Earth Pressure, Retaining Walls, Tunnels and Strutted Excavations

Poussée des terres, murs de soutènement, tunnels et puits dans les sols

General Report

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Introduction

In this Report I shall deal with the various aspects of earth pressure under the following headings; *General theory and experiment*, *Anchored sheet pile walls*, *Strutted excavations*, *Tunnels*, and *Abutment pressures*. In each case I shall consider some of the more important papers published since the Second Conference in 1948, as well as those submitted to the present Conference. With regard to the papers published between 1948 and 1953 only three authors sent me copies of their work specifically for discussion in this Report. I shall mention several others, but if any publications have been overlooked I apologise to the authors concerned.

(1) General Theory and Experiments

Although not an original contribution to theory, one of the important publications concerned with earth pressure which has appeared since 1948 is the *Code of Practice on Earth Retaining Structures*, issued in 1951 on behalf of the Civil Engineering Codes of Practice Joint Committee by the Institution of Structural Engineers, London. This document, of over 200 pages, accepts fully the modern methods of earth pressure calculations, based on soil mechanics principles. After describing these, and giving the relevant equations and graphs, the Code goes on to give construction details of gravity walls, reinforced concrete walls, sheet pile walls, cribwork, revetments and sea walls. There are nine appendices, one of which describes fourteen failures of various types of earth retaining structures, with diagrams and photographs. The Code was drafted by a number of committees, under the general chairmanship of Mr. *Wentworth-Sheilds*, and the various drafts were co-ordinated by the Research Secretary of the Institution.

An outstanding contribution to earth pressure calculations was made by *Caquot* and *Kerisel* (1949) in Part IV of the 2nd edition of their book «Traité de Mécanique des Sols». Based on the assumption that all stresses increase in direct proportion to the distance from the free surface, these authors have solved the *Boussinesq-Resal* equations of equilibrium in a cohesionless material using a step-by-step method of integration. The results are applied to the determination of the active and passive pressure on retaining walls with any values of wall friction, inclination of wall and sloping backfill; provided the wall tilts about the base. The coefficients are set out in Tables[1]), and are a little less favourable than those obtained by the graphical logarithmic spiral method (with no restriction on the position of the centre of rotation). These coefficients may be considered to be the final solution to the particular problems dealt with by the authors.

In the realm of more speculative theory *Tschebotarioff* raised some interesting points on the question of "arching", at a Conference held in Essen, January 1952. He differentiates between redistribution of pressure by shear forces, "silo action" and true arching; suggesting that the latter condition is possible only in sands and only where practically unyielding "abutments" can exist with an almost complete absence of any slip surface in the sand. There must also be downward movement of the soil beneath the zone of arching. In practice, the author considers that examples of true arching are provided by

(i) anchored sheet pile walls in sand where there is little if any outward yield of the anchors; the arching being developed in the sand at and above the level of the anchor (e.g. tests at Princetown and a sheet pile wall studied by *Martin Duke*, 1952),

(ii) in the upper zone of sand behind a strutted excavation (e.g. Sixth Avenue Subway N.Y.), and

(iii) in certain special cases such as that described by *Pennoyer* (1945) where sand had been eroded out of the bottom of some of the cells of a double wall cofferdam and the remaining sand, above water level, with a thickness of about 23 ft., remained stable with an arched or domed undersurface spanning some 27 ft.

In the above mentioned examples the value of the earth pressure coefficients in the zone of arching are computed by *Tschebotarioff* to be of the order 2 or 3. In contrast, if the sand is slipping downwards relative to the wall the maximum value

[1]) The Tables have been published in English translation. Gauthier-Villars, Paris.

of the earth pressure coefficient is that corresponding to the passive state with negative wall friction and this is not greater than unity, as shown by *Caquot* and *Kerisel* (loc. cit.).

Reference should also be made to the chapter on "Lateral Earth Pressures" in *Tschebotarioff*'s text book published in 1951. The majority of the results given in this chapter were naturally obtained before 1948 but, in collecting together the available information (theoretical, experimental and from the field) and discussing it from an up-to-date and consistent point of view, the author has rendered a useful service.

Although *Rowe*'s paper (1952) will be considered in more detail under the heading "anchored sheet pile walls" some of his experimental results have a more general interest and may be mentioned here. A model pile wall 3′ 6″ high and 7′ long was used, and in this wall 47 pressure gauges and 104 strain gauges were mounted. Initially, sand was placed on both sides of the wall up to its top. Without allowing any yield of the tie rods, sand was then removed from in front of the wall. As this process occurred the total pressure on the back of the wall progressively decreased until, for ratios of height above dur-edged level to total height of more than 0.6, it reached a value equal to that given by the *Coulomb* wedge theory (with the angle of wall friction $\delta = 20°$). The distribution of pressure was then markedly different from linear, there being a strong concentration of pressure at the level of the anchor and a correspondingly small pressure near mid-span of the piling. This distribution was similar to that found under similar conditions by *Tschebotarioff* (1949) and others. However, the anchors were than allowed to yield, and in all of the 16 tests (with various positions of the anchor and various values of surcharge) the pressure distribution became triangular, although the total earth pressure remained unchanged. The yield of the wall at anchor level required to bring about this triangular distribution never exceeded $L/1000$, where L is the height of the wall: and this movement is less than that which may be expected to take place in practice, due to elastic extension of the tie rods and horizontal movement of the anchor block. The sand was loosely packed, with $\varphi = 30°$. The tests also showed that a passive pressure corresponding to $\delta = 0$, was mobilised at a movement of about $0.002\,D$ (where D is the penetration of the piling) but full passive pressure (with $\delta = 20°$) was mobilised only after a movement of about $0.05\,D$. It is to be hoped that *Rowe* will repeat these tests with a more densely packed sand, but the conclusion at the present time would seem to be inescapable, namely that it is unwise, in general, to depend upon any reduction in bending moment in anchored sheet pile walls due to a distribution of active pressure more favourable than that given by the classical assumption of a basically linear distribution.

In another series of tests *Rowe* measured the distribution of passive pressure on a stiff plate which could be rotated about its upper or lower edge. For rotation about the upper edge the pressures were very small near the surface and then increased rapidly with depth, the centre of pressure being at a height of considerably less than $D/3$ above the bottom of the plate (where D is the penetration of the plate in the sand). For rotation about the lower edge the pressure built up to high values quite close to the surface of the sand and then decreased with depth, the centre of pressure being at about $D/2$ in loose sand and appreciably higher than $D/2$ in dense sand. As will be seen later, these results are highly significant in the behaviour of sheet pile walls.

In an important paper by *Terzaghi* (1953) on "*Anchored Bulkheads*", which will also be more fully considered in the following section, many points of general theory are discussed. These are, the effects of wall movements on the mobilisation of shear in sands, the relation between the coefficient of passive pressure and the magnitude of wall friction, the calculation of unbalanced water pressures on walls in sand, a detailed treatment of line and point surcharge loads, the influence of yield of the wall and anchors on active and passive pressure distribution (in which *Rowe*'s tests are given prominence) and a summary of modern ideas on shearing resistance of soils as applicable to earth pressure calculations. This paper should be read by all civil engineers, and there is no need for me to make further comment on the general aspects in this Report.

The only theoretical paper on earth pressure contributed to the Conference is by *Brinch Hansen* (7/5) who presents a method by means of which the magnitude and point of action of the earth pressure can be determined for a specified position of the centre of rotation of the wall. The assumptions are that the slip surface takes the form of a circular arc, that *Kotter*'s equation holds good, and that the stresses at the surface of the ground, where the slip circle emerges, are given by a special boundary condition evolved by the author from a consideration of conjugate stresses. I am not certain that it is strictly justifiable to use *Kotter*'s equation for an arbitarily chosen slip surface, such as a circle, but the results obtained appear to be interesting from the brief summary given in the paper. For example, in sand, with rotation about the top of the wall, in the active case, the earth pressure coefficient in 0.355 and the centre of pressure is situated at $0.48\,H$ above the base of the wall (for $\delta = 0$ and $\varphi = 30°$). Similarly for the same values of δ and φ and for rotation about the top of the wall, in the passive case, the coefficient is 2.6 and the centre of pressure is at a height of $0.16\,H$. This result is comparable to that obtained in *Rowe*'s passive pressure tests.

(2) Anchored Sheet Pile Walls

A paper of basic importance on the problems of anchored sheet pile walls in sand was published by *Rowe* (1952). Brief reference to this has already been made, but in this section attention must be drawn to the chief point in the investigation, namely the influence of the flexibility of the piling on the bending moments set up by the earth pressures. Model walls varying from 20 to 36 inches in height were used covering a range of flexibilities, expressed by the parameter log (L^4/EI), from —3.7 to —2.1 (see Fig. 1). In each test the tie rods were allowed to yield by the small amount necessary to destroy any arching in the sand, and the stresses in the wall were measured, by strain gauges. To an accuracy of about $\pm 10\%$ the ratio of the actual

Fig. 1 Bending Moment—Flexibility Curves: Rowe (1952). Sheet Pile Walls in Sand
Courbes des moments de flexion et de flexibilité. Ecrans de palplanches dans du sable

bending moment to the moment calculated on the assumption of the "free end" condition remained constant for any given wall for all practical values of the depth of dredging, level of the tie rods and surcharge pressures. In contrast, this ratio of the actual to the "free end" moment decreased in a very marked manner with increasing flexibility, as shown in Fig. 1, and depended also on the relative density of the sand. Almost identical curves were obtained for sand, pea gravel and rock chippings. The reduction in moment is attributed, primarily, to a rise in centre of pressure of the passive resistance in the sand in front of the wall. This is in accordance with the tests mentioned earlier. The passive pressure on very stiff walls is similar to that measured on a plate rotating about the sand surface (in front of the wall); while on a very flexible wall the passive pressure is similar to that on a plate rotating about its lower edge, when the centre of pressure is much higher than with the stiff wall. Moreover, that some such factor is involved has been demonstrated by *Rowe* (1952, p. 642ff.) in later tests in which the "active" pressure was applied by water, in which, of course, there can be no possibility of arching or shear transfer. Nevertheless a substantial reduction in bending moment was observed with increasing flexibility.

The well-known series of model tests at Princetown by *Tschebotarioff* (1949) were found to be in agreement with *Rowe*'s results (see Fig. 1). But the Princetown tests cover only a part of the range of moment reduction, and the simplified design rules for calculating bending moments, suggested by *Tschebotarioff*, are, on the basis of *Rowe*'s tests, not on the side of safety for relatively stiff walls.

Rowe put forward a method of design for anchored sheet pile walls in sand, based on his moment reduction curves. Briefly, this consists in calculating the depth of penetration, the anchor pull and the bending moment, by the classical "free end" procedure (with a suitable factor of safety on the passive pressure) and then choosing a section of sheet piling such that the bending moment as reduced in accordance with the curves in Fig. 1 (for the particular value of L^4/EI of the piling) will set up stresses which do not exceed the allowable working stresses of the material of the piling. In typical cases, for steel sheet piling, the bending moment will be of the order 25 to 60 per cent less than the "free end" moment. In principle, the tie rod loads are also reduced by flexibility of the piling, but this reduction is typically only 15 to 30 per cent of the "free end" value and, on account of differential yield or pressure concentration at the anchor level, *Rowe* suggests that little reduction should be allowed in practice.

In his paper on "Anchored Bulkheads", to which reference has already been made, *Terzaghi* adopts *Rowe*'s design method for sands, and suggests a unified procedure applicable to all soils. This procedure may be summarised as follows.

(I) The depth of penetration should be 20 per cent greater than that calculated from classical "free end" theory, using a factor of safety of 2 or 3 on the coefficient of passive pressure in sands and 1.5 or 2 on the shear strength in silts and clays. The lower factors of safety in silts and clays arise from the rather conservative expression used by *Terzaghi* for passive pressure in these materials. The 20 per cent extra penetration arises from a recognition of the fact that considerable lateral displacements are associated with the mobilisation of passive pressures.

(ii) The bending moment is taken as being equal to the "free end" value multiplied by a reduction factor, depending upon the flexibility of the piling *and* the nature of the ground. In clean sand the reduction factors are those given by *Rowe* (Fig. 1). In dense or medium silty sand the reductions are less, and in loose silty sands or silty sand filling no reduction is allowed. Nor is any reduction allowed in clays. But, on the other hand, it is suggested that for walls in clays rather higher steel stresses, than have been usual in the past, could be accepted.

(iii) The tie rod loads are treated in the same manner, namely *Rowe*'s factors are applied when the backing is sand, smaller reductions are applied in dense and medium silty sands and no reduction is applied in loose silty sand, silts or clays.

This procedure, which is only outlined here, is of great interest, and chiefly because it makes use of the hitherto almost neglected factor of flexibility of the piling and because it presents a unified approach applicable to all soil types, yet taking into account the major difference in behaviour of these various soil types.

The question naturally arises as to what changes the method based on moment reduction curves is likely to bring about in the safety and economics of sheet pile design. It is too early for any definite statements to be made, but in my opinion the new method is basically more logical than the older design procedures and will, as experience grows, lead to improvements from both points of view. The outstanding need at the present time is for reliable observation on earth pressure and stresses in full scale sheet pile walls and it seems that further advance is now largely dependent upon such observations.

Meanwhile, however, an instructive series of comparisons have been made, by *Packshaw* and *Lake* (1952) between the designs resulting from the use of *Rowe*'s method and the normal "fixed end" method, for walls in sand. In a total of ten cases considered, the *average* differences in length of piling, in maximum beading moment and in tie rod load, were small, yet in *individual* cases the differences amounted to ± 10 per cent (in length), ± 40 per cent (in bending moment) and ± 15 per cent (in tie rod load).

Whereas *Rowe*'s work was entirely experimental, *Blum* has published (1951) a monograph in which the influence of flexibility can be calculated, using a modulus of subgrade reaction which is assumed to increase linearly with depth below dredge level. The computational work involved appears to be too lengthy for use as a standard design procedure, and requires a knowledge of the modulus for the two-dimensional conditions of a sheet pile wall; and the author does not indicate how this is obtained.

In a paper to the Conference by *Roisin* and *Verdeyen* (7/9) a modification of the usual "fixed end" method, less radical in its changes than that of *Rowe* and *Terzaghi*, is put forward for sheet pile walls in sand. This allows for pressures greater than the active in the zone above the tie rod, coupled with a reduction in active pressure below dredged level, and assumes that the reaction of the "equivalent beam" is situated at the point of zero net pressure. The bending moments appear to be greater than those computed either by the fixed end or by *Rowe*'s method. The authors also apply their suggested pressure distribution to the problem of calculating strut loads in braced cofferdams in sand. More detailed treatment of the anchored wall and cofferdam problems will be found in two papers by *Verdeyen* and *Roisin* (1952).

Turning now from theory and experiment to field work, we should note a series of observations reported by *Duke* (1952) on a sheet pile wall 55 ft. high (above sea bed level) and penetrating 22 ft. into sand. The lower half of the wall above bed level was supported by a rock fill dyke, and the tie rods were placed 13 ft. below cope level. Behind the wall a hydraulic fill was placed, consisting chiefly of fine to medium sand. Pressures on the sheeting were measured directly by pressure cells; loads in the tie rods were also measured, as were pore pressures in the fill and deflections of the sheeting. During placing of the fill the pressure cells recorded a roughly linear increase in pressure with depth, with an earth pressure coefficient computed by the author to be about 0.7. The piezometric heights of the pore pressures indicated a downward flow of water in the fill at low tide (when most of the earth pressures were recorded). This would reduce the earth pressure coefficient to an average value of about 0.6. This value is, however, higher than would be expected even for the earth pressure at rest; and since the effective deflection (the total deflection relative to the walling was about $3\frac{1}{2}$ inches or 0.35 per cent of the total length of piling) of the sheeting was about 3 inches relative to the tie rod wallings, it would seem more probable that K was actually at the active value of about 0.3 or 0.35. I am inclined to think that this discrepancy is due to the calibration of the pressure cells in the laboratory not applying accurately to their behaviour in the fill. Nevertheless, until more field information is obtained, these observations suggest that caution should be used where hydraulic fills of fine sand are involved.

Subsequent to completion of the filling, the pressures on the wall above the tie rods increased considerably, while the pressures below decreased. The author attributes this effect to settlement of the fill, causing "arching" in the sand and a downwards drag on the tie rods. This is the example to which *Tschebotarioff* refers (1952), as mentioned previously.

Two reports on sheet pile walls in clay have been presented to the Conference. The paper by *Feld* (7/4) describes the failure of a quay wall built on a marsh. Dredged level was 23 ft. below coping, and at low water, tide level stood 11 ft. below coping. The ground behind the wall consisted of about 15 ft. of filling (partly dredgings and partly silty sand deposited by crane) overlying 19 ft. of marsh deposits and very soft silt. Beneath these, lay a stiff silt and a very stiff clay with pebbles. The wall consisted of heavy section steel sheet piling driven about 20 ft. into the stiff strata, and tied at 10′ 6″ centres to pairs of battered anchor piles, also driven into the stiff silty clay, and joined by a concrete beam. Shortly after the filling and dredging had been completed, the wall moved outwards, reaching a maximum displacement of 6 ft. at the top near the centre of the length of wall. The anchor beam moved upwards and outwards by roughly 2 ft. A week or two before the failure, filling (10 ft. deep) had been deposited for a road situated 500 ft. or 600 ft. from the wall. It set up "mud waves" in the underlying soil and the author attributes the wall failure to this effect. The simpler explanation would seem to be that the failure was due to insufficient anchorage to resist the forces set up by the weight of the filling immediately behind the wall, but since no soil mechanics data are given in the paper it is, perhaps, difficult to draw any conclusions of value.

The second paper concerning sheet piles in clay is that by *Lea* (7/7). This describes the partial failure of a wall in 1944 due to the placing of an ore pile on the ground behind it. The strata consisted of about 10 ft. of slag filling and sand, overlying 30 to 40 ft. of varved clay which, in turn, rested on a few feet of silt and then hard boulder clay. Coal had previously been stored on the site, and some consolidation had occurred in the clay. Vane tests in 1951, showed the clay to have a shear strength of about 1050 lbs./ft^2. The sheet piles were rather more than 60 ft. long, and penetrated the boulder clay for a depth of 6 or 7 ft. The top of the wall was tied back to an anchor wall, 1100 ft. back from the sheet piles, and the anchor wall itself was tied to the pile cap of the foundation supporting the shear leg of the gantry at other side of the ore pile. An outward movement of 1″ of the piling was observed in 1943, but a year later, when the height of the ore pile was substantially increased, a further 2″ movement occurred and there is evidence that during this movement the tie rods to the anchor were stressed to their yield point. Using the assumption of "free earth" support for the sheet piling, the author has calculated the tie rods loads at this stage using a form of wedge theory and also an assumption that the lateral pressures due to the surcharge are twice those given by elastic theory. From the information in the paper it is not possible to follow these calculations in any detail; and therefore little comment can be made.

It should, however, be pointed out that the surcharge load under the maximum height of the ore was about 4,500 lbs./sq.ft., which is equal to at least 4 times the shear strength of the clay. Yet the ultimate bearing capacity of the clay was probably not greater than 6 times the shear strength. Hence the factor of safety against the ore pile breaking into the ground on which it was placed was probably less than 1.5. And it is reasonably certain that under such conditions the lateral pressures developed in the clay, and hence on the wall, would have been of dominant importance. Under similar conditions *Terzaghi* (1950) briefly records serious movement of a quay wall supporting clay strata on which an ore pile had been placed, although the factor of safety against ultimate foundation failure never fell below 1.5 .

(3) Strutted Excavations in Clay

Since 1948 there have been, to my knowledge, two papers published reporting observations regarding the loads set up in the framing of deep excavations in clay coupled with reliable data on soil properties. The first, by *Skempton* and *Ward* (1952) related to a cofferdam in the soft estuarine clays of the Thames esturary at Shellhaven. These clays (which had a sensitivity of 5 or 6) extended to a depth of 46 ft., below which there was about 6 ft. of firm sandy clay and then sandy gravel. The walls of the cofferdam consisted of steel sheet piling 65 ft. long, the width was 50 ft. and the depth of excavation was 32 ft. The piling was supported by three frames of steel joists. The soil properties were investigated in detail and measurements were made of the loads transferred from the piling to the walling, by means of vibrating wire gauges. These gauges are described in a paper to the Conference by *Cooling* and *Ward* (7/3) who show a cross section of the cofferdam and a typical set of load readings. Strains in the struts were also measured, using stick-on electrical resistance gauges but these did not prove very satisfactory. The vibrating wire load gauges, on the other hand, were completely successful.

When excavation was completed the loads in each of the three frames (spaced at the half and quarter points of the depth) were practically identical (see Table 1). On driving foundation piles at 6 ft. centres within the cofferdam, the soft clay was disturbed with a consequent loss in strength, and the load in the lowest frame increased by about 60 per cent. At any given stage of construction the variations in strut loads in a particular frame were of the order ± 50 per cent.

The earth pressures acting on the sheet piling were calculated, with $\varphi = 0$ in the clay in the normal manner (as given in the *Civil Engineering Code* of Practice in Earth Retaining Structures 1951) and the centre of pressure of the active pressure in the depth of excavation H was at a height of $y = 0.30\, H$ above the bottom of the excavation. It was then shown that, with this classical earth pressure distribution, the corresponding loads in the frames depended to a marked degree on the deflections of the sheet piling at the level of the lower two frames (relative to the upper frame) prior to placing these frames. If, for example, no deflection had occurred the loads were as in Table 1 column (2) but for other deflections, two of which are given in the Table, the calculated loads were close to those observed.

Table I Calculated and Actual Frame Loads, Shellhaven Cofferdam tons per ft. run

Deflection at Frame II *Deflection at Frame III* *(1)*	*0* *0* *(2)*	*1.0 in.* *1.7 in.* *(3)*	*1.4 in.* *2.25 in.* *(4)*	*Actual Loads* *(5)*
Frame I	1.4	3.0	4.1	3.6
Frame II	0.4	3.9	3.8	3.6
Frame III	12.8	4.9	3.1	3.5
Total Load	14.6	11.8	11.0	10.7

Moreover, these deflections were similar to those measured by *Peck* (1943) in a sheet pile cofferdam under comparable conditions in Chicago. *Skempton* and *Ward* therefore concluded that, in spite of the actual frame loads being practically equal, it was possible that no redistribution of active pressure in the clay had occurred. Hence, as a lower limit for the centre of earth pressure $y/H = 0.30$. The true value of y/H was probably rather greater as it is likely that some small degree of pressure redistribution had occurred.

The significance of these conclusions can be seen in Fig. 2 where the values of y/H for various strutted excavations in clay are plotted against K_e the equivalent coefficient of earth pressure, defined by the equation

$$P_a = K_e \frac{\gamma H^2}{2}$$

where P_a is the active pressure in the depth H. Of the points in this graph those for Park Village East (*Golder*, 1948) and Chicago Contract D6E (*Peck*, 1943) are definitive. In these

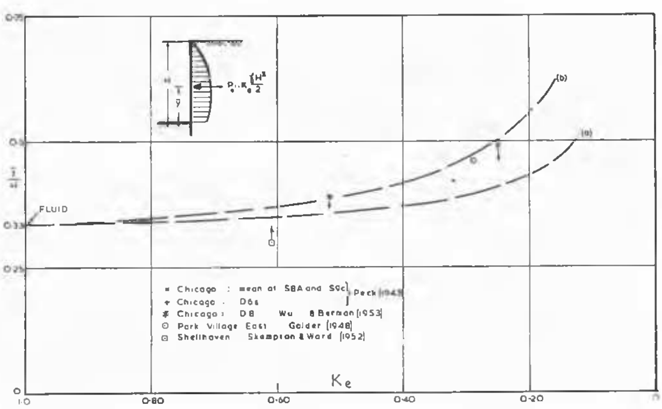

Fig. 2 Relation Between Position of Centre of Pressure and Equivalent Depth of Excavation in Clays
Rapport entre la position du centre de pression et la profondeur équivalente d'excavation dans les argiles

two excavations there was no passive pressure on the sheeting, for the simple reason that the sheeting did not penetrate below the bottom of the excavation. Hence P_a equalled the total strut load and from the strut loads y/H could be uniquely determined. At the other locations in the Chicago excavations, Contracts S8A and S9C, the passive pressures were small. Consequently, as an upper limit, but with little loss of accuracy, the value of y/H can be determined for these sections in the same manner. The average result of the two sections is shown in Fig. 2.

The second paper on cofferdams in clay, published by *Wu* and *Berman* in 1953, enables a fifth point to be plotted. The excavation was also in Chicago (Contract D8), but was considerably deeper (63 ft.) than those previously reported by *Peck*. There were five frames and the strut loads were measured by 10 inches *Whittemore* strain gauges on a total of fourteen profiles. The maximum variations in load in a particular frame were of the order ± 45 per cent.

The ground consisted of 14 ft. of sand followed by 42 ft. of soft to medium clay. The last 7 ft. of the excavation was in harder clay and the sheet piles penetrated this clay for a further 7 ft. There would have been an appreciable passive pressure, and therefore the actual value of y/H was lower than that given by the strut loads. Also, as in the case of Shellhaven, deflections of the sheet piling prior to placing the frames would lead to a higher value of y/H, as deduced from the strut loads, than that of the earth pressure itself. Consequently the centre of pressure of the strut loads (equal to 0.39 H) is an upper limit for the centre of earth pressure, and is so indicated in Fig. 2.

The importance of the Shellhaven and the Chicago D8 observations is that, for the first time, they give information on the probable centre of active pressure for excavations which are deep compared with the critical height of the clay. From these observations the conclusion can be made that for excavations with K_e of the order 0.5–0.6 the centre of active pressure is located at a height of about 0.35 above the bottom of the excavation. In contrast, the previously available data was for cuts where K_e was about 0.30 and in those cases the value of y/H was about 0.45. Therefore, there is a strong indication that, as would be expected, the height of the centre of pressure decreases as the clay becomes softer (relative to the depth of excavation) and approaches in the limit the value of 0.33; which is, of course, the value for fluid loading ($K_e = 1$).

In 1943 *Terzaghi* indicated a method for calculating y/H ("Theoretical Soil Mechanics", pp. 186–188). Assuming the slip surface is circular and that it intersects the ground surface at right angles, P_a can be expressed in terms of y/H. The equation is:

$$P_a\,(1 - y/H) = \frac{\gamma H^2}{2}\left(\frac{2}{3} - 2.764\,\frac{c}{\gamma H}\right)$$

where c is the shear strength of the clay, with $\varphi = 0$. If it is then further assumed that, in all cases, P_a has the value given by classical theory, namely

$$P_a = \frac{\gamma H^2}{2}\left(1 - \frac{4c}{\gamma H}\right)$$

or $K_e = 1 - 4c/\gamma H$ assumption (*a*)

$$\text{then } \frac{y}{H} = 0.309 + \frac{0.024}{K_e}$$

Alternatively it can be assumed that P_a has, in all cases, the value given by the circular slip method, namely

$$P_a = \frac{\gamma H^2}{2}\left(1 - \frac{3.83c}{\gamma H}\right)$$

or $\quad K_e = 1 - \frac{3.83\,c}{\gamma H}$

then $y/H = 0.278 + \frac{0.055}{K_e}$ assumption (*b*)

The curves relating y/H to K_e for assumptions (*a*) and (*b*) are plotted in Fig. 2 and it will be seen that they serve as reasonable limits to the field data.

From the more immediately practical point of view it is to be noted that in all the cofferdams so far observed, the strut loads are given approximately by *Peck*'s empirical method of design (*Peck*, 1943). This method is therefore proving a valuable aid to the engineer.

(4a) Tunnels in Clay

The opportunities of carrying out field measurements on the stress in the linings of tunnels in clay are not of frequent occurrence. Consequently the paper by *Cooling* and *Ward* (3/3), in the Conference Proceedings, is an important contribution, since it contains records of such measurements in three tunnels at depths of about 100 ft. in the London Clay. Two of those tunnels were adjacent to each other. They had a diameter of 25 ft. and the normal type of cast iron lining used in the London area. Vibrating wire strain gauges were fixed at 18 positions in one ring in each tunnel. In tunnel No. 2 South the stresses were found to be practically uniform around the ring, and subsequent to a period of two months after construction the stress remained constant with time. Moreover the average value of these stresses was practically equal to the stress corresponding to that set up by a hydrostatic pressure equal to the total overburden pressure acting on the tunnel.

In contrast, the stresses measured in tunnel No. 1 South were non-uniform around the ring and their average value was only about 45 per cent of the stress corresponding to full overburden pressure acting on the tunnel.

These tunnels formed the southern pair of four closely adjacent tunnels, No. 1 South being in the outer position. This may account for the differences between the results, but it should be noted that only one ring in each tunnel was observed and the differences may be merely due to scatter as between any two rings in the same tunnel. Accepting this as the simplest explanation, the average stress in both tunnels amounts to about 70 per cent of the stress corresponding to full overburden pressure.

In the third tunnel 13 rings were investigated, using vibrating wire and hydraulic load gauges; the former being used to measure both radial and circumferential pressures. The behaviour of the vibrating wire gauges is reported to have been entirely satisfactory and the results of this investigation, when fully presented, will undoubtedly be of great value. In the paper to the Conference, however, only two typical results are given, from which it is difficult to draw any definite conclusion. Nevertheless these results indicate that the loads in the tunnel lining attained a constant value about 2 months after construction (and in a personal communication from the authors I learn that this constancy has been maintained over the past year) and that this value is about 60 per cent of the load corresponding to full overburden pressure. The tunnel was 9 ft. in diameter, at a depth of 90 ft.

The chief value of this paper is perhaps in showing that the vibrating wire type of gauge is very reliable under practical conditions and over considerable periods of time. Yet, although any deduction as to the earth pressure acting on the tunnel is probably premature, pending more detailed publication of results, there does seem to be a strong suggestion that on the average this pressure is roughly 60 or 70 per cent of the full overburden pressure. When it is realised that London Clay, at depths of about 100 ft., is a stiff material with a shear strength of the order of 50 lbs./sq.in., this result is not surprising.

The only observations in London Clay other than those described by Cooling and Ward were published by *Skempton* in 1943. These were made on a single ring in a cast iron lining of a 12 ft. diameter tunnel at a depth of 109 ft. and they showed stresses in the lining corresponding to full overburden pressure. They were therefore similar to those reported by *Cooling* and *Ward* for tunnel No. 2 South. But it would be unjustifiable to attach too much significance to measurements on one ring only and I do not consider that the general conclusion, at present, can be other than that the average stresses correspond to about 70 per cent of full overburden, with variations of about $\pm$ 50 per cent on that figure (i.e. from 40 to 100 per cent of full overburden pressure).

(4b) Rock Tunnels

The stresses in rock surrounding tunnels or cavities have been the subject of a notable theoretical paper by *Terzaghi* and *Richart* (1952). Elastic theory was used to compute the stresses, and numerical solutions are presented for circular and elliptical tunnels and for spherical and spheroidal cavities. It is shown that the stresses depend to a very marked degree on the state of stress in the rock prior to excavation. If N be the ratio of the horizontal to vertical stress prior to excavation, then the following Table summarises the influence of this ratio on the horizontal stresses σ_h at the crown of a circular tunnel at a depth of H in rock of density γ

N	0.25	0.5	1.0	2.0
$\sigma_h/\gamma H$	-0.25	$+0.5$	$+2.0$	$+5.0$

Thus the stresses change from tensions where N is 0.25 (corresponding to normally consolidated material) to compressions which can be very large where N is greater than unity (as with heavily over-consolidated rocks or those which have been subjected to tectonic forces). The authors point out that since, in their opinion, reliable methods of determining N by physical data, such as the results of strain gauge measurements, are not available, the uncertainties in estimating the stresses around a tunnel are considerable; although in many instances the geological history of a rock formation limits the values of N to a certain range and hence an approximate estimate of the stresses can be made.

From France a number of interesting papers have been published of work, under the general direction of *Mayer*, concerning measurements of the "elastic" moduli of rocks in-situ; measurements which are significant for the design of concrete dams as well as for tunnel linings. The standard method used by the French engineers is that of carrying out a loading test on the rock in-situ by jacking on to a plate about 10 inches diameter against one wall of a gallery; the reaction being obtained from the opposite wall. The modulus is calculated from the observed relation between pressure and deformation, using classical elastic theory equations. The principles of this method were described by *Habib* (1950) who made laboratory jack tests on a mass of concrete, and compared the results with values of the modulus E obtained by various other means such as unconfined compression tests, and the velocity of sound. He also described an in-situ test on jointed quartzite at the site of the Tignes dam. In the tests on concrete all methods gave approximately the same value for E. In the rock, however, the jack test led to a value for E equal to only one-sixth of that determined on individual (unfissured) specimens, owing to the presence of joints or fissures in the natural rock.

Further in-situ jack tests were then reported by *Delarue* and *Mariotti* (1950) on rocks in Morocco. In accordance with the test results these authors were able to classify rocks into three broad groups:

(i) hard and compact rocks behaving to a first approximation as elastic materials;

(ii) fissured or jointed rocks exhibiting an important degree of compaction during the first cycle of loading but behaving more or less elastically after several cycles of loading, the value of E in this latter stage being far greater than in the compaction stage;

(iii) soft rocks showing large irreversible deformations under each cycle of loading.

A typical load deformation curve is given for each type of rock.

In a paper to the Conference, *Bernard* (7/1) reports more results of jack loading tests carried out in various rock formations encountered in a tunnel at Im'font in Morocco, and compares the results with tests carried out by measuring the diametral strains in test galleries subjected to internal water pressure. The two methods are in agreement as to the order of magnitude of the modulus but the differences in any particular rock are often considerable; the jack test usually giving the lower value. The author discusses the difficulties in interpretation, and draws the general conclusion that the simple jack test is adequate, providing that account is taken of its tendency to give too low values of E in fissured rocks, where the compaction during the first cycle of the loading test is probably excessive owing to the disturbance of the rock caused by excavating the tunnel. *Bernard* suggests that a reasonable approximation, for the purpose of designing tunnel linings, is to take the mean of the values of E derived from the first and the last cycle of loading in a jack test. The author also shows, by means of jack tests and gallery tests, the greatly increased value of E obtained by grouting a heavily fissured rock.

In connection with the linings for tunnels in rock that have to carry water under high pressures *Mayer* (7/8) presents to the Conference a description of a method for pre-stressing these linings by means of an expansive cement. The lining, which consists of pre-fabricated sections, is put in place and the annular space between the lining and the rock face is packed with aggregate. A special cement grout is then injected into the voids of the aggregate, this grout being one which, if unrestrained, swells with time. In the confined annular space, however, where swelling is practically impossible, pressures are built up and hence compression stresses are induced in the lining. These will balance the tension stresses which are set up in the lining by the pressure in the water in the tunnel. The grout is composed of 60% Portland cement, 20% blast furnace slag and 20% gypsum and *Mayer* reports measurements which show that pressure as high as 30 kg/sq.cm can be developed within a period of about one month. This process is one of considerable interest and should find important applications.

The foregoing papers are chiefly concerned with the measurement of E in rock and the design of linings in rock tunnels. In two papers by *Habib* and *Marchand* (1952) and by *Tincelin* (1952) an ingeneous method is described for measuring the stresses in the rock in the sides of a tunnel. This method has been used in an iron ore mine in Louraine, and consists of placing strain gauges in the rock face, then cutting a deep slot in the rock near the gauges observing the release of strain over the gauge length. A *Freyssinet* flat jack is next placed in the slot, and the pressure is noted which has to be applied to the jack to reduce the strains back to zero. This pressure is taken as a measure of the stresses in rock at the point under test.

Finally, in the field of rock tunnelling, *Caille* and *Barbedette* (7/2), in a Conference paper, give an account of a technique by which the construction of a tunnel in fissured water-bearing rock can be expedited by forming an "umbrella" or hood of grouted rock ahead of excavation. This treatment is described in terms of two jobs in which it has been successfully applied. The "umbrellas" were advanced up to 90 metres ahead of the working face, and grout pressures as high as 100 kg/sq.cm were used.

(5) Abutment Pressures Due to Soil Creep.

A particular type of earth pressure problem, and one of unusual interest, is discussed in a Conference paper by *Haefeli*, *Schaerer* and *Amberg* (7/6). In 1930 one of *Robert Maillart*'s beautiful reinforced concrete bridges was built over the river Landquart at Klosters. In 1938 cracks appeared in the structure and investigations were undertaken which proved that the trouble was caused by hill-creep of the soil on the left bank of the river, dragging past the abutment and hence pushing it towards the river. The average ground slope behind the left abutment is about 25° and the average rate of creep of the order 15 mm per year. Observations in deep pits indicated that the creep movement extended to a depth of at least 40 ft. As remedial measures, a strut was placed between the two abutments, the right abutment being strengthened, and a relieving arch built around the foundation of the supports of the left approach spans. These measures were described by *Mohr*, *Haefeli et al.* in 1947 together with the method used for estimating the thrust which would be developed in the strut. This method consisted of assuming that the passive pressure would be developed by the soil moving past the left abutment, and also that friction would be mobilised along the sides of the abutment. The thrust, in the strut was, in other words, taken to be that corresponding to the force required to push the left abutment into the soil—that being equivalent to the actual movement of soil past the abutment. In order to obtain an upper limit it was assumed that $\varphi = 45°$, which gave 1650 tons as the thrust. The strut was placed in position towards the end of 1944. By 1949 the thrust had built up to a value of about to 800 tons, the exact value depending upon the interpretation of the observations, and in the following 3 years there seems to have been little increase. It would be interesting if the authors could say to what value of 7 this thrust corresponds and how the calculated value of 7 agrees with test results on the soil. In general, information on the soil properties could usefully be increased so that the valuable data gained from the Landquart bridge could be more widely applied.

Conclusions

Any attempt at drawing general conclusions from the contribution since 1948, including those to the Conference, must be largely one of personal opinion. The most significant advances seem to me to be: the recognition of flexibility of sheet piling as a vital factor in the design of anchored sheet pile walls and in the interpretations of strut load observations in cofferdams; the further developments of reliable load gauges for measuring earth pressures on walls or tunnel linings; and the advances in our knowledge of rock tunnels.

Since 1948 there have been some notable field observations; in deep cofferdams in clay, in anchored sheet pile walls, in tunnel both in clay and rock, and on the pressures set up by hill creep.

Proposals for Discussion

In conclusion, I suggest that the following topics are those which will be most interesting for discussion.

(1) *Rowe*'s model tests on sheet pile walls show that the elastic extension of the tie rods and the usual small forward movement of the anchor are together sufficient to prevent any "arching" in sand, and the pressure distribution is of the classical form. Is there any field evidence which does not support this conclusion, and if there is, then what explanation is offered to account for the observations?

(2) There seem to be good reasons for considering seriously the method of design of sheet pile walls by the "free end" method coupled with reduction factors applied to the bending moments, as suggested by *Rowe* and *Terzaghi*. Yet this represents a departure from the standard method of design. More comparisons between the two methods, such as the comparisons made by *Packshaw* and *Lake*, would be useful, and the new method could well be discussed, both from the point of view of the physical principles on which it is based and of adapting it to drawing office use. The methods suggested by *Roisin* and *Verdeyen* could also be discussed.

(3) By taking into account the deformation of sheet piling prior to placing the struts, in a deep cofferdam in clay, *Skempton* and *Ward* have shown that the pressure distribution exerted by the clay on the sheet piling could have been of the classical form, in spite of the fact that the loads in the struts were practically independent of their depth. Is it possible that in earlier cases where "parabolic" or other similar distributions of loads have been reported as being in the struts, the actual earth pressure itself may have been linear with depth—or, at least, that the departure from the classical distribution was far less than was considered at the time? Also, is there any additional field data that can be added to the information in Fig. 2 concerning the centre of pressure in clay?

(4) The observations by *Cooling* and *Ward* indicate that in stiff clay the average pressure developed on a tunnel is about 60 or 70 per cent of the full overburden. It would be interesting to compare this result with observations in other types of clay, and with any theories concerning pressures on tunnels in clay.

(5) Under the general direction of *Mayer*, a valuable series of field tests have been carried out on the measurement of *Young*'s modulus of rock in situ, and on the stresses in rock surrounding a tunnel. The methods used are novel and could usefully be the subject of discussion. Comparisons with the results obtained in other countries would also be interesting.

(6) *Haefeli*, *Schaerer* and *Amberg* show that great pressures have been developed on the abutments of the Landquart bridge due to soil creep. Are there any other cases where pressures due to this cause have been measured, and under what conditions are such great pressures likely to be encountered? In other words, to what degree is the Landquart experience exceptional?

Propositions pour les discussions

Pour conclure, je propose que les discussions portent sur les sujets suivants:

1° *Rowe*, dans ses essais sur des palplanches de modèle réduit, constate que l'extension élastique des entraits et le faible déplacement avant de l'ancrage qui a lieu en général, sont dans l'ensemble suffisants pour empêcher l'effet de voûte dans le sable, la distribution de pression restant de forme classique. Rencontrons-nous dans la pratique des résultats qui contredisent cette théorie, et, dans l'affirmative, quelles raisons peuvent être avancées pour expliquer ces observations?

2° Il semble y avoir de bonnes raisons pour considérer sérieusement le calcul des rideaux de palplanches «libres au pied» en introduisant conjointement des facteurs de réduction appliqués aux moments, ainsi que l'ont suggéré *Rowe* et *Terzaghi*. Cependant ceci constitue une déviation de la pratique courante de calcul. D'autres comparaisons entre les deux méthodes, comme celles faites par *Packshaw* et *Lake*, seraient utiles, et la nouvelle méthode pourrait très bien être considerée, aussi bien du point de vue des principes physiques sur lesquels elle est basée, que sur celui des moyens de l'adapter à l'établissement des projets. Les méthodes de calcul suggérées par *Roisin* et *Verdeyen* pourraient aussi être discutées.

3° En tenant compte de la déformation subie par des palplanches avant la mise en place des étançons dans un profond batardeau fondé dans de l'argile, *Skempton* et *Ward* ont démontré que la répartition de la pression de l'argile sur les palplanches, peut être de forme classique, bien que la charge dans les étançons soit pour ainsi dire indépendante de leurs profondeurs. Est-il possible que, dans les cas antérieurs où une répartition parabolique ou semblable des charges a été observée dans les étançons, la poussée réelle de la terre ait pu être de forme classique, ou que, tout au moins, la différence avec la distribution classique ait été bien moins importante, que celle qui avait été envisagée? A-t-on recueilli d'autres résultats obtenus en pratique qui puissent être ajoutés aux renseignements, concernant le centre de pression dans l'argile, donnés dans la Fig. 2?

4° Les constatations de *Cooling* et de *Ward* montrent que dans de l'argile dure, la pression moyenne portant sur le revêtement d'un tunnel équivaut à peu près à 60 ou 70 pourcent de celle des terrains de couverture. Il serait intéressant de comparer ces résultats à des constatations dans d'autres genres d'argile et à tout autre théorie relative aux pressions sur les tunnels dans l'argile.

5° Sous la direction générale de *Mayer*, une intéressante série d'essais pratiques a été exécutée pour déterminer le module d'élasticité des roches en place et mesurer les contraintes régnant autour d'un tunnel en rocher. Les méthodes employées sont originales et pourraient constituer le sujet de discussions utiles, invitant en plus des comparaisons entre ces résultats et ceux obtenus dans d'autres pays.

6° *Haefeli*, *Schaerer* et *Amberg* ont montré que de fortes pressions se sont développées dans les butées du pont de Landquart, causées par le cheminement du sol. Avons-nous connaissance d'autres cas où des pressions de même provenance aient été mesurées? Dans quelles conditions, des pressions de cette importance ont-elles des chances de se produire? Autrement dit, jusqu'à quel point l'expérience de Landquart est-elle exceptionnelle?

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