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Road Construction Over Peat Bogs Has Presented a Difficult Problem in Michigan

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FILL SETTLEMENT IN PEAT MARSHES[1]

By V. R. BURTON, Engineer on Special Assignments, Michigan State Highway Department

THE MICHIGAN State Highway Department has just completed a special study of fill settlement in peat marshes as a part of the general soil survey it has undertaken. Settlement in peat marshes has caused serious trouble in many places, because it was unknown how much fill material would be required and even after the fill had been completed there was no assurance that it would maintain the original grade line. It was therefore desired to ascertain the physical characteristics of the peat and decide on the field data necessary to properly locate and estimate the cost of a road over a peat deposit. It was hoped that it would be possible to devise a method of construction more economical than that then in use, and to adopt better maintenance methods to restore sunken fills and prevent further settlement.

A brief description of peat deposits will be necessary if any adequate idea is to be had of what actually exists under the surface of a "sink hole." Peat deposits are classified and mapped stratigraphically in essentially the same way as mineral upland soils. For general peat studies, peat materials have been described by layers in terms of composition, texture, structure, and color. This gives a fairly good idea of the main differences between separate peat layers and localizes their relative positions.

VARYING CHARACTERISTICS OF PEAT LAYERS AID STUDY OF FILL SETTLEMENT

The classification of peat strata was made in accordance with the system largely developed in this country by Dr. Alfred P. Dachnowski, of the Bureau of Plant Industry, United States Department of Agriculture, and he personally supervised a number of surveys. The following description of peat deposits is taken from an article on the "Stratigraphic Study of Peat Deposits"[2] by Doctor Dachnowski. "Distinction should early be made between muck and peat. Muck is the thin surface layer of disintegrated peat and is entirely different in appearance and properties to the relatively unaltered peat below it. This distinction is made because many engineers use the terms interchangeably with no knowledge that there is a difference in the two."

Table 1 is from the article just referred to, and gives the classification of characteristic peat layering.

Pulpy peat is formed in water basins under conditions of poor drainage from transported organic sediment carried from its origin and redistributed by water currents. There are no visible bedding planes although the original sediment was laid down in practically horizontal position under conditions differing by rate of deposition, seasonal changes, etc. The plant remains are small in size. In texture the peats vary from coarse to very finely divided particles, while in structure a layer may vary widely in its degree of compaction. Fresh from the deposit and while still moist the colors vary from gray, green, and brown to black.

DRAINAGE DITCH THROUGH PEAT BOG

As with most peats, the pulpy variety quickly loses its character by oxidation on exposure to the air and drying. It shrinks enormously and becomes a hard, water-resisting substance when quickly dried out. It will not then absorb anything like the original quantity of water. Doctor Dachnowski states that "it may be accepted as an axiom that undrained deposits of peat contain from about 70 to 95 per cent of water."

The outstanding characteristic of fibrous layers of peat is their matted or felt-like, porous nature which has its origin in the slightly altered remains of moss, roots, rootlets, and rhizomes of herbs. The peat layers differ widely in character. Their water content is enormously high, the solid matter commonly composing only from 5 to 15 per cent of the peat mass. In texture they vary from coarse to very finely fibered material usually presenting a rather loose, porous appearance, with colors ranging from gray through yellow to red brown and dark brown. The layer may be interbedded or it may grade into a layer of woody of pulpy peat by admixture or it may occur in overlapping beds of other kinds of fibrous peat.

TABLE 1.—*Structural units of peat deposits and some of their physical characteristics*

Groups of peat-forming vegetation	Types of peat	Character of peat layers	Color of peat layers	Texture of peat layers	Structure of peat layers
Aquatic	Macerated Colloidal	Pulpy (sedimentary)	Olive green, brown to black.	Coarse to very finely divided.	Compact, impervious, stiff, plastic or loose, friable.
Marsh	Reed Sedge Brown moss	Fibrous	Gray, red or yellow brown to dark brown.	Coarse to fine fibered	Dense, matted, felty, or porous, spongy.
Bog	Bog moss Heath shrub				
Swamp	Willow-alder shrub Deciduous forest Coniferous forest	Woody	Dark brown to blackish brown.	Coarsely fragmented to granular.	Compact and granular, or loose, wicker-like.

[1] Paper presented at the fifth annual meeting of the Highway Research Council, Washington D. C., December, 1926.
[2] Soil Science, vol. 17, No. 2, February, 1924.

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Woody peat may occur either as the dominant component or as a prominent admixture in other types. The woody plant remains are broken down partly into granular débris and partly into irregularly shaped woody fragments, which are almost unaltered by decomposition. The woody peats differ widely in composition and character. They may be as finely divided as sawdust or as coarse as a tangle of waterlogged brush and logs. Several layers may be found as, like the fibrous peats, they are developed on moist flat land under conditions of a rising or fluctuating water table. Land may grow up to forest, be submerged to form a marsh, and then emerge again to become a forest. In one peat deposit examined on this survey three distinct forest layers were plainly visible.

Marly phases of peat are formed from calcareous plant remains and shells of mollusks and are usually found admixed in the macerated types of peat. Extensive deposits of marl with peat occur mainly in regions underlain by limestone rock or where the soil adjacent to and underneath a peat area is derived from limestone drift. Deposits vary in color from gray to brown and are usually soft plastic masses with particles from fine to comparatively coarse, depending on the amount and character of plant or shell remains present.

The lake clay found at the bottom of most deep peat deposits is structureless, mainly siliceous material from plant and animal remains consisting of diatomaceous shells and drift débris. It is gray to gray blue in color, fairly compact, but with a high percentage of water in it and is quite plastic and sticky. It is, however, a much more compact and weighty material than any of the peats or marls.

In carrying out the study of fill settlement it was decided to cross section marshes accurately by borings, which would show the various peat layerings and the exact position of the filled material with reference to them. Samples of the various classes of peat were to be collected and sent to the laboratory for physical tests in an effort to discover some relationship between their bearing power and the known settlement for various water contents. If a large number of existing fills were thus examined, it seemed logical that general conclusions might be drawn to serve as a guide on future work.

FILL SETTLEMENT NOT AFFECTED BY CHARACTER OF PEAT

Fills over eight different peat deposits were cross sectioned at 83 different points where the depth of the peat ranged from 1 to 66 feet. The shape of the fill material was accurately determined with a 2-inch Empire drill. Cores from the peat were taken with the Davis peat sampler where possible, but if the peat was compacted beyond a certain amount the Empire drill and jet was used. Enough test holes were put down on the cross section chosen to show the fill shape fairly accurately and an effort was made to extend the cross section in both directions from the fill sufficiently far to get out of the zone where the layers were disturbed by filling. In a few cases it later became evident that the test holes at the extremities of the cross section were not placed far enough from the center line.

Samples of the different peat layers were collected at various depths and sent to the laboratory for physical tests. Results were very disappointing in this regard. It was impossible to maintain the peat in its original condition on exposure to the air. Peat as it comes from the ground is only partially oxidized and at the greater depths there is actually some reducing action. Consequently, immediately it is handled in air, oxidation begins and if it is attempted to dry it, a totally different material results. Laboratory bearing value tests were not satisfactory. It is certain that the amount of compression under loading in a vessel such as is used in the standard bearing value test which does not permit of a free displacement in all directions will not give results at all indicative of the behavior of the same material in its natural place.

The only way, then, to secure the solution of the various problems involved in this study was by a careful study of the cross sections showing the fill shape and the peat layerings in the disturbed position after filling. An effort was made to reconstruct the position of the layering as it originally existed from the position of the layers outside the distorted area of the cross section. A study of the fill cross section and the distortion of the original layers as reconstructed led to the conclusions drawn from these effects.

The various factors which it was thought should determine the amount of settlement were depth and character of peat layering, height of the water table, slope of the mineral subsoil beneath the peat, height and width of fill. The weight and character of the fill material would also be a factor but the difference in weight of the materials studied was so small as to be neglected. All fills in this investigation were earth and varied from a clay to sand. No rock fills were studied.

So far as could be determined the character of the peat layering is very slightly responsible for differences in fill settlement. There is, to be sure, a decided difference between the top layer of peat a foot or two in depth where the material is fairly well decomposed, but this is due more to a smaller water content than in the actual character of the material itself. The lake clay which occurs in the bottom of most of our deeper peat deposits is enough different in character to make it necessary to consider its effect separately. It is also probable that if any considerable thickness of pure marl with a heavy texture were encountered it should be grouped with the clay or given separate treatment. As it happened in this investigation no very great amounts of marl were found except as they occurred as a peat admixture. A soft soupy marl should probably be classed as peat in considering its effect on fill settlement. The preliminary work of sounding a peat bog preparatory to estimating fill settlement is thus considerably simplified for the engineer, as where a uniform marl is absent, only two layers, the peat and lake clay, need be considered. No differentiation of peat layering is necessary for estimating fill settlement.

DEPTH OF PEAT AND LAKE CLAY THE MOST IMPORTANT FACTORS

If there were any great variation in the height of the water table compared to the depth of the deposit, this factor should certainly show its effect. It so happened that the road ditches were about the same depth through all of the shallow peat deposits studied and in the deposits of greater depth the depth of the ditch was very small in comparison to the total depth of the peat. The water table in most cases varied from 1 to 3 feet below the fill surface. The varying height of water table, which was to a certain extent dependent on the time of year as well as on the particular situation of the cross section, was not taken into account. Some inconsistencies in the shallower depths are probably due to the neglect of this factor.

The height of the fill above marsh surface is of course a factor, although not so great a one in the case of loading over a wide area as one would expect. In peat depths of less than 10 feet the settlement is small and is affected by factors other than height of fill and results are inconsistent. The height of fill produces a marked effect in medium depths of peat of from 10 to 25 feet. In depths beyond 25 feet the total depth of fill is so great in proportion to the height above marsh surface that the amount of this additional loading is not important. The above refers to completed fills studied and not to new construction methods in which the height of fill placed in the early stages is of considerable importance.

The slope of the mineral subsoil produces a marked effect on settlement and the bottom of the fill nearly always takes the same direction of slope as the marsh bottom. This is due to two reasons; first, the flow of peat beneath the fill is aided by gravity, and, second, the greater depth of peat down slope induces greater settlement. Most of the inconsistencies in amount of settlement from the determined normal occur above sharp slopes in the mineral subsoil. The amounts varied so much, however, that no general law could be observed. It is safe to say that in general if the mineral subsoil does slope quite sharply, the normal amount of settlement will be increased.

One very important factor met in special cases is the effect on fill settlement of the peat compression by neighboring fills. The compressed peat is much more resistant to flow than is the uncompressed peat, and hence flow on the side adjacent to an old fill is very much smaller than on the free side.

The factors of greatest importance and the ones least difficult of determination are simply the depth of the peat layer and the depth of the lake clay beneath it. Other factors undoubtedly have their influence but the depths alone, so far as could be determined from the information available, directly determine the amount of settlement which will occur. No other factors operated with the consistency or effect of these.

Let us consider what happens when a fill is placed over a peat deposit. The top crust is first subjected to compression beneath the interior of the fill and to shear and tension at the edge. The communication of this compression to the interior of the peat is opposed by the resistance to flow of the peat or its internal friction. On account of the inertia of the peat particles, water is squeezed out of the material nearest the fill and the material is compacted, and this process extends with continually decreasing effect as distance from the fill increases until at some distance the peat is practically undisturbed.

Lateral flow of peat begins at an early stage in the placing of a fill and is due to the horizontal component of pressure due to the internal friction of the material and also to the inclined position of the settling load. Peat is forced outward and upward, until in the course of time a condition of equilibrium is reached.

It is easily understood, then, that the width of fill is a function of the amount of settlement which takes place. The fills in this investigation do not vary greatly in width, viz, from 24 to 30 feet, and so no account is taken of this factor. It is important, however, in considering the cause of the greater settlement in deeper deposits.

When deposits are relatively deep in proportion to their width, considerable vertical movement can take place before any marked reaction is experienced from the mineral subsoil below. In this case, lateral flow is comparatively small at first and a large amount of settlement is possible before there is much resistance; the elastic crust ruptures in tension and shear and the whole weight of the fill is suddenly thrown on the soft material below. This sudden application of weight very quickly compacts the peat below to a degree at which lateral flow begins in a considerable amount and we have a "sink hole." A portion of the peat flowing from beneath the load of earth is forced up at the sides. Due to the lighter weight of the peat as compared to the fill material, a correspondingly

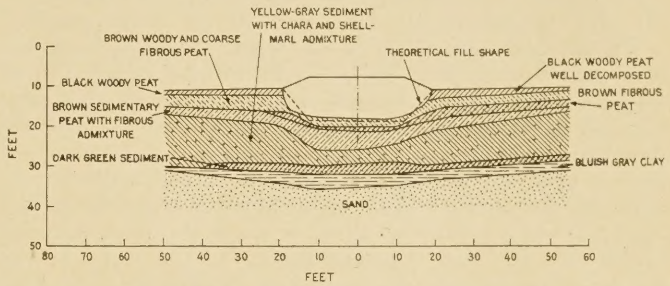

Fig. 1.—Fill of Normal Cross Section in a Peat Marsh About 22 Feet Deep from Surface to Clay

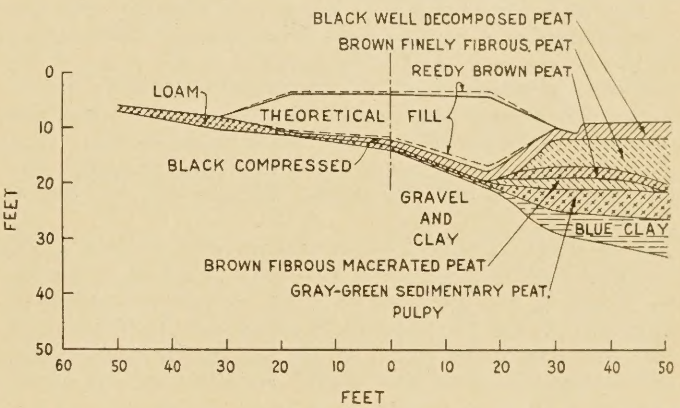

Fig. 2.—A Fill Partly on Upland Soil and Partly on Peat Marsh

larger amount of peat must be raised to balance the heavier fill. Subsequent loading increases this heaving until the weight of the heaved peat brings about a condition of equilibrium.

SLOPE OF SOIL BENEATH PEAT BED AFFECTS FILL SHAPE

In the later stages of fill settlement the rate of settlement is very slow, but even a rate of a foot a year is decidedly objectionable. The rate progressively decreases until finally, after a long time, equilibrium is established. The length of time required to reach a stable condition in some cases is known to have been more than five years. We have arbitrarily established the rate of five-hundredths of a foot in 30 days as being the rate of settlement at which filling may be stopped. Since the rate will usually decrease progressively, this should give a settlement of less than 6 inches per year.

Unfortunately, at times, if the early settlement is not rapid, some later cause may accelerate the rate of settlement. Fills completed in the fall under earlier methods were quite apt to settle rapidly the following spring. The vibration of traffic also tends to increase the rate of settlement after a road is first opened. Large ditches cut along an old road are quite apt to start considerable settlement. As a matter of fact, unless settlement during filling has been accelerated in order to insure its completeness, some subsidence is almost sure to take place later.

A number of cross sections illustrating the various shapes taken by fills under different conditions are shown. Figure 1 shows a fill of normal cross section in a peat marsh about 22 feet deep from surface to clay. The theoretical shape of fill is shown by the dashed line and is obtained by a method described later. This deposit is of nearly equal depth on both sides of the fill and therefore the shape of fill is quite regular.

Figure 2 shows a fill which is partly on upland soil and partly over peat marsh. It will be noticed that the mineral subsoil slopes sharply toward the peat deposit and for this reason the proportion of the fill beneath the surface of the peat is considerably increased above that shown on the normal cross section. It will also be noticed that on account of the flow of the material only a very small amount of the peat remains under the fill, and that the layers to the right of the fill are considerably thickened by reason of their flow from beneath the fill and subsequent expansion due to the decrease of pressure.

Figure 3 shows a fill which is still settling into the peat marsh at a rapid rate. The slope of the bottom of the fill is quite noticeably parallel to the slope of the mineral subsoil beneath the light clay and most of the distortion of the peat layers has occurred on the low side of this mineral subsoil. It is quite evident that even with this considerable depth of peat beneath the fill, pressure is communicated throughout the entire mass.

The effect of the slope of the mineral subsoil upon the direction of pressure is still more marked in Figure 4, where it will be noticed that, except immediately adjacent to the fill, practically all of the disturbance of the peat has taken place to the left of the center line, indicating that flow has largely occurred in this direction.

Figure 5 shows a fill which was placed adjacent to a railroad fill at the right, but not shown in the figure. In this instance it will be noted that the compacted peat to the right of the center line reduced the amount of fill necessary in a material manner. A very considerable amount of distortion of the layers is shown on the left of the figure. In this instance the peat is heaved above the normal level of the marsh some 10 or 12 feet and slopes away from the crest of the heaved material in a fairly uniform manner.

Figure 6 shows the deepest cross section encountered in this investigation, a fill with a depth of about 66 feet. The marsh, over which this fill was made, lies between two fairly large lakes, which are at present connected by a small stream. It is quite evident that the location of the highway in this particular instance was very unfortunate in that the center line of the highway at the cross section shown is practically on the center line of the old glacial drainage channel between the lakes in the early stage of their formation. A location of center line 30 feet to the right of its present location, which would have been possible, would have eliminated about 25 feet of fill. The entire amount of the lake clay which probably nearly filled the old channel, has been forced to one side by the load of fill above it. The irregular shape of the cross section on the right is probably due to filling a hole left by the peat heaving 8 or 10 feet at the side during construction.

METHOD OF PREDICTING FILL SETTLEMENT AND SHAPE DEVELOPED

From the whole series of cross sections of the character of those illustrated, most of which, however, are considerably more regular, diagrams giving the amount of settlement and shape of fill have been prepared. Depths of peat and marl on center line of fill were

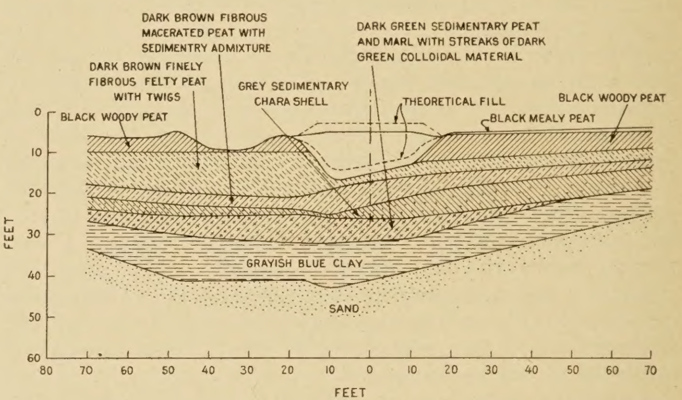

FIG. 3.—SHAPE OF FILL IN PEAT CONSIDERABLY AFFECTED BY SLOPE OF MINERAL SOIL AT BOTTOM OF PEAT BED

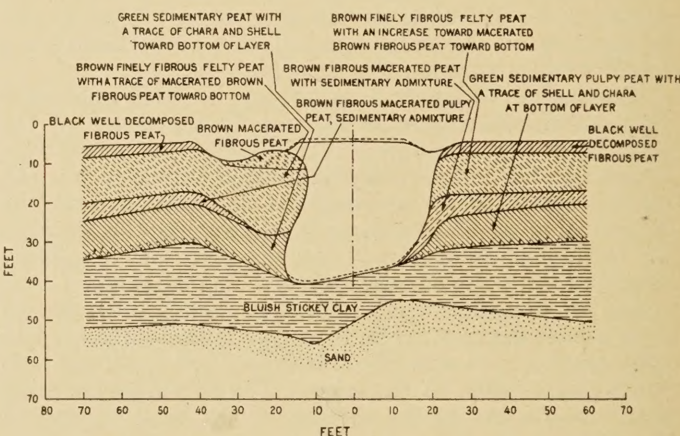

FIG. 4.—DISTORTION OF PEAT FILL TO THE LEFT DUE TO SLOPE OF FIRM SOIL BENEATH PEAT BED

platted against the total depth to which the fill penetrated the peat and marl as measured from the original marsh surface. Three curves were then drawn, one representing the average of fills with a height above marsh level of 1 to 3 feet, one for 3 to 4 foot fills, and the third for fills from 4 to 5 feet in height. These three curves, shown in Figure 7, indicate that the amount of settlement increases uniformly but is not great up to a marsh depth of 20 feet. As the marsh depth increases from 20 to 26.5 feet the settlement increases rapidly and when the depth exceeds 26.5 feet the fill goes completely through the peat in every instance, so that the penetration is equal to the depth and the three curves become common and make a straight line with a slope of 45°. Therefore, for peat and marl depths of less than 26.5 feet the depth of lake clay need not be taken into account, except as a factor should settlement be different from the average, and anticipated settlement is read directly from the diagram. For depths of peat and marl over 26.5 feet the settlement will probably be the full depth of the deposit plus some penetration into the lake clay. The amount of penetration into the latter material is read directly from the curve marked lake clay, which has been plotted, using the combined depth of peat and clay as abscissas.

Suppose we have a peat marsh consisting of 30 feet of peat and marl and 5 feet of lake clay. The peat curve gives us complete penetration of the full 30 feet of peat and from the 35 feet total depth of the marsh deposit we read off from the clay curve 2 feet for clay penetration. There will then be a distance from bottom of fill to marsh surface of 32 feet. Again, if we had a peat deposit consisting of 38 feet of peat and 2 feet of lake clay we should get 38 feet of peat penetration plus 4 feet of clay penetration. The total, 42 feet, exceeds our total depth of deposit and the fill therefore stops at the marsh bottom with a 40-foot penetration. This last case is not common except in the very deep deposits, of 50 feet or more. It will usually be found that in marshes of such extreme depth with ordinary proportions of peat and lake clay, the fill goes to the very bottom.

No claim of scientific reason is made for the shape of the curves shown in Figure 7. They are purely empirical and represent merely the average behavior of a considerable number of fills. Extreme variations of actual from expected settlements as derived by this method have been found to be as much as 100 per cent for an indicated settlement of 5 feet, 60 per cent for 10 feet, 30 per cent for 20 feet, 20 per cent for 25 feet, 10 per cent for 30 feet, and less for greater amounts. Generally accuracy of the method is well within these limits.

The method of estimating fill quantities is illustrated by Figure 8. The typical shape shown was arrived at

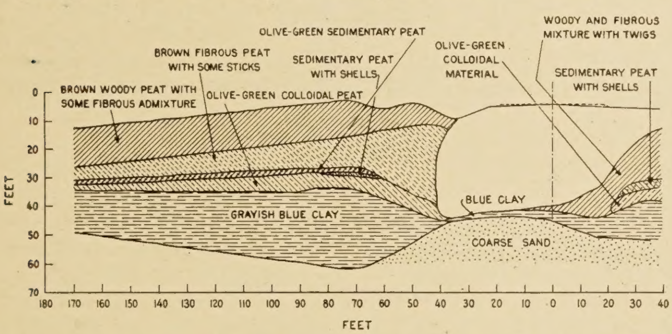

FIG. 5.—A FILL ADJACENT TO A RAILROAD FILL AT THE RIGHT BUT NOT SHOWN. COMPACTED PEAT ON THE RIGHT SIDE REDUCED THE AMOUNT OF FILL CONSIDERABLY

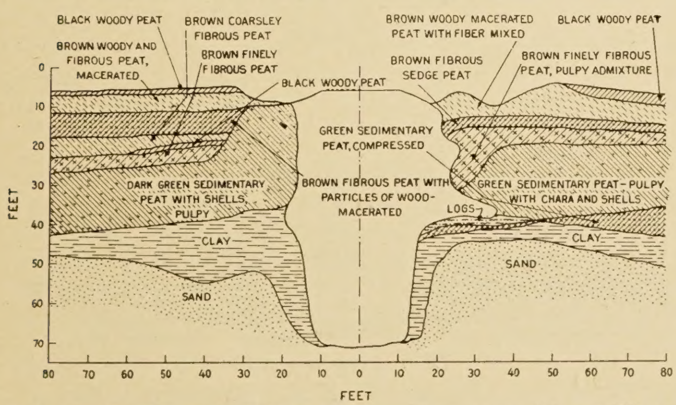

FIG. 6.—AN UNUSUALLY DEEP FILL. SOUNDINGS WOULD HAVE INDICATED THIS CONDITION AND PERMITTED MORE ADVANTAGEOUS LOCATION

by drawing on cross-section paper all cross sections of equal depth, superimposed on each other, with the marsh level as a base line, and then fitting an average shape to them. A side slope of embankment above marsh level of 1 to 3 feet was used, since most of the fills studied had this slope. The fills quite generally slope inward from the outermost point of the embankment to a point directly beneath the shoulder line. The total area of fill is composed of two trapezoids and can be calculated by the following formula, the symbols representing distances as indicated in Figure 8.

$$\text{Area} = (W+3F)\ (F+H)$$

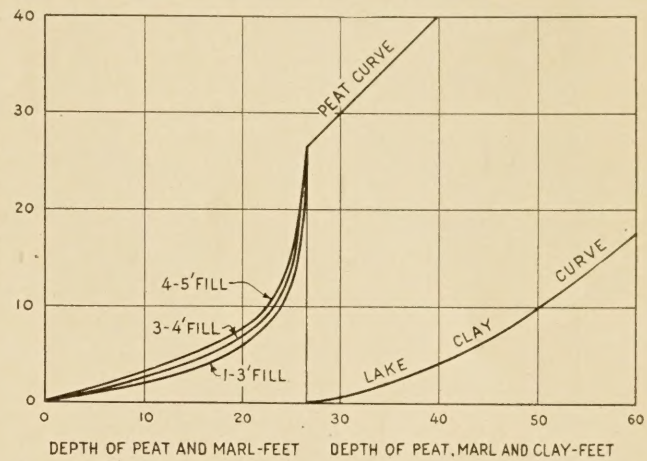

FIG. 7.—DIAGRAM GIVING PROBABLE DEPTH OF PENETRATION OF FILLS IN PEAT MARSHES

In each case H must be determined by use of the curves shown in Figure 7. In Figure 8 the bottom of the predicted shape of fill is shown as a horizontal line and this was very nearly the case in most instances, the center of fill being only slightly lower than the outside. The depth of penetration, as indicated by Figure 7, takes account of these variations and gives the average depth to the bottom of the fill.

FILL SETTLEMENT SHOULD BE ACCELERATED

It may be surprising to note that the slope of the fill sides beneath the surface of the peat is toward the center of the road, but, with a little consideration, the reason for this is quite apparent. In the first place, the fill load as placed varies from zero at the toe of slope to its maximum at the shoulder line, and hence should produce a fill shape corresponding to this loading. Also, in a good many instances the fill has been carried across the marsh at less than full width while heavy settlement was taking place, and this produced a narrower fill at the bottom. Undoubtedly the drag of the compacted peat at the side also tends to narrow the fill width as it sinks. This is indicated by the fact that the center is quite generally lower than the rest of the cross section.

One of the striking features of this investigation was the irregularity shown in the topography of the marsh bottoms. In a number of instances a slight shift in line would have saved thousands of yards of earth in filling. It is therefore necessary when the location of the road requires the crossing of a peat marsh to map the topographical features of the marsh bottom quite accurately. This is more important than on upland soil because visual inspection of the proper location can not be made but must be determined by the topographical information secured from sounding.

In places where there is any considerable depth of peat it is highly necessary that settlement during filling be encouraged in every way possible. This settlement is bound to take place sooner or later, and it is important that it be practically complete before any rigid surfacing is placed. Construction of brush mats to float the fill is not advisable in the case of highway work where hard-surface pavement is to be placed. These mats only delay settlement and do not stop it. This

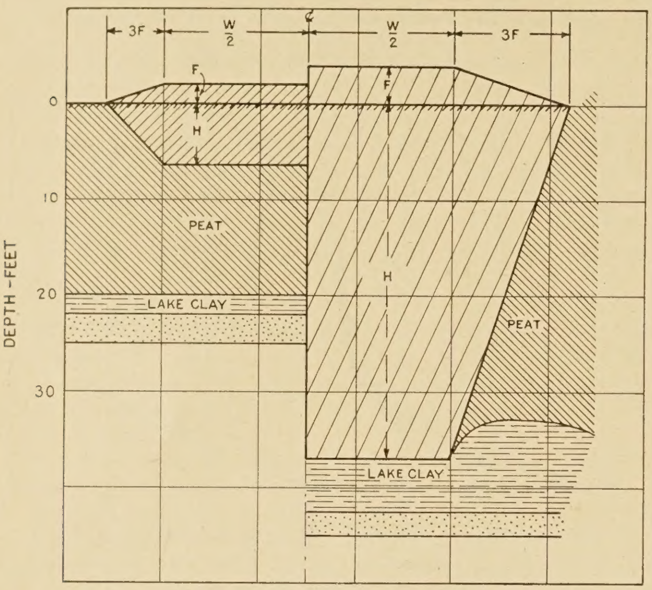

Fig. 8.—Diagram Illustrating Typical Shape of Peat Fills for Use in Estimating Earthwork Quantities

is contrary to the practice of some railroads, but in railroad work it is a simple matter to add ballast and bring the track back to grade as settlement progresses without interrupting traffic. In case of any considerable settlement on a hard-surfaced highway, the subgrade must be refilled and a new surface placed. This always involves more or less interruption to traffic besides the greater expense of placing the new surface.

The length of the bog should be considered in choosing a method of filling. Settlement over short bogs, even in small amounts, is much more objectionable than on longer stretches because of the sharper deflection of the settled surface, and the liability to offsetting of the pavement slabs. A long gradual settlement of considerably greater amounts is not so noticeable either to the eye or in the riding qualities of the pavement. Of course, settlement over either long or short bogs is objectionable if the amount is great enough to interfere with the drainage. Settlement sufficient to cause trouble on this account is not likely to occur in marshes of less than 6 feet in depth.

STANDARD METHODS OF FILLING DEVELOPED

Figures 9 and 10 illustrate methods of filling over peat bogs now standard with the Michigan State Highway Department, one or another of which is used depending upon the length and depth of the bog to be crossed. Method No. 1 simply consists of placing the fill on the peat surface and is used where the marsh is less than 6 feet deep, more than 300 feet wide and undrained. The bog is long enough so that, if settlement should take place, it would be rather gradual, and the 24-inch height of fill above the original marsh will usually absorb this settlement without bringing the top of fill too close to the marsh surface.

Method No. 2 is used in drained peat marshes up to 6 feet in depth and over 300 feet in width. It is similar to method No. 1 except that the marsh drainage ditch goes entirely through the peat deposit. The ditch is excavated prior to the placing of the fill. This type of construction results in a much drier subgrade, and the lowered water table helps to prevent considerable settlement. This scheme should be used where conditions of the marsh drainage system require the use of ditches of considerable size.

Where bogs are less than 300 feet in width, and not more than 6 feet in depth, and even small settlement is undesirable, method No. 3 is used. This method requires that all the peat to a width 4 feet greater than the proposed pavement slab be excavated to marsh bottom, and the excavation filled with good earth. This method is not so expensive as it would seem, because the peat is very easy to move and can usually be disposed of with a very short haul. Peat excavation of this type is not practical for depths greater than 6 to 8 feet on account of the large amounts of peat which slide into the hole from the sides.

The above methods are not suitable for bogs much in excess of 6 feet in depth. In the deeper deposits settlement is almost sure to take place and should be aided while the fill is being placed. Method No. 4 in Figure 10 illustrates a Michigan practice in deposits over 6 and less than 20 feet in depth. In the first stages, two narrow fills, spaced 16 feet on centers, are carried across the marsh.

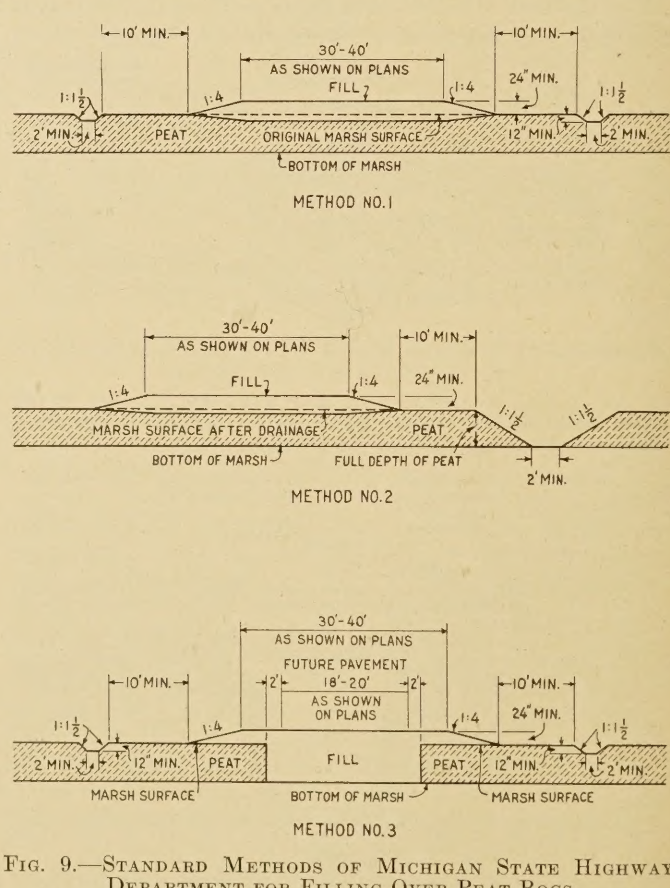

Fig. 9.—Standard Methods of Michigan State Highway Department for Filling Over Peat Bogs

The supporting power of the peat crust is destroyed by blasting on the center line of each of the fills. The trenches thus made are backfilled with earth in 4-foot lifts and carried simultaneously across the marsh, trapping a certain amount of peat between the two fills. When the first two lines of embankment are complete and settlement has slowed up to a considerable extent, charges of dynamite are placed in the peat on 4-foot centers, from 2 to 3 feet below the bottom and on the center line of each embankment, and exploded. This second blasting drives the peat from beneath the wedges of earth and induces further rapid settlement. After refilling the settlement caused by the shooting, the second loading of earth is added in a 4-foot lift on the center line of the road. This second loading helps drive the wedges of earth still farther into the peat and insures a maximum compaction in the center so as to eliminate the possibility of center settlement resulting from the method used in the first loading. This loading is maintained by refilling the settlement until the rate of settlement is less than five-hundredths of 1 foot in 30 days. The excess earth is then spread to form the complete embankment.

Bogs from 6 to 20 feet in depth usually give more trouble than any others, because they rarely settle rapidly, unless excessively loaded, and surfaces are laid on fills long before complete settlement has been attained. It is much more difficult, in bogs of these depths, to secure complete settlement due to the early resistance of the peat to compaction; and it is for this reason that the two narrow wedges of earth are used, the effort being to secure as deep a penetration as possible. The final loading under this scheme gives a static load which is nearly twice the normal load, and it is felt that little danger of further serious settlement need be feared. To insure the acceleration of settlement, it is required to maintain the grade to within 1 foot of the designed elevation at all times.

In long marshes, in order to avoid long backing by trucks in end dumping, provision is made for shooting part of the fill at a time. It is believed, however, that at least a hundred feet of fill should be held for settling at all times before any shooting is done.

For depths of peat over 20 feet, the center loading is used at once after the surface of the peat has been broken up by dynamite. A fill 12 feet wide at the top and 4 feet high is carried across on the center line. This fill is maintained at grade as nearly as possible until it is entirely across the space over which this method is to be used. As soon as it can be maintained at grade entirely across the marsh, a second 4-foot lift is made without any preliminary shooting. It is felt that with a fill width of this amount, dynamite would not displace the peat sufficiently to justify its use. In addition to this, the greater depth of the marsh makes settlement so much easier that the loading alone produces sufficient settlement. After the settlement has slowed down to a rate of five-hundredths of 1 foot in 30 days, the fill is unloaded and the excess material spread to form the remainder of the cross section. The compressed peat on both sides of the narrow fill will usually support the additional width with little settlement.

Any combinations of these methods may be used in crossing peat marshes of considerable length, suiting the method to the depth and width. The first requirement in all marsh filling where pavement is to be placed is to insure settlement, and not attempt to avoid it. Unless the fill has become stabilized, this settlement may occur over long periods and the former practice of attempting to "sneak across" with light loadings has been responsible for a great deal of maintenance expense in locations of this character. The proper time to take care of this settlement is during construction, as it is extremely difficult to maintain in a satisfactory way a surface which is continually settling.

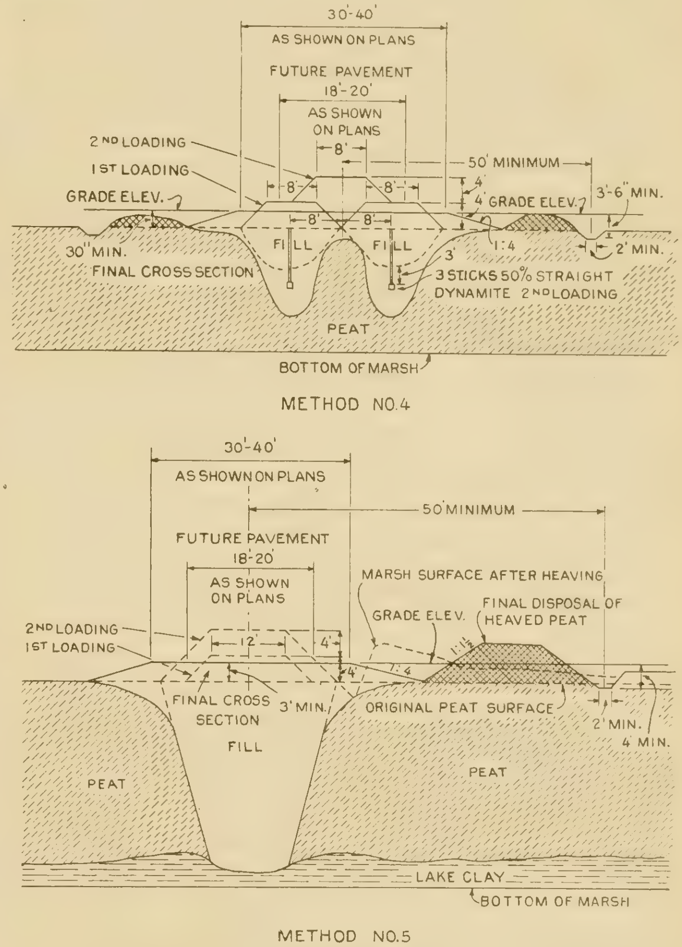

Fig. 10.—Standard Methods of Michigan State Highway Department for Filling Over Peat Bogs

FILL CONSTRUCTION SHOULD PRECEDE BRIDGE CONSTRUCTION

If any bridges are to be placed over streams through deep peat deposits, the approaches should always be completed first. Filling should continue toward the center of the stream so that excavation for footings will occur in the filled earth. In some cases, where streams are small, it will be found advisable to carry this filling completely across the old stream bed. The expense of later cleaning out the earth above stream bottom and taking care of the stream flow during filling is money well spent.

The above may seem ridiculously obvious to many engineers, but the frequency with which bridge failures have occurred during the past because of ignorance of the troubles which may arise through neglect of this item justifies it. When a bridge rests on piling with lengths of over 10 feet penetrating soft peat before

(Continued on page 247)

DETERMINATION OF CONSISTENCY OF SOILS BY MEANS OF PENETRATION TESTS

Reported by Dr. CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

WHEN walking over the ground, one's foot may either sink into the ground to a depth of more than an inch, the soil being squeezed out laterally under pressure or the foot may leave hardly any impression with many intermediate possibilities. Judging from the effect of the pressure exerted by the foot, we say, that one soil is less consistent than the other.

The effect of the pressure exerted by the foot is obviously connected with what is known as the bearing capacity of the soil. Since, in addition, it is connected with what is commonly called the consistency, both engineers and geologists have attempted to establish some scale for expressing the consistency of soils for practical purposes. The basic idea is this: First to devise some simple standard test, whose outcome depends on the consistency of the material and then to correlate the *consistency coefficient* obtained by means of such a standard consistency test with the bearing capacity of the soil. This plan seemed to be particularly promising in all those cases, where the penetration of the loaded area is due merely to a lateral flow of the soil—e. g., penetration of a loaded area into soft fat clay.

The possibility of expressing the bearing properties of a soil by a *consistency coefficient* would certainly represent an ideal solution of an important problem. The purpose of this paper is to discuss related attempts and their chances for success.

TERMS DEFINED

The term *consistency* is by no means physically as clearly defined as are the terms *viscosity* or *hardness*. As it is commonly used, the term *consistency* means the resistance of a material to flow, yet resistance to flow is in itself a rather complex property. Suppose we have a mass of paraffin and a mass of soft clay, both with a plane, horizontal upper surface. Then we select a weight, which, if placed on top of the clay, is heavy enough to produce a marked impression without completely sinking into the material. If we take another weight, exactly alike and put it on the surface of the paraffin, we will notice that the impression produced by the weight is far less significant. Yet, if we observe the two weights during a sufficiently long period, we will notice that the speed with which the weight penetrates the clay rapidly decreases and after a certain time its downward movement stops completely. If the weight is carefully removed and again applied, the increase in penetration will be exceedingly small. On the other hand the speed with which the weight penetrates the paraffin remains fairly constant, although it is very small, even at the beginning. As a consequence, after a certain length of time the penetration of the weight into the paraffin may considerably exceed the penetration produced in the clay, although the initial penetration was very much smaller. If the weight is removed and again applied, the penetration will continue to increase about at the same rate as it did at the beginning of the process. The question arises: Which one of these two materials is the more consistent?

After a short period of time, the depth of penetration in paraffin will be very much smaller than in clay. After a sufficiently long period the opposite will be true. In order to understand this apparent contradiction, we must consider the following facts: The speed with which the weight penetrates the material depends on viscosity, a property common to both solids and liquids. If a material has only viscosity, it has no static bearing capacity, though in every other respect it may exhibit all the properties of a solid body—e. g., sealing wax or paraffin. It flows under any pressure, although it may flow very slowly. Certain other materials, as, for example, soft clay, have, compared with sealing wax, a rather low viscosity, which means that the soft clay is very much more *fluid* than is the paraffin. Yet in contrast to the sealing wax, under very low pressures the clay does not flow at all. In order to start the flow, the pressure must exceed a certain limiting value, known as the *yield point*. The yield point represents the ultimate bearing capacity of the clay. Although the soft clay is much more fluid and much less viscous than the paraffin, yet its yield point is very much higher. For this and similar reasons, the consistency of a material can not possibly be expressed by a single figure. It must be represented by a graph.

However, when dealing with subgrades we are merely interested in what the subgrade can stand without the deformation going on indefinitely. Hence, if we use the term *consistency* in connection with foundation problems at all we should confine it strictly to the maximum pressure under which the material is still apt to to come to equilibrium, that means to the yield point. This statement determined the fundamental requirement for standard consistency tests for subgrade or foundation purposes. The figures furnished by the tests should be in a constant and well-defined relation to the yield point of the material. At the same time it should be kept in mind that such figures inform us about one phase only of what the physicists call the consistency. The other phase of this property, viz, the behavior of the material in the state of continuous flow, under a pressure above the yield point, may be interesting from a scientific point of view, but it has no bearing on our foundation problems.

It is obvious, that the yield point, that means the maximum pressure per unit of area under which the material is still apt to come to a state of equilibrium, depends on its state of confinement and on the way in which the pressure acts. Thus, for instance, the yield point of a thin layer of soft clay inclosed between a hard road surface and a more consistent under-ground is very much higher than the yield point of the same material if only loaded over an area of several square inches or if tested in an unconfined state. Yet these conditions are a matter of detail and will be considered later. The essential point merely resides in the possibility of an ultimate state of equilibrium under a locally applied pressure which, in the case of sealing wax or paraffin, does not exist.

The most logical procedure for rapidly evaluating the yield point of a material seems to be that of loading the surface of the material with a cone of standard weight. The deeper the cone penetrates into the material the smaller becomes the pressure per unit of surface of contact between the cone and the material. Hence the depth at which the cone refuses to penetrate farther into the material can be used as a scale for expressing the resistance of the material to penetration. Due to the apparent efficiency of the procedure, the idea of using it was independently conceived by investigators working in different parts of the world; by the Swedish Railroad Commission for the investigation of earth-slides, operating during the years 1914–1922; by the writer in Constantinople in 1923; by Dr. E. M. Kindle in Ottawa, Canada (1925), and quite recently by the committee for the standardization of loading tests of the Austrian Society of Civil Engineers.

The following paragraphs contain a brief description of some of the experimental facts brought out by these tests.

TESTS OF THE SWEDISH RAILROAD COMMISSION [1]

For testing the consistency of drill samples the Swedish Railroad Commission used cones with weights of 60 and 100 grams and with vertex angles of 60°. In addition to this, systematic investigations were carried out with cones having weights of 10, 30, 200, and 300 grams, and with points of 30° and 90°. The material was contained in a soil cup, with a diameter of 6 centimeters. Prior to the test a cone was fixed in such a position that the point just touched the surface of the material. The cone was released and allowed to freely penetrate the material and the measurement of the depth of penetration was made as soon as the cone came to rest.

The depth of penetration of the 60-gram (or of the 100-gram) cone served as a basis for computing the consistency factor of the material. The consistency factor is the weight of the cone required for producing a standard penetration of 4 millimeters. Determination of the consistency factor was made by means of a diagram obtained in the following manner:

Suppose we increase the water content of a sample of clay by adding water to it. As a result of this procedure both the softness of the clay and the depth of penetration of a cone with a standard weight of 60 grams increases. Then let us plot a diagram, wherein the abscissas represent the softness of the clay and the ordinates the depth of penetration of cones. If we consider the depth of penetration of the standard cone with a weight of 60 grams as a scale for the softness of the clay, the penetration curve of the standard cone will be a straight line, passing through the origin and bisecting the angle between the two axes. In order to find the penetration curves for cones 1, 2, 3, etc., with a weight other than 60 grams, we take a sample with a defined water content, into which the standard cone penetrates to a depth of say t millimeters, determine the depth of penetration t_1, t_2, t_3, etc., of the other cones, and plot all the values thus obtained on the same ordinate which corresponds to the depth of penetration t of the standard cone. By repeating the same procedure on samples into which the standard cone penetrates to a depth of t', t'', etc., we obtain a series of points, t_1, t_1', t_1'', etc., t_2, t_2', t_2'', etc. These points determine the penetration curves for the cones 1, 2, 3, etc. From this graph one can easily derive another one, in which the abscissas represent the depth of penetration of the standard cone, whereas the ordinates represent the weight of cone required to produce a standard penetration of 4 millimeters. Since it was agreed to call these weights *consistency factors*, the curves thus obtained are called the *consistency curves*. By means of the consistency curves, the consistency factor of any sample can be determined, provided the depth of penetration of the standard cone is known. Hence the reliability of the procedure depends essentially on the reliability of the consistency curves.

The results of the investigations, which obviously involved a very considerable amount of time and labor, disclosed some facts which seriously affect the value of the method. It was found that the depth of penetration of the standard cone into the undisturbed material was invariably smaller than the penetration of the cone into the same sample after the sample was remoulded, the water content and the density of the sample remaining unchanged. This means that the consistency of the material depends not only on the water content, but also on the history of the sample. Furthermore, the ratio between the consistency factor of the undisturbed and the disturbed sample was found to be very different for different materials. Hence no conception of the consistency of a soil deposit can be derived except from tests performed on undisturbed samples. Finally it was found, that any consistency curve is valid for one definite family of soils only. Different soils have different consistency curves.

An analysis of the last fact mentioned above led the writer to the following conclusions: If the depth of penetration of the standard cone depended on nothing but the *hardness* of the material, the consistency curves should be valid for all fine-grained soils, irrespective of their physical character. Hence the lack of universal validity of these curves indicates that the depth of penetration depends not only on the hardness but also on a foreign factor which has nothing to do with consistency. This other factor seems to reside in the friction between the soil and the sides of the cone, combined with purely viscous resistances acting while the material is flowing.

YIELD POINT RELATED TO CUBE STRENGTH

Before passing to the investigations of the writer, it seems advisable to insert a brief statement concerning the theory of plastic flow, this theory being essential to a clear understanding of the nature of the processes involved in the cone tests. The theory of plastic flow produced by local application of an external pressure has been worked out during the last few years by Kármán, Prandtl, and Hencky.[2] From the investigations of these authors, we know that the hardness of plastic materials (resistance against penetration of a solid body into the material) is a simple multiple of the cube strength of the material. In turn, by definition, the hardness of plastic materials corresponds to the yield point, because under a pressure greater than the hardness the material flows. Hence the yield point and the cube strength of plastic materials are connected with each other by a simple and well-defined relation. This theoretical conclusion has been repeatedly verified by experiment. Thus, according to theory, the resistance of a material against penetration of a body with a long, narrow cross section should be equal to 2.57 times the cube strength of the material, regardless of

[1] Statens Järnvägars Geotekniska Kommission 1914–1922, Stockholm, 1922.

[2] L. Prandtl, Über die Härte plastischer Körper. Nachrichten von der K. Gesellschaft der Wissenschaften zu Göttingen, Math. phys. Klasse, 1920, and L. Prandtl, Anwendungsbeispiele zu einem Hencky' schen Satz über das plastische Gleichgewicht. Ztschr. für angewandte Mathematik und Mechanik. Bd. 3, Heft 6, 1923.

whether the material is hard, like a metal, or as soft as putty. Tests performed with steel edges pressing against the plane surface of wrought iron showed that the ratio between resistance against penetration and cube strength was, for this particular case, equal to 2.31, against the theoretical 2.57.

Recognizing the importance of the relation between cube strength and consistency (hardness, resistance against penetration, yield point, etc.) the writer started in 1920 to develop a method for measuring the cube strength and the elasticity of very soft materials. The results furnished by the tests were intended to be used in estimating the bearing capacity of natural soil deposits. The test method proved to be successful. However, the time required for testing one sample amounted to at least two hours. In order to simplify the procedure, the writer started in 1923 to investigate whether or not it was possible to compute the compressive strength of any soft material from the penetration of a cone with a standard weight, dropping from a standard height, or from the penetration of a cone forced into the material by a static pressure.

The investigations were carried out with a set of samples, extracted with drills from a mud deposit located at the Golden Horn in Constantinople, on similar sets of samples from drill holes through glacial clays in Detroit, Mich.[3] and on samples obtained from a soft deposit of glacial clay in Cambridge, Mass.[4] The samples were obtained by means of special samplers attached to the lower end of a set of rods. Whenever the ground encountered in the test boring seemed to change, the sampler was forced into the ground to a depth of 2 to 3 feet below the bottom of the drill hole. Then the sampler was withdrawn and detached from the rods. Finally both ends of the sampler were covered with paraffined cardboard and sealed by dipping each end three times into liquid paraffin, so as to keep the water content of the sample unchanged.

Two types of apparatus were used for making the tests: a Vicat needle device in which the needle was replaced by a cone with a 90° point, having a standard weight of 112 grams, dropping freely from a standard height of 4 centimeters (apparatus *A*); and a lever device with an automatically sliding weight, acting on a cone with a 90° point, whose point, at the outset of each test, just touched the plane surface of the sample (apparatus *B*).

The investigations included the following types of tests: Penetration tests with both apparatus *A* and *B* on undisturbed samples (soil still contained in the sampler), on disturbed samples (soil taken out of the sampler and transferred to a cylindrical mould), and finally cube tests, performed for the purpose of finding out whether or not a definite relation exists between the cube strength of the materials and the figures furnished by the penetration method.

Figure 1 shows the results of tests obtained by means of apparatus *A* on remoulded samples from the mud deposits located at the shore of the Golden Horn in Constantinople. Since the material was very soft, it was transferred from the samplers into a cup with a depth of 1½ inches and a diameter of 3 inches. The ordinates represent the cube strength of the samples, determined by unconfined compression tests. When selecting the abscissas for the diagram, the writer was guided by the following consideration:

Let

t be the penetration of the dropping cone in millimeters

r the radius of the hole, punched by the cone

q the cube strength of the material, and

Q the force equivalent to the impact produced by the dropping cone.

Since the vertex angle of the cone was equal to 90°, we have

$$r=t$$

and the vertical projection of the area over which the force Q acted was equal to

$$\pi r^2=\pi t^2$$

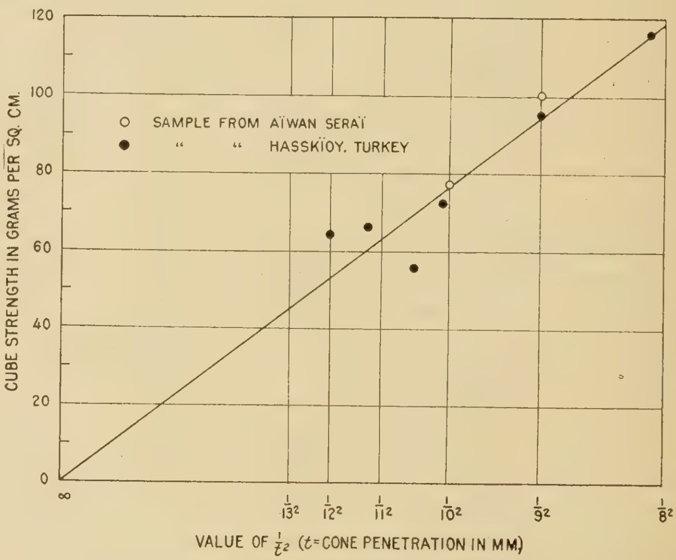

Fig. 1.—Relation Between the Cube Strength of Mud Samples and the Corresponding Penetration t

According to the theory of plastic flow, the yield point is equal to

$$c_1 q$$

wherein c_1 is a constant for all plastic materials. The cone ceases to penetrate the material as soon as the pressure per unit of area becomes equal to the yield point. Hence, combining the aforementioned relations, we obtain

$$\frac{Q}{\pi r^2}=\frac{Q}{\pi t^2}=c_1 q$$

or

$$q=\left(\frac{Q}{\pi c_1}\right)\frac{1}{t^2}=\text{constant}\times\frac{1}{t^2}.$$

If the formula is correct the relation between the cube strength and the quantity $\frac{1}{t^2}$ should, in a diagram, be represented by a straight line. For this reason we have plotted in the diagram the values $\frac{1}{t^2}$ as abscissas. As a matter of fact, the points obtained by actual observation are very nearly located on a straight line, which indicates that the cone test furnished fairly reliable values for the relative "consistency" of these particular mud samples.

[3] Tests made by Metcalf & Eddy, consulting engineers, Boston, Mass., according to instructions furnished by the writer.

[4] Tests by E. S. Sheiry, Massachusetts Institute of Technology, under the supervision of the writer.

Figure 2 shows the results of similar tests performed on drill samples from the blue clay deposit located beneath the Massachusetts Institute of Technology in Cambridge, Mass. The apparatus used for these tests was identical with the one used for the tests of Figure 1. The procedure was as follows: After removing the seals from the two ends of the sampler, a piston was forced into the sampler from one end, thus forcing about one-half inch of the material out of the tube. After removing the material projecting out of the tube, the surface

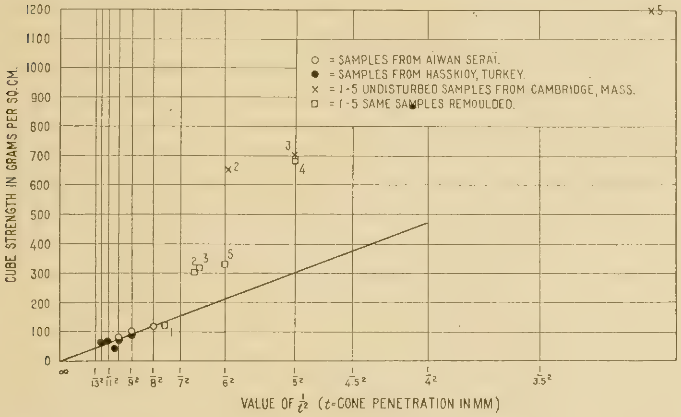

Fig. 2.—Relation Between the Cube Strength of Clay and Mud Samples and the Corresponding Penetration t of a Standard Cone, Freely Dropping From a Standard Height of 4 Centimeters

thus exposed was smoothed off by means of a straightedge, and the cone was dropped in its center. This test furnished the depth of penetration in the "undisturbed" sample. Then an additional quantity of about 1 cubic inch of clay was forced out of the sampler, and molded into a cylindrical mold with a height of 1 inch. The cone test performed on the smooth top surface of this sample furnished the depth of penetration in the "disturbed" sample.

RESULTS OF CONE TEST FOUND TO BE MISLEADING

If the penetration of the cone depended on nothing but the yield point of the material, the points obtained from the tests should be located on the straight line determined in Figure 1 and shown also in Figure 2. Figure 2 shows, however, that this is by no means true. The points are scattered over the area in a rather erratic manner. In contrast to the Constantinople samples, the nature of the material of the Cambridge samples varied between rather wide limits. The observed facts suggested the following conclusions: The relation between the yield point and the penetration of a freely dropping cone is very different according to the nature of the material. The results furnished by the test may be misleading, unless the test is used for estimating the relative consistency of materials with a very similar character.

On account of the rather discouraging results with apparatus A the construction of apparatus B was decided upon. The material was prepared in the same manner as for the former tests. The force acting on the cone increased from zero at a constant rate of 30 grams per second. Every fifth second the depth of penetration of the cone was observed and recorded. Figure 3 shows the result of tests performed on an undisturbed and on a disturbed sample of a pale blue, glacial clay from a depth of 90 feet below the surface, with a liquid limit of 48.4 per cent, a plastic limit of 24 per cent, and a water content of 44 per cent. Figure 4 shows the results of similar tests performed on a clay from a depth of 44 feet, liquid limit of 49.7 per cent, plastic limit of 23 per cent, and a water content of 40 per cent.

The relation between the load acting on the cone and the corresponding depth of penetration should be as follows:

Let

Q be the force acting on the cone,

t the depth of penetration of the cone in millimeters, and

$r=t$, the radius of the vertical projection of the area over which the force Q acts.

According to the theory of plastic flow, the yield point should be independent of the area over which the pressure acts. Hence, if we plot the values of Q as abscissas and the values of t^2 as ordinates, the points obtained from each test should be located on a straight line, because

$$\frac{Q}{\pi r^2}=\frac{Q}{\pi t^2}=\text{constant}$$

or

$$Q=(\text{constant})\times t^2=c_1 t^2$$
$$\pi\times$$

Figures 3 and 4 actually show that the straight line relation expressed by this formula approximately exists. In addition to this, they show in a very striking manner the importance of the influence on the yield point of remolding the samples of unaltered

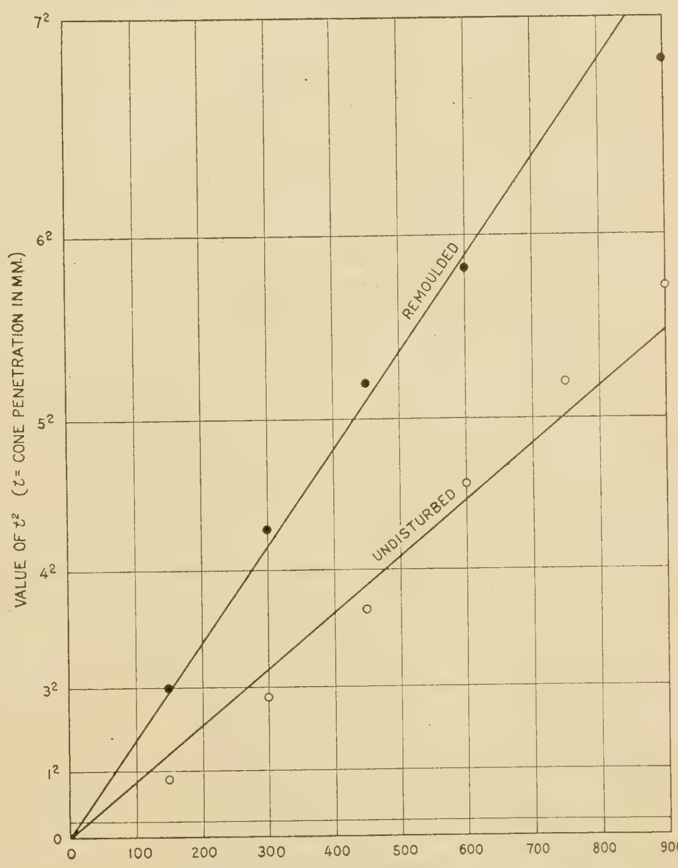

Fig. 3.—Relation Between Weight Acting on Cone and Square of Penetration of the Cone Into Sample No. 2

water content. The abscissa of the intersection between the penetration lines and a horizontal line with ordinates equal to 4^2 obviously corresponds to the consistency factor of the Swedish Railroad Commission.

YIELD POINT OF UNCONFINED SPECIMEN

In order to correlate the results of cone penetration tests with the yield point of the material, compression tests have been made on unconfined, cylindrical specimen of different clays. Figure 5 shows the

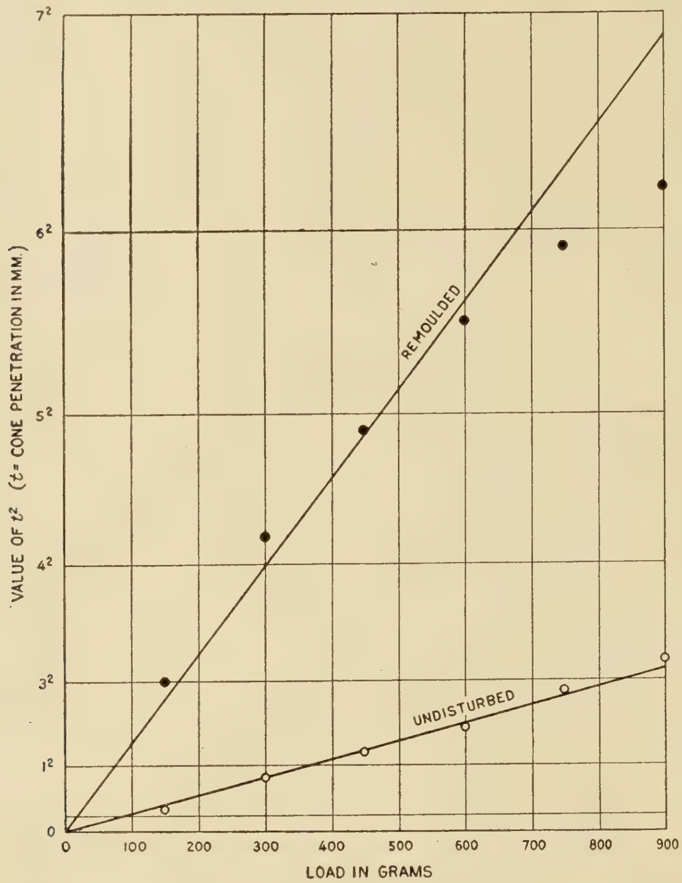

FIG. 4.—RELATION BETWEEN THE WEIGHT ACTING ON CONE AND SQUARE OF PENETRATION OF THE CONE INTO SAMPLE No. 5

results of the compression tests performed with undisturbed and the disturbed materials of Figure 3, and Figure 6 the results of similar tests made on the materials of Figure 4. The tests were made on cylinders 1 inch high, with a diameter of 1 inch. During the tests the cylinders were surrounded by a humidifier, and the remolding was done so rapidly that the water loss due to evaporation associated with the remolding process was practically negligible.

The load was changed (increased or decreased) by equal increments in equal time intervals at such a rate, that every test lasted about two hours, the time required for the time observations *a*, *b*, *c*, etc., at constant load excluded.

If in such a test a new increment is added to the load, there follows an almost instantaneous compression. Then, as time goes on, the compression gradually increases at a decreasing rate, until finally equilibrium is reached, the time increase of compression becoming equal to zero. On the other hand, on removing precisely the same load increment, the specimen almost instantaneously expands. As time goes on, the expansion continues at a decreasing rate, until finally, after a couple of hours, expansion ceases. These phenomena are characteristic of the effect of pressure below the yield point and at the same time correspond in every respect to the effect of pressures on solid bodies. Thus, from the results of experiments with various metals, Michelson[5] concluded that a change in stress of a solid body involves four different strain effects: (1) Rapid elastic yield; (2) rapid inelastic yield (lost motion); (3) slow elastico-viscous displacement; and (4) slow viscous (nonreversible) displacement. The relation between time and each one of these four types of strain effects in solid bodies closely resembles those which exist for clays and similar nonliquid materials, regardless of how stiff or fluid they may be, provided the pressure which causes the strain effects be below the yield point. A discussion of the causes of the lag (retardation) of the strain has been presented elsewhere.[6] For our purposes it is sufficient to retain the following two facts:

The application of a pressure on clay always involves a viscous flow, regardless of whether the pressure be below or above the yield point.

If the pressure is below the yield point, the speed of flow decreases at constant pressure until it finally becomes equal to zero.

The maximum pressure (or, in the case of cube tests, the maximum load per unit of area) under which a state of final equilibrium can still be reached, corresponds to the *yield point*. Hence the time-compression curves for loads below the yield point approach a horizontal tangent, while the corresponding curves for loads above the yield point have no such tangent.

In Figures 5 and 6 the yield point corresponds to the abscissa of the vertical tangent to the pressure-compression curve marked "T" because the pressure compression curve joining this tangent means infinite increase in compression at no increase in load.

Tests can not be carried to the point where the compression curve becomes vertical but the position of the vertical asymptote can be estimated with a sufficient degree of accuracy. In the right lower corner of Figures 5 and 6, the increase of compression at constant load for different loads are shown for both undisturbed and remolded samples.

The degree of elasticity of the samples is determined by the reversible part of the compression and can be expressed by the modulus of elasticity, which in turn is equal to the cotangent of the angle formed between the pressure axis and the hysteresis loops.

The samples represented by Figures 5 and 6 had the following characteristics:

	Modulus of elasticity	Yield point	Ratio of yield point to modulus of elasticity
	Kg. per sq. cm.	*Kg. per sq. cm.*	
Sample No. 2 undisturbed	37	0.65	0.0176
Sample No. 2 remolded	25	.30	.0120
Sample No. 5 undisturbed	66	1.20	.0182
Sample No. 5 remolded	25	.33	.0132

[5] The Laws of Elastico-Viscous Flow, by A. A. Michelson, Proceedings of the National Academy of Sciences, May 15, 1917.
[6] Terzaghi, Erdbaumechanik, Wien, 1925.

COMPRESSIVE STRENGTH AND CONE PENETRATION

The results of the cube tests furnished the basis for correlating the yield point with the data furnished by the penetration tests. From these results, the relation between the cube strength q of the material and the unit resistance $p=\frac{Q}{\pi r^2}=\frac{Q}{\pi t^2}$ to the penetration of the cone (yield point) was found to be as follows:

Sample	q	$p=\frac{Q}{\pi t^2}$	$\frac{p}{q}$
	Gms. per sq. cm.	*Gms. per sq. cm.*	
No. 2, undisturbed	650	956	1.47
No. 2, remolded	300	548	1.83
No. 5, undisturbed	1,200	2,960	2.47
No. 5, remolded	330	600	1.82
No. 3, undisturbed	700	1,310	1.87
No. 3, remolded	320	711	2.23

If the test with apparatus B were an ideal consistency test, the ratio $\frac{p}{q}$ should be a constant and entirely independent of the nature of the material. In spite of all the tests having been made with blue glacial clay, the ratio varies by about ± 25 per cent, the average value being equal to 1.95.

The most conspicuous result brought out by the tests concerns the wide range in variation of the ratio between the cube strength of disturbed and undisturbed samples. The cube strength of the remolded samples ranges anywhere between 0.55 and 0.25 of the cube strength (yield point or consistency) of the undisturbed samples. Hence there seems to be no chance to evaluate the actual consistency of a soft, water-bearing deposit, except by experimenting with undisturbed samples, and the technique of securing undisturbed samples represents as important a problem as the technique of testing the samples.

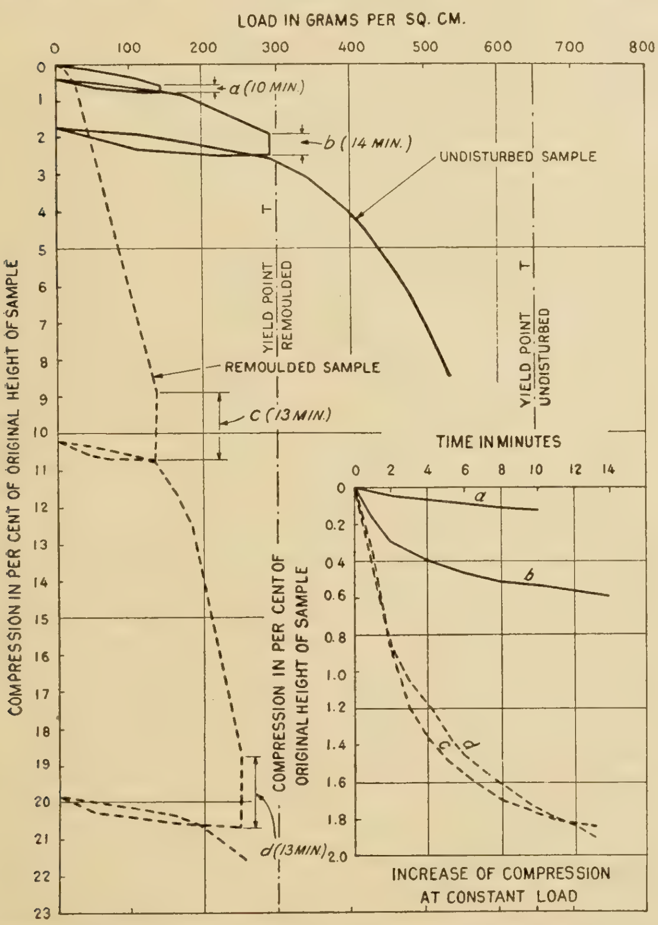

Fig. 5.—Compression Test on Unconfined Specimen of two Identical Clay Samples (No. 2) With Identical Water Content one of Which was in the Original State and the Other Remolded

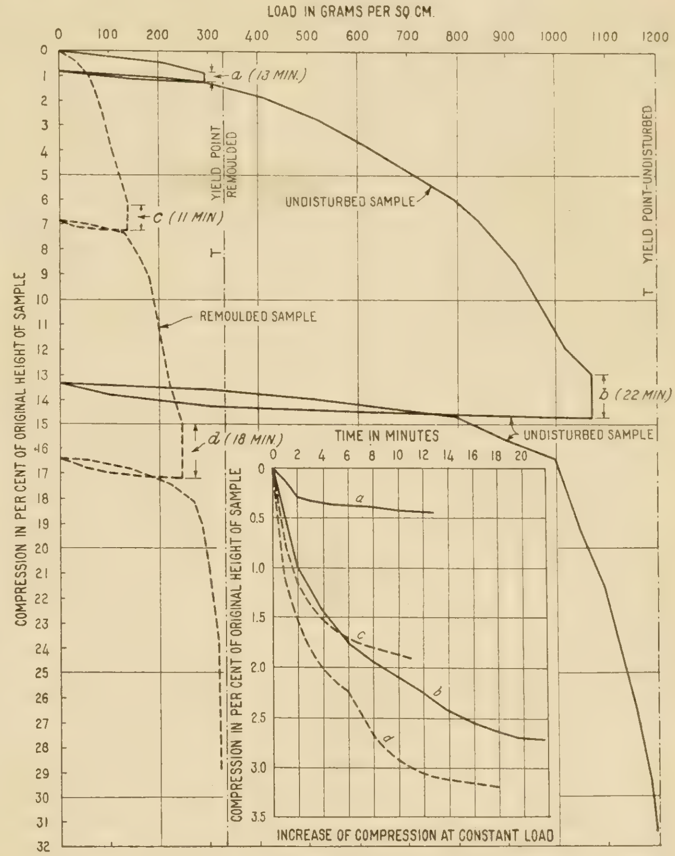

Fig. 6.—Compression Tests on Unconfined Sample of Two Identical Clay Samples (No. 5) With Identical Water Content One of Which was Undisturbed and the Other Remolded

FRICTION AFFECTS VALUE OF KINDLE'S TEST

In contrast to the writer, Doctor Kindle attacked his problem from a purely empirical point of view. He compared materials ranging in hardness between talcum and cohesionless sand with the successive stages through which a mixture of neat cement and water passes during the process of setting, and tried to work out an experimental method for correlating the consistency of any soft material with the corresponding consistency of a mixture of neat cement and water. This method of attack is very original and appealing on account of its apparent simplicity. It furthermore differs essentially from the methods used by the Swedish Railroad Commission and by the writer, inasmuch as it is intended to include not only clay and mud, but all other kinds of soft materials, plastic and brittle. It involves, however, from the very beginning a serious difficulty. Consider, for example, the two following materials: A sand whose grains are slightly cemented together and a soft clay. Both materials may have, according to any consistency test (cube test, bearing-capacity test, cone-penetration test, etc.) precisely the same resistance. Nevertheless it would be most confusing to consider the consistencies of these materials as equivalent, because there is obviously a second factor which must be taken into consideration, the deformability of the material According to the degree of deformability, the mechanical behavior may or may not be governed by the laws of plastic flow, a difference which is at least as important as a difference in consistency. Hence the consistency of the materials can not possibly be represented by a single figure, even in the restricted sense of the word, as used in this paper.

Disregarding this initial difficulty, we may pass to the test method itself.[7] In order to compare the consistency of two materials, Doctor Kindle measures the penetration of a one-half inch rod with a sharp point, freely dropping from an elevation of 10 inches. Upon analyzing the process of penetration, one realizes that the depth of penetration depends not only on the cohesion of the material, but also on the friction between the surface of the rod and the adjoining material. This second factor is something completely independent of cohesion. In addition to this the intensity of the frictional resistance is strongly influenced by dynamic lubrication phenomena. Thus the coefficient of friction between the point of the rapidly penetrating rod and the adjoining material is apt to amount to only a small fraction of the coefficient of friction between the same material and the surface of a slowly penetrating point. In turn, each of these two coefficients may be very different from the coefficient of friction between the point and a mixture of cement and water of equal consistency.

These friction phenomena are most troublesome wherever they develop in connection with experimental work. Due to the effect of dynamic lubrication it is impossible to establish a universally valid pile-driving formula. The same effect introduced an erratic element into the cone tests described in the first paragraphs of this paper. Since the outcome of Doctor Kindle's penetration test depends to a large extent on friction, the figures furnished by the tests can not possibly be considered as a standard for comparing consistencies. They represent the combined result of two independent factors—consistency and friction. The friction effect is complicated by dynamic lubrication and seems to have a far greater influence on the outcome of the tests than the consistency.

Three years ago the writer investigated the effect of the shape of the penetrating body on the ultimate bearing capacity of cohesionless sand[8] and found that the resistance against penetration of a cylindrical body with a flat bottom and with a diameter of 3.9 centimeters acting on the plane surface of the sand is about 20 per cent greater than the resistance against penetration of a cylinder of equal diameter, terminating in a cone with a vertex angle of 60°. On the other hand, several years ago, similar tests were made near Vienna, on cohesive loess.[9] The diameter of the loaded area was equal to 28.1 centimeters. In contrast to what has been stated above for the sand, the resistance of the loess against penetration of the conical bodies (vertex angle of 90° and of 35°) seemed to be somewhat in excess of the ultimate bearing capacity for a load acting on a plane surface. Hence the ratio between the bearing capacity of the ground for plane and for conical loaded surfaces may be different for different materials. This fact ought to be kept in mind when attempting the interpretation of cone penetration tests.

CUBE STRENGTH MOST PROMISING INDEX OF CONSISTENCY

From the preceding remarks we learn that the proposed methods for determining the "consistency" of a material are still far from ideal. For the time being the most promising method consists in measuring the cube strength directly, by a simple, nonconfined compression test, and to consider the yield point of the cubes as a measure of the consistency because, according to the aforementioned theory of plastic flow, the

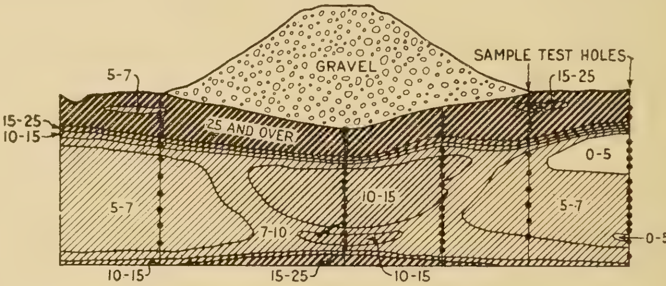

FIG. 7.—DIAGRAM SHOWING CONSISTENCY FACTORS OF SOIL BENEATH A GRAVEL FILL AT SMEDSEROD, SWEDEN. COPIED FROM REPORT OF SWEDISH RAILROAD COMMISSION

yield point of an unconfined cylinder strictly determines the ultimate resistance of a plastic material under any other conditions of confinement.

It remains to decide to what extent a consistency coefficient thus obtained may serve a practical purpose. There are three principal fields of application for consistency tests:

(*a*) Applied geology, the consistency test serving as a means for expressing the hardness of materials (rocks, soils, muds, etc.) softer than talcum.

(*b*) Foundation engineering, the consistency test to be used for investigating the degree of stiffness of soil intended to support structures and artificial loads.

(*c*) Subgrade investigations, the consistency test to be used for expressing the resistance of the toplayer of the ground, located immediately beneath the road surface.

The application of the consistency test to soft geological formations is severely handicapped by the great variation in the physical character of the softer formations. The common rocks and minerals included in the traditional scale of hardness represent practically homogeneous solid bodies. The homogeneous solid bodies belong, physically speaking, in one class, and their hardness can be described by a single figure.

In contrast to this, the soft materials may be either porous solid bodies (chalk, etc.) or two-phase systems (saturated silts, clays, etc.) or three-phase systems (dry clays or loams). These three groups of materials are physically so different that it is impossible to establish a scale of consistency valid for all of them. The hardness of the materials of the first and the third group may best be expressed by the cube strength (breaking strength) of the material or by any other method used for measuring the hardness of brittle solid bodies. The consistency of the materials of the

[7] "A proposed scale of hardness and cohesion for rocks," an unpublished article by E. M. Kindle.
[8] Terzaghi, Erdbaumechanik, Wien, 1925, p. 232.
[9] From an unpublished report by Dr. Ing. Fritz Emperger.

second group is also primarily determined by the cube strength, but the cube strength has in this case a different meaning. It corresponds, not to the breaking strength of brittle substances, but to the yield point of plastic materials. Hence, in order to define the hardness of a soft material, it is not sufficient to mention the cube strength. It is necessary to state in addition, at least, whether the material was crushed or whether it flowed under pressure, and whether the material disintegrates in water or whether it retains its strength. For clays it will, in addition, be required to determine the consistency in an undisturbed state.

The most promising field of application of the consistency tests concerns the stiffness and the bearing capacity of compressible substrata (muck, silt, clay) located beneath the depth of seasonal variations in water content. Figure 7 shows the change in consistency of a bed of silt produced by the superimposed weight of a gravel fill. The investigations which led to this diagram were made by the Swedish Railroad Commission, by means of the cone penetration method described in this paper. The diagram shows very clearly the general subsidence associated with an increase of the consistency of the strata located beneath the fill. Yet the data have a relative value only, because the tests have been made with remolded samples. At present our efforts concentrate on improving the methods for obtaining undisturbed samples and on testing these samples in their original condition.

When attempting to apply consistency methods to the problem of predicting the behavior of a subgrade to be located immediately beneath the surface of a road, one meets the same difficulties which make it impossible in the laboratory to go beyond an attempt to classify the raw material of the subgrade. In the field the subgrade undergoes periodical changes between the state of almost complete saturation and the state of dryness. As a consequence its consistency varies between very wide limits. In the preceding paragraphs it has been shown that the consistency of two identical samples with identical water contents may be very different according to the preceding history of the sample. It would be still more difficult to estimate in advance the minimum consistency of a material under conditions which we can not possibly reproduce in the laboratory. Hence in subgrade investigations the consistency tests can not possibly be utilized except in connection with laboratory procedure for classification purposes, and without expecting the test results to have a direct bearing on the behavior of the soil in the field.

(Continued from page 239)

hard bottom is reached, the bending stresses induced in the piling during backfilling can easily cause failure where sufficient penetration of the mineral subsoil can be had to give a good piling anchorage. If penetration of the piling is limited by a hardpan layer below the peat, sliding of the bottom of the piling may take place. If the deposit is deep, the piling may be flexible enough so that it acts as a cantilever fixed at the bottom and the whole structure may be moved in one direction. The forces due to the compression and flow of the peat during filling have been the cause of a number of bridge failures.

In one case in Michigan in 1916, a 40-foot bridge was put in, supported by 50-foot spliced piling. Some time after filling of the approach was started the bridge began to move, slid along the axis of the roadway and inside of a week had completely disappeared from sight. In another case in a peat deposit about 40 feet deep, a 40-foot bridge rotated about 5° about one end, and settlement then began in sufficient amount to destroy the bridge. Had the approach fills been made first, these failures would not have occurred.

The maintenance of a road surface which is settling is a comparatively difficult matter to handle. In a number of cases where the amount of settlement was not over a foot or two and a concrete surface had been placed, the slabs have been jacked back to grade and the space beneath the slab refilled with tamped earth. It is for this reason that mesh reinforcement is desirable, as it has been found that reinforced slabs can be jacked back into place with very much less breakage than the plain slab. In other cases, where the settlement has been accompanied by considerable slab breakage, it has been necessary to remove the surface, bring the fill back to grade, and replace the surface. Where the amount of settlement is considerable it may be possible to effect a remedy by removing only a part of the pavement at each end of the settlement and placing new sections which will join the sunken slab properly to the surface adjacent to the settlement.

It is unfortunate that the present methods of filling have been used for so short a time, as it is impossible for us to accurately estimate the amount of filling material necessary with methods 4 and 5. Quantities in each case will be less than those indicated by curves derived from an investigation of fills which have been placed full width on the surface of the undisturbed peat. It is planned in the near future, as soon as a sufficient number of these examples are available, to cross section and determine the amount necessary, using the new method. We are confident that the methods proposed will eliminate a great deal of the settlement formerly encountered, and will involve the use of smaller amounts of material in filling.

CALIFORNIA ROAD SURVEY DEMONSTRATES THE ECONOMIC POSSIBILITIES OF SUBGRADE STUDIES

Reported by C. A. HOGENTOGLER, Highway Engineer, Bureau of Public Roads

EXAMINATION of the data on subgrades and road condition collected in the study of California highways in 1920[1] shows convincingly the value of considering subgrade conditions in highway design.

In this survey the concrete roads were classified according to condition as follows:

(*a*) A pavement in which the plainly visible tranverse cracks do not exceed the normal number expected of a pavement constructed without expansion joints, and which has no plainly visible longitudinal cracks.

(*b*) A pavement having more than the normal number of plainly visible transverse cracks or with some "crowfoot" cracks at the edges or with both.

(*c*) A pavement similar to classes *a* and *b* and with one plainly visible longitudinal crack or with a considerable number of "crowfoot" cracks.

(*d*) A pavement so cracked transversely and longitudinally that numerous slabs are formed of less area than class *c*, but that do not average less than about 50 square feet.

(*e*) A pavement in which the plainly visible transverse and longitudinal cracks are so numerous that it is broken into slabs having areas less than about 50 square feet but in which no general disintegration appears.

(*f*) A pavement badly broken and with disintegrated portions.

The subgrade classification was made in accordance with the practice of the U. S. Bureau of Soils as follows:

Class 1. Clay and adobe soils (includes clay, silty clay, clay-loam and clay).

Class 2. Marsh lands (includes salt marsh and peat lands).

Class 3. Loams (includes loam, clay-loam, silty loam and silty clay-loam).

Class 4. Sandy loam (includes coarse sandy loam and fine sandy loam).

Class 5. Sand, and sand and gravel.

TABLE 1.—*Condition classification of concrete roads on different types of subgrade, based on study of 1,200 miles of California road in 1920*

Type of subgrade[1]	Class of pavement					
	A	B	C	D	E	F
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sand	68.4	21.4	10.2			
Sandy loam	44.2	38.8	11.0	3.6	0.9	1.5
Loam	34.8	39.0	18.1	4.2	1.1	2.8
Adobe and clay	21.4	37.8	20.4	10.8	4.6	5.0

[1] Types classified in accordance with the practice of the U. S. Bureau of Soils.

In the California report, about 1,200 miles of concrete surface and the corresponding subgrade were classified according to the above systems, with the result shown in Table 1. These roads were practically all of the standard California 4½-inch cross section used prior to 1920. They were constructed during the period 1912 to 1920 and the construction was distributed over different types of soils in approximately proportionate amounts throughout this period.

In the large mileage of pavement inspected and classified there were sections of various ages from one to eight years and sections which had been subjected to traffic of various degrees of density from light to heavy on each of the types of subgrade. In other words, there were pavements laid on sand subgrades which had been in service for varying periods, and there were pavements laid on adobe and other types of soil which had also been in service for various lengths of time; similarly, there were pavements on sand subgrades which had been subjected to light traffic and others which had been subjected to heavy traffic, and the same may be said of the pavements laid on adobe and other types of soil. In view of these conditions and the large mileage involved it may be assumed that major differences found to exist in the percentage and degree of failure of pavements on the various types of subgrades were due in greater degree to the character of the subgrade than to the age of the surfaces or the density and weight of the traffic.

There may be some differences of opinion as to the details of road and soil classification, whether the road condition classes should be based on the spacing of cracks and area of resulting blocks or on length of cracks and extent of breakage, and whether the soils should be classified according to type or according to certain laboratory tests. Whatever the opinion in regard to these matters it can influence but little the outstanding conclusions to be drawn from the comparative data in Table 1. Classification by other proposed systems which definitely identify road condition and soil characteristics would warrant the same conclusions.

These conclusions are as follows:

1. That subgrade character has a controlling influence on road condition and outweighs the traffic variable in general comparisons.
2. That a surface design which is adequate for one type of soil may be inadequate for an inferior type.
3. That within a given soil type certain undesirable conditions of subgrade may develop which will make inadequate a pavement design adequate where these conditions do not exist.
4. That, other things being equal, these undesirable subgrade conditions increase with increase of clay content.
5. That soils can be grouped with regard to the occurrence of these undesirable conditions of support, although the best classification may not yet be determined.
6. That information on the relative tendency of the various soil types to develop undesirable subgrade conditions and on the causes and corrective measures for

[1] Report of a study of the California highway system, 1920.

these undesirable conditions can result in enormous savings in the ultimate cost of roads.

Table 1 indicates that, in general classifications, the traffic is subordinate to the subgrade variable. While no correction was made for differences in traffic, the percentage of good pavement consistently decreases and the percentage of bad pavement correspondingly increases as the supporting value or some other desirable characteristic of the subgrade decreases.

In comparing but two roads, difference in traffic undoubtedly will be reflected, but when large portions of highway systems are used for study the traffic variable seems subordinate.

This coincides with the findings of the highway research board which were to the effect that the influence of traffic on concrete pavement condition was subordinate to those of age, subgrade and design of slab.[2]

Table 1 demonstrates that a design of pavement which is adequate on sand subgrades may be inadequate on clays. This can be better demonstrated by a second tabulation. Since the California report, in further explanation of the road condition classes, states that classes A, B, and C were equally serviceable for traffic, and also since the adoption of the center joint design would place class C pavements with classes A and B, these three classes are grouped in Table 2 as desirable pavements and classes D, E, and F are grouped as undesirable pavements.

TABLE 2.—*Pavements on different types of subgrade classed as desirable or undesirable*

Type of subgrade	Desirable	Undesirable
	Per cent	*Per cent*
Sand	100.0	
Sandy loam	94.0	6.0
Loam	91.9	8.1
Adobe and clay	79.6	20.4

Table 2 shows clearly that the design of pavement used was adequate for sand subgrades but that other subgrades require either a change in design or corrective measures in varying amounts. This indication is supported by findings of the highway research board and also by behavior of certain roads in both New York and Delaware.

Table 2 indicates that road failures are caused by certain undesirable conditions found in portions of the various soil types. Even on the worst subgrade (clay) about 80 per cent of the pavements were in desirable condition. A further study of this table indicates that but three soil classifications are needed in this case and the revision is shown in Table 3.

TABLE 3.—*Pavements classed as desirable or undesirable using only three subgrade classifications*

Type of subgrade	Pavement condition	
	Desirable	Undesirable
	Per cent	*Per cent*
Sand	100.0	
Loam	93.0	7.0
Clay	79.6	20.4

[2] Proceedings of the Fifth Annual Meeting of the Highway Research Board, 1925, Part II.

This tabulation plainly indicates that the undesirable conditions increase with increase of clay content and also that soils can be classified with regard to their influence on pavement condition, thus supporting indications 4 and 5.

As an example of the value of subgrade information to the engineer let it be assumed that 1,000 miles of pavement are to be constructed; that the subgrades consist of 200 miles of sand, 300 miles of loam, and 500 miles of clay, and that conditions are similar to those existing in California.

In the absence of definite information on subgrade influence, a pavement of uniform design and adequate to meet the undesirable conditions in the clays and loams would probably be adopted. Referring to Table 3, this means that on the loams a pavement more than adequate for 93 per cent of its length would be used because of undesirable conditions existing on but 7 per cent, and in the clays a pavement more than adequate for 80 per cent of its length must be laid because of undesirable conditions found on 20 per cent, or under the assumptions stated above, 879 miles of our total length would be more than adequately improved because of undesirable conditions existing on 121 miles. Under the above conditions and with an assumed pavement cost of $21,000 per mile, the cost of our system would be $21,000,000.

In contrast to this procedure, if the highway engineer had definite information on the types of soils in which undesirable conditions exist and also on the location, cause and corrective measures for these conditions, a design adequate for the 879 miles of pavement would be used and corrective measures would be added on the 121 miles.

Just what these corrective measures would cost is not known, but certain roads in Delaware and New York indicate that the difference in support between natural sands or sands and gravels and undesirable conditions found in clay subgrades can be equivalent to at least 1 inch of pavement thickness or about $3,000 per mile. If this indication is correct, it can be seen that on the 879 miles of pavement there exists the possibility of saving $2,637,000 or 12½ per cent of the total cost. Whether or not these actual figures are accepted, the California data plainly show that if accurate subgrade information is obtained and utilized a considerable saving of road funds will result.

These data also point out that the logical method of procedure in subgrade studies is to make:

1. General surveys for determining the influence of various types of subgrades on different types and designs of pavements.
2. Detailed investigations of road failures to determine the causes of undesirable conditions developing in the several soil types.
3. Detailed surveys of subbases, subgrade treatments, drainage types, and change in pavement design to determine the efficiency of the various corrective measures for overcoming undesirable subgrade conditions.

At present 15 States consider subgrade support in pavement design, a fact which reflects increased recognition of the importance of the subject. Practice in Colorado can be described as a good example of the practical application of the present knowledge of

(Continued on page 251)

DESIGN OF A CONSTANT TEMPERATURE MOIST CLOSET

By WALLACE F. PURRINGTON, Materials Engineer, New Hampshire Highway Department

THE TESTING laboratory of the New Hampshire State Highway Department recently found it necessary to design a moist closet to meet the conditions of the new specification of the American Society for Testing Materials, which reads as follows:

"The temperature of the room, the materials, the mixing water, the moist closet and storage tank water shall be maintained as nearly as practicable at 21° C. (70° F.) and the mixing water, moist closet, and the water in the storage tank shall not vary from this temperature more than 3° C. (5° F.)."

This is a much more rigid requirement than that in the older specification, which stated that the air and water should be maintained as nearly as practicable at a temperature of 21° C. (70° F.) and did not give a definite tolerance limit. Testing laboratories may find it necessary to install new equipment to meet the new conditions. In the Northern States the problem is to maintain the necessary amount of heat in the fall and spring, while in the Southern States the greatest difficulty is in reducing heat in summer.

In designing a moist closet for New Hampshire conditions the following features were considered—

1. Insulation against heat and cold.
2. Equipment to generate and control the necessary heat.
3. Equipment to maintain the maximum degree of humidity.

The closet sidewalls were constructed of 6-inch hollow tile and lined with 1½-inch cork board coated with a bituminous mastic one-quarter inch thick. The cork board was cut into slabs 12 by 36 inches in size and cemented to the tile by a rich cement-sand mortar. The cork was treated with a chemical sealer to prevent staining by the mastic. Three coats of a cement-stucco paint were applied to the mastic coat after the wall-board was placed. The room has been in operation for three months and there is not the least sign of

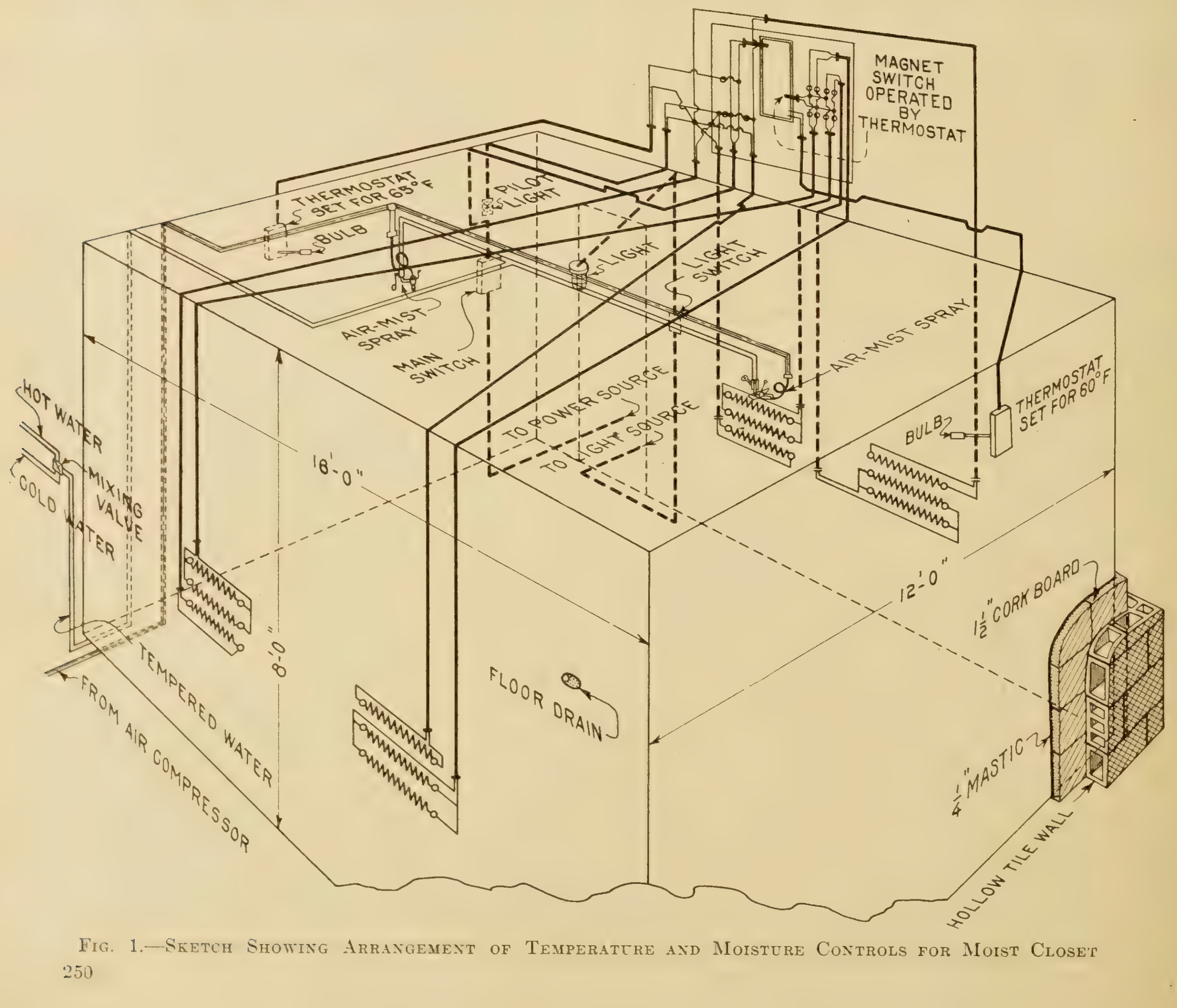

FIG. 1.—SKETCH SHOWING ARRANGEMENT OF TEMPERATURE AND MOISTURE CONTROLS FOR MOIST CLOSET

the mastic causing discoloration. The floor is made of concrete, and the top of the closet is constructed of 4 inches of reinforced concrete. The door is of wood and is lined with cork. This construction has been found to give entirely satisfactory insulation.

HEAT CONTROL EQUIPMENT

Two possible sources of heat were considered, steam from the heating plant of the building and electricity. It was decided that steam heat could not be regulated satisfactorily, while electric heating could be easily controlled, and in addition would be available at all times, although more costly.

The heating units used were 500-watt capacity heaters. Four of these were connected in series and eight in parallel. The series group was connected to a liquid-type thermostat set at 65° F. and the parallel group was connected to a similar thermostat set at 60° F. The latter were wired through a 110-volt, 60-ampere remote control switch. But little use has been made of this group as the temperature has not fallen to the point where it comes on. Figure 1 shows that the heating units are arranged in four groups, each consisting of one of the units in series and two of those in parallel. It is necessary that the heating units, switches, thermostats, and connections be of the best type and not affected by corrosive action in the warm humid atmosphere. These heaters require considerable current and it is necessary that the wires and insulation be of the very best. The main feed consists of No. 6 rubber-covered wire while all outside wiring is No. 4 common covered wire. Owing to the excessive heat and dampness, the wires connecting the fuse block and control switch are No. 14 and of the slow-burning type.

At the present time a recording thermometer has not been installed and it is necessary to take readings at different periods of the day. Typical observations covering a week have been recorded as follows:

Date	Temperature in degrees Fahrenheit		
	8.30 a. m.	12 m.	5 p. m.
Nov. 15	68.7	66.9	66.9
Nov. 16	66.2	68.5	70.7
Nov. 17	70.1	68.9	67.5
Nov. 18	65.7	68.0	68.0
Nov. 19	67.1	69.1	67.1
Nov. 20	66.9	67.1	

EQUIPMENT TO MAINTAIN HUMIDITY

Two humidifiers supply a very fine fog spray. They are connected with water and compressed air. By adjustment it is possible to get nearly any degree of humidity required. During the fall and winter the water is very cold. Unless it is tempered somewhat by the use of warm water and a mixing valve there is a great demand on the heaters to keep the room to the desired temperature. The purpose of the mixing valve is simply to reduce the expense of operating. The heaters will keep the temperature up, however, if tempered water is not introduced. The humidifying apparatus also serves to keep the atmosphere in the chamber in constant motion and at a uniform temperature throughout.

Pans for the storage of briquettes, and 2 by 4-inch cylinders have been installed in this room. The pans are arranged in such a manner that a small trickle of water empties into a holding tank which allows the water to come to the temperature of the room and the water overflows into a pan in which briquettes are stored. In the second pan there is an overflow pipe which empties into another pan below. Five pans, including the storage pan, are placed in this way and are supported by a frame made of angle irons, the joints of which have been electrically welded. This was done to avoid the rusting of screws, bolts, and washers. Two pans much longer and deeper than the one described above are used for the storage of cylinders. The room is large enough to store all the concrete cylinders the laboratory will be called upon to test.

A large stock bottle of distilled water for mixing is kept in this room at all times. Enough is drawn off for each test and as a result the requirement of keeping the mixing water within the tolerated limits can be carried out. Immediately upon making the pats for initial and final set and for the steam test they are placed in this room upon a slate-covered bench and are kept there during the period required.

(Continued from page 249)

subgrades. Prior to construction a detailed survey is made of all subgrades and soil samples are obtained for laboratory test. These test results then determine both the locations for and character of treatment. Subgrade treatments of sand are provided for all locations in which the lineal shrinkage exceeds 5 per cent and the thickness of the sand layer used depends on the extent by which the moisture equivalent exceeds 20 per cent. This is determined by reference to a curve which shows the percentage of sand required to reduce the moisture equivalent from any amount to 20 per cent. While the method of treatment and the limits of 5 and 20 per cent might be subject to variation, the practice of using subgrade information forecasts its utilization in the future on all highways.

EFFECT OF HIGHWAY SLASH ON INFESTATION BY WESTERN PINE BEETLE STUDIED

The Bureau of Entomology of the United States Department of Agriculture has recently completed a four-year investigation of the effect of slash from highway clearing on insect infestation of surrounding timber. The work was conducted by J. E. Patterson, assistant entomologist, on a highway in the Cascade Mountains of southern Oregon.

It has been known that slash in all sections of the country is usually infested by one or more species of two groups of forest insects commonly known as bark beetles and borers. Until recently, however, little had been done to make a scientific analysis of the relation between such infestations and infestations in surrounding timber.

The Bureau of Entomology began its study in 1920 with the clearing of a highway through a larger timber area consisting of yellow pine, sugar pine, Douglas fir, white fir, and true fir. The slash consisted of entire trunks and tops of trees dragged to either side of a clearing 60 feet wide and 24 miles long. In this region the western pine beetle is the principal insect enemy of the mature western yellow pine.

To investigate the relationship of the slash to the attack and breeding of bark bettles, 1 square foot of bark was taken from the middle of the butt cant of each felled tree after the emergence of the broods and entrance and exit holes counted. These data were then compared with data from standing trees which had been attacked by bark beetles.

The conclusions were as follows:

Line slash of the character here considered is very attractive to the bark beetle *Dendroctonus brevicomis*, practically all such slash being attacked by this insect.

The attack of this bark beetle on the slash is not so heavy as its attack on mature standing timber. In the particular case studied approximately one-half as many beetles attacked a unit area of bark on the felled trees as attacked an equal area on standing timber.

The broods developing in slash are characterized by abnormal mortality. The increase of beetles developing in the slash studied was only 64 per cent of the number of beetles making the attack, whereas the corresponding increase in adjacent standing timber at the same time was 135 per cent.

Bark beetles from the surrounding standing timber are attracted to the slash at the time of attack, and a temporary concentration of infestation occurs in its immediate vicinity. Normal distribution of the infestation is resumed within a year.

The concentration just mentioned and the breeding of beetles in line slash do not increase or greatly influence infestations in the surrounding forests. The cycle of an infestation continues regardless of the slash.

This study indicates that the infestation of line slash by *Dendroctonus brevicomis* is not a serious menace to neighboring mature timber, and may be disregarded when the problem of slash disposal is under consideration.

THE ACTION OF CALCIUM CHLORIDE ON CEMENTS[1]

Some interesting observations as to the action of calcium chloride on cements are contained in an article by M. Anstett in Le Génie Civil for October 2, 1926. This article is the result of an investigation on the corrosive action of calcium chloride on steel reinforcement.

It was found that very dilute mixtures of calcium chloride, such as a few grams per liter of mixing water, retarded the setting of the cement quite noticeably. Concentrated solutions, such as 100 to 400 grams per liter, accelerated the setting and hardening. The calcium chloride admixture is a stabilizer of mortar, due no doubt to its affinity for free lime such as is present in varying amounts in all cements. The elimination by combination of this free lime without delay removes a frequent cause of swelling and disintegration of concrete.

Mixed with a strong solution of calcium chloride, the cement acquired considerable compressive strength in a very short time, but samples of a similar mixture disintegrated as a result of immersion in water a few moments after setting. Other specimens that had cured in air for 15 to 20 hours remained intact during submergence in water. Other factors are known to influence the effect of calcium chloride.

[1] Translated and abstracted by C. S. Jarvis, associate highway engineer, Bureau of Public Roads.

To illustrate the influence of this admixture on the soundness, 18 cement pats were prepared, 9 of which were mixed with pure water, and the remainder with 5 per cent by weight of calcium chloride. After curing for a few days in water at 15° C., 3 of the 9 pats mixed with pure water had developed cracks, but all of those with the admixture were intact.

Efflorescence increases when calcium chloride is added in strong proportions, due, no doubt, to the deliquescence of the chemical and the constant humidity of the cement as thus maintained.

Table 1 illustrates the influence of calcium chloride admixture on the time of setting.

TABLE 1.—*The influence of calcium chloride admixtures on the time of setting of cement*

Cement	Cement mixed with pure water		Cement with about 3 per cent of $CaCl_2$		Cement with about 6 per cent of $CaCl_2$	
	Initial set	Final set	Initial set	Final set	Initial set	Final set
	H. m.	*H. m.*	*H. m.*	*H. m.*	*H. m.*	*H. m.*
Aluminate	4 10	6 40	0 20	1 05	0 12	0 35
Holderbank	3 --	6 45	0 16	1 20	0 04	0 11
Ordinary Portland	3 15	6 45	0 15	2 30	0 04	0 17
Slag	3 15	19 --	3 45	11 30	1 45	6 15

It must be observed in passing that the 6 per cent admixture is not to be recommended for use with the Holderbank and ordinary Portland cement, as the four-minute time of initial setting is too short a period to employ in practice.

Table 2 indicates the resistance to compression in pounds per square inch, and the influence of calcium chloride as an admixture.

TABLE 2.—*The influence of calcium chloride admixtures on the compressive strength of cement*

Cement	Test specimens cured					
	In water at 15° C.				In air at minus 20° C.	
	Mortar mixed with pure water		Mortar mixed with 6 per cent $CaCl_2$		Mortar mixed with 6 per cent $CaCl_2$	
	2 days	7 days	2 days	7 days	2 days	7 days
	Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
Aluminate	3, 300	4, 600	2, 350	2, 370	1, 720	2, 480
Holderbank	950	2, 740	2, 340	2, 510	1, 340	3, 020
Ordinary Portland	336	1, 510	1, 070	1, 770	700	1, 316
Slag	266	826	28	168	28	826

The admixture of 6 per cent calcium chloride was apparently harmful where the temperature of the bath was 15° C., except for ordinary Portland cement. It is of some advantage where exposed to temperatures as low as minus 20° C., for the specimens mixed with pure water developed practically no strength at this temperature.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
*Report of the Chief of the Bureau of Public Roads, 1926. 5c.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94C. TNT as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60M. Federal Legislation Providing for Federal Aid in Highway Construction.
62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.

FARMERS' BULLETINS

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*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *739Y. Federal Aid to Highways, 1917. 5c.
*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-Span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying Out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JANUARY 31,1927

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,985.09	1,298.3	$ 1,834,960.34	$ 889,114.85	101.9	$ 5,519,632.52	$ 2,604,130.30	296.7	$ 141,817.16	$ 70,908.93	13.3	$ 3,606,535.83	Alabama
Arizona	10,949,878.25	5,863,772.35	729.8	521,097.34	352,011.23	35.8	1,343,390.41	919,184.58	75.9	22,957.56	16,607.49		3,522,667.35	Arizona
Arkansas	18,364,544.50	7,666,698.35	1,323.0	1,761,995.60	805,926.30	115.6	3,735,948.91	1,815,344.33	248.5	1,448,975.67	715,201.88	95.2	1,880,529.14	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0	5,242,897.06	2,542,537.22	171.4	9,088,252.84	4,404,837.95	194.2	412,370.87	167,114.97	8.4	4,438,169.56	California
Colorado	13,905,904.64	7,127,288.18	745.0	958,040.35	522,177.14	37.9	5,912,091.75	2,878,868.76	266.8	256,076.31	143,620.34	20.1	3,030,377.58	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	684,812.28	245,719.74	13.7	5,699,574.28	1,580,996.95	70.4				879,063.51	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	1,038,483.07	452,067.18	28.0	591,038.05	241,755.90	17.6	183,700.71	87,799.94	7.1	276,394.38	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	2,120,466.67	1,049,833.65	62.7	9,007,138.20	4,040,758.40	229.1	1,135,928.26	415,082.24	24.7	1,654,368.39	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	4,265,434.44	2,102,653.99	255.7	9,346,042.77	4,573,997.15	406.0	199,260.48	99,680.23	4.4	1,970,592.77	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	1,920,785.49	1,055,775.24	102.9	2,054,615.81	1,291,376.25	162.5	208,283.78	130,082.19	13.0	1,135,473.62	Idaho
Illinois	44,116,611.86	20,619,995.74	1,377.7	2,406,008.82	1,177,396.86	81.9	9,740,785.37	4,668,106.54	347.8	665,850.50	325,680.26	24.1	6,195,447.60	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	3,638,469.22	1,738,093.43	116.9	16,204,004.17	7,590,742.41	456.4				2,630,165.97	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	3,731,392.31	1,720,815.87	233.3	11,897,224.91	5,765,995.30	618.0	3,493,361.51	1,683,798.77	94.5	433,649.96	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	1,762,045.93	792,649.00	142.7	13,306,357.44	5,284,820.81	721.3	2,516,823.08	883,972.21	93.4	1,981,011.73	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	1,759,367.93	692,505.85	78.0	7,574,192.37	3,602,891.90	357.3	472,126.06	229,167.79	25.6	1,614,108.21	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1,056.9	886,239.31	393,861.22	31.8	4,824,231.35	2,307,333.02	220.6	435,627.20	197,677.90	11.2	1,242,103.87	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	1,461,181.52	538,004.58	43.5	2,612,258.85	951,262.17	73.9	87,912.01	43,956.00	5.6	1,419,891.86	Maine
Maryland	10,924,943.10	5,112,991.22	423.3	356,216.42	168,190.63	24.2	1,376,382.60	640,064.42	64.2				638,929.73	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	412,120.06	121,949.75	5.1	5,416,681.35	1,483,354.92	78.1	1,085,305.70	308,090.10	20.7	2,627,170.61	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	1,324,273.80	612,755.75	43.1	14,256,486.78	6,461,572.97	403.9	533,994.00	218,610.00	14.6	3,437,064.98	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,181.9	7,860,229.10	3,430,029.11	461.6	3,514,912.74	775,498.90	212.2	3,214,512.67	357,000.00	159.3	1,563,876.43	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1,129.0	1,168,069.25	581,407.96	74.0	7,985,181.01	3,895,056.87	406.5	196,624.31	98,312.15	17.3	1,446,585.92	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	9,587,678.62	4,204,288.49	284.4	11,598,617.86	4,607,458.42	313.0	2,007,316.24	899,956.78	105.3	1,743,892.46	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	1,294,991.62	857,209.60	82.8	1,575,476.67	1,353,362.30	130.3	856,661.87	452,047.21	90.2	5,980,299.00	Montana
Nebraska	11,533,401.62	5,474,202.52	1,768.3	2,334,199.91	1,152,552.23	258.3	12,690,828.64	6,182,149.79	1,331.4	430,139.87	203,245.97	40.2	3,208,222.49	Nebraska
Nevada	7,558,195.51	5,130,934.59	538.8	2,418,585.71	2,078,456.00	246.9	1,612,460.46	1,393,756.26	210.9				1,140,578.15	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6	520,363.47	241,809.41	17.8	1,000,087.73	453,480.17	27.3				462,377.35	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	5,881,939.07	2,397,022.27	26.0	4,500,662.42	855,223.56	55.0	331,087.93	104,910.00	7.0	946,532.96	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0	36,773.39	21,877.50	0.6	3,248,332.74	2,239,874.76	278.6	282,955.23	236,525.13	9.3	2,321,214.23	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	4,686,404.51	1,678,684.42	107.6	38,720,873.00	10,025,600.20	628.6	4,697,900.00	1,060,797.50	57.6	7,003,372.69	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	5,677,359.00	2,274,655.41	124.7	4,380,616.71	2,025,475.48	135.5	490,853.80	238,440.59	20.1	1,714,652.58	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	3,323,775.06	1,610,686.24	486.2	4,611,036.04	2,517,465.95	676.3	672,701.96	336,351.42	80.1	1,447,246.61	North Dakota
Ohio	47,699,532.90	17,371,787.03	1,364.1	3,298,360.96	1,316,121.47	112.8	12,291,043.88	4,720,402.19	362.8	1,377,279.96	542,093.66	17.9	4,543,600.65	Ohio
Oklahoma	28,247,950.33	13,159,999.15	1,178.9	1,177,814.08	553,488.56	47.0	3,686,908.89	1,511,084.32	201.1	1,624,340.02	558,021.83	90.3	2,029,084.14	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,278,772.59	1,816,407.15	127.8	409,163.92	215,220.95	12.0	1,033,065.19	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1,188.8	838,497.58	216,460.04	14.5	29,574,896.82	8,442,326.00	591.6	2,722,312.77	1,008,481.91	59.3	3,446,516.01	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	274,781.19	113,520.00	7.6	2,024,903.87	531,795.00	35.5	239,058.07	74,175.00	4.9	754,874.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	1,694,832.67	646,425.42	75.3	5,304,063.44	2,299,639.32	186.7	99,046.24	42,921.58	10.3	1,101,202.75	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,181.2	1,331,175.93	664,464.75	222.7	3,422,826.30	1,734,580.84	610.2	220,867.18	89,849.59	23.8	1,294,131.85	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	1,589,779.61	747,492.21	49.1	9,062,838.88	3,936,706.92	258.8	424,027.84	80,000.00	7.7	1,854,573.85	Tennessee
Texas	69,183,673.48	27,440,254.72	4,920.2	5,240,853.93	2,366,551.01	299.9	17,330,477.66	7,811,296.00	741.2	3,317,531.66	1,505,050.36	79.8	5,980,550.91	Texas
Utah	8,253,178.03	5,098,440.68	546.4	825,999.14	615,614.06	79.1	1,666,310.69	1,262,142.56	129.9	734,811.96	563,860.83	42.7	1,125,626.88	Utah
Vermont	4,242,042.64	2,017,699.51	134.5	314,565.23	134,066.49	6.6	1,967,767.73	759,594.56	36.5				722,771.44	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	2,005,236.91	891,387.53	62.5	5,851,503.28	2,452,570.58	158.8	723,986.58	97,955.23	18.6	1,116,586.55	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	300,577.74	116,642.49	8.7	4,382,234.09	2,045,600.00	94.9	140,581.59	52,000.00	7.0	1,280,156.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	951,130.88	432,685.36	26.5	5,634,425.23	2,201,847.66	164.3	1,434,758.96	529,950.92	64.8	840,600.41	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	894,671.96	416,931.79	45.0	8,191,712.72	3,915,680.88	363.0	64,419.42	32,209.71	2.8	4,551,741.89	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	1,370,684.55	875,427.00	150.3	1,977,666.96	1,251,602.63	135.8				1,332,726.32	Wyoming
Hawaii				343,664.15	97,440.00	6.5	1,439,661.16	464,352.64	23.1	346,662.73	98,010.00	6.5	805,975.36	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	106,032,067.23	49,111,077.21	5,134.8	352,033,430.94	146,589,426.94	13,936.8	40,359,973.64	15,214,439.56	1,510.7	107,406,352.71	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 112,824,428.82 Federal aid $ 46,788,763.54 Miles 4,098.0

Nightingale