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PERFORMANCE EVALUATION OF LONGITUDINAL PIPE UNDERDRAINS

by

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A Report on the Investigation of Methods to Improve Pipe Underdrain Performance in Illinois

Project IHR-R25
Illinois Cooperative Highway and
Transportation Research Program

A Cooperative Investigation Conducted by the
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Experiment Station
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16. Abstract

This research was conducted under four study phases. Phase 1 involved a full scale laboratory study to evaluate three different underdrain pipe and envelope systems. It was found that an open graded FA-4 envelope material without a geotextile wrapped pipe was a viable design option. In Phase 2 a study was conducted to relate the pipe slot size to the envelope gradation. A testing procedure was developed. A 2 mm pipe slot prevented FA-4 envelope material from passing into the pipe. In Phase 3 it was found that a modified FA-4 material with control of the 0.075 mm aterial could be produced which would have an acceptable saturated hydraulic conductivity value for good subdrainage performance. The study in Phase 4 was inconclusive in terms of selecting geotextiles that would not clog in subdrainage applications. Neither the Gradient Ratio Test nor the Hydraulic Conductivity Ratio Test identified conclusively whether soil-geotextile combinations would clog in the field.

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Introduction

The objective of this research was to evaluate the IDOT longitudinal pipe underdrain design procedure and develop guidelines and recommendations for improved performance and cost savings. The research program was conducted at the Advanced Transportation Research and Engineering Laboratory (ATREL) in the Department of Civil and Environmental Engineering at Urbana-Champaign, Illinois. The research tasks were outlined as follows:

1. Review design and performance of existing IDOT longitudinal pipe underdrains.
2. Evaluate the interaction of present subdrainage hydraulics, materials selection, and system geometrics on performance and cost.
3. Investigate pavement subdrainage systems, materials, and construction procedures that provide for improved performance and cost savings.
4. Conduct a laboratory testing and evaluation program on several promising and cost effective longitudinal pipe underdrain systems.
5. Prepare a final report on the study findings.

Because of the broad scope of this research program, the Technical Review Panel requested that the research effort be concentrated on those areas that were of immediate concern to IDOT District Engineers. These research areas were identified and conducted in four phases as follows:

Phase 1. Full scale laboratory study of three longitudinal drain design alternatives proposed by IDOT and the University of Illinois research team.

Phase 2. A study on the relationship between drainage pipe holes or slots and the aggregate envelope and the amount of fines that migrate into the pipe.

Phase 3. An investigation of the hydraulic properties of IDOT FA4 gradation and its suitability as an envelope material.

Phase 4. A study on the use of geotextiles for soil filtration to prevent clogging of the drain system.

Phase 1. Full Scale Laboratory Study

Three full scale models of the proposed longitudinal pipe underdrains were constructed in the laboratory and tested. The pavement section for the models is shown in Figure 1 and a photograph of the set-up is shown as Figure 2. A typical IDOT highway drainage section was built in a 6 ft wide by 6 ft long by 4 ft deep steel box. A 2 ft deep by 10 in. wide trench was excavated at the pavement edge. The longitudinal pipe drain was placed in the excavated trench with the corresponding envelope materials. The drain was covered with an asphalt shoulder. A 7 in. wide by 14 in. thick steel I-beam was placed on the pavement edge (next to the pipe underdrain) as a loading foot. A 12.5 kip dynamic load was applied to the top of the I-beam using a haversine wave form with a 0.1 s load period followed by a 0.9 s rest period. A water supply trench maintained a constant head of water at the top of the BAM base layer. Pavement deflection and water flow were monitored as functions of load repetitions. Pavement deflection along the edge was measured with LVDTs. Water flow was measured with a beaker and a weight scale.

The three underdrain models were designated as Cases 1, 2 and 3. All Cases had the same basic pavement and trench geometry as shown in Figure 1, but with different trench materials. The pavement layers consisted of 6 in. of PCC, a 4 in. asphalt concrete base layer (BAM), a 12 in. layer of lime stabilized subgrade, and 12 in. of untreated subgrade (AASHTO A-6, 26% sand, 61% silt,13% clay, PI of 5). A 4 in. open graded aggregate layer was located at the bottom to facilitate back saturation of the subgrade. The three underdrain cases were as follows:

Case 1. A 4 in. diameter polyethylene pipe surrounded with a filter sock and an aggregate envelope meeting the IDOT FA1 gradation. The trench was not lined with a geotextile.

Case 2. A 4 in. diameter polyethylene pipe without the filter sock and an aggregate envelope meeting the IDOT CA16 gradation. The trench was lined with a 6 oz/yd^2 nonwoven geotextile with an AOS of 0.212mm (AMOCO 4506).

Case 3. A 4 in. diameter polyethylene pipe without the filter sock and an aggregate envelope meeting the IDOT FA4 gradation. The trench was not lined with a geotextile.

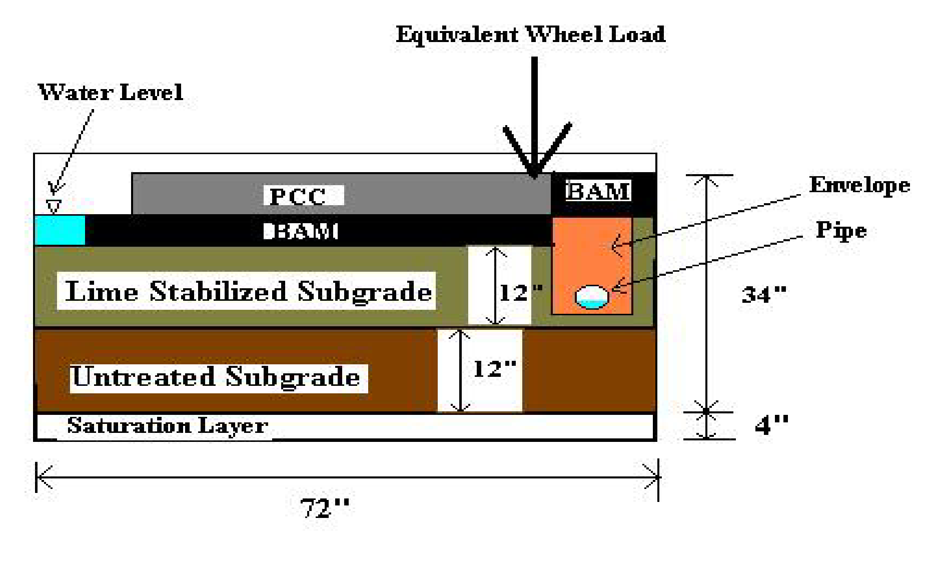

Figure 1. Large Model Test

Figure 2. Pavement Test Set-up at ATREL

Full Scale Model Testing Procedure

Full scale model tests were conducted to meet two criteria. The first criterion was to simulate field conditions as closely as possible. The second criterion was to accelerate testing to achieve results economically in the shortest possible time. The first of these criteria was met by constructing the edge drain with geometry similar to IDOT field design as shown in Figure 1. A 10 in. wide by 24 in. deep trench was placed at the slab edge for the edge drain. The edge drain trench was filled differently for each case as the design variables changed. Each drain was covered with a 6 in. asphalt concrete shoulder to the level of the PCC slab. A constant head water supply trench was placed on the opposite side of the slab and the water level was maintained at the top of the BAM layer by use of a float valve.

Loading at the pavement edge was intended to simulate the effects of a moving 9 kip wheel load on a typical 10 in. thick concrete pavement. This was accomplished by first measuring the modulus of subgrade reaction on the 6 in. slab using a 12 inch circular steel plate. Using Westergaard theory, a corresponding edge deflection was calculated for a 9 kip load on a 10 in. slab. This deflection was then chosen as the desired response of the 6 in. slab to simulate pressures on the subgrade as if it was protected by a thicker slab. The load required to make the slab deflect by this amount with the steel beam was determined experimentally to be 12.5 kips. This load level was held constant for the three case studies.

The second criterion for testing economy and accelerated results was achieved in a number of ways. The PCC slab used for the tests was reused instead of casting a new one for each test. To insure a uniform load to the underlying layers, a sand and cement grout was placed on the top of the BAM layer. The BAM layer was the only layer that was completely reconstructed for each test. The top 2 in. to 3 in. of the lime stabilized subgrade layer was reconstructed. The edge drain was completely reconstructed. As for the testing procedure itself, pavement loading was accelerated by making it continuous at 1 cycle per second. The continuous 0.1 s load followed by a 0.9 s rest period would correspond to a daily traffic load that would rarely occur. Therefore the soil pore pressures were not able to dissipate during testing. The results of these tests can not be considered as indicative of field performance, but only as an indicator of how the different designs perform under the same accelerated loading conditions.

Case 1 Drainage Section

The Case 1 Drainage Section was constructed over a period of 7 weeks. The average dry untreated soil density was 105.4 pcf or 95% of AASHTO T-99. The average dry density of the 2% lime treated soil was 114.6 pcf or 104% of T-99. The bulk density of the BAM was131.9 pcf. The water flow was 5 cm^3/s to 5.6 cm^3/s in the subdrain prior to loading. This changed to 7.3 cm^3/s to 8.1 cm^3/s after the first 10 loads, and 13.3 cm^3/s to 15.4 cm^3/s after the next 2000 loads.

The PCC slab was loaded along the edge using a large steel I-beam to maintain a uniform edge deflection, similar to what a 10 in. slab would deflect with a 9000 lb wheel load. The actual load applied along the PCC (6in. thick by 70 in. edge length) was 12.5 kips. The loads were applied with a 0.1 s period, and a 0.9 s rest interval. LVDTs measured the displacement of the slab at the middle edge (LVDT "C") and 20 in. on either side of the middle (LVDTs "E" and "W"), as well as at the middle rear of the slab away from the loaded edge (LVDT "Back").

The test was discontinued at 21100 cycles when it was observed that a large mass of sand material had pumped from the pavement edge next to the underdrain, to the opposite side of the pavement. The pumping began at approximately 14000 cycles when the rear transducer away from the slab edge began to accumulate a positive deflection (slab was lifting). This corresponds to a change in slope of the resilient deflection. Resilient edge deflections next to the underdrain increased from 0.020 in. at the beginning of the test to 0.040 in. when the pumping began. Permanent displacement also steadily increased during the test, reaching a negative displacement of about 0.08 in. at time of failure.

Case 2 Drainage Section

The Case 2 Drainage Section was constructed over a period of 5 weeks. The top 2 in. of the lime-stabilized soil layer from Case 1 was removed and the top of the lime-stabilized layer was reconstructed. The BAM layer was rebuilt. The BAM was a coarser mix than that used in Case 1, due to the fact that this was what was being produced at the hot-mix plant at the time. The coarser mix was difficult to compact in the laboratory and a Bulk Specific Gravity of 2.12 to 2.22 was obtained (8% to 12% voids). During the pre-saturation of the test section some water movement through the BAM layer was observed. The fabric used to wrap the CA16 was an AMOCO 4506 with a permittivity of 1.5 s^{-1}, a weight of 6-oz/yd^2, and an AOS of 0.212 mm.

The water flow was 4.9 cm^3/s prior to loading . The flow increased to 10.3 cm^3/s after 5000 loads. At 10,000 loads the effluent was muddy and the flow had dropped to 6.1 cm^3/s. At 12,800 loads the effluent was still muddy and the flow had dropped to 1.7

cm^3/s. The flow had practically ceased by 15,000 loads, dropping to 0.3 cm^3/s. Shortly after flow had ceased, at 15,300 loads, pumping near the edge of the slab became obvious. Spurts of muddy water between the BAM and the edge of the steel box occurred with each load cycle, similar to what might occur in the field at a joint during wet weather.

As in Case 1, the PCC slab was loaded along the edge using a large steel I-beam to maintain a uniform edge deflection similar to what a 10 in. slab would deflect with a 9000 lb wheel load. The actual load applied along the PCC (6 in. thick by 7 in. edge length) was 12.5 kips. The loads were applied with a 0.1 s period, and a 0.9 s rest interval. LVDTs measured the displacement of the slab at the center of the loaded edge (LVDT "C") and 20 in. on either side of center (LVDTs "E" and "W"), as well as at the rear of the slab away from the loaded edge (LVDT "Back"). The resilient and permanent displacements during the test are shown in Figure 3.

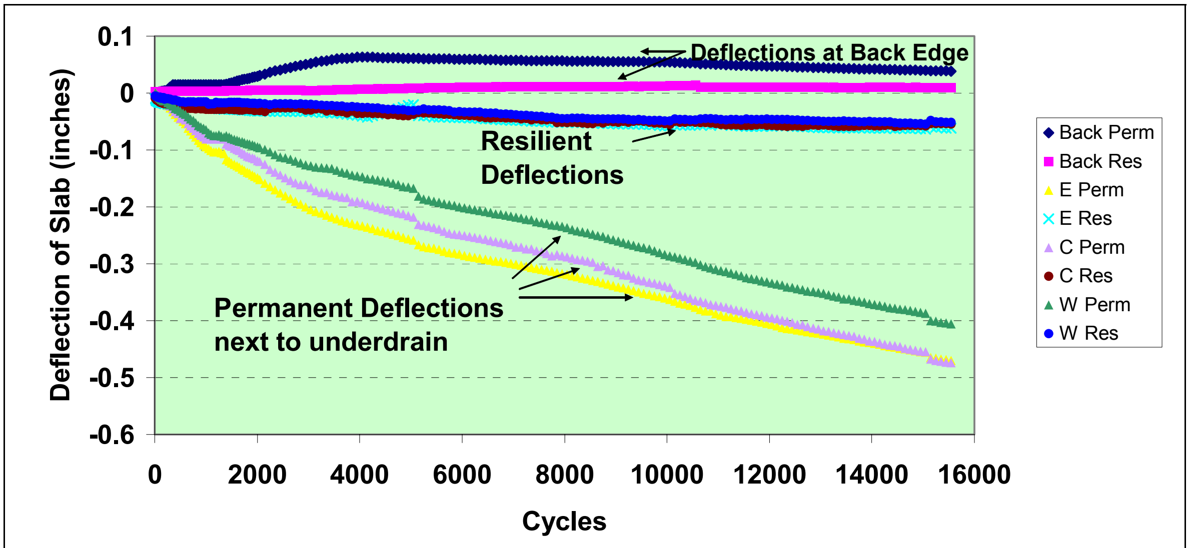

Figure 3. Deflections vs. Load Cycles for Case 2

Permanent displacement in Case 2 increased at the edge at a much higher rate than in Case 1. Whereas it took approximately 15000 load cycles to reach a permanent displacement of 0.1 in. in Case 1, in Case 2 it only required 1650 load cycles. It is not clear whether this was due to material loss or was due to a change in the density of the subgrade. Resilient displacements increased at approximately the same rate for both models. This suggests that resilient deflection is controlled more by the slab thickness and the relative stiffness of the subgrade rather than subgrade erodability.

After reviewing the results of the first two tests it is difficult to come to a conclusion about which one of the two models is preferable. Both tests showed signs of pumping at around 15000 to 17000 load cycles. In Case 1, the FA1 envelope pumped as well as the subgrade. In Case 2, the geotextile clogged with fines from the subgrade and the subgrade pumped. Permittivity tests on the geotextile facing the pavement edge were run after the loading test (see Figure 4). Although the permittivity results are from disturbed samples and are not the same as in-situ measurements, they do indicate that the geotextile clogged, particularly along the top of the trench.

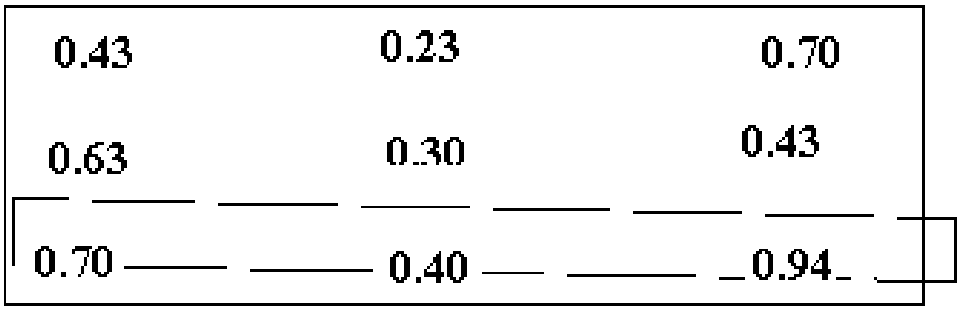

Figure 4. Geotextile Permittivity Results at Points in Trench (values are in s^{-1}).

Case 3 Drainage Section

The Case 3 Drainage Section with FA4 envelope material was constructed during a 2 month period. The top 2 in. of the lime-stabilized soil layer from Case 2 was removed and the top of the lime-stabilized layer was reconstructed. After re-constructing the top of the lime-stabilized layer, it was observed that the reconstructed soil was not bonding to the underlying subgrade. Therefore the loose material was scraped off until the top of the subgrade was uniform. The BAM layer was rebuilt. The BAM was a surface mix similar to that used in Case 1.

The water flow was 3.9 cm^3/s prior to loading. The flow decreased gradually to 3.3 cm^3/s after 20000 loads, but increased to 4.2 cm^3/s at 29600 loads. At 3000 loads the effluent became hazy, indicating the loss of fines. The water became more turbid throughout the loading, and was muddy by the end of the first day of testing. Loading was interrupted at 30,000 loads, but water flow was left on overnight. After 15 hours with no loading, the flow decreased slightly to 3.8 cm^3/s, but effluent was clear. This indicates that subgrade erosion is a load related phenomenon. As soon as loading was resumed, the effluent became muddy. Flow was 4.3 cm^3/s after 35,000 loads. The flow measurements during the test are shown in Figure 5

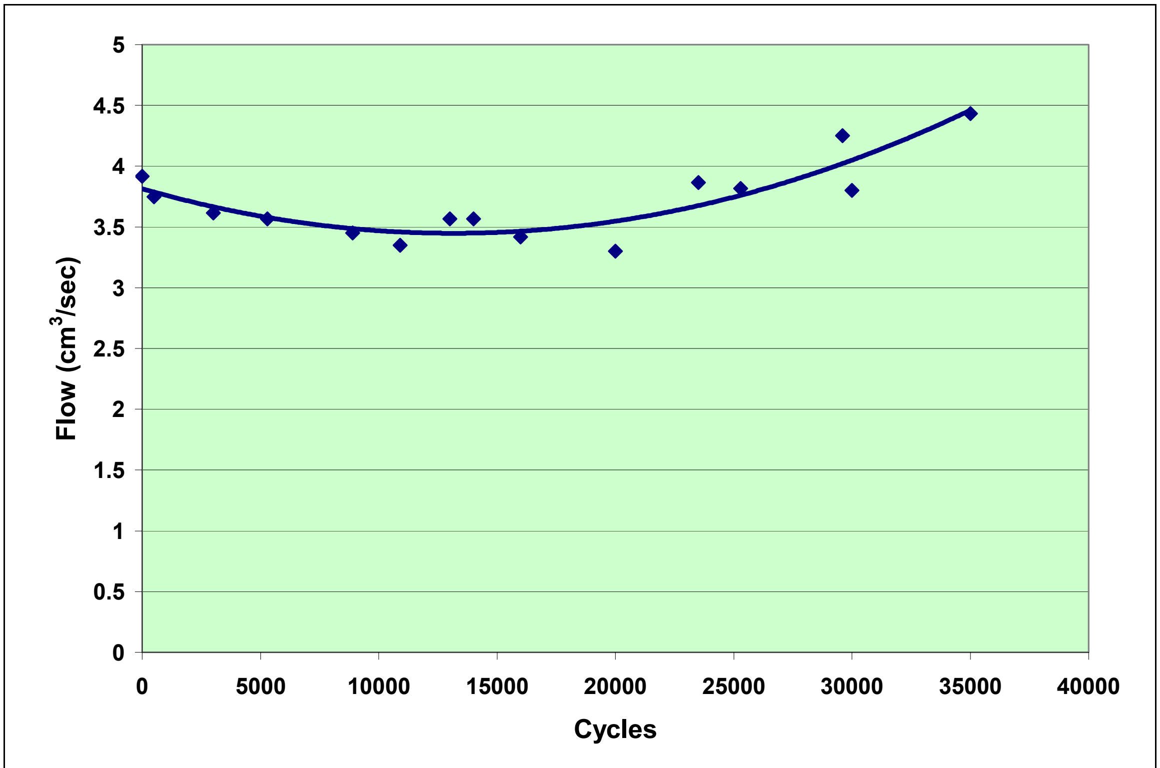

Figure 5. Water Flow vs. Loading Cycles for Case 3

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As in Cases 1 and 2, the PCC slab was loaded along the edge using a large, steel I-beam to maintain a uniform edge deflection, similar to what a 10 in. slab would deflect with a 9000 lb wheel load. The actual load applied along the PCC (6 in. thick x 70 in. edge length) was 12.5 kips. The loads were applied with a 0.1 s period, and a 0.9 s rest interval. LVDTs measured the displacement of the slab at the middle edge (LVDT "C") and 20 in. on either side (LVDTs "E" and "W"), as well as at the rear of the slab away from the loaded edge (LVDT "R"). The permanent displacements during the test are shown in Figure 6.

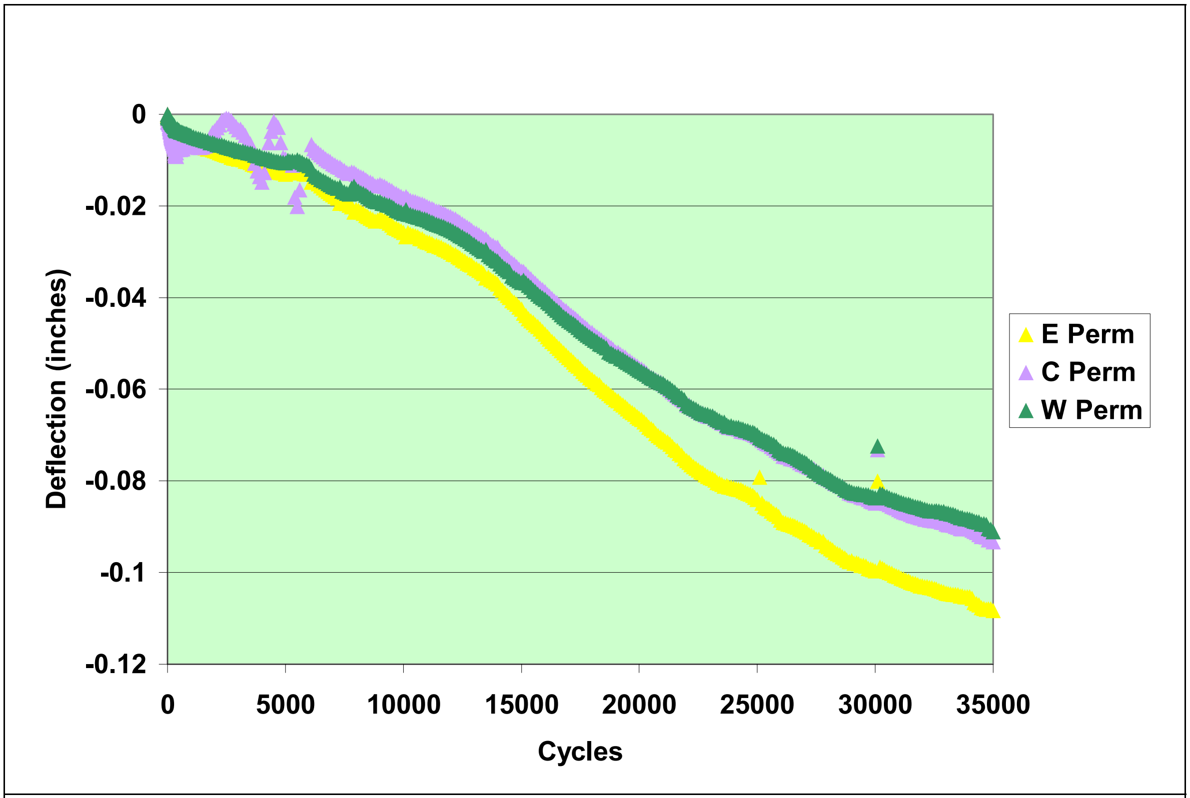

Figure 6. Permanent Deflections vs. Loading Cycles for Case 3

Permanent displacement in Case 3 increased at the edge at a lower rate than in Cases 1 and 2. Whereas it took approximately 15000 load cycles to reach a permanent displacement of 0.1 in. in Case 1, and Case 2 required 1650 load cycles, Case 3 did not reach 0.1 in. until 30000 load cycles. Some of the difference may be due to the fact that this was the last test run and the subgrade may have been consolidated by the previous tests. Also loose material had been scraped off the subgrade surface prior to constructing the BAM layer.

Samples were taken of the FA4 at the pavement interface and washed gradation tests were conducted. The percentages of fines smaller than 0.075mm for the samples at various points in the trench are shown in Figure 7. The original FA4 gradation indicated that less than 0.1% of material finer than 0.075mm should be present. Therefore these percentages represent subgrade fines that have mixed with the envelope material. These percentages do not indicate the amount of fine material that was washed through the envelope and pipe, however.

When the pipe was removed, it was washed out over a #200 sieve and a gradation was run on the remaining material. The gradation of this material is shown in Table 1. The total amount of this sand size material that migrated into the 6 ft. long pipe section was 51 grams.

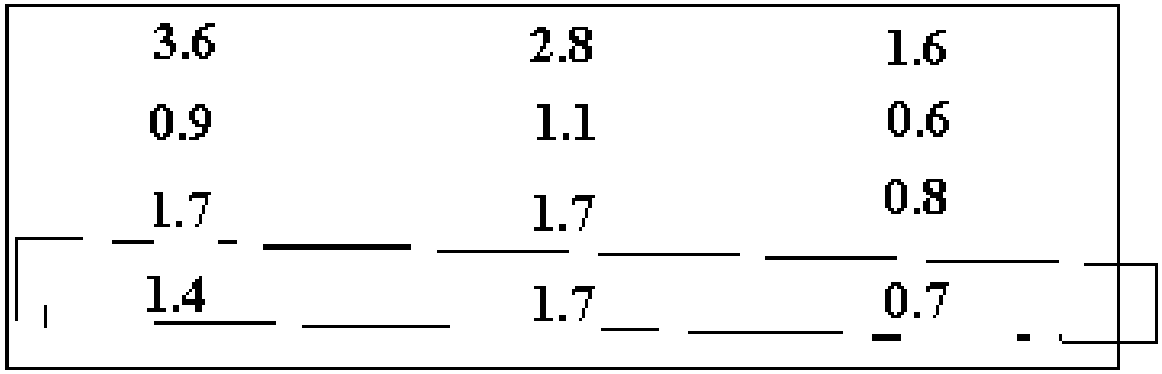

Figure 7. Percentages Finer than 0.075mm present in Envelope for Case 3

Total Sand Wgt.	51.13 gm	
Particle Size (mm)	Weight Retained (gm)	% Finer
4.75	0	100.0%
2.36	2.06	96.0%
2	3.05	94.0%
1.18	11.8	76.9%
0.6	32.37	36.7%
0.425	42.35	17.2%
0.18	48.77	4.6%
0.15	49.31	3.6%

Table 1. Gradation of Sand Residue in Pipe for Case 3

Summary of Laboratory Model Tests

The best performing drainage section in the laboratory study was Case 3 which used the FA4 envelope without a geotextile. As shown in Table 2 the water flow was constant at about 4 cm^3/sec during the 35000 loads and erosion at the pavement edge (indicated by slab tilting) was the least of the three models. In Case 1 with the FA1 envelope, pumping of the sand envelope occurred after about 15000 loads. Case 2, which had a CA16 envelope wrapped with a geotextile liner, indicated clogging or blinding of the geotextile (water flow dropped from 6.1 cm^3/s to 0.3 cm^3/s) leading to pumping of the stabilized subgrade after 15000 loads. There was no indication of problem with using the

CA16 as an envelope; the problem was due to the geotextile used in this test. Water flow versus load cycle for all three cases is given in abbreviated form in Table 2. Slab tilting versus load cycles for the three cases is shown in Figure 8. Figure 8 also indicates less slab tilting for Case 3 during the laboratory testing program.

An extrapolation of the results to what happens in the field requires some engineering judgment. The limitations of the test set-up were the small size of the slab, the loading geometry, load timing, and the hydraulic conditions. The testing box could not accommodate a full-size slab, so that as the subgrade at the edge of the trench eroded, the slab might have tilted more than it would in the field, allowing a rapid build-up of pore pressure. This may have helped cause the FA1 envelope to pump in Case 1. On the other hand, in the field, water would enter the trench not only horizontally between the subgrade/pavement interface but also vertically along the interface between the pavement edge and asphalt shoulder. Earthquake studies of sand liquefaction have indicated that, "Sands most susceptible to liquefaction have coefficients of permeability in the range of 10^{-5} to 10^{-3} m/s." (Soil Mechanics in Engineering Practice, 3rd Edition, Terzaghi, Peck, and Mesri, 1996, p.193).

The FA1 used in Case 1 had a coefficient of permeability (i.e. hydraulic conductivity) of 4.5×10^{-4} m/s. As the fines from soil along the trench began to erode and began to mix with the FA1 envelope, the permeability of the FA1 may have been less than when the test started. Therefore, it can be concluded from the results of the test and the literature on soils liquefaction, that given the right conditions, the FA1 sand can pump, and even if it does not pump, the FA1 does not rapidly dissipate pore water pressures along the edge of the pavement.

Load Cycle	Case 1	Case 2	Case 3
0	5.3	4.9	3.9
10	7.7	-	-
500	-	-	3.8
2000	14.2	-	-
5000	-	10.3	-
5300	-	-	3.6
10000	-	6.1	-
10900	-	-	3.4
12800	-	1.7	-
13000	-	-	3.6
15000	-	0.3	-
16000	-	-	3.4
25300	-	-	3.8
30000	-	-	3.8
35000	-	-	4.4

Table 2. Water Flow vs. Load Cycles for Full Scale Tests (cc/s)

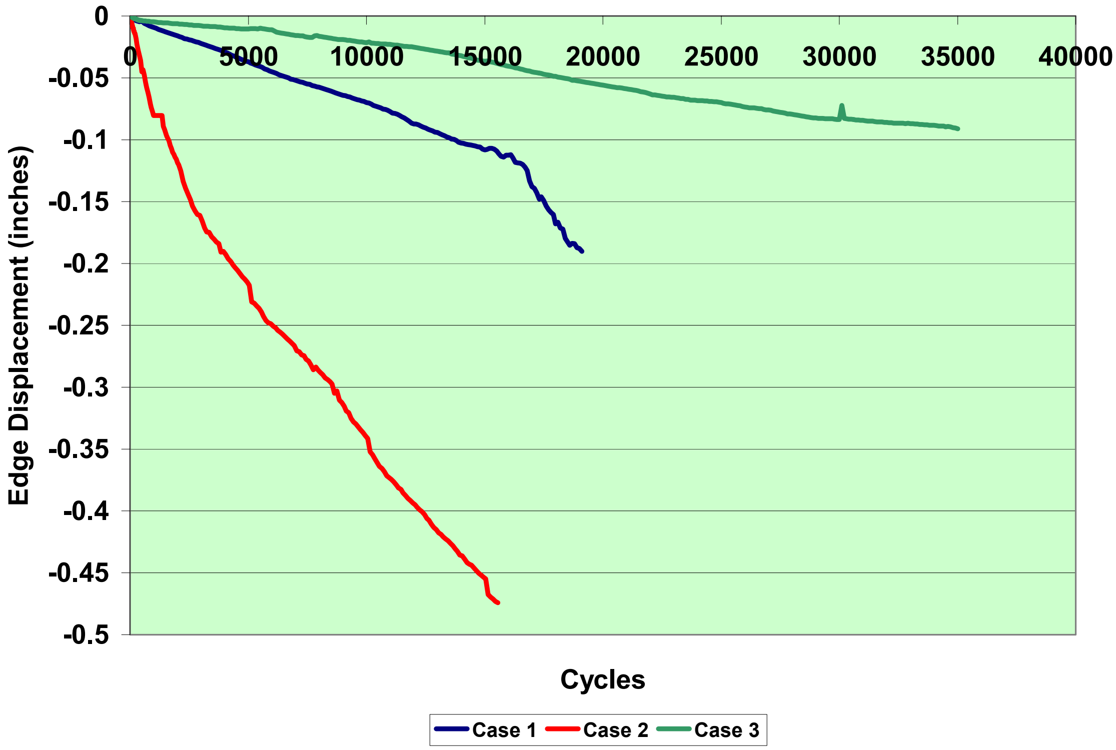

Figure 8. Slab Tilting during Full Scale Tests (Indicates Erosion at Edge)
Case 1: FA1 envelope Case 2: CA16 with Geotextile Wrap Case 3: FA4 envelope

Phase 2 Development of Pipe Slot Infiltration Test

Introduction

Current Illinois practice for installation of pipe drains is to sheathe the pipe with a geotextile sleeve. This practice creates the possibility of fines clogging or blinding the geotextile. IDOT engineers have proposed that this geotextile sleeve be removed. Removal of the geotextile, however, will allow some sand size particles from the envelope material to migrate into the pipe. The University of Illinois was given the task of determining the relationship between pipe slot size, envelope gradation, and the amount of sand entering the pipe. University researchers decided that due to the large number of variables involved the best approach to this problem was to develop an index test for different combinations of envelope materials and pipe slot sizes. The pipe slot test was then used to compare the relative performance of four envelope aggregate gradations with three slot sizes. Therefore rather than specify slot sizes for every conceivable envelope gradation, the purpose of this research was to develop a test that could indicate the stable relationship between pipe slot size and envelope material, and demonstrate how the test could be used to evaluate the four aggregate samples (designated FA1, FA4, FM4, and CA16) provided by IDOT.

Test Apparatus and Method

A Plexiglas frame was constructed to hold a 12 in. section of pipe. A Plexiglas cap with thumb clamps and a swivel garden hose fitting was made to fit the end of the pipe sample. The apparatus was made to fit inside a rotating aggregate washer. A photograph of the Plexiglas frame and a typical pipe sample is shown in Figure 9. A photograph of the test in progress is shown in Figure 10.

Figure 9. Pipe Slot Infiltration Test Frame and Pipe Sample

The test procedure was as follows:

1. Dry the envelope material at ambient temperature.
2. Take a random sample of the envelope material and perform a sieve analysis according to AASHTO T27.
3. Take a second sample of the envelope material large enough to fill a 12 in. section of pipe. The sample size will be approximately 4500 to 5200 grams.
4. Measure the width of ten randomly selected pipe slots with feeler gages, being careful not to force the gages into the slots. Record the average opening size.
5. Clamp the pipe section into the bottom of the test frame and weigh the frame with the pipe.

6. Compact the pipe with envelope material. The material should be compacted in four lifts, using a uniform effort. At the University of Illinois this was done with a 250 gram drop hammer, using 30 blows per lift and a 4 in. drop height.
7. Weigh the apparatus with the envelope material. Calculate the initial weight of envelope material.
8. Fasten the cap to the top of the pipe and tighten the thumb screws. Insert the frame into the rotating aggregate washer and clamp it to the drum. Connect the garden hose to the swivel hose fitting.
9. Turn on water flow. Water flow should be uniform between tests. The University of Illinois researchers set the water flow at 2 gal/min. Start rotating the aggregate washer. Run the washer for 15 minutes.
10. Turn off aggregate washer and water flow.
11. Empty envelope material into a pan. Dry the remaining material in an oven for 15 hours at 105 degrees Celsius. Weigh the oven dry material and calculate the percentage of material lost through the pipe slots.

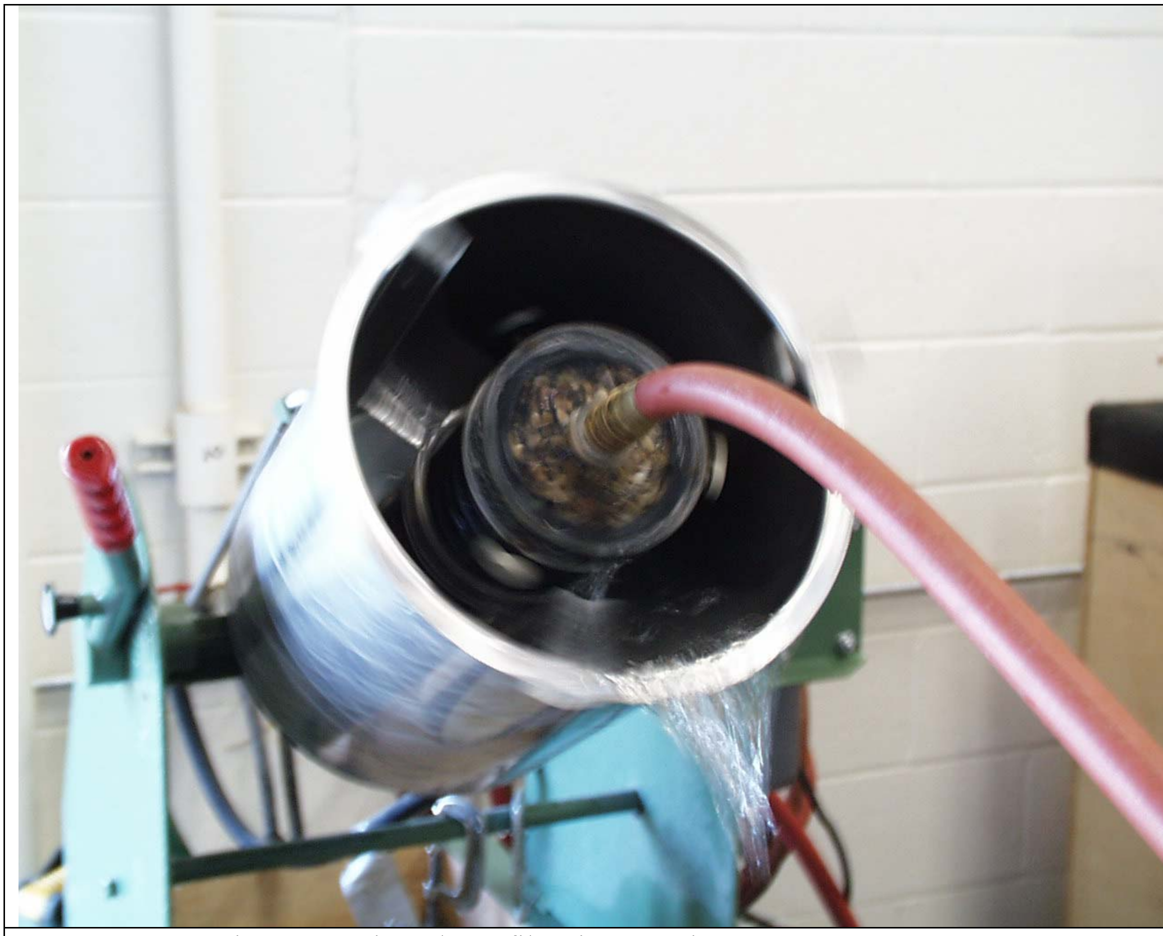

Figure10. Pipe Slot Infiltration Test in Progress at ATREL

<u>Envelope Materials and Pipe Slots Tested</u>

The pipe sections tested were typical black polyethylene corrugated pipe. The pipe comes with slots in two standard sizes of 0.6 mm and 1.6 mm widths. The pipe sections used in the tests had slots that were 0.6 mm, 1.9 mm, and 3.0 mm in width. The larger slot widths were made by enlarging the manufactured slots with a rotary drill.

The four envelope materials tested were provided by IDOT District 4. These materials met IDOT CA16, FA1, FA4, and FM4 gradations. The gradations for the four envelope aggregates are shown in Table 3. All four aggregates were composed of uncrushed natural gravels and sands. The CA16 was a coarse aggregate "pea gravel". The FA1 was a well-graded sand. The FA4 was a coarse sand with few fines. The FM4 was a uniform coarse sand.

Size Passing (mm)	CA16	FA1	FA4	FM4
12.5	100	100	100	100
9.5	98.87	100.00	100.00	100.00
4.75	27.49	100.00	91.94	99.97
2.36	0.68	93.39	22.70	3.08
1.18	0.24	76.80	5.11	0.64
0.6	0.16	60.08	1.67	0.53
0.3	0.06	13.52	0.22	0.27
0.15	0.04	1.39	0.05	0.12
0.075	0.00	0.00	0.00	0.00

Table 3. Envelope Aggregate Gradations Tested for Pipe Infiltration

Test Results

Each combination of envelope material was tested four times with the three slot widths. The only exception was the FA1 envelope with the 3 mm slot width. The FA1 and 3 mm slot width combination was too unstable for a meaningful test (i.e. the aggregate poured out of the slots during filling). A summary of test results is shown in Table 4. The test results were analyzed using a statistical computer program[1]. The statistical analysis provided the means, variance, and indicated whether any statistical difference existed in the test results for the different slot sizes.

Envelope-Slot Size Combination	Average Wgt. Loss (%)	Variance σ^2 (Square of Standard Deviation)	Significant Difference? (95% confidence)
FA1-0.6mm	2.1	0.03	No
FA1-1.9mm	16.8	112.3	No
FA4-0.6mm	1.4	0.04	No
FA4-1.9mm	2.2	0.3	No
FA4-3.0mm	4.8	0.8	**Yes**
FM4-0.6mm	0.8	0.007	No
FM4-1.9mm	0.9	0.04	No
FM4-3.0mm	3.2	1.6	**Yes**
CA16-0.6mm	1.1	0.09	No
CA16-1.9mm	0.7	0.01	No
CA16-3.0mm	1.4	0.06	No

Table 4. Results of Statistical Analysis for Pipe Infiltration

Discussion of Test Results

In general, as the width of the slot increased the envelope material-slot combination became less stable. As long as the slot opening was below some critical width or "stability threshold", the amount of material washed through was more or less the same. Once the stability threshold was exceeded, soil piping occurred and large amounts of material began to wash though the slots. This behavior was clear in tests conducted on the FA4 and FM4 envelope aggregates. Statistically there was no difference between the results for the 0.6 mm and 1.9 mm slot widths. When the slot width was increased to 3.0 mm the amount of fines lost through the slots showed a significant increase.

The tests did not identify a significant pipe slot combination for the CA16 and FA1 envelope aggregates. For all practical purposes, the amount of fines washing through the slots was the same for the CA16 between 0.6 mm and 3.0 mm. This suggests that the threshold limit for slot width is greater than 3.0 mm for the CA16. The statistical analysis for the FA1, however, is somewhat misleading. Although there does not appear to be a significant difference in the results for the 0.6 mm and 1.9 mm slot widths, a comparison between the average weight loss and variance of the two combinations suggests otherwise. The amount of FA1 fines lost through the 0.6 mm slots ranged from 1.9% to 2.2%. The amount of FA1 fines lost through the 1.9 mm slots ranged from 7.7% to 31.4%. The variance of the results for the FA1 for the 0.6 mm slot was only 0.03% but this jumped to 112.3% for the 1.9 mm slot. The results of the 1.9 mm slot combination statistically skewed the results from the 0.6 mm, making the separation of these means impossible. The threshold limit for slot width for the FA1 is apparently somewhere between 0.6 mm and 1.9 m

Summary of Pipe Slot Study

A comparison of the test results with the gradation of the envelope materials does not indicate a correlation between the percent weight loss, the pipe slot size, and the percentage of envelope material finer than that size. This is shown graphically in Figure 11. Lines representing each of the three slot widths have been superimposed over the envelope gradation curves. At the intersection of these lines, the average percentage loss from the testing is indicated. If the amount of material lost were proportional to the slot width and the percent of the gradation passing, then it is hard to explain why the FA1-0.6mm combination lost only 2% through the pipe relative to 61% passing that grain size in the original sample, whereas the FA4-3.0mm combination lost 5% through the pipe relative to approximately 48% passing that grain size in the original gradation. Therefore a variable in slot stability is the overall structure of the aggregate matrix, which is a function of more than one grain size percentage.

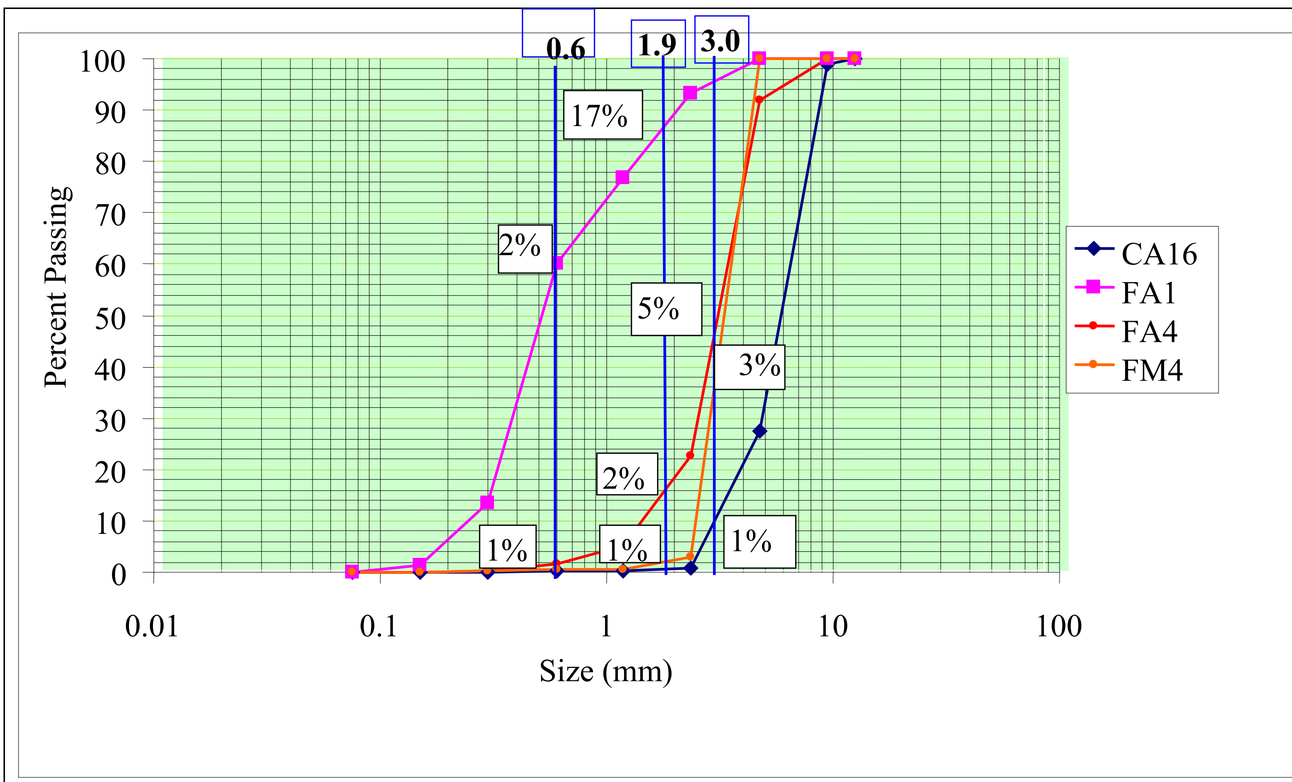

Figure 11. Percentage Losses at Different Slot Sizes Superimposed on Envelope Material Gradations

The test developed by the University of Illinois is a useful empirical test for identifying envelope aggregate vs. pipe slot width combinations that may become unstable in the field. It may be possible to use this test to identify threshold slot widths for typical envelope aggregate gradations. This would require a larger number of samples than have been tested at this time. Only one gradation each was tested that met the IDOT CA16, FA1, FA4 and FM4 specifications. The slot stability of these materials could change with different samples from different suppliers.

Phase 3 Hydraulic Conductivity of FA4 Gradation

Introduction

In Phase 1, full scale models of highway edge drains were built at the Advanced Transportation Research Engineering Laboratory (ATREL, University of Illinois) using the standard FA1 envelope material, and a sample of the fine aggregate from District 4. The pavement slab next to the drain was saturated and subjected to dynamic edge loads. The FA4 gradation sample supplied by District 4 performed well and did not show the same pumping problems as the FA1, nor did it show any signs of clogging over time. The only noticeable problem was that the FA4 envelope did not act as an effective filter for eroded fines from the lime-stabilized subgrade.

The fact that this particular sample of FA4 performed well did not insure that all FA4 samples will have desirable drainage properties. The IDOT FA4 gradation is a very loose specification that controls at only two points. IDOT specifies that an FA4 must have 100 percent passing the 9.5 mm sieve and 0% to10% passing the 1.18 mm sieve. If the IDOT FA4 specification is to be approved for drainage envelopes, then it becomes necessary to know whether a sample that meets this specification has a low saturated hydraulic conductivity. ATREL researchers developed a method to approximate the lower limits of hydraulic conductivity for the IDOT FA4 specification.

Test Procedure

The test procedure for the hydraulic conductivity was a modification of ASTM D 2434 Standard Test Method for Permeability of Granular Soils (Constant Head). The test was conducted in a permeameter specially built at the University of Illinois for testing geocomposites and granular materials (see Figure 12). The University of Illinois permeameter is large enough to test a 30 cm length of granular fill, permitting more accurate measurements of hydraulic gradient and hydraulic conductivity than is possible with small laboratory samples.

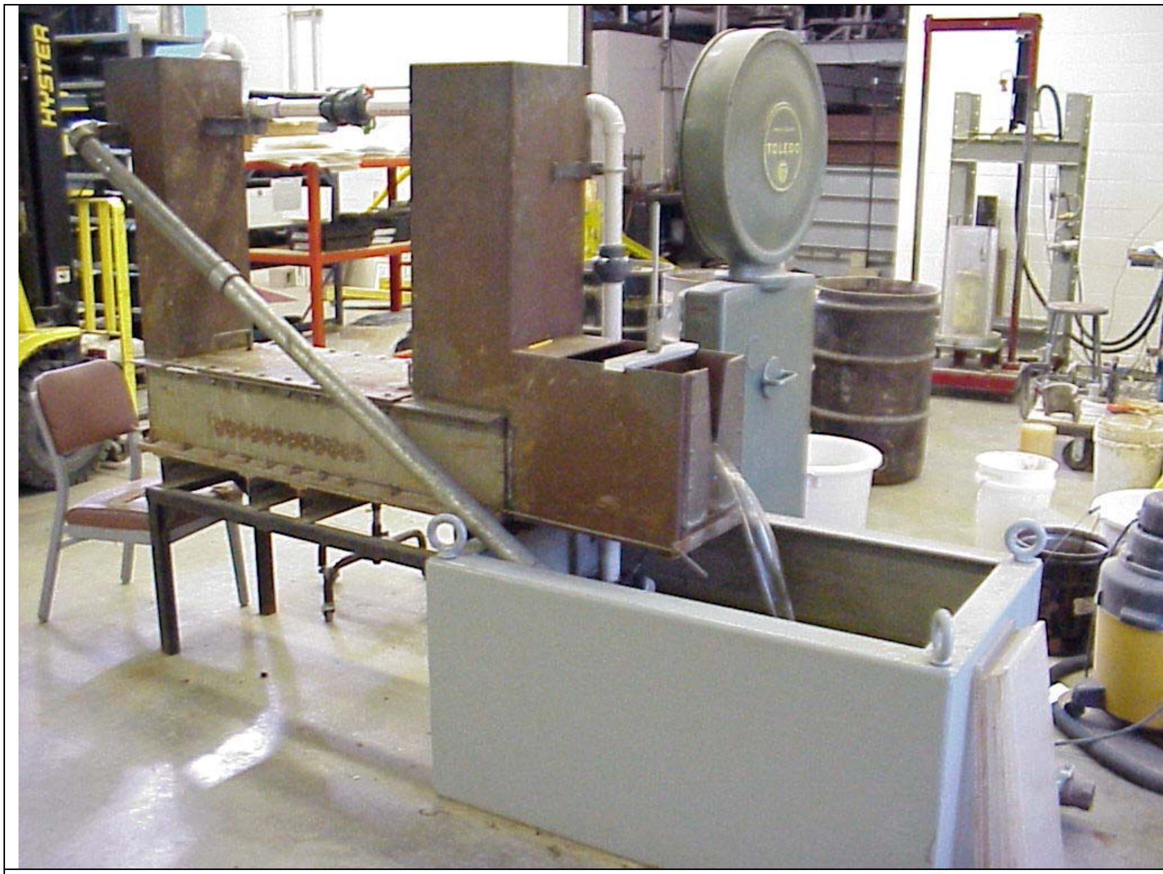

Figure 12. University of Illinois Constant Head Permeameter

Test samples were compacted at optimum moisture content (based upon AASHTO T-99) in three uniform lifts into a removable steel chamber using a vibratory pneumatic hammer. The sample chamber was weighed and a moisture sample was taken to calculate approximate dry density based upon the internal dimensions of the chamber. The chamber was placed into the permeameter and the sample was saturated overnight. Hydraulic gradient was calculated from the difference between the height of the water behind the upstream spill plate and the height of the water behind the downstream spill plate, divided by the length of the aggregate sample. Several measurements were made at different gradients by changing the height of the downstream spill plate.

Materials and Gradations

The original material provided by District 4 was a clean natural sand. Other samples tested in the study were made in the laboratory by separating the original IDOT material into individual sizes and recombining these sizes to arrive at the desired gradation. The three gradations made in the laboratory were designated "FA4-Max" (for maximum density), "FA4-Fine" (for greatest amount of minus 0.075mm material), and "FA4-New" (the possible maximum density for a new specification recommended to IDOT for drainage envelopes).

The gradation designs are based on the fact that the IDOT FA4 specification already limits the effective size. The smallest effective size allowed is 1.18 mm. This left only two variables to examine, porosity and the percent of minus 0.075 mm material. Porosity is inversely proportional to the dry density of a soil or aggregate. By increasing the dry density, the voids through which water can travel are closed or restricted. A gradation that allows the closest packing of particles for a given maximum size can be calculated using the Talbot equation :

$$P_i = (d_i/D_{100})^m \times 100$$

where

P_i is the percent finer than the ith diameter
d_i is the ith diameter
D_{100} is the diameter of the maximum particle size
m is an exponent ranging between 0.33 and 0.50, assumed as 0.5.

The gradation that approximates the maximum amount of material finer than 0.075 mm can be approximated using the Terzaghi filter equation to insure that the fine particles do not wash out of the material. According to Lowe[2], the internal stability of an aggregate drainage filter can be checked by dividing the aggregate gradation into a coarse fraction (filter) and fine fraction (soil), and making sure the filter fraction can retain the soil fraction. In this particular case the grain size used to divide the gradation into filter and soil was 1.18 mm. The gradation of the coarse fraction remained the same as in the Talbot modified gradation, and the fine fraction was modified to maximize fines. The Terzaghi filter equation is

D_{15} (filter)/d_{85}(soil) = 5

where

D_{15} (filter) is the particle size at the 15% passing of the filter material, and
d_{85} (soil) is the particle size at 85% passing of the soil.

The gradation representing a new specification for IDOT FA4 is based upon the recognition that the gradation curves for well draining materials have a steep slope with the fine end of the curve being small or truncated. The recommendation to IDOT was to add a restriction to the FA4 specification that would allow no more than 4% of 0.600 mm material. Such a limitation would have the added advantage of restricting the amount of minus 0.075 mm material. Again the Talbot equation was used to create the densest possible material while staying within specifications.

A comparison of the four sample gradations of FA4 along with the IDOT FA4 gradation specification and recommended modification is shown in Table 5. The gradation curves for all four samples are shown in Figure 13.

Size (mm)	Sieve Size	IDOT FA4 Specification	FA4 (Dist 4)	FA4 "fine"	FA4 "max"	New FA4 Specification	FA4 New
9.5	3/8"	100	100	100	100	100	100
4.75	#4		92	71	71		71
2.36	#8		23	50	50		50
1.18	#16	0 -10	5	10	10	0 - 10	10
0.6	#30		2	9	7	0 - 4	4
0.3	#50		0	7	5		3
0.15	#100		0	6	4		2
0.075	#200		0	5	3		1

Table 5. FA4 Specifications and Gradations Tested for Conductivity

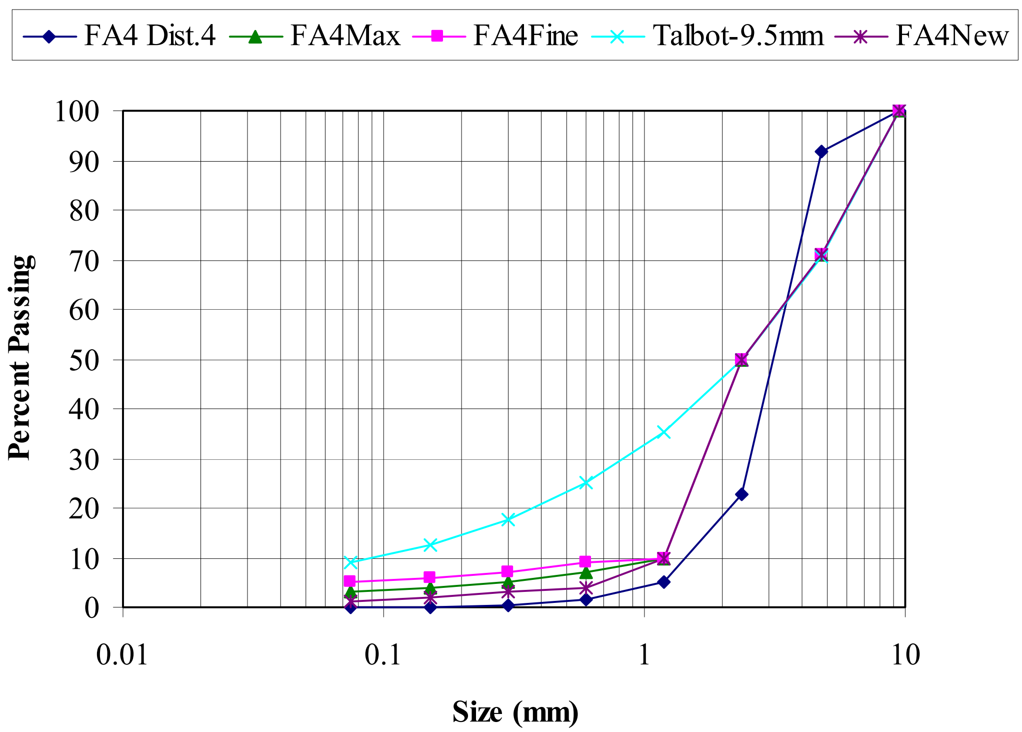

Figure 13. Gradation Curves for FA4 Hydraulic ConductivityTest Samples

<u>Saturated Hydraulic Conductivity Test Results</u>

The dry density of the four test gradations and estimated values for permeability are shown in Table 6. The results of the hydraulic conductivity tests are shown in Figure 14. The hydraulic conductivity of the FA4 from IDOT District 4 ranged between 1530 m/day to 1900 m/day. The hydraulic conductivity of the FA4 designed for maximum density ranged between 75 m/day to 135 m/day. The hydraulic conductivity of the FA4 with a maximum amount of stable non-plastic fines ranged between 120 m/day to 195 m/day. The hydraulic conductivity of the FA4 based upon a new specification ranged between 360 m/day and 490 m/day. Therefore the hydraulic conductivity tests showed that existing IDOT FA4 gradation specifications permit material suppliers to provide IDOT with fine aggregates with widely varying hydraulic conductivity coefficients.

Gradation	FA4Dist4	FA4Max	FA4Fine	FA4New
T99 max dry density (kN/m^3)	17.0	19.0	19.8	18.9
T99 w optimum (%)	7.0	8.0	8.0	7.5
Test dry density (kN/m^3)	17.6	19.6	19.8	19.2
n	0.33	0.24	0.24	0.26
D_{10} (mm)	1.51	1.18	1.18	1.18
P_{200} (%)	0	3	5	1
Calculated k (m/day) [Moulton]	infinity	10	7	35
Calculated k (m/day) (Hazen)	1970	1205	1205	1205
Lowest k result (m/day)	1530	75	120	360
Highest k result (m/day)	1900	135	195	490

Table 6. FA4 Index Properties and Comparison of Calculated Conductivity

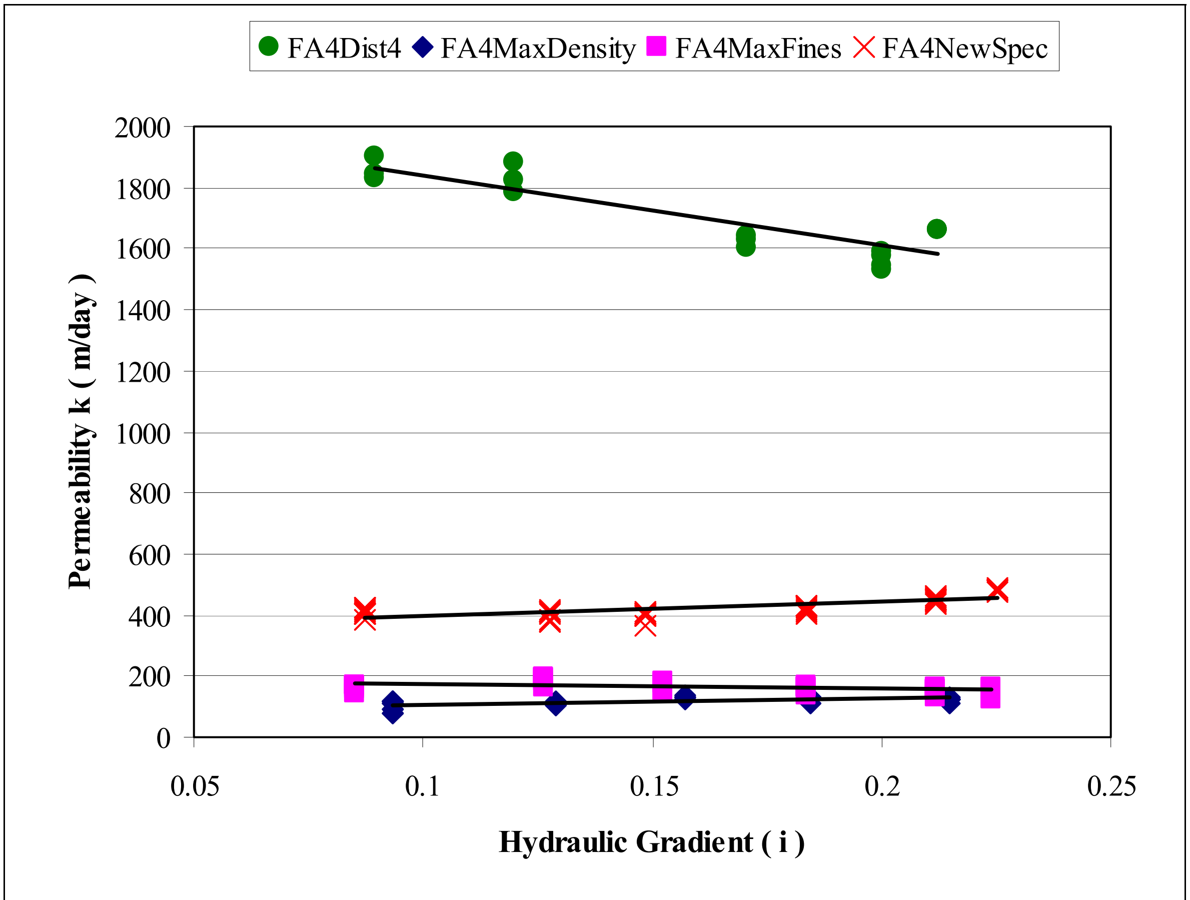

Figure 14. Permeability vs. Gradient for Tested FA4 Gradations

Comparison of Results with Hazen and Moulton Formulas

There are two equations that have been used for estimating the permeability coefficient of granular materials, the Hazen equation and the Moulton[3] equation. Neither equation gave good results for the FA4 Gradation. This reflects the fact that both equations were empirically derived from correlations of biased test data. The Hazen equation was derived from the testing of clean, cohesionless sands. The Moulton equation was derived from multiple materials that included some with high percentages of minus 0.075 mm particle sizes.

The Hazen equation is:

$$k = 2835\ D_{10}^{2}$$

where k is the hydraulic coefficient in feet/day
D_{10} is the particle size in mm at 10% passing.

The Moulton equation is:

$$k = \frac{6.214 \times 10^5 D_{10}^{1.478} n^{6.654}}{P_{200}^{0.597}}$$

where: k is hydraulic coefficient in feet/day
D_{10} is the particle size at 10% passing
n is the porosity approximately equal to $1 - \gamma_d/(62.4 \times G)$
γ_d is the dry unit weight in pcf
G is the specific gravity of the solids (2.70 approx.)
P_{200} is the percent passing the No. 200 sieve

A comparison between material properties, calculated hydraulic conductivity coefficients, and test results is shown in Table 6. The best correlation between a calculated value and a test result was for the FA4-Dist4. The Hazen equation predicted a value of 1970 m/day and the highest test result was 1900 m/day. The Hazen equation overestimated the hydraulic coefficient, and the Moulton equation underestimated the hydraulic coefficient for the other samples tested.

Summary and Conclusions of Phase 3

The test results indicated that is possible to have a poorly draining material that will meet IDOT FA4 gradation specifications. Earthquake studies have indicated that sands and soils most susceptible to liquefaction have hydraulic conductivities in the range of 10^{-5} to 10^{-3} m/s, or 8.5 to 85 m/day.[4] The laboratory gradation was based on the Talbot equation to achieve the densest packing of particles ("FA4Max") produced a hydraulic coefficient as low as 75 m/day. The gradation based upon Lowe's application of the Terzaghi filter equation produced a hydraulic coefficient as low as 120 m/day. The two results suggest that there may be other possible gradations that meet the IDOT FA4 specification that have even lower hydraulic conductivity coefficients. Therefore to insure a margin of safety, a modification of the IDOT FA4 specification would be justified.

A modification of the gradation specification that would give a margin of safety was subjected to a trial similar to the one for the original specification. The trial sample performed well, giving a low result of 360 m/day. This would provide a margin of safety even if the trial sample did not represent the lowest possible hydraulic conductivity for that gradation specification.

Laboratory tests did not correlate well with the two most commonly used equations for predicting hydraulic conductivity from gradation. This was not surprising since many other studies have indicated similar results. The Hazen equation tends to over-estimate hydraulic conductivity, and the Moulton equation tends to underestimate hydraulic conductivity. The best correlation between prediction and test result was for the original FA4 from IDOT District 4. The reason for this correlation is because this sample is similar to the type of sands that Hazen tested to derive his empirical formula.

The approach adopted for testing the lower limits of hydraulic conductivity of an aggregate gradation specification was shown to be useful. It is less costly and more efficient to prepare approximate worst case gradations in the laboratory, than to sample all possible materials from all possible suppliers. If gradation specifications do not provide an adequate margin of safety against poor drainage, specifications can be modified and retested using the same approach. Envelope materials should be selected based on laboratory tests and not on grain size. If laboratory hydraulic conductivity testing is not possible, then tighter gradation limits (that have been previously verified to produce acceptable hydraulic results) should be used.

Phase 4 Geotextile Clogging Research

Introduction

The University of Illinois was asked to study IDOT's current geotextile specifications for highway underdrains and investigate whether a laboratory test can be used to identify geotextile-soil combinations likely to clog or blind. District 4 of IDOT provided ten soil samples with varying silt content from the west central region of Illinois. University Researchers obtained four different geotextiles and attempted to identify clogging potential with two different tests, ASTM D5101 "Gradient Ratio" and ASTM D5567 "Hydraulic Conductivity Ratio".

Gradient Ratio Testing

The Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ration (ASTM D5101) is performed by measuring the hydraulic gradient within a soil layer and comparing it to the hydraulic gradient across the soil-geotextile interface. The method assumes that as the textile clogs the hydraulic gradient at the interface should increase relative to the soil gradient which should remain constant. The apparatus for the test consists of a stiff wall permeameter with piezometers connected to fittings at several locations relative to the soil and geotextile.

During the testing, University of Illinois researchers observed that flow channels were forming along the wall of the permeameter within the soil layer. This raised questions about the reliability of the piezometer readings and the validity of the results. Upon further investigation and consultation with Dr. Robert Koerner of Drexel University, University of Illinois researchers concluded that the Gradient Ratio test is not suitable for testing fine grained soils due to the tendency of these soils to pipe at the wall of a rigid permeameter. Dr. Koerner recommended ASTM D5567 for testing textiles with fine grained soils.

Introduction to Hydraulic Conductivity Ratio Testing

The Standard Test Method for Hydraulic Conductivity Ratio Testing of Soil/Geotextile Systems (ASTM D5567) is used to determine clogging potential of soil-geotextile combinations. The test is similar to the Gradient Ratio test with the exception that the apparatus uses a flexible wall permeameter and uses the change in hydraulic conductivity of the system to determine clogging potential.

Materials Tested

The Illinois Department of Transportation, District 4 provided eleven soil samples to test. The soils were numbered 1 through 11 and consisted of varying percentages of sand, silt and clay (see Table 7). Samples 1 and 2 were the same soil, so these two samples were combined and designated as Soil #1. Three samples, #1, #7 and #10 were chosen to represent the soils covering the broadest range of percentages of clay, silt and sand. Table 7 shows the properties of each sample.

Soil	1	2	3	4	5	6	7	8	9	10	11
%Sand	26.3	26.3	6.3	10	1	1	1.7	1.7	2.1	1	1.7
%Silt	55.5	55.5	84.4	81.8	89	88.1	88	73.1	77.8	74.7	75.8
%Clay	18.2	18.2	9.1	8.1	10	10.8	8.6	25.2	20.1	24.3	22.5
LL	0	0	27	23	22	21	24	34	36	40	30
PL	0	0	25	21	19	19	24	25	28	30	23
PI	0	0	2	2	3	2	0	9	8	10	7

Table 7. Soil Properties of Samples for Geotextile Clogging Tests
(Test samples used are highlighted)

Geotextiles used in the testing are currently used for highway drains although they do not necessarily meet IDOT specifications. The textiles were provided by BP-Fabrics and Fibers (formerly AMOCO). Two non-woven and two woven geotextiles were tested. The properties of the geotextiles are indicated in Table 8.

Geotextile	**Visual Description**	**Polymer Type**	**Manufacturing Process**	**Apparent Opening Size (AOS) (mm)**	**Permittivity (sec -1)**
4553	Black, Calendared One Side	Polypropylene	Non-woven, needle punched fabric	100 sieve, (0.15)	1.5
4551	Black, Calendared One Side	Polypropylene	Non-woven, needle punched fabric	70 sieve, (0.212)	1.5
2019	Black	Polypropylene	Woven Fabric	70 sieve, (0.212)	0.28
2016	Black	Polypropylene	Woven Fabric	40 sieve, (0.425)	0.7

Table 8. Geotextile Properties for Clogging Tests

Experimental Design and Test Procedure

The testing plan was set up as a 4x3 experiment. Four geotextiles were tested according to ASTM 5567 (the Hydraulic Conductivity Ratio) with each of the three different soils. A test frame was built according to test specifications that consisted of three flexible wall permeameters, a pressurized container, water reservoirs, and manometers (see Figure 15). Soil layers were compacted at optimum moisture content (AASHTO T99) to a depth of 2 in. The diameter of the soil samples and geotextile layers was 6 in. The hydraulic gradient used for the tests depended on the soil type and the guidelines set forth in ASTM 5567. In general, the gradients used were high since University of Illinois researchers felt that this would best simulate the high soil pore water pressures that develop under dynamic loading conditions at the edge of highway pavements, as well as accelerate the development of any textile clogging.

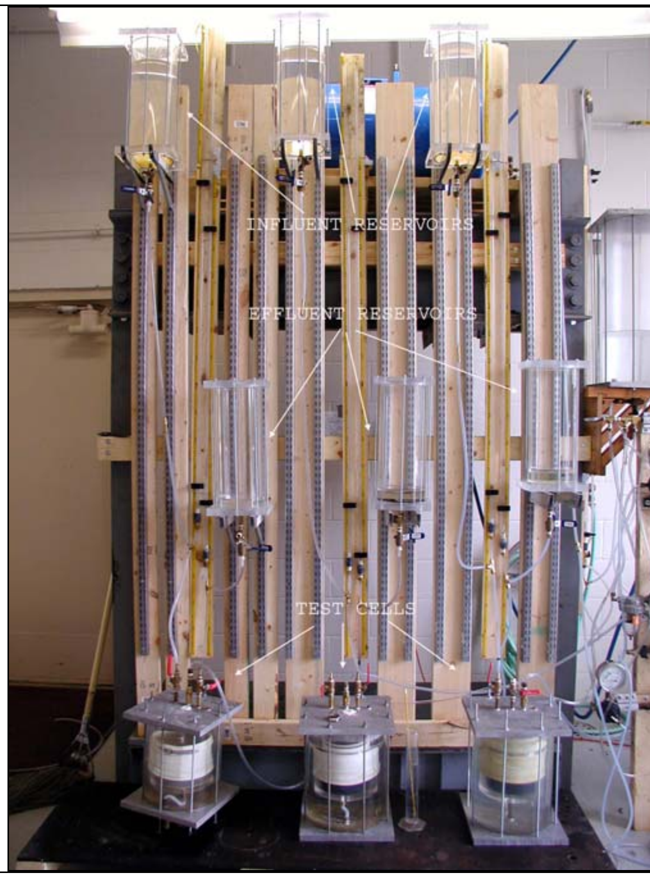

Figure 15. HCR Testing Frame at ATREL

Test Results

Most of the combinations did show a drop in hydraulic conductivity for the duration of the test (see Figures 16, 17, and 18 for typical plots of hydraulic conductivity vs. time). Figure 16 shows the hydraulic conductivity test results for the [Soil 1-Geotextile 2019] combination over a period of month. The hydraulic conductivity dropped from 7×10^{-8} cm/s to 1×10^{-8} cm/s. Figure 17 shows the hydraulic conductivity test results for the [Soil 7 – Geotextile 2016] combination over 16 days. The hydraulic conductivity dropped from 2 x 10^{-5} cm/s to 1 x 10^{-5} cm/s. Figure 18 shows the hydraulic conductivity test results for the [Soil 10 – Geotextile 4554] combination over 10 days. The conductivity dropped from 3 x 10^{-5} cm/s to 3 x 10^{-7} cm/s. The beginning and ending hydraulic conductivity test results for all combinations is presented in Table 9.

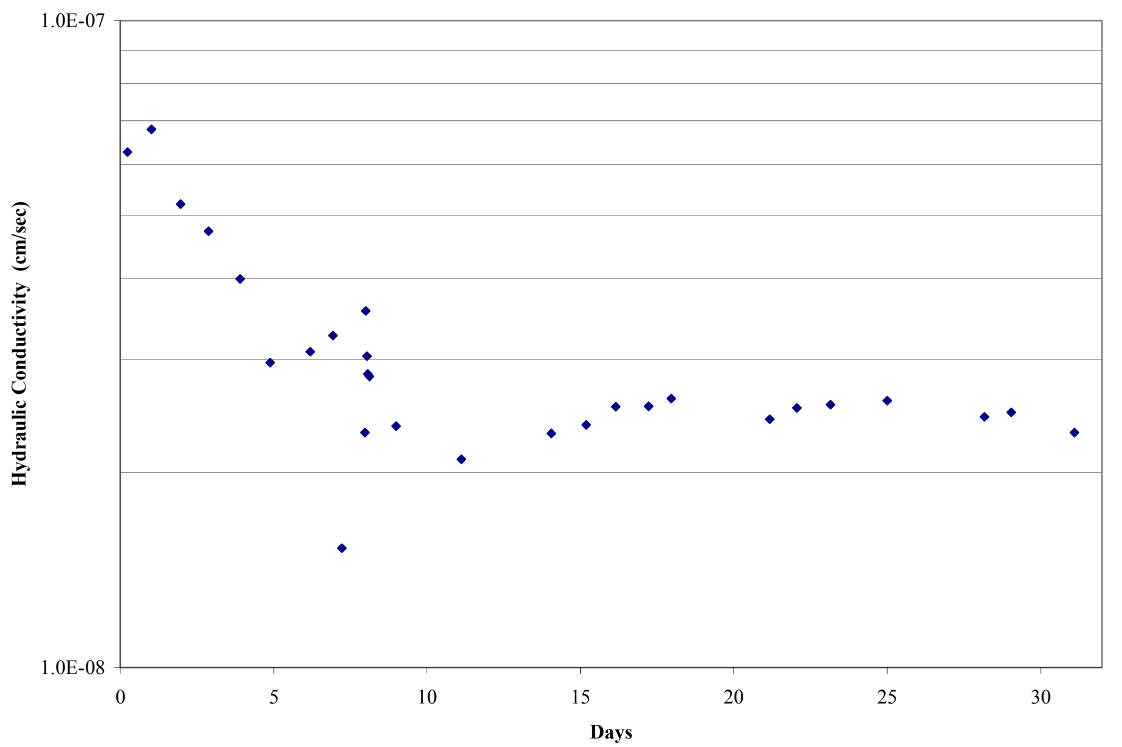

Figure 16. Hydraulic Conductivity as a Function of Time
[Soil 1-Geotextile 2019] Combination

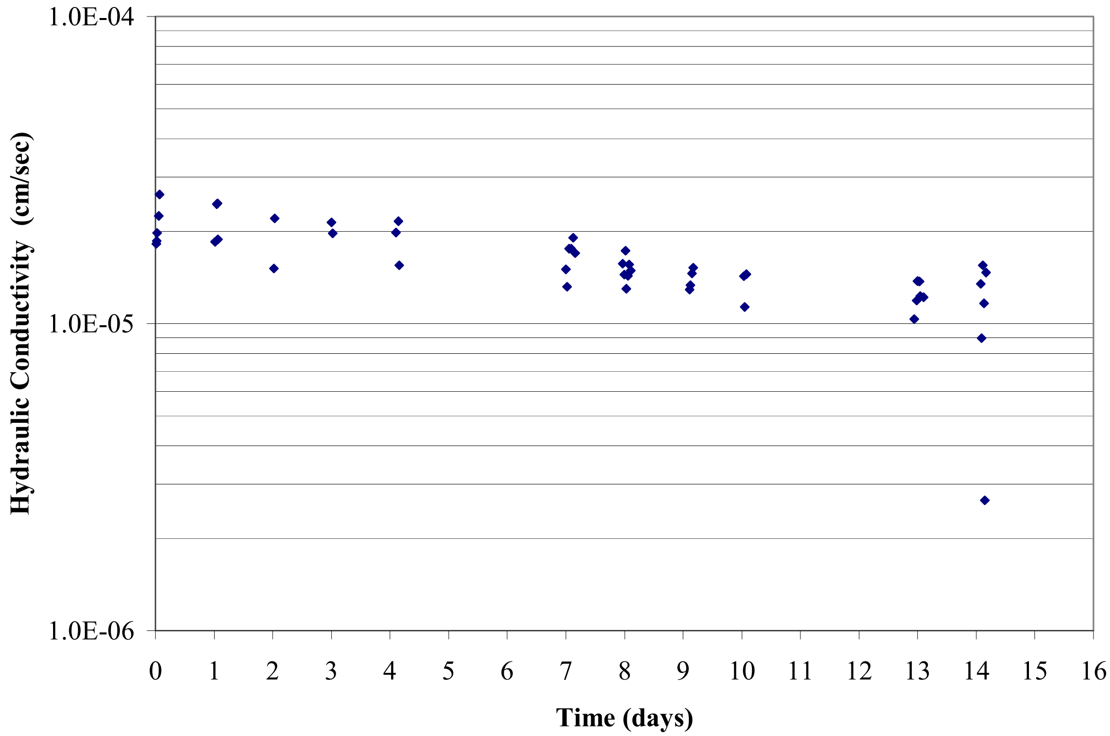

Figure 17. Hydraulic Conductivity as a Function of Time
[Soil 7- Geotextile 2016] Combination

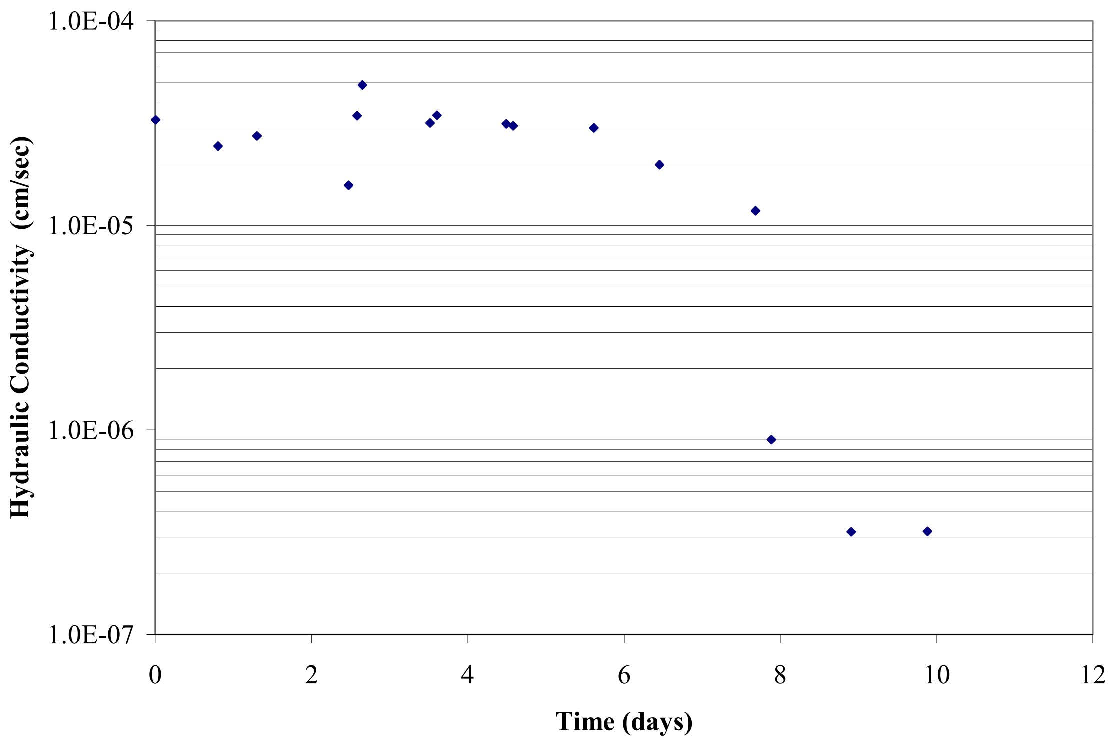

Figure 18. Hydraulic Conductivity as a Function of Time
[Soil10- Geotextile 4553] Combination

Discussion of Test Results

A statistical analysis of the results indicated that the decreasing conductivity could be explained as part of the overall measurement variability, and not necessarily due to the geotextile type. In other words, if the soil type and test apparatus controls the hydraulic conductivity at the beginning of the test, the geotextile combinations showing a clogging potential should have hydraulic conductivities at the end of the test that are significantly different from average conductivity measurements of all the tests. If the ending hydraulic conductivities are not significantly different then it is unlikely that the variability of the results can be due to the geotextile.

The procedure for the statistical analysis shown in Table 9 was to approximate the beginning hydraulic conductivity by taking an average of the first four hydraulic readings for each soil-geotextile combination. The final hydraulic conductivity was approximated by taking the average of the last four hydraulic readings for each geotextile. This data was pooled together with similar data for the three other geotextiles for the same soil for a total of 32 data points per soil. A statistical computer program was used to perform a means separation for the 8 means (4 geotextiles at 2 conditions each: beginning and end) using a 95% confidence level. The computer program groups the means that have a 95% probability of coming from the same population of measurements.

The results of this statistical procedure indicated at least one “clog” for each soil. The ending hydraulic conductivity for soil #1 was significantly different with the BP-2019 geotextile. The ending hydraulic conductivity for soil #7 was significantly different with the BP-2016 geotextile. The ending hydraulic conductivity for soil #10 was significantly different with the BP-4553 geotextile. The problem with taking these results at their face value is that, with the exception of soil #10, the geotextiles that "clogged" had larger opening sizes than geotextiles that showed no indication of clogging. A simpler explanation for the decrease in hydraulic conductivity of these tests is that they are due to changes in the soil structure (e.g., consolidation, self-clogging), or incomplete saturation at the beginning of the test.

Soil-Geotextile Combination	Hydraulic Gradient	k (cm/sec) beginning (average)	k (cm/sec) end (average)	HCR	Time (days)	Pore Volumes	Significant Difference? (95%)
1-2016	35	1.8 E-8	1.0 E-8	0.56	29	0.6	No
1-2019	40	8.1 E-8	2.3 E-8	0.28	36	1.3	**Yes**
1-4551	39	5.8 E-8	2.7 E-8	0.47	34	1.9	No
1-4553	29	1.8 E-8	2.7 E-8	1.5	36	0.4	No
7-2016	21	2.0 E-5	1.0 E-5	0.5	14	25	**Yes**
7-2019	22	1.7 E-5	1.1 E-5	0.65	15	24	No
7-4551	21	1.8 E-7	2.5 E-7	1.4	9	11	No
7-4553	21	1.0 E-6	2.6 E-6	2.6	13	53	No
10-2016	23	7.2 E-6	1.0 E-5	1.4	7	48	No
10-2019	24	5.3 E-6	3.3 E-6	0.62	10	42	No
10-4551	22	1.8 E-5	1.1 E-5	0.61	11	83	No
10-4553	10	2.5 E-5	3.4 E-6	0.14	10	62	**Yes**

Table 9. Statistical Summary for HCR Tests

Conclusion and Future Research Suggestions

The use of an open-graded FA4 sand back-fill as an envelope material without a geotextile wrap in highway edge drains is a viable design. The FA1 gradation specification and sands with large amounts fines may not be suitable to dissipate dynamic pore water pressures when placed along the pavement edge. The CA16 gradation specification is a suitable envelope material from a drainage stand point, but requires a suitable filter material to prevent soil intrusion. In the laboratory test, the AMOCO 4506 geotextile blinded and prevented water from entering the CA16.

The FA4 gradation needs to be modified to control the amount passing the 0.075 mm size to ensure that hydraulic conductivity is above 86 m/day. Envelope materials should be selected based upon large sample saturated hydraulic conductivity tests instead of estimated hydraulic conductivity from gradation.

The standard pipe slot size of less than 2 mm in width is small enough to keep most of the FA4 envelope from infiltrating into the pipe. Threshold slot sizes that would not create a stability problem were not determined for the FA1 and CA16 samples, but the tests indicated that for these samples slot widths of less than 0.6 mm would probably be acceptable for the FA1 tested and slot widths less than 3 mm would be acceptable for the CA16 tested. It may be necessary to test other combinations of envelope materials and pipes in the future. The test developed at ATREL is a good indicator for this purpose. The stability of an envelope with respect to slot width is a function of the entire envelope gradation, not just one "effective" grain size. A well-graded aggregate can remain stable with a relatively wider slot than a uniform graded or gap-graded aggregate.

Neither one of the standard tests for geotextile clogging, the Gradient Ratio Test nor the Hydraulic Conductivity Ratio Test identified soil-geotextile combinations that would clog in the field. The large scale model with the geotextile wrap that was tested in the laboratory, Case 2, did show blinding/clogging. The main difference between the large scale test and the standard small tests was the dynamic loading and the orientation of the geotextile along a vertical wall in the large model; instead of the absence of loading and the orientation of the geotextile along a horizontal surface in the standard geotextile tests.

The time and funding constraints made it impossible to come up with a definitive answer on how IDOT's geotextile specifications should be modified. A new test needs to be developed that better simulates the combined effect of hydraulic gradients, dynamic loading, and the erosion of the soil at the geotextile interface.

Other research needs to be conducted as to the optimal design of highway drainage systems in Illinois. What is the best geometry of the drain in terms of trench width, depth, and location from the loaded area of the pavement? How should pavement layers be modified to improve pavement drainage, while still meeting structural and constructability requirements? Answering these questions will require a combination of empirical and mechanistic approaches. Empirical testing will be required to determine hydraulic parameters likely to be encountered in the field. A mechanistic model will need to be developed that takes into account dynamic loading stresses and seepage stresses on the subgrade and the drain over time.

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Technical Staff

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Phase 3: Andrew Wright
Phase 4: Joseph Comar