steel (0.7 mm thick) was supported by struts placed as excavation proceeded. Spring loaded struts were developed for this purpose, with stiffnesses of 70 N/cm and 105 N/cm. Movements of the sheeting and strut deflections were measured by means of probes attached to dial gauges which were mounted on the outside of the bin. Pore water pressures were measured in the retained clay at a number of locations indicated in Fig. 3 by means of pressure transducers. Two types of pressure transducers were used for this purpose, one was an EOS miniature pressure transducer, the other being a Tyco Bytrex AB type pressure transducer.

Kaolin was used for the tests and this clay was prepared at water contents indicated in Table I and then placed by hand in the bin. The clay preparation and testing was carried out in a constant temperature room at 20°C. After placement in the bin the clay was left to stand for two days before excavation commenced (estimated time for 90% consolidation was 3 years). The undrained shear strength (c_u) of the clay was measured by means of a laboratory vane shear apparatus before and after each test. The average values of shear strength throughout the depth of the clay have been quoted in Table 1.

4 TEST RESULTS

Some of the pertinent test data has been summarized in Table 1. The strut load data obtained from Test 1 was somewhat limited in view of the fact that only two struts were placed before failure by base heave occurred. In the second test three struts were placed before failure occurred at a relatively shallow depth. For both of these tests, failure by base heave could have been predicted

TABLE 1

SOIL PROPERTIES AND TEST DATA

Test No.	1	2	3	4	5
Water Content - %	79	78	62	62	60
Saturated density - ρ (Mg/m^3)	1.50	1.54	1.57	1.59	1.58
Average Shear Strength - c_u (kN/m^2):					
before test	0.59	0.86	1.52	1.96	2.12
after test	0.61	1.01	1.70	1.98	-
Excavated depth, H, (cm)	41.2	37.6	58.5	59.1	60.0
No. struts placed	2	3	4	4	4
Maximum N	10.2	6.6	6.0	4.7	4.4
N_D	18.2	12.9	7.6	5.9	5.5
Max. sheeting deflection (mm) at base of excavation	86	41	23	29	19

since the calculated value of the bearing capacity factor N_c was approximately 6.2. This is to be compared with the magnitudes of the stability number N of 10.2 and 6.6 for tests 1 and 2 respectively. Table 1 also indicates the significantly larger magnitudes of deflections of the sheeting for these two tests compared with those observed for the later tests. In most cases the maximum deflection of the sheeting was found to occur around the bottom of the excavation. The lateral movement of the top of the sheeting in all tests was quite small. In all cases the pore water pressures at the four locations shown in Fig. 3 decreased as the excavation proceeded. The pore water pressures started to decrease before the excavation reached the level of the transducer.

5 STRUT LOADS

To facilitate comparison between observed and calculated thrusts over the full height of the excavation the observed thrust was evaluated by adding the measured strut loads together with a

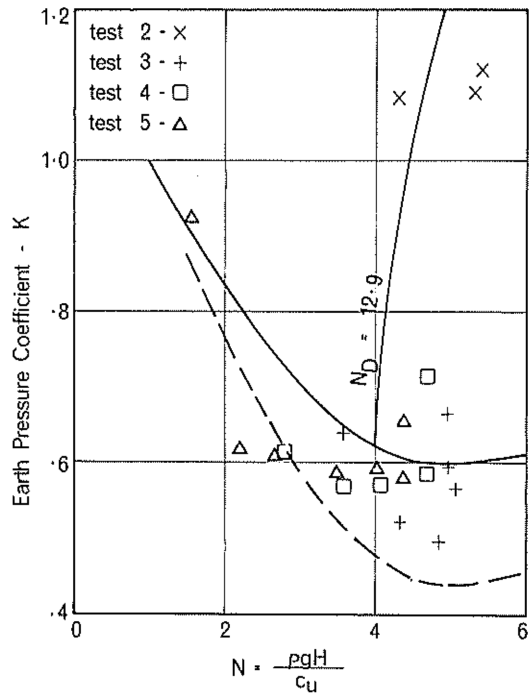

Fig. 4 Observed Lateral Thrusts

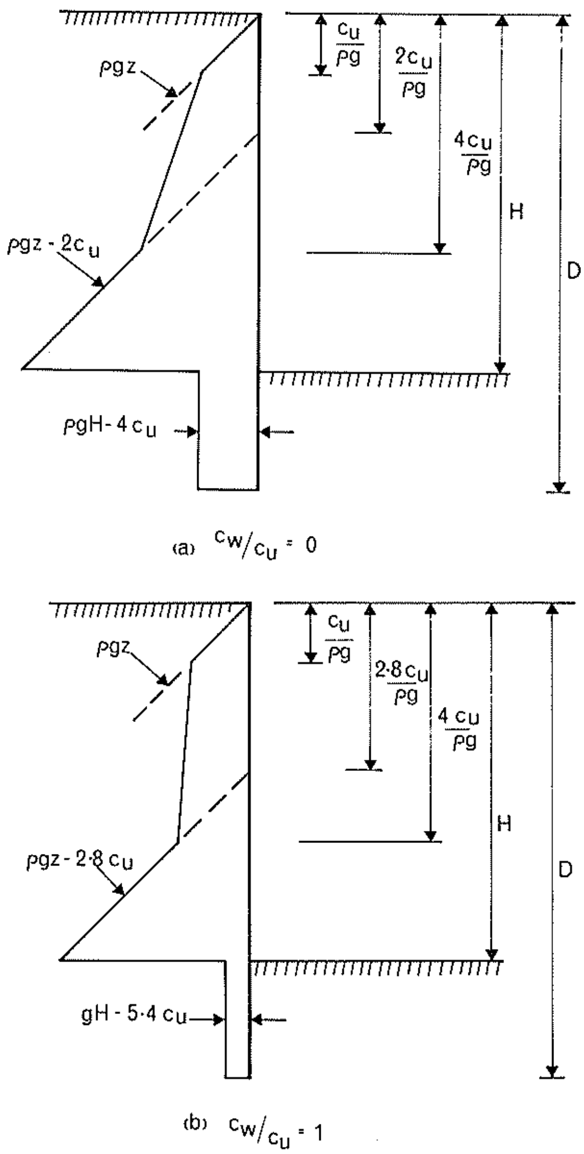

Fig. 5 Proposed Distributions for Calculation of Lateral Thrusts

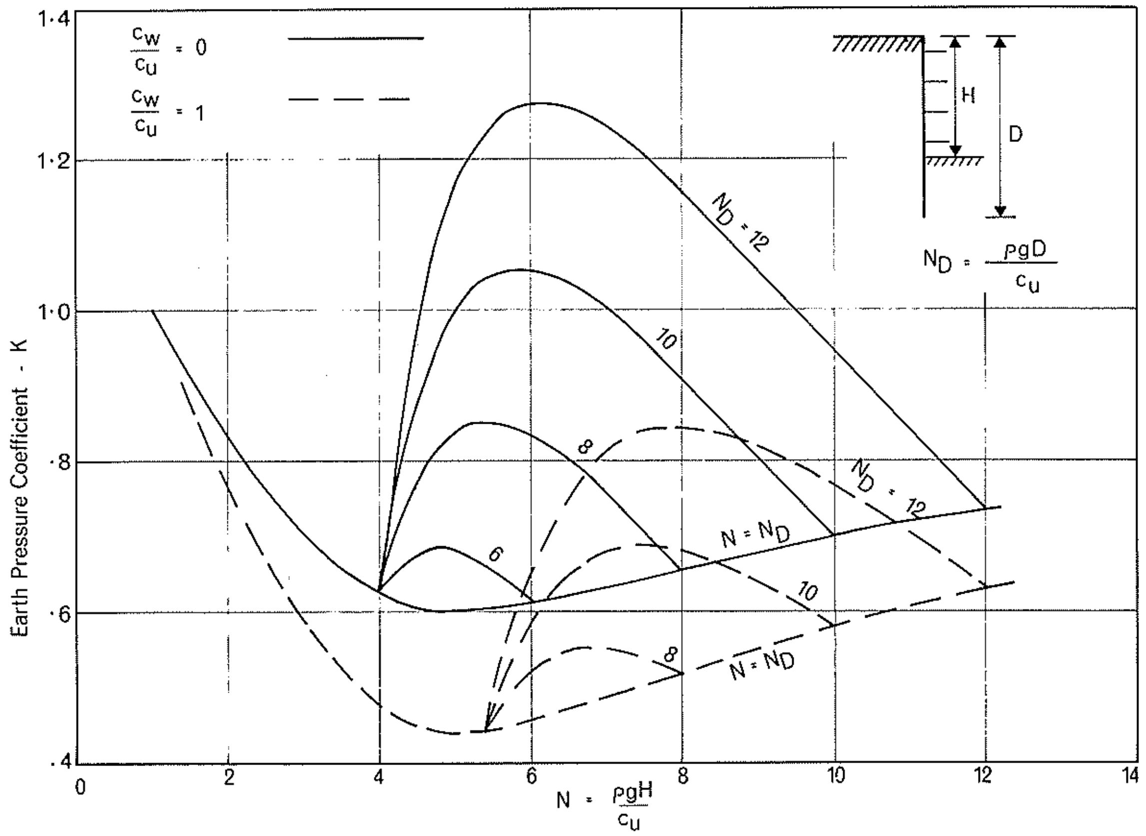

Fig. 6 Equivalent K Values for Proposed Pressure Distributions

correction for the estimated shear force in the sheeting at the base of the excavation. This shear force was estimated by the same procedure recommended by Terzaghi and Peck (1967) in which the apparent earth pressure at the bottom of the cut was assumed to be the same as that computed from the lowest strut. For comparative purposes the thrusts have been expressed in terms of an equivalent earth pressure coefficient K based upon a triangular distribution of lateral pressure. The observed results have been presented as a function of the stability number N in Fig. 4. Of the apparent pressure distributions shown in Figs. 1 and 3 the only one that yields values of the earth pressure coefficient K in approximate agreement with the observations is that proposed by Flaate (1966) for values of m in equation (4) lying between 0.4 and 0.8. Since Flaate's earth pressure distribution is based upon an envelope of observed apparent pressure distributions it cannot be expected that the total thrust calculated from his distribution will be in agreement with any observed value.

In an attempt to devise an earth pressure distribution which may be used for the sole purpose of calculating the total thrust to be carried by the struts, the earth pressure distributions for smooth and rough walls are proposed in Figs. 5(a) and (b) respectively. With these pressure distributions it has been assumed that at-rest pressures will act over the sheeting down to a depth corresponding to N = 1. The pressure distribution then gradually merges into the Rankine active pressure distribution at a depth corresponding to N = 4. The value of N = 4 was chosen since in this series of tests it was observed that significant movements of the sheeting commenced at values of N of approximately 3 to 4. Terzaghi and Peck (1967) have made a similar comment in relation to field observations. At depths shallower than this it was considered to be highly unlikely that active or passive pressure would be mobilised. As shown in Fig. 5 the full active and passive pressure distributions that exist on either side of the sheeting below the excavated depth H have also been included in considering the total thrust to be carried by the struts. Observations on the Oslo subway have shown that full active and passive pressure distributions existed on either side of

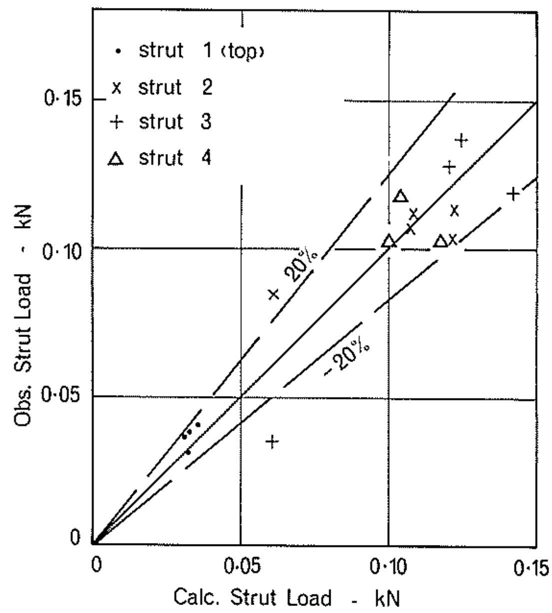

Fig. 7 Calculated and Observed Strut Loads

the sheeting. The equivalent earth pressure coefficients K, corresponding to the earth pressure distributions in Fig. 5 have been shown in Fig. 6 as a function of the stability number N. For N less than 4 it has been assumed that no passive pressure has been mobilized and the pressures on either side of the sheeting below the depth of excavation are equal. If portion of the theoretical curves in Fig. 6 are reproduced in Fig. 4 it is seen that a rough measure of agreement is obtained between calculated and observed values of the earth pressure coefficient K.

If an appropriate value of K is determined from Fig. 6, the corresponding total thrust to be carried by the struts may be calculated from equation (1). If this total thrust is now distributed over the height of excavation H in accordance with the shape of the apparent earth pressure distribution proposed by Flaate (Fig. 1) the calculated magnitudes of the individual strut loads may be compared with those observed. The individual strut loads have been calculated from the apparent earth pressure distribution by means of the same technique discussed by Terzaghi and Peck. These calculated strut loads have been compared with the observed strut loads in Fig. 7. It is seen that the majority of the observed strut loads fall within a range of ±20% of the calculated strut loads.

6 CONCLUSIONS

A method has been proposed for the calculation of the total thrust to be carried by the struts and approximate agreement has been obtained with observed values. Using this calculated thrust and the shape of Flaate's apparent earth pressure distribution individual strut loads have been calculated and the observed strut loads are found to be within ±20% of the calculated values.

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