# **Checking up on Vertical Sand Drains**

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**VERTICAL** sand drains, or sand piles as they are sometimes called, have been more and more widely used in recent They represent an attempt to years. augment the consolidation of unstable saturated soils by providing vertical drainage outlets for the water presumed to be squeezed from the soil by the weight of the The objective of consurcharge or fill. solidation by such artificial means is to develop increased soil resistance and support for superimposed loads, usually consisting of earth fills in highway or airport construction.

Whether or not sanddrains are actually effective and do produce the results represented by the theory has been a debated The writer has always been question. skeptical of the theory of consolidation in principle and little impressed with its attempted application in practice. The consolidation theory conceives that settlement is caused by squeezing water out of the voids of a saturated soil under the applied pressure. This theory postulates that the movement of moisture is caused by porewater pressure or a differential hydrostatic pressure as distinguished from the pressure components acting on the soil mass as a whole which results in shearing stress. It ignores the fact that adsorbed moisture in fine-grained soils frequently exceeds the solids in volume and that the same molecular forces which must be overcome in order to force out the water are the source of cohesion or shearing resistance of the soil. Under the situation most representative of saturated soils in which the solid particles are dispersed in a semifluid medium, it has always been difficult for the writer to understand how the water could be forced out without carrying the solid particles with it. In other words, in such soil masses as encountered in practice without artificial confinement, it would seem that displacement or shearing failure would take precedence over consolidation.

The consolidation theory as formulated cannot be applied to unsaturated soils in

which settlement, as observed in practice, differs only in magnitude from that taking place in saturated soils. Some of the uncertainty involved in its application to saturated clays are discussed by its foremost proponents who recognize some of its shortcomings (1). In discussing an apparent lag in the reaction of clay to applied loads characteristic of plastic solids, it is stated in the reference that:

These delays in the reaction of clay to a change in stress, like the secondary time effect and the influence on  $c_V$  (coefficient of consolidation) of the magnitude of the load increment, cannot be explained by means of the simple mechanical concept on which the theory of consolidation is based. Their characteristics and conditions for occurrence can be investigated only by observation.

Its inaccuracy in application to practical conditions is likewise recognized as in the following statement:

It is obvious that the results of a settlement computation are not even approximately correct unless the assumed hydraulic boundary conditions are in accordance with the drainage conditions in the field. Every continuous sand or silt seam located within a bed of clay acts like a drainage layer and accelerates the consolidation of the clay, whereas lenses of sand and silt have no effect. If the test boring records indicate that a bed of clay contains partings of sand and silt, the engineer is commonly unable to find out whether or not these partings are continuous. In such instances the theory of consolidation can be used only for determining an upper and lower limiting value for the rate of settlement. The real rate remains unknown until it is observed.

The extension and elaboration of this uncertain theoretical concept to the design of a roadway fill or airport runway involving sand drains includes the calculation of several definite quantitative components of the completed facility. These include the spacing of sand drains, the height of surcharge required to produce a required degree of improvement, a specified consolidation period, and finally, a completed fill with a definite safe load. To the uninitiated client or to the contractor who bids on the job, the precise evaluation of these items in plans and specifications as quantitative requirements or objectives to be accomplished not only implies but gives assurance that the whole subject of sand drains has been established beyond any reasonable doubt. Critical In the few instances in which the writer has had the opportunity to investigate the effectiveness of sand drains on an actual construction project, the results have been definitely negative. Perhaps these were unfortunate coincidences, but this experience only reinforced some rather



Figure 1. Fill settlement and slide planes at north side runway Stations 45-47.

examination of such evidence as has been presented indicates that application of the principles involved has scarcely progressed beyond an experimental stage and that the claims of successful application are seldom supported by more than fragmentary and inconclusive evidence.

Occasionally one hears of failures in connection with sand-drain installations. and if these failures are exhaustively investigated, they are seldom, if ever, so reported for public consumption. Accounts of supposedly successful installations which are more numerous usually presume that the lack of complete subsidence or failure has been accomplished by the sand drains. Generally, these claims are not supported by a factual stability analysis to demonstrate that the original bearing capacity was clearly deficient or that there was a definite improvement due to consolidation. Settlement is classified as consolidation settlement, but no clear demonstration is given as to how much was displacement and how much consolidation. The writer has yet to see in a published report a comprehensive set of reliable factual data on moisture content, soil density, or shearing resistance before and after consolidation from an actual full-scale sand-drain installation.

strong convictions expressed in this discussion with regard to the theoretical background of consolidation. It is possible that there are materials and stratified soil deposits in which vertical sand drains are of material benefit. The writer has not encountered such conditions and suspects that if they do exist, they are the exception rather than the rule.

Having a deep interest in the practical value of sand drains, the opportunity to conduct a comprehensive investigation of the results produced by an installation of sand drains on a major project came as a distinct opportunity. The project with which this investigation was concerned was the extension of a runway into a marshy area at the Norfolk Naval Air Station. The investigation was made for the Contractors. Ralph E. Mills Company of Salem, Virginia, and Blythe Brothers Company of Charlotte, North Carolina, and was made with the permission of the representatives of the Bureau of Yards and Docks of the U.S. Navy. The objective of the investigation was to determine the reason for a number of difficulties which had been encountered during the construction. This determination boiled down to whether or not these difficulties were the result of improper construction methods or were inherent in the design as presented by the



Figure 2. Fill settlement and slide planes at north side runway Stations 45-47.

plans and specifications.

The difficulties referred to which presented some serious problems to both the Navy and the contractor may be described briefly as follows:

First, due to slides and displacement of the softer strata near the surface, the amount of fill material being required was mounting up to an excessive overrun involving increased cost beyond the funds which had been provided for this purpose.

Secondly, the slides and displacement destroyed the settlement platforms which had been placed for the purpose of determining the final volume of fill and made it necessary to establish some other method of determining the final pay quantity. In other areas, the displacement of the soft materials took place as fast as the sand fill was placed, and it had been impossible to establish a working platform from which the settlement platforms could be placed.

Third, in other, more-stable areas, the contractor had been able to place the working platform, drive the sand drains, and place part and sometimes all of the surcharge called for by the plans and specifications. However, in several of these areas failures developed producing the most-extensive slides and mud waves that were encountered in the project. It should be noted in connection with this problem that the plans called for the full surcharge to be placed immediately after the sand drains



Figure 3. Mud waves on taxiway.



### Figure 4. Mud waves on taxiway.



LOCATION PLAN

Figure 5.

had been completed and provided for a 90day consolidation period. In such areas where it had been impossible to place the surcharge fill without serious failures developing, the contractor was forced to place the fill in small lifts which were then left in place until it was presumed that the consolidation had been completed, after which the rest of the surcharge was placed.

Fourth, aside from the delays and increased cost which somebody would have to bear, the continued excessive displacement

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raised a serious question as to whether there was any consolidation being accomplished by the surcharge fill and whether a useable runway could ever be produced under the procedure called for by the plans and specifications.

Fifth, in certain areas after a consolidation period of from 2 to 5 months, it was presumed that the objectives of the design had been accomplished and the contractor was instructed to remove the surcharge and blanket on top of the sand piles. This removal was also required in the final operation when the sand surcharge was to be removed and spread on either side of the finished runway and taxiway to form the final cross section. It had been the contractor's experience that the continuous removal of this mud wave in order to maintain temporary ditches continued to promote the excessive fill settlement and displacement. The question which this





spread it in flat slopes on either side of the runway and taxiway. However, after the surcharge had been removed, it was found that the settlement continued in the runway at such a sufficiently high rate that there was some question as to whether the contractor would be able to maintain final grade for the paving operation within the tolerances laid down by the specifications.

Sixth, during the entire operation the contractor was required to remove the mud waves which had piled up on either side of the runway and taxiway and which were furnishing a counterweight tending to stabilize the fill. This removal was being required in order to provide temporary drainage to carry away the water which was supposed to be coming out of the porous raised was whether or not this counterweight could be removed at any time without setting off a new series of slides.

To give some idea of the character and magnitude of the fill settlement and mud waves, there are presented the first four figures, photographs taken in the field in the early part of September 1951. Figure 1 and Figure 2 show the failure areas which were first encountered toward the west end of the runway. Figure 3 and Figure 4 show very clearly the mud waves caused by displacement of the soft clay during the placing of the working platform and the driving of the sand piles along the taxiway.

Obviously the most-essential part of an investigation of the enumerated problems was a comprehensive soil investigation,





Figure 8.

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including borings and all of the laboratory tests necessary to fully describe existing soil conditions. The original boring logs, which served as a basis for preparation of the plans and specifications, were available to the contractor being incorporated in the plans. Available data included moisture content in percent by dry weight, the dry weight per cubic foot in a general description of the soil. The predominant material was described as a gray silty clay, but there were several areas with sand or sandy soil both near the surface and at greater depths.

As a part of the contractor's investigation, there was a total of 45 check borings made, 11 of which were deep borings with undisturbed samples being taken for testing at the Soil Mechanics Laboratory at the University of Michigan. The remaining 34 borings were made primarily to determine the depth of penetration of the sand fill. The borings and undisturbed sampling were done by the Raymond Concrete Pile Company under the Michigan specifications requiring core samples 1% inches in diameter and approximately 7 inches long. A complete description of the sampling procedure is available elsewhere and will not be repeated in this discussion (2).

Figure 5 is a location plan which shows the general layout of the runway and taxiway extension and the location of borings and critical soil areas, which will be discussed further. The open circles along the centerline of the runway and taxiway are used to designate the original borings made prior to preparation of the plans and specifications. The solid circles designate the 11 check borings made as part of the contractor's investigation from which undisturbed samples were obtained. Borings 1, 2, 5, and 6, along the centerline of the runway, and Borings 8, 9, and 11, along the centerline of the taxiway, were taken in areas where the surcharge fill of varying height had been in place for consolidation periods varying from 2 to 5 months. Undisturbed samples of the saturated clay in these areas are presumed to represent the condition of the soil after it has been subjected to the prescribed consolidation periods. Borings 3, 4, and 7, in between the taxiway and runway in the open marsh area, were taken for the purpose of obtaining representative samples of the original soil in its unconsolidated state.

Boring 13 was taken outside of the runway area to compare with Boring 1 at a point where the soil conditions varied substantially from those over the greater portion of the runway and taxiway extension.

Selection of the location of the check borings was predicated on an analysis of the soil conditions as portrayed by the From these original original borings. data, areas which were presumed to represent the more-critical soil conditions were selected and have been designated on Figure 5 as soil area 1. In this area, the soil test data from the original borings indicated that there was gray silty clay of high moisture content from the surface of the ground to depths varying from 50 to 60 feet. This soil was originally described as very soft to soft silty clay with moisture contents varying from 65 to 95 percent by dry weight and 60 to 80 percent by volume. Tests performed as a part of the contractor's soil investigation indicated that the predominant soil was a silty clay containing 30 to 35 percent clay and colloids, 30 to 50 percent silt and 20 to 30 percent sand. The plastic limit of this material varied from 20 to 30, the liquid limit from 50 to 60, with an average plasticity index of approximately 30. In general, the natural moisture content of this material varied from 10 to 20 percent above the liquid limit, indicating that the soil should be classified as a flocculated clay.

In the vicinity of Borings 1 and 13, sand, substantially granular in character, was found from the surface to a depth of 20 to 25 feet, followed by a relatively thin body of the very soft silty clay approximately 15 feet in depth, underlain again by a body of sand which continued for the full depth of the boring. In the vicinity of Boring 5, similar conditions existed, except that the depth of the soft silty clay between the surface sand and underlying sand was approximately 35 feet, somewhat more critical than in the case of the other sand area but presumably more stable than the critical areas designated as Area 1. Practically the entire length of the taxiway has been designated as a critical soil area with the soft silty clay extending from the surface to a depth of 60 to 70 feet. Near the east end of the widened taxiway in the vicinity of Boring 8, the same classification of soil is found in the original borings although the natural







moisture content is substantially less than in the highly saturated clay indicating a more stable soil.

There is also shown (Fig. 5) the areas in which the major slides and displacements were encountered. In general, these areas agree with the classification of critical soil areas based upon the original soil test data and classification. This includes a major slide area in the vicinity of Boring 2, another similar area in the vicinity of Boring 6 and a continuous area of subsidence and displacement along the major portion of the taxiway from near Boring 9 to past Boring 11.

As has been previously indicated in this discussion, the primary objective of the contractor's investigation was to obtain comprehensive, factual data on the moisture content and shearing resistance of the saturated clay soil in the critical soil areas before and after consolidation. This information was required in the first place to determine whether or not the unsatisfactory performance was inherent

in the existing soil conditions and the procedures called for by the plans and specifications and whether it was physically possible for the contractor to accomplish the objective set forth in these plans and specifications. Having accepted a contract to produce a completed facility planned by others, a contractor is frequently in the unfortunate position of having to accept the burden of proof when faced with the possible contention that the failure to achieve the planned objective is a matter of construction methods and his legal responsibility. Furthermore, from the standpoint of everybody concerned, the situation had reached a stage in which it was highly essential to establish whether it was physically possible to provide a completed runway which could be safely used by the Navy over a reasonable period of years.

The results of the tests to determine the degree of consolidation which had been produced by the sand piles and the surcharge fill which had been in place over periods varying from 2 to 5 months are

shown on a series of graphs presented by Figures 6 to 12 inclusive. Figures 6 through 11 present the moisture content before and after consolidation for individual borings. In Figure 6, the boring log from the original boring is shown on the left-hand side of the figure and the log of the check boring is shown on the right-hand side. At the top of the righthand boring log is shown the surcharge grade, both as designed and as measured at the time the boring was made. The thickness and bottom elevation of the sand fill on the check boring can be compared with the ground-surface elevation on the original boring to indicate the depth to which the sand fill has settled. In Figure 6, the indicated settlement is approximately 4 feet.

The primary purpose of Figure 6 through 11 is to present the moisture content measurements in the central chart. Moisture contents and solids are shown in percent of the total volume with the moisture or liquid volume being shown on the right as the complementary percentage of the volume of solids which is shown on the left-hand side of the chart. Results from the original test data are shown by the open circle, the results from the check boring by the solid circle. At the location of Boring 2 in Figure 6 in the original soil below the sand fill, moisture contents vary from 60 to 70 percent with the volume of solids varying from 30 to 40 percent. The moisture content in the sand fill is approximately 20 percent, corresponding to the percentage of voids in such a granular material when compacted. Particular attention is directed to the fact that there is no measurable change in moisture content for the entire depth of the original soft silty clay under discussion.

It may be noted, however, that the textural classification or description of this soil does vary from the original description of a soft, gray, silty clay to the textural classification of gray silt shown in the check boring. In the latter case, the descriptions of soil type are from visual examination by the boring crew superintendent. It is the practice in the Michigan laboratory not to alter the soil



Figure 10.

description given by the boring contractor but to make a separate classification based on the mechanical analysis where such classification is required. As previously noted, the laboratory classification of the predominant saturated soil was a silty clay in substantial agreement with the original classification. All through the apparent decrease in moisture content after the period of consolidation in the region from Elevation 0 to Elevation -10. However, this apparent decrease in moisture content is clearly related to the textural differences which are shown due to the subsidence of the surface layer of sand under the surcharge fill.



#### Figure 11.

comparative tests in the six figures under discussion, similar textural variation will be in evidence and where these variations are of any particular significance, special comment will be made.

Figure 7 is a similar chart for results from Boring 5. This area has not been classified as soil Area 1 although as previously noted the thickness of soft silty clay beneath the surface stratum of sand is sufficient to raise some question as to the ultimate stability of this area. It may be noted that the height of surcharge as planned and measured is much less than in the previous case and the amount of settlement is somewhat less being only a little over 3 feet. Again there is no measurable change in moisture content in the silty clay at greater depth but there is an

Boring 6 shown in Figure 8 is in another critical area where original soil conditions indicated more than 50 feet of unstable soil starting at the surface. There is, in this case, a marked difference in moisture content before and after consolidation, but there is also a marked textural change in the check boring. The reasons for this are quite clear, as the coarse, granular material encountered at the greater depth is obviously that used in the sand piles. Thus, the check boring happened to strike such a pile for part of its depth. At the only depths below the penetration of granular fill where the original soil appeared to be present, moisture contents are quite comparable before and after consolidation.

Boring 8 in Figure 9 is in the middle

portion of the widened taxiway and was designated as less critical based on original soil test data. In the check boring, the textural classification is even moredefinitely granular near the surface. This particular boring was beyond the general failure area along the taxiway and the check boring indicates a smaller fill settlement. Moisture contents before and after consolidation are remarkably close together for the entire depth of the borings, even though the soil is more from above by subsidence of the fill. On this boring between Elevation 0 and Elevation -15 is the only evidence of decreased moisture content after consolidation which is not clearly accounted for by a difference in soil texture. However, such a result could very easily be produced by the vertical displacement from above of soil having lower moisture content. Unfortunately, no tests were reported from the original borings above Elevation 0, although the trend toward lower moisture





granular and presumably more permeable.

Boring 9 in Figure 10 is at the point of transition from the wide to the narrow taxiway and is an area designated as critical and where fill failures were general. In such areas, borings made along the centerline were generally in a portion of the soil mass and fill between the slide planes from either side of the taxiway. Thus, they do not represent the deepest penetration of the fill in the actual slides but may be an area in which a mixture of original soil and fill has been carried down contents near the surface is indicated. Below Elevation -15 for the balance of the depth in the saturated clay, moisture contents are the same before and after consolidation.

Boring 11 in Figure 11 is midway between two of the original borings in one of the more-critical areas along the taxiway. Fill settlement is approximately 15 feet. Below the fill in the depth of soft saturated soil above the underlying sand, moisture contents are in the same range before and after consolidation, although it must be kept in mind that the original borings are somewhat removed from the check boring. Any differences in moisture contents at the greater depths are definitely related to textural changes, with the underlying sand being encountered at varying elevations in the three borings compared.

Figure 12 is a composite of the data obtained from the individual borings in Figure 6 to 11 presented to represent the same comparison of moisture content before and after consolidation on the project as a whole. Moisture contents are shown for the original borings by open circles. Moisture contents from check borings made outside the filled area are shown by a circle with a cross and from check borings made within the filled area after the period of consolidation are designated by a solid circle. An envelope has been drawn indicating the range of moisture content in the original borings. With the exception of the sand fill and in the granular materials underlying the soft, clayey silt, moisture contents after consolidation fall within this envelope with no indication that there has been any significant change in moisture content before and after consolidation.

Even more than moisture content, the shearing resistance of the soil before and after consolidation should provide direct evidence of the degree of consolidation produced by the sand drains and surcharge fill. If shearing resistance tests were conducted in connection with the original borings. these data were not made available to the contractor. Consequently, as previously noted, Borings 3, 4, and 7 were made in the open marsh area between the runway and taxiway. Shearing resistance tests were conducted in the laboratory on samples from borings used to represent the original soil condition and from the borings taken from the filled area to represent shearing resistance after consolidation.

Following the customary practice at the Michigan laboratory, two types of shear test were conducted on all samples: (1) the ring-shear test developed by the University of Michigan Soil Mechanics Laboratory and (2) the unconfined-compression test used by a number of other soil mechanics laboratories.

The ring-shear test is a measure of what may be called the static yield value or shear stress greater than which the soil will suffer progressive deformation. In these tests, observations are made of the rate of shearing deformation for each load increment applied. From this may be determined, by extrapolation, the actual load at which progressive deformation occurs. The final results are thus independent of dynamic resistance and represent that applied stress which may be sustained in static equilibrium.

The shearing resistance, as determined by the unconfined-compression test conducted in accordance with generally accepted procedures, has not been corrected for dynamic effects, so the test may be termed a rapid shear test. The load is applied at a continuous rate until failure is produced and in a much-shorter period of time (5-minute loading period). Over several thousands of parallel tests, it has been found that shear values obtained from these unconfined-compression tests are close to four times those obtained from the ring-shear tests in the case of plastic clays of the truly cohesive type or in the case of saturated clays. It has also been found that for stiff clays and for materials which have granular characteristics the ratio of the shear values obtained from these two tests is quite erratic and, in general, higher than in the case of plastic clays. In presenting the results from the unconfined compression, it has been the practice to plot these shear values to a scale four times greater than is used for the ringshear test, which brings the divergent results into close focus in most cases. In reporting the shearing resistance from unconfined compression, it has also become common practice to report the equivalent shearing resistance to the reduced scale or a quarter of the measured value in the rapid shear test.

In connection with the shearing resistance test on this soft, silty clay, one other observation should be made. In the borings outside the filled area, the equivalent shearing resistance from the unconfinedcompression test runs generally lower than the yield value determined by the ringshear test, a characteristic which has been found common to flocculated clays of high moisture content that have not been disturbed or remolded. While the writer can offer no well-supported explanation of this phenomenon, it appears to be related to the sensitivity of the clay and the fact that, after remolding, the highly sensitive clay seldom, if ever, regains its original shearing resistance. While this departure from

the usual ratio between shear values from these two tests is apparent on most of the samples from those borings, it does not hold for the samples from borings within the filled area, where the results from in place for periods of time varying from 2 to 5 months. Shear profiles of these two groups of borings have been combined to form a composite shear profile from which average shear values may be determined as



#### Figure 13.

these two tests is close to the usual ratio of 4 to 1. In this connection, it may be noted that in borings under the runways the soft, silty clay may have been remolded by pile driving and by displacement under the weight of the overload fill, from which it might be presumed that the unusual result obtained in the case of the undisturbed flocculated clay is a function of a flocculated structure.

In connection with the contractor's investigation, Borings 3, 4, and 7 have been selected as representative of soil area 1 in its original condition before it has been altered by construction operations. Borings 2, 9, and 11 have been selected as representative of the same soil area under the runway and taxiway after the sand piles have been driven and the overload has been representative of the soft silty clay under discussion.

The comparison of shearing resistance before and after consolidation in terms of the transverse shearing resistance or ringshear test is shown in Figure 13. Composite boring logs are indicated on the left for Borings 3, 4, and 7 and on the right for Borings 2, 9, and 11. The composite shear profile for the original soil in Area 1 is shown on the left-hand portion of the central chart and the composite shear profile after consolidation is shown on the right-hand portion of the chart. Average shear values at several different depths have been indicated by the heavy dashed line. These shear values are shown to the nearest 5 lb. per sq. ft. and are based on the arithmetic average of shear test results for the

silty clay only. All shear test results are shown on the charts including those from samples of sand and in the case of Boring 7, a more-highly consolidated chalky silt found only in that boring around Elevation -30. Such unrepresentative and exceptional test results were not included in the average shear curve shown by the heavy dashed line.

For the purposes of the present discussion, a comparison of shear values must be limited to the original soil below the depth to which the sand fill has penetrated and which has not been displaced by the superimposed load. It should be noted in Borings 3, 4, and 7 on the left-hand portion of the central chart that the uppermost layer of clay down to approximately Elevation -10 has almost negligible shearing resistance and has, in all probability, been almost completely displaced by the sand fill. This soft, silty clay sometimes would not even support the weight of the small steel ring in the ring-shear test (which produces a shearing stress of approximately 4 lb. per sq. ft.) and this is the value that has been reported for this material in a number of the tests. In the case of the unconfined-compression test on this softest clay when the 3-inch samples were extruded from the liner, the small cylinder slumped under its own weight and it was impossible to apply any compression load.

**Omitting comparisons in the softest clay** strata above Elevation -10, which has largely been displaced by sand fill, it may be seen that the average transverse shearing resistance in those borings representing the original soil conditions varies from 130 to 195 lb. per sq. ft. within the depths at which these tests were performed. The range in the average shearing resistance for these same depths from the borings taken in the filled area after consolidation is from 45 to 125 lb. per sq. ft. indicating a measurable loss in shearing resistance, due presumably to the disturbance caused by pile driving or other construction operations and not recovered during the consolidation period.

Figure 14 presents a similar comparison based upon the results from the unconfinedcompression test. In this case, the unconfined - compression test on samples from the borings representing original soil conditions and undisturbed clay gave an equivalent shearing resistance from 85 to

160 lb. per sq. ft. In those borings from the filled area after the consolidation period. the equivalent shearing resistance from unconfined compression varied from 55 to 110 lb. per sq. ft., again indicating a definite loss in shearing resistance, presumably due to remolding or other disturbance during the construction period which was not recovered by consolidation. It should also be noted that the equivalent shearing resistance from unconfined compression is measurably less than the transverse shearing resistance in Borings 3, 4, and 7, representing the original soil conditions and undisturbed clay, but this variation was not established by a similar comparison for the remolded soil.

While the direct comparison between moisture contents and shearing resistance before and after consolidation were the primary objectives of that part of the investigation being presented in this paper, there were several other subjects studied in the contractor's investigation which are of passing interest in the present discussion. A stability analysis was made to determine the overload ratios imposed on the original soil mass by the proposed heights of surcharge called for by the plans and specifications. This analysis indicated that the overload ratios with respect to the uppermost layer of softest clay were so high that it was quite obvious that complete displacement could be anticipated before the full surcharge could be placed. Overload ratios were computed only for Area 1 which had been judged to be the most critical from preliminary investigations. However, such critical classification covered the major portion of the length of the taxiway, and a considerable portion of the length of the runway. In these areas, the marked correlation between high overload ratios, failure areas, and points of maximum settlement left no doubt as to the source of excessive settlement and subsidence.

## CONCLUSION

In summarizing the results of the soil investigation made by the contractors, the following conclusions are presented:

1. Laboratory tests on undisturbed samples from borings in the critical failure area, designated as Area 1, identified the soft, silty clay extending from the surface to a depth of approximately 60 feet as a flocculated clay of high sensitivity which





has been found to suffer a loss in shearing resistance upon being disturbed or remolded.

2. Shearing-resistance tests on undisturbed samples from borings outside the runway area representing the original soil indicated that the yield value or static shearing resistance based on the average of three borings varied from 30 to 195 lb. per sq. ft. in ring shear and from 25 to 160 lb. per sq. ft. in the unconfined-compression test. The flocculated clay on top in several cases was so soft that shear values were not measurable.

3. In the group of three borings taken along the centerline of the runway and taxiway in soil Area 1, the softest clay on top had been largely displaced by the sand fill, and the shearing resistance in the lower strata, which had been disturbed and remolded by driving of the sand piles, was definitely less than in the original state, even after having been subjected to periods of presumed consolidation by the overload fill in place for periods of 2 to 5 months. In this case, the static shearing resistance, or yield value, from the ring-shear test, based on the average of three borings, varied from 45 to 125 lb. per sq. ft., as compared to 130 to 195 lb. per sq. ft. in the undisturbed or original condition.

4. Comparison of moisture contents from the present investigation between the undisturbed soil and that which had presumably been consolidated for 2 to 5 months under the overload fill indicated that there had been no measurable change in moisture content due to consolidation. This conclusion was also confirmed by the comparison of moisture contents from the original borings, in which there was striking agreement for comparable samples between moisture contents before and after consolidation.

5. The stability analysis made as part of this investigation showed that overload ratios in the critical soil area investigated were far in excess of those certain to cause soil displacement. The coincidence between high overload ratios, failure areas where subsidence and mudwaves were observed, and points of maximum settlement leave little question that excessive fill settlement was due to exceeding the supporting capacity of the soft subsoil.

6. Settlement of the fill was excessive following the consolidation period until the final paving was completed in the summer of 1952. This excessive settlement has continued and in approximately a year has amounted to a maximum of 1.5 feet in the more-critical areas which coincide with those designated by the contractor's investigation. As a final testimony to the ineffectiveness of the sand-drain piles under the soil conditions of the Norfolk project and impracticability under those conditions of a design predicated on the general theory of consolidation, it may be noted that the Navy, during the past fall, found it necessary to arrange for the restoration of the runway extension which has been the subject of this discussion. Those most familiar with the present status of the project anticipate that the maintenance of the runway will be a serious problem for a long time to come.

#### REFERENCES

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2. Procedures for Testing Soils - American Society for Testing Materials, 1950.

# Discussion

L.A. PALMER, Engineering Consultant, Soil Mechanics and Paving, Bureau of Yards and Docks, Department of the Navy -There have been instances of success in deep soil stabilization by vertical sand A description of a notable indrains. stance of such success in Navy experience is found in the 1949 Transactions of ASCE in a paper; "Failure of Quay Wall at Mare Island," by L.C. Coxe. There have been other instances of only partial success and in a few instances, the installation of vertical sand drains has been entirely futile in accomplishing any noticeable degree or extent of deep soil stabilization.

The instance described by Housel tends to fall in the second category. The easterly end of the runway extension at NAS Norfolk has sunk at least 16 inches, following completion of construction, and indications are that the settlement will continue.

This writer believes that mud waving sheared off a considerable number of the vertical sand drains in this case and that installation of the drains should have been accomplished after placing the excess surcharge over the site and not prior to this operation. The sequence of operations should have included first the placing of the excess fill to drive the water out of the mud. This would cause mud waves which should have been allowed to reach an equilibrium condition and without removal of the displaced mud to displace such equilibrium. Thereafter, the vertical sand drains could have been constructed without being sheared off by displacement of the soft, organic silt under pressure of the surcharge.

Any of us can be misled by technical publications which indicate that the speed of consolidation of the soft soil by the sand drains can be predicted with a fair degree of accuracy. Since estimating the coefficient of permeability in the horizontal direction is mostly guesswork, such a prediction is not possible. After there is full realization of the fact that one usually cannot obtain a reliable estimate of the rate of settlement after sand-drain installation, one tends to proceed with more caution and prefers, whenever possible, to install the drains after and not before the surcharge loads are applied.

If it were possible to now locate the points where sand drains were installed and covered over with fill at NAS Norfolk, one could take borings down through the sand-drain locations. If this were done, it would likely develop that the drains were broken off and displaced in shear by the flowing mud.

Usually, the purpose of sand drains is to accelerate the settlement and have the bulk of it completed prior to construction. Where the soft soil is deeply seated and located at appreciable depths below the proposed elevations of footings of a structure, this is the primary consideration. But when it is necessary to build up bearing capacity in the soft soil as well as to complete its consolidation before construction, then one must make a careful study of the remolding characteristics of the soil to be stabilized by sand drains. The disturbance, caused by the installation of sand drains at 8 to 12 feet, on centers, may weaken a soil much more than its subsequent drainage under load into sand drains can strengthen it. This condition was observed by this writer at a proposed warehouse site in the Norfolk area that is about 3 to 4 miles from the air station. The deep layer of organic silt at this site proved to be sensitive to remolding, and for this reason, sand drains were not recommended.

Neither are sand drains efficacious in peat soil. This writer made an extensive study of the settlements of concrete barricades at Port Chicago, California. Here sand drains had been installed and overloaded with extra surcharge which, after several weeks, was cut back to finished grade with subsequent construction of the barricades. These have settled from 4 to 9 feet or more since 1944. The soil in this case is disintegrated (not fibrous) peat with an organic content of more than 30 percent. In consolidation tests, the rate of strain was independent of the thickness of In other words, the rate of the sample. consolidation of peat (not peaty or peatlike soils) is not determined by the coefficient of permeability, and hence it is independent of the distance of flow of the escaping water.

A thorough field investigation of sand drains under controlled conditions is long overdue. It is not a problem that can be worked out altogether in the laboratory. Much more basic and practical information should be made available before one can have a reasonable degree of confidence in the use of vertical sand drains.

Basically, the theory of soil consolidation is probably sound. However, there is much wrong with its use and application. One must be an irresponsible optimist to believe in the possibility of precise estimates of the subsidence of millions of cubic yards of soil on the basis of the observed laboratory behavior of a few pounds of the stuff. It is just as detrimental to the interest of the public to oversell the utility of soil mechanics as it is to destroy all confidence in its use.

Water held by physiochemical forces in fine-grained soil, referred to by Housel as absorbed water, is probably never lost to any appreciable extent by normal conditions of loading. Also, the bulk of such adsorbed water is retained in the soil at the boiling temperature of water. But free water, lost on air drying, can and is released by the soil under external load application. The release of this mechanically held water into pervious layers that bound the layers of finegrained soil is adequately described by the theory of consolidation, the mathematics of which is about identical to that for the unidirectional flow of heat in an insulated metal rod or the diffusion of a salt into a Difficulties aries when one solvent. idealizes certain conditions. such as "previous boundaries," the continuity of stresses (due to external load) through multiple layers of dissimilar types of soil, and the assumed homogeneity of large masses of fine-grained soil with respect to permeability. The near-ideal conditions do sometimes exist, and they lead to fairly accurate settlement predictions which receive altogether too-much publicity.

WILLIAM S. HOUSEL, Closure - The written discussion of this paper by Palmer was welcome, inasmuch as he had firsthand knowledge of the particular sand-drain installation under discussion. The author was glad to have his somewhat-qualified confirmation of the conclusion that the sand drains were "entirely futile in accomplishing any noticeable degree or extent of deep soil stabilization." There were also a number of other points brought up by Palmer on which the author wishes to comment. The question of shearing off the sand drains by fill displacement is an example of only one of the important uncertainties about sand-drain installation which are all too frequently taken for granted without any real effort made to determine the final result achieved. The continuity of the vertical sand drains is largely a matter of speculation, as it is difficult to determine by direct observation. It is quite probable that they have been sheared off where there has been excessive fill settlement. However, there are other areas where fill settlement has

not been excessive and, in fact, substantially less than anticipated from assumed consolidation. In these areas, there is no reason to expect abnormal disturbance of the sand drains from subsequent fill settlement. In locating the check borings, the attempt was made to obtain data in some of these relatively undisturbed areas. The moisture content and shearing resistance data presented in the paper show the same lack of consolidation effects in all areas which is the essential consideration under discussion.

Shearing off of the sand drains is only one phase of soil disturbance in sanddrain installations. Disturbance due to placing of the sand drains is another reality which has been passed over too lightly. The steel pipe mandrel used in placing the vertical sand drains at Norfolk was 14 inches in diameter and, in a depth of 60 feet, involves a volume displacement of approximately 2.5 cu. yd. Years of pile driving experience has shown that piles driven in soft, saturated clay soils result in practically complete volume displacement. Specifications for heave and alignments of all piles recognize the serious nature of this volume displacement. Occasional collapse and damage to steel shells of cast-in-place concrete piles furnish further evidence of this phenomenon. In the light of all this accumulated experience, it is something more than optimism not to recognize the probable damage to unsupported columns of sand without any structural continuity whatsoever. In the author's opinion, it is extremely problematical that the vertical alignment of sand drains is sufficiently consistent to produce effective vertical drainage aside from all other questions raised as to their effectiveness.

Another aspect of soil disturbance due to installation of the sand drains which does not have to be left to speculation is the loss of shearing resistance due to the remolding effect on flocculated clays of high sensitivity. This point was brought up in the original paper and also emphasized by Palmer. The variation in shearing resistance before and after consolidation clearly established the detrimental effects and more permanent character of remolding under the soil conditions at the Norfolk site. This led Palmer to place such soils in the classification in which vertical sand drains

would not be practical. He also adds peat to the soils not benefited by vertical sand drains which would appear to the author to leave only a restricted range of rather exceptional soil conditions for their effective use.

Palmer cites two sand drain projects in his discussion; the quay wall at Mare Island, California and fill construction at Port Chicago, California, both in the San Francisco Bay area. By strange coincidence, the author was asked to investigate both of these projects during the period in which he was stationed in this area while serving with the Navy. Both projects were in some difficulty and the assignment was to make an evaluation of them and submit any suggestions which resulted to the officer in charge.

It has always troubled the author to see Port Chicago included among the examples of sand-drain installations which are presumably cited as successful applications.<sup>1</sup> His evaluation of it at the time of construction was not much different from that of the present Norfolk project, except that the fill displacements were not as extensive. No change in procedure was suggested as the project was then nearing completion. As now recalled, the author summarized this project as an unsuccessful attempt to make the soil behave in accordance with a theory which did not represent a realistic concept of soil behavior. The results of the study which Palmer made some years later is rather surprising confirmation of this previous evaluation.

The quay wall at Mare Island was only one of several perplexing problems in a difficult area that was then under development. The author made no special study of it, although it was included under a suggested program of investigation which included undisturbed samples, shearing resistance tests, and complete stability anal-The author had no further contact vses. with these problems or knowledge of the quay wall until the final results were reported by Commander L.C. Coxe as part of the "Symposium on Lateral Earth Pressures on Flexible Retaining Walls" in the 1949 Transactions of the American Society of Civil Engineers. The essential phases of this project may be summarized as fol-

<sup>&</sup>lt;sup>1</sup>"Vertical Sand Drains Speed Consolidation of Soft Water-Bearing Subsoil", <u>Engineering News Record</u>, 1947, Vol. 138 p. 386.

lows: The quay wall and dock first failed under a superimposed load which exceeded the supporting capacity of the underlying "plastic clay mud." It was then rebuilt with certain changes in design which included both an anchorage system and vertical sand drains. As described in the article, "Concrete anchors supported by wood, vertical and batter piles and connected to the quay walls by steel tie rods 2 inches and 2.5 inches in diameter were not included in the original design but were installed after failure of the wall." After the tie rods were connected, there was still "appreciable movement" and sand drains were installed. The exact sequence of these several steps is not entirely clear and the "appreciable movement" was also influenced by the load of a sinking tug moored to the dock.

At any rate after the installation of both anchorage system and sand drains was complete, the movement was reduced to a more-acceptable magnitude. Data given in the paper up to 1946 indicate a maximum of additional vertical settlement of 0.43 feet or about 5 inches in 42 months prior to Graphs indicate a continued that date. progressive settlement at some points of approximately a uniform rate of 1.5 inches There are several interesting per year. statements in this connection which deserve special attention and may be quoted as follows:

The action of the sand drains and surcharge was very effective although no actual data as to the reduction in pore pressures are available. This method of stabilization combined with the anchored tie rods completely checked any further motion of the quay wall and effective stabilization was realized in December 1942, approximately two months after the sand drain installation was completed.

The statement that the movement of the quay wall was "completely checked" is not borne out by subsequent data taken up until 1946 and noted in the previous par-That the relative improvement agraph. was due to reduction in so-called pore pressure was an assumption unsupported Without going into the by factual data. hypothetical ramifications of pore pressure, it might have been just as reasonable to suppose that the anchorage system that was added was the more responsible factor in the reduction in movement of the wall.

In the paper presented by the author,

the statement was made with respect to sand drains "that the claims of successful application are seldom supported by more than fragmentary and inconclusive evidence." After careful consideration of the description of the Mare Island quay wall, it would seem that it might serve as a fair example of the fragmentary and inconclusive evidence referred to and in the author's opinion, this would be a very generous evaluation of that project.

Aside from these areas of substantial agreement, there are several points brought up by Palmer which require either further clarification or represent points of Understanding that the some difference. primary purpose of vertical sand drains is to permit the gradual consolidation of soft unstable soil deposits without displacement, it would seem that placing of the surcharge fill first would defeat the declared purpose of the sand drains. If as stated in the discussion the displacement under the surcharge fill was "allowed to reach equilibrium," there would no longer be any point in having the vertical sand drains. In connection with this same discussion, it would appear to the author that Palmer does not clearly differentiate between "placing the excess fill to drive the water out of the mud" which is consolidation and allowing the mud waves to reach an equilibrium which is clearly a function of shearing displacement.

In the original paper, the author definitely questioned the validity of the theory of consolidation and would continue to hold Actually, the difference that viewpoint. in viewpoint is not too great when Palmer says, "Basically, the theory of consolidation is probably sound" and the author would say, "Basically, the theory of consolidation is probably not sound." Both positions leave room for an adjustment of ideas as additional evidence is presented. In supporting the negative position, the author would fall back on the definition of theory as a description of the results of observation and experiment and the contention that when theory and practice disagree, It is certainly the theory must give way. in this area that the theory is being questioned and the experimental data presented in the original paper are capable of standing on their own feet.

In the closing part of his discussion, Palmer brings up the important subject of adsorbed water and the conditions and forces involved in its removal from the soil. His differentiation between free and adsorbed water by evaporation at the boiling point would seem to be an over-simplification not liable to clarify the confusion and misconceptions which may arise in a discussion of these phenomena. The author, on the other hand, would regard all moisture in a soft, saturated clay as being adsorbed to a varving degree, depending upon the magnitude of the forces of molecular attraction with which it is held. Much of the water in such physical combination, sometimes called the water of placticity, may be subject to evaporation at the boiling point of water. The mostclosely held water sometimes considered as chemically combined would be far bevond the effects of ordinary physical pressure and temperature less than the boiling

point of water.

In the original paper, the attempt was made to be quite specific on the probable range of consolidation effects involving anplied pressure and its relation to shearing resistance which controls mass displacement. The only amplification of the view presented which appears to be required at this point is to acknowledge that pressure concentration which may be applied within the limit of shearing displacement does represent a range of actual consolidation which might be achieved under practical conditions. To the author, this represents the only range within which the consolidation theory may be considered valid. It is felt that within this range, consolidation is too long and too little to be of more than negligible value under practical conditions in the types of soil under discussion.