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16. ABSTRACT

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The present paper is focused upon foundation subsidence, primarily as it produces losses in elevation and non-uniformity of grade. Several highway projects in the California State Highway System are described. Typical foundation soils data are summarized for each project, the foundation treatment method is described, and project settlement records are presented. The projects represent a wide range of soil conditions, and periods of records up to 40 years.

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SUBSIDENCE OF CALIFORNIA HIGHWAYS

By

H. R. Cedergren and W. G. Weber

Prepared for
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H. R. Cedergren* and W. G. Weber**

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The construction of highways over compressible ground produces some of the difficult problems in Soil Mechanics. When highway construction was relatively small business, "rule of thumb" and dependence upon individual personal experience were reasonably adequate; however, with expansion of highway construction into the biggest construction project of all time, existing problems have increased in importance. High speed, wide freeways mushrooming in areas where low speed narrow roadways previously had been adequate puts greater emphasis on the rational soil mechanics methods. The best use of these methods depends upon observing the actual behavior of projects which have been designed and constructed. By looking back in this fashion the adequacy of the methods that we use can be evaluated and if we are observant, improvements can be made in the use of these methods.

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The present paper is focused upon foundation subsidence, primarily as it produces losses in elevation and non-uniformity of grade. Several highway projects in the California State Highway System are described. Typical foundation soils data are summarized for each project, the foundation treatment method is described, and project settlement records are presented. The projects represent a wide range of soil conditions, and periods of record up to 40 years.

INTRODUCTION

Soils which provide the foundations for highway fills and other structural purposes vary substantially both in amount and rate of compression. The rate is governed by the most variable soil property of all - permeability. The projects described in this paper have soils with permeabilities that vary several thousand times. Since the speed of consolidation is directly proportional to permeability, and time therefore is inversely proportional to permeability, it is obvious that the physical time for consolidation to occur in the field can be expected to vary over a tremendous range. Also, the time for consolidation varies with the square of the thickness of the consolidating layers which further magnifies the range that is possible. These points are illustrated by the settlement patterns for the projects described in this paper. Complete consolidation for these projects varies from a few weeks to a couple of centuries.

The influence of variable soil conditions on the rate of consolidation may be seen vividly by comparing the settlement curves of the Santa Margarita and La Trianon projects (Figs. 8 and 10) with settlement curves for the projects having mud foundations (Figs. 1, 5, 6 and 11). The Santa Margarita formations consolidated almost as quickly as load was applied, whereas the thicker and more impervious soils under the east approach fill of the San Francisco-Oakland Bay Bridge may require about 200 years for the completion of consolidation. The indicated ratio between these two extremes is in the order of 5000 to 1. In relation to practical construction this time range is of very great significance.

The amount of settlement that can be attributed to consolidation of foundation soils can be estimated with reasonable accuracy from laboratory consolidation tests. When soft foundation soils are loaded to the point where stresses approach or exceed their strengths, excessive settlements often occur because of plastic deformation of the soil. As these deformations may continue for many years after construction they tend to indicate that consolidation is occurring more rapidly than is actually the case. The fills at Tule Lake and Tormey Valley illustrate this condition.

Occasionally soft foundation soils are underlain by firmer strata which are assumed to be incompressible under the loading of the fills. Sometimes these underlying layers, though firm in relation to the overlying deposits, are not entirely incompressible, and the total settlements are measurably greater than the predicted amounts. This factor is believed to have an influence on some of the projects reported here.

Overloads, sand drains and controlled rate of loadings often are used in the construction of fills on compressible foundations. In the more pervious soils which existed at Santa Margarita Bypass and at La Trianon this type of treatment was very successful. Sand drains, overloads, and a controlled rate of loading were employed on the Bayshore Freeway in an attempt to obtain nearly complete consolidation during construction. Partial stripping, overloads and a waiting period were used on the east approach to the San Francisco Bay Bridge to reduce settlement after construction. A study of the records furnished in this paper for the above projects shows that substantial settlements did occur after construction for the projects having low permeability foundation soils.

The use of present day methods would indicate that in general when fills are constructed over impervious muds, overloads have to remain in place for several years to be effective in minimizing after-construction settlements. Several projects now planned or under construction in the California Highway System are to have waiting periods of one to three years, controlled rates of loading of one to three feet per week, and substantial overloads. Design studies for a number of other important projects on mud or peat foundations have led to the decision to construct bridges or other structure to eliminate troubles associated with earthwork construction over weak, compressible foundations.

In some projects stability is a major concern, and maintenance due to settlement of secondary importance. The open water fill at Candlestick Cove is an example of this type of project.

THE PERFORMANCE OF SEVERAL HIGHWAY PROJECTS

Selection of the Projects

Many highway projects in the California State Highway System have had within their limits sizeable deposits of highly compressible foundations. Approach fills to numerous structures have been constructed on moderate to heavily compressible foundations which have been treated in some manner to reduce settlements after construction. A number of California projects have been described in published engineering reports and papers.[1,2,3,4,5] In the present paper, several projects are described which give an indication of the scope of soil conditions encountered and treatment methods employed to reduce settlements after paving of the highways. The cases presented cannot be completely representative of California highway projects in which subsidence has been important, since every project is in some way different from every other. They have been selected to demonstrate the wide variety of soil conditions that occur in the State of California and the treatment methods that have been employed. Typical data for the several projects are shown in Table I.

General Discussion

When highways are constructed over compressible foundations, several methods of treatment may be considered. The one or combination selected is the least costly that will satisfy the requirements of the job. The methods used in California to reduce settlements after construction are:

1. Stripping to firmer soil and replacing with low compressibility fill.
2. Displacement of compressible material by blasting and overloading.
3. Placing fills at controlled rates of loading to prevent failures, and providing waiting periods to allow settlement to occur.
4. Use of overloads to accelerate consolidation.
5. Use of sand drains to accelerate consolidation.
6. Combinations of methods.

On projects which have foundation problems borings are made and "undisturbed" samples secured for laboratory testing. Laboratory consolidation tests are used for estimating the amount and rate of settlement that may be expected. The coefficient of consolidation is determined from the t_{50} or t_{90} obtained from laboratory time-consolidation curves. On projects of major size the vertical permeability is determined using the coefficient of consolidation obtained as described above and the field permeability test is performed[8] at several points in the foundation. This test measures the combined horizontal and vertical permeabilities. The rate of consolidation is then calculated using: (a) field permeabilities, and (b) laboratory permeabilities. The field permeabilities usually are two or more times greater than laboratory permeabilities, which is believed due to the fact that horizontal permeabilities usually are somewhat greater than vertical permeabilities. Rates of consolidation measured during construction usually agree better with theoretical rates estimated with field permeabilities than with rates estimated with permeabilities determined in the laboratory. A study of the examples shows generally close agreement between computed and measured amounts of settlement. As can be expected, calculated rates of settlement are subject to greater differences.

The measured settlements reported in this paper are the subsidences produced by the fills. A major portion or all of this subsidence is due to consolidation of the underlying foundation soil. Any remainder may be due to: (a) plastic deformation of the foundation soil, or (b) compression within the fill. In some of the projects settlements were not measured until after construction of the fills. In those cases settlement points were placed on the surface of the fill. In others, settlement platforms were placed on or near the original ground, and settlements were recorded from the start of construction.

The theoretical settlements are estimates of the amounts of settlement caused by consolidation of the compressible foundation soils, and do not include allowances for plastic flow, compression within the fills, or consolidation of firmer underlying strata.

The calculations of theoretical settlement rates are based upon the vertical flow of pore water only. In thick deposits of compressible soils there may also be an appreciable horizontal flow of pore water. This horizontal flow of pore water may be important where the fills are narrow in relation to the depth of the compressible soil.

Example No. 1 - Eastern Approach Fill to SFOBB

In 1933, foundation investigations for the approach fills east of the toll plaza for the San Francisco-Oakland Bay Bridge revealed soft silty clay to depths up to 80 feet. This soil, known locally as "Bay Mud," is capable of supporting fills only 5 or 6 feet high without failures. The mud is a silty clay of recent geological origin, of high moisture content, impervious and highly compressible. Its in-place strength is nearly zero at the surface and increases at a rate of 12 to 15 pounds per square foot for each foot of depth. In a fully consolidated state it exhibits a "true" coefficient of friction of about 28°.

This approach fill was constructed by dredging out the upper 12 feet of the soft foundation soil and replacing with sand fill. The fill was constructed to about 10 feet above the original mud line, at which time a waiting period of about one year was allowed. A small surcharge was then removed and the roadway placed.

The measured settlements at typical locations are shown in Fig. 1. The settlements represent changes in the elevation of points set on the surface of the completed fill and do not include settlement which occurred during placement of the fill.

Theoretical time-settlement curves are given in Fig. 2. These curves were obtained using consolidations and permeability factors obtained from a limited number of consolidation tests. Very little consolidation data are available for this project; hence, the time comparisons must be considered only approximate. The mud left in place could be expected to undergo ultimate reductions in volume of 20 to 25% under the increased loads produced by the heavier fill that replaced the dredged mud, and the thickness of fill placed above original ground. Since the thickness of soft mud left in place ranged from about zero to 70 feet, ultimate settlements up to about 12 feet could be expected due to consolidation within the foundation (see Fig. 3). Measured settlement in areas where 10 ft. or more of mud was left in place ranged from 2.5 to 4 feet after 5 years, and from 2.5 to 7.5 feet 18 years after paving (see Fig. 1).

In the 27 years that have elapsed since the initial paving of this road, the approach pavements have required only moderate maintenance. On the whole this has been a very successful project.

A study of the settlement records for this project shows that the portions that were constructed over shallow thicknesses of soft mud (10 - 20 ft.) stabilized in about 10 years, whereas, progressively longer times are being required over the deeper mud.

Fig. 3 compares several locations on this fill in relation to the thickness of mud left after dredging. It shows the amount of settlement recorded before paving, the amount that took place in the first 18 years after paving, and the estimated total ultimate settlements. The measured settlements that appear to be completed are somewhat in excess of the theoretical ultimate amounts of consolidation. It is believed that plastic flow of the soft foundation soil which may have occurred during early stages and compression within the fill can account for this settlement.

Example No. 2 - Bayshore Freeway

In 1947 several miles of freeway were constructed along the west side of San Francisco Bay south of San Francisco. This is a portion of the "Bayshore" freeway. It is founded on bay mud in varying thicknesses up to 60 feet. This mud is a soft silty clay of high moisture content, low strength, and low permeability rather similar to that under the eastern approach fills to the San Francisco-Oakland Bay Bridge, discussed above. At some locations the soft mud is underlain by sand and at others by a stiff bay mud. The soil conditions are, therefore, conducive to variable drainage. Sand drains were used for acceleration of consolidation of the foundations of fill approaches to structures. Considerable settlement data have been collected for this portion of highway. In the present report, settlement records are given for four locations, two in sand drain areas and two in non-drain areas (see Figs. 4 & 5). In each kind of construction area one shallow mud location was selected and one deep mud location. Thus, in the sand drain-treated sections, Location A had a mud thickness of 42 feet, and Location B 12.5 feet. In the non sand-drain areas, one section (Location C) had a mud thickness of 39 feet and the other (Location D) a mud thickness of 13 feet.

The sand drains were installed with a closed mandrel. They were backfilled with sand and drained by means of a layer of sand fill placed over a thin working table of earth fill. The rate of loading was 3 feet per week and a 5-foot surcharge was placed in the sand drain areas. Extensive failures occurred on this section of roadway during construction, and at some locations the sand drains may have been sheared or damaged by these failures. The examples used in this report are in areas where no shear failures were known to have occurred; however, plastic flow of the bay mud may have taken place throughout much of the length of this project since the loading was at a comparatively rapid rate.

By reference to the settlement curves for the non-drain areas (Fig. 5) it is seen that settlement at the shallow mud location (13 ft. of mud) appeared to be essentially complete within 8 or 10 years, but the deeper area (39 ft. of mud) probably will require 90 to 100 years. In contrast to these times, the sand drain installation in 12.5 feet of mud (Fig. 4) was well consolidated within a year (quicker than average for the project) and the deeper area (42 feet of mud) appeared to be well stabilized 6 or 7 years after construction.

The examples illustrate that wide variations in rate of settlement were recorded in the non sand-drain and in the sand drain-treated portions of this freeway. Broad conclusions of the benefits of sand drains should not be attempted from the few examples presented here. A review of the records at 50 or more locations has suggested that the sand drain-treated foundations generally consolidated several times more rapidly than comparable non-treated foundations. However, failures during construction, variations in natural drainage, and possibly other factors, have influenced the behavior of the foundations. Sand drains evidently were speeding up consolidation, but because of the very slow nature of the whole process, a fairly long time still was required. An evaluation of the efficiency of sand drains is beyond the scope of this paper. The data reviewed here are presented only to permit a comparison of this project in relation to the other examples.

Example No. 3 - Candlestick Cove "Open Water" Fill

One of the most outstanding mud displacement projects ever attempted is the "open water" fill which was constructed across Candlestick Cove in the period from 1952 - 1955[4,5]. "Bay Mud" varied in depth from 30 to 80 feet on this two-mile stretch of highway. The soft mud is a low permeability silty clay of high moisture content, and low strength. Boring data showed sand under the mud in some of this area; hence, it is possible that double-drainage exists for portions of this foundation.

In a series of contracts, fill was constructed by forcefully displacing the mud as completely as possible, after initial efforts to "float" the fill had proven unsuccessful. A total of 4,007,000 cubic yards of imported fill material was placed across the cove. Borings after placement showed that mud had been displaced to depths as great as 70 feet and that as much as 80 - 90% of the soft mud may have been displaced under portions of the road section. The best displacement was obtained by maintaining continuous filling operations with the front in the shape of a wedge having an angle of about 45 degrees on each side of centerline. Blasting was employed periodically to keep the mud on the move and minimize its entrapment beneath the fill. With all of these efforts it was not possible to remove all of the mud, which is evident from the boring records and the slightly undulating profile the road has developed. The settlement curves presented with this report (Fig. 6) indicate that moderate additional differential settlements can be expected for some years to come. For comparison with the measured settlement rates on this and other comparable Bay Area projects, theoretical settlement curves are given in Fig. 7. These curves cover the range of thickness of mud left in place as estimated from borings after construction. As previously noted, there is evidence that underlying strata may furnish some drainage to the compressing mud. In developing Figure 7 double drainage (top and bottom) was assumed in the theoretical calculations. In most areas of San Francisco Bay, bottom drainage appears to be relatively non-existent; hence, single drainage usually is assumed. Multiplying the time scale in Fig. 7 by four will convert the time-settlement curves to single drainage.

This project is believed to be a good example of a successful attempt to construct a highway over fairly deep mud by the displacement method.

Example No. 4 - Santa Margarita Bypass

In the summer of 1955 several highway fills up to 37 feet in height were constructed over alluvial deposits of sandy silt to sandy silty clay containing some gravel. Beneath 20 to 40 feet of these moderately compressible formations were less compressible sands, sandy clays, and sandy gravels. Field and laboratory studies indicated that settlements up to 2½ feet could be expected with a substantial part occurring during construction. To reduce settlements after construction, overloads were prescribed for this location. Strength was not a problem, and no other special measures were required.

Records for this project showed that settlement occurred almost as rapidly as the fill was placed. Minor additional settlements of 0.1 to 0.2 foot (believed to be secondary consolidation) occurred in the first three years after construction. Although settlements were fairly large, no serious problems developed at this section of roadway.

Permeability information obtained on several "typical" soils in this foundation are shown in Table II.

Typical soils information for this project may be found in Table I. The rate of loading, and measured and theoretical settlement curves are given on Fig. 8. In making settlement calculations the soil at this location was assumed to be a two-layer system. The top 15 feet is a sandy silt which lies partially above the water table. Underlying this layer is 12 feet of sandy silty clay underlain by silty sand to unknown depth. All drainage was assumed to be vertically upward into dry soil or to the underlying sand. The compression of the soil above the water table was assumed to occur during loading. Laboratory time-consolidation curves were used for calculation of permeability and consolidation constants. The field permeability determinations were made using a one-foot Norton tube piezometer after A. Casagrande's design.

Combining the two layers and determining the theoretical rate of settlement assuming single drainage of the two layers back-to-back, the field permeability indicated 100 percent primary consolidation at the end of construction, while the laboratory permeability indicated 56 percent consolidation at the same time. The measured settlement is in close agreement with the thoretical rate determined from the field permeability.

<u>Example No. 5 - Fill Across Tormey Valley</u>

The construction of a parallel bridge over the Carquinez Straits near the northern end of San Francisco Bay required some of the heaviest earthwork in California highway history. Simultaneously with construction of the new bridge, several miles of traffic bottleneck to the north and south of the Straits were eliminated by improving the road to freeway standards and re-aligning the road south of the Straits. The new alignment south of the Straits gave rise to the largest single cut in California highway history up to that time. The "Big Cut," required 8 million cubic yards of excavation through a large hill of highly weathered shale. To the south of the "Big Cut" from Hercules to Tormey Valley, fills (up to 170 ft. height) were required. Borings in the foundations of these large fills disclosed comparatively stiff clays that would have been of no concern for "ordinary" fills but were of borderline properties for the large fills required.

In the foundation area across Tormey Valley there is a deep deposit of semi-consolidated alluvium. One boring was drilled to a depth of 87 feet before a firmer underlying stratum was encountered. Moisture contents of the alluvium ranged between 25% and 30%. Unconfined compression tests resulted in shear strengths from 1000 to 2500 pounds per square foot. The averages of all tests in three borings were 1400, 1600 and 1700 pounds per square foot shear strength, respectively.

Because the indicated foundation strengths were borderline for fill of the planned height, buttress fills 30 feet high and 130 feet wide were placed at the sides of the fill. Readings taken on heave stakes in the struts and native soil during construction showed that movements up to one foot occurred in both the vertical and horizontal directions, indicating that plastic flow was taking place. Movement of the heave stakes continued at a greatly reduced rate for a time after construction. No failures occurred; however for a time some concern was felt over stability.

The height of fill was increased at a rate of 3 or 4 feet a week (see Fig. 9) which was considered safe from the standpoint of stability. Consolidation computations indicated that substantial settlement could be expected and that a major part would occur during construction. The only special measure adopted for the control of settlement was placement of an overload.

Because of the magnitude of this fill settlement platforms and piezometers were installed so that a record of settlement and pore pressures could be maintained during construction. These readings indicated that possibly 50% or more of the added load was temporarily carried on the pore water. These pressures dropped substantially in the first year after construction. About two years after completion the fill was dressed to grade and paved. Since paving, settlement has been less than one foot and has not noticeably altered the smooth profile of the roadbed. A comparison of theoretical and measured rates and amounts of settlement is given in Fig. 9. The agreement is within limits of error normally expected.

Example No. 6 - Fill West of La Trianon

In 1956 a fill from 5 to 45 feet in height was required in Lake County at the edge of a swampy area near a small community known as La Trianon. Explorations in the foundation area disclosed wet soft plastic silty clay to sandy silt containing rock fragments of broken Franciscan sandstone and shale. The water table was at a depth of 7 to 8 feet. Unit weights of the soil ranged from 106 to 138 pounds per cubic foot, and in-place water contents below the water table were predominately between 18% and 33%. Liquid limits were from 25 to 32% and plasticity indexes from 5 to 11%.

Permeabilities calculated from laboratory consolidation curves ranged from 8×10^{-5} to 4×10^{-6} ft./hr., and averaged 3×10^{-5} ft./hr. Field permeability determinations were made in two borings, with the following values being obtained at four points: 2×10^{-2}, 7×10^{-4}, 5×10^{-4}, and 5×10^{-4} ft./hr.

The average of the three lower values is approximately 50×10^{-5} ft./hr., which is 16 times the average value obtained from laboratory consolidation tests. As the laboratory consolidation tests were run on the fine grain soil matrix this difference in permeability would be expected.

Typical curves showing the rate of construction of the fill, measured settlement, and theoretical time-settlement rates are given in Fig. 10, together with the typical soil cross-section. The total settlement measured after 3 years was 2.5 feet. The amount estimated from laboratory P-e curves was 2.6 feet.

Calculations of theoretical rates of settlement with sand drains installed to shorten the path of escape of excess water were made using curves developed by R. Barron(7). These calculations indicate that settlement should have been 80 percent complete at the time 44 feet of fill was in place. Because of the favorable performance that was expected with sand drains at this location, the design specified sand drains at a rectangular spacing of 15 feet under the heavier parts of the fill. Fill was placed at an average rate of 5 feet a week without surcharging. No struts were specified as it had been concluded that sufficient strength would develop during construction. No evidence of failures of any type developed during construction.

The in-place shear strength of the weaker sediments in this foundation as indicated by unconfined compression tests on "undisturbed" samples was predominately in the range of 200 to 400 pounds per square foot and averaged 280 pounds per square foot. Without any special treatment or control over rate of fill placement, 15 to 20 feet of fill probably could have been placed here. With the use of struts and possibly a controlled rate of loading to prevent failures it is likely that if this road had been so constructed, and paved a year after construction, post construction settlements would have been small. As constructed, settlements after paving have been negligible and the grade is smooth and uniform.

The performance of this sand drain foundation job has been so satisfactory that one might question their need. Had sand drains not been used, however, and ample alternative measures not been employed it is possible that serious trouble may have developed here, due to the low strengths of the 25 to 30 feet of saturated sediments in the foundation.

Example No. 7 - Tule Lake Fill

Many lake (lacustrine) deposits form under conditions of very poor drainage which lead to high compressibility and unusually low strength. The conditions which exist during the building-up process undoubtedly have an important influence on the structure that is formed. A factor that usually has been ignored is the degree of freedom with which excess water can escape as the deposits compress under their own weight. The authors have compared clay deposits which were formed under good drainage (some valley deposits) with similar deposits which were formed under poor drainage (bay and lake deposits) and have found the well drained deposits to have developed considerably greater strengths than the poorly drained deposits.

The Tule Lake deposits (in Lake County) are among the poorest low-organic lake deposits in the California highway system. These soils are very week, highly compressive and relatively low in permeability. The highway across an arm of Tule Lake has been progressively built up over a period of 40 years. Fortunately, settlement records have been kept over most of this period. This section of road furnishes an excellent long-time record of settlement.

The progressive changes in the cross-section of the fill, as the road has been improved are shown in Fig. 11. The original road across Tule Lake was constructed to 9 or 10 foot height in 1922. By 1950 the roadway surface had subsided to only three feet above the surrounding marsh area, and had to be closed periodically due to flooding. In 1950 the roadway was reconstructed to a height of about 17 feet above the estimated original ground level. During the 1950 reconstruction a failure occurred when the fill reached a height of about 15 feet. After this failure, berms were increased and the fill was completed to the 17 foot height. By 1955 the roadway had subsided five feet to a height of about 12 feet above the surrounding marsh, which was below high water stages. The fill was then reconstructed to approximately 18 feet above original ground, which is the last major work on the fill. At the present time (1962) the roadway is about 15 feet above original ground. Total settlements of up to 18 feet have been measured.

Borings in the foundation of this fill disclosed saturated, soft, dark gray silty clay and clay up to 83 feet in depth. The test unit weights averaged 93 pounds per cubic foot and the water contents 85%. There was no appreciable trend toward increased density at greater depth or reduced water content with depth.

Theoretical settlement curves are compared with the recorded rate of settlement on Fig. 11, wherein the curves for the three stages are plotted one above the other.

During the 1955 reconstruction, water level type settlement platforms were installed at Stations 312+00, 316+00 and 320+00. These platforms are at the same locations as platforms which were installed for the 1950 construction, so as to permit continuation of settlement records. Four piezometers were installed in the soft clay.

By reference to the settlement curves in Fig. 11 there is evidence that considerable plastic flow has occurred, particularly during the 1950 construction. It is known that the loading rate in the 1950 construction led to a failure and probably set plastic flow in motion. The shape of the time-settlement curve for the period following this construction seems to verify this conclusion. Also, it may be noted that the actual total settlement has substantially exceeded the amount calculated from consolidation tests. In this kind of soil rather good agreement usually is obtained between calculated and actual amounts of ultimate settlement. The recorded amount of settlement is strong evidence that some of the settlement may have been caused by flow.

Evidence of plastic flow on this job is also provided by the movement of monuments set in the berms and outside the fill area at the time of the 1950 construction. Iron pipe monuments at the toe of the berm show an outward movement of one to two feet between 1950 and 1955. Elevations on the iron pipes indicate that a slight upward heave occurred in 1950, and downward movement of about 0.4 foot between 1951 and 1955. These pipes were located five feet outside the toe of the berm. Two sets of bronze monuments were set in the berm, one at the toe of the main roadway prism, and one near the outer edge of the berm. Those at the toe of the slope showed that settlements of 0.6 to 0.7 foot and lateral movement of approximately 0.5 ft. occurred between 1950 and 1961. The monuments at the outer edge of the berms recorded 0.3 ft. of settlement and small, erratic outward movements of 0.3 to 0.6 feet. It is estimated that plastic flow may account for one to two feet of the settlement of the roadway prism that was measured.

Measurements made in 1961 on piezometers installed in the soft mud under the fill, showed pore pressures equal to about 75% of the loading of the fill. The large pore pressures, and their slow rate of dissipation, indicate that consolidation of the soft mud is far from complete. The theoretical calculations of rate of consolidation indicate that about 100 years will be required for full consolidation. When working with impervious soils it is normal to estimate the amount of settlement that can be expected in a 20- or 30-year period, this being the useful life of a road. For this project about one-half of the estimated ultimate settlement can be expected 20 or 30 years after construction.

CONCLUSIONS

Several important conclusions may be made from a study of the settlement records of fills that have been constructed over compressible foundations:

1. The rate of consolidation of thick impervious soils is very slow, requiring decades or centuries for completion.
2. Rational soil mechanics methods can furnish useful indications of the probable rate and amount of settlement.
3. Methods used for reduction of settlements after construction should not be considered "cure alls." They should be applied only with the competent use of present knowledge of soil mechanics principles.
4. The stability of the fills must be carefully studied and construction methods used that will keep plastic deformation of the foundation soil to a minimum. If it is once set in motion, it may continue for many years.
5. A broader recognition of the extremely wide variations in soil behavior would be highly beneficial in reducing the number of cases of bad project experience caused by delayed settlements. Soils engineers are well aware of this facet of soil behavior. It is hoped that the information presented in this paper will be a useful demonstration of the broad range of soil conditions that are regularly encountered in the foundations for highways in the State of California.

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The projects reported above are all part of the State Highway System of California. They were constructed over a period that goes back 40 years. Acknowledging all those who were in responsible charge of the numerous projects cannot be attempted. The present study was possible through the use of records in the files of the Materials and Research Department which is under the direction of F. N. Hveem, Materials and Research Engineer. At the time of the studies, the Foundation Section was under the supervision of A. W. Root.

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TABLE I

ILLUSTRATIVE DATA FOR PROJECTS DESCRIBED

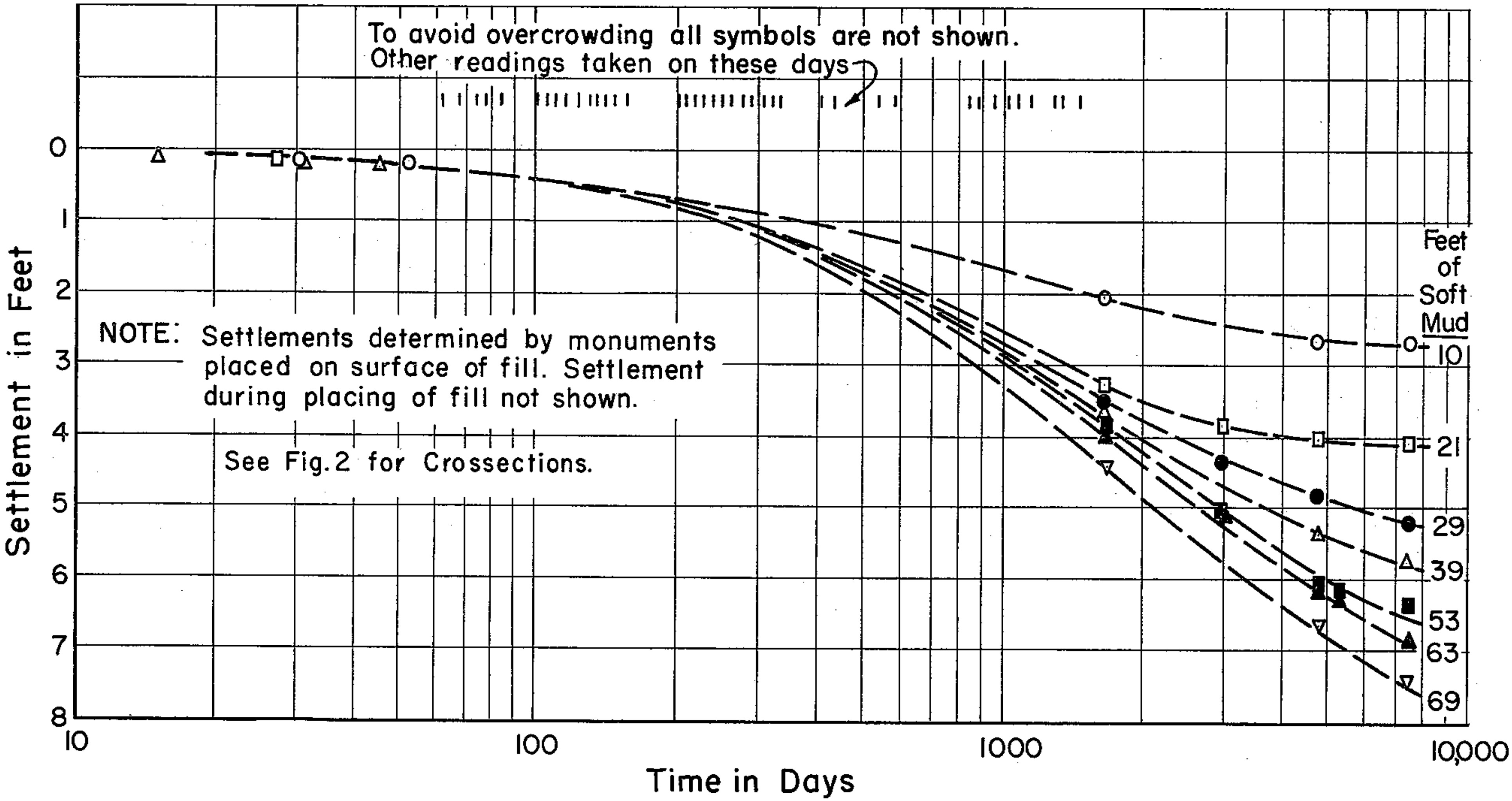
Example No.	Project	Original Thickness of Soft Soil	Fill Ht. Above Orig. Ground	Depth of Dredging or Displacement	Typical Soils Data w	e	LL	PI	Field k	Lab k	C_v
		(feet)	(feet)	(feet)	%		%	%	(ft/hr)	(ft/hr)	(ft^2/lb)
1	East Approach S.F.O.B.B.*	10-70	10-12	12							8×10^{-4}
2	Bayshore	5-45	5-10	None	100	2.8	90	50	1×10^{-5}	2×10^{-5}	2×10^{-3}
3	Candlestick Cove	25-80	15	Up to 80	60-80	1.9	55	30	2×10^{-5}	1×10^{-5}	6×10^{-3}
4	Santa Margarita	30	30	None	20-30	0.7	30	7	3×10^{-1}	5×10^{-4}	5×10^{-2}
5	Tormey Valley	0-90	135	None	30	0.8	50	30	7×10^{-6}	3×10^{-6}	6×10^{-3}
6	La Trianon	35-40	5-45	None	20	0.7	25	6	1×10^{-3}	1×10^{-5}	3×10^{-2}
7	Tule Lake	20-80	15	None	80-100	1.7	70	30	5×10^{-5}	6×10^{-5}	1×10^{-3}

*San Francisco-Oakland Bay Bridge

TABLE II

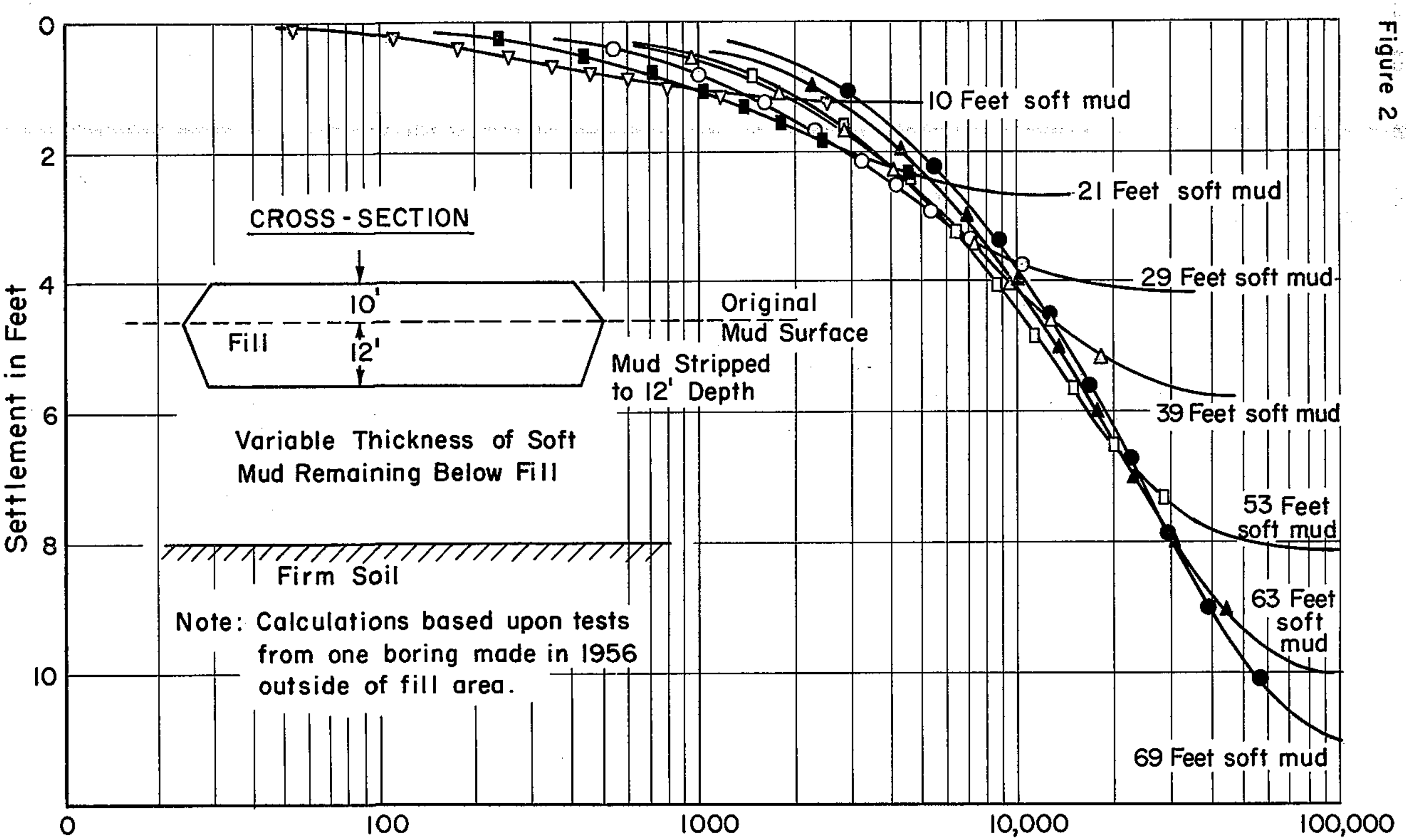
Comparison of Soil Permeabilities
Santa Margarita Bypass

Description of Soil Tested	Field "k" Before Construction (ft/hr)	Field "k" After Construction (ft/hr)	Lab "k" (ft/hr)
Sandy Loam	2×10^{-1}	-	5×10^{-5}
Clayey Silt	2.5×10^{-4}	2.9×10^{-5}	2.9×10^{-5}
Clayey Silt	2×10^{-1}	-	1.4×10^{-5}
Clayey Silty Sand	4×10^{-5}	-	3×10^{-5}
Clayey Silt with some Sand	4×10^{-5}	1×10^{-5}	2×10^{-5}

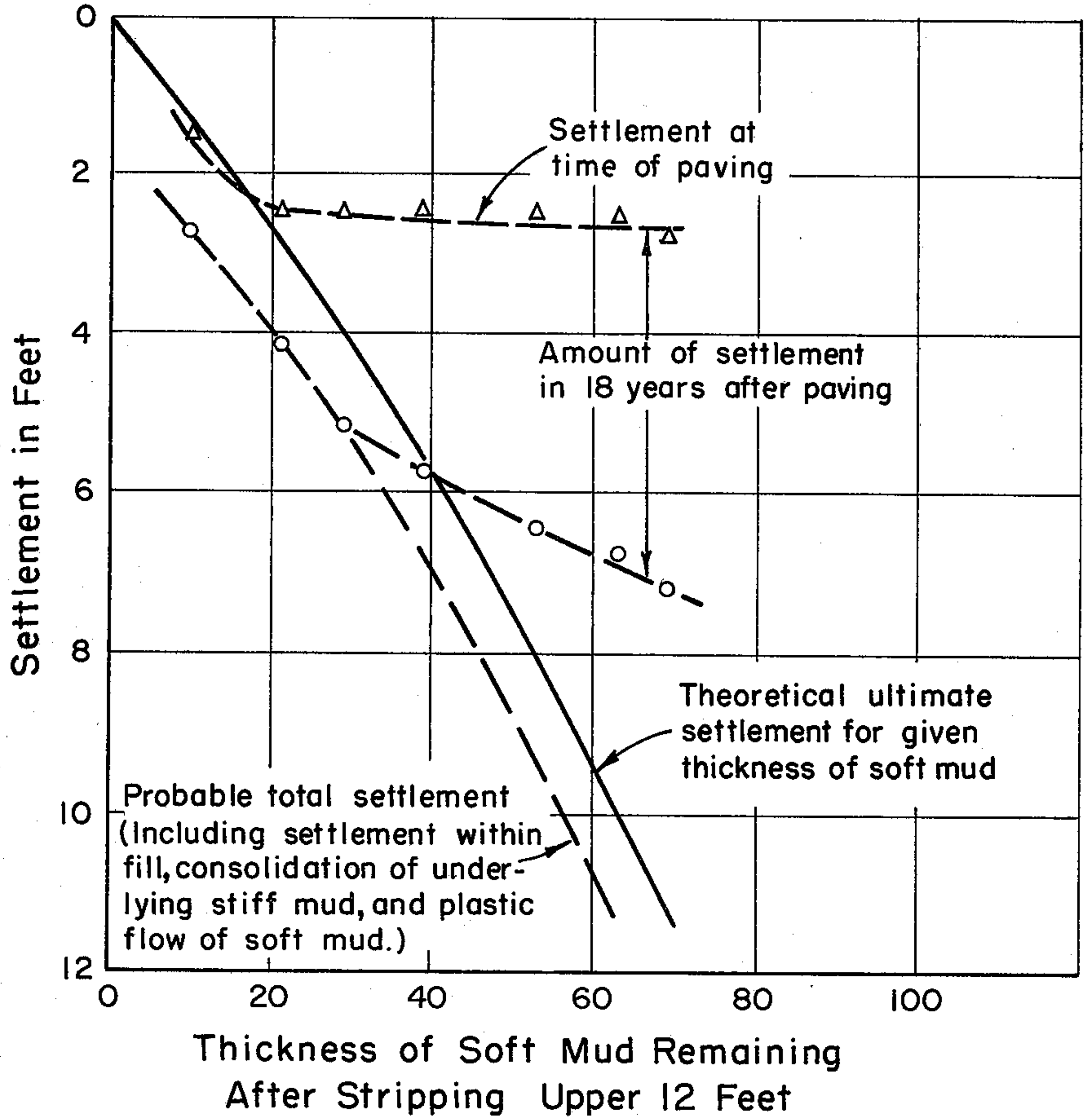


MEASURED RATE OF SETTLEMENT
EAST APPROACH
SAN FRANCISCO - OAKLAND BAY BRIDGE

Figure 2

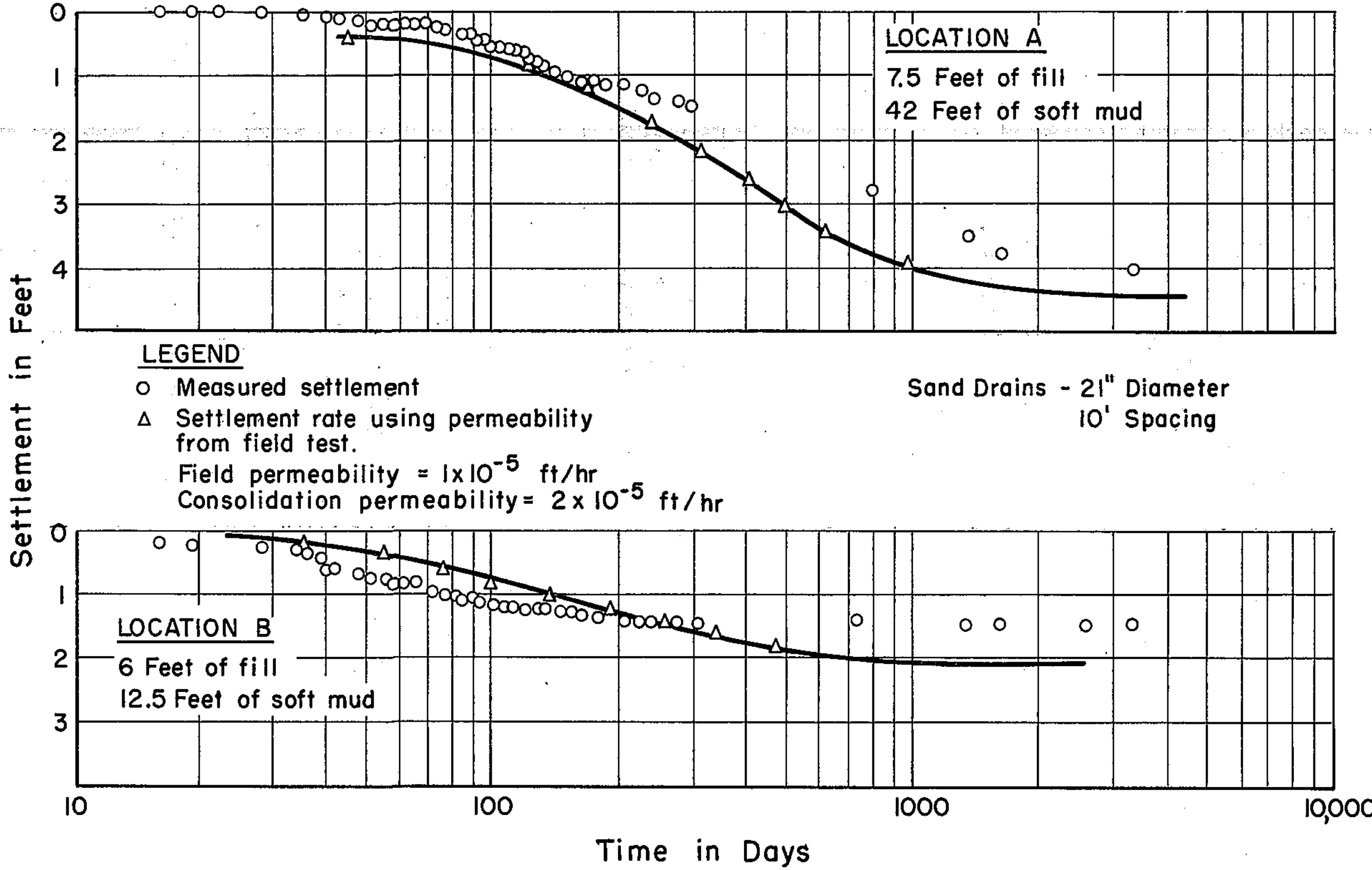


THEORETICAL RATES OF SETTLEMENT
EAST APPROACH
SAN FRANCISCO-OAKLAND BAY BRIDGE



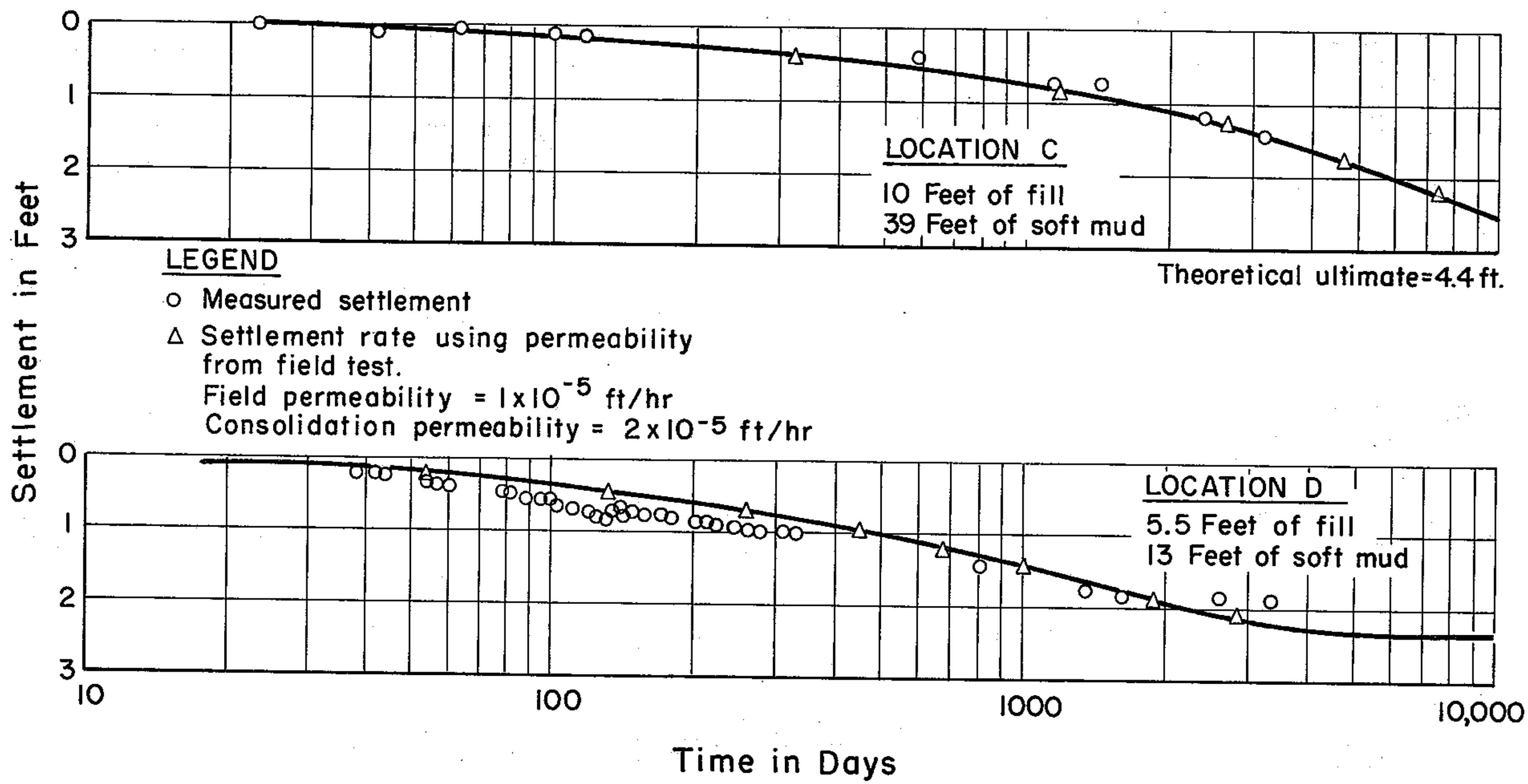
THEORETICAL AND MEASURED SETTLEMENTS
EAST APPROACH
SAN FRANCISCO - OAKLAND BAY BRIDGE

Figure 4



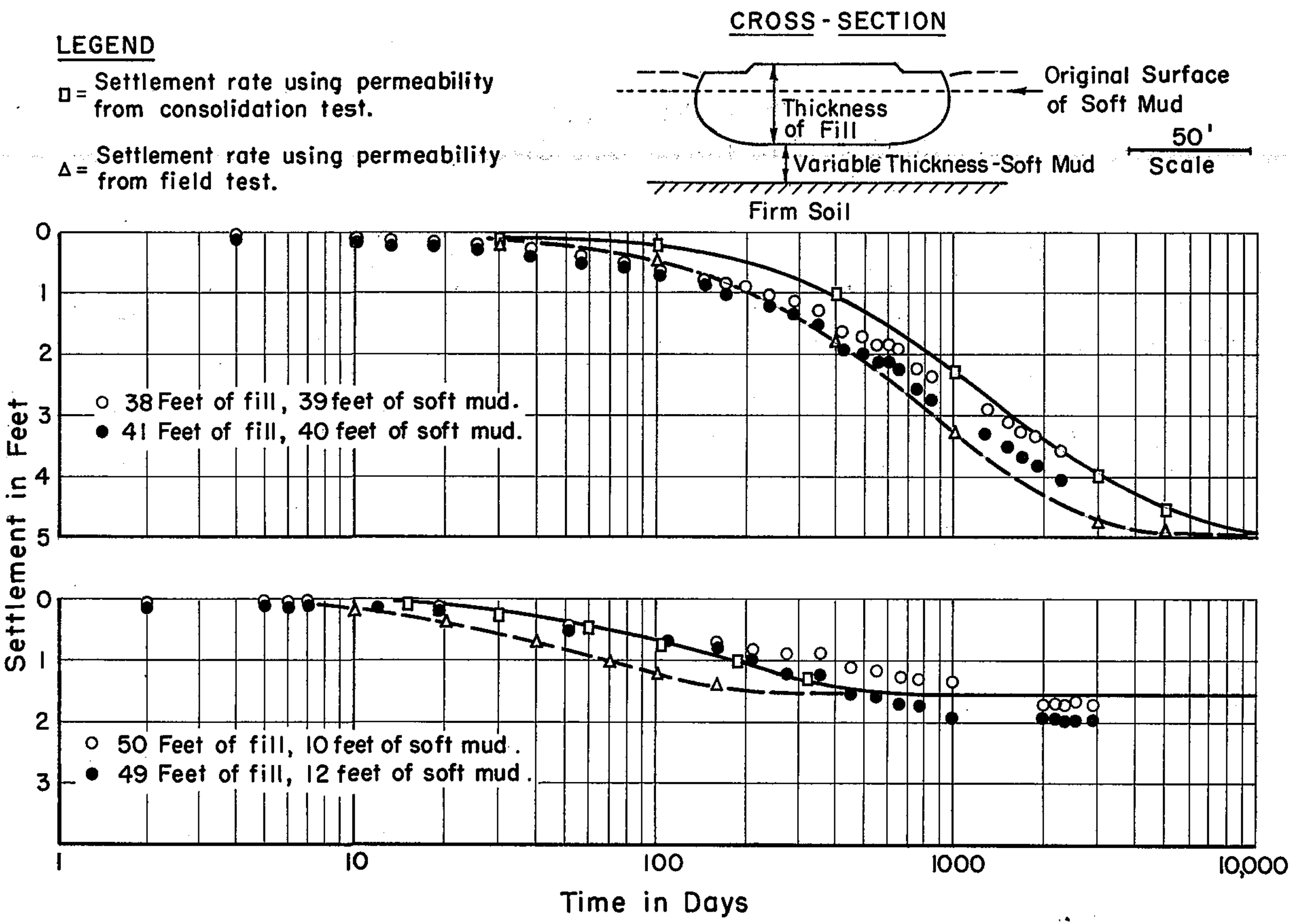
COMPARATIVE RATES OF SETTLEMENT
SAN FRANCISCO BAYSHORE FREEWAY
SAND DRAIN AREAS

Figure 5



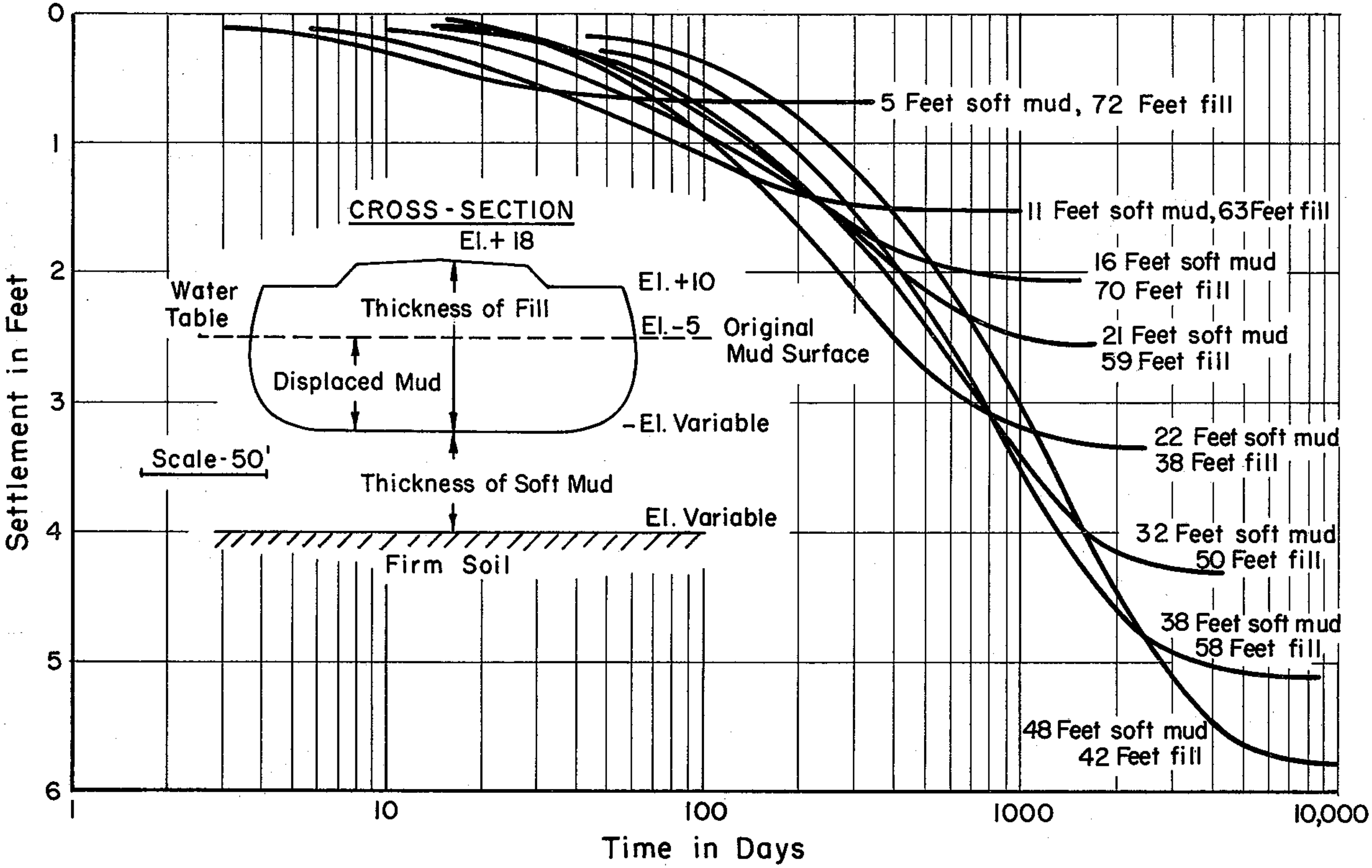
COMPARATIVE RATES OF SETTLEMENT
SAN FRANCISCO BAYSHORE FREEWAY
NON SAND DRAIN AREA

Figure 6



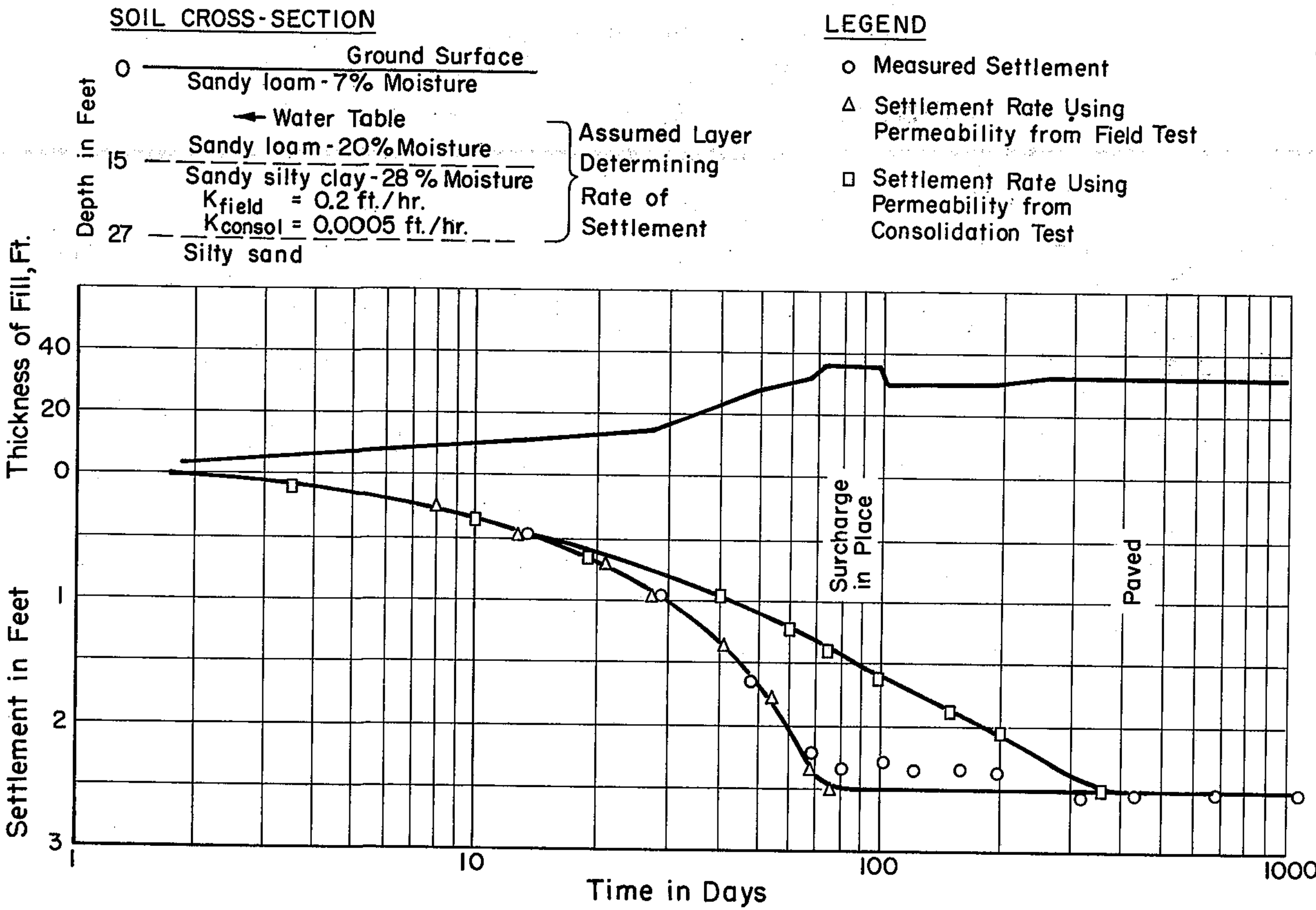
COMPARATIVE RATES OF SETTLEMENT
CANDLESTICK COVE "OPEN WATER" FILL

Figure 7

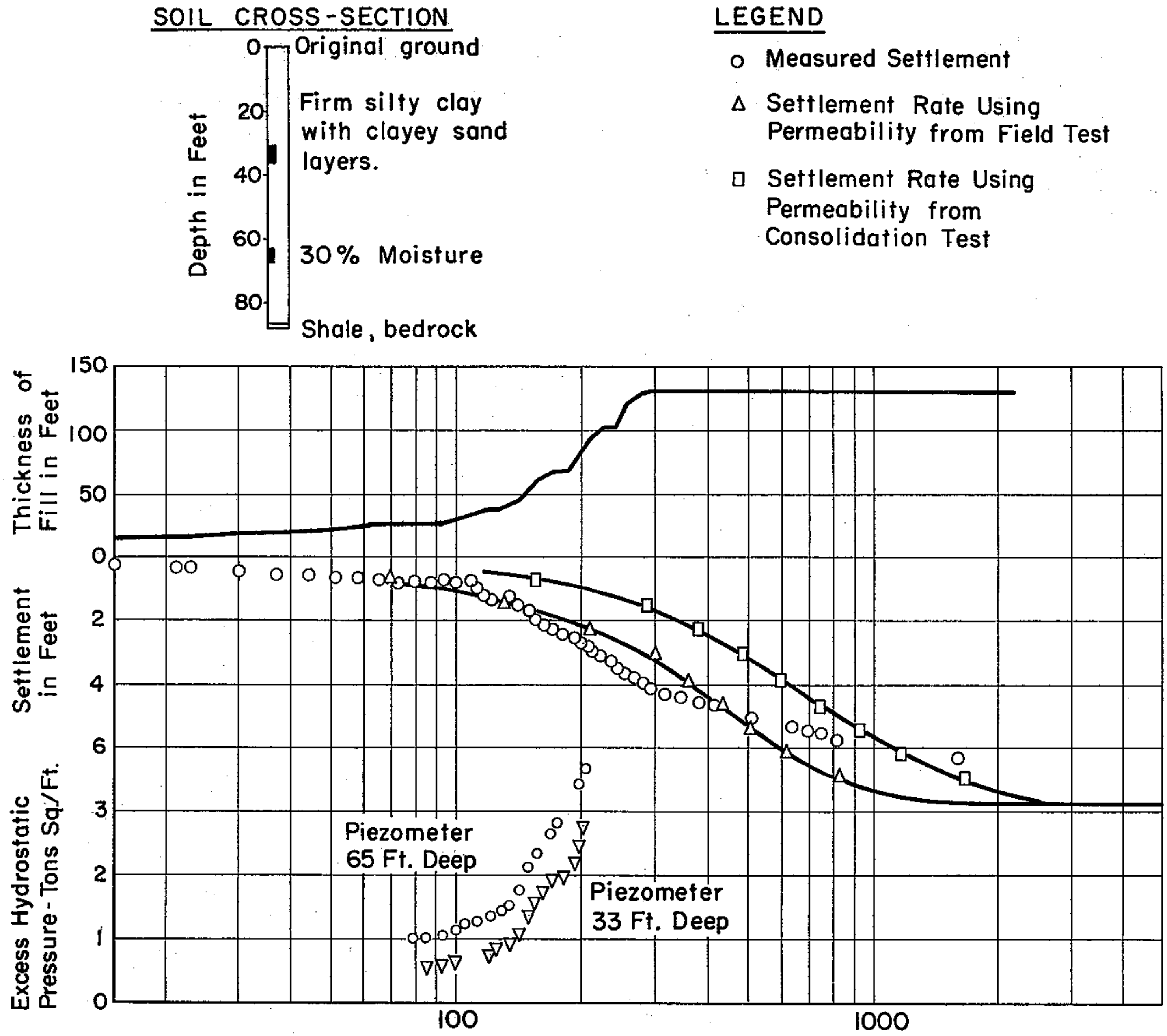


THEORETICAL RATES OF SETTLEMENT
CANDLESTICK COVE
(Assuming Double Drainage)

Figure 8

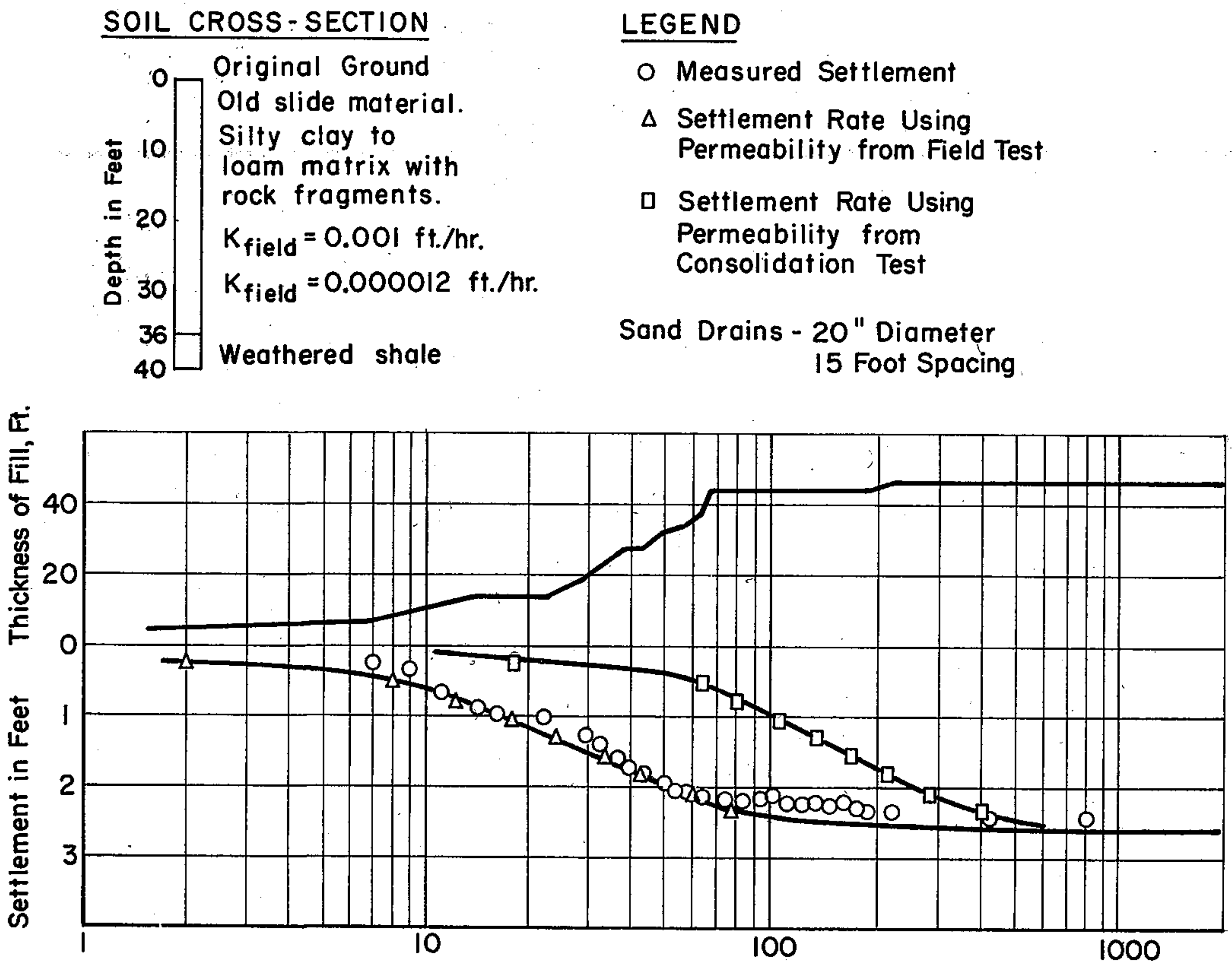


COMPARISON BETWEEN THEORETICAL & MEASURED RATES OF SETTLEMENT
SANTA MARGARITA



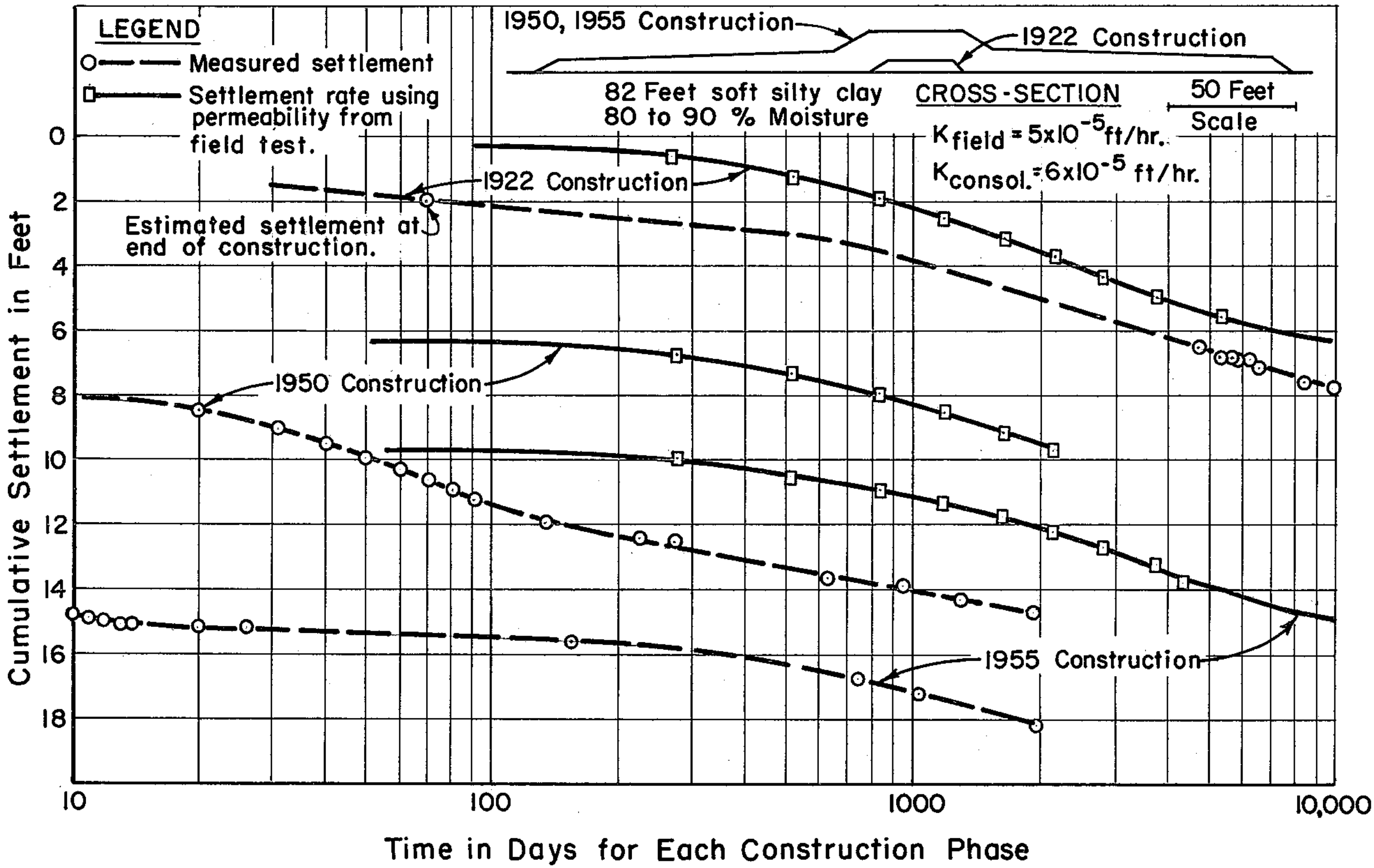
COMPARISON BETWEEN THEORETICAL & MEASURED RATES OF SETTLEMENT - FILL ACROSS TORMEY VALLEY

Figure 10



COMPARISON BETWEEN THEORETICAL & MEASURED RATES OF SETTLEMENT - FILL AT LA TRIANON

Figure II



COMPARISON BETWEEN THEORETICAL & MEASURED SETTLEMENTS
FILL ACROSS TULE LAKE - STA. 316 + 00