UNDERDRAIN PRACTICE OF THE CONNECTICUT HIGHWAY DEPARTMENT

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SYNOPSIS

Present underdrain practice in Connecticut is based on a rational study of the factors that affect proper performance. Soil mechanics is a necessary tool in the solution of some of the problems. Five topics are discussed: (1) types of pipe, (2) location of pipe, (3) location of openings, (4) filter materials, (5) installation procedures.

The important factors in considering types of pipe are filtering characteristics, strength and life, cost and availability. Open-joint pipe and perforated pipe with holes up require stone covering over the openings if a fine filter (backfill) is used. Metal pipe is preferred in instable ground because of its strength.

Location of a side-hill intercepting underdrain is in the gutter on the uphill side of the road. In level areas, one underdrain under either gutter (or shoulder) usually suffices. Depth of pipe is governed chiefly by depth of frost and capillary rise of water in soil. Variation of frost heave with depth of water table is being investigated. Some soils have too great a capillary rise to make an underdrain economically justifiable.

If perforated pipe is used and the water table is about level, the holes are placed down. Where the table has a substantial grade and the pipe may intersect "dry" strata which would do harm if wetted, the holes are up. If the pipe also carries surface water, holes are up. A recent proposal to place the holes just below the middle but leave the invert unperforated seems to be the right solution for most cases.

Concrete sand and clean bank-run gravel are best backfill materials for all-purpose use. The latter does not require a stone covering in the cases mentioned above and is usually much cheaper, but its quality may change rapidly in a bank. In a great many installations in glacial tills, ½-in. stone or grits (⅜-in. stone) is fine enough, as the soil in these cases has a substantial percentage of gravel and coarse sand. Larger stone is never used now, as experience and experiments proved it often became clogged. Investigations by soil mechanics engineers have established "piping ratios" for stability against clogging.

After the trench is excavated, the installation is completed as promptly as possible, due to danger of rainfall washing soil into the installation. The top 8-inches is bank-run gravel surfaced with bitumen or sod, for this reason.

The "raisons d'être" of this paper are the timeliness of the subject, due to the anticipated large volume of highway, airport and other construction after the war, the importance of underdrainage and the scarcity of investigation and literature on the various problems connected with underdrains. This scarcity is especially noticeable in the field of highways. The author hopes this paper will help to stimulate investigational work on underdrains, as much is needed, especially on the problems involving soil mechanics. Apparently only a minute fraction of highway underdrain installations in the past have been made after rational engineering studies and analyses of the factors affecting their performance.

The Connecticut Highway Department has done considerable work on underdrains for eight reasons:

(1) The average annual precipitation is 45 in., greater than in most states.

(2) The hilly topography, with abundance of ledge and impervious glacial tills, creates high ground water tables at numerous highway cuts.

(3) The hilly topography means that generally a low discharge point can be found a short distance from the section to be drained.

(4) Frost heaves and frost boils ("Spring

break-up") have inflicted considerable damage to the highways, particularly the older ones.

(5) Our frost-heaving soils are often of rather low capillarity, so that lowering the ground water table by an underdrain often will stop or materially reduce a heave.

(6) The abundant snow and the numerous shady areas caused by high hillsides, trees and brush make icy pavements and shoulders a serious traffic hazard and make sanding and ice-picking costly items.

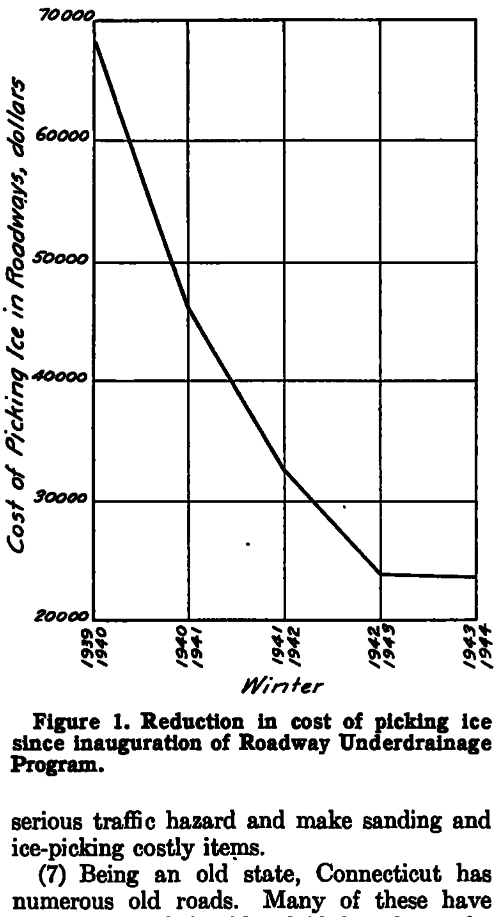

Figure 1. Reduction in cost of picking ice since inauguration of Roadway Underdrainage Program.

(7) Being an old state, Connecticut has numerous old roads. Many of these have pavements and shoulders laid directly on the virgin soil, without benefit of a well-draining non-heaving subbase where needed, or else have a subbase only under the original narrow width of pavement. Where such roads give trouble, it is nearly always better economics to install an underdrain than to remove the pavement and replace with a subbase and a new pavement. (A possible future alternative for such cases is chemical injections into the soil; this is being investigated by a Highway Research Board sub-committee and others.)

(8) Deep open ditches are not feasible because of the deep cuts, the often expensive right-of-way costs and the severe requirements imposed by the State courts against highway accidents.

DEVELOPMENT OF PRESENT PRACTICE

As in many states, Connecticut's underdrain practice began with the use of a telford base, with or without a tile pipe running at a shallow depth under the roadway center-line. Evolution has gradually changed this to the present practice of a pipe at considerable depth beneath one gutter (occasionally one under each gutter) with a clean but comparatively fine granular backfill. Our present practice dates back to the Summer of 1939 when the Associate Highway Maintenance Engineer in charge of roadway drainage spent much of the summer investigating various old underdrains which were not preventing bad conditions. His chief findings were: (a) backfill of ¾-in. stone or larger often became clogged with soil brought into the trench by the seeping underground water, and (b) 3½-ft. was frequently not a sufficient depth of pipe to prevent some water from flowing beneath the underdrain and coming to the pavement some distance downhill or to prevent frost heaving fed by capillary rise.

The following year a program of modern underdrain installation was begun. In that year and the succeeding four years about 33 miles of underdrains on some 300 projects have been laid by the Roadway Maintenance Bureau. This work has brought many great improvements in the roadways thus treated and has resulted in large savings in checking destruction of pavements and in maintenance costs. One saving due to this work is shown by the reduction in ice picking costs, which has fallen steadily from a peak of $68,000 in the winter of 1939–40 to $24,000 last winter (1943–44) (See Figs. 1 and 2). This saving alone would pay for the cost of the 33 miles of underdrain installation in about 15 years. Considerable work has also been done by the Bureau of Roadside Development in installing subsurface drains on slopes where sloughing due to seepage has occurred. Since the war, the Construction Bureau has done little work, but in the extensive plans for post-war work, advantage has been taken of the experience gained by the other bureaus by using modern underdrain design. The Bureau of Materials Engineering has coöperated in the last 2½ years of this work through the services of its soils division. At each proposed subsurface drain, the soils engineer inspects subsurface conditions, samples the soils, tests them in the laboratory, analyzes the test results and reports his recommendations. He uses the following check list in this work; some of the items can be considered briefly in many of the projects.

1. Conditions to be remedied, their location, severity, etc.
   a. Frost heaves.
   b. Frost boils ("Spring break-up").
   c. Icy pavement and shoulders.
2. Topography.
   a. Source of water.
      If source is a swamp, can it be drained?
   b. Gradients of ground water, highway and adjacent land.
   c. Drainage area.
3. Soil at proposed underdrain.
   a. Locations of ground water table, wet and dry seasons.
   b. Soil types and grain size distributions.
      "Frost heaving" types.
   c. Densities of "undisturbed" samples.
   d. Water contents and degrees of saturation.
   e. Height of capillary rise.
   f. Depth of frost.
   g. Filter (backfill) required.
   h. Permeability of soils and backfill.
   i. Extent of subbase (if any) beneath existing pavement and shoulders.
4. Construction.
   a. Approximate center line of pipe.
   b. Depth of lowest perforations, or invert if open-joint pipe.
   c. Gradient of pipe.
   d. Location and elevation of outlet.
   e. Perforations up or down.
   f. Filter (backfill) material available.
   g. Will surface water also be carried by pipe?
   h. Estimated flow through pipe.
   i. Bleeders or special connections to springs.

SOIL TYPES

Connecticut's soils are chiefly of glacial origin. Geologically the two chief types are ground moraines, covering most of the state, and glacio-lacustrine deposits found at various elevations in all but the smallest valleys (1).[1] The former are clayey to sandy tills with boulders. Often the till is found as a single thick stratum of clayey or silty sand-gravel with occasional sand lenses 1 in. or less thick. Sometimes the till consists of strata, 1 to 10 ft. thick, of sand-gravel with varying amounts of silt, lying in slight or radical unconformity on one another. The glacio-lacustrine deposits are usually sand-gravel, sand or sandy silt strata, changing gradually in texture from one stratum to the next and lying horizontally or nearly so. The well-known gray and red varved clays of glacial Lake Connecticut,

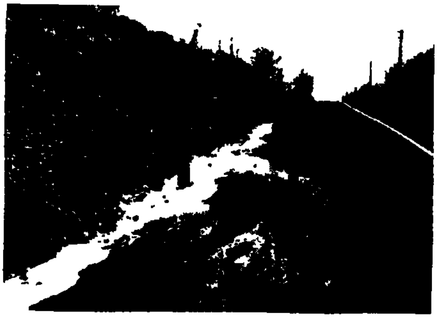

Figure 2. Sidehill seepage just above gutter results in ice on gutter and shoulder.

which at East Hartford extend down at least to elevation −210, are generally covered with a mantle of later glacio-lacustrine sand or recent flood-plain river deposits, so they are not often encountered in underdrain work.

About 75 per cent of our modern underdrains are in moraine deposits on gentle (Fig. 3) or steep hillsides. These installations are generally simply intercepting underdrains placed under the gutter on the side of the road from which comes the seepage. The balance of our installations are in level areas where it is desired to lower a level ground water table. Flow net analyses can be used on the latter, if desired, because each of the strata is usually quite homogeneous and can be well located

[1] Numbers in parentheses refer to the list of references at the end of the paper.

without excessive field explorations. A valuable tool in such cases is the recent work by McClelland on large scale model studies of underdrains (2).

PRESENT PRACTICE

Description of Connecticut's underdrain practice can be divided into five topics: types of pipe, location of pipe, location of pipe perforations, filter (backfill) materials, installation procedures.

Type of Pipe

The important factors in choosing underdrain pipe are: (a) filtering characteristics, (b strength and life, (c) cost and (d) availability.

Figure 3. Seepage out of Gentle Hillside. Note the reference stakes for proposed underdrain.

Factor (a) was investigated for six types of pipe by the U. S. Engineer Department at the Vicksburg Experiment Station (3), using a 36-ft. flume and filter material consisting of fine gravel and coarse sand, all between the $\frac{3}{8}$-in. and the No. 40 sieves. All the pipe was 6-in. diameter and the head of water was the same in each test. Porous concrete pipe (sealed joints) and asphalt-coated corrugated metal pipe with perforations down gave excellent performances—0.03 lb. of filter material per lin. ft. of pipe entered the pipe. The metal pipe with perforations up allowed 0.2 lb. per linear foot to enter. Perforated concrete and clay pipe, with open joints, allowed from 1.3 to 3.7 lb. per lin. ft. to enter. Plain pipe and skip pipe, with open joints, allowed 8 lb. per lin. ft. to enter the pipe. The open joints mean concentration of flow and higher velocities there, hence greater ability to carry this filter material into the pipe. If a filter as fine as the above or finer (like concrete sand) is used, perforated pipe with sealed joints and holes down or porous concrete pipe can be used without protection at the openings. If the former pipe with holes up is used, we place a 2-in. to 3-in. covering of $\frac{1}{2}$-in. stone over the holes before adding the sand. When open-joint pipe is used, we place a "collar" of $\frac{1}{2}$-in. or $\frac{3}{4}$-in. stone around the joints before adding the sand, or else use clean bank-run gravel for the filter. However, where the soil permits use of a filter of $\frac{1}{2}$-in. stone or grits ($\frac{3}{8}$-in. stone), as discussed later, these precautions are unnecessary. Recently an organization manufacturing large quantities of corrugated metal pipe proposed decreasing its pipe perforations to $\frac{5}{16}$ in. and placing them in two or three sets of rows, the sets being on opposite sides of the pipe, but below the center. The result would be an unperforated invert about 100 deg. of arc in width. It is said that tests show that a coarse sand filter will not enter through these perforations in appreciable amounts, and the solid invert will prevent loss of water, during a normal flow, into "dry" strata through which the pipe may pass. If the pipe is placed "upside down," the unperforated invert will be an arc of 225 deg. Another point in regard to openings is that if silt or clay should get through the filter to the pipe, it is less likely to remain deposited on the pipe opening if the pipe wall is thin than if it is thick. If the flow should stop during dry weather, the deposit would dry and perhaps harden permanently; succeeding cycles would add to the deposit and finally plug the opening.

Factor (b)—Strength—is important where settlements or lateral movements are expected. Examples are swampy soil, a "soft" trench whose bottom cannot be well stabilized with coarse material, a slope that tends to slough, and unstable ground near large wheel loads. Movement of pipe may cause opening of the joints or crushing; both are undesirable. Metal pipe with metal couplings is the best in these two respects. Its resistance against corrosion has not been proven, but it is said that the bituminous coating makes it practically the equal of tile and concrete pipe.

Factor (c)—Cost—is a variable item. The higher material cost of metal pipe, compared to tile and concrete, is nearly balanced by its lower labor cost, as it is in 20-ft. lengths. However, the cost of furnishing and laying pipe is only about one sixth of the total installation cost on our projects, so choice of pipe affects the total cost very little.

Factor (d)—Availability of pipe is normally not a factor in Connecticut, as manufacturers are close at hand. During the war a scarcity of metal pipe has developed, temporarily narrowing our choice, but when peace comes, all types will again be available.

Location of Pipe

On existing roadways and slopes, ground water conditions are usually fairly well known and poor performance tells the experienced engineer approximately what he will find beneath the surface. Test pits 5 ft. to 6 ft. deep usually give him the complete picture. On new construction, auger borings, or test pits if the soil is gravelly, suffice if practicable. Most of Connecticut's projects have deep cuts and frequently the soil is "bony." Hence our soil surveys are usually not as complete as those of States with flat topography and our underdrains are only tentatively located until excavation is at or near subgrade, when the foregoing methods are used.

On existing roads, a new underdrain is placed under the gutter to avoid injuring the shoulder and pavement. On new construction, an intercepting underdrain is put under the gutter, but one in a level area may be placed under the shoulders to effect a lower water table beneath the pavement. McClelland's studies (2) are very useful in the latter cases. Occasionally on a dual-lane highway we have underdrains under both outside gutters. When the underdrain also carries surface water, it is beneath the gutter, to avoid long curves into the catch basins. A detail that is important is extension of the underdrain up the road to the farthest point past which ground water seeps, as that water may otherwise seep down the road to plague it at a bad area. This is especially true of an old road with a French drain down its center-line. Another important detail is to have the discharge end of pipe 6 inches above the ground or floor of basin or manhole, to avoid deposition of soil in the pipe, particularly if the pipe carries surface water.

Depth of pipe is extremely important and the objective is simple: to lower sufficiently the water table beneath the roadway. We have found many cases of old drains placed only 3½ ft. below edge of shoulder which were obviously too shallow (Fig. 4). See also references (4), (5), (6). Water seeped beneath the pipe to cause trouble under the roadway and capillary rise caused severe frost heaves. Accordingly our modern installations are always at least 4½ ft. deep, usually 5 or 5½ ft. deep and occasionally 6 ft.

Where the water table is about level, the question of pipe depth depends on how great a degree of capillary saturation can be tolerated within the frost zone and the subgrade, which will be discussed below. Certain soils, such as silts, have capillary rises of 10 ft. or more and an underdrain would have to be too deep to be economically justified. These soils are also so impervious that an underdrain would not have sufficient time between normal

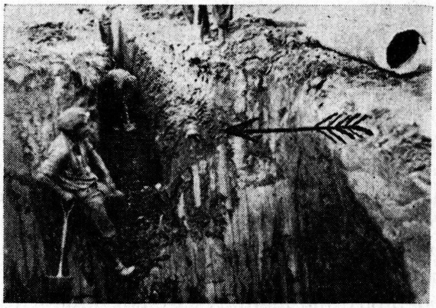

Figure 4. Old shallow underdrain (shown by arrow) being replaced by deeper installation.

rainfalls to lower materially the water table, as shown by McClelland's experiments. The same is true of clays; fortunately clays are so very impervious that they require a long time to suck up water by capillarity to feed their frost lenses. Hence clays do not develop as severe heaves as silts in similar circumstances. In silts and clays on new construction we use 12 inches to 24 inches of sub-base, depending on depth of frost, and place an underdrain or bleeders to remove water which gets into the sub-base. In silts and clays on existing roads without sub-bases, we excavate the pavement and subgrade and install an adequate subbase, and if ground water is plentiful, we also install an underdrain when none exists; if there is a thin subbase, we may merely install an underdrain so that it bleeds water from the inadequate subbase, in the hope that heaving will be greatly reduced. The latter is done when the frost heaving area is extensive, as installing an underdrain is usually much cheaper than removing pavement, shoulders and subgrade over a large area and replacing with subbase and new pavement and shoulders.

Where the ground water table slopes transversely to the centerline of road (usually revealed by the more severe heaves and boils on the "uphill" side of the road), we assume the water table under the pavement will be lowered at least to the level of the pipe, as the latter is placed on the "uphill" side. Again minimum depth of pipe is governed by the degree of capillary saturation we can tolerate. If the soil is an impervious till, which is the situation in 75 per cent of our cases, we watch for pervious strata or lenses of sand which, although very thin, carry most of the flow because their permeability coefficient is 10 to 100 times greater than that of the till. If these are intercepted, draw-down is fairly rapid after installation and prolonged wet spells. Sometimes when a water-bearing pervious stratum is found less than a foot beneath the necessary depth of pipe, the grade is arbitrarily lowered to intercept it, to avoid the possibility of this water flowing downhill below the pavement to a spot where it would be close enough to the pavement to cause trouble. As mentioned previously, our topography and highways are not often flat and our soil strata, especially tills, often have radical changes in direction, dip and thickness over short distances. Hence strata are difficult to trace and we find it advisable to control intercepted water by discharging it where it will do no harm. Underdrains in rock are handled like those in impervious tills with pervious lenses, except the former are not as deep, due to absence of capillary rise in rock and to save expense.

As mentioned previously, depth of pipe at the roadway is determined chiefly by the capillary saturation which can be tolerated under the pavement. Ignoring the often substantial surface infiltration through joints and cracks in pavement and shoulders, soil physics and soil mechanics should enable us to calculate rather closely the required depth, knowing the capillary characteristics of the soil strata and, in the case of frost heaves, the depth of frost. To date science apparently has not progressed far enough in the exploration of unsaturated moisture laws to enable an accurate answer to be made. Most of the research in that direction has been by soil scientists interested in its application to agronomy. The excellent paper by Russell and Spangler (7) summarizes the results of work by themselves and other investigators in studying capillary potential and the energy concept of moisture in soils and their effect on unsaturated soil moisture movements. Of special importance to the highway engineer is their statement on page 447:

> "If at any time the rate of removal of water from an unsaturated soil, by growing plants or by evaporation, or in the case of highway subgrades by the growth of ice plates or lenses typically associated with frost heave, exceeds the ability of that soil to transmit water from regions of saturation, a reduction in moisture content will result. This decrease in moisture content is accompanied by a further reduction of the pressure in the soil water which in turn rapidly reduces the ability of the soil to transmit water. This cycle, sometimes "vicious" and sometimes "benign," results in progressive reduction in moisture content of the soil from which water is being removed."

This probably explains why tight clays having a water table at only 4 ft. to 5 ft. below ground do not heave badly. It also explains why a requirement for substantial frost heaving is almost complete capillary saturation at the start of or during the heaving process. When making soil investigations in summer and fall months for proposed underdrains at frost heaving pavements, the writer has found from undisturbed samples that the frost-heaving soils were from 90 to 100 per cent saturated. Ground water table was from ½ ft. to 3½ ft. below the pavement, excepting that at silts and sandy silts it was often deeper. The high degree of saturation shows fulfillment of the above requirement.

Vertical capillarity tests have been run by the writer using dry soil in 4-in. diameter lucite tubes with a supply of free water maintained in a pan at the base of the tube. Two or 3 per cent of water was usually added when mixing the soil to avoid segregation while placing it. These tests showed that capillary rise in such frost-heaving soils as silty tills is about 6 ft., in clayey tills, about 8 ft. and in pure silts, over 10 ft. The tests had the well known phenomenon that degree of saturation decreases with increasing height above the free water surface, the degree being 50 per cent saturation at roughly half the height of total capillary rise. It seems practicable to consider that in a soil with capillary moisture above a moderately deep water table, the upper portion with less than 50 per cent saturation would not in most cases receive enough surface infiltration to become almost completely saturated for any appreciable time; hence this portion would not fulfill the requirement of saturation given above and it can be considered as practically non-frost-heaving. Hence, for want of better knowledge, the writer uses a rough rule that the ground water table should be lowered to a depth below the pavement equal to depth of frost plus the height of capillary rise at which there is at least 50 per cent saturation. In Connecticut frost averages about 2 ft. deep in the southern half increasing to about 3 ft. at the Massachusetts line, excepting that in the northwest district it reaches 3½ to 4 ft. An underdrain, for example, in the southern half would be laid at least 5 ft. deep if in a soil that has 50 per cent saturation by capillary rise at 3 ft. above water table.

It is obvious that there are many other important factors in determining depth of pipe, such as variation in frost due to exposure, shade, snow removal, etc., amount of heave tolerated, uniformity of heave, infiltration of surface water through cracks and joints, percentage of clay in the soil and variations in soil and water conditions between test pits and between borings. As in most soils problems, a large dose of good judgment is needed to evaluate the variations and less-known factors to arrive at a correct solution of the problem.

Another factor may be the aid furnished by concrete sand backfill to capillary rise in the adjacent soil. To investigate this, three vertical capillarity tests were run by the writer in 6-in. diameter lucite tubes—one had silt alone, a second had silt and concrete sand side-by-side, and the third had silt and ½-in. stone. Rate of rise in the silt was about the same in the first and third tests but was about 40 per cent faster in the second. Heaving of silt in two 5-ft. pits in the yard behind the laboratory is much more rapid in the one having sand beside it than in the one with silt alone. However, until more tests with better control are run, the writer cannot evaluate the magnitude of this aid.

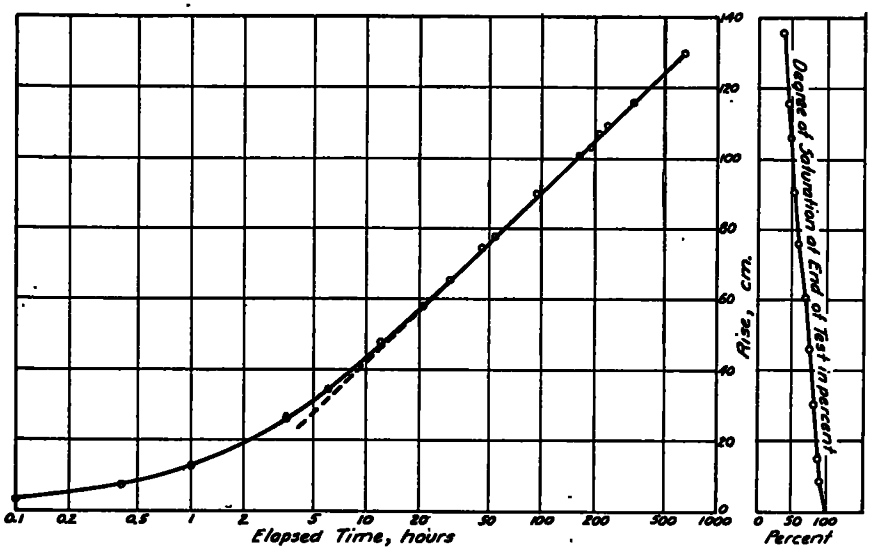

Figure 5. Typical Curve of Capillary Rise for a Silty Sand

Of interest in the various capillarity tests mentioned is the observation that when the soil was a frost-heaving type (Casagrande's definition is a uniform soil and a well-graded soil having 10 and 3 per cent, respectively, finer than 0.02 mm.), the rate of rise, plotted as a semi-logarithmic graph, was always similar in shape to the example in Figure 5. The straight-line portion of the graph was steeper for the finer soils, as would be expected. This straight-line phenomenon should be very use ful in investigating the laws of unsaturated moisture flow and probably it has already been used by investigators; lack of time has not allowed the writer to study much of the literature on this subject as yet. The graphs do not agree with the theoretical formula

$$t = -\frac{nh_c}{k}[\log(1 - z/h_c) + z/h_c]$$

where t = elapsed time
z = rise at time, t
h_c = ultimate capillary rise
n = porosity of soil
k = coefficient of permeability for saturated flow,

for rise in a soil whose pores are considered to be a bundle of capillary tubes, and it seems to

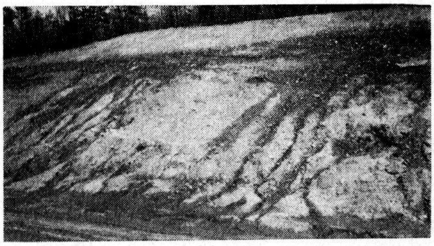

Figure 6. Seepage Line at contact between pervious sandy till and impervious clayey till. Irregular seepage line at irregular pervious lenses. Two subsurface drains now collect 5 g.p.m. each.

be generally conceded that the latter is of little value in the determination of capillary rise in soils.

To conclude this topic, depth of subsurface drains may be discussed briefly. These drains have the sole purpose of intercepting ground water before reaching the surface. Practically all our sloughing slopes are of impervious till with thin pervious strata or lenses. The pipe invert is about 1 ft. below the bottom of the pervious layer if it is clearly defined (Fig. 6). If it is not, the pipe is placed at the head of the sloughing or wet area and about 4 ft. below ground surface, but a foot or two deeper if wet pervious lenses are found at that depth. It is well to keep in mind the possibility of the pervious stratum becoming frozen uphill from the trench (Fig. 7). In this case, seepage often cannot flow through the frozen pervious stratum to the trench and it may break through the former to the ground surface. The proper design is to dig the trench so that it intersects the bottom of the water-bearing pervious stratum at a point below the frost zone.

Location of Perforations

The old debate as to whether pipe perforations should be up or down can be settled, I believe, by analysis of the facts. Where the ground water on both sides of the installation is and will continue to be higher than the pipe, such as in level areas, ground water can flow into but not out of the perforations. Therefore, perforations should be down. Where the ground water table slopes transversely across the pipe line, as in side hill areas, and it is or will be lower than the pipe at a nearby point under the road, the holes should be up,

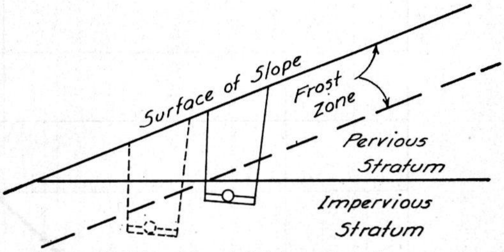

Figure 7. Trench on right can receive seepage from pervious stratum before it reaches frost-zone. Trench on left would be too far downhill.

provided the soil has pervious strata or lenses that could carry ground water to such a point. This is the case in the majority of our installations in Connecticut. But if the soil has no such pervious media, such as a pure clay, holes should be down.

As mentioned previously, a large manufacturer of metal pipe proposes placing two or three rows of holes on each side of the pipe, slightly below center. The resulting solid invert would carry a flow about 1 in. deep in a 6-in. pipe (and more in larger sizes) without spilling water out of the holes. This would eliminate the need for having holes up, unless ground water flow were large or the pipe also carried surface water. In the latter cases, the proposed pipe could be used "upside down," unless the flow were very heavy.

Placing perforations down has the advantage of eliminating the need of stone covering over the holes when a sand filter (backfill) is used. It also requires slightly less depth of excavation, particularly with the larger pipes, as the water table in the trench is governed by the elevation of the lowest perforations.

Filter (Backfill) Materials

There are six criteria in selecting materials to backfill the subsurface drain, as follows:

(a) Fine enough to hold back the soil.
(b) Coarse enough not to enter the pipe perforations or joints.
(c) Good permeability ("free draining").
(d) Consistent quality of material.
(e) Cost of material and handling.

Criterion (a) is probably the most important and it has given us the most trouble in the past. As stated before, one of our two major troubles with old underdrains has been clogging backfill of $\frac{3}{4}$-in. stone or larger with fine sand, silt and clay. (See Figs. 8, 9, 10). A Highway Research Board questionnaire (8) indicates that as recently as four years ago only about one-fourth of the State highway departments used backfill finer than $\frac{3}{4}$-in. stone. Clogging usually occurs slowly, over a period of three to ten years; if it occurs rapidly it is due either to a large amount of seepage or else to failure during construction when rain has washed in surface soil before the backfill was covered at the ground surface. The latter does not necessarily indicate faulty filter design.

In investigating the criterion for stability against clogging, soil mechanics engineers consider that if the finer particles of the filter, at and near the plane of contact with the soil, can hold back the coarser soil particles, the latter will hold back all of the soil. A few fine soil particles near the plane of contact may slip into the filter, but these are of small consequence. Therefore they consider the "15 per cent size" of the filter (15 per cent by weight is finer than this size) and the "85 per cent size" of the soil. The "piping ratio" is the former divided by the latter, or D_{15F}/D_{85S}. Apparently the first to investigate and use correctly designed underdrain filters was Terzaghi in about 1920. At that time he found that if the piping ratio were four or less, the design was safe. Bertram's tests in 1939, using a 2-in. tube and applying no shocks or jars to the tube, indicated Terzaghi's rule to be rather conservative. Three years ago the U. S. Engineer Department issued a directive advising the use of a piping ratio of 5 for a well-graded filter and soil and 4 for a uniform filter and soil; these are based on tests at Vicksburg (3) which I believe were too severe, due to using a layer of soil only $\frac{1}{4}$ in. to $\frac{1}{2}$ in. thick (9).

Figure 8. Two-inch crushed stone backfill entirely clogged with soil at pipe level and nearly so above pipe.

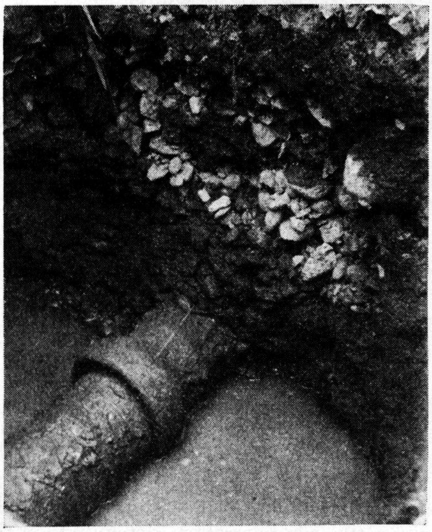

Figure 9. Two-inch to $\frac{1}{2}$-in. screened gravel backfill clogged with soil up to 8 in. above pipe.

This will give too conservative a design. The Providence District, U. S. Engineers, in 1942 published findings (10) from filter tests by its soils laboratory and others; the criterion for stability is reproduced here (Fig. 11). The piping ratio used is D_{15F}/D_{15S} and its value varies greatly with the uniformity of the soil, D_{60S}/D_{10S}. A soil of uniform grain size is inherently less stable than a well-graded soil, hence the Providence District's approach to the problem seems one step nearer to the correct solution. By applying this chart to typical soils it is seen that the Providence allowable piping ratios are a little higher than

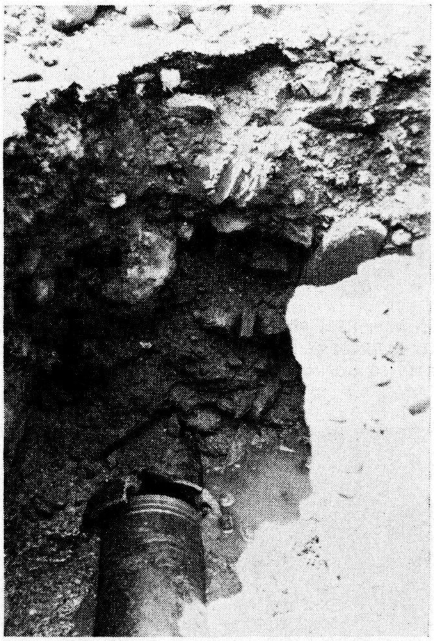

Figure 10. Backfill of broken field stone choked with soil.

4 for uniform soils and considerably higher than 5 for well-graded soils. Our filter tests, run in a 4-in. lucite tube similar to those referred to and in a box 24 in. long by 16 in. wide by 18 in. high (Figs. 12 and 13) check well with the Providence results, so we use the latter in our filter design. All of the above are for cohesionless soils. For clayey soils, piping ratios can be greater, depending on the cohesion of the soil and the judgment of the engineer.

Figures 14 and 15 illustrate typical uniform and well-graded soils which I believe will be barely stable with backfills of ½-in. stone and grits, respectively. Finer soils than these require concrete sand or bank-run gravel backfill. In the case of pure silt which theoretically requires a finer filter than concrete sand, we found that if the stratum is thick, an underdrain would not be economically justifiable, and if it is thin, the other (pervious) strata

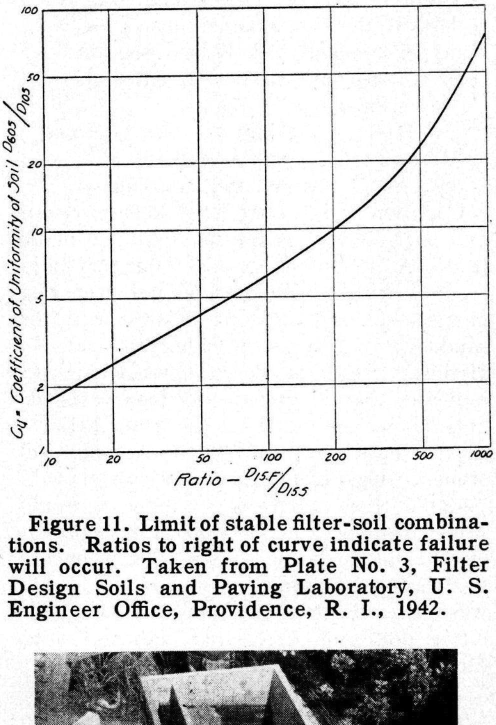

Figure 11. Limit of stable filter-soil combinations. Ratios to right of curve indicate failure will occur. Taken from Plate No. 3, Filter Design Soils and Paving Laboratory, U. S. Engineer Office, Providence, R. I., 1942.

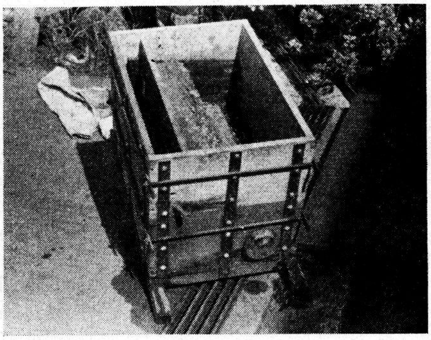

Figure 12. Filter Test Box. Left to Right: 3-in. Forebay for Headwater, 60-mesh Brass Screen Nailed at top to 1-in. board, 4-in. Soil Layer, 9-in. Filter Layer of ½-in. stone (nearly all removed) over 4-in. Pipe with Holes Up.

discharge nearly all the flow. Where there is no flow there can be no clogging.

The consideration of backfill being coarse enough not to enter the pipe was discussed previously under "Types of Pipe." As was seen, concrete sand has the drawback of requiring a stone cover when perforations are up or joints are open.

Good permeability is obviously necessary if all of the seepage entering the trench is to be drawn down to the pipe. All of the materials shown in Figure 16 have permeability coefficients of over 10. $\times 10^{-4}$ cm per sec., which is

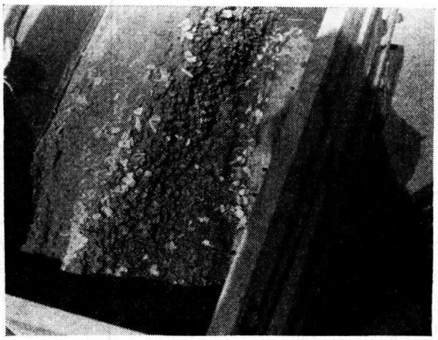

Figure 13. Closer View of Box in Figure 12 showing how soil, a fine sand, washed into ½-in. stone filter and into pipe. Stone near right side of box was removed after test.

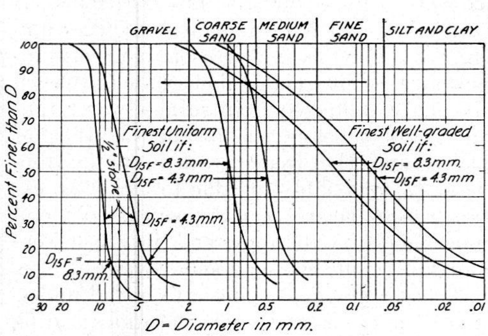

Figure 14. Finest uniform and well-graded soils with which ½-in. stone backfill may be used.

good. However, if "bleeders" without pipe are installed, the design should be calculated from the expected flow, permeability of backfill, hydraulic gradient and cross-section area below the free water surface. A case in mind was a small bleeder in the form of a T. Seepage was about 1.0 G.P.M. into the cross of the T and discharge was through the stem, about 36 in. wide, on a 1:5 gradient. Permeability of the concrete sand backfill was 0.043 cm. per sec. This gave a depth of flow of 30 in. through the sand in the stem, which was undesirable as the bleeder was not deep. The concrete sand was blamed, but actually the fault was with the design; the stem should have been wider or deeper or graded more steeply, or two stems should have been used.

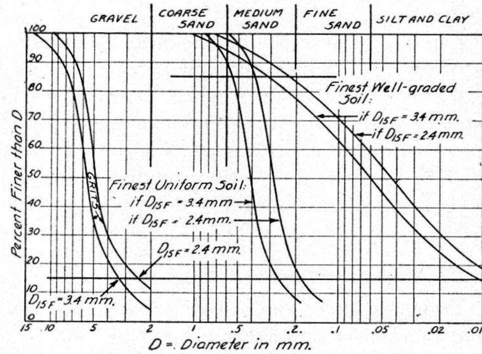

Figure 15. Finest uniform and well-graded soils with which grits may be used.

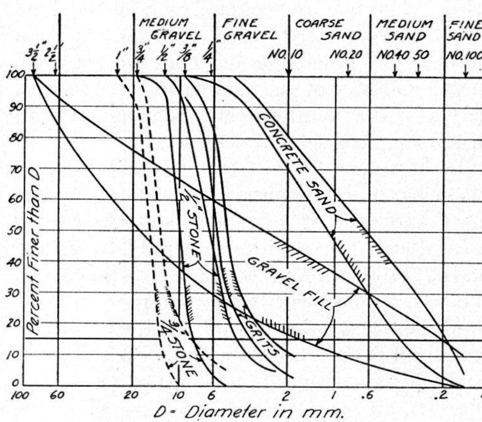

Figure 16. Backfill Materials. ¾-in. stone is no longer used but is shown here for comparison. Gravel fill is a clean bank-run gravel.

Consistent quality of backfill materials offers no problem to us, as they are bought from commercial plants, excepting bank-run gravel. The latter requires stripping overlying topsoil and often other fine strata, which may become mixed with the gravel if the loading is not carefully watched, and often our good gravel strata "thin out" to nothing as digging progresses.

Suitable bank-run gravel costs us from 10¢ to 20¢ per yard in the bank plus about 60¢ for loading, which is considerably lower than concrete sand at 90¢ to $1.10 or ½-in. stone at $1.50 to $1.90 per yard loaded, respectively. However, the quantity required on our average underdrain installation is only about 300 cu. yd., hence getting it from a commercial plant is generally quicker and not much costlier.

Using one material for backfilling a trench is of course cheaper and quicker than using two. We use two only when concrete sand backfill is used and the pipe openings require a stone protection, as described under "Types of Pipe." This protection requires only a small amount of material but is a bother to install.

Installation Procedures

Trenches along the gutter or shoulder are dug by a power shovel, usually with a ⅜ cu. yd. back hoe, which of course enables the machine to work away from the excavated area. Sometimes a crane with clamshell bucket is used. The hoe or bucket digs a 30 to 32 in. wide trench, which is a little wider than necessary. Trenching machines are not used, as the majority of our installations are in hard clayey or silty sand-gravel. Shoring the sides of the trench is seldom necessary.

If the trench floor is soft, it is stabilized by placing granular material, such as the proposed filter material. If the pipe is perforated, with holes down, 3 in. of filter material is then laid on the stabilized floor, the pipe is installed and the trench filled with filter material. If perforations are up, the pipe is laid directly on the stabilized floor and soil from the excavation is backfilled and firmly tamped up to 2 in. below the bottom rows of perforations. Then the trench is backfilled with filter material; however, if the latter is concrete sand, a 1-in. layer of sand is first placed over the soil backfill and a 2-in. protective cover of ½-in. stone is placed over the holes. With larger pipe, as 15-in. diameter, the edges of the stone cover may rest against the trench wall. A laborer with a shovel pulls such stone toward the pipe before placing the concrete sand. If open-joint pipe is used, we generally use either a ½-in. stone or a bank-run gravel filter, depending on the soil, thus obviating the need for special protection at the joints; the procedure is the same as for pipe with perforations down. If concrete sand is used, we place a collar of ½-in. stone around each joint after laying the pipe; since this is a "fussy" detail we rarely use the sand with open-joint pipe.

When the trench floor is soft, it is important to have the pipe openings above the soil by the required amounts, as there is almost certain to be some settlement of the pipe while the trench is being backfilled. Also important is to tamp the backfill while placing it, particularly against the "wet" side of the trench. This is because there is often a slight slow movement of the soil into the filter, about ¼ to ½ in., until the larger soil particles are wedged in the finer voids of the filter. Tamping on the wet side will also prevent small cavities in the filter at the trench wall. Such cavities might result in concentrated flow and movement of the soil at those points. Tamping is difficult to enforce, as it slows down completion of the work, but it is a good precaution nevertheless.

The top 12 in. of our trenches are bank-run gravel, sealed with an impervious material such as grass sod or oil. It has been found by Adams (4) and others that if surface water is allowed to drain directly into the trench, it carries silt and fine sand into the filter and clogs it. For this same reason, it is very important that filter material placed in the trench should be protected from surface water running into it before the installation is completed and the trench sealed at the top. Some of our failures are due to this. An extreme example was a 6-in. perforated metal pipe, capped at its upper end, in which were found clay, silt, fine sand and leaves.

Another practical point is to install underdrains in the less pervious locations during spring or summer, if possible. This will give the drain time to lower the ground water table beneath the road before frost arrives.

No mention has been made of underdrains and weep holes behind walls. Design principles are the same as those discussed in this paper. In the past, ¾-in. stone in bags was used for filter material, but our specifications will probably be changed to concrete sand at least 2 ft. wide or clean bank-run gravel sloping on a 1½ to 1 slope from the underdrain. The latter gives a better design, as seepage forces are more favorable, since they act in the direction of flow. A rather impervious earth fill with a vertical filter next to the wall will result in nearly horizontal seepage through the earth until the filter is reached, when the flow drops vertically. This horizontal seepage creates a horizontal force through the earth and against the wall.

In closing, the writer wishes to express his admiration for the work of Mr. Homer R. Turner, Associate Roadway Maintenance Engineer in charge of drainage for his bureau. Through his vigor and intelligence, Connecticut's modern underdrain practice was successfully launched five years ago and he has continued this excellent work, with the desire to improve designs and methods from experience in the field and research in field and laboratory.

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CAPILLARITY IN SANDS

By Raul Valle-Rodas, *Bolivia, South America*

SYNOPSIS

A summary is given of the findings made during an experimental comparison of active and passive capillarity in sands. The investigation was performed by the author in the Soil Mechanics Laboratory of Princeton University in partial fulfillment of the requirements for the degree of Civil Engineer.

Tests were made with a sand separated by sieving into seven groups of different uniform size. The heights of active capillary rise observed in open tubes are compared graphically to the heights of passive capillary rise determined by means of the negative head capillarimeter as well as to the theoretical values computed by means of conventional formulas. The causes for the appreciable differences in the results as well as their trends are analyzed.

The observed non-uniform distribution of capillary water along the height of active capillary rise is recorded. The observed changes with time in the rate of active capillary rise are indicated. The observed decrease with time of the water content along the height of passive capillary rise and its distribution after flooding and drainage of the test specimens is also presented graphically. The effect of varying admixtures of finer sand particles on the observed height of passive capillary rise is reported.

The importance of the different problems encountered in highway construction and earth structures in general, which are due to the presence of capillary water in soils, prompted