the calculated ones. This might indicate that the water, held in the sandfill, acts as a load on the foundation during a certain period.

DETERMINATION OF THE FINAL RATE OF SETTLEMENT FOR THE AREA YET TO BE RAISED, TAKING ACCOUNT OF THE SECULAR EFFECT.

For the area yet to be raised, the calculation of the rate of settlement is completed along similar lines as for the area already raised. Both layers overlying identical geological foundations, a hydrodynamic period of 15 years is assumed, this being the average for the former area. As an example the results for borehole 5 are shown in fig. 5.

NOTE.

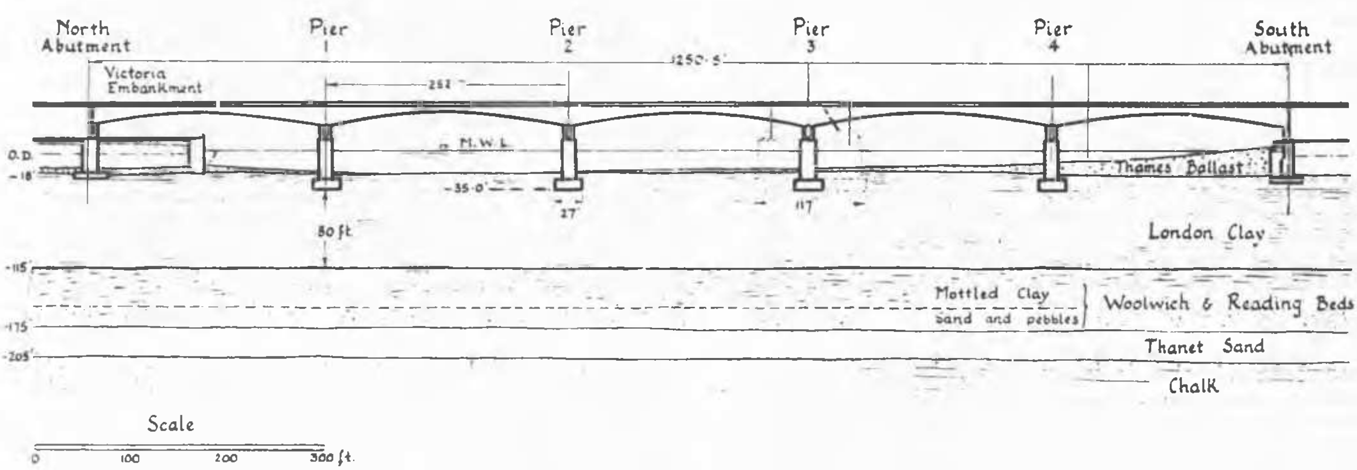
Notwithstanding the numerous approximations and assumptions it seems likely that the actual time settlement curves will approximate closely the calculated ones. The rate of "secular" settlement after the hydrodynamic period is mainly based on compression constants found from compression tests. The corresponding calculations are thus far checked by observations during a few years only. Not before prolonged observations in such areas on weak foundations are made, will it be possible to detect substantial discrepancies between theory and fact.

-o-o-o-o-o-o-

VI a 9

SETTLEMENT ANALYSIS OF WATERLOO BRIDGE

L.F. COOLING, M.Sc.
(Soil Mechanics Division, Building Research Station, D.S.I.R., Great Britain)



New Waterloo Bridge - Geological Section

FIG.1

I. SITE CONDITIONS AND SAMPLING OPERATIONS

The new Waterloo Bridge crosses the River Thames and the Victoria Embankment in London, from Wellington Street, off the Strand, to Waterloo Road. The geological conditions at the site, mainly interpreted from Geological Survey records, are shown in Fig. 1. The estimation of the amount of settlement of the bridge was mainly concerned with the properties of the London clay.

The pier foundations were excavated in an open cofferdam of steel sheet piling, and sampling operations were begun when only a few feet of clay were left to be removed. Six inch diameter lined boreholes were put down with a hand derrick to a depth of 40 feet below foundation level underneath each pier.

Subsidiary samples, consisting of pieces of disturbed clay taken from the auger, were taken at 2 ft. intervals of depth. Undisturbed samples, 4¼ inch diameter and 15 in. long, were taken at intervals of about 6 - 8 feet by means of a sampling tube driven into the clay by percussion; the sampling tube having an 'area ratio' of about 22%.

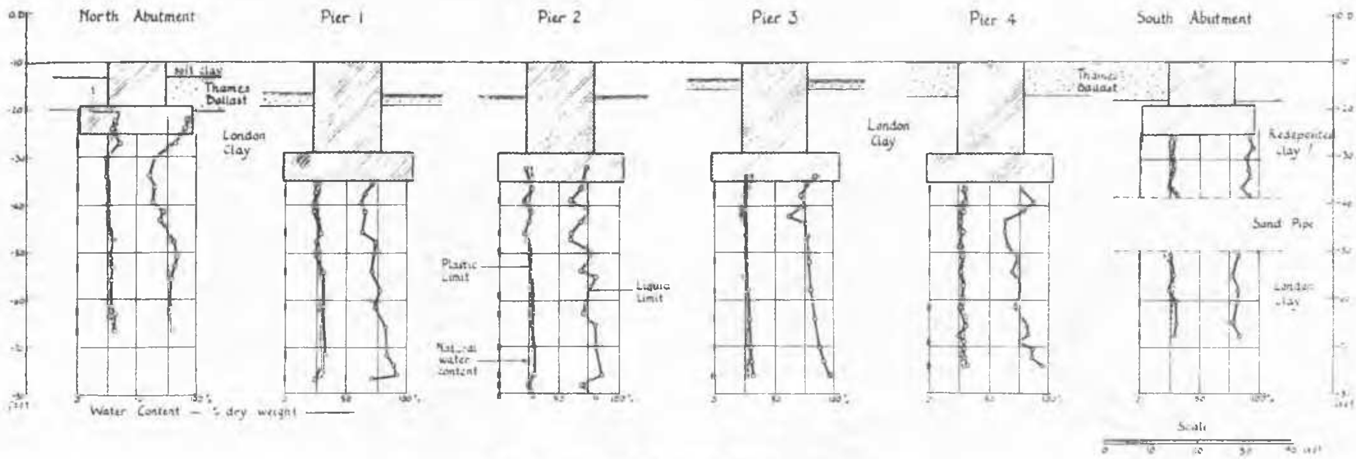
Out of the fifty undisturbed samples taken, it was found possible to prepare satisfactory test specimens from all but four, in spite of the friable nature of the clay.

II. LABORATORY TESTS

In the laboratory, index property tests were carried out on all samples on both the subsidiary samples and the core samples, in order to study the degree of variation in the London Clay stratum. 1) The mechanical properties of the clay were determined by tests on undisturbed samples only.

Index-Property Tests

The index properties used for this inves-



Variation of Index Properties

FIG.2

tigation were the natural water content, the liquid limit and the plastic limit. The variation of these properties with the depth in each of the boreholes is shown in Fig. 2, and it will be seen that throughout the natural water content is very near the plastic limit. There is a certain amount of variation, the clay in some places being more silty than in others. On the whole, however, the stratum was found to be much more nearly uniform than was expected, and the analysis showed that there was no significant difference between the average soil properties for the various piers.

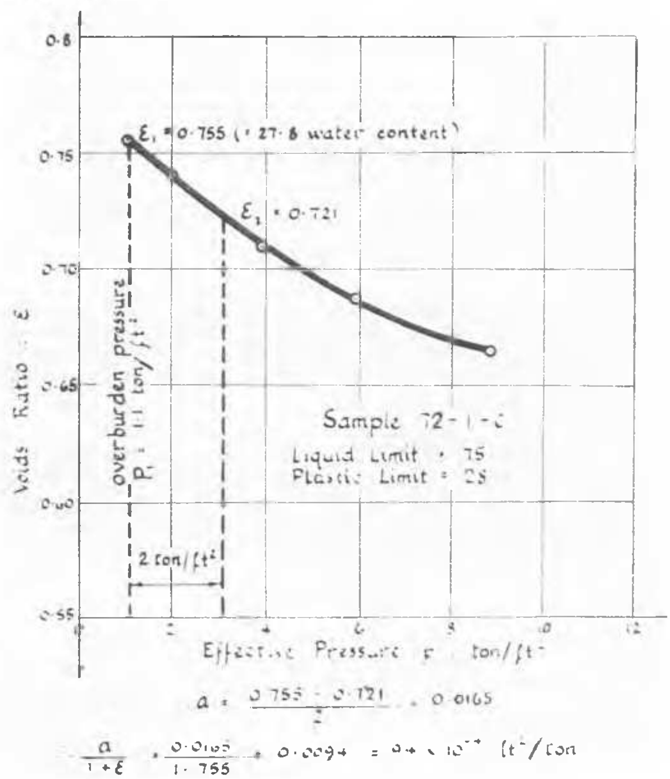
Consolidation Tests

Consolidation tests were carried out on 46 samples. The size of specimen used was 3 in. diameter and 7/8" thick and the initial pressure applied to the samples was 1 ton/sq.ft. The pressure was then increased successively to 2, 4, 6 and 9 tons/sq.ft. It was found that equilibrium was practically reached after each increment had been maintained for 24 hours, which was the usual time employed. In a few cases the increment was maintained for 72 hours and it was found that the average compression movement during the last 48 hours was only 5% of that for the first 24 hours. A typical p-ε curve ratio is shown in Fig. 3. The weighted means for the compressibility $\frac{a}{1+\varepsilon}$ of the soil in the different boreholes gave very similar values and for calculation purposes the compressibility was taken as 100 x 10^{-4} sq.ft./ton.

A typical time-consolidation curve is shown in Fig. 4, in which the degree of consolidation, μ , (calculated with reference to the compression after 24 hours) is plotted against the square root of the time after the application of the pressure increment. The theoretical curve with the same initial slope as the experimental curve is shown dotted and the simple method used for calculating the coefficient of consolidation (c) from the initial slope of the $\mu/\sqrt{t}$ curve is indicated. The error involved in neglecting 'secondary compression' was examined and found to be quite small: since this correction also involves a corresponding correction in the p-ε curve, the simple method indicated in Fig. 4, which does not involve correction of the p-ε curve, was adopted. Here again the mean values of c for the different boreholes were substantially the same, and were equal to 8.5 x 10^{-4} sq.in./min.

Compression Tests

Unconfined compression tests were carried out on cylindrical specimens prepared from the undisturbed cores in order to determine values of the compression modulus for estimating the 'immediate' settlement of the bridge. The method adopted allowed for plastic creep during the test, and precautions were taken to prevent drying out of the specimen by evaporation. The value obtained for the 'instantaneous' modulus was about 5,000 lbs/sq.in., but it was appreciated that for a fissured clay like London clay unconfined compression tests were not wholly satisfactory. Unfortunately no triaxial compression apparatus was available at the time, but subsequent work suggests that with a lateral pressure about equal to the overburden pressure the 'instantaneous' modulus would be higher than the value quoted above.

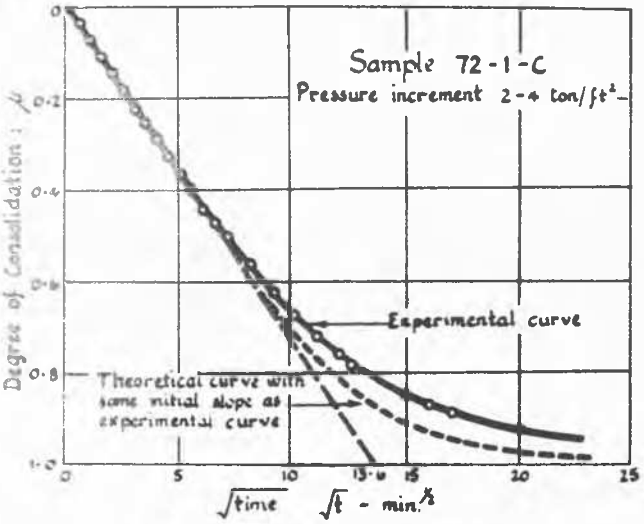


Typical Pressure - Voids Ratio Curve

FIG.3

III. THEORETICAL ESTIMATION OF SETTLEMENTS

The calculations concern the estimation



$$d = 0{\cdot}432 \text{ in.} \quad \frac{\mu}{\sqrt{t}} = \frac{1}{13{\cdot}6} \text{ min.}^{-1}$$

$$\frac{\mu}{\sqrt{t}} = 2\sqrt{\frac{c}{\pi d^2}}$$

$$\therefore c = \frac{\pi}{4} \times \frac{0{\cdot}432^2}{13{\cdot}6^2} = 8{\cdot}0 \times 10^{-4} \frac{\text{in}^2}{\text{min}}$$

Typical Time - Consolidation Curve

FIG.4

of (a) the 'immediate' settlement due to the elastic yield of the clay stratum beneath the loaded area and (b) the long term settlement due to consolidation. The loading details are given in Fig. 5.

Immediate Settlement

As a theoretical basis it may be assumed that a thick clay stratum behaves as an elastic medium, and in this problem the mean deflection (Δ) of a uniformly loaded area on the surface is given by the equation:

$$-\Delta = M\frac{p\sqrt{A}}{E}\left(1-\frac{1}{m^2}\right)$$

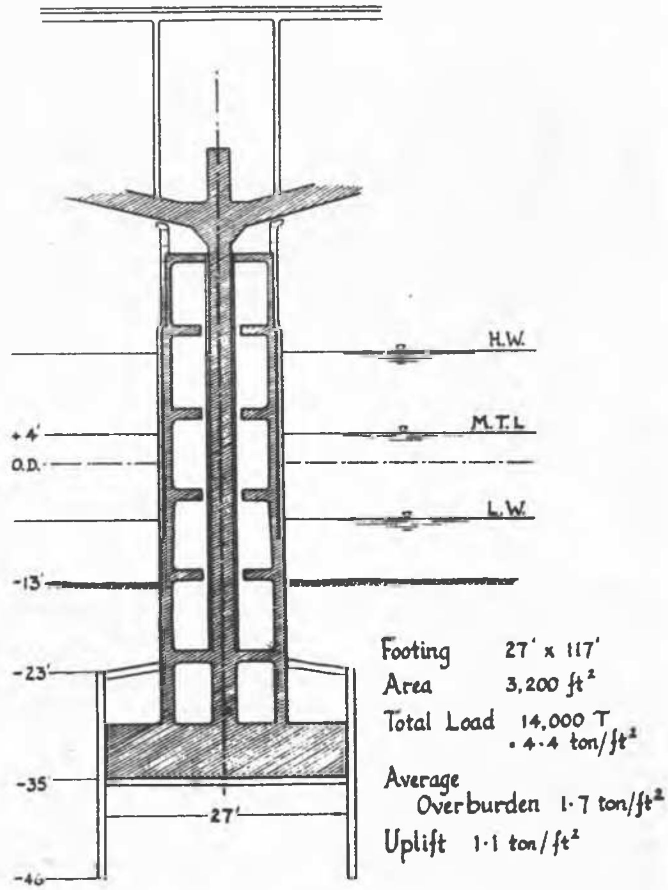
where p = pressure intensity, A = area,
E = modulus of elasticity,
$1/m$ = Poisson's ratio, is
and M=a factor depending on the shape of the loaded area.

However it has been pointed out 2) that estimates made on this basis are considerably greater than the movements which actually take place, especially in the case of large raft foundations located well below the surface. Since results were already available for the immediate settlement of similar piers in the London Clay at Chelsea Bridge (Buckton and Fereday) 3) a check on the method was made by estimating the theoretical movements obtained using the value for E (5000 lbs/sq. in) determined by the tests quoted above. This check calculation showed that the method considerably overestimated the movements.

The 'immediate' settlement of Waterloo Bridge was therefore estimated by direct analogy with Chelsea Bridge, using the observed movements of the latter as a basis. By this means the 'immediate' settlement was estimated to be about .75 in., and was the same for each pier.

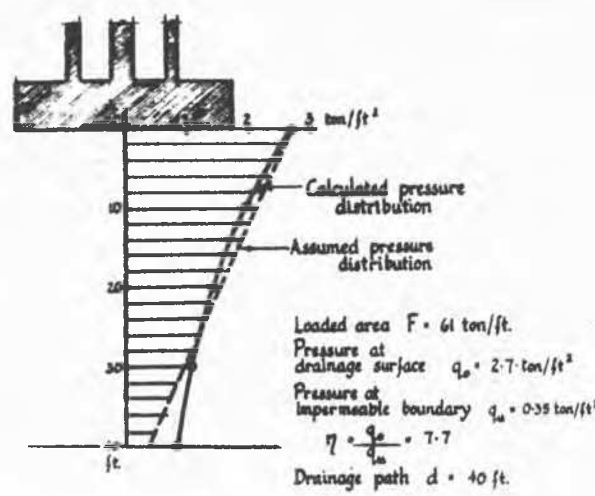
Consolidation Settlements

The conditions assumed for the calculation of the consolidation settlements are given in Fig. 6. The foundation slab was assumed to



Typical Pier

FIG.5



Consolidation - Settlement Calculations

FIG.6

be rigid, and the pressure distribution in the ground beneath the slab was worked out assuming that the pressure under a rigid footing is equal to the mean pressure under a flexible footing. The drainage was assumed to be through the concrete of the foundation only; this was justified because the concrete was found by laboratory tests to be at least five times as permeable as the clay.

As will be mentioned later, provision was to be made to measure the actual settlements of the piers relative to datum points 40 ft. below foundation level.

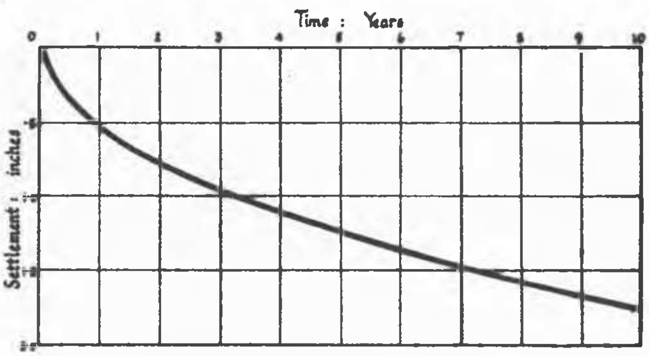
The consolidation settlement was therefore worked out for a 40 ft. trapezoidal loading with an impermeable boudary with an impermeable boudary at the bottom. The characteristics of the stratum were taken as the mean values determined in the tests, namely compressibility 0.01 sq.ft./ton and coefficient of consolidation 8.5 x 10^{-4}sq.in./min. or 3.1 sq.ft./year.

The progress of consolidation settlement with time calculated on this basis is given in Fig. 7, and again is the same for each pier.

IV. APPLICATION TO PRACTICAL PROBLEM

In the theoretical calculations above, the following simplifying assumptions were made: (a) that the clay would return to its original condition when the load became equal to the original overburden pressure; (b) that the consolidation settlement was due to a single increment of load equal to 2.7 tons/ sq.ft.

In the practical case, however, two features in particular were present which would seriously influence the settlement movements,



Time - Settlement Curve. Pier 4.

FIG.7

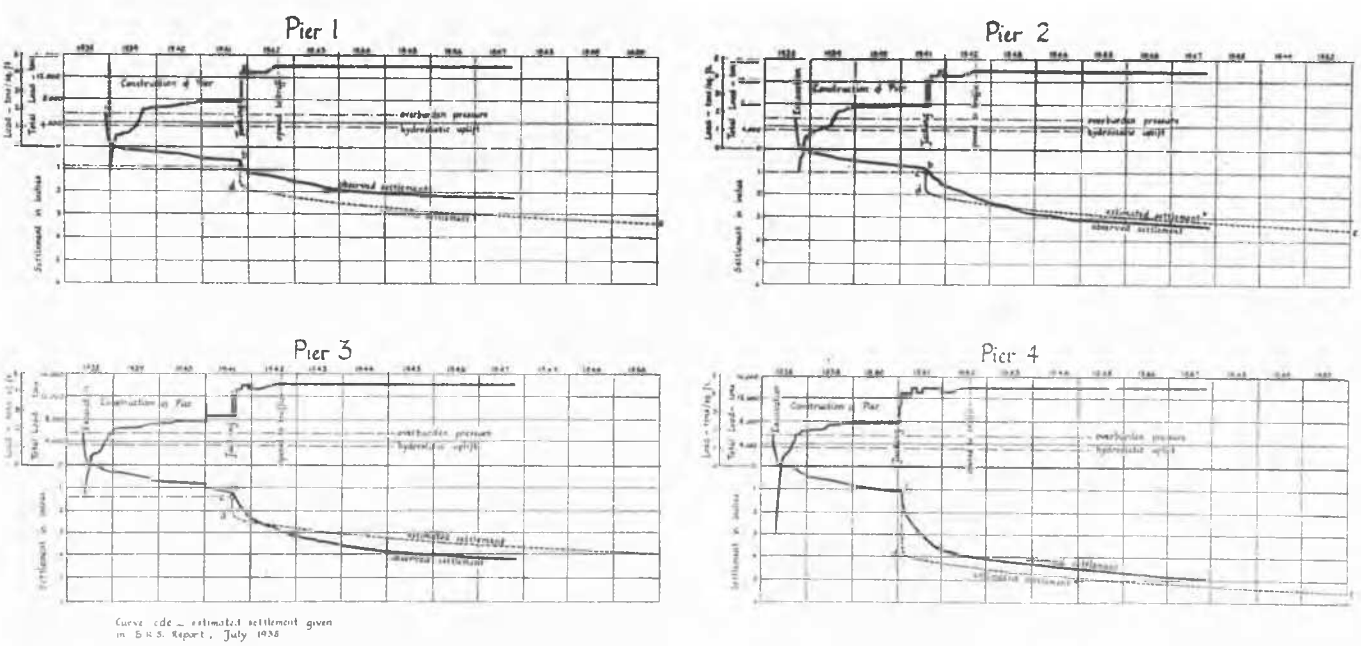
especially in the early stages. One was the rise of clay at the bottom of the cofferdams which occurred during excavation, and the other was the fact that the bottom ten feet of the sheet piling was to be left in and made integral with the foundation slab.

Rise of Clay in Cofferdams

Careful observations were taken by the Engineers of the rise of clay at foundation level during the excavation. Reference points were established along the centre line and near the sheet piling, and readings were begun when 5 - 10 ft. of clay were still to be removed. The resulting movements for the various piers are shown in Fig. 8.

In the case of Pier 4, which was the first to be constructed, the rise of the clay before casting the foundation was as much as 3 in. at the centre and 1 in. near the sheet piling. The rise took place while the clay was covered with a thin concrete mat (only 3 in.) which cracked and admitted water to the clay. In all subsequent piers and abutments, the excavatior of the last 5 ft. of clay and the placing of the concrete mat were carried out in sections and the mat was made 18 in. thick. This substantially reduced the rise of clay, which in all other piers was only about 1 in. at the centre and less than 1/3 in. near the sheeting.

On investigating the cause of these movements, only a small proportion could be accounted for by elastic recovery and normal swelling of the clay due to absorption of water. It was concluded that probably the rise was mainly due to the fissures in the clay being opened up by water pressure. The clay at Waterloo Bridge was of a highly fissured type, and it was thought that disturbance due to the excavation operations and heavy pile driving may have slightly opened up a network of fissures. Water then entered and opened them up still more. This idea was supported by the observation that in the cofferdam for the South Abutment, where the clay was of a fairly intact type, the rise was very much less than in the others.



Load Settlement Records

FIG.8

The fact that the rise of clay was different in the various cofferdams pointed to the probability that the subsequent settlements would vary accordingly. Had the constructional operations merely involved the excavation of the soil and its replacement by concrete, these movements could reasonably have been expected to be recovered soon after the structural load came on to the clay. However, in the problem concerned, the sheet piling around the foundations introduced complications, and its influence was assessed in the following manner.

Influence of the peripheral sheet piling on the probable settlement movements

As indicated in Fig. 5, the bottom ten feet of the sheet piling was left in and made integral with the reinforced concrete foundation slab. Under these conditions, it was considered that in the early stages of construction most of the structural load would be carried on the sheet piling by skin friction in the firm clay, and that the disturbed clay immediately beneath the foundation slab would be screened from load. The piling round the piers would support at least 4000 tons, and probably nearer 6,000 tons, by skin friction. Since the uplift pressure of water on the base was of the order of 3,500 tons, it was expected that movements would be small until the load reached between 7,500 tons and 9,500 tons. No attempt was made to assess the consolidation settlements under these conditions. Similarly it was impossible to assess the settlements of the north and south abutments owing to the large quantity of sheet piling helping to support them.

It will be seen from Fig. 8 that for about 30 months during the construction of the piers, the load was less than 7,500 tons. The load of the superstructure was then transferred to the piers in effectively one increment by jacking; this increased the load from 7,500 tons to 14,000 tons. The latter load would definitely overcome the skin friction and throw the load on to the clay under the slab. It was therefore expected that there would be rapid settlement due to closing of the fissures and elastic deformation of the clay. Both these processes would take a certain amount of time to complete, but since it was not possible to estimate their progress with time, the movements are assumed to coincide with the time of jacking. In Fig. 8, 'bc' represents the closing of the fissures which recovered the original level; 'cd' the 'immediate' settlement due to elastic deformation; the progress of the dotted curve 'de' represents the calculated consolidation settlement to date.

V. COMPARISON OF ACTUAL SETTLEMENTS WITH COMPUTED VALUES

The actual settlement of each pier was measured with reference to a datum point consisting of a 1¼" pipe firmly embedded in the clay about 40ft. below foundation level. The datum pipe was projected by a 2 in. sleeve pipe, and the two were carried upward through the pier construction to an accessible point. The outer sleeve pipe was fixed in the foundation slab, and the relative movement between the top of the datum pipe and the sleeve pipe was measured by means of a micrometer dial reading to 1/1000". Each pier was fitted with one such measuring device, and the results of the observations are shown by the full lines in Fig. 8.

Comparison of the actual and computed settlements over a period of six years shows a degree of agreement which is considered reasonable in view of the complicated nature of the foundation problem.

VI. ACKNOWLEDGEMENTS

The observations of the settlements of the bridge were taken by the consulting engineers, Messrs. Rendel, Palmer and Tritton, and by the Chief Engineers Department of the London County Council, to both of whom the author is indebted for permission to publish. The remainder of the work described in the paper was carried out at the Building Research Station of the Department of Scientific and Industrial Research, and is published by permission of the Director of Building Research. The author also wishes to acknowledge the helpful collaboration of his colleagues in the Soil Mechanics Division of the Building Research Station, in particular Mr. A.W. Skempton, M.Sc., A.M.I.C.E.

REFERENCES

1) "A Laboratory Study of London Clay", by L.F. Cooling and A.W. Skempton, J.Inst. of C.E., Vol.17, No. 3., 1942.
2) "Settlement of Structures" by K. Terzaghi, Paper F 16, Proc.Int.Conf. on Soil Mechanics, Vol. III, Harvard, 1936.
3) "The Reconstruction of Chelsea Bridge" Buckton and Fereday. J. Inst. C.E. Jan.1938 Vol. 3. (1937-38).

-o-o-o-o-o-o-