Review of Uses of Vertical Sand Drains

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Vertical sand drains have been for the stabilization of soft and compressible soils when the foundation soil is either too weak to support a proposed fill or structure or is so compressible that large and long continued settlements would occur following completion of construction. Credit for the idea of using sand drains for stabilizing soils apparently belongs to Daniel E. Moran, who submitted a patent application for sand drains in 1925 (filed August 5, 1925) which was granted in 1926 (patented August 31, 1926, Patent No. 1,598,300). This patent has been used solely for the protection of the engineering profession. Moran apparently clearly understood the mechanics involved for his patent claimed " . . . the method of strengthening a body of earth which consists in forming drains at numerous points in the area of the mass and compacting the material laterally also at numerous points to force the water out of it by way of such drains."

The first application of Moran's invention to highway fill foundations was proposed by him as a means of stabilizing the mud foundation beneath the easterly roadway approach to the San Francisco - Oakland Bay Bridge. This proposal led to laboratory and field experiments with sand drains by the California Department of Highways in 1933 and 1934, which were described by O.J. Porter in a paper published in the First International Conference on Soil Mechanics and Foundation Engineering in 1936 (1). The success of the laboratory experiments and field test sections led to further use and development of the method, first by the California State Highway Department, and then, in the eastern United States by the Corps of Engineers in 1940 - 1942, and by many state highway departments since that time. The number of sand drain installations which have been made now total about 100 and installations have been made in 16 different states and in several foreign countries. Sand drains have been used for the stabilization of weak and compressible foundation soils beneath earth fills, primarily highways and airfield fills; beneath warehouse floors; within cellular cofferdams; and in earth dams.

The use of sand drains covers a period during which theoretical design procedures were not initially available and in which the design procedures changed from largely an empirical approach to one based upon theoretical concepts as well as past experiences. An equally substantial advancement occurred simultaneously in field installation techniques and equipment. While a large number of records have been published in which sand drains have been successful some installations have not been satisfactory, the number of the latter has not been known and causes of failure have not been analyzed. In order to determine the usefulness and possible limitations of vertical sand drains for the stabilization of soils, the Bureau of Yards and Docks, Department of the Navy, decided to undertake a review to determine what is known and what is not known about the use of sand drains. The results of this review, which has been conducted by the firm of Moran, Proctor, Mueser and Rutledge, will be discussed briefly to the extent that it is complete. The review has included a thorough examination of available theoretical design methods and of experiences in the use of vertical sand drains.

DESIGN METHODS

Design of sand drains refers primarily to determination of rate of consolidation of soft and compressible soils in which sand drains have been installed and which subsequently are loaded by the weight of fill. Secondarily, the shear strength of the soft soil, the rate of gain of shear strength with consolidation and the over-all stability of the fill during its stages of placement are a vital part of design. In the primary design, factors to be determined for a soil profile determined by borings include: diameter and spacing of sand drains, thickness of drainage blanket, rate of placement of fill, amount and duration of surcharge fill loading, amounts of settlement to be anticipated during and subsequent to construction period, and values of settlements or pore water pressures to be used for control of construction operations. In the secondary but equally important part of the design, stability analyses for all stages of the construction operations are required because construction controls should, in most cases, be based on maintenance of stability.

The theory for the primary design of sand drains is based upon an extension of Terzaghi's basic work on the consolidation of clay soils and was largely developed by R.A. Barron during 1940-42. Prior to his work, Rendulic, under Terzaghi's direction formulated and solved the differential equation for consolidation by radial flow to a well in 1935. Carrillo worked on the same problem about the same time as Barron and published his results in March 1942. Barron's work, which was the most extensive, was done independently of the work of Rendulic and Carrillo and was presented in complete form in the 1948 Transactions of the ASCE (2). This paper constitutes a basic reference on the theory of vertical sand drains. In order to determine if the theoretical design procedures are sound, F.E. Richart of the University of Florida made a detailed review for our office of the mathematical theory of consolidation of soils for both vertical and radial drainage conditions. This review found that the mathematical solutions are correct and that the reliability of sand drain design analyses is limited primarily by the closeness with which the assumptions made as to soil behavior agree with the actual properties of the soil. These assumptions and uncertainties include those of Terzaghi's consolidation theory and others such as:

1. The effect of disturbance of the soil caused by installation of the drains on the coefficient of consolidation. This disturbance includes a smearing action of the soil at the surface of the drain plus a disturbance of the soil to some unknown distance from the drain, both of which affect the coefficient of consolidation.

2. The effect of more rapid consolidation, and hence settlement, of soil near the sand drain in causing arching of the overlying foundation soil and fill.

Barron evaluated the effect of a smeared zone of finite thickness having a reduced permeability adjacent to the drain. During the course of our review, Richart developed charts which simplify the use of Barron's analysis in computing the effect of a smeared zone on the rate of consolidation and which show that the effect of smear is to reduce the effective diameter of the sand drain. These charts will be included in a final report to be published by the Bureau of Yards and Docks, Department of the Navy. The assumptions for the thickness of the smeared layer and the coefficient of permeability in the affected zone are still matters of judgment, for our review did not disclose that any satisfactory field data have been developed for evaluating the effects of smear caused by driving of sand drains. Pile driving observations show that serious remolding occurs for a distance of one-half to one diameter outside of a pile. It is, therefore, necessary for each installation to be considered individually, on the basis of judgment and possibly with the benefit of consolidation tests performed on undisturbed and remolded samples, to estimate the effects of smear and disturbance.

Barron also obtained theoretical solutions for two limiting cases: one of no arching, a "free-strain" case; and a second case where arching occurs and redistributes the fill and foundation loads to result in an equal settlement, an "equal-strain" case. Fortunately, the solutions to these two limiting cases do not differ substantially when used for sand drain design purposes with the exception of evaluating piezometer observations.

One of the assumptions made in deriving the theory of consolidation and in the theoretical work by Barron is that the voids of the soil are completely filled with water. Practically, however, many soils in which sand drains are installed contain or evolve gas. It has been observed that open-ended piezometers installed in organic soils have frequently discharged gas which could be ignited and would burn for sometime. Our investigation has developed analyses to indicate the effect of gas on the consolidation process. If the load on a sample of soil containing gas is suddenly increased the gas will compress practically instantly and the sample will, in effect, be partially consolidated under the increment of load even though there has been no drainage of water. As time proceeds, water drains from the soil with the result that part of the load carried by the water is transferred to the soil grains and the hydrostatic pressure decreases. The volume of gas simultaneously increases with the result that the observed settlement is less than the settlement corresponding to the volume of water which is drained during any interval of time. The over-all effect of gas in the soil is to increase the coefficient of consolidation during the period when the loads are being increased and to decrease it after all loads have been applied. The results of analyses of the effect of gas on the rate of consolidation are too involved to include in detail but demonstrate that the effect of gas can be substantial and should be given consideration in evaluating the results of laboratory tests and in analyzing piezometer observations. The effect of gas in the soil has another effect also, which is to reduce the heaving and disturbance caused by a close spacing of sand drains.

SOIL PROPERTIES AFFECTING SAND DRAIN DESIGN METHODS

Coefficient of Consolidation

The most significant soil properties entering into the design of sand drains are the compressibility in the vertical direction and the permeability in vertical and horizontal directions. The coefficient of consolidation for drainage in the horizontal direction, defined as

$$c_{v-r} = \frac{k_r \, (1 + e)}{a_v \quad w}$$

governs the consolidation process and is the most important single soil property in design, but unfortunately it is not readily determinable. Laboratory consolidation tests of the usual type determine the coefficient of consolidation for vertical instead of horizontal drainage. If a consolidation test is performed on a sample which is rotated 90 deg., the direction of drainage corresponds to prototype conditions but the compressibility is determined for differently oriented soil particles and the results may not be comparable to the field behavior of the soil. Normally performed consolidation test results can be corrected to furnish a value for horizontal drainage by: (a) performing permeability tests in the laboratory or in the field for both horizontal and vertical drainage directions; or (b) assuming a ratio between the horizontal and the vertical permeability on the basis of experience and inspection of the soil samples. Field methods of determining the coefficient of permeability in vertical and radial directions are receiving considerable attention and may prove to be reliable if done carefully. Neither method by itself is particularly satisfactory. In many cases the ratio of the horizontal to the vertical permeability has been assumed to be higher than it really was. This ratio is greatly affected by the presence of even thin layers of silt or sand which can be found only if continuous sample borings are made, however, there is a possibility that smear caused by installation of the sand drains can nullify the effect of thin pervious layers. Promising work is in progress at Northwestern University on a new type of consolidation test apparatus which provides directly the coefficient of consolidation for horizontal drainage with vertical settlement and it is hoped that it will soon be practicable to use it in consolidation testing for design of sand drains.

Disturbance

The effect of disturbance of the soil on its consolidation properties is to lower the coefficient of consolidation. The effect decreases with increasing water contents and increasing loads for organic silts and clays with water contents between 37 percent and 98 percent. For these materials and loads at the preconsolidation stress the coefficient of consolidation in the undisturbed state was from 2 to 24 times as great as in a remolded state. The ratios decreased to between 2 and 7 for loads equal to one ton per sq ft above the preconsolidation stress. The variability of the coefficient of consolidation with load, and with disturbance, makes it necessary to use conservative values in sand drain designs. The assumption made in the theoretical analyses that the coefficient of consolidation is constant is satisfactory but approximate. This soil property decreases greatly for loads in the vicinity of the preconsolidation stress but is reasonably constant for greater loads.

Secondary Compression

Many, but not all, soils in which sand drains are installed exhibit large secondary compressions which are not directly related to excess pore water pressures and hence are not accounted for in the Terzaghi theory of consolidation nor in its adaptation by Barron to the design of sand drains. These compressions or settlements occur simultaneously with primary consolidation but continue after primary consolidation is complete. The relative importance of secondary compression is greater on sand drain installations than on most applications of the theory of consolidation. This is partly due to the higher amounts of secondary compressions usually exhibited by soils in which sand drains are installed. An important reason, however, is that the time of primary consolidation is greatly reduced, with the result that the amount and relative importance of secondary compression following completion of primary consolidation is greatly increased. The general concept of secondary settlements is that they are the result of a plastic time lag or plastic resistance to compression but relatively little is known about this phenomenon. Tests have shown that the rate of secondary compression is proportional to the logarithm of time and the amount is directly proportional to the thickness of the compressible layer. The amount of secondary compression is often as high as, or higher than, 0.03 ft per ft thickness of layer per cycle of time. Thus, for a ten year period immediately following a normal construction program, the secondary compression for a fifty foot compressible stratum would be 1.5 ft if surcharge fills were not used or were not maintained long enough. Of this amount, 0.45 ft would occur during the first year after construction and 1.0 ft during the first five years. The second ten year period would, however, show a secondary compression of only 0.45 ft and the third ten year period a settlement of 0.27 ft. These figures illustrate that secondary compressions can be of practical importance for some soils and that repaving may be required fairly soon after construction if surcharge fills are not used.

It is believed on the basis of both field, laboratory and theoretical considerations, that secondary compressions can be largely and possibly entirely eliminated by preloading fills provided that the surcharge load is maintained for a long enough time to reach consolidation equivalent to ultimate under a load which is greater than the final load remaining after the surcharge has been removed. The degree of preconsolidation required to eliminate future secondary compressions can only be estimated approximately at this time.

Stratification

The details of stratification are difficult to determine but have a profound effect on the rate of consolidation. Continuously sampled borings in representative locations are necessary to define stratification. Soils may be stratified in many ways but the effect is similar though the degree of influence may differ greatly. If a soil contains layers of more pervious material, the effect will be to accelerate the component of the rate of settlement which is due to vertical drainage. The effect of more pervious strata depends upon their permeability, spacing, continuity and thickness. If a sand layer is thin, installation of the drains may smear the sand, separating it from the drain and thereby greatly reducing the efficiency of the sand layer in accelerating horizontal consolidation. If a soil is highly stratified, sand drains may be unnecessary, or, if drains are used, may make an accurate estimate of their effect difficult or impossible.

SUMMARY RE DESIGN

The dependence of sand drain design methods upon the Terzaghi theory of consolidation makes it necessary to inquire into the validity and applicability of the theory, especially because of doubts which have been expressed concerning the physical concept that loading causes hydrostatic excess pressures which result in drainage and settlement with time. Our review found that: (a) the mathematical solutions are correct for the basic assumptions made, (b) the physical concepts involved have been recognized and understood by some practical engineers before the mathematical theory of consolidation was formulated, and (c) the applicability of the theory of consolidation to practical work has been verified. The conclusion is reached, therefore, that the theory of consolidation, and sand drain design procedures, are applicable but their accuracy is limited by the degree of agreement between assumed and actual soil properties.

The influence of actual soil properties leads to two important limitations in the use of vertical sand drains and of theoretical methods of design. The first is that sand drains are effective only in accelerating settlement due to primary consolidation and that current design methods apply only to such primary consolidation. Settlements due to secondary compressions are essentially independent of whether or not sand drains are used. The second limitation is that some soils have such an extremely low coefficient of consolidation that sand drains at customary or economical spacings cannot effect a significant amount of consolidation in the short time generally available for construction. Additional limitations involving the sensitivity of a soil to disturbance may exist but cannot be formulated at this time.

The practical significance of the first limitation is that in soils exhibiting large secondary compressions, sand drains will probably not be effective in eliminating future settlements because they are effective only in accelerating primary consolidation. However, if the soil profile also includes very soft soil with a large primary consolidation, sand drains may be effective in increasing the rate of gain of shear strengths and providing stability under loads otherwise not possible. This limitation in the use of sand drains can be made evident from the results of laboratory consolidation tests and design analyses. If soils exhibit mainly secondary compressions with rapid primary consolidation, sand drains should be considered only for increase in shear strengths. Surcharge loading fills may be applicable, if maintained long enough to compensate for secondary compressions under the weight of the final fill. No other specific statement of the applicability of sand drains to particular conditions is needed because any advantages of using sand drains become evident if tests and analyses are made.

The practical significance of the second limitation in the use of sand drains is that: (a) a complete laboratory testing program is necessary to recognize the highly impervious and slow consolidating soils, and (b) whether or not this limitation applies is dependent upon the particular case involved and especially the time available for surcharge loading. In many highly plastic and impervious soils a three or six month loading period is not long enough to accomplish significant consolidation with any practical spacing of drains and magnitude of surcharge loading.

EXPERIENCE WITH SAND DRAINS

Installations Made

In studying experiences with sand drains the records of over 100 sand drain installations have been reviewed and the field performance data from 25 installations have been analyzed in detail to determine past experiences and especially to uncover cases where troubles have been encountered. The greatest single application of sand drains has been to stabilize foundations beneath highway fills. The drain diameters and spacings most generally used are summarized below:

Item	Range	Most Frequent Values
Diameter of sand drains	6 in. to 30 in.	75 percent between 18 in. and 20 in.
Drain spacing:		
(a) "n" values[a]	4 to 42	35 percent between 4 and 6 75 percent are less than 9
(b) feet	6 ft to 20 ft	30 percent between 6 ft and 8 ft 75 percent between 6 ft and 10 ft

[a] $n = \frac{d_e}{d_w}$

The types of difficulties experienced on sand drain installations can generally be grouped as follows:

1. Shear slides during construction.
2. Slow rate of consolidation.
3. Excessive settlements following construction.

The above types of troubles have been experienced in almost every area where sand drains have been installed, but to widely varying degrees. Over a dozen installations have experienced major troubles of one type or another. In some cases the troubles were so serious that the sand drains did not perform any useful function. In other cases the troubles were corrected during construction with satisfactory results. Of particular interest and importance are the experiences at some installations where shear failures took place and where the sand drain system was redesigned, new drains installed, and the fill completed without difficulty.

Shear Slides

The most serious single type of trouble on sand drain installations has been with slides during construction. These slides, which extend through the weak soil in which the drains are installed, have occurred at practically every stage of construction. Slides have occurred when the sand blanket was being placed, during placement of a working mat, as the fill was being placed, and when the fill was almost completed. In some cases the slides were arrested by berms and by halting construction before any apparent damage to the sand drains had taken place. In other cases the slides sheared the drains which became practically ineffective. In several cases very large mud waves developed with heights in the range of half the fill height to one extreme which was even higher than the fill. The damage to the drains by slides was sufficient in several cases to require installation of new drains in the area affected.

The histories of sand drain installations reveal one especially significant fact. Serious slides have occurred only on projects where no stability analyses were made prior to construction. In all cases where adequate stability analyses had been made no slides occurred or they were local and easily corrected or their possibility had been anticipated and control measures established. Minor slides at the beginning of work on several jobs were used as a basis for correcting the assumed shear strength used in stability analyses and the remainder of the work was completed without slides.

There has unfortunately developed an apparent belief among some engineers that sand drains automatically increase the strength of a weak soil at a fast enough rate so that shear failures of the foundation cannot occur. It cannot be sufficiently stressed that the design of a sand drain installation where the stability of the foundation is critical, the usual case, requires not only careful design of the sand drains themselves but also complete stability analyses of the fill and foundation and studies of how the contractor can and should place the sand blanket, install the drains, and place the fill. One result of our review is the conclusion that, while many shear slides have occurred on sand drain installations, the stability analysis methods developed by soil mechanics are satisfactory in preventing this type of trouble.

Slow Consolidation

Several installations consolidated so slowly that the rate of fill placement had to be decreased and the surcharge fill could not be maintained for the length of time anticipated. In reviewing these records we usually found that the spacing of the sand drains and the probable rate of consolidation had been determined from either very crude approximations or the results of previous installations with no use made of the thoretical design procedures developed by Barron. In some cases the design charts in Terzaghi's "Theoretical Soil Mechanics" were used and the engineer was unaware that they had been superseded by Terzaghi's revised charts in an article in Civil Engineerin in October 1945. In other cases the results of using sand drains were predicted before the results of laboratory tests were available and the predictions were not revised. Review of the data available from one unsuccessful installation, designed under severe time restrictions, showed that shear slides should have been anticipated, that consolidation would proceed very slowly and that the scheduled surcharge loading period was so short that little consolidation could result. We have concluded from our review that existing laboratory consolidation tests, despite their shortcomings when applied to the design of sand drains, are still capable of predicting the general rate of consolidation under field conditions. Failure to make carefully performed tests on good undisturbed samples and to use the theoretical design procedures available are the main causes for disappointments in the rate of consolidation.

Excessive Post-Construction Settlements

The settlements following construction have been sufficient at a number of installations to require repaving of highways once or twice within the first ten years of use. The settlements have been so rapid on some jobs that repaving was necessary within two or three years after construction was complete. In reviewing the records and soil information available, it became apparent that the reasons were either that the primary settlement of the foundation was not complete at the end of construction or that high secondary compressions were occurring. When the primary settlement was found to be incomplete at the end of construction, it was apparent from the properties of the soils that this result could have been expected for the time permitted for consolidation. Cases where the field rate of primary settlement was sharply lower than would be expected from laboratory consolidation tests were not found.

Settlements due to secondary compressions are dependent upon whether or not the soil was preconsolidated by surcharge fills to loads in excess of the final fill loads. It was found that settlements due to secondary compressions were high when no surcharge was used and that they decreased as the ratio of the surcharge to the final fill loads increased. At one sand drain installation where no surcharge fill was used, and where the soil conditions consisted of 5 ft of fibrous organic matter and organic silt overlying 20 to 25 ft of soft dark gray clayey silt, primary settlements were complete by the end of construction but large secondary settlements were experienced. Within two years after construction the roadway had to be repaved. This was repeated four years later by which time the maximum secondary settlements had reached nearly one foot.

On another installation a five to seven foot surcharge fill was left in place for over a year and practically complete primary consolidation under it was obtained. When the surcharge was removed, a rebound of 0.1 ft to 0.2 ft occurred but, nevertheless, later settlements occurred following paving which are attributed to secondary compressions. The amount of settlement in a three year period was as high as 0.25 ft and decreased with increasing amount of load removed. This case illustrates the secondary settlements that may occur even with an apparently generous surcharge loading period. While moderate uniform settlements following construction can generally be tolerated, surcharge fills appear necessary to keep secondary settlements to a minimum near bridge abutments and at transitions to hard ground. In a few cases where comparisons of field and laboratory secondary compressions were possible, it was found that the agreement was relatively good. On this basis it is believed that the individual loads in the laboratory consolidation tests should be maintained long enough to define the slope of the secondary compressions and that these data should be used as a guide in estimating probable field secondary compressions. Estimation of the probable amount of secondary compression should be regarded as part of the design procedure although this has not generally been done. Considerable field and laboratory research is needed, however, on the phenomenon of secondary compressions.

Construction Control

The review of cases where troubles have been experienced showed that both design and practical considerations are important and that neither can be slighted. It is especially important that the specifications provide adequate controls for such items as:

1. The normal maximum allowable rate of fill placement.
2. Varying the rate or temporarily stopping fill placement to permit dissipation of

temporarily high hydrostatic pressures.

3. The permissible lift thickness of fill and maximum end and side slopes during placement.
4. Disposition of surcharge fill.
5. Placement methods for sand blanket and working mat, need for mats or casting of material into position for sand blanket and working platform.
6. Control measures such as piezometers, settlement plates and side stakes.
7. Gradation of sand drain fill, sand blanket and drainage windrows, if required.

In several cases where troubles were experienced the specifications were found to have been violated or adequate controls for construction operations were not provided. Proper inspection during construction is probably more essential on sand drain work than on almost any similar construction activity.

Technical Control

Settlement plates, piezometers and side stakes to determine lateral and vertical movements have been used with success in controlling field operations and in checking the field behavior against results of design analyses. This finding is, however, only partially true with respect to the behavior of piezometers. The piezometers commonly used are of either the closed system type with a Bourdon gage or are of the open Casagrande type utilizing a small diameter (3/8 in.) standpipe. The closed system type of which there are several variations, has been the most common.

In a large number of cases the piezometers failed to drop as fast as they should have for the period following the application of all loads. In many cases the piezometer readings remained stationary or actually increased. The causes are not known but it is believed likely that accumulation of gas in and around the piezometer point and riser pipes is responsible. We have not found any cases where the piezometers have been tested to determine their basic time lag and response in the manner recommended by Hvorslev (5). These simple tests reveal the presence of gas in the piezometer system and are recommended for every piezometer installation. In connection with the use of piezometers for determining excess hydrostatic pressures, it should be remembered that the placement of a fill occupying a fairly large area generally raises the normal or static ground water level beneath the fill.

Emergency Corrective Measures

Sand drains are often used under exceptionally difficult soil conditions and despite careful and complete investigations and designs, troubles may result because of variations in the soil profile or soil properties not revealed by the investigational program or because of an effort on the part of the engineer to use the minimum possible factor of safety. In addition, the need for the completed work may require an accelerated construction schedule, or the contractor may have fallen behind in his work. If the above conditions develop, emergency measures of one type or another may be required, and our review has shown that many have been used.

If slides develop during construction, placement of fill should be immediately stopped and stability analyses made to determine the shear strength of the soil at failure. Borings to determine unexpected changes in the subsoil profile may also be required. With this information and revised stability analyses, a decision can be made as to the need for local or general corrective measures. These measures may consist of:

1. A decreased rate of loading.
2. Berms
3. Additional sand drains.
4. Lowering the ground water level in some drains by installing wellpoints.
5. Lowering the hydrostatic pressure in an underlying more pervious stratum, if one exists.
6. Use of vacuum type wellpoints in the sand drains.
7. Accelerating drainage by electro-osmosis.

If a need develops during construction to accelerate the rate of consolidation, this may be done by adding sand drains or by increasing the rate of fill placement and the amount of surcharge to the limit permitted by stability considerations. The permissible rate of filling and amount of surcharge can be increased by adding berms. A surcharge can also be effectively added by pumping from the drains as in methods 4 to 7. Thus, methods exist for improving the behavior of a sand drain installation after it is in operation, if the need arises. The sooner these methods are used the more satisfactory will be the results.

SUMMARY

The review of installations with unsatisfactory performance records showed that the reasons were, in general:

1. Failure to make complete stability analyses.
2. Improper design of the sand drains including determination of anticipated rates of consolidation.
3. Lack of control requirements in the specifications on the contractor's operations.
4. Violation of the drawings and specifications by the contractor, or lax or inadequate inspection.

Of the above causes, a failure to make stability analyses was the most common reason for failures. In a few cases reviewed the limited data available did not permit the conclusion to be drawn that sound design or construction procedures had been violated nor that sand drains did not behave according to the theory of consolidation. Thus, while it cannot be stated that the drainage of water from soil can always be facilitated by using sand drains, it can be stated that for the cases which were reviewed, there is not one instance where sand drains were properly and completely designed, installed and inspected and still failed to effect an increased rate of consolidation.

The results of the review, both of the theoretical aspects and of actual sand drain installations, forcibly demonstrates the need for a thorough initial design and that layout of sand drains on a purely empirical basis is not satisfactory. The initial design should be based on an adequate number of continuously sampled borings, good undisturbed samples, and carefully performed strength and consolidation tests on representative undisturbed samples.

The use of sand drains involves many minor uncertainties which need to be resolved on the basis of full-scale field investigations. These include such items as:

1. Smear
2. Disturbance
3. Permissible sizes and spacings of drains installed by driven closed end mandrels
4. Secondary compression

Item 4 above actually represents a major research study requiring intensive theoretical, laboratory and full-scale field investigations. In the meantime collection and analyses of data from field installations will provide guides for practical design and invaluable basic data for the intensive research.

ACKNOWLEDGMENT

The project summarized in this paper has been carried out by the firm of Moran, Proctor, Mueser and Rutledge, Consulting Engineers, New York, under contract with the Bureau of Yards and Docks, Department of the Navy. The officer in charge of this contract has been Captain A.S. Klay (CEC) USN and the supervising engineer has been L.A. Palmer of the Bureau of Yards and Docks. For the engineers Philip C. Rutledge has been supervising partner and Stanley J. Johnson has been the engineer in charge of the investigation. Theoretical analyses have been made by F.E. Richart, Jr., Department of Civil Engineering, University of Florida, as a part time employee of the engineering firm and as a consultant.

Special acknowledgment is made of the excellent cooperation received from all state highway departments contacted and from several consulting engineers and toll road authorities. The information so received has made possible the review of actual experiences in the use of sand drains.

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Discussion

W.S. HOUSEL, Professor of Civil Engineering, University of Michigan, and Research Consultant, Michigan State Highway Department — This paper has been awaited with a great deal of interest ever since it became known that such a comprehensive study was being made for the Bureau of Yards and Docks of the Department of the Navy. The writer desires to comment on several phases of the subject and compliment the authors on their unusually thorough and objective review of a somewhat controversial subject. It should be noted that their final report will be published by the Navy and will include the detailed data which have necessarily been summarized in the present paper.

The authors' frank recognition of the importance of mass stability is particularly welcome and the following statements under the heading of "Shear Slides" cannot be overemphasized:

> The most serious single type of trouble on sand drain installations has been with slides during construction. These slides which extend through the weak soil in which drains are installed have occurred at practically every stage of construction. . . when the sand blanket was being placed, during placement of the working mat, as the fill was being placed, and when the fill was almost completed.
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> It cannot be sufficiently stressed that the design of sand drain installations where the stability of the foundation is critical, the usual case, requires not only careful design of the sand drains themselves but also complete stability analyses of its fill and foundation and studies of how the contractor can and should place the sand blanket, install the drains, and place the fill. One result of our review is the conclusion that, while many shear slides have occurred on sand drain installations, the stability analysis methods developed by soil mechanics are satisfactory in preventing this trouble.

While heartily endorsing this positive statement of an important basic problem, one cannot help but be somewhat amused by the authors' reluctance to call the underlying phenomenon by its real name, shearing displacement or plastic flow, instead of "secondary compression." For many years the writer has been objecting to the studied efforts of the Terzaghi school of soil mechanics to relegate the basic phenomenon of plastic flow to the role of secondary compression and to pretend that this was one of the unsolved mysteries of soil mechanics. It seems that this paper may present the opportunity to bring the difference in basic concepts to some sort of a climax that may clarify the issues involved.

As the writer sees the situation, the difference originates in the basic idea of the theory of consolidation, which pictures the soil-water system as a two-phase system in which water and soil solids act as separate entities. In this concept Terzaghi first defined and has never changed the view that cohesion was the product of internal pressure created by the surface tension of adsorbed water, with shearing resistance a function of internal friction (1) (2). In discussing this question, it may be well to outline several past discussions that are significant in illustrating the conflict of ideas.

This concept of the function of surface tension led Casagrande in 1932 to the view that a saturated clay, submerged as it generally is below the surface water table, would be reduced to a suspension of solid particles in water without any static resistance to displacement. He voiced this conclusion as follows (3):

> I have tried to illustrate that the whole problem of building foundations on clay boils down to these two simple principles: first, do not disturb the natural structure of the clay: if you do, no human being is able to restore its original strength; second, decide on a certain rate of settlement which you do not wish to exceed and determine that pressure which will cause that rate of settlement; the difference between the building load and the above pressure is the weight of soil which must be removed before erecting the building.
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> A definite bearing value of clay does not exist . . . . The engineer must learn that the kind of questions he asks an expert regarding the properties of a clay underground should not be, "How much load may I put on this soil?" Or, in an apparently more scientific manner, "What is the bearing capacity or bearing value of this clay?" His question should be, "How must I design my foundation so that the rate of settlement under a given building load will not exceed certain limits?"

The writer took issue with this denial of a definite bearing capacity which eliminates static shearing resistance due to cohesion and declared that a definite bearing power of clay does exist (4). In support of this declaration the results of several series of plate loading tests were presented, with subsequent settlement measurements on full-size structures which were successfully designed for a limited settlement on the basis of the loading tests.

The last time the writer heard the statement that plastic clay had no shearing resistance was in another discussion by Casagrande at the Purdue Soil Mechanics Conference in 1937. This discussion was never published, although it would have been useful to document changing views on soil mechanics. Complete recognition of the shearing resistance of cohesive clays has long since ceased to be a matter of debate. All soil mechanics laboratories conduct shearing resistance tests of one kind or another and use the results to compute bearing capacity by some one of a dozen or more available formulas. It seems rather odd, however, that shearing displacement, the Siamese twin of shearing resistance, must still be called "secondary compression." Furthermore, it should be noted that although the view that soil has no static shearing resistance has gone into oblivion, the basic concept of soil as a two-phase system is still with us accompanied by other complexities, some old and some new.

The next episode in the story took place at the International Conference on Soil Mechanics and Foundation Engineering at Harvard University in 1936. It was at that conference that Terzaghi began laying the groundwork for an explanation of a continuing rate of settlement at a substantially uniform rate as a secondary time effect or secondary consolidation. The writer's views were expressed at that time in a discussion, from which the following abstracts have been taken:

> Next, I wish to comment on the subject of the settlement of structures and certain aspects of continued settlement. According to my observations of time-settlement relations, there appear to be two basic phenomena which may be represented by time-settlement curves.

In the first place there is consolidation of the soil due to volume changes which represent the compression of void spaces in the soil structure. In porous soils this part of the settlement may be relatively large, but in well-consolidated materials it may be relatively small. This consolidation will take place over a period of time, which may be four or five years in a large mat foundation, two or three months in the case of a pier footing, or considerably less than an hour for a smaller test area. In clay soils with water filled voids and a relatively high degree of impermeability, I have yet to encounter conclusive evidence that the migration of water through the soil due to applied pressures within yield value of the soil under plastic flow is of more than negligible importance.

After the period of consolidation one of two situations may arise. For a certain intensity of pressure one may say that the consolidation has been complete, the pressure being less than the yield value there is no continued or progressive settlement and the settlement curve approaches a horizontal asymptote. For a higher intensity of pressure the consolidation is also complete but the load is greater than the yield value of the soil and settlement continues. It appears, as mentioned by Dr. Terzaghi earlier in the discussion, that such settlement continues at a uniform rate and the settlement curve approaches a sloping asymptote.

I cannot see, however, anything about this situation new or awaiting explanation by investigators of soil mechanics. This is entirely in accord with the conceptions of plastic substance, outlined, I believe, by James Clerk Maxwell approximately in the middle of the last century. It is not at all surprising that plastic clays follow the laws of plastic flow which are quite well known, in fact it would be surprising if they didn't.

According to these principles, Bingham, Nadai, and others, define a plastic material as a substance which will sustain a certain shearing stress without movement but at a higher stress will be deformed gradually without rupture, the rate of deformation being directly proportional to the stress in excess of the field value.

The determination of yield value in my opinion is the most important factor which practical foundation engineering has to consider. Incidentally this point bears on a question put to the Conference which, so far as I am aware, has not been definitely answered. The yield value according to definition as applied to cohesive soils is the shearing resistance at zero normal pressure assuming, of course, that no dynamic effects are introduced due to rapid load application.

* * *

These examples are not all, but many investigators have uncovered similar evidence. Thus, in addition to consolidation we have with us plastic flow of plastic soils if that be strange.

In the past twenty years there are many times when the subject of plastic flow versus consolidation has been discussed without any significant change in opposing viewpoints. There have also been many occasions within the writer's recollection when failure to look this problem full in the face and recognize shear failures for what they were, has led practicing engineers astray.

It is the overemphasis on consolidation to the exclusion of shearing displacement that has led to a number of notable failures of the consolidation theory to provide reliable control as in the case of the Norfolk Naval Air Station (5). While it may be over-simplification, the writer has attempted to summarize his objection to the theory by the statement that the water cannot be squeezed out of clay without shearing displacement when that moisture is held by the same molecular forces that are the source of shearing resistance due to cohesion.

Although failure to recognize shearing displacement has resulted in many failures it may be that renewed interest in the subject, via the current investigation of sand drains, may succeed in clarifying the relation between consolidation and shearing displacement where other efforts have failed.

One new idea that seems to have appeared in the present paper has to do with the effect of gas on sand drain installations. As stated by the authors the theory of consolidation has always been limited to saturated soils, where it is assumed that the voids are filled with water. In reading the comments on this point, one wonders if the introduction of gas into the discussion is a first step in the attempt to extend the theory of consolidation to unsaturated soils, with further complications in theoretical soil mechanics.

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P. C. RUTLEDGE and S. J. JOHNSON, Closure—Professor Housel's discussion is welcomed because it calls attention to basic differences in the understanding of soil mechanics terms.

Professor Housel is either indulging in a play on words or taking advantage of the differences in understanding of terms when he implies that the authors, and what he calls "the Terzaghi school of soil mechanics", ignore shear strengths and plastic deformations in clays. Life for soil mechanics engineers would indeed be simpler if secondary compression or consolidation could be dismissed simply as a plastic deformation which occurs only at stresses above some determinable yield value as indicated by Housel.

Perhaps the basic difference in viewpoints is in separation of phenomena. The authors, and many of their colleagues in soil mechanics, prefer to separate the phenomena of volume change in soil, which is consolidation, from those of shear deformation and shear strength which do not necessarily involve volume change and which, in clays, do take place without volume change. To clarify these two groups of phenomena, and some soil mechanics terminology, the following definitions are offered:

Primary consolidation is decrease in volume of a soil through decrease in volume of its pore spaces, accompanied by a compression or squeezing out of pore fluid, whether gas or liquid or both. Primary consolidation is independent of shear stresses and occurs under conditions of equal stress in all direction when shear stresses and deformation are zero, although the more common case of one-dimensional consolidation in laboratory tests and in nature does involve some shear stresses and deformations.

Secondary compression is a volume change phenomenon which continues after completion of primary consolidation and is characterized by a straight-line relation between volume change and logarithm of time. The proportionate magnitude of secondary consolidation in comparison with the primary, by which it is invariably preceded, varies with soil type, state of stress and temperature, being a maximum for highly organic soils. However, secondary compression continues after primary consolidation at all magnitudes of stress, with the proportion between the two being approximately independent of magnitude of stress. Secondary compression has been attributed by some research investigation to a plastic readjustment of stress between soil grains and theories have been developed on this basis. However, no theory put forward to date explains adequately all of the physical phenomena, such as effects of temperature, observed in secondary compression.

Shear deformation is the phenomenon of change in shape under the action of stress and invariably requires unequal stresses in coordinate directions; in other words, the presence of shear stresses. This type of deformation can occur without volume change, and in clay soils does. Since shear deformations do not require volume change, which in saturated clays can only proceed at a slow rate, they frequently precede consolidation. In other words, stability or freedom from large shear deformations is a primary requisite in successful foundations and earthworks, including sand drain installations, but it does not preclude subsequent settlements due to consolidation. An example is an earth fill of uniform thickness but of large areal extent completely covering a deposit of soft or compressible soils. Once such a fill is in place no significant shear stresses are created in the soft soil, but settlements of large magnitude can take place due to consolidation. This is the basic concept in the Casagrande paper of 1932, with which Housel takes issue by selective quotation out of context.

Plastic deformation is a restricted part of shear deformation defined in two different ways. In classical mechanics, plastic deformation is that part of change in shape which is not completely recovered upon release of stress. In more sophisticated mechanics it is shear deformation, which occurs gradually under constant stress and hence is a time phenomenon. By the latter definition some materials exhibit plastic deformation under all magnitudes of stress, whereas others deform plastically only when some stress, called the yield value, has been exceeded. It must be emphasized that plastic deformation requires shear stress, but does not require volume change. In fact, in most metals plastic deformation takes place at constant volume.

These definitions make it self-evident that secondary compression, shear deformation, and plastic deformation are not different names for the same physical phenomenon, as suggested by Housel, but are separate and distinct behavior characteristics which must be treated separately if a clear understanding of the behavior of soils is to result.

Professor Housel's quotation and discussion of Casagrande's 1932 paper on "The Structure of Clay in Foundation Engineering" appears to miss completely the point of the quoted material. Two important conclusions that are exemplified by Casagrande's paper are that (a) excessive primary consolidation can occur without exceeding the bearing capacity of the soil, and (b) the bearing capacity can be exceeded before the amount of consolidation is of practical significance. Thus, Casagrande's statements do not deny the existence of a definite or limiting bearing capacity of a soil; his statements only emphasize that, before the safe bearing capacity has been reached, the amount of consolidation that will ultimately be experienced can be and generally is the factor that determines how much load should be applied to a foundation. Thus, " the Siamese twin of shearing resistance, shearing displacement" cannot and has not been called "secondary compression". Apparently Housel does not observe the important difference between the definitions previously given. Unless these differences are observed there is no common meeting ground for factual discussion of the various phenomena involved. This does not imply that secondary compression, shear deformation and plastic deformation are not all related to the existence of shear stresses in the soil. What it does say, however, is that these three behavior characteristics are not one and the same to which three different names have been applied.

Professor Housel flatly rejects the idea that consolidation of a clay can occur without shearing displacement, although he recognizes that this may be an over-simplification. This opinion is erroneous and misleading. The consolidation process should be evaluated on its basic assumptions as a simple matter of water flowing through clay due to hydraulic gradients created by applied loads, ignoring theoretical concepts of molecular forces. The applicability of the theory of consolidation to normal field conditions is dependent upon whether or not decreases in water content are actually found under field loading conditions. They have been found at so many sites that the basic applicability of consolidation theory to field loading conditions has been adequately demonstrated.

Although Housel appears to be trying to discredit the theory of consolidation, in reality he is discussing details about which there is not real disagreement, despite his implications. For example, his statement that "I have yet to encounter conclusive evidence that the migration of water through the soil due to applied pressure within yield value of the soil under plastic flow is of more than negligible importance", merely says, in effect, that for low loads, probably less than the preconsolidation load, the expected settlement and water content decreases are so small that they may be difficult to detect in the field.

Those who use the theory of consolidation do not deny—rather they endorse the concept that settlement, the preconsolidation stress, the applied load, shear deformation, secondary compression, and plastic deformation are related in over-all sense. A relationship between these items is obvious, in that maximum previous loading affects the preconsolidation load, and hence settlements, as well as the existing shear strength of the soil, which affects shear and plastic deformations. These factors are not grouped, but are considered separately by the school of soil mechanics criticized by Housel.

The authors and Professor Housel are in complete agreement on the necessity of considering both shearing displacement and consolidation theory in the design and use of vertical sand drains and for providing field construction control. The authors view this need as imperative and believe that any failure to do so, or to have done so in the past, merely demonstrates an incomplete analysis of a problem and a temporary stage in the application and correct use of soil mechanics to practical problems. As ideas and experiences are exchanged and discussed by engineers the required design procedures to meet the needs of the situation will be formulated and become more frequently employed.