the moment on the wall can be determined. If triangular intensity of pressure is assumed, also the magnitude of the earth pressure can be determined.

Up to now tests have been done with crushed stones of a grain size 35 - 75 mm and an angle of internal friction $\varphi = 40^o$. If the filling was brought on successively against the wall (see fig. 10), the pressure was $J_a = 0{,}22 \cdot \gamma \cdot h^2/2$. According to the friction theory it would have been $0{,}217 \gamma h^2/2$. The deflection of the wall was 0,60 mm at the upper edge and 0,20 mm at the lower one. The relation between the deflection at the upper edge and the height of the wall was then 1/3300.

Tests have also been made with load on the filling. If the measured moment on the wall is assumed to be caused by an uniform horizontal pressure on the wall the horizontal pressure was equal to 0,22 times the vertical load. When the load was taken away 60 - 70 % of the pressure caused by the load still remained.

Up to now all the tests have shown that elastic and plastic deflections of the size mentioned above are sufficient to reduce earth pressure at rest to active pressure.

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SUB-SECTION V b

EARTH PRESSURE AGAINST FLEXIBLE VERTICAL WALLS

V b 1 MEASUREMENT OF PRESSURE IN TIMBERING OF A TRENCH IN CLAY

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This paper describes some measurements of the pressure in the timbering of a deep excavation behind a retaining wall in the London Clay.

The plan and section of the wall are shown in figures 1a and b. The wall, which was about 40-ft high, was built in 1901-02. In 1920 serious movement of the wall was noticed and six counterforts were constructed behind the wall. Movement of the wall was thus almost arrested until 1939 when further movements occurred. It was decided to construct a continuous counterfort behind the wall joining four of the previous isolated counterforts and extending beyond them as shown in the plan. The new counterfort was to be 6-ft deeper than the existing wall foundations and was to be constructed in seven sections, each about 25-ft long carried out in the order shown in figure 1a.

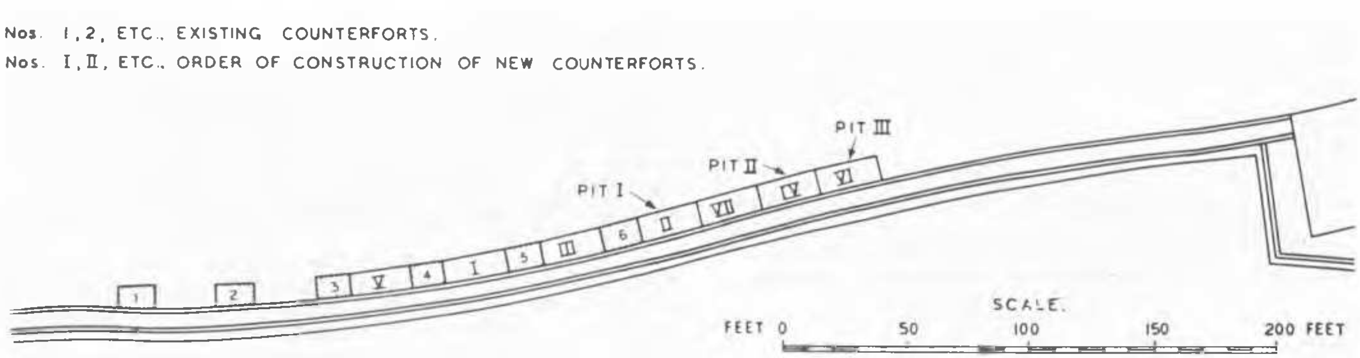
The opportunity was taken to measure the pressure in some of the timber struts supporting three of the excavations.

METHOD OF MEASUREMENT.

One of the conditions laid down by the Railway Company was that any work carried out should not interfere in any way with the Contractor's progress on the job. This ruled out the use of steel struts and elaborate pressure measuring devices.

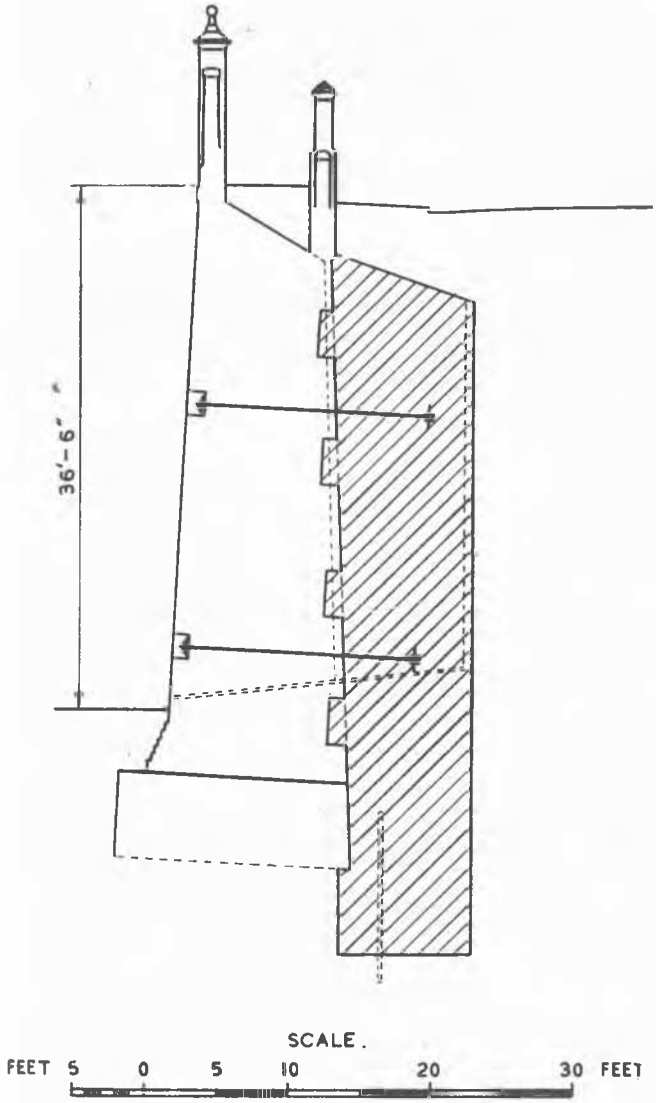
Preliminary laboratory tests with a surface strain gauge had shown that this method was of no use with timber struts since the drying tensions in the surface of the timber sometimes exceeded the compression due to the applied load. It was decided therefore to attempt to measure the reduction in length of a large length of the strut under applied load. The measurements were made over a gauge length of 5-ft with a micrometer reading to 0.01 mm.

The technique of measurement had been developed at the Building Research Station to measure the shrinkage of brickwork and was applied with one or two minor modifications to the present problem. The micrometer was



Plan of wall

FIG.1a

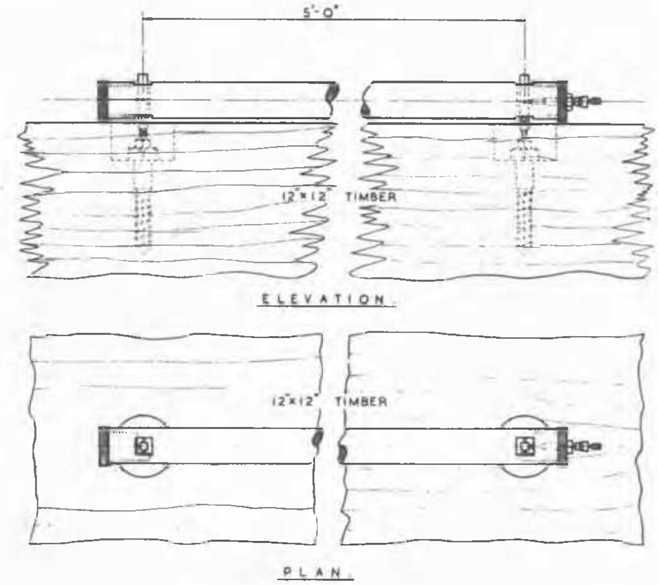


Section of wall showing counterforts
(cross-hatched)

FIG.1b

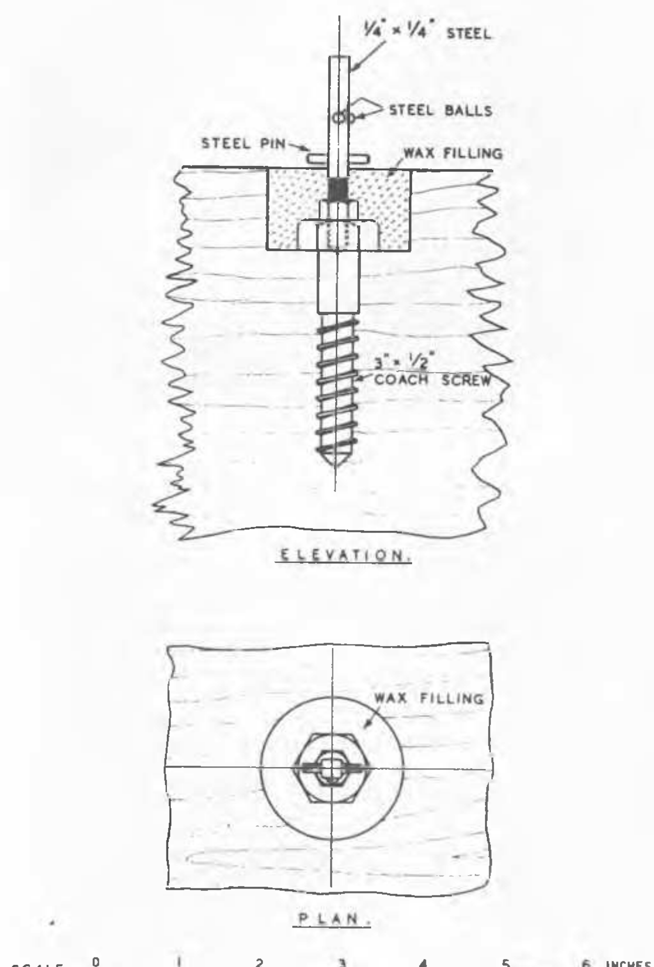
mounted in one end of a steel tube, in which were cut two slots, one near each end, to admit the measuring points. Three stainless steel plane surfaces were fixed in the tube in such a way that it always took up exactly the same position when these surfaces were in contact with the measuring points. The micrometer was then screwed up to a ball fixed on the measuring point, and any change in length of the gauge length was thus obtained by difference between successive micrometer readings. The arrangement is shown in figure 2a.

In order to make sure that the movement of the measuring points represented a fair mean for the whole cross-section of the strut, two 3" x $\frac{1}{2}$" coach screws were sunk into each side of the strut 5-ft apart and with their heads 1" below the surface of the timber. Steel measuring points with steel balls fixed in their sides were then screwed into the heads of the coach screws and left projecting above the surface of the strut. The hollow in the wood was then filled with molten paraffin wax. (See figure 2b). Preliminary tests with a strut in a testing machine indicated that, even with eccentric loads the deflection was



Measuring rod in position.

FIG.2 a



Details of measuring point.

FIG.2b

given nearly as accurately by the mean of the readings on any two opposite sides of the strut as by the mean of all four readings. In practice, therefore, measurements were only made on the top and bottom of the struts.

Measuring points were fitted to five struts before their installation in the excavation. These struts were used in three different excavations, thus giving three sets of pressure measurements. The points were protected from accidental damage by wooden blocks screwed to the strut. The struts are shown in position in the photograph, figure 3. Readings were taken before the strut was placed in position, at intervals during the excavation of the clay and back-filling of the pit with concrete, and after the removal of the strut. After removal from the last excavation the elastic modulus of each strut was measured in a testing machine. The load in the strut was thus calculated from the measured deflections.

FIG.3

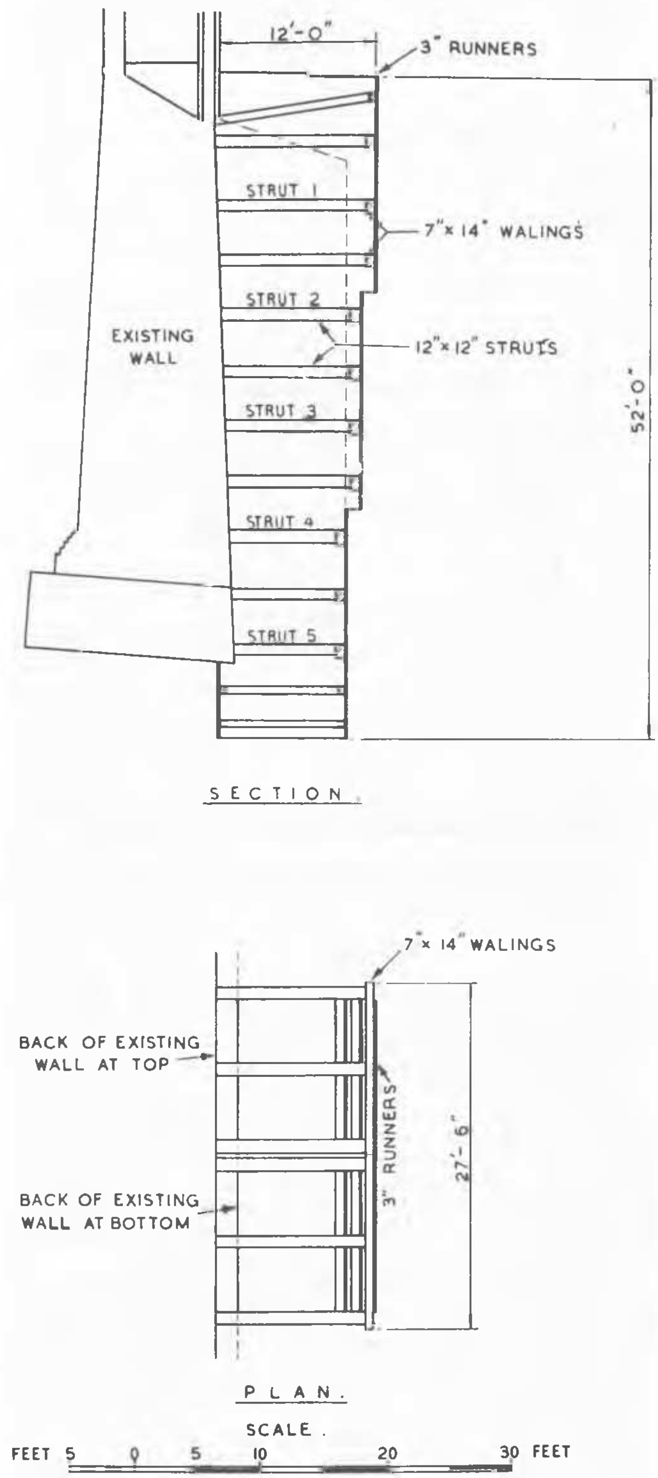
At first, a temperature correction was applied to the readings by taking a reading on an Invar-rod after each reading on a strut. To do this it was necessary to take the Invar-rod into the pit. For convenience in handling the rod it was removed from its heavy mounting, but it was then found to be so flexible that it gave misleading results. A series of tests in controlled temperatures established the fact that it was just as accurate to apply a temperature correction to the reading directly. The procedure was therefore adopted of laying a thermometer beside the measuring tube while each reading was taken, noting the temperature and making a correction to the reading.

It was found in practice that it was quite possible to repeat a reading to $\pm$ 0.01 mm even under the very severe winter weather conditions which were encountered in the first pit.

The estimate of load in a strut is probably correct to within $\pm$ 2-tons.

RESULTS OF MEASUREMENTS.

The timbering consisted of vertical runners supported by horizontal walings and struts. Each waling was about 12'-6" long and was supported by three struts, one near each end and one in the middle. The measurements were made on struts in the centre of a waling, five struts vertically below one another being chosen as shown in figure 4. The horizontal distance from centre to centre of the struts supporting each waling was 6-ft. The vertical distance between each frame of timbering was about 4-ft.

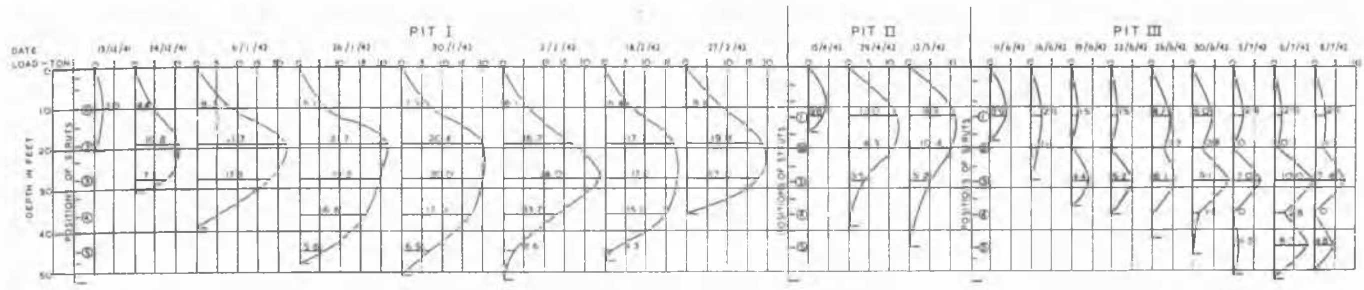


Details of timbering.

FIG.4

The measurements in Pit I were carried out between 13/12/41 and 18/2/42; those in Pit II between 15/4/42 and 12/5/42, and those in Pit III between 11/6/42 and 14/7/42. The forces in the struts at different depths of excavation are shown in figure 5. A sample sheet of readings (for Strut 2 Pit I) is shown in Table I. All readings were corrected to a standard temperature of 50° F. before calculating the load. The value of the modulus of elasticity for each strut is given in Table II.

In order to convert the load per strut



Forces in struts

FIG.5

TABLE I

TYPICAL SITE READINGS

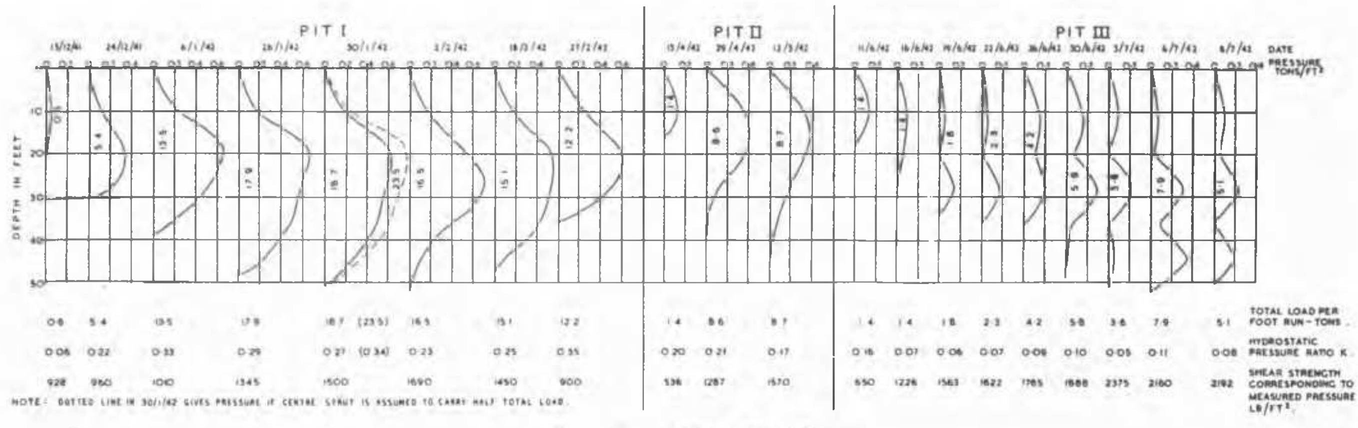
	Pit 1 Strut 2 Depth to centre 18"-9" below road level.									
Date	Temp. oF.	Corr.^{n}	Side A			Side B			Mean diff	Load Tons
			Reading	Corrected	Diff.	Reading	Corrected	Diff.		
13/12/41	52	+0.02	16.98	17.00	0	18.79	18.81	0	0	0
24/12/41	52	+0.02	16.99	17.01	+0.01	18.54	18.56	-0.25	-0.12	10.8
6/1 /42	33	-0.17	17.09	16.92	-0.08	18.58	18.41	-0.40	-0.24	21.7
26/1/42	32	-0.18	17.14	16.96	-0.04	18.55	18.37	-0.44	-0.24	21.7
30/1/42	38	-0.12	17.07	16.95	-0.05	18.53	18.41	-0.40	-0.225	20.4
2/2/42	35	-0.15	17.13	16.98	-0.02	18.61	18.46	-0.35	-0.185	16.7
18/2/42	36	-0.14	17.11	16.97	-0.03	18.60	18.46	-0.35	-0.19	17.2
27/2/42	37	-0.13	17.09	16.96	-0.04	18.55	18.42	-0.39	-0.21	19.0
13/3/42	41	-0.09	17.03	16.94	-0.06	18.83	18.74	-0.07	-0.065	Load zero
20/3/42	49	-0.01	16.98	16.97	-0.03	18.78	18.77	-0.04	-0.035	Load zero

TABLE II

DATA ON STRUTS				
Strut No	SIZE	Modulus of Elasticity lb/in^{2}x 10^6	Water-content of wood x)	
			SIDE A	SIDE B
1	12"x12"	1.38	30	33
2	12"x12"	2.14	27	26
3	12"x12"	2.06	33	39
4	12"x12"	2.15	37	30
5	12"x12"	2.05	30	29

x) Mean water-content of wood taken from hole bored for measuring point.

into a pressure distribution diagram it is necessary to make an assumption about the equivalent length of waling supported by each strut and the length of runner supported by each waling. It is reasonable to assume that each waling supports half the span of runner both above and below it. Two assumptions are possible with regard to the span of waling supported by each strut. The first is that the strut supports half the span, the second is to consider the waling as a beam uniformly loaded and continuous over two spans in which case the centre reaction is 5/8 of the total load. In figure 6 the pressure distribution calculated according to the latter assumption is given, but a typical curve based on the former assumption is given for purposes of comparison. In this figure the total earth pressure per foot run is given and also the hydrostatic pressure ratio based on a weight for the clay, of 121 lb/cu.ft. (Mean value of many tests, range 117-126 lb/cu.ft.).



Pressure distribution.

FIG.6

SOIL TESTS.

The excavations were made in the London Clay. This is a stiff-fissured clay of Eocene age, normally grey in colour but oxidised near the surface to brown. The liquid limit of the clay varied from 75 to 98 and the plastic limit from 27 to 32.

At this site the top 36-ft were brown. Water was met with at a depth of about 39-ft and was of course percolating through the fissures.

A large number of undisturbed soil samples were taken as excavation proceeded and the shear strength of the clay was measured by means of unconfined compression tests and also box shear tests.

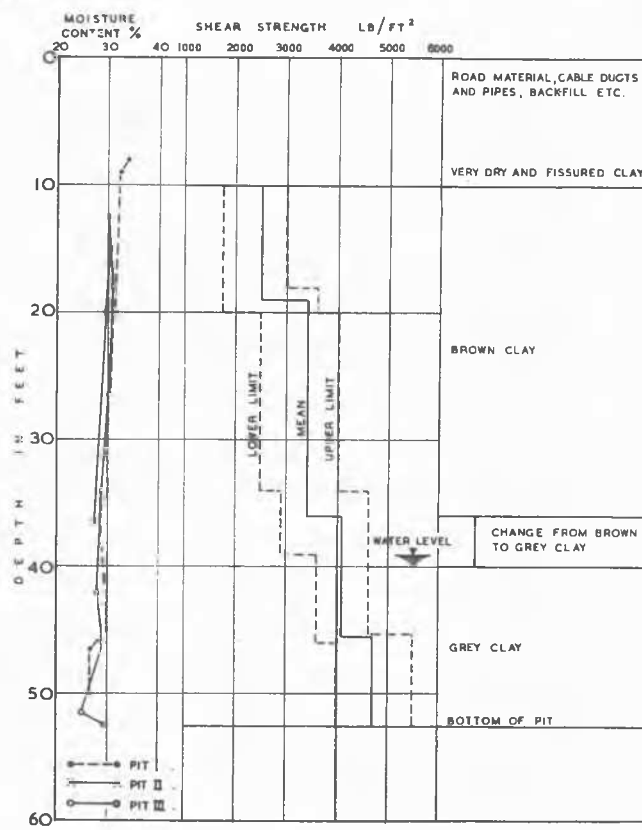
As is usual in the London Clay there was a considerable variation in the test results, but it was found possible to divide the profile into several zones for which fair average values of the shear strength could be chosen. These are given in figure 7. The variation of moisture content with depth is also shown in figure 7.

DISCUSSION.

As has been shown by previous measurements of this type, 1) the pressure distribution is not triangular, but approximates to a parabola. In Pit III this parabolic form is not so pronounced and the last two diagrams might be considered to approximate to a triangular distribution by anyone who wished to prove that this form was correct.

The magnitude of the measured pressure was much greater in Pit I than in the other two, and was greater in Pit II than Pit III. The measured shear strength did not differ greatly from one pit to the other, being highest in Pit II and lowest in Pit I. The moisture content, which can be considered to some extent an index of shear strength, was, as can be seen from figure 7, very much the same throughout the samples tested. However, since the shear strength did not increase from Pit I to Pit III as the pressure diminished, variation in shear strength cannot be considered the cause of this reduction in pressure.

Two other differences existed between the conditions of the three pits. The first was the time of year. Pit I was excavated in the very cold and wet weather of the winter of 1941-42. Pit II was made in the following Spring, while Pit III was dug in the summer. It is very probable that, although the shear strength of the body of the clay did not



Shear strength of clay.

FIG.7

vary, the strength in the fissures was higher in the summer than in the winter.

The second difference is that when the first counterfort was being constructed the whole wall was moving forward slowly. This movement was gradually arrested as one counterfort after another was completed. When the measurements were made in Pit III the wall was comparatively stable and it seems likely that the pressure on the timbering could be reduced by horizontal arching of the soil between the adjacent sections of the wall. The effect on Pit II would be expected to be between that for Pits I and III. The author believes that this arching action was an important factor in causing the difference in pressure, which is without doubt a real difference.

The measured strength of the clay is such that the vertical face of clay would be expected to stand without support, i.e. the calculated active pressure is zero. This result was more or less expected, since it is known that the behaviour of the London Clay in stability problems is usually controlled by the strength, not of the mass of the clay, but of the clay surfaces exposed in the fissures.

The shear strength of a homogeneous mass of clay which would exert the measured pressure is given in figure 6. This value increases with depth which suggests that the strength in the fissures is lowest near the surface; a conclusion which would accord with the known fact that the wall moved forward farthest at the top, thus allowing the fissures to open widest near the ground surface. Whether these figures can be taken as an estimate of the actual strength of the clay along a surface passing through the fissures is doubtful as the problem is complicated by the tension crack which can exist at the surface of the clay.

The whole wall was moving forward when the measurements were made in Pit I. An estimate of the average shear strength along any possible surface of failure can therefore be made by treating the problem as analogous to the stability of a vertical bank. This shear strength lies between 800 and 1.000 lb/sq.ft. depending on the assumptions made. The active pressure on the back of the wall, calculated from this shear strength is 22 to 27 tons, which is of the same order as the maximum pressure measured i.e. 18.7 tons to 23.5 tons according to the assumptions made.

ACKNOWLEDGMENTS.

Permission to carry out the work was given by the Chief Engineer of the L.M.S. Railway Company, and acknowledgment is made to Mr. Hope, District Engineer, for his helpful collaboration. The work was carried out under the supervision of Mr. L.F. Cooling and is published by permission of the Director of Building Research.

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Vb 2

LATERAL EARTH PRESSURE AS A PROBLEM OF DEFORMATION OR OF RUPTURE

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SUMMARY.

The paper outlines some of the results of an extensive program of research concerning lateral earth pressures which has been in progress at Princeton University since 1943 under the sponsorship of the Bureau of Yards and Docks, Navy Department, Washington, D.C.

Tests were performed at a 1:5 and a 1:8 model scale with flexible anchored bulkheads. Supplementary tests were carried out under pressures of prototype intensity in a specially designed laboratory device termed the "Lateral Earth Pressure Meter." The soils used included submerged sands, clays and mixtures of sand and clay. Clays were placed in a completely fluid condition and measurements were continued through all stages of consolidation. The effects of movements of the support were studied. The results of various types of laboratory shear strength tests and of conventional methods of lateral pressure computations were compared to the experimental values.

Conclusions are presented which suggest limitations and revisions of several conventional concepts. A possible method of approach to the design of flexible bulkheads is outlined.

INTRODUCTION

Coulomb's treatment (1776) of lateral earth pressures was based on the conditions prevailing at the moment of rupture along a definite surface in the backfill. Rankine's (1857) treatment of the matter was equivalent to an assumption that the soil is in a state of incipient failure without any motion being necessary to induce such failure. Rànkine originally related the lateral pressures to the angle of internal friction of the soil which was usually assumed as being equal to the angle of repose. Later modifications of Rankine's theory by Résal (1910) and others extended the analysis to clays and introduced the value of the "cohesion" which in determined from various procedures of laboratory strength tests carried to complete failure.

Terzaghi's large-scale model studies with rigid retaining walls and granular backfills led him to introduce the concept of earth pressure "at rest" which was shown to be somewhat greater than the "active" pressure. A slight outward movement of 0.1 per cent of the height of the retaining wall was shown to be sufficient to achieve the transition to an "active"