LABORATORY AND FIELD CONSOLIDATION OF SOFT CLAY

MESURES EN LABORATOIRE ET SUR LE TERRAIN DU TASSEMENT DU A LA CONSOLIDATION SUR ARGILE TENDRE

ПОЛЕВЫЕ И ЛАБОРАТОРНЫЕ ИССЛЕДОВАНИЯ КОНСОЛИДАЦИИ МЯГКОЙ ГЛИНЫ

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SYNOPSIS.- Two full scale embankments representing two-dimensional (plane strain) and three-dimensional (axisymmetric) conditions were constructed on soft clay deposits. The degree of consolidation for each test embankment was determined from field measurements of excess pore water pressure and compression. It was compared with the consolidation predicted on the basis of laboratory test results. The results of one-dimensional laboratory tests used with one-dimensional consolidation theories did not predict the actual consolidation accurately. For the axisymmetric case, however, it was possible to predict the actual rate of consolidation fairly well if three-dimensional theories were used and the anisotropic nature of the soil was taken into account. For the two-dimensional case, it was not possible to predict the actual primary consolidation because of an extensive amount of secondary consolidation that appeared to occur.

I.- INTRODUCTION

In Southeast Asia there exist many deposits of soft marine clay which present problems in settlement and stability. It is of general interest, therefore, to determine the time required for primary consolidation to occur both from the standpoint of settlement analysis and the prediction of pore pressures for use in stability computations.

The consolidation characteristics of the soil are generally assessed from the results of one-dimensional laboratory consolidation tests and the test results are then used together with one-dimensional consolidation theories. In reality, however, most actual problems are either two-or three-dimensional. Furthermore, considerable differences may exist between field and laboratory conditions, particularly, in terms of the macroscopic soil structure such as fissures, etc.

This paper considers the primary consolidation of full scale test embankments and compares the behavior predicted from laboratory tests with the observed behavior in the field. Two embankments representing both two-and three-dimensional conditions are considered. In both cases, 3 m high embankments were instrumented and constructed on layers of soft and weathered clay extending to a depth of approximately 10 m. The degree of consolidation was determined on the basis of the measured pore pressures and settlements and compared with the consolidation predicted from laboratory results.

II.- METHODS OF EVALUATING CONSOLIDATION RESULTS

The most common method of determining the rate of consolidation is to determine a value of the coefficient of consolidation, c_v, from laboratory tests and to then apply this value in the Terzaghi consolidation theory. The values of c_v are commonly determined from consideration of the geometry of the settlement-time curve using the Casagrande curve fitting method or the Taylor curve fitting method (Lambe and Whitman, 1969). In the Casagrande method the observed settlement-time curve is matched to the theoretical curve at the point corresponding to 50% consolidation, whereas in the Taylor method the curves are matched at a time corresponding to 90% consolidation.

In both methods almost all of the primary consolidation must have occurred before the value of c_v can be determined. However, in some instances such as sensitive clay, the shape of the settlement-time curve may sometimes not be suitable for use with either of these two methods. Also, in cases where full scale structures are being observed, it is frequently impractical to monitor the response for a time period sufficiently long to allow 90% consolidation to occur.

Scott (1961) determined the degrees of consolidation, U(t) and U(Nt), corresponding to times, t and Nt, where N is any number > 1.0. Scott presented a solution for the consolidation ratio U(t)/U(Nt) as a function of the time factor, T,

$$T = c_v t/H^2 \tag{1}$$

where H is the length of the drainage path.

The consolidation ratio U(t)/U(Nt) can be determined by the ratio of the soil compression occurring at times, t and Nt. Considering various values of t and N several values of c_v can be computed and adjustments made until a uniform value of c_v for all times is obtained.

The application of one-dimensional consolidation theory to actual field conditions is valid only for a loaded area of infinite extent. Davis and Poulos (1972) obtained solutions for two-and three-dimensional boundary conditions. The solutions are presented in the form of charts showing the percent consolidation as functions of the time factor for various conditions of drainage. They also obtained solutions for anisotropic soil wherein the horizontal permeability is different from the vertical permeability.

III.- COMPARISON OF LABORATORY AND THREE-DIMENSIONAL FIELD RESULTS (THA CHANG BRIDGE)

Site Description

Construction of the Tha Chang Bridge over the Chao Phraya River at Bangkok entailed the construction of an embankment approximately 3 m high at one of the approaches. As part of this, four test embankments were constructed to investigate the effectiveness of vertical sand drains that had been installed to accelerate settlement. A complete description of the testing program and analysis of the results is presented by Ciridon (1972).

The soil profile and layout of the instrumentation beneath one of the embankments is shown in Fig. 1. The soil was basically a very soft to soft silty clay to a depth of about 18 m underlain by stiff clay. A blanket of sand about 1 m deep had been placed over the entire site sometime before construction of the test area, but all pore pressures due to this blanket had dissipated before the test loads were constructed. The test load was constructed by filling a 6 m square wooden box to a depth of 3 m with sand having a density of 1.92 ton/cu.m. All loading was accomplished within a 12 hr period.

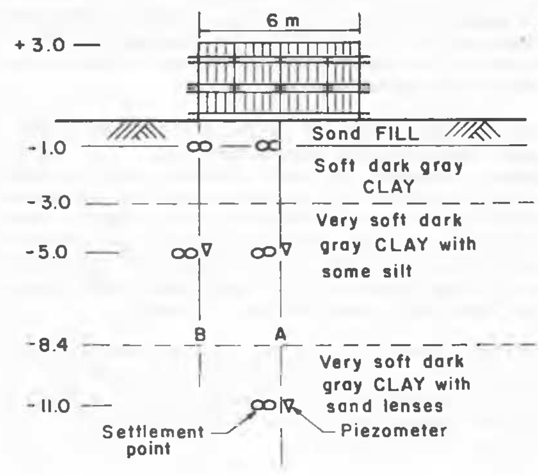

FIG. 1. TEST EMBANKMENT AT THA CHANG BRIDGE

Settlement points were installed at depths of 1 m, 5 m, and 11 m along the center line (line A) and depths of 1 m and 5 m along a line at the edge (line B). A settlement point consisted of an auger head which was screwed into the ground to which a rod was attached. A separate outer pipe eliminated friction along the inner rod. The movement of each point was determined from the elevation of the top of the inner rod.

Bishop type piezometers were installed at depths of 5 m along lines A and B and 11 m at point A. They were installed by pushing them into the soil by means of a special attachment designed to fit the piezometer on the drill rod. The rod could be withdrawn leaving the piezometer in the ground, and special sealing was not necessary, therefore, because as soon as the rod was withdrawn the soft clay collapsed around the piezometer. The pressure was read on a mercury manometer.

Field Results

The compression of the upper layer of clay could be determined from the settlement points at 1 m and 5 m. Similarly, the compression of the lower layer and the total 10 m of clay could be determined. The compression as a function of time is shown in Fig. 2. The settlement point at 5 m along line A exhibited anomolous results, and consequently, the results shown in Fig. 2 are presented for the settlement points along line B. The 11 m settlement was taken from line A.

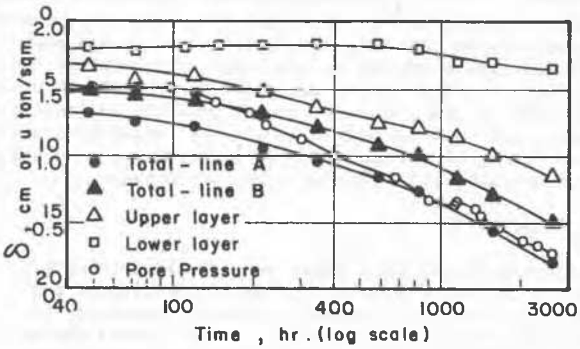

FIG. 2. COMPRESSION OF CLAY AND EXCESS PORE PRESSURE (THA CHANG)

The pore pressures measured at 5 m are also shown in Fig. 2. The pore pressure measured at 11 m was negligible during the entire time of the experiment indicating drainage at or above that depth as well as at the surface (i.e., double drainage).

Scott's method of evaluating the consolidation coefficient was applied to the curves shown in Fig. 2, and the results are shown in Table I. It can be seen that relatively good agreement was obtained between the values of c_v from all curves. The use of Scott's method entails the selection of an initial compression which is subtracted from each reading so as to yield the same value of c_v at all values of t. The values of initial compression that were used to determine c_v from each curve are also shown in Table I.

The initial compression may represent somewhat the instantaneous elastic compression of the soil. The values shown in Table I agree roughly with observed values of instantaneous compression (Ciridon, 1972).

The degree of consolidation as determined by Eq. 2 is shown in Fig. 3.

$$U = \frac{\delta - \delta_i}{\delta_f} \quad (2)$$

Table I - c_v and δ_i from Scott's Method of Curve Fitting Applied to Field Results (Tha Chang Bridge)

Layer	c_v cm^2/sec	δ_i cm
1-5 m	360×10^{-4}	1.5
5-11 m	360×10^{-4}	0.0
1-11 m Line A	258×10^{-4}	4.0
1-11 m Line B	263×10^{-4}	2.3

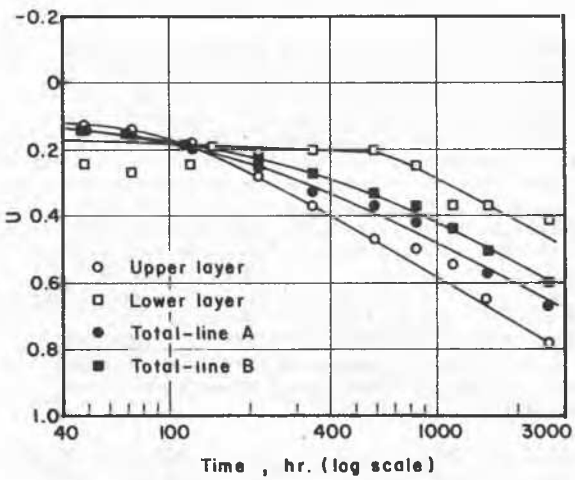

FIG. 3. CONSOLIDATION BASED ON MEASURED SETTLEMENTS (THA CHANG)

where δ = measured compression,
δ_i is the initial settlement, and
δ_f is the final compression of the clay layer computed using Terzaghi's equation.

Good agreement exists between the curves for the total thickness of the clay layer.

Laboratory Results

Standard laboratory consolidation tests were performed on undisturbed samples taken from various depths beneath the embankment. The consolidation coefficient was determined using the curve fitting methods proposed by Casagrande (log t), Taylor ($\sqrt{t}$), and Scott [U(t)/U(Nt)]. The results are presented in Fig. 4. Since c_v varies with the magnitude of the load applied, those values shown in Fig. 4 are for the average loading applied to the soil by the embankment.

It can be seen that the values of c_v are nearly the same for all three curve fitting methods within the upper 6 m. However, in the lower layer considerable scatter exists, probably because more sand and silt is present at the lower depths. The sand content was probably not uniformly distributed, and some variation undoubtedly existed between the different soil

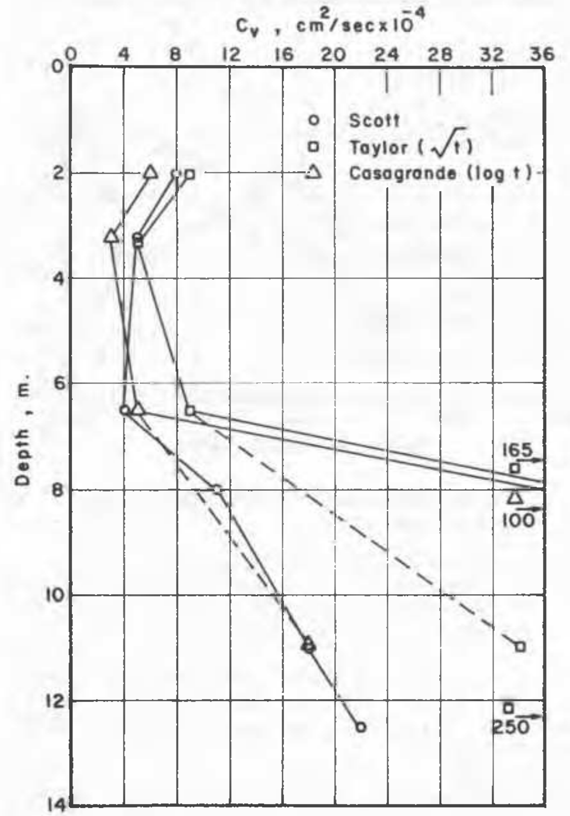

FIG. 4. LABORATORY VALUES OF c_v (THA CHANG)

samples. Scott s method gives the lowest value and exhibited the smallest amount of scatter.

Comparison of Field and Laboratory Results

A comparison of the degree of consolidation determined by several different methods is shown in Fig. 5. Curve 1 represents the consolidation determined using $c_v = 5 \times 10^{-4}$ cm^2/sec for the upper layer and 14×10^{-4} cm^2/sec for the lower layer. These values agree with values obtained using Scott's method of curve fitting. The curves obtained using values obtained by Casagrande's method and Taylor's method do not differ greatly from curve 1 and therefore are not shown. The consolidation of the total layer was computed using Eq. 3 which assumes that consolidation of the upper and lower layers occur independently of each other.

$$U = \frac{U_1 + r\,U_2}{1 + r} \qquad (3)$$

where U is the consolidation of the total layer,
U_1 is the consolidation of the upper layer,
U_2 is the consolidation of the lower layer, and
r is the ratio of the final compression of the lower layer to the final compression of the upper layer.

Curve 3 was computed from the equation

$$U = 1 - \frac{u}{u_i} \qquad (4)$$

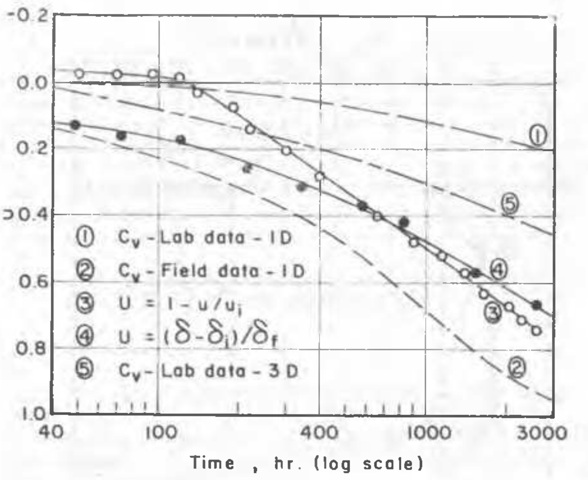

FIG. 5. COMPARISON OF CONSOLIDATION FROM VARIOUS METHODS (THA CHANG)

where u is the measured pore pressure,
u_i is the initial pore pressure, taken as the point at which the pore pressure first levels off.

It can be seen that for times greater than 300 hours curve 3 agrees quite well with curve 4 showing the consolidation determined from measured values of compression (Fig. 3).

The solution given by Davis and Poulos (1972) for the three-dimensional case was used to determine curve 5. The values of c_v used therein were the same as those used for curve 1. It is evident that, whereas the application of the three-dimensional solution gives somewhat better agreement between predicted and observed values of U, there still exists some difference. The reason for this is probably the anisotropy of the soil.

Duangkhae (1970) observed that for Bangkok Clay, the horizontal permeability may be considerably greater than the vertical permeability. Davis and Poulos (1970) show that the time required to reach 50% consolidation in an isotropic soil is about 4 times longer than that for a soil with the same vertical permeability but a horizontal permeability ten times greater. This is in good agreement with the results shown in Fig. 5.

IV.- COMPARISON OF LABORATORY AND TWO-DIMENSIONAL FIELD RESULTS (AIT, RANGSIT)

Site Description

The development of the new campus at Rangsit, 40 km north of Bangkok entailed the construction of a dike around the entire 400 acre site. Due to stability and settlement problems involved in constructing a large embankment on soft clay, an 80 m long test section was constructed and fully instrumented to provide comprehensive data on stability and settlement for the final design. A complete description of the entire site and instrumentation is given by Moh et al (1972). The profile of the test embankment is shown in Fig. 6. Much more instrumentation than that shown in Fig. 6 was included but only the instrumen-

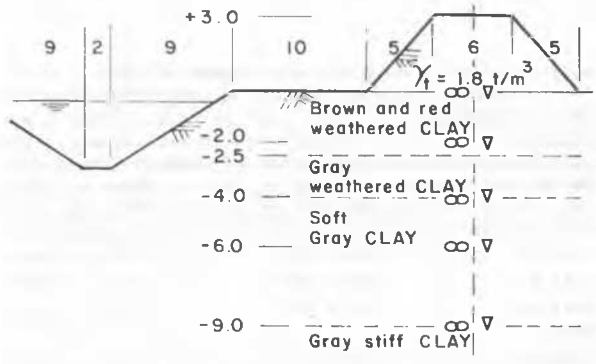

FIG. 6. TEST EMBANKMENT AT AIT, RANGSIT

tation which is of consequence to this paper is shown therein.

The soil at this site consisted primarily of three main zones, the weathered, soft and stiff clay. The soft clay is a fairly homogeneous deposit with no apparent stratification except for occasional very thin silt seams and slickensides. The weathering of the top few meters of the soft clay resulted in a surface crust with vertical cracks to 1.5 m below the ground surface. From this depth to approximately 2.7 m numerous small vertical holes were found to exist at the test site. Actual weathering of the soft clay at Rangsit appears to exist to depths in excess of 5 m (Moh et al, 1972). The stiff clay is an overconsolidated deposit (OCR > 5) which is quite heavily fissured with layers of coarse sand. Its relatively high strength and low compressibility make it comparatively unimportant with regard to settlement and stability.

The settlement points used in this investigation were the same as those used at the Tha Chang bridge site. Bishop type piezometers were inserted in sand bags, placed in bore holes and sealed with bentonite. The boreholes were backfilled with compacted clay that had been removed during drilling.

Field Results

Compression of the upper and lower layers (0-4 m and 4-9 m) and the entire thickness of weathered and soft clay (0-9 m) are shown in Fig. 7. To account for the duration of the construction period (2 weeks) the time was arbitrarily measured from the middle of the construction period, and the end of construction occurred at the end of 7 days.

Scott's method of curve fitting to determine c_v was applied to the curves shown in Fig. 7 and the results are shown in Table II. Because of the impermeability of the compacted clay from which the embankment was constructed, the layer was assumed to have only single drainage to the bottom. Fairly uniform results were obtained for the values of c_v thus obtained (Table II).

The degree of consolidation determined from Eq. 2 is shown in Fig. 8 for the two zones as well as the total layer. It can be seen that according to this method of determining the consolidation, only a small amount of primary consolidation had occurred by the end of the observation period.

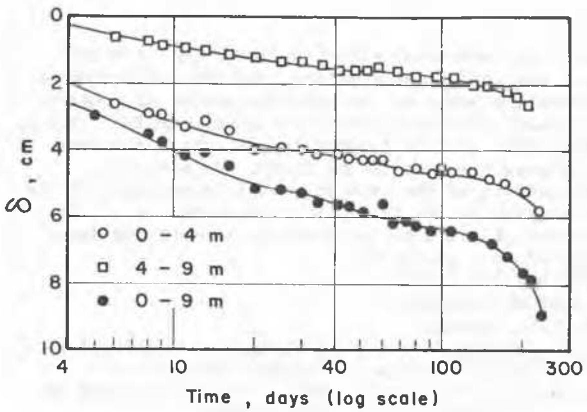

FIG. 7. COMPRESSION OF CLAY (AIT, RANGSIT)

Table II - c_v and δ_i from Scott's Method of Curve Fitting Applied to Field Results (AIT, Rangsit)

Layer	c_v cm^2/sec	δ_i cm
0-4 m	1600×10^{-4}	1.3
4-9 m	1480×10^{-4}	0.3
0-9 m	1740×10^{-4}	1.7

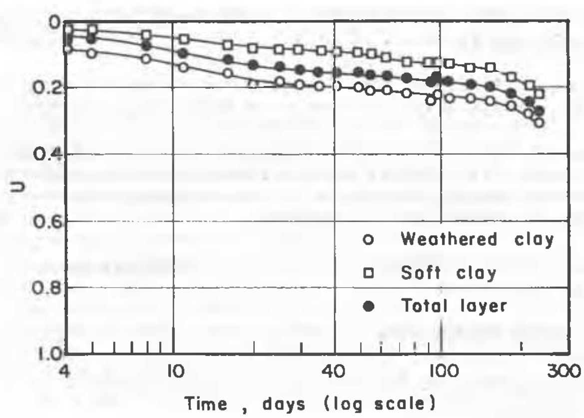

FIG. 8. CONSOLIDATION BASED ON MEASURED SETTLEMENTS (AIT, RANGSIT)

The consolidation determined from Eq. 4 is shown in Fig. 9. The initial values of pore pressure were determined by assuming that the pore pressure response was equal to the change in deviatoric normal stress (Nelson, 1972). This value was used instead of the maximum pore pressure response because of evidence that drainage had occurred in the upper zone during construction (Moh et al, 1972). However, at the 6 m depth, the pore pressure response indicated that little or no drainage had occurred in the soft clay layer during construction.

It can be seen from Fig. 9 that 100% primary consolidation had occurred throughout almost the entire depth by the end of approximately 70 days. However, after about 100 days the pore pressure exhibited a sudden increase coincidental with an increase in the rate of settlement as seen in Fig. 7. It is believed that this further increase in settlement resulted from local yielding caused by a collapse of the soil structure. This will be discussed later.

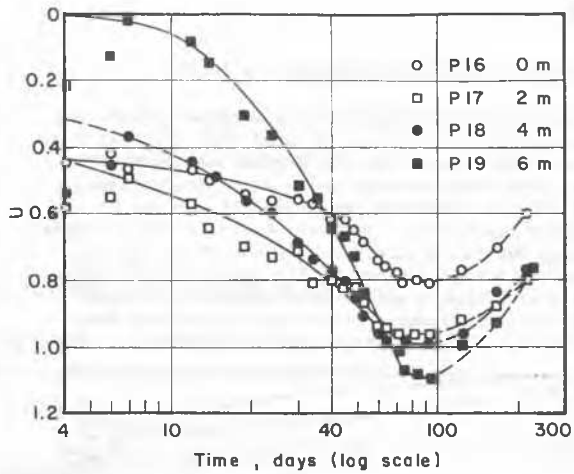

FIG. 9. CONSOLIDATION BASED ON MEASURED PORE PRESSURES

Laboratory Results

The results of laboratory oedometer tests performed on undisturbed samples taken from beneath the test embankment were reported by Kangsasitiam (1970). Values of c_v were determined only on the basis of Taylor and Casagrande curve fitting methods, and are shown as a function of depth in Fig. 10. Except for a few points at 2 m fairly uniform results were obtained throughout the entire layer.

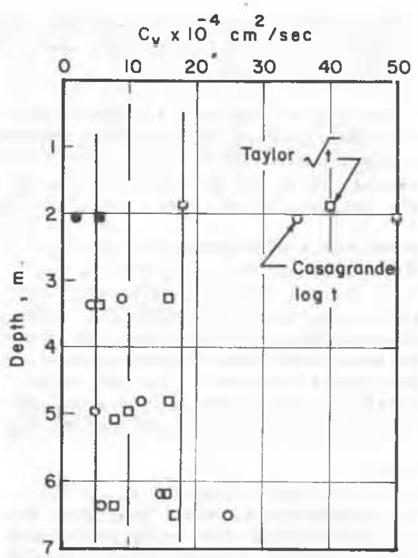

FIG. 10. LABORATORY VALUES OF c_v

Comparison of Field and Laboratory Results

The consolidation determined by various methods are shown in Fig. 11. It can be seen that for this case the one-dimensional and two-dimensional solutions give quite close results which agree fairly closely with the consolidation determined on a basis of observed settlement. However, curves 1 and 2 representing the field determined values of c_v and the consolidation determined on the basis of measured pore pressure, give much faster consolidation and indicate that primary consolidation had been completed in a period of less than 100 days.

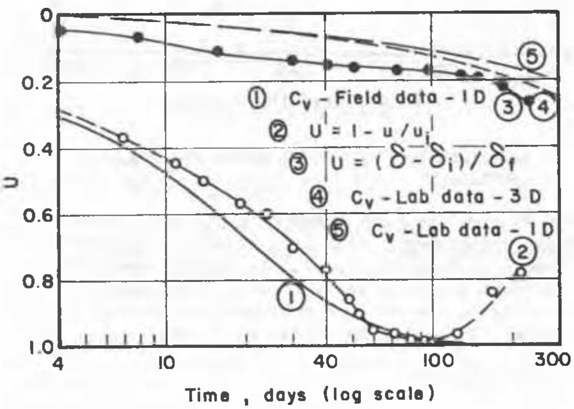

FIG. 11. COMPARISON OF CONSOLIDATION FROM DIFFERENT METHODS

It is believed that although curves 1 and 2 represent the actual degree of primary consolidation, in this case the secondary consolidation is of main concern. The laboratory results from which curves 3 and 5 were determined probably included the effect of the secondary consolidation which undoubtedly took place in the small laboratory samples.

The apparent collapse of the soil structure after 100 hours indicates that primary consolidation may have again been renewed. This point is in need of much more research and raises the question of what is really primary and what is secondary consolidation.

Since the intermittently occurring collapse of soil structure and development of pore pressure may occur in laboratory oedometer tests the laboratory results will include this secondary consolidation. Consequently, the settlement predicted from the results of laboratory tests will include both primary and secondary consolidation effects. In that case, the primary consolidation will form only a small part of the overall response of structures placed on the soil.

V.- CONCLUSIONS

The rates of primary consolidation predicted from the results of one-dimensional laboratory tests were compared with actual field measurements on test embankments representing two-and three-dimensional conditions. On the basis of these results, it was concluded that the use of one-dimensional laboratory data along with two-or three-dimensional theories predict, fairly well, actual field behavior if the anisotropy of the soil can be taken into account.

In one case, secondary consolidation formed a major part of the total consolidation, and the prediction of consolidation based on the relative amount of compression agreed fairly well with laboratory results. This, however, bore no relationship to the actual dissipation of pore pressure in the field. In addition, local yielding of the soil and possible collapse of the soil structure after the pore pressure had initially dissipated resulted in increased settlement and development of pore pressure.

VI.- ACKNOWLEDGEMENTS

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