In Situ Permeabilities for Determining Rates of Consolidation

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This paper presents the use of an in situ permeability test to predict the rate of consolidation of foundation soils. The test method is described and the method of calculation presented. The sources of error in performing the test are discussed. The in situ permeability is used to estimate the rate of settlement on several projects. The rate of settlement as estimated using the in situ permeability agreed with the rate measured by settlement platforms. Using the data from the consolidation tests, the rate of settlement frequently varied from the measured rate of settlement. This relationship was found to exist in a wide range of foundation soil types.

•THE rate of consolidation of foundation soils is a difficult item to estimate during the design of embankments. However, the effectiveness of sand drains, the required height of overloads, and the duration of overloads are determined from the rate of consolidation. Thus, for design studies of embankments constructed upon yielding foundations, a reasonable estimate of the rate of consolidation becomes necessary.

A common method of determining the rate of consolidation is by the use of the data from the consolidation test. The rate may be estimated from the time consolidation curves using the relation that the time is a function of the square of the drainage path. In the work at the California Division of Highways this has proved to be an approximation at best. Another method is to determine the time for a given percent of consolidation to occur, and to determine from this test data the value of the coefficient of consolidation, C_v. This involves comparing a theoretical time with a measured time in the consolidation test. However, when the void ratio, e, coefficient of compressibility, a_v, and permeability, k, are known the coefficient of consolidation, C_v, can be calculated directly. The e and a_v are readily determined with a reasonable degree of accuracy in the consolidation test. However, the permeability is difficult to determine in the laboratory. As a result, the California Division of Highways in 1953 undertook a study of the various methods of determining the in situ permeability of soils in conjunction with an evaluation of the effectiveness of sand drains. A method was developed using a piezometer as a variable head permeameter (1). This new test method showed promise, although the techniques used were very crude at the time.

Work was continued with the piezometer method on several construction projects where settlement and pore pressures were being observed. The techniques of performing these tests were improved. The results indicated that the piezometer method could be used in a wide range of soil types. The piezometers would serve a dual function—measurement of permeability and measurement of excess hydrostatic pressures during construction. A report was received from the U. S. Army Corps of Engineers (2) that confirmed the theoretical work concerning the use of piezometers to determine in situ permeabilities.

PIEZOMETER TEST

The test is performed using nonmetallic, porous tube type piezometers. The installation of these devices by the California Division of Highways has been described else-

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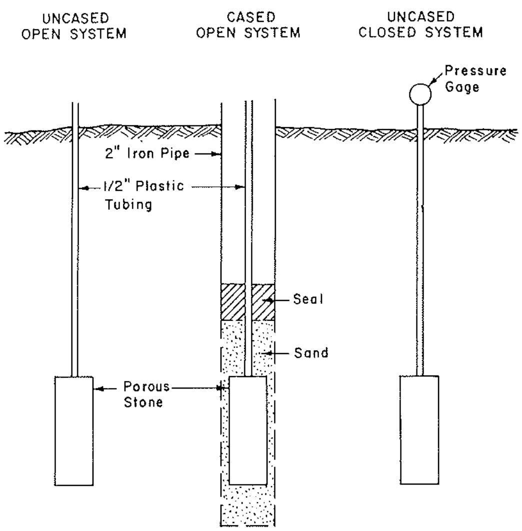

Figure 1. Schematic of piezometer installations.

where (1). The piezometers consist of the porous stone, with or without sand filter, placed in the soil mass. A ½-in. plastic tube is connected to the porous stone and extends vertically to the ground surface. A schematic of the piezometer installation is shown in Figure 1. The test is normally conducted using the open-type system; however, it can be conducted using the closed-type system.

In conducting the test using the open piezometer system, the water level in the plastic tubing is lowered about 5 ft. This is accomplished by means of a hand vacuum pump connected to a ¼-in. plastic tube placed inside the ½-in. plastic tubing. The depth to the water level is then measured at various time intervals (see Fig. 2). The pressure head at a given time interval is then divided by the amount of the total reduction in head (H/H_0). The time interval is plotted against the logarithm of the head ratio. Typical examples of the field data are shown in Figure 3. From these data the basic time lag, the time for H/H_0 to equal 0.37, is determined.

It may be noted that these time lag curves do not always form a straight line through the zero time and 1.00 H/H_0 point. This is primarily due to air in the soil or piezometer system. By lowering the water level in the ½-in. plastic tubing, the pressure is reduced and the air expands, partially escaping. This is one of the reasons for the use of the rising head test instead of the falling head test, in which water is introduced into the system to increase the head. The correction for the air is made by parallel shifting of the straight line portion so as to pass through the zero time and H/H_0 equals unity point. This parallel shifting of the curve assumes that the air has not affected the volume of water passing through the porous stone, which is approximately true when small amounts of air are present. This restricts the use of this test to saturated soils.

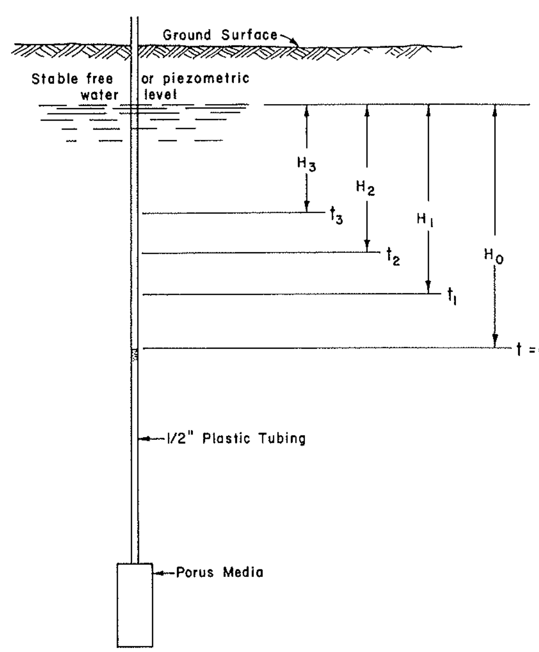

Figure 2. Open piezometer system, indicating water levels at various time intervals.

These time lag curves are the basis for calculating the permeability of the soil surrounding the piezometer. There are three physical dimensions that must be known to calculate the permeability: the length of the permeameter, its diameter, and the diameter of the standpipe. These variables can all be measured with reasonable accuracy. Using the following equation the horizontal permeability can be calculated (4):

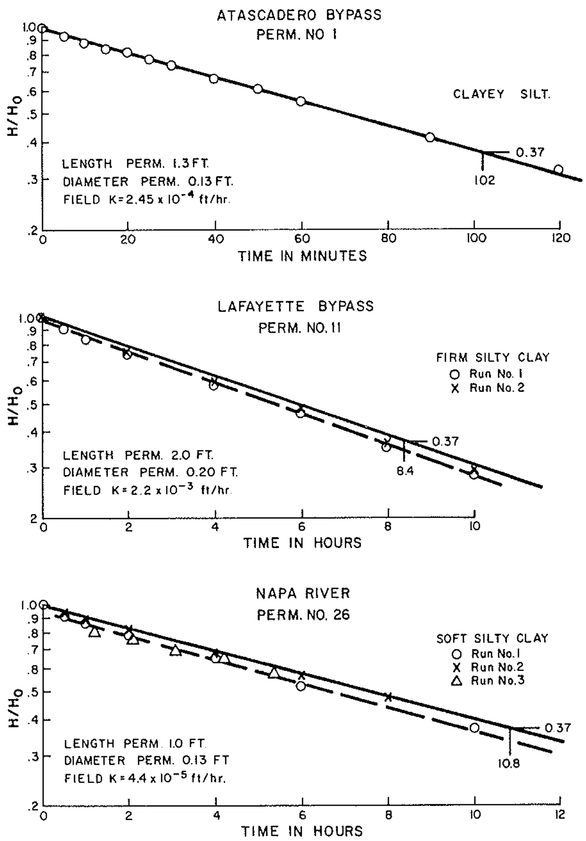

Figure 3. Typical field time lag curves.

$$K_h = \frac{d^2 \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8\,LT}$$

where

K_h = permeability in the horizontal direction,
d = diameter of standpipe,
L = length of permeameter,
D = diameter of permeameter,
ln = natural logarithm,
m = square root of the ratio of horizontal to vertical permeabilities (assume m = 1 for first approximation), and
T = basic time lag.

A correction factor, m, is used to correct the horizontal permeability where the vertical permeability is not the same. This correction factor is generally small. However, concern existed over the effect on the factor m of using different lengths of permeameters. Figure 4 shows one set of data on varying the lengths of the permeameter in a uniform soil. As was expected the basic time lag was decreased from 18.6 to 5.4 hr by lengthening the permeameter; however, only a minor change in permeability resulted. This was found to occur in every instance where variable length permeameters were used in uniform soils. The standard size permeameter now being used is 1½ in. in diameter and ½ to 2 ft in length with a ½-in. OD plastic tubing as the riser.

Factors Affecting the Test

The factors affecting the test were studied on several projects with widely varying soil conditions. It was found that the test was only valid in saturated soils. However, this presented no major problem because the settlement in nonsaturated soils would generally occur during the loading of the embankment.

The range of soil permeabilities successfully measured varied from 10^{-2} to 10^{-6} ft per hour. This range of permeabilities adequately covers the normal range encountered in California soils. Thus, the same test could be used in all soils by simply varying the length of the porous medium from ½ to 2 feet. The time to perform the test could thus be held to one day for all soils. Also the same installation could be later used to measure piezometer pressures developed when the embankments were constructed.

It is frequently necessary to conduct permeability tests on tidal flat soils. Considerable concern existed about the effects of the tide on the results. At three locations near the ocean, a series of tests was performed to evaluate this tidal effect. At two locations a soft silty clay existed and at the other a soft clayey silt existed. At all locations the tide had no effect on the excess hydrostatic pressures or the permeability data as long as the tide was below the elevation of the ground surface. However, when the tide was above the ground elevation a measurable effect on both of these readings

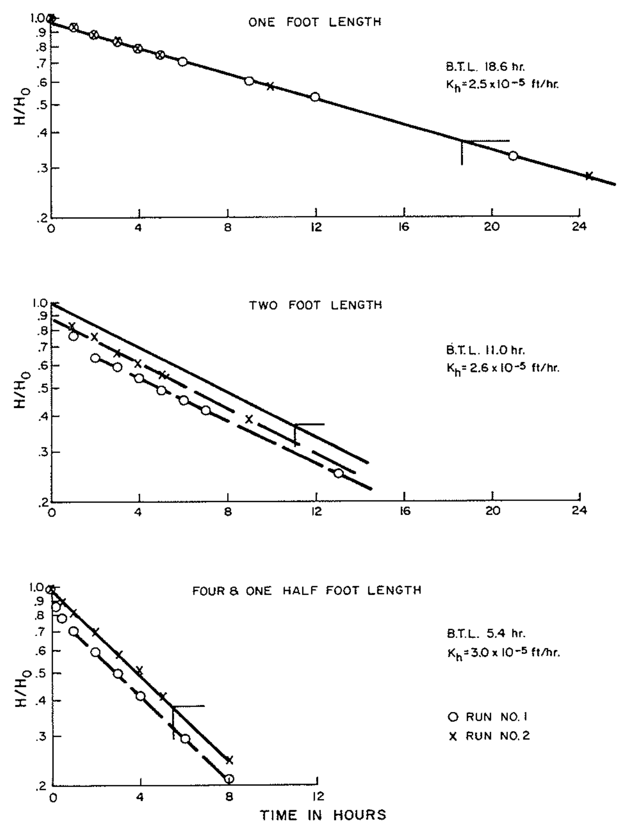

Figure 4. Comparison of various lengths of permeameter.

TABLE 1

SUMMARY OF SOURCES OF ERROR IN DETERMINING BASIC TIME LAG AND EXCESS HYDROSTATIC PRESSURE[a]

Type of Error	Preventable	Precautions and Remarks
Stress adjustment time lag	No	Allow time for soil stress adjustment to take place—usually 3 to 5 days—to minimize effect.
General instrument and installation errors	Yes	Careful workmanship.
Seepage—between soil and conduit	No	No known prevention. Study soil conditions existing at location and use judgment as to best method of installation. Note conditions on log to aid future evaluation of results.
Seepage—between conduit and plastic tubing	Yes	Careful check of bentonite seal.
Gas in lines	Yes	No difficulty in open systems. Use double lines in closed systems.
Gas in porous medium and soil	No	No known prevention. Note on test if suspected that gas exists in porous medium or soil. Careful workmanship will reduce this item.
Internal clogging of porous medium	Yes	Use only clear, clean water in installations. Keep corks on tubing in open systems.
External clogging of porous medium	No	Keep pressure head differences small at all times so as to reduce water flow across porous medium.

[a]From Weber (3).

was noted. These soils had a permeability of 10^{-4} ft per hour or less. It is obvious that at a somewhat higher permeability the tide will have some effect on the readings. The distance between the installation and the tidal waters would also have some effect. These installations had at least 100 ft between the test locations and the tidal waters. Thus, permeabilities can be obtained in tidal flats when proper care is taken.

A summary of common sources of error is given in Table 1, with indications as to how they can be minimized. A discussion of many of the causes of errors in the permeability determination is also included in Hvorslev (2). Most of these errors can be minimized by care in conducting the test, and in making the piezometer installation.

COMPARISON OF RATES OF SETTLEMENT

The primary purpose of this study was to improve the reliability of the estimated rate of consolidation. A comparison of the theoretical rates calculated by the time consolidation curves and the in situ permeability methods with the measured rates would indicate the reliability of the two theoretical methods. The two theoretical methods will be presented and then several comparisons of actual project records will be given.

Time Consolidation Method

The time consolidation method is the normal way of estimating the rate of settlement in the California Division of Highways. The time consolidation curves are plotted for the various loadings. After correcting for the zero and 100 percent consolidation points, the 50 percent consolidation point is determined. From this the time for 50 percent consolidation is determined. Using this time and the corresponding theoretical time factor, a C_V is calculated. This C_V is then used to calculate the times in days for various percentages of consolidation to occur, using the time factors as presented by Taylor (4).

Permeability Method

When the rate of settlement is calculated by the permeability method the e and a_V are obtained from the pressure void ratio curves. From this data a C_V is calculated by the following equation:

$$C_V = \frac{K(1 + e)}{a_V \gamma_W}$$

where
- C_V = coefficient of consolidation,
- K = in situ permeability,
- e = initial void ratio,
- a_V = coefficient of compressibility, and
- γ_W = unit weight of water.

Using this C_V the time in days for various percent consolidations are calculated as in the previous method. In this method, the C_V is calculated from measured test data.

PISMO OVERHEAD

Sand drains and surcharges were used to accelerate the settlement of a yielding foundation soil under an approach fill to a bridge on the Pismo Overhead project. Settlement and excess hydrostatic pressure data were obtained in both the sand drain and non-sand drain areas. A comparison in the non-sand drain area will be presented.

The foundation soil on this project is a heterogeneous mixture of sedimentary deposits. The top 8 ft is a loose, fine-to-medium sand, to a sandy silt, that is pervious and moderately compressible. Because this layer was above the water table the settlement was assumed to occur as the loading was applied. This sand is underlain by a 5-ft layer of wet, soft silty clay to clayey silt that is somewhat pervious and fairly compressible. A 6-ft layer of wet, compact, sand and gravel underlies the silty clay.

TABLE 2

COMPARISON OF PERMEABILITIES AT PISMO OVERHEAD AS DETERMINED FROM CONSOLIDATION TEST AND FROM PIEZOMETERS

Elevation	Soil Type	Permeabilities in ft/hr	
		Piezometer	Consolidation Test
10	Damp, soft, sandy loam Damp, loose, sand		
0	(Original ground) wet, soft, sandy silt Wet, compact, sand and gravel	4.0×10^{-3}	8.1×10^{-7}
-10	Wet, soft, silty clay	3.2×10^{-5}	2.9×10^{-6} 1.8×10^{-6}
-20	Wet, soft, silty clay	1.3×10^{-6}	9.1×10^{-7}
-30			
-40	Wet, compact, sand and gravel		

This sand and gravel layer is free draining and for the settlement studies was considered incompressible and continuous. The sand and gravel layer is underlain by 28 ft of soft, silty clay that is impervious and compressible. This silty clay layer contributes the major subsidence after construction. The silty clay layer is underlain by a wet, compact, sand and gravel layer that appears to be extensive.

A soil profile and the permeabilities obtained are shown in Table 2. The in situ permeabilities indicate that the soil is more permeable than the permeability calculated from the consolidation test. This is especially noted in the upper soil layers. In the lower compressible layer there is only a minor difference in permeabilities.

A 30-ft fill above original ground, with a 5-ft overload, was constructed. The construction rate was about 3 ft of fill per week. The surcharge remained in place about 180 days.

The comparison of the measured and theoretical rates of settlements of the original ground are shown in Figure 5. The consolidation test data indicate a slower rate of settlement than that indicated using the in situ permeability data. This is mainly due to the differences in indicated permeabilities in the upper foundation soil layers. The measured settlements are within reasonable agreement with the settlement indicated by

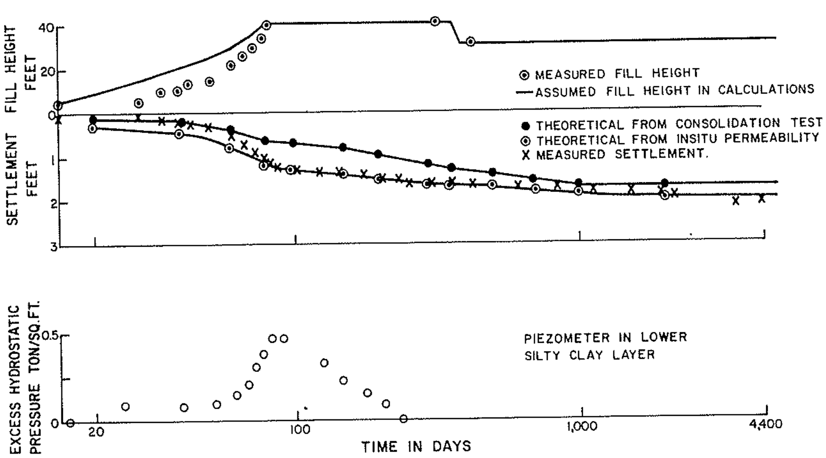

Figure 5. Rate of settlement comparison, Pismo Overhead.

the in situ permeabilities. During construction the measured settlements followed the rate indicated by the in situ permeabilities, then continued at a slower rate than indicated by either method after construction.

One piezometer was placed in the center of the upper sandy silt layer. This piezometer indicated only minor (less than 0.1 ton per sq ft) excess hydrostatic pressure (EHP) during loading and returned to zero about 30 days after loading ceased. Two piezometers were placed in the lower silty clay layer. One piezometer was placed near the upper quarter depth of the layer (Fig. 5). This piezometer indicated about 0.5 ton per sq ft EHP upon completion of loading and decreased to zero EHP in about 200 days. The piezometer in the center of the lower silty clay layer indicated about 0.7 ton per sq ft EHP when loading was completed and ceased to function about 60 days later, when it indicated about 0.5 ton per sq ft EHP. The piezometers indicated that the primary consolidation was completed when the surcharge was removed.

LAFAYETTE BYPASS

A portion of the Lafayette Bypass passes through a valley consisting of soft-to-firm silty clays. A small stream, a county road, and an E.B.M.U.D. pipeline was crossed by means of a bridge on the west side of the valley. The east approach to the bridge consisted of a 40-ft fill over approximately 55 ft of compressible foundation soils. A 9-ft surcharge was used to accelerate the settlement of the fill. Settlement platforms and piezometers were installed to measure the consolidation of the foundation soils.

The foundation soil consisted of three compressible layers underlain by a compact sand. The top layer was about 5 to 10 ft in thickness and consisted of a damp sandy, clayey silt. As this layer was above the water table, instantaneous settlement with loading was assumed.

The center layer is about 20 ft thick, consisting of a damp, soft-to-firm silty clay with varying amounts of sand and gravel; it is below the water table. Using the piezometers as permeability tubes, an average permeability of 3.0×10^{-5} ft/hr was obtained. Using the consolidation time curves from the consolidation test, an average permeability of 6.5×10^{-6} ft/hr was obtained. In calculating the rate of settlement, this layer was assumed to drain upward into the sandy clayey silt layer. This layer contributed a major portion of the longtime settlement.

The lower compressible layer is a damp firm silty clay, with varying amounts of sand, about 30 ft thick. The firm silty clay is underlain by a compact sand to clayey sand that was assumed to be free draining and incompressible. Using the piezometers as permeability tubes, an average permeability of 2.0×10^{-3} ft/hr was obtained for this silty clay layer. Using the consolidation time curves from the consolidation test, an average permeability of 4.7×10^{-5} ft/hr was determined. In calculating the rate of settlement this layer was assumed to drain downward. The major difference in the theoretical rates of consolidation during construction was due to this layer.

The theoretical and measured settlements of original ground at one location are shown in Figure 6. The theoretical settlement using the in situ permeabilities indicates a rapid settlement during loading with no further settlement after the surcharge is removed. The theoretical settlement obtained using the consolidation test data indicates about 20 percent of the consolidation would be expected at the completion of loading and about 70 percent when the surcharge was removed. The measured settlements indicated that the settlement was completed shortly after completion of loading.

Two piezometers were used, one in the center of each compressible layer. The piezometer in the upper silty clay layer indicated about 1.0 ton per sq ft EHP at completion of loading and ceased to function shortly thereafter. Piezometers at other locations in this layer indicated a slow decrease in EHP after loading. The piezometer in the lower silty clay layer indicated a EHP of 0.75 ton per sq ft when loading was completed and had reduced to about 0.3 ton per sq ft when the surcharge was removed. The piezometers on this project indicated that primary consolidation was nearing completion when the surcharge was removed.

The estimated settlement on this project was about twice the measured settlement. This was due to a failure to consider the preconsolidation of the lower silty clay layer.

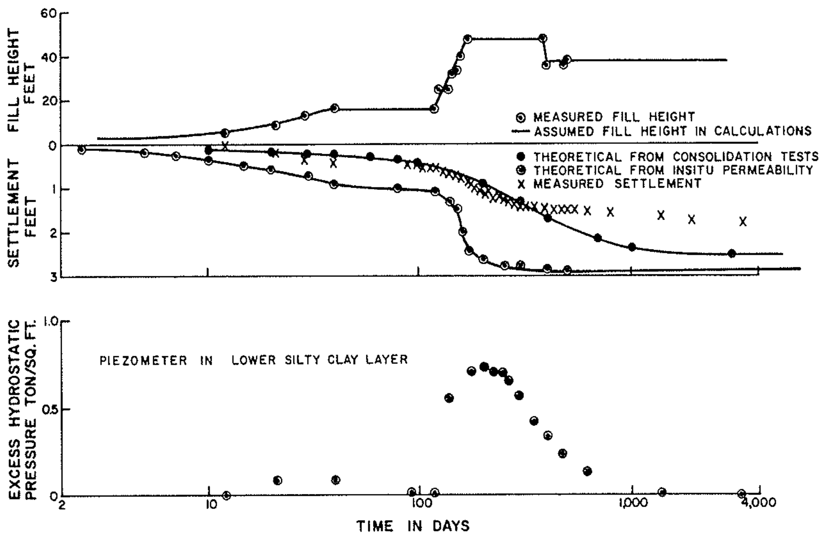

Figure 6. Rate of settlement comparison, Lafayette Bypass.

Generally no significant EHP developed in this layer until 25 to 30 ft of fill was in place (Fig. 6).

ATASCADERO BYPASS

The Atascadero Bypass consisted of about 5 miles of freeway through gently rolling hills. At many of the valleys, cattle or creek crossings, or similar structures were built into the fills. Surcharges were used on many of the fills to eliminate long-term settlements.

The location was a small alluvial valley through which an intermittent stream flows. The fill across this valley contained a culvert for this stream and a cattle crossing, which were constructed by the imperfect trench backfill method. The foundation soil consisted of 19 ft of a sandy silt and was underlain by 12 ft of sandy silty clay. The water table was 2 to 3 ft below the ground surface. The entire area is underlain by a pervious, incompressible silty sand with sandstone fragments. For the settlement calculations it was assumed that the top layer drained to the surface and the lower layer drained to the underlying silty sand layer; also that all settlement above the water table took place during loading.

The permeabilities indicated by the piezometers were generally much higher than those indicated by the time consolidation data. The in situ permeability obtained was 2.1×10^{-1} ft per hour and from the consolidation test data it was 4.6×10^{-6} ft per hour in the lower sandy silty clay layer. In the upper sandy clayey silt layer both methods of determining the permeability indicated a permeability of about 10^{-2} ft per hour. It was thought that the reason for the discrepancy in the lower sandy silty clay layer was that the finer grain portions of the material were tested in the consolidation test while the in situ permeameter tested a larger representative sample. This resulted in the in situ permeability indicating complete settlement as loading occurred. The settlement indicated using the consolidation test data was that about 50 percent of the settlement would occur during loading and about 300 days would be required for completion of the primary settlement (see Fig. 7). This was typical of the results obtained at several locations on this project.

The measured settlements of original ground indicate that the settlement occurred during loading. The piezometer in the lower sandy silty clay layer did not indicate a significant excess hydrostatic pressure during loading. A piezometer placed in the

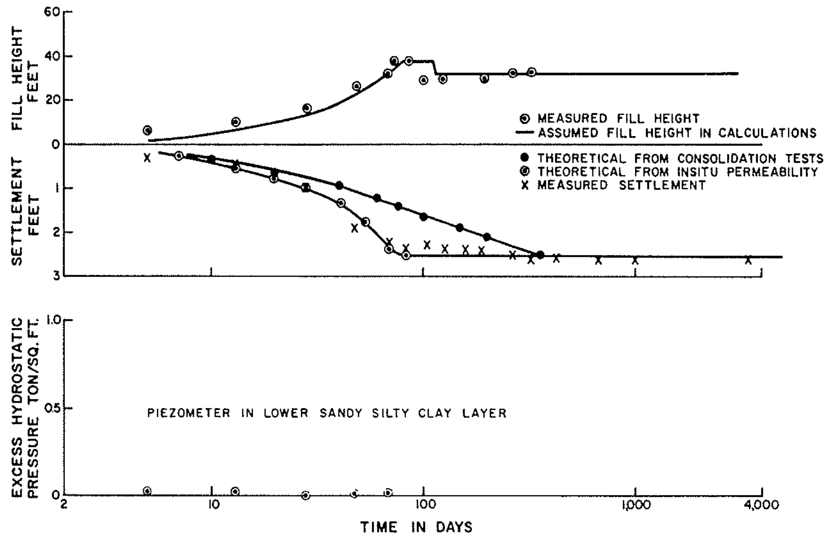

Figure 7. Rate of settlement comparison, Atascadero Bypass.

lower portion of the sandy clayey silt layer was destroyed when the fill was approaching 20 ft in height, and did not indicate a measurable excess hydrostatic pressure during the time it was functioning. At all of the locations studied on this project the in situ permeabilities indicated rates of settlement that were in agreement with the measured rates of settlement and excess hydrostatic pressure readings.

LA TRIANON

The realignment of the existing highway near Blue Lake in northern California required the construction of a fill in excess of 40 ft in height. During the design of this fill it was determined that the foundation soil had insufficient strength to support this height of fill under normal construction procedures. In situ permeameters were installed to determine the anticipated rate of consolidation using various foundation treatments. Whereas the previous in situ permeabilities were determined at the start of construction, the permeabilities were obtained during the design stage on this project and were used to estimate the rate of settlement during construction. Sand drains on 15-ft centers without an overload were used to construct a stable fill.

The foundation soil for the fill consisted of an old slide that blocked the westward drainage of Blue Lake. The material consisted of soft plastic silty clay with fragmented Franciscan sandstone and sand to a depth of 40 ft. The material was extremely heterogeneous and only the finer grain portions of the soil could be tested in the laboratory.

TABLE 3

COMPARISON OF PERMEABILITIES—LA TRIANON

In Situ Permeability, ft/hr	Permeability From Consolidation Test, ft/hr
1.8×10^{-3}	3.2×10^{-6}
5.0×10^{-4}	3.3×10^{-5}
2.3×10^{-2}	1.9×10^{-5}
7.2×10^{-4}	1.4×10^{-5}

A summary of the permeabilities obtained is given in Table 3. The permeabilities from the consolidation test data were all in the range of 1×10^{-5} to 3×10^{-6} ft per hour. The in situ permeabilities were in the range of 2×10^{-2} to 7×10^{-4} ft per hour. The in situ permeabilities were obtained using a 5-ft long permeameter while the consolidation tests contained a 1-in. high sample. It was felt that this was the reason for the differences in permeability.

The estimated rates of settlement, using Barron's work (5), are shown in Figure 8.

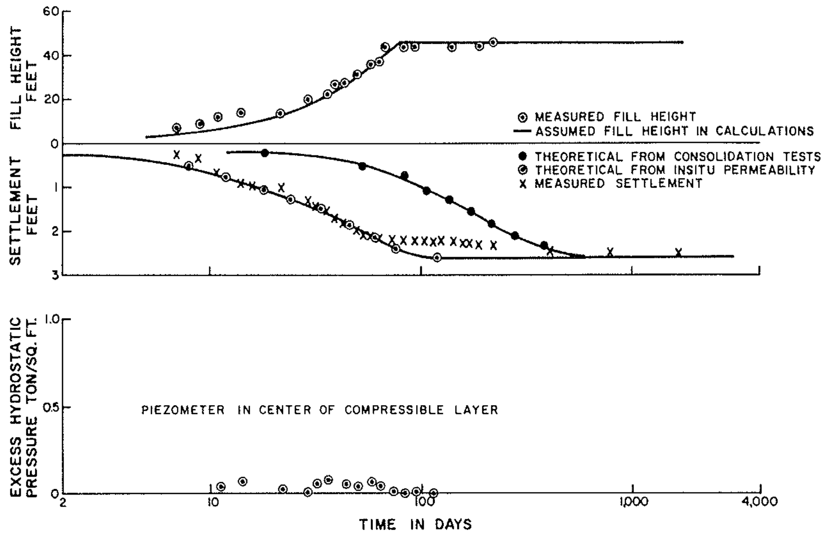

Figure 8. Rate of settlement comparison, La Trianon.

Using the in situ permeabilities it was estimated that 90 percent of the settlement would occur during loading. Using the data from the consolidation tests it was indicated that 20 percent of the settlement would occur during loading. An average of 6.5×10^{-3} ft per hour was used for the in situ permeability and 9.6×10^{-5} ft per hour for the permeability from the consolidation tests. The measured settlements indicated that all of the primary settlement occurred during loading. The piezometers showed only minor increases in excess hydrostatic pressures, indicating that primary consolidation was occurring during loading. The long-term settlement readings indicate minor secondary consolidation is now occurring. The embankment was stable at all times.

NAPA RIVER

The west approach to the Napa River bridge at Mare Island was constructed on 60 to 65 ft of soft silty clay (San Francisco Bay mud) underlain by a stiff silty clay. This fill was constructed as an experimental section with both sand drain and non-sand drain sections. The permeabilities of this foundation soil were determined by both the consolidation tests and by in situ permeabilities (see Table 4). The average in situ permeability was 6.8×10^{-5} ft per hour and the average permeability from the consolidation test data was 2.9×10^{-5} ft per hour. For a report on this project, see Weber (6).

The method used in analysis of the permeabilities and coefficient of consolidation obtained from the consolidation test data is shown in Figure 9. The k and C_V were calculated for various loadings in the consolidation test. These values were then plotted against the logarithm of the average pressure as

TABLE 4

COMPARISON OF PERMEABILITIES—NAPA RIVER

Depth, ft	In Situ Permeability, ft/hr	Permeability From Consolidation Test, ft/hr
10	8.4×10^{-5}	3.9×10^{-5}
20	5.0×10^{-5}	4.0×10^{-5}
30	5.1×10^{-5}	2.2×10^{-5}
10	11.3×10^{-5}	3.4×10^{-5}
20	4.9×10^{-5}	1.7×10^{-5}
30	4.6×10^{-5}	4.9×10^{-5}
10	10.8×10^{-5}	2.5×10^{-5}
20	7.3×10^{-5}	2.2×10^{-5}
30	3.9×10^{-5}	1.4×10^{-5}

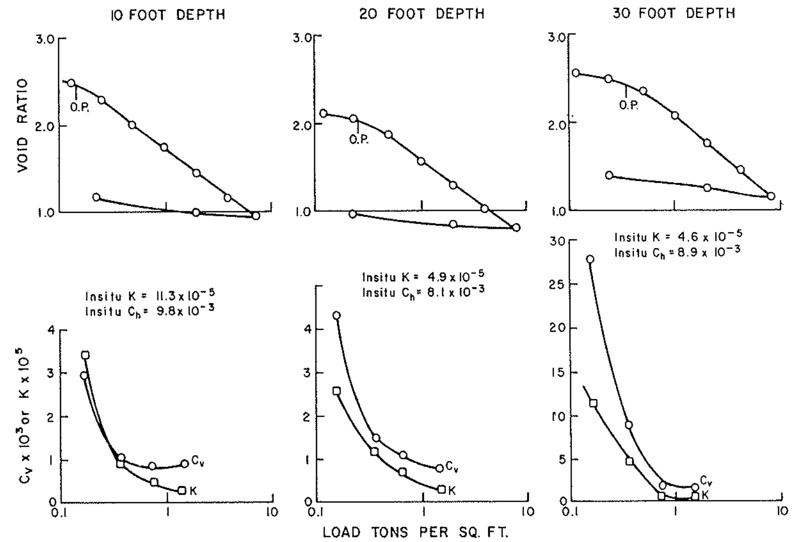

Figure 9. Determination of permeability from consolidation test, Napa River.

shown in Figure 9. The k and C_V at the existing pressure were used to obtain the values to compare with the in situ permeameter values. This procedure was the same for all permeability studies that were conducted.

Using the in situ permeabilities, there tended to be a decrease in permeability with depth (see Table 4). This is as would be expected in a normally consolidated uniform soil deposit. The piezometers installed prior to construction indicated that no EHP existed in this soil. The permeabilities from the consolidation test data are of a random nature. During the design of this project the average in situ permeability was used to estimate the rates of consolidation for the various foundation soil treatments.

The theoretical rates of consolidation for the 15-ft height of embankment and non-sand drain section are shown in Figure 10. The rate of settlement determined from the

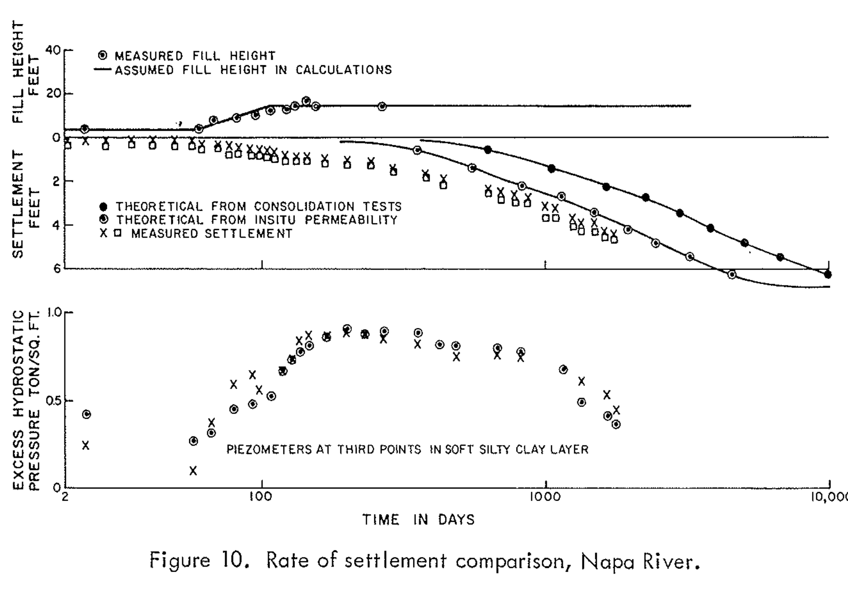

Figure 10. Rate of settlement comparison, Napa River.

consolidation data indicated that it would require about twice the time indicated by the in situ permeability data. As there was a thin peat and/or sand layer between the soft silty clay and stiff silty clay, double drainage was assumed. The measured settlement at two points in this test section are shown in Figure 10. The measured settlements occurred at about one-half the time as indicated by the rate determined by means of the in situ permeability. This non-sand drain section was 150 ft in length with sand drain sections on each end. It was felt that these adjacent sand drain sections may have contributed to the acceleration of the measured rate of settlement. The excess hydrostatic pressure readings of two piezometers, at about the third points of the soft silty clay layer thickness, are shown in Figure 10. The excess hydrostatic pressures are in reasonable agreement with the measured settlement data. Typical examples of the variation of EHP with depth are given by Weber (6). Attempts were made to correlate the settlements as determined by the piezometers with the measured settlements. The measured settlements at the example cited were in reasonable agreement with the settlements as determined by the piezometers. At many locations the measured settlements were greater than the settlements determined by the EHP readings due to plastic flow of the soft foundation soil. In the sand drain areas the rates of settlement were affected by the method of placement of the sand drains and do not directly compare with either theoretical solution for rates of settlement (6).

GENERAL DISCUSSION

The in situ permeameters measure primarily the horizontal permeabilities of the foundation soils. Generally, they indicate a larger permeability than the data from the consolidation tests. There was originally considerable concern that the in situ permeabilities would indicate too rapid a rate of consolidation. However, the measured rates of settlement were generally in reasonable agreement with the settlement rate as determined by use of the in situ permeabilities. This would indicate that either considerable horizontal drainage was occurring in the foundation soils or that the in situ permeabilities were primarily affected by the vertical permeability of the foundation soil.

Comparative permeabilities have been obtained on a large number of projects with a wide variation of soil conditions. In every case the measured settlements have been in reasonable agreement with the rate determined by the in situ permeameter. Thus, a useful tool is available for estimating the rates of settlement with reasonable accuracy. The time required for surcharges to remain in place can be estimated with reasonable accuracy during design operations. On several projects, such as La Trianon, this has resulted in the successful elimination of overloads.

The volume of soil tested by the two methods varies greatly. The in situ permeameter tests from 1 to 3 cu ft of soil in-place. There is some disturbance of the soil mass in installing the permeameters, but it is not felt that this disturbance is large. The consolidation test is conducted upon a sample of soil with a volume of about 3 cu in. This soil sample must be removed from the foundation soil and generally has some disturbance during sampling operations. Also, further disturbance occurs when the sample is placed in the consolidometer and reloaded. Thus, a partially remolded sample of small volume is used in the consolidation test. It is felt that this disturbance and size of the sample account for the differences in permeability obtained by these two methods.

CONCLUSIONS

A simple, consistent test is available for determination of in situ permeabilities of foundation soils. This in situ permeameter will also serve as a piezometer during construction. The principal sources of error have been studied and techniques for performing the test developed.

The rates of settlement determined from this in situ permeability agree well with the measured settlements. Thus, a useful tool is available with which to estimate the amount and duration of time for overloads necessary to eliminate primary consolidation after completion of construction. The permeabilities can also be used for estimating the effectiveness of special foundation treatments, such as the use of sand drains.

ACKNOWLEDGMENTS

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