74

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16. Abstract

The Virginia Department of Transportation constructed a 2.52-acre parking lot of porous asphaltic pavement in Warrenton, Virginia. Runoff from the lot was collected and monitored for quantity, detention time, and quality. Prior to the lot opening for public use, three storms were successfully monitored. After the lot was opened to the public, four storms were monitored to ascertain whether traffic affected the lot's performance. It was determined that the lot was not performing as desired in the detention of runoff because of the slope across the parking lot, thus this portion of the research effort was terminated. The structural strength of the pavement has been monitored and has performed well for four years.

Two recommendations that address design considerations were made as a result of this study.

1. The permeability of the underlying soil layers should be determined and considered as the main criterion in the design.

2. The subgrade and riding surface of a porous asphaltic pavement should not have any slope.

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74

FINAL REPORT

FIELD PERFORMANCE OF A POROUS ASPHALTIC PAVEMENT

David C. Wyant
Senior Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

Virginia Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the
Virginia Department of Transportation and
the University of Virginia)

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

July 1992
VTRC 92-R10

750

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ABSTRACT

The Virginia Department of Transportation constructed a 2.52-acre parking lot with a porous asphaltic pavement in Warrenton, Virginia. Runoff from the lot was collected and monitored with regard to quantity, detention time, and quality. Prior to the lot being opened for public use, three storms were successfully monitored. After the lot was opened to the public, four storms were monitored to ascertain whether traffic affected the lot's performance. It was determined that the lot was not performing as desired with regard to the detention of runoff because of the slope across the parking lot; thus, this portion of the research effort was terminated. The structural strength of the pavement has been monitored and has performed well for 4 years.

Two recommendations that address design considerations were made as a result of this study:

1. The permeability of the underlying soil layers should be determined and considered as the main criterion in the design.

2. The subgrade and riding surface of a porous asphaltic pavement should not have any slope.

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FINAL REPORT

FIELD PERFORMANCE OF A POROUS ASPHALTIC PAVEMENT

David C. Wyant
Research Scientist

INTRODUCTION

Background

During the 1970s, the Virginia Department of Transportation (VDOT) strengthened and expanded its program for protecting streams and impoundments from adverse effects of highway construction. The majority of VDOT's effort was directed toward controlling erosion and retaining silt on construction projects. These efforts included educating both contract and state construction personnel on environmental concerns and developing erosion control measures and silt retention devices for use on projects.

Although VDOT was concentrating its water quality research efforts in the area of erosion and siltation control, researchers in other national agencies were embarking on research in managing stormwater in urban areas.[1–3] One of the techniques developed for stormwater management in these studies was the use of a porous asphaltic pavement or an open-graded asphaltic friction course. This stormwater management technique utilizes a porous asphaltic paving layer over an open (high void content) aggregate base, which is used as a reservoir. These two layers allow the rapid infiltration of rainwater and good permeability through the layers while providing temporary storage for the rainwater.

Using porous asphaltic pavement is one means of decreasing the number of drainage facilities necessary to catch the surface runoff, improving water quality through filtration and bacterial means, providing temporary detention, recharging the ground water, and reducing the volume and peak quantity of stormwater runoff, thus decreasing the chance for downstream flooding. This type of pavement is recommended for use in areas where the traffic volume and vehicle weight are low. Such areas include fringe or overflow parking areas, residential streets, recreation surfaces, and small airport taxiways and runways where stable subgrade soils with moderate permeability exist.[4]

Planning and Design Considerations in the Construction of Porous Asphaltic Pavement

In the planning and design stages of a porous asphaltic pavement, three major factors need to be considered: (1) the permeability of the underlying soil, (2) hydrologic and hydraulic design, and (3) pavement design. Other factors (such as slope of terrain, underground drainage, and adjacent drainage) may need to be considered in some designs.[1,4,5]

Permeability of the Underlying Soil

Since recharging the ground water is one of the purposes of using this technique, the permeability of the underlying soil is critical. This technique should be used with sandy to sandy loamy soils that have an infiltration rate equal to or greater than 0.5 in/hr. This infiltration rate is fast and is equivalent to a permeability of 3.5×10^{-4} cm/sec or 1 ft/day. Porous asphaltic pavement is not recommended for clay or clayey loamy soils that have an infiltration rate of less than 0.27 in/hr (equivalent to a permeability of 1.9×10^{-4} cm/sec or 0.54 ft/day). If soils such as silty loam are present (infiltration rate of 0.27 to 0.5 in/hr), auxiliary drainage is recommended. Two means of providing additional drainage are providing storage ponds and vertical wells or shafts. Storage ponds constructed close to the porous asphaltic pavement can retain the runoff from the drainage layers while infiltration is occurring both under the pavement and in the storage pond. Vertical holes or wells filled with sand or any stable, fast-draining material or geosynthetic vertical drains can carry the runoff to soil layers with a higher permeability beneath the layer of soil at the surface. Therefore, the permeability of soil layers below the top layer of soil is of importance in areas with a low surface permeability. Soil borings are required in these areas to aid in the design of the system so that the more permeable soil layers can be located and used.[4–6]

Hydrologic and Hydraulic Design

The size of storm the pavement should be designed to handle must be selected, compared with the local hydrologic data, and adjusted to match past storms. After the design storm has been finalized, the quantity of precipitation that may be delivered to the pavement system can be ascertained.

The permeability of the underlying soil layers, especially the top layer, dictates whether there will be positive drainage from the system after detention or retention due to slow soil infiltration rates. No matter whether retention or detention of the runoff is planned, the storage capacity of the system must be determined. The determination of the storage capacity of the system should include consideration of the depth of storage, if any, of the underlying soil (Table 1). The Maryland Department of Natural Resources recommended that drainage from the storage reservoir be completed 72 hr after the storm has stopped.[5] From the size of storm selected, the percolation rate of the asphaltic pavement, infiltration rate and storage depth of the underlying soil, and total time of detention, one can calculate the required total storage capacity of the aggregate reservoir. In 1972, Thelen felt that complete drainage of the porous pavement should occur within 10 days.[1] Figure 1 is a graph used by these researchers to correlate the soil permeability with the maximum daily rainfall and the time the designer desires the base reservoir to be completely drained. Drains are required for less permeable soils after the 10-day drainage time selected.

Table 1

MAXIMUM DEPTH OF STORAGE IN INCHES FOR A SOIL VOID RATIO OF 0.4

Soil Type		Sand	Loamy Sand	Sandy Loam	Loam	Silty Loam	Sandy Clayey Loamy	Clayey Loamy	Silty Clayey Loamy	Sandy Clay	Silty Clay	Clay
I_R (In/Hr)		8.27	2.41	1.02	.52	.27	.17	.09	.06	.05	.04	.02
Maximum	24	496	145	61	31	16	10	5	4	3	2	1
Storage	48	992	290	122	62	32	20	11	7	6	6	2
Time (hr)	72	1489	434	183	93	49	31	16	11	9	7	4

I_R = minimum infiltration rate (inches/hour).

These values represent infeasible solutions.

Source: Maryland Department of Natural Resources. 1984. *Maryland standards and specifications for stormwater management infiltration practices*. Annapolis.

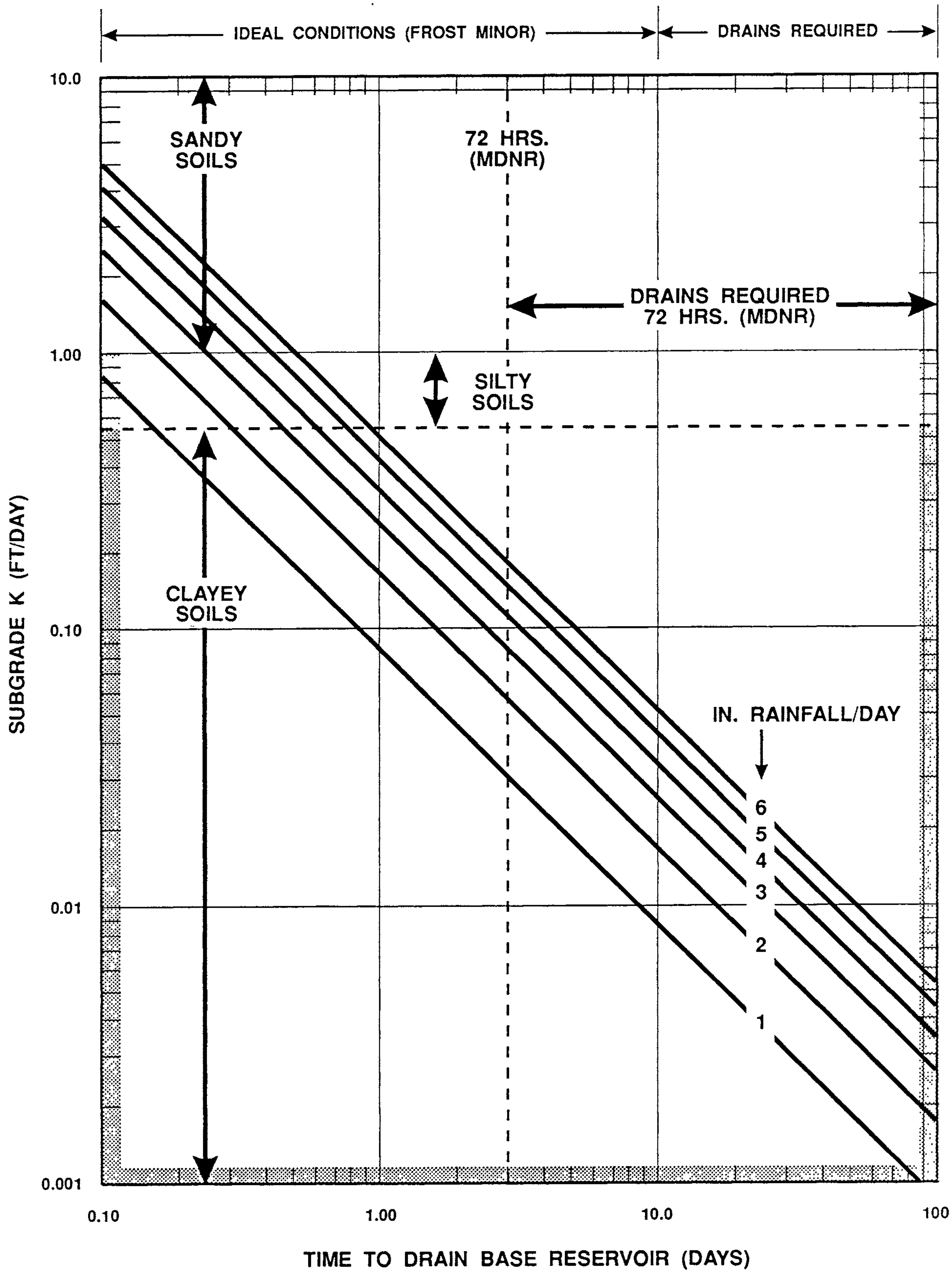

Figure 1. Requirements for subgrade soil permeability.

Since 1972, porous pavement designers have desired faster drainage and less storage. Figure 1 shows that drains are required for more soils to achieve 72 hr of complete drainage than when complete drainage is desired in 10 days. For example, for rainfall of 2 in/day, the designer would require drains for a soil with a permeability equal to or less than 0.06 ft/day or 2.0×10^{-5} cm/sec for 72 hr of drainage.

Another consideration in the drainage design is the potential for frost heave. With the faster drainage conditions desired today, the chance of frost heave is reduced. Figure 1 shows that frost heave was minor for the ideal soil and rainfall conditions. In Publication MS-15 of the Asphalt Institute, frost heave was reported to be a design consideration when the total clay and silt content of the soil exceeded 40 percent by weight of the soil composition.[7] When the total clay and silt content is less than 40 percent by weight, the soil is more open graded and free draining, thus less likely to swell during freezing and cause pavement failures.

Pavement Design

The structural design of the porous pavement system needs to be determined, as with any pavement system. The load-bearing capacity of the various materials in the system from the subgrade to the riding surface needs to be considered along with the traffic volume and loads. If frost heave is a potential problem, the reservoir layer may need to be extended below the frost line even though the structural capacity calculations indicate a shallower depth. In addition, if the drainage layer does not have complete drainage within a short period of time, then saturation of the underlying soil is a concern. Water that is continually in contact with silty or clayey soils can cause the strength of the soil to decrease. Determination of the 4-day-soak California bearing ratio (CBR) would be a good indication of the soil's load-carrying capability and swell potential after extended saturation.

Table 2

MINIMUM THICKNESS OF POROUS ASPHALTIC PAVEMENT IN INCHES

	CBR of the Soil				
Traffic	> 15	10–15	6–9	< 5	ESAL
Light	5	7	9	*	<5
Medium light (max. 1,000 VPD)	6	8	11	*	6–20
Medium (max. 3,000 VPD)	7	9	12	*	21–75

*For these classes of traffic and a soil CBR of 5 or less, studies indicated that the soil can be improved to a CBR of 6 with 2-in size crushed stone.

Note: VPD = vehicles per day; ESAL = average number of equivalent 18-kip, single-axle loads per day.

Source: Thelen, E. 1972. *Investigation of porous pavements for urban runoff control.* NTIS PB-227-5. Springfield, Va.: Environmental Protection Agency.

Table 2 provides the minimum thickness of the porous pavement for various traffic volumes and CBR values of the subgrade. These values are minimums and should be used only as guides. The design of the pavement should be determined using site-specific traffic volumes and loads, soil and aggregate CBR values, and the other factors discussed.

Test Site and Related Factors

In 1985, VDOT decided to install a porous asphaltic pavement in a commuter parking lot north of Warrenton, Virginia. This decision was made after preliminary investigations by VDOT and several discussions with the Division of Soil and Water Conservation (DSWC) of the Department of Conservation and Recreation (DCR) and the Northern Virginia Planning District Commission. Since the parking lot is located within the Occoquan Watershed, the disposition of the stormwater runoff is regulated and monitored by the Northern Virginia Planning District Commission (the reason for their interest in the project). The DSWC was the lead state agency in developing and implementing stormwater management techniques, such as the porous asphaltic pavement, as required by the 1972 Federal Water Pollution Control Act Amendments (P.L. 92-500).

After the site was selected, the design of the pavement was a joint effort of the Warrenton Residency and the Culpeper District. The porous pavement areas were designed by the Hydraulics Section of the Culpeper District's Location and Design Division with assistance from the Materials Division and Residency. A soil survey of the proposed site was performed by the Materials Division. Soil samples were secured and tested by the Materials Division for design parameters, such as CBR, permeability, gradation, and classification. In addition to the Residency and the District, the Virginia Transportation Research Council (VTRC) was asked to be a part of the effort. The Culpeper District and the DSWC requested that VTRC monitor the porous pavement and evaluate its performance. This report is the documentation of that monitoring and evaluation program. Through the cooperative efforts of the Residency, the District, and VTRC, benefits for all with minimal effort from each could be derived for the design, construction, inspection, monitoring, and evaluation of future porous asphaltic pavements.

Prior to the commencement of the construction, additional soil samples from four locations at the site were secured by VTRC for classification, routine engineering tests, CBR tests, and permeability testing. In addition to the subgrade soils testing, aggregate base materials from the scheduled supplier were secured and tested for the same parameters. To verify the test results, field permeability tests were performed at the job site.

VDOT proposed to construct an access road and a roadway through the parking lot using normal pavement design methods and asphaltic pavement mixtures (B-3 base and S-5 surface) so the sections could carry heavy commuter buses traveling to the Washington, D.C., area (Figure 2). The design range of the asphaltic mixtures are shown in Table 3. The total paved area planned for construction was 2.52 acres. The porous asphaltic pavement area is large enough to park 213 cars. If utilization of the parking lot is satisfactory, an additional 2 acres of parking will be constructed at a later date.

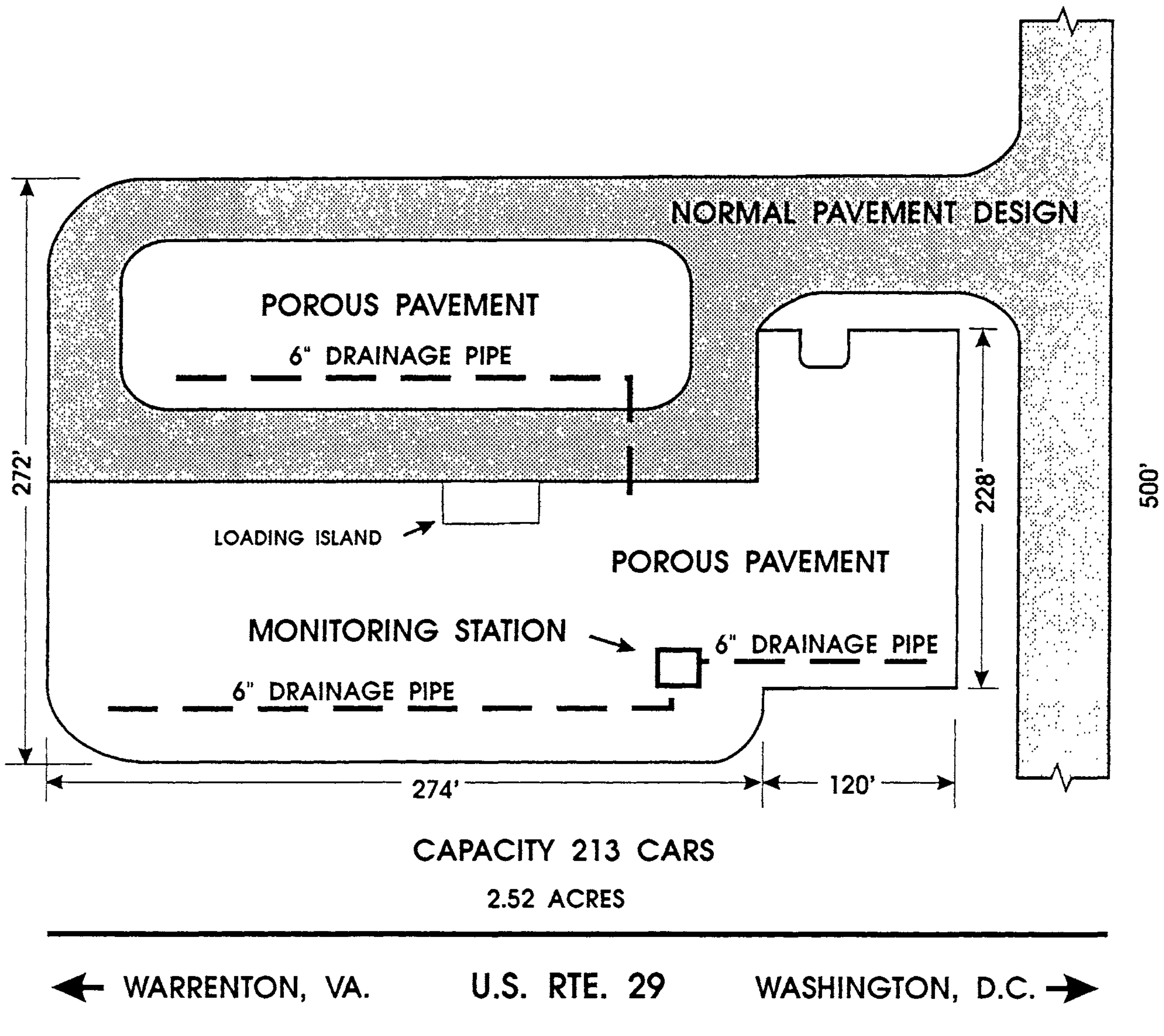

Figure 2. Site plan.

760

Table 3
DESIGN RANGE OF ASPHALTIC MIXTURES

Mixture	Percentage by Weight Passing Sieve								%
	1 1/2 in	3/4 in	1/2 in	3/8 in	No. 4	No. 8	No. 30	No. 200	Bitumen
B-3	100	73–85			38–48	28–35		2–6	4.0–7.0
S-5			100		53–67		19–27	4–8	5.0–8.5
S-8			100	85–100	15–32	0–7		0–0.5	6.0–12.0

The pavement design of the access road and roadway to the loading island was determined to require 12 in of 21-A aggregate base material (Table 4), 4 in of B-3 bituminous concrete base mixture, and 2 in of S-5 bituminous concrete surface mixture (Figure 3). The underlying soil is a clayey soil with a good bearing capacity.

In the porous pavement areas (Figures 2 and 4), 2 in of an S-8 bituminous concrete surface mixture, 4 in of No. 19 aggregates, 8 in of No. 57 aggregates (Table 4), and a geotextile were placed over the clayey soil. The No. 57 aggregate material was designed to provide 37,208 ft^3 of storage for the stormwater. This storage approximately doubled the required storage of 19,440 ft^3 as determined by the county of Fauquier and the Northern Virginia Planning District Commission using a 2-year frequency storm. The No. 19 aggregate material provides some additional storage volume but is primarily a leveling course for the S-8 porous asphaltic layer (Figure 5).

The average slope across the parking lot was approximately 0.25 in to the foot. The access road and roadway through the lot were on the high side of the parking lot. Drainage from the lot would go toward Route 29 and pass through a culvert under Route 29.

As shown in Figures 2 and 4, 6-in drainage pipes were installed in the porous pavement areas to drain the overflow from the storage reservoir (No. 57 aggregate base material). One pipe was used in the upper lot to convey the overflow water

Table 4
DESIGN RANGE OF AGGREGATE LAYERS

Size No.	Percentage by Weight Passing Sieve										
	2 in	1 1/2 in	1 in	3/4 in	1/2 in	3/8 in	No. 4	No. 8	No. 10	No. 40	No. 200
19	82–95	82–95		67–83					26–41	14–24	4–5
21-A	100		94–100			63–72			32–41	14–24	6–12
57	100		90–100		26–60		0–7	0–3			

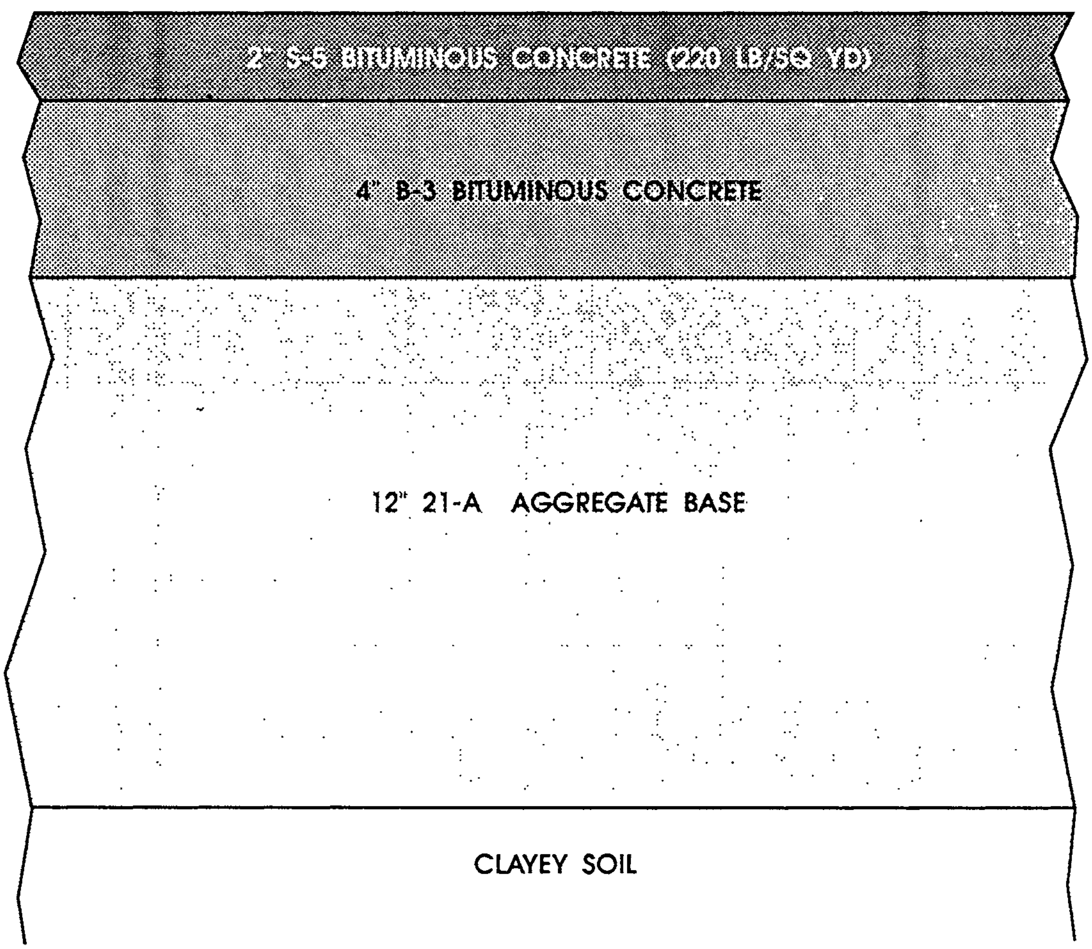

Figure 3. Normal pavement design.

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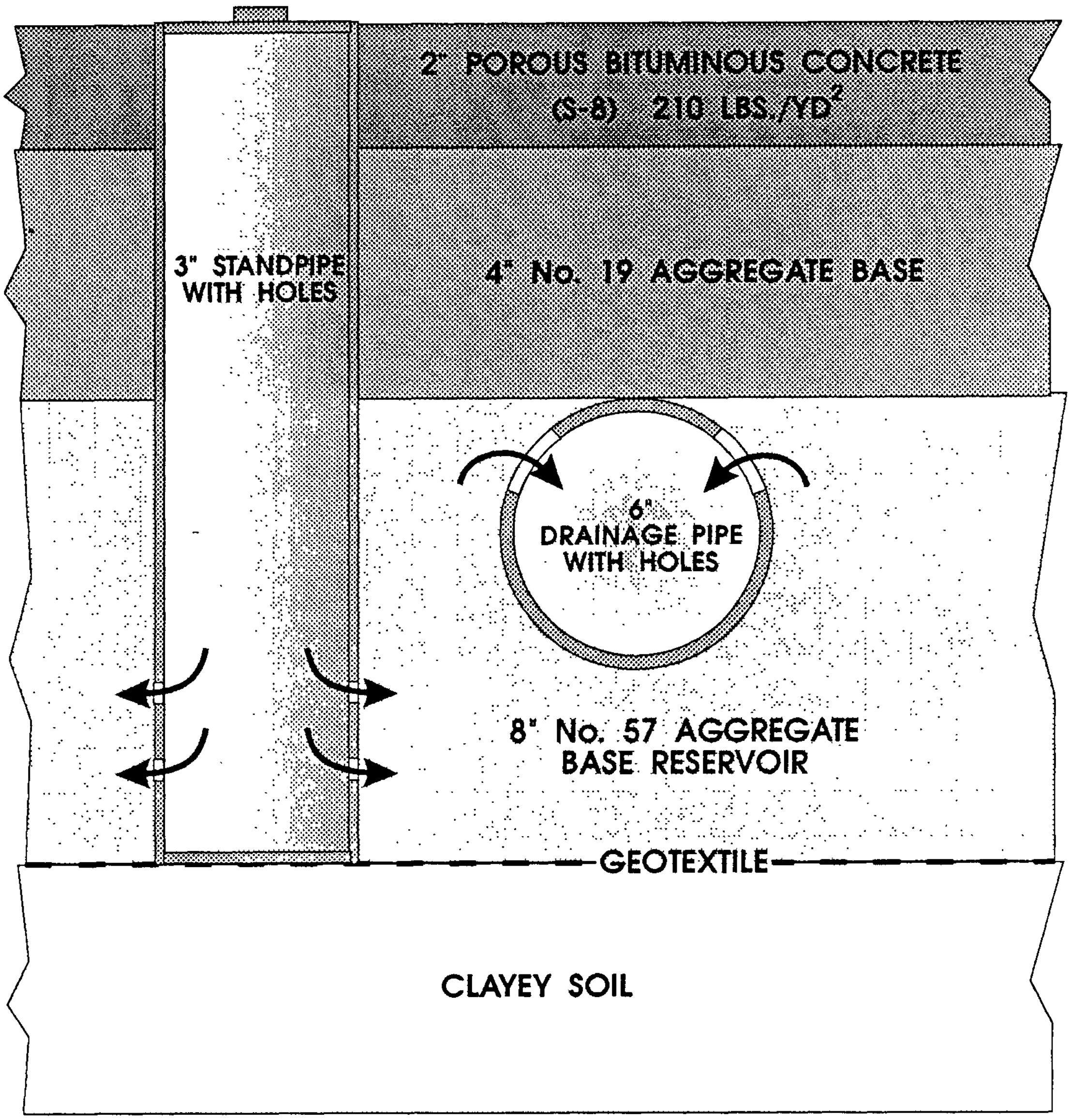

Figure 4. Porous pavement design.

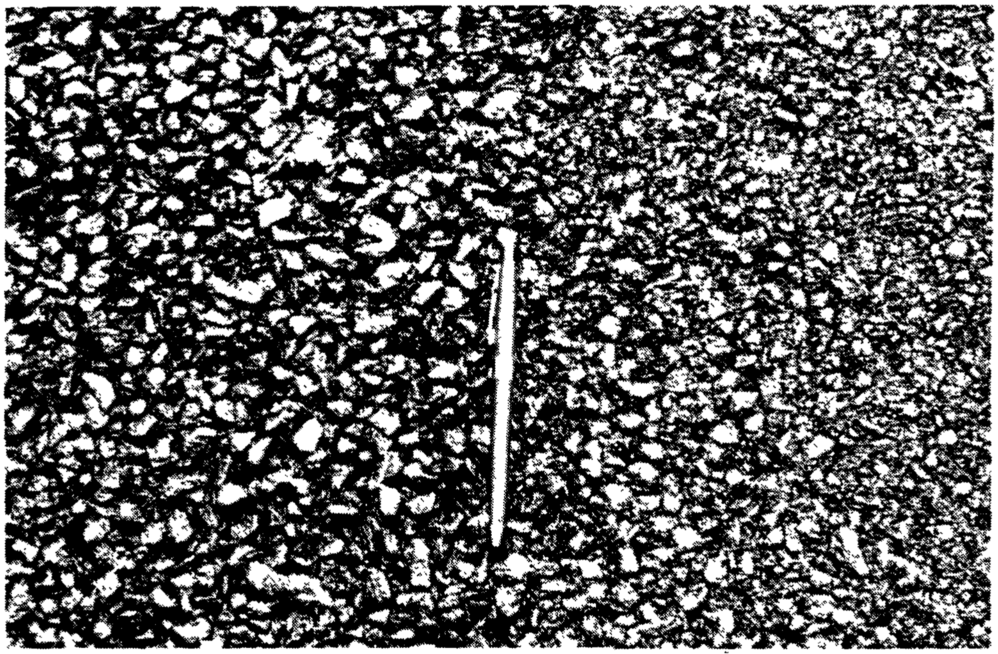

Figure 5. Surface mixture S-8 on the left and surface mixture S-5 on the right.

underneath the roadway by the loading island into the lower porous pavement parking area. The two remaining segments of pipe were placed along the lower e of the porous pavement areas to carry the overflow into a drop inlet that was in th storm sewer system. This drop inlet also served as the monitoring station for this study. As shown in Figures 4 and 6, the 6-in drainage pipe was placed at the top the No. 57 aggregate base layer. The drain holes in the pipe were within 2 in of th top of the pipe, so theoretically, 6 in of water was designed to be in the base layer prior to having any overflow. These pipes were wrapped with a geotextile during placement. A woven geotextile was used as the wrap as well as the separator between the No. 57 aggregate base material and the subgrade soil (Figure 7).

Commencement of the construction of the parking lot was delayed several months by the contractor. Actual construction commenced around March 1987. The earthwork, laying of pipe and utility lines, placement of aggregate, and other work required prior to placement took approximately 6 months to perform. The porous asphaltic pavement was placed on August 24, 1987, but the parking lot was not opened to the public until December 1987.

76

Figure 6. Pipe wrapped in geotextile in No. 57 aggregate base layer.

Figure 7. Placement of stone on geotextile.

PURPOSE AND SCOPE

This study focused on the effectiveness of the porous asphaltic concrete pavement to affect the runoff quantity and quality. The quality of the runoff was ascertained by the periodic monitoring of several key parameters of water quality.

The major objectives of this study were to determine the quantity and intensity of the rain that fell on the parking lot, the volume of stormwater runoff delivered to the monitoring station via the 6-in drainage pipes, the delay in this discharge reaching the monitoring station, and the infiltration of the stored runoff into the soil (stormwater management).

METHODS

To accomplish these objectives, a collection box was designed to fit inside the drop inlet or monitoring station (Figure 8). All the discharge from the two drainage pipes was collected in the box for sampling and discharge measuring. The effluent drained from the box through a triangular weir, in one side, which provided an easy means to measure the discharge. The effluent then dropped into the storm sewer that drained the adjacent wooded areas around the parking lot. Flow measuring and water sampling equipment was installed in a large secure box resting on top of the drop inlet and above the collection box (Figures 9 through 11). The sampling and flow measuring tubes were anchored securely in the collection box since their location was critical to obtaining representative samples and measurements. In addition, still, calm water areas were created in the box for the flow measuring tube since flow is determined by the depth of water flowing over the weir.

Through the efforts of personnel from the Warrenton Residency, the equipment was activated when a storm was imminent. VTRC was then notified that the equipment was turned on. Flow measurements were continuously taken around the clock, and sampling of the discharge was generally performed every 15 min during the storm.

The rainfall intensity and quantity were measured with a tipping bucket rain gauge at a site near the parking lot (Figure 12). This equipment operated continuously for 8 days without requiring servicing.

In order to ascertain the rate of infiltration into the subgrade of the runoff collected in the storage reservoir of the parking lot, depth measurements to the water level in the eight 3-in standpipes shown in Figures 4 and 8 were taken periodically both during and after a storm. The standpipes had holes near the bottom and were placed at various locations throughout the parking lot. The amount of runoff stored in the reservoir at any time was determined from these measurements (Figure 13).

76

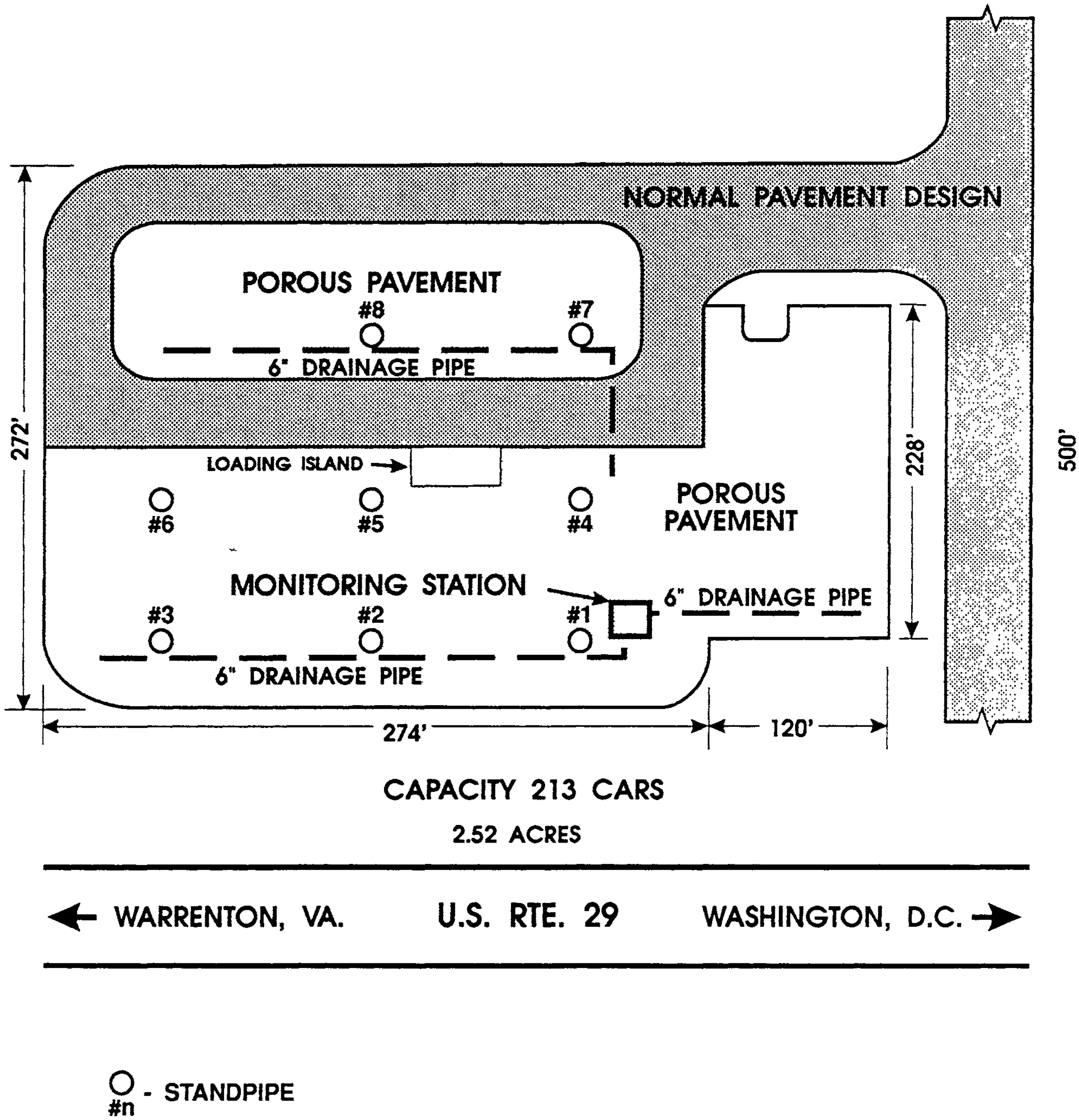

Figure 8. Standpipe locations.

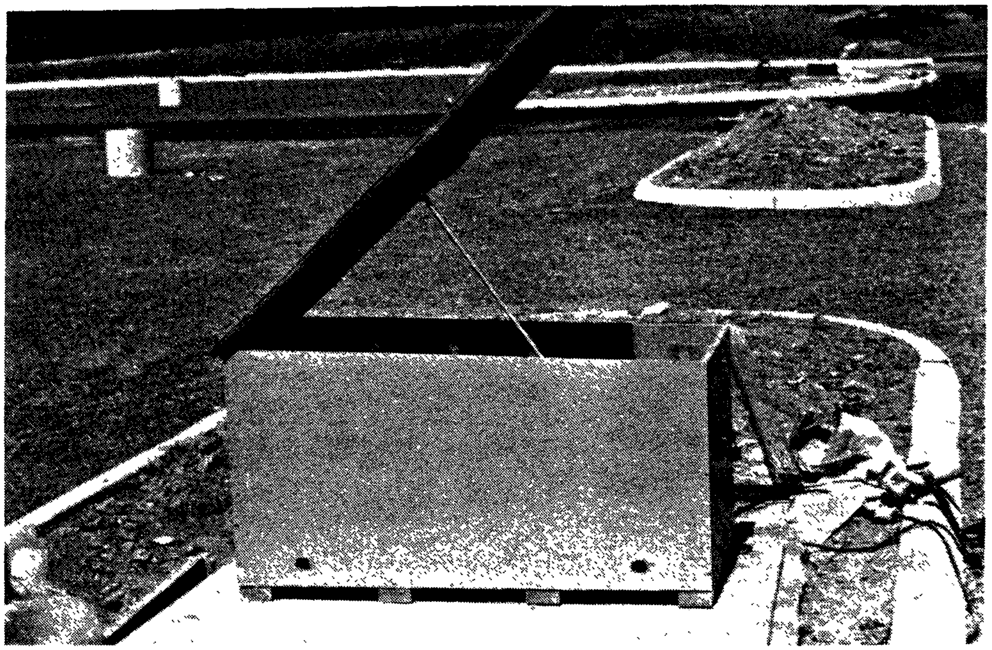

Figure 9. Equipment box on top of drop inlet.

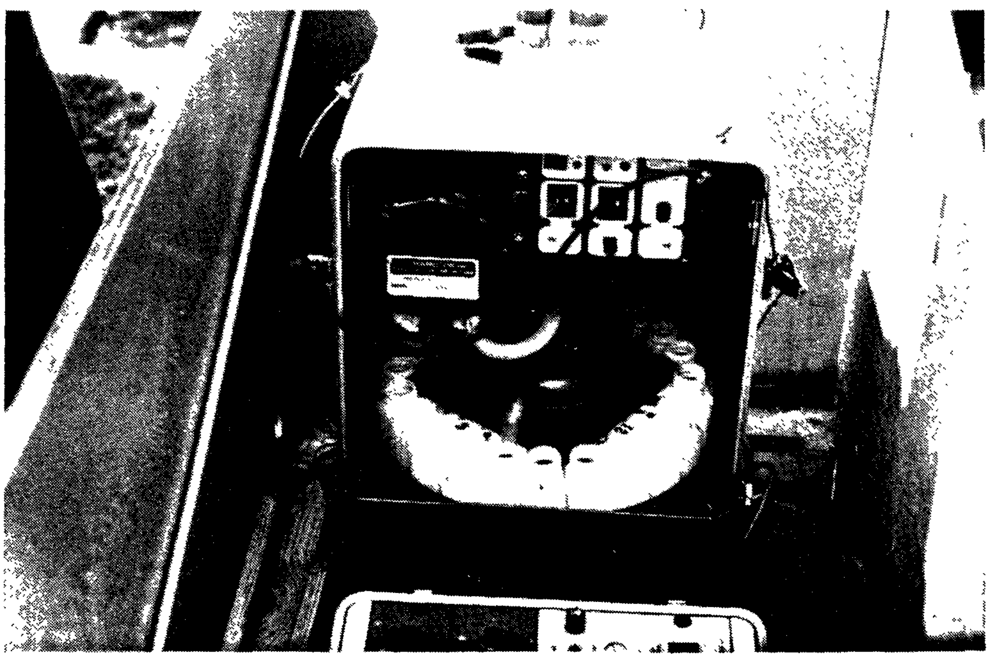

Figure 10. Water sampler.

Figure 11. Flow meter with batteries.

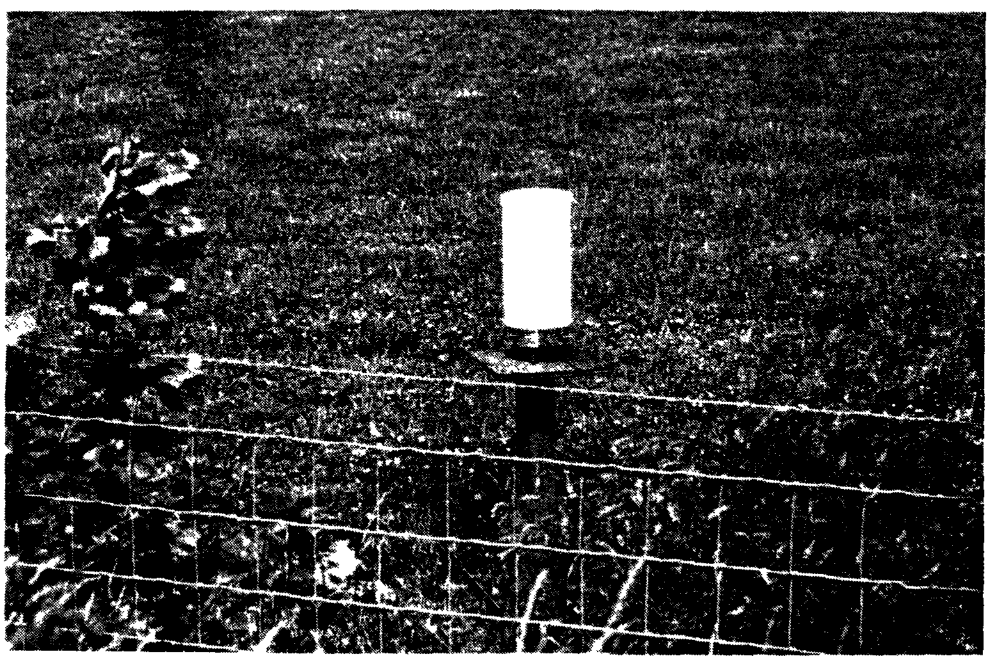

Figure 12. Rain gauge.

Figure 13. Checking standpipe for presence of water.

The water samples collected during the storms were processed to measure total suspended solids, conductivity, pH, and oil and grease content. The amount of total suspended solids was determined for every water sample collected, and the other parameters were periodically determined to assess the temporal effect. This monitoring schedule was dictated by budget limitations, sampling difficulties, storage problems, and personnel availability.

The conductivity of the discharge, which is an indication of the total concentration of ionized substances dissolved in the effluent, was periodically measured to ascertain if there were any major changes in the water quality. The pH of the discharge should indicate any significant chemical changes, and the oil and grease content should provide data on any changes of these deposits on the parking lot. Through a monitoring program of this type, trends in these parameters (and thus the water quality) can be studied. Significant changes in any of these parameters would warrant additional and more precise testing to determine the exact cause of the change.

Another concern raised about porous pavement is the effects on its structural strength. With the storage of water in the aggregate reservoir beneath the open-graded asphaltic layer, questions have been asked about the loss in strength due to the different design as compared to a normal pavement (i.e., greater chance of freezing of the materials and the strength loss of the soil from being saturated). To address these concerns, deflection measurements were taken after the construction but prior to opening to traffic. Deflection measurements were repeated twice a year thereafter. If there was any strength loss due to the factors discussed, the results from these measurements would indicate so.

Use of the parking lot was monitored to determine if the number and types of vehicles and the deposits left on the lot affected the pavement performance. In addition, the traffic count indicates whether there was a need for the commuter lot, how well it is accepted by the traveling public, and when, if ever, to expand the lot.

RESULTS AND DISCUSSION

Vehicle Usage

Use of the parking lot steadily increased over the first 4 years. In the first several months, an average of 73 cars per day were parked in the lot. In the next 3 months, the average rose to 84 cars per day and after 1 1/2 years to an average of 103 cars per day. Within the first 1 1/2 years of use of the parking lot, usage increased 48 percent and has steadily increased to an average of 123 cars in 1991, or more than 50 percent of the capacity of the lot. Effects of the increased usage were not noticeable during this study.

Materials

Soil samples collected by VTRC had the following properties:

- *maximum density*: 99.9 pcf
- *optimum moisture*: 24.1 percent
- *liquid limit*: 43
- *plasticity index*: 13
- *permeability*: 2.5×10^{-7} cm/sec
- *classification*: A-7-5(4).

The classification identifies the soil as a clayey soil with greater than 40 percent –200 material and a typical permeability rate (extremely slow) for these soils. This permeability rate was determined from both the field tests and the laboratory tests. The laboratory tests were performed on disturbed samples compacted near optimum moisture content and maximum density. In Figure 1, this permeability is equivalent to 7×10^{-4} ft/day and is below the bottom of the figure. These soils are undesirable below a porous asphaltic pavement since water will essentially not drain. With the high percentage of –200 material, the soil is susceptible to swelling during freezing.

The Culpeper District reported in their design notes that the soil on the site was a silty loam. In Table 1, a silty loamy soil is shown to have an infiltration rate of 0.27 in/hr, or 1.9×10^{-4} cm/sec.[5] In earlier studies, it was recommended that auxiliary drainage, such as storage ponds or vertical wells, be provided with silty loamy soils.[3–5] This site was not designed with any auxiliary drainage.

The aggregate base materials (No. 19 and No. 57) were very open and had excellent permeability (4.0×10^{-2} and 5.1×10^{-2} cm/sec, respectively). These materials were excellent products for fast permeability rates, but the No. 19 material presented problems to the paving contractor because of its instability. As the contractor attempted to advance the paver, the tracks dug into the No. 19 leveling course. This problem was probably due to the minimum compactive effort (2 or 3 passes with the roller) used on the open-graded aggregate bases and the lack of fines in the base material to prevent it from becoming a dense mixture. Since these pavers carry and push heavy loads, the potential for this problem to occur with almost any stone used for porous pavements is probable since extensive rolling is not desirable.

The geotextile chosen for the project was a woven product. Typically, woven geotextiles are not the best type to use to wrap drainage pipes because of their ineffectiveness as a filter. However, their use as a strength enhancement layer is common. On this project, most geotextiles would have performed satisfactorily since the wrapped pipe is located in the No. 57 aggregate layer (Figure 4) and is not likely needed to filter any fines (less than No. 100 sieve size particles). In addition, the material with the most fines is the soil, and that is covered with the same geotextile.

Water Monitoring

The runoff from the parking lot was monitored before and after the lot was opened to the public.

Before Opening

The parking lot was not open for public use for 3 months after the placement of the porous asphaltic pavement. During this period, attempts were made to collect runoff data at least six times. For various reasons (water sampler or flow meter malfunctions, cold weather problems, etc.), only data from three storms were usable.

The first usable background data were from a storm on October 3, 1987. The total rainfall was 0.37 in. Figure 14 shows the cumulative rainfall curve as well as the flow from the porous pavement drainage pipes. The rainfall started around 1000 hr and had an almost constant intensity until 1700 hr. Figure 14 shows that the flow through the monitoring box started around 1300 hr, or after 0.25 in of rain had fallen. For a rainfall of similar intensity, the delay in drainage from this parking lot would occur approximately 3 hours after the storm started.

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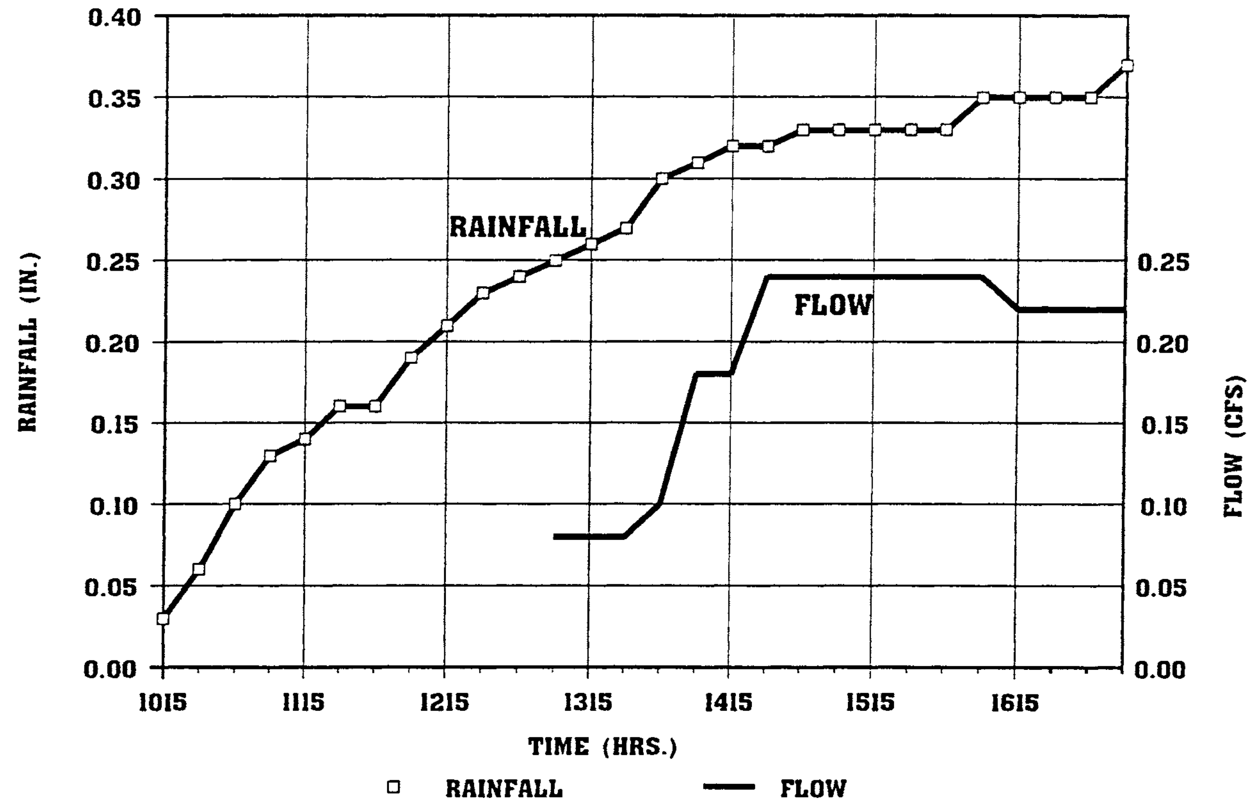

Figure 14. Rainfall and flow vs. time (10/3/87).

When the rain began, the person responsible for turning on the water sampler would go to the parking lot to activate the sampler. (The flow meter and rain gauge were continuously operating devices that did not require service as frequently as the water sampler.) At the time of activation, it was difficult for the residency person to ascertain how long the storm would last, what the intensity of the rainfall would be, and how the porous pavement would perform in terms of detention time and quantity of effluent. It was felt by this researcher that the earlier portion of a storm could be the most important time since the first flushing of the pavement might provide significant results when higher concentrations of pollutants could be in the runoff. Considering these factors, the fact that the maximum number of water samples that could be collected during any one time period was 28 (maximum number of sample bottles), and the desire to secure frequent samples, this researcher made the decision to collect a water sample every 15 min, which would cover the first 7 hr of a storm.

For the October 3rd storm, the water sampler was activated at 1030 hr, but meaningful data were not acquired until approximately 1300 hr since flow into the monitoring box did not start until that time. Small volumes of water were collected by the sampler prior to 1300 hr from the reservoir of water kept in the bottom of the monitoring box after each storm to prevent the sensing tube of the flow meter from being exposed to the atmosphere and ruining the pressure transducer. Figures 15 through 17 show the suspended solids, pH, and conductivity results of the October 3rd storm. Comparing these figures with Figure 14, it appears that interference

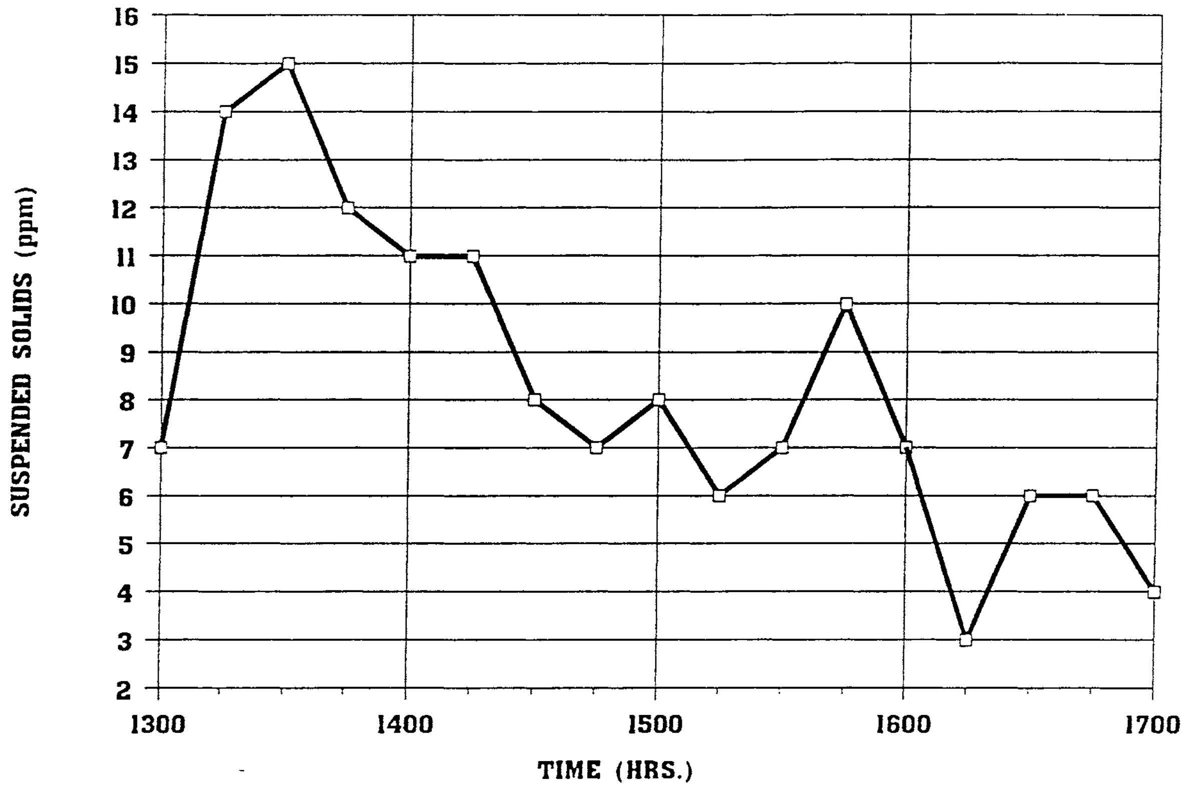

Figure 15. Suspended solids vs. time (10/3/87).

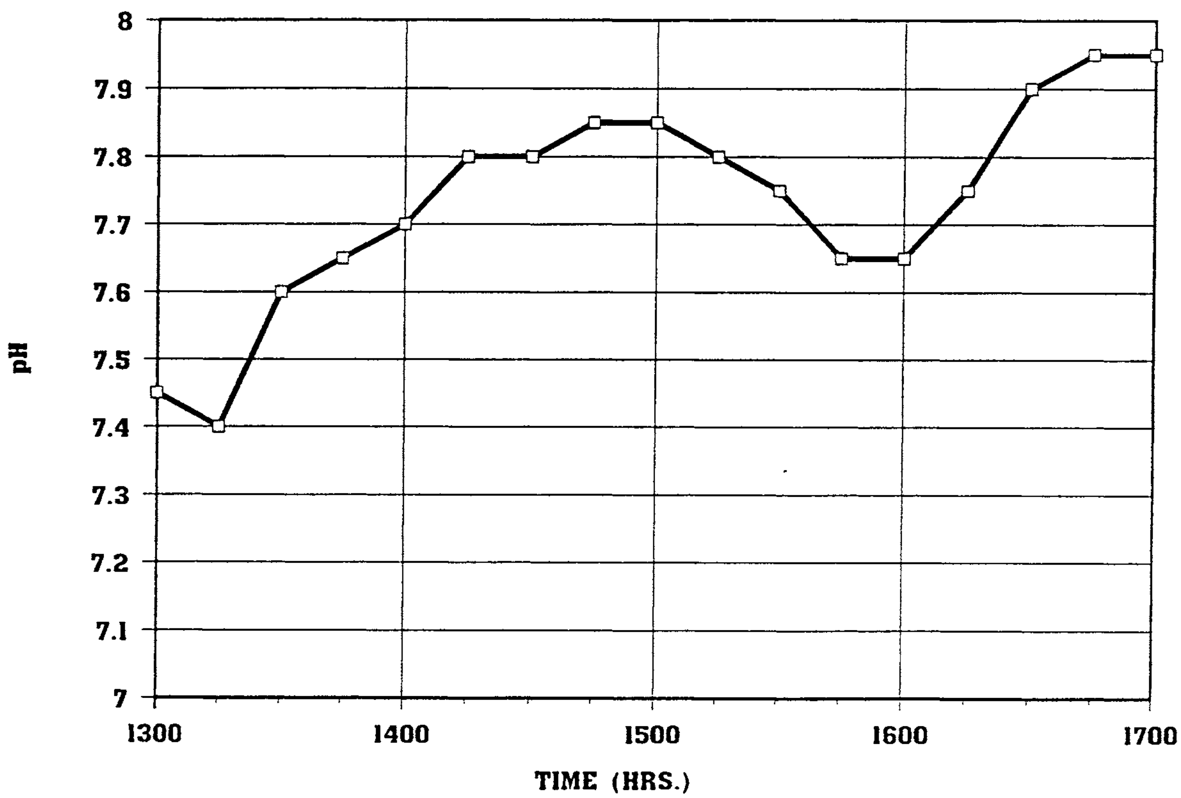

Figure 16. pH vs. time (10/3/87).

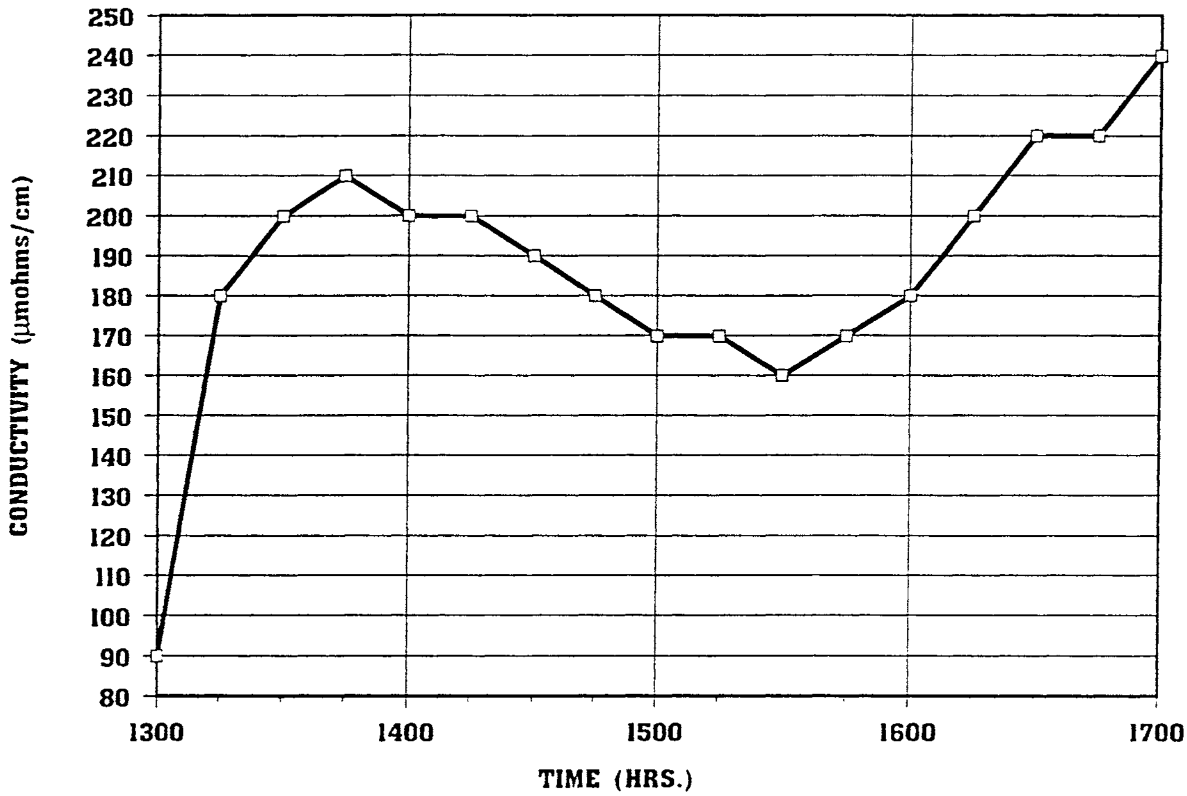

Figure 17. Conductivity vs. time (10/3/87).

from the residual left in the box affected the results until at least 1345 hr, when the flow started to increase or be significant. These erroneous data are more evident on Figure 15 where the suspended solids are highest at 1330 hr.

The second storm prior to opening the parking lot for public use occurred on October 27, 1987. It started to rain very slowly at 0800 hr, and the water sampler was not turned on until 1410 hr. The activation of the water sampler this late after the rain commenced did not hamper data collection for this storm. At 1410 hr, the total rainfall was 0.20 in, and no water was entering the monitoring box or was present in the aggregate reservoir. Figure 18 shows the cumulative rainfall curve. As shown in Figure 18, the intensity of the rain started to increase at 1430 to 1830 hr, when a large increase occurred for the next 40 min. Figure 19 is a graph showing the flow into the monitoring box from the aggregate reservoir. As with the first storm, flow into the box started after approximately 0.25 in of rain had fallen. The flow curve shows that the peak flow (0.48 cfs) occurred 1 hr (2010 hr) after the rainfall ceased. Comparing Figures 18 and 19, one can see that the two curves do not have a direct relationship to each other, as one would suspect. For instance, the large intense rainfall between 1830 and 1910 hr does not show on the flow curve in Figure 19 when comparing the two curves. If the flow curve indicated a steeper slope later than the earlier portion of the curve, then one would conclude that there is a direct relationship between the two curves (i.e., equal volume of water out for equal volume of water in). Since the slopes of the curves in Figures 18 and 19 are the opposite of each other (less intense rainfall with larger flow rate and most

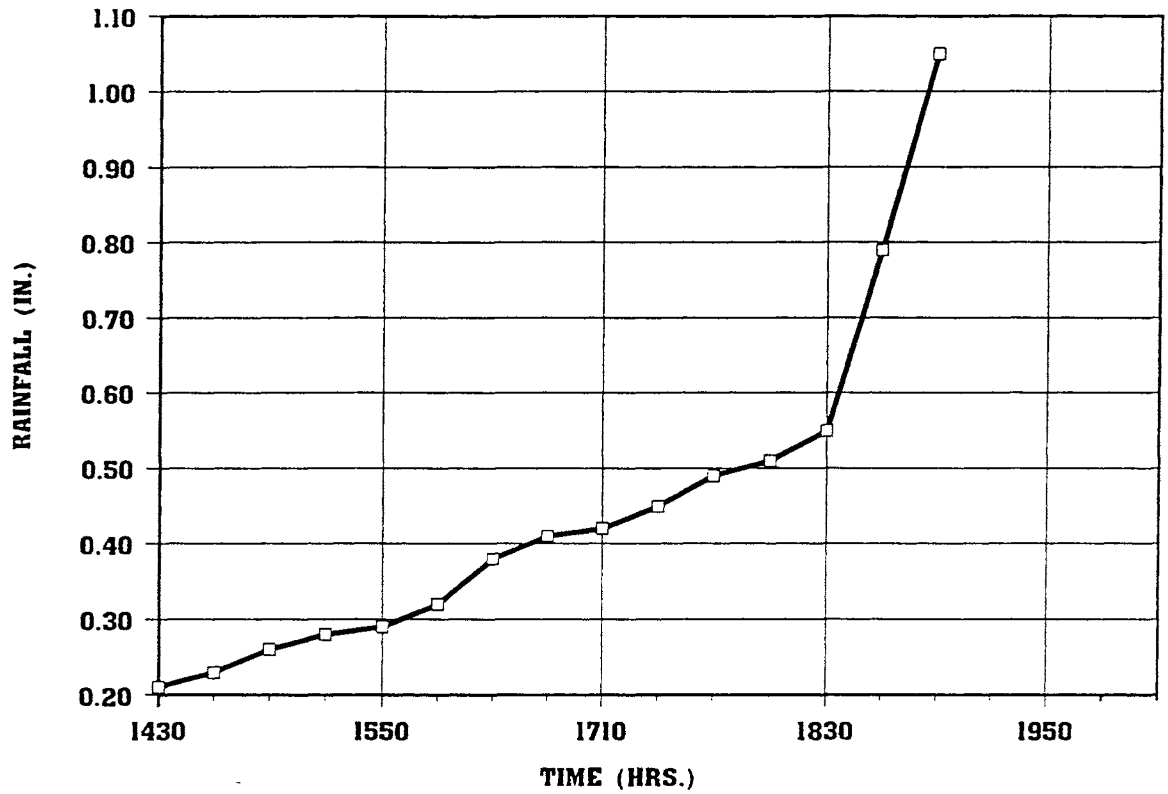

Figure 18. Rainfall vs. time (10/27/87).

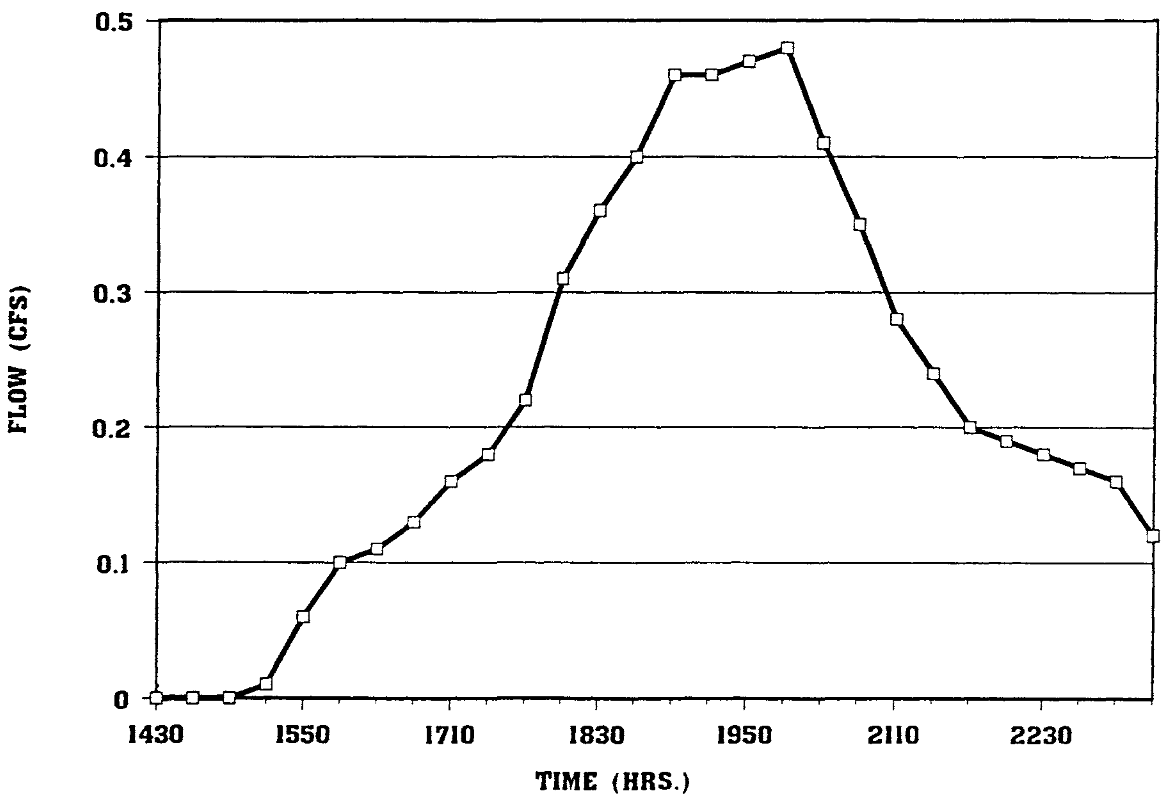

Figure 19. Flow vs. time (10/27/87).

intense rainfall with lower flow rate), one would expect something to be occurring other than detention or direct runoff. As will be explained later in more detail, the collected rainfall in the aggregate reservoir was flowing back up through the pavement and running over the lower grassed area rather than being detained within the porous pavement system. This was occurring while the 6-in drainage pipes were running less than full capacity.

Figure 19 indicates that the first measurable flow into the monitoring box occurred at 1530 hr. When there was an extended period of time between storms, the first water samples from the box contained a large percentage of impurities, such as dust and other fallout, that had collected in the box since the last storm. These samples had high concentrations of these impurities because of the amount of deposits accumulated in the box and the small quantity of water extracted from the box by the sampler due to the small volume of water kept in the box at all times. (Since a pressure transducer was used to measure the depth of water and consequently the flow into the monitoring box, water had to remain in the box and over the end of the pressure transducer tube so the transducer would not be destroyed.) Figures 20 and 21 confirm this occurrence: the first several data points are high and out of line with the other data. This also was true for the October 3rd storm as well as several other later storms. If the sampler was turned on too early (as for the first two storms by about 2 hr), the samples were collected and most of the data were useless and not shown on the enclosed figures.

Figure 20 indicates that the maximum suspended solids (140 ppm) occurred at 1910 hr, just as the most intense rainfall ceased. The values for the suspended solids during the earlier stages of the storm ranged between 20 and 50 ppm, which is significantly more than the values for the October 3rd storm (Figure 15) but still low. Since the parking lot was not open to the public and very little traffic from others was on the pavement, most of these initial values could be attributed to the fines within the aggregate reservoir.

The pH and conductivity curves (Figures 16, 17, 21, and 22) for these first two storms indicate an increase in both parameters as the monitoring of the storm continued. However, through closer study, it can be seen that these increases started to occur when the flow was beginning to decrease. This may be due to the increased concentration of the water in the monitoring box. For both storms, the pH was a little above neutral, or in the upper 7 or lower 8 range, and the conductivity was generally in the 150 to 200 range.

It was decided prior to the commencement of the study that only one oil and grease determination would be made for most storms due to the cost of running an oil and grease test. An oil and grease sample was collected at 1750 hr, which was during the earlier stage of the October 27th storm. It was determined from that one sample that there was 4.4 mg/l of oil and grease in the runoff. However, this value is probably in error since the sample was not preserved or tested in a timely manner.

The last rainfall successfully monitored prior to the parking lot being opened for public use occurred on November 10, 1987. This storm had a total rainfall of

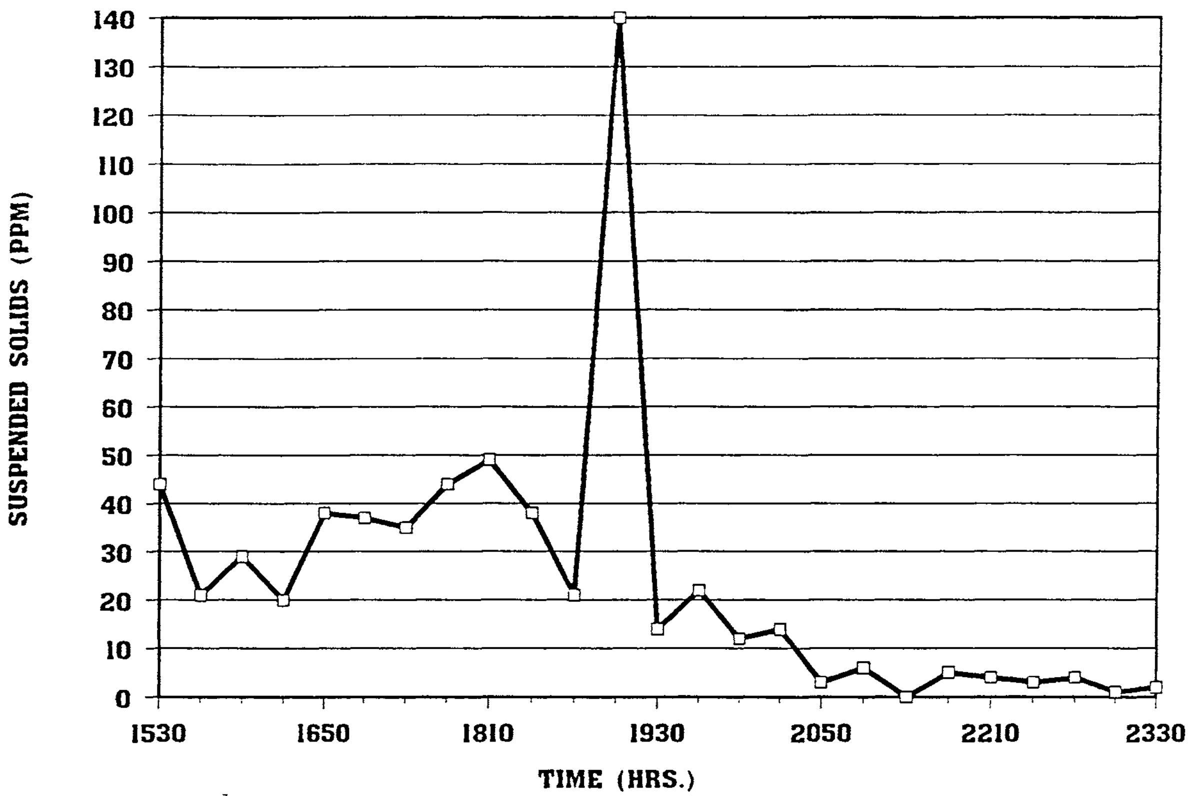

Figure 20. Suspended solids vs. time (10/27/87).

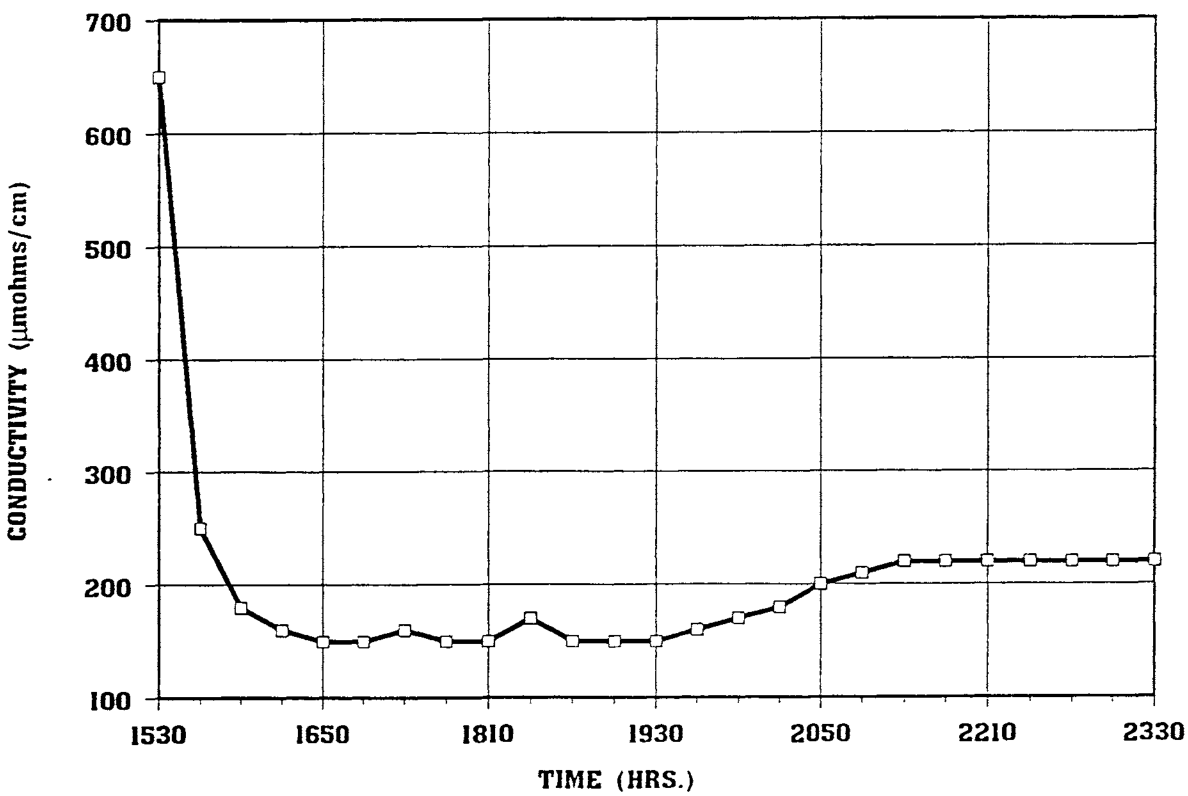

Figure 21. Conductivity vs. time (10/27/87).

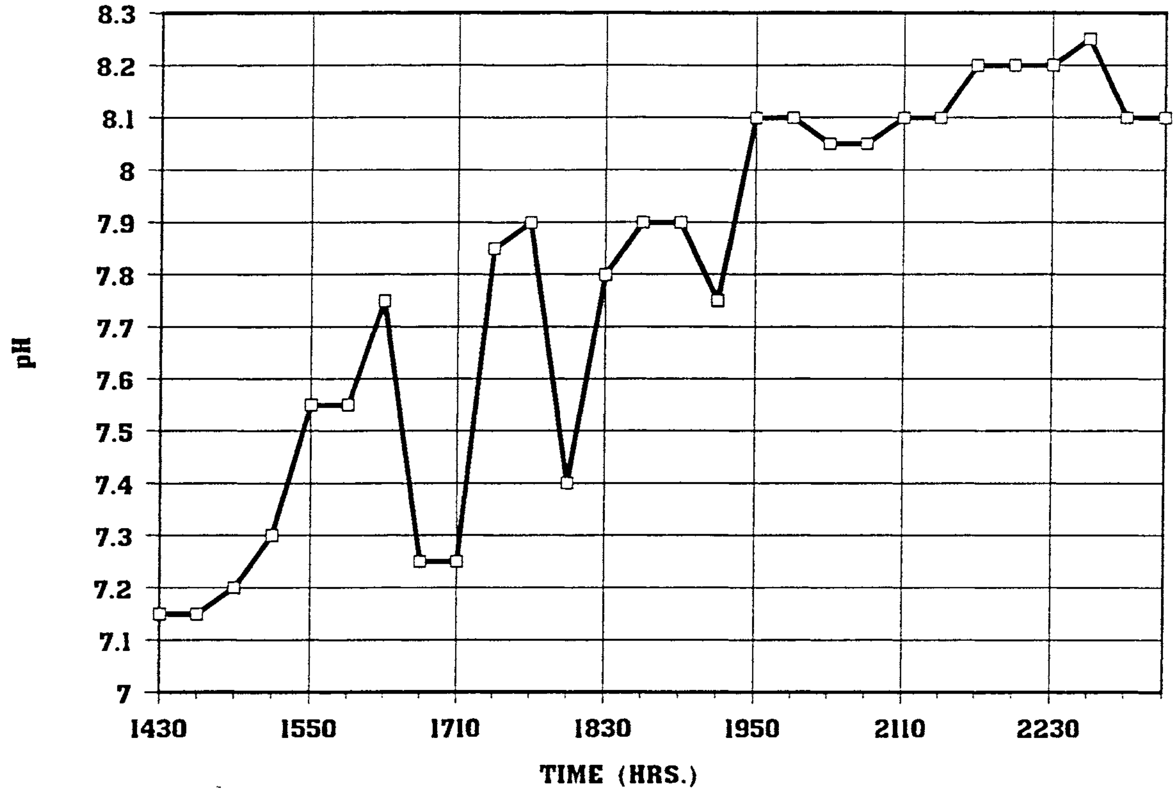

Figure 22. pH curve (10/27/87).

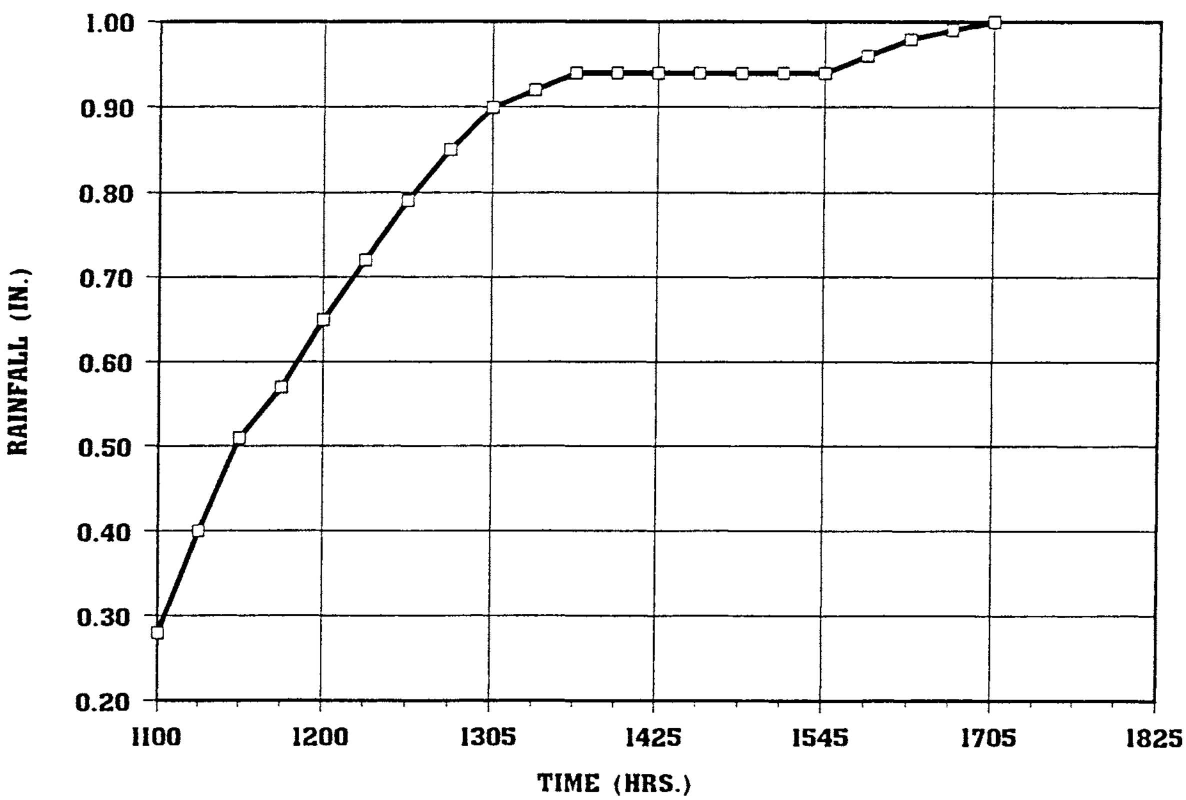

Figure 23. Cumulative rainfall curve (11/10/87).

1.00 in and a different intensity than the October 27th storm of 1.05 in (Figure 23). As happens more often than not in this type of work because of the conjecture about the weather and (sometimes) the time the rain storm occurs (early a.m. hours), the water sampler was turned on a little late. Field notes kept on the project and about this storm indicate that water was starting to enter the box slowly at 1030 hr, which is the time the sampler was turned on. However, upon arriving on the project at 1000 hr, the VTRC technician noted that no water was entering the box or was present in the standpipes, which is an indication of the depth of water in the aggregate reservoir. At 1000 hr, the technician decided that he would wait until a flow was being received in the box before activating the sampler in order to obtain the most information on the storm with the limited number of bottles in the sampler. When he turned the sampler on at 1030 hr, he guessed that flow into the box would not be critical for an additional 30 min; thus, the first sample was delayed until 1100 hr. As would happen, a significant volume of discharge occurred between 1030 and 1100 hr. The peak flow during this storm was 0.58 cfs at 1200 hr (Figure 24). Nevertheless, meaningful data were collected, as shown in Figures 25 through 27. Figure 25 indicates that there were more suspended solids early in the monitoring. Since these higher values occurred during the early portion of the flow curve, it is possible that they are due to the residue accumulated since the last storm. In any case, the values are no higher than the values for the early portion of the October 27th storm, nor are they high values.

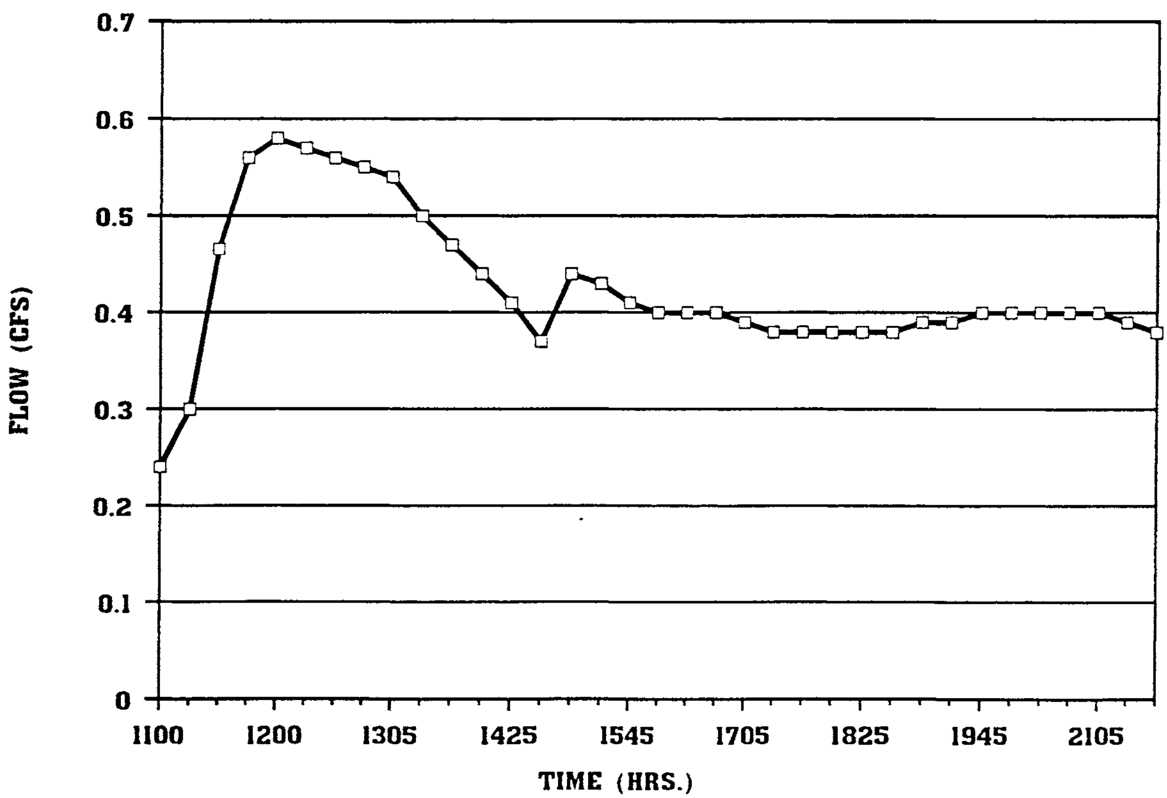

Figure 24. Flow curve (11/10/87).

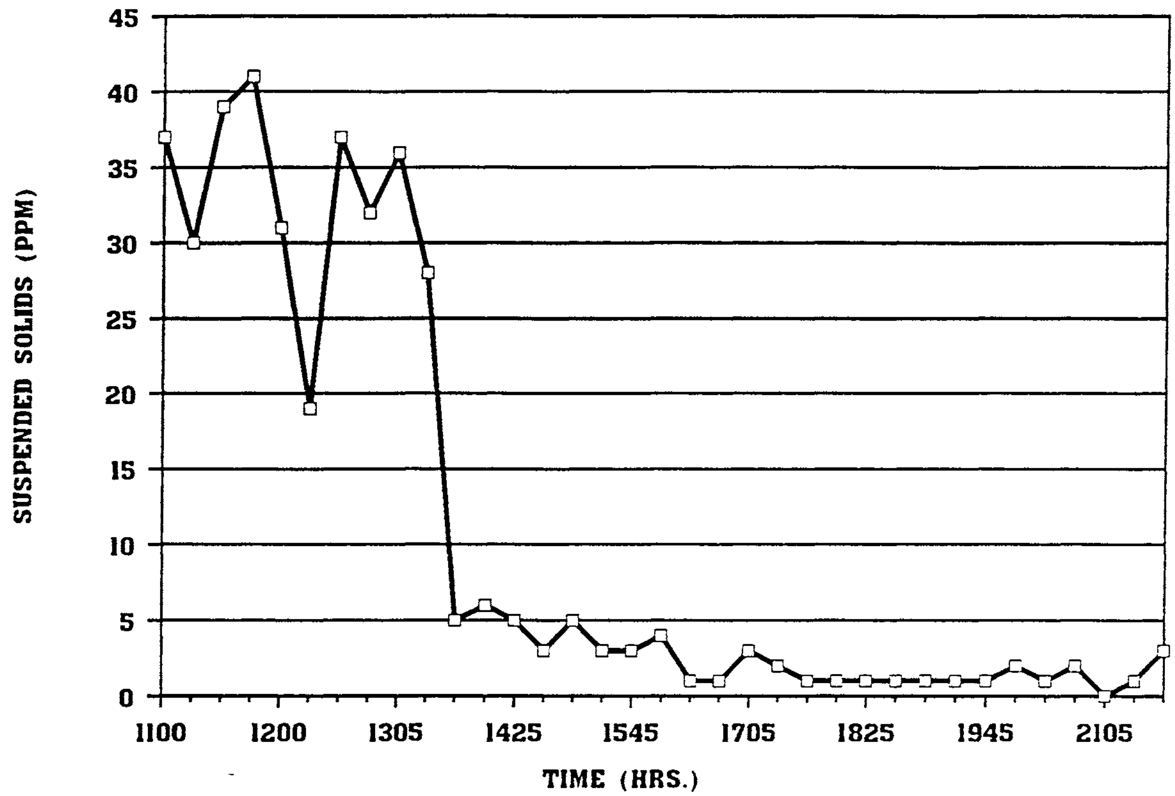

Figure 25. Suspended solids curve (11/10/87).

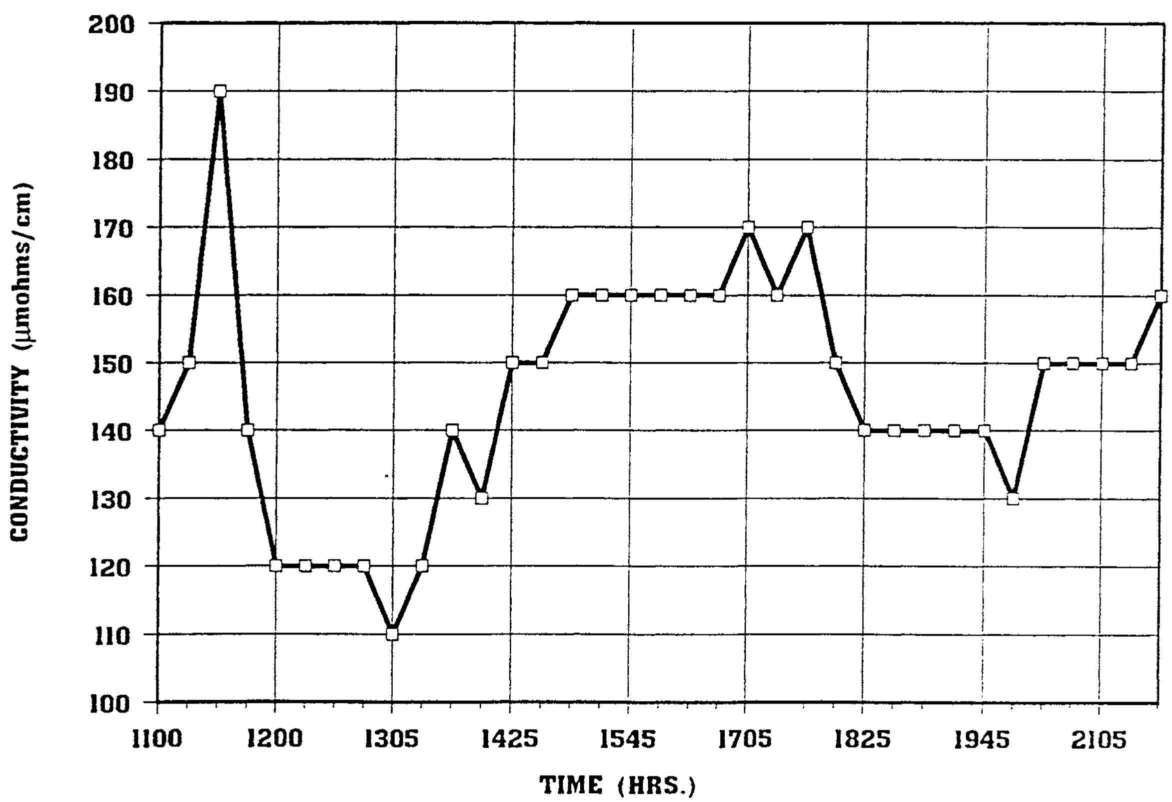

Figure 26. Conductivity curve (11/10/87).

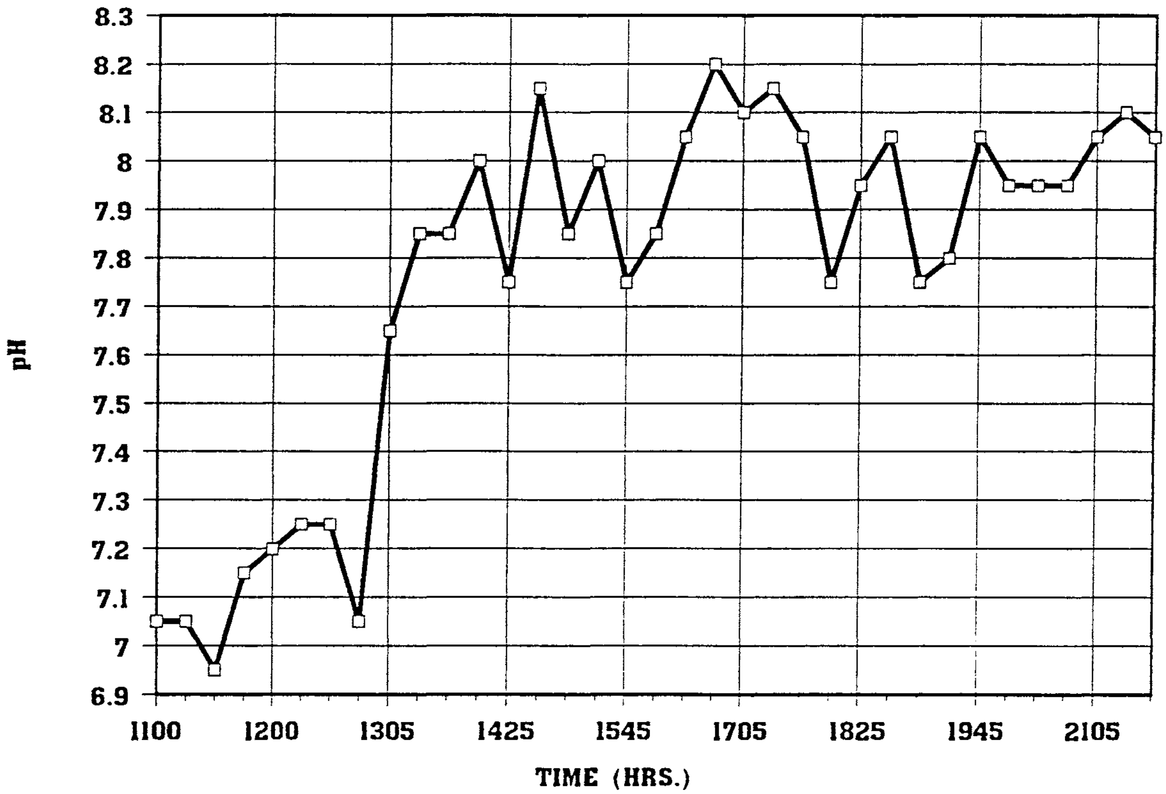

Figure 27. pH curve (11/10/87).

Figure 26 tends to indicate from the conductivity values that the effect of the residue in the box was evident in the first several samples. In addition, the conductivity and pH readings for this storm are similar to the October 27th storm (Figure 27).

Two water samples were secured at 1100 and 1300 hr to determine oil and grease content. These samples were preserved and processed in a timely manner. The oil and grease content for the 1100 hr sample was 0.01 mg/l and for the 1300 sample was 0.06 mg/l. The oil and grease content of both samples was essentially nil since the accuracy of the test method is 0.2 mg/l.

On December 7, 1987, three water samples were collected from the runoff for an oil and grease determination. The samples were taken with minimum delay between samplings and after allowing the effluent from all the entering pipes to mix together. Proper preservation and storage procedures were followed with these three samples. The amount of oil and grease in the three samples was determined to be 0.13, 0.47, and 0.73 mg/l. These values indicate that the variability of the test is more than the value for the results for the November 10th storm.

After Opening

Due to cold weather and other problems, usable data from storms were not secured again until April 18, 1988. At that time, the parking lot had been open for public use for approximately 4 months, with 80 vehicles being parked in the lot.

This first storm after opening of the lot for public use did not provide very good data for analysis. The storm was small (0.31 in of rainfall), there was a 3-hr stoppage of rainfall in the middle of the storm, and the sampling terminated just as the rain stopped due to the cycle being completed through the bottles (Figures 28 through 32). With the 3-hr stoppage in the middle of the monitoring, one could claim that Figures 28 through 32 cover two storms, and that would be a valid claim. However, it was decided by this researcher that the monitoring period covered the two periods of rain and should be displayed and discussed as a single storm. Even though the storm was not a very good storm for analysis, there are several observations of data that need to be noted.

The flow curve (Figure 29) indicates a slight increase (0.01 cfs) in the flow at 1410 hr when the rain had stopped approximately 1 1/2 hr earlier. This indicated increase is small and probably due to an equipment problem, such as a slight movement of the pressure sensing tube. In any case, it is highly unlikely that the flow would increase in the box when there had not been any rain falling on the parking lot for 1 1/2 hr. As with the previous storms, the values for suspended solids were low and in the same range (20 to 50 ppm). The pH values (Figure 31) were also very similar to the results for the previous storms. However, the conductivity curve (Figure 32) presents several interesting data points. During the early stages of the storm, there were two occasions on which the conductivity showed significant increases. These values cannot be considered anomalies because the trend of the curve for prior data points leads to the peak values of 680 and 1620 μmohms/cm.

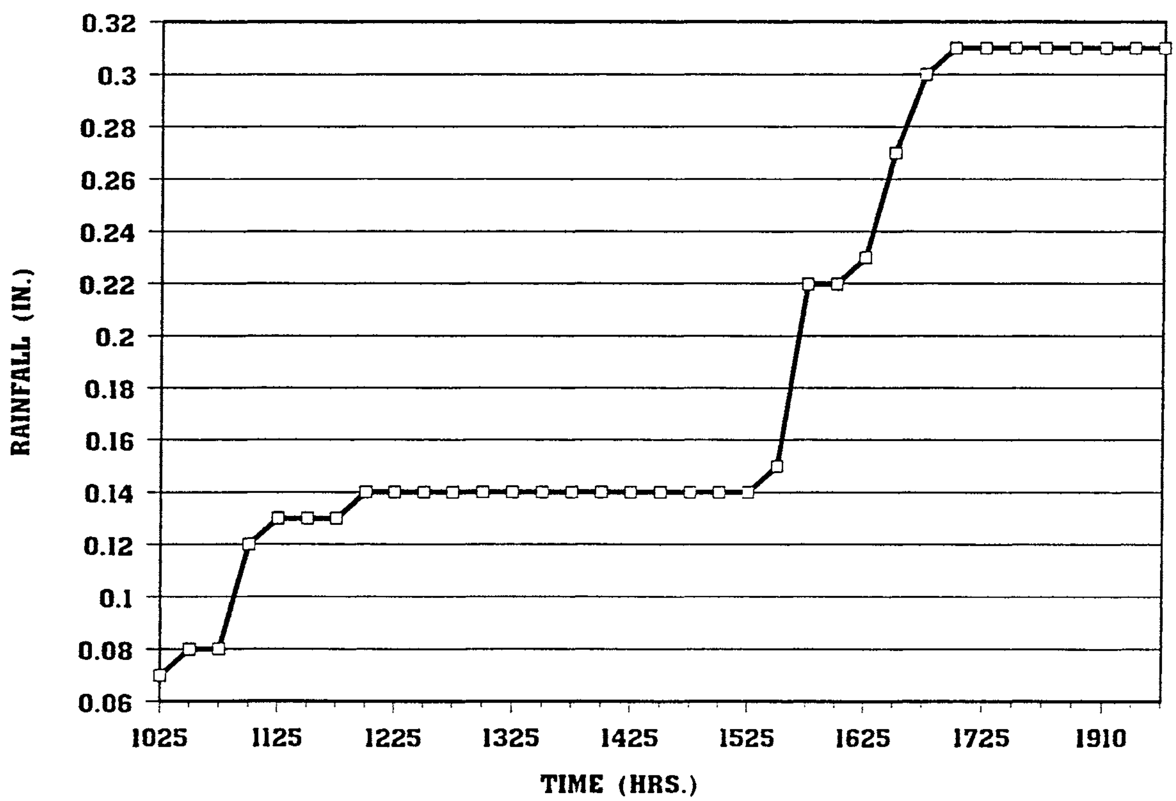

Figure 28. Rainfall curve (4/18/88).

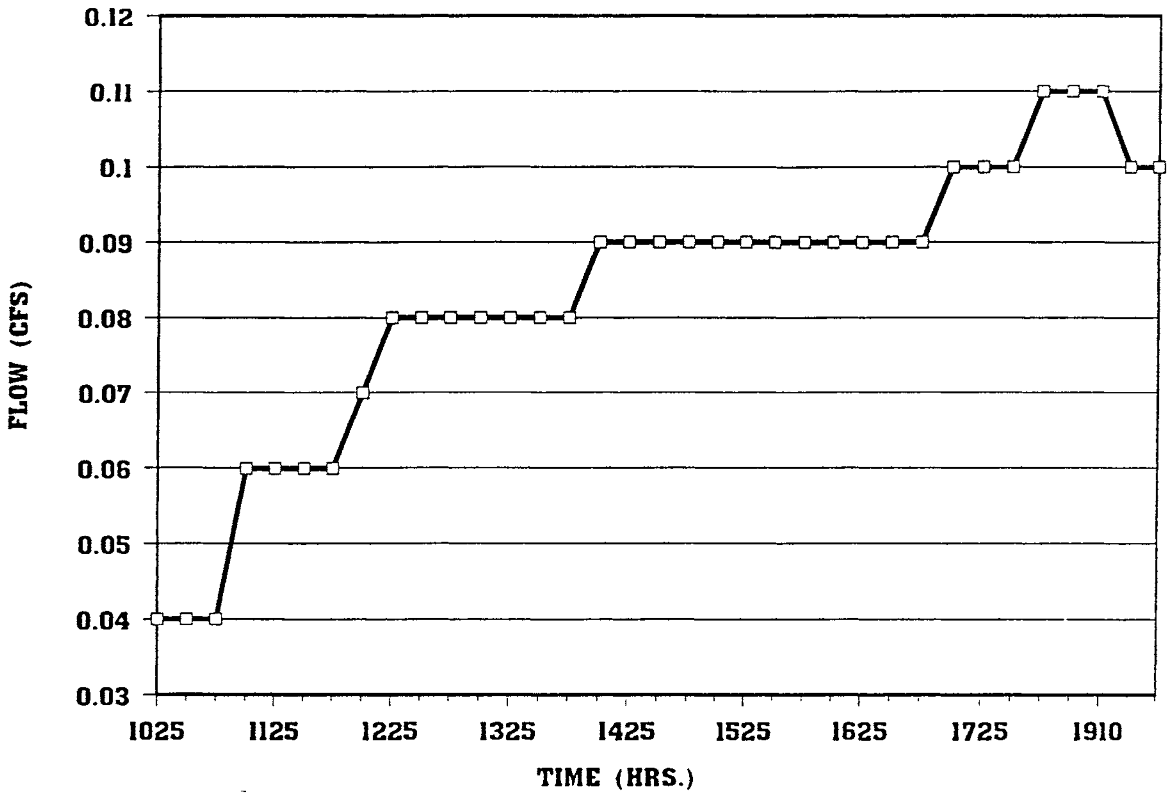

Figure 29. Flow curve (4/18/88).

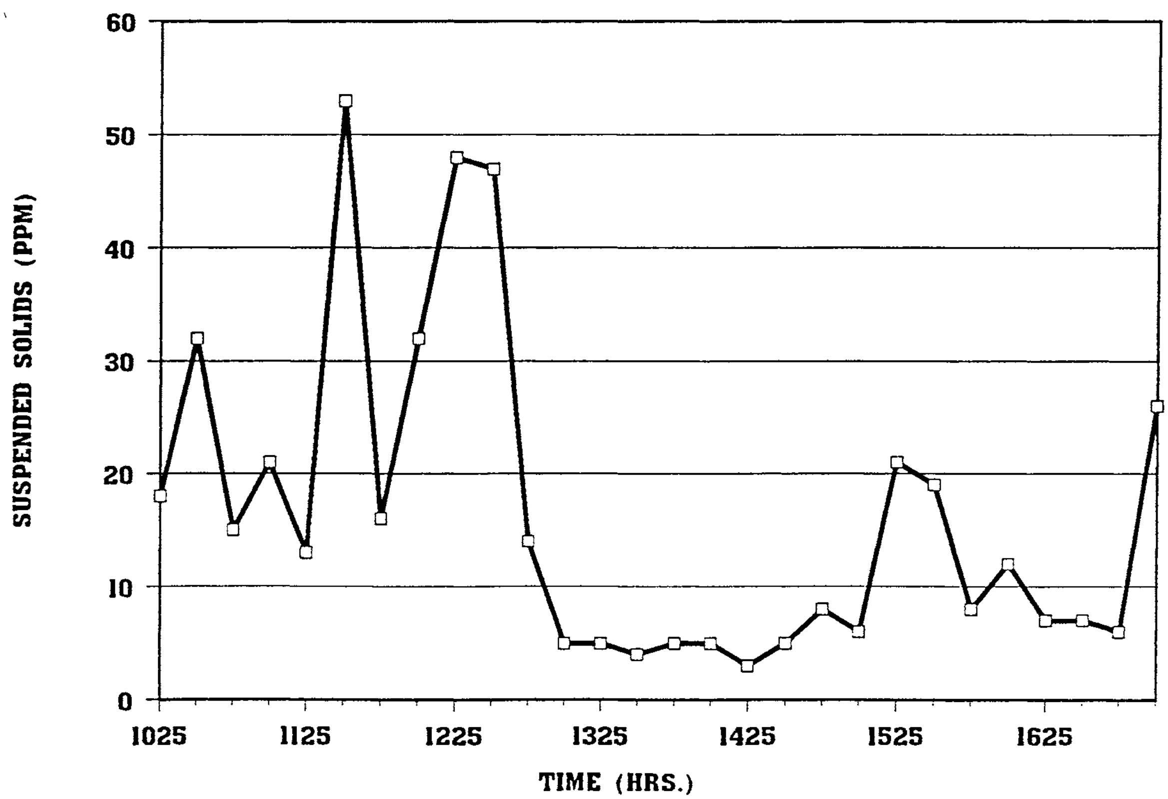

Figure 30. Suspended solids curve (4/18/88).

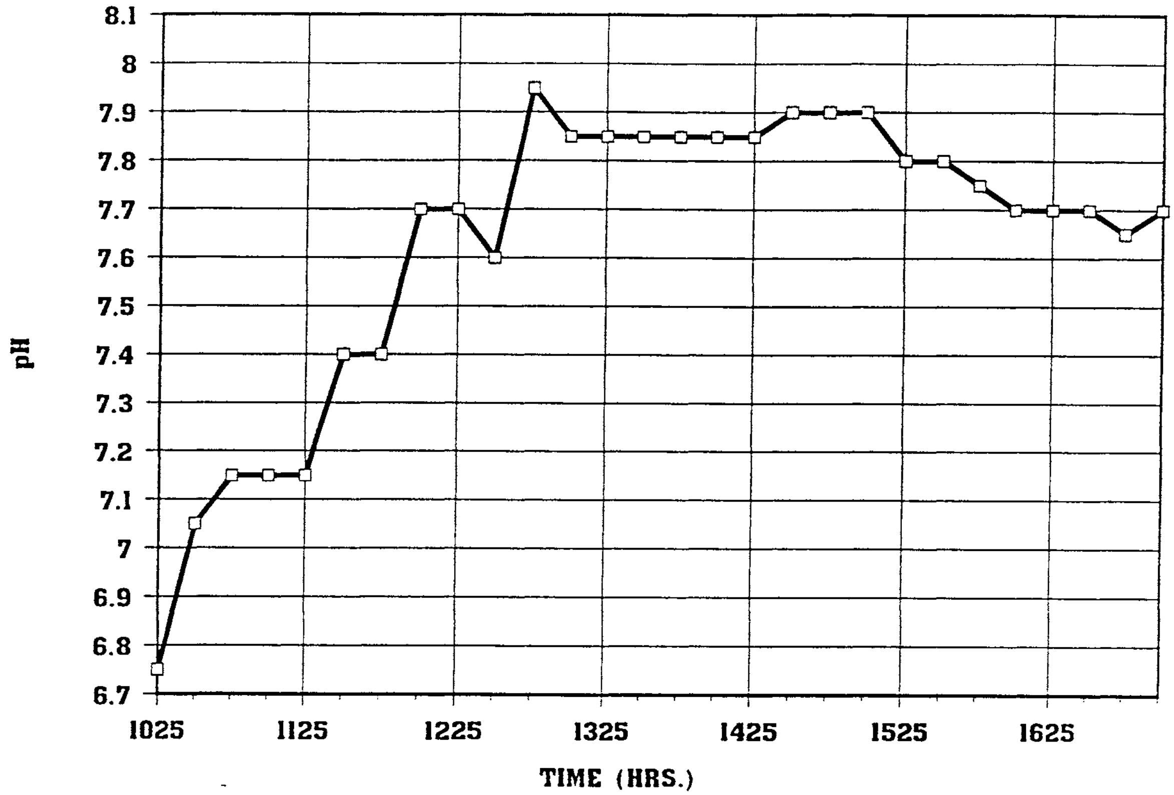

Figure 31. pH curve (4/18/88).

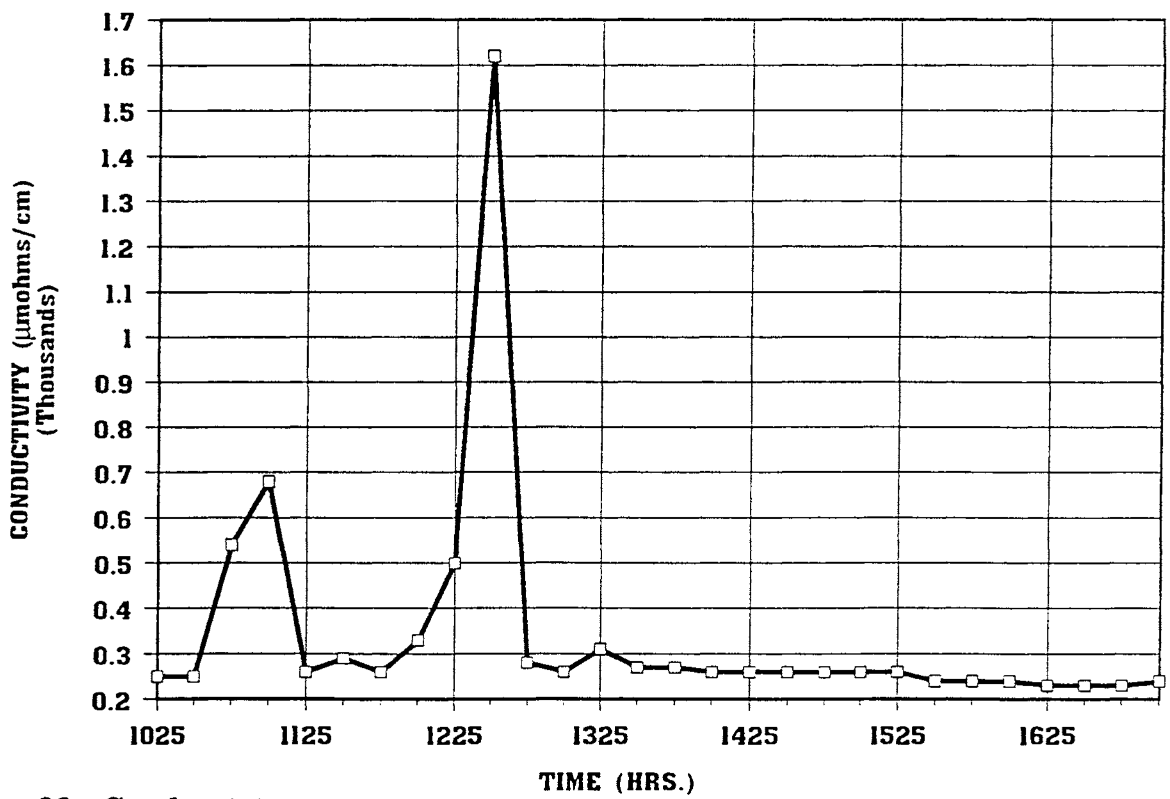

Figure 32. Conductivity curve (4/18/88).

During the later intense rainfall, starting at 1525 hr, no significant changes in the conductivity of the runoff were evident, which would suggest that there were ions in the water that caused the peaks during the earlier periods.

On May 4, 1988, rain started at 1500 hr. VTRC personnel visited the parking lot on the morning of May 5th at 0700 hr anticipating the collection of water samples for processing. They found no samples. The sampler was immediately turned on and found to be inoperable. After another sampler was secured and installed, the first sample was obtained at 1420 hr. In addition to the sampler malfunction, the bubbler tube for the flow meter came unfastened and caused the flow readings prior to 1420 hr to be in error. The total rainfall from 1500 hr May 4th to 1420 hr May 5th was 1.35 in. At no time during that storm were samples obtained. The initial portion of Figure 33 indicates the decrease in flow into the box from the rainfall.

Around 1420 hr on May 5th, it started raining at the parking lot. Figure 34 shows the rainfall curve for this storm. The curve indicates that it steadily rained for about 11 hr but at various intensities. At 2020 hr, the most intense portion of the rain occurred. Comparing the rainfall curve with the flow curve (Figure 33), it appeared that the porous pavement delayed the runoff 30 to 45 min. The peak runoff (0.60 cfs) occurred at 2150 hr, and the intense rainfall started at 2020 hr and ended at 2150 hr. The earlier portion of the flow curve (prior to 1550 hr) is a result of the unmonitored 1.35-in storm on May 4th.

The values for the suspended solids in the runoff from the porous pavement were low, as with previous storms (Figure 35). The first two data points on the

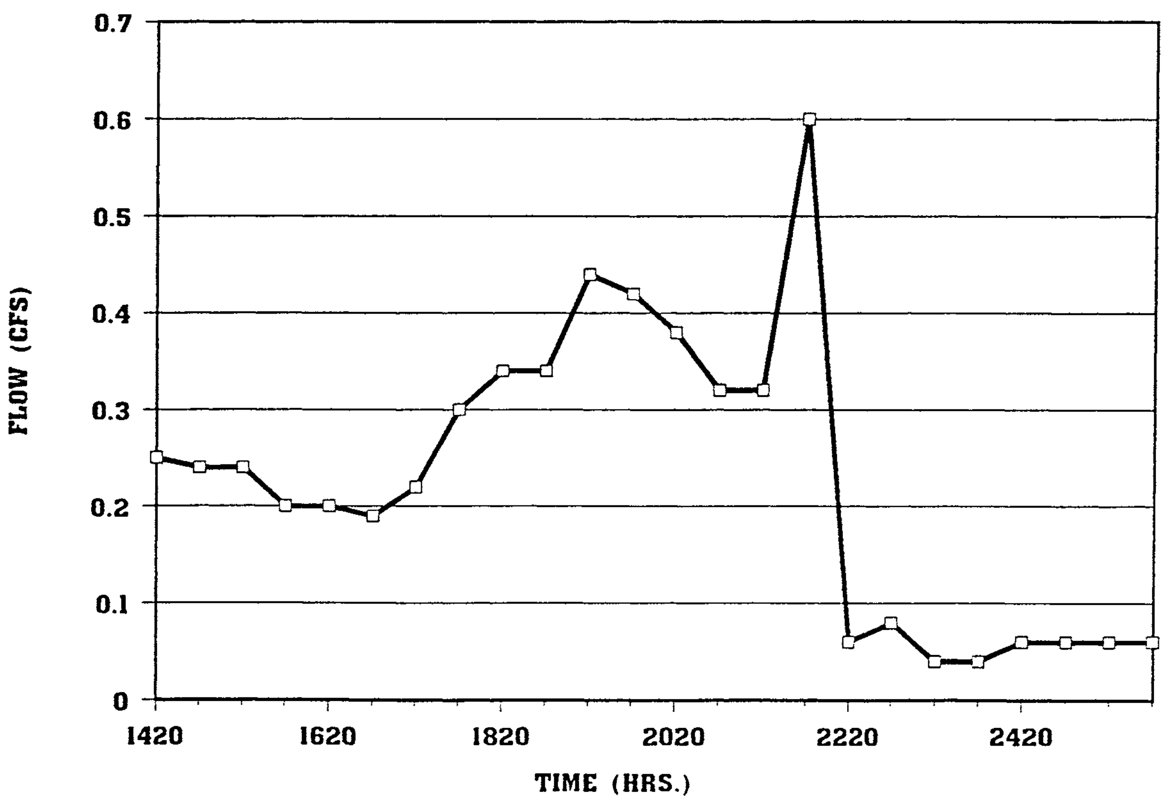

Figure 33. Flow curve (5/5/88).

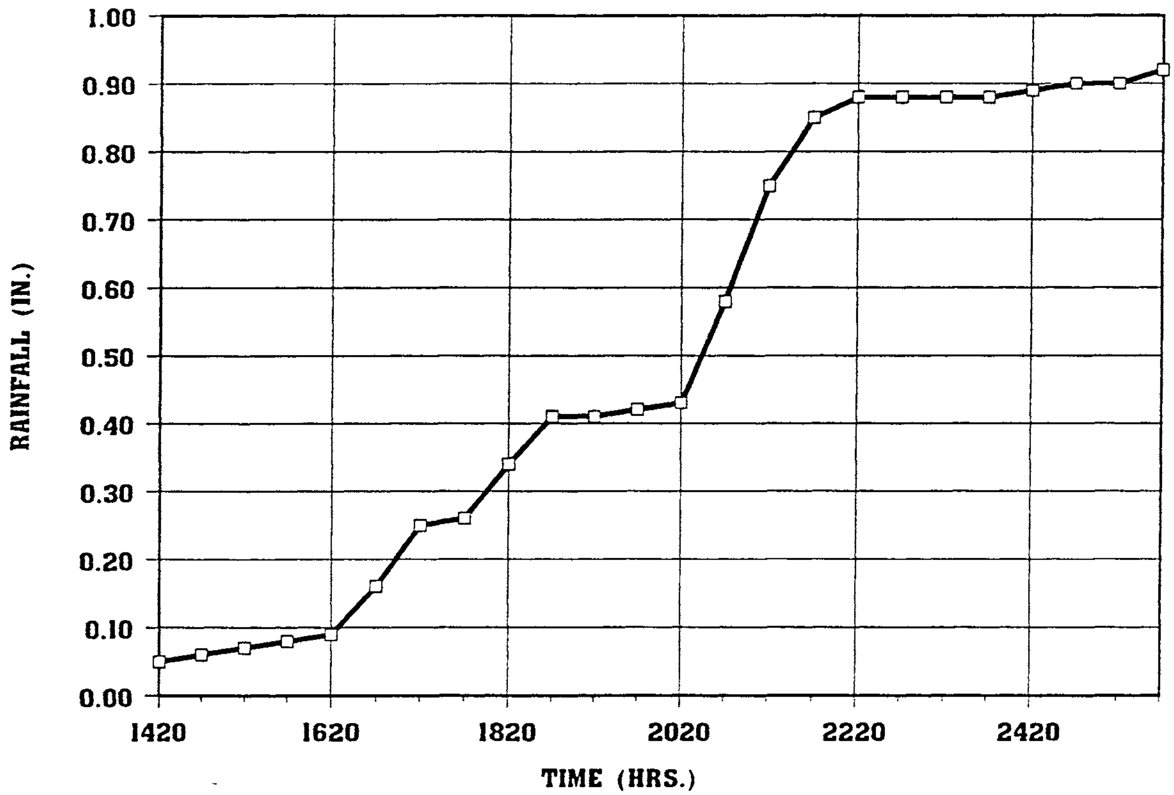

Figure 34. Rainfall curve (5/5/88).

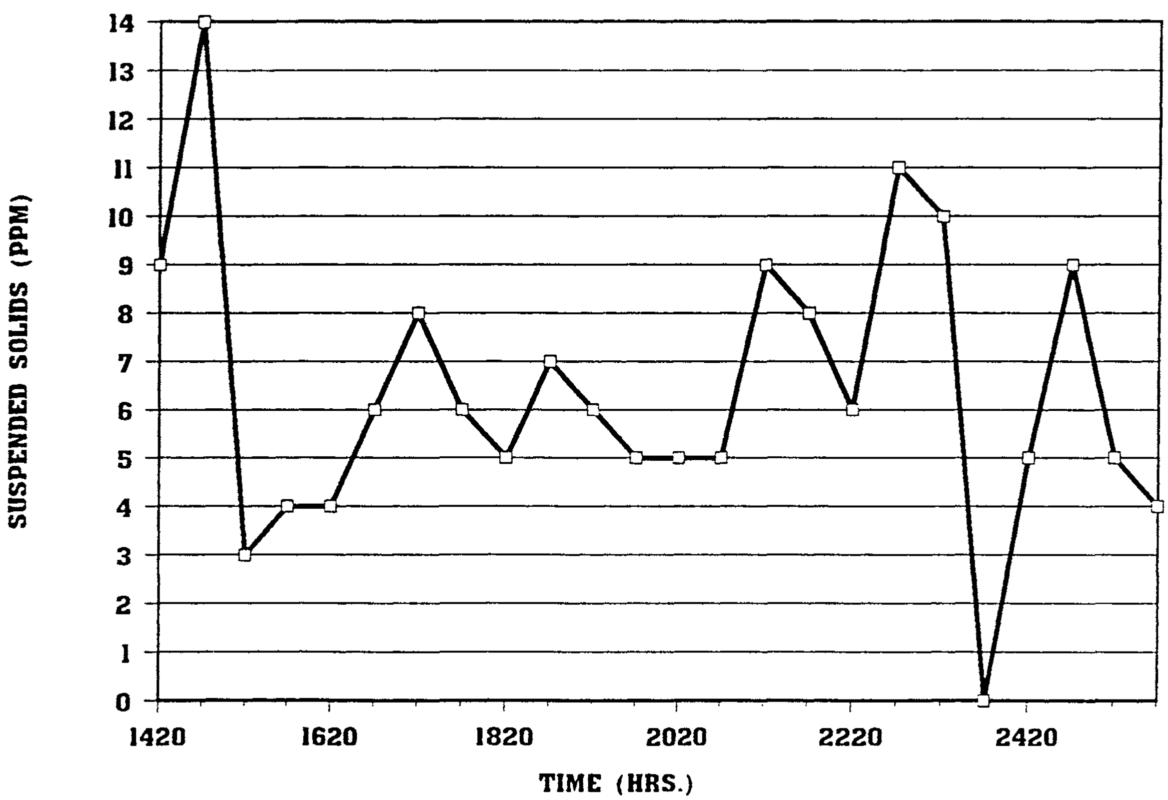

Figure 35. Suspended solids (5/5/88).

suspended solids curve show the higher values for suspended solids as the flow was decreasing from the previous rainfall missed. Also, the trend of the curve indicates a slight increase in the values for suspended solids as the flow increased around 2150 hr.

Figures 36 and 37 are the pH and conductivity curves, respectively, for the May 5th storm and are similar to those for previous storms. However, the conductivity curve indicates increases when the flow from the pavement system decreases (i.e., ion concentration is increasing).

A 0.76-in storm was monitored the following day (May 6th). The results are similar to the results for other storms (Figures 38 through 42).

A water sample for the determination of oil and grease content was collected on May 6th and tested. There was 0.01 mg/l of oil and grease in the sample. As shown earlier, this value is much less than the variability of the test method.

The last storm monitored occurred on May 17th. The total rainfall was slight (0.20 in). Results for this storm are similar to those for previous storms (Figures 43 through 47). Oil and grease results from two samples indicate very low amounts (0.06 mg/l for each sample).

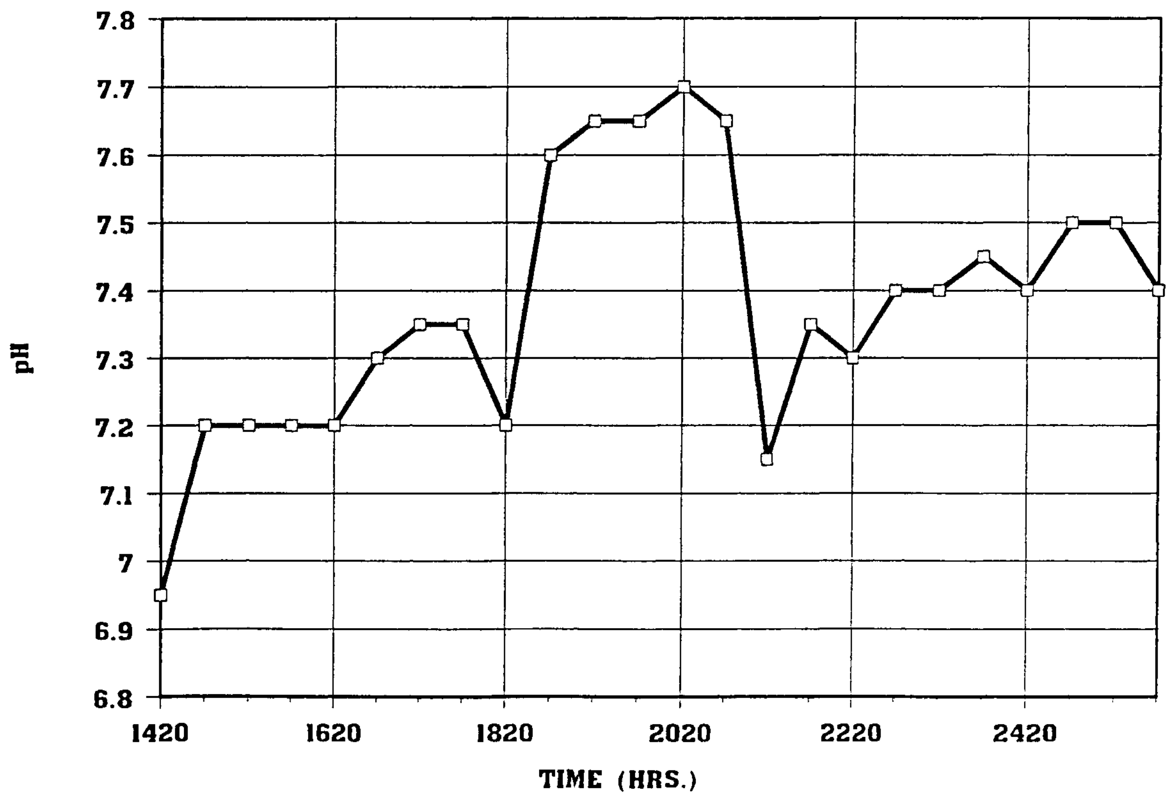

Figure 36. pH curve (5/5/88).

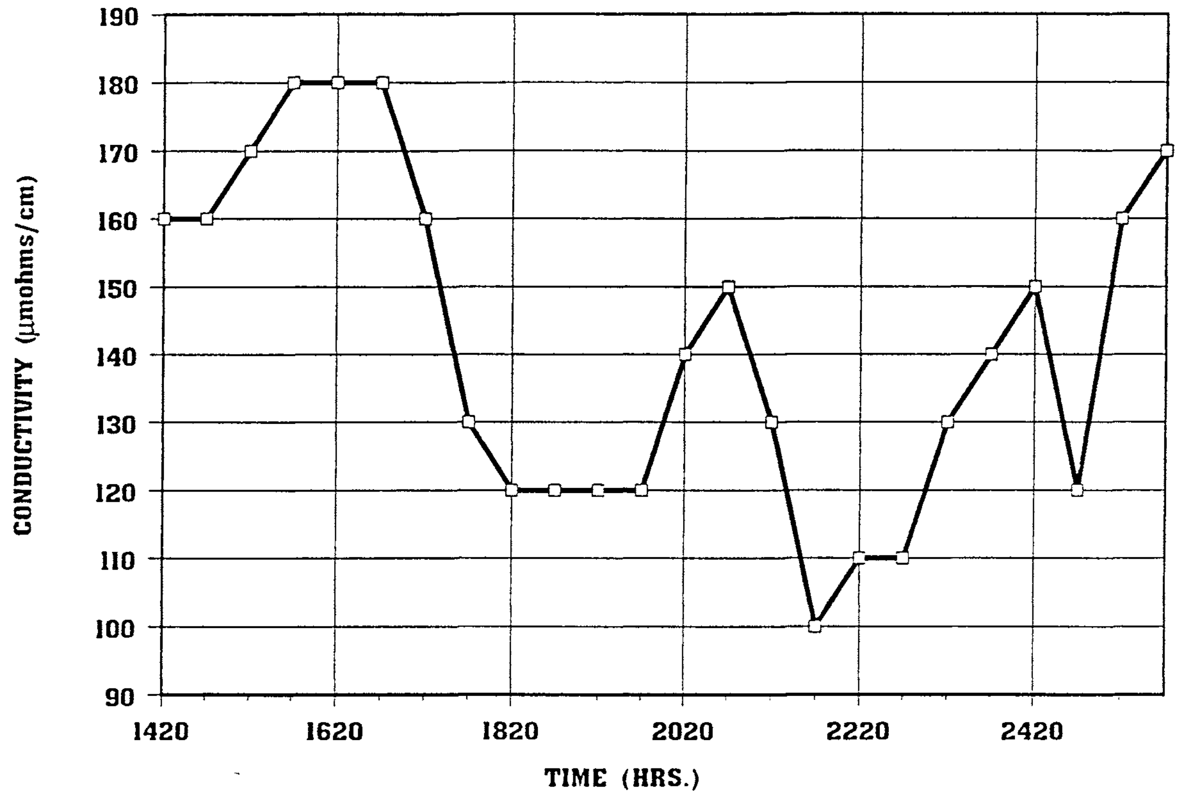

Figure 37. Conductivity curve (5/5/88).

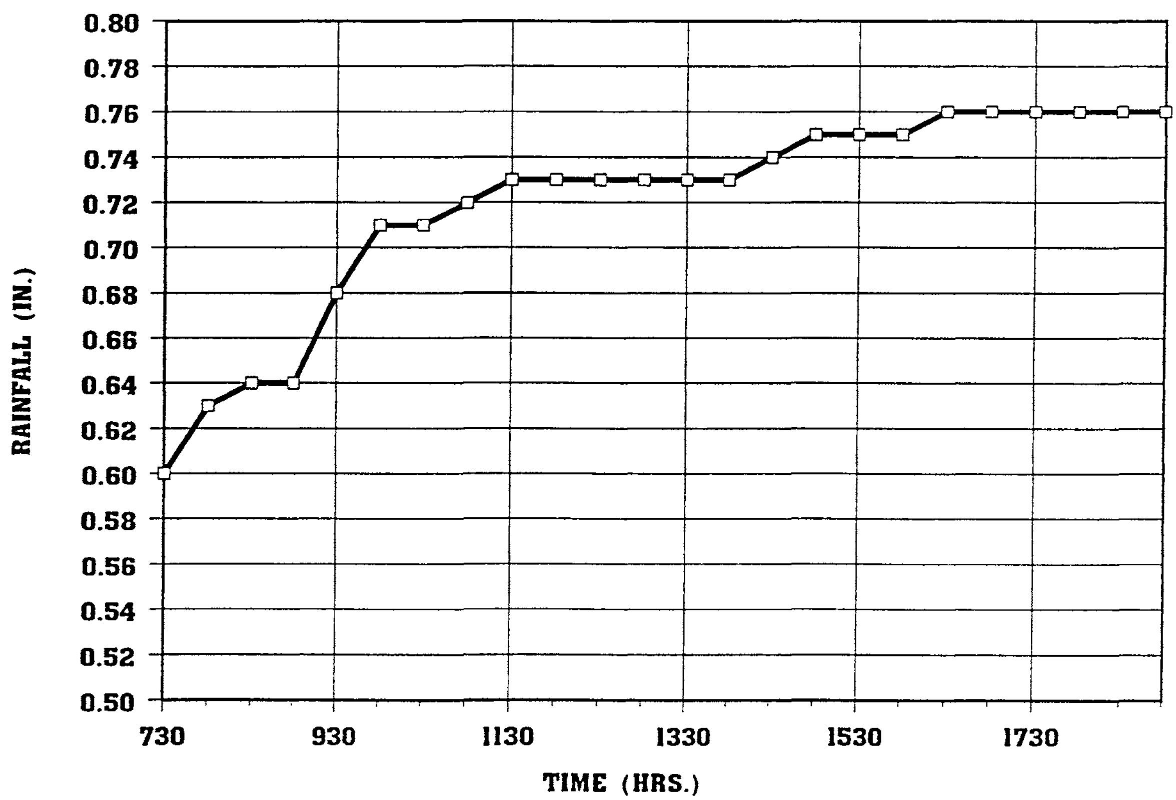

Figure 38. Rainfall curve (5/6/88).

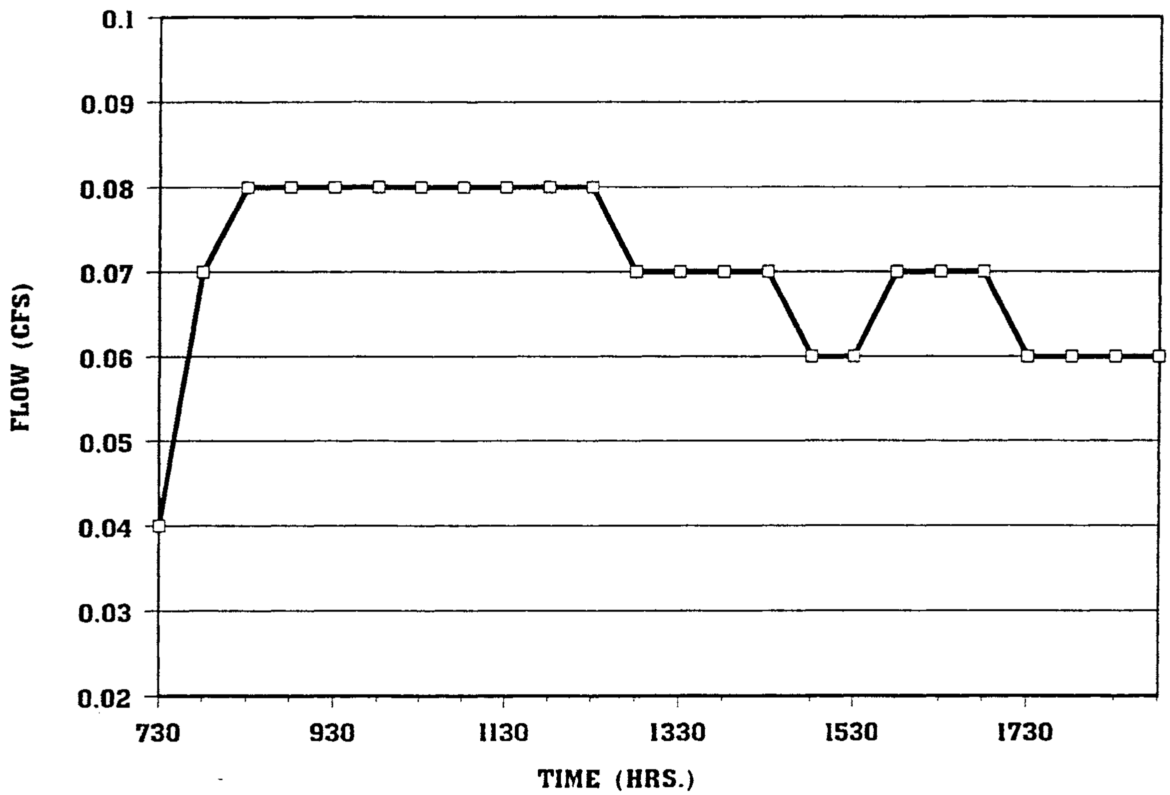

Figure 39. Flow curve (5/6/88).

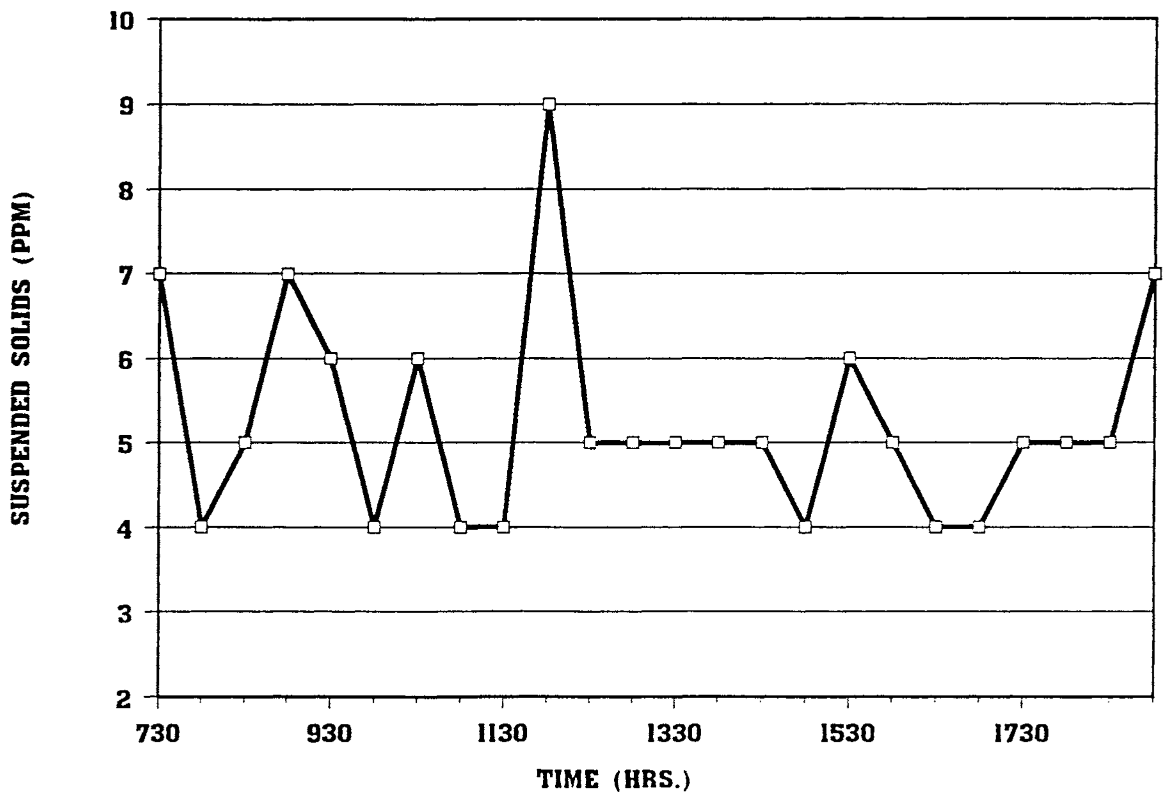

Figure 40. Suspended solids curve (5/6/88).

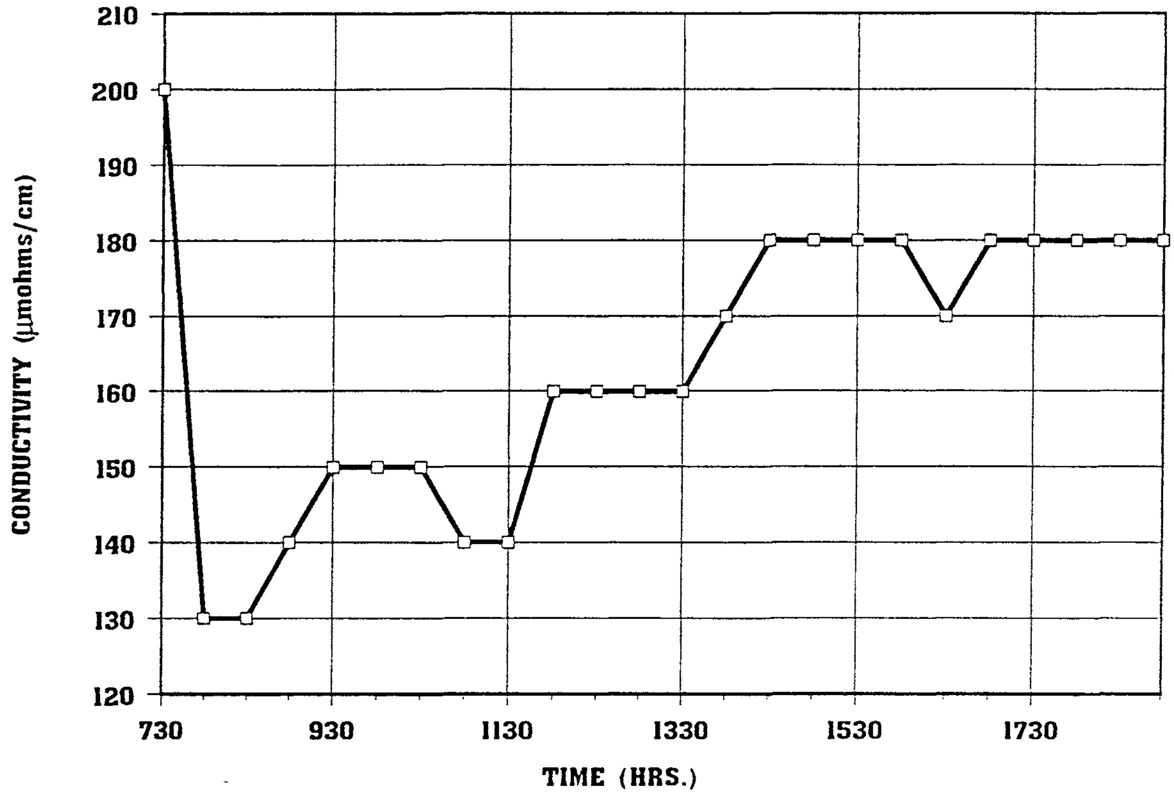

Figure 41. Conductivity (5/6/88).

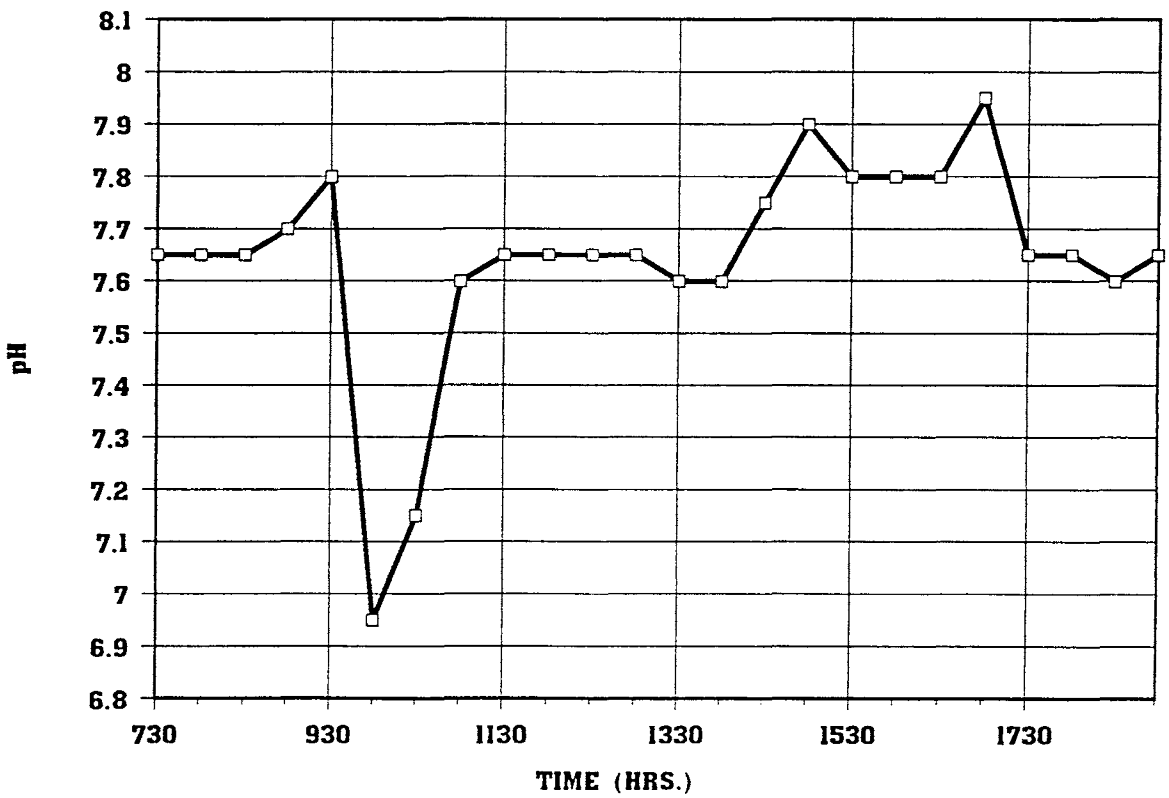

Figure 42. pH curve (5/6/88).

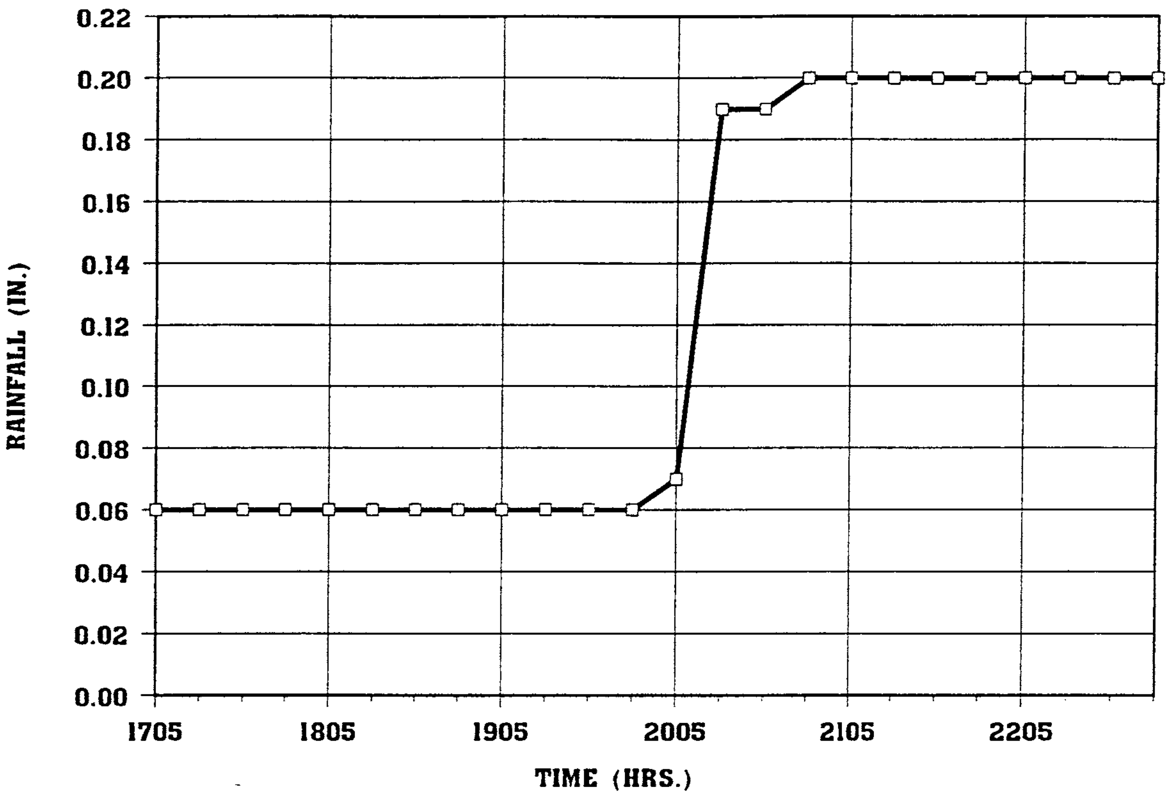

Figure 43. Rainfall curve (5/17/88).

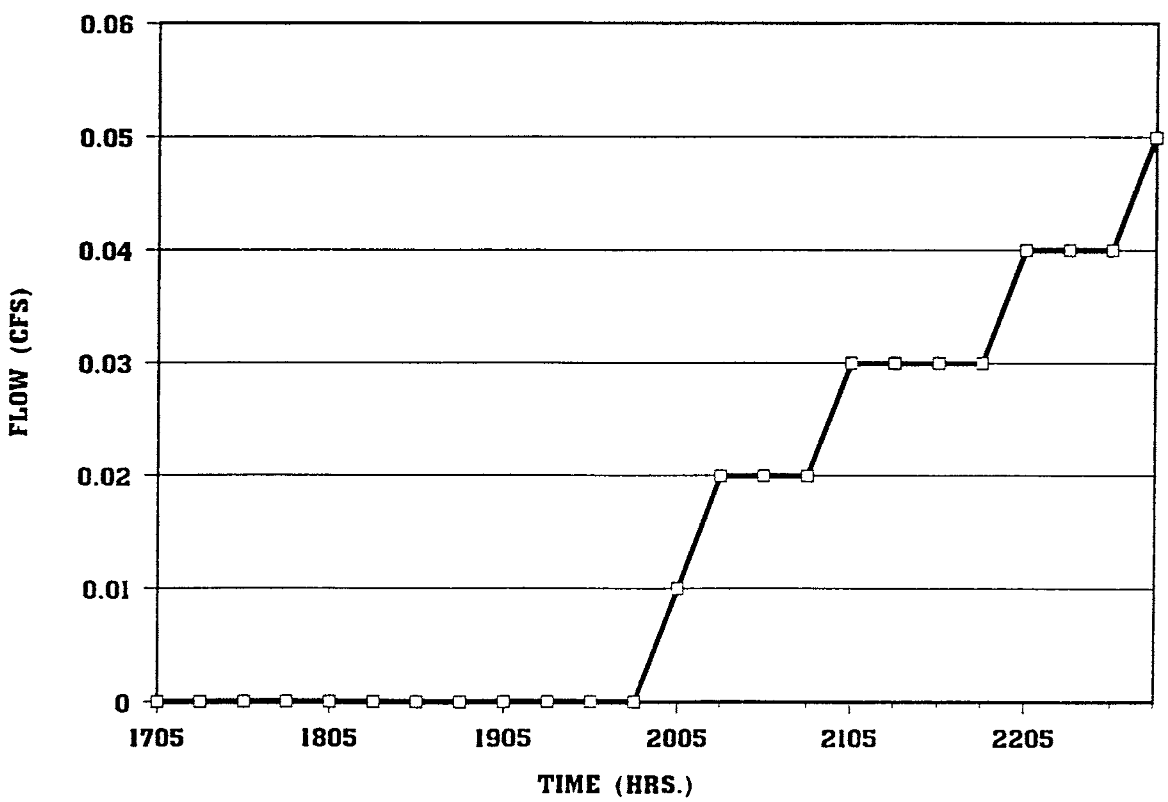

Figure 44. Flow curve (5/17/88).

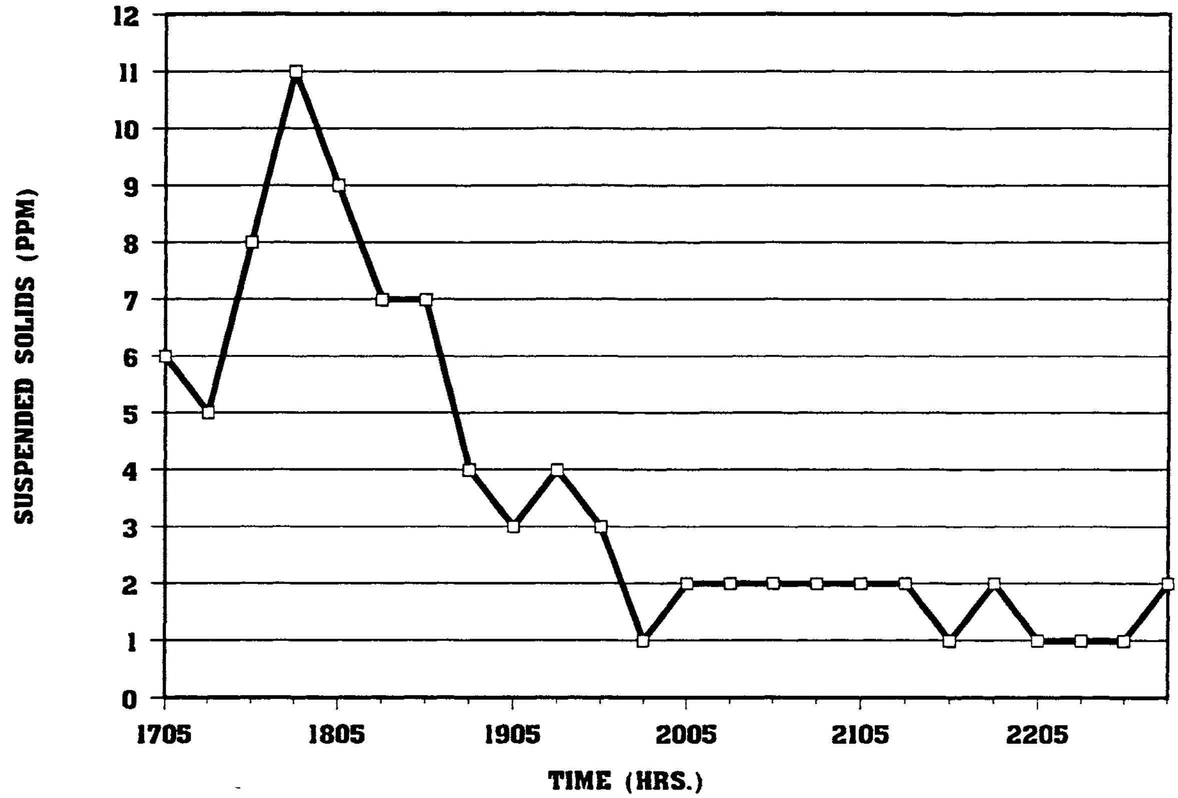

Figure 45. Suspended solids (5/17/88).

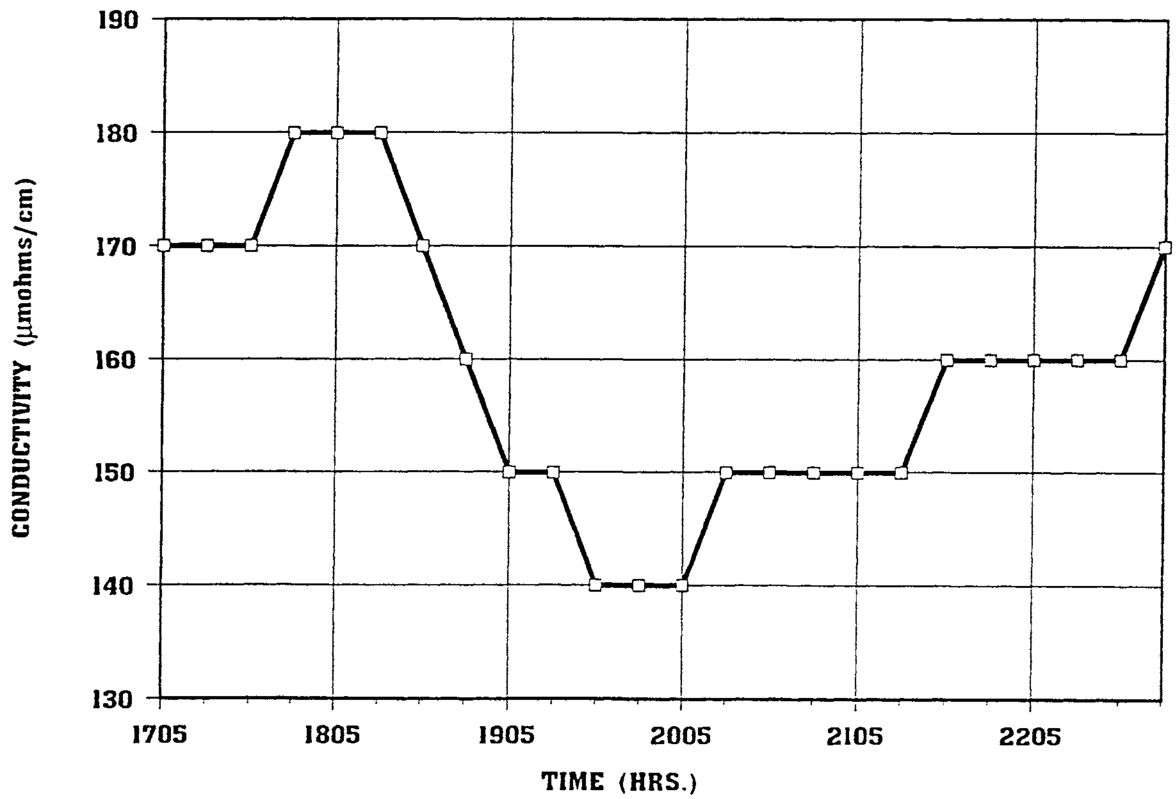

Figure 46. Conductivity curve (5/17/88).

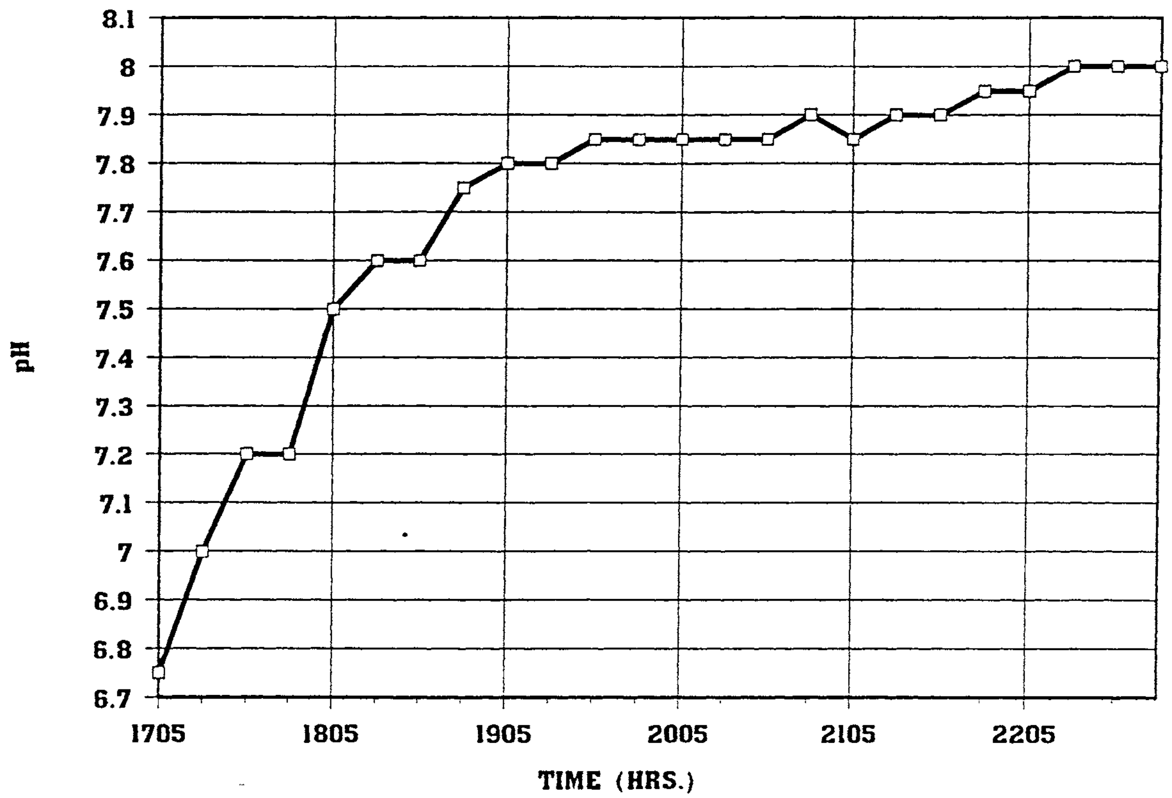

Figure 47. pH curve (5/17/88).

Reservoir Storage

The detention and storage of the runoff in the aggregate reservoir were much less than what was desired and what the reservoir was designed to provide. The standpipes shown in Figure 4 (their locations are shown in Figure 8) were installed to monitor the depth of water in the aggregate materials: 37,208 ft^3 of storage was provided in the No. 57 stone, with additional storage (which was known to be available but not designed into the system) in the No. 19 stone. The depth of water in each standpipe was checked at least once during and after each storm. At no time was water found in standpipes 4 through 9 (Figure 8). In standpipes 1 and 3, there were small quantities (0.25 to 0.50 in of depth) of water present during the larger storms of longer duration (such as the storm on May 4–6). The maximum depth measured at any time was in standpipe 2. During the storm on May 4–6, the water rose to a height of 2 in.

With storage being one of the objectives for selecting porous asphaltic pavement and storage appearing to be lacking at this site, this researcher began to ascertain the reason. After investigating the site, it was determined that the water passing through the stone layers was mostly being forced back up through the system and the adjacent soil on the lower edge of the lot (below standpipes 1 through 3) by the hydrostatic head being created by the slope across the parking lot. The hydrostatic head on the water was calculated to be approximately 360 lb/ft^2. Water

Figure 48. Blowout and runoff channel at lower end of parking lot.

was bubbling up through the system and the adjacent soil and running across the vegetated area between the lot and Rt. 29 to a drop inlet in the middle of the grassed area (Figure 48). The loss of water from the system through the lower end also caused the flow curves to respond differently. For example, Figure 19 shows little increase in flow when the most intense portion of the rain storm had just occurred (Figure 18).

During a very intense rainstorm that occurred prior to the installation of the monitoring equipment in early September 1987, VTRC personnel observed large quantities of water entering the box through the two 6-in pipes from the aggregate base materials (Figure 49). Both pipes were flowing full during this storm. At that time, no runoff appeared to be bubbling up from the lower end of the lot or running across the surface of the lot. During this very intense rain, measurements were taken in the eight standpipes. Only standpipes 1 through 3 had any water in them. Standpipe 3 had 2.5 in, and standpipes 1 and 2 had 6 and 7.5 in, respectively.

After taking these measurements and observing this storm and the happenings at the parking lot, this researcher calculated the total capacity of storage available before the collected water would start to come back through the pavement surface. It was determined that only 21 percent of the total design capacity of the storage reservoir could ever be used if the rest of the system performed as designed. With the highly permeable materials being used in this design (Figure 50), the runoff would start to come to the surface when the two drainage pipes are running full and the water level has reached a point 56 ft back from the lower end of the lot.

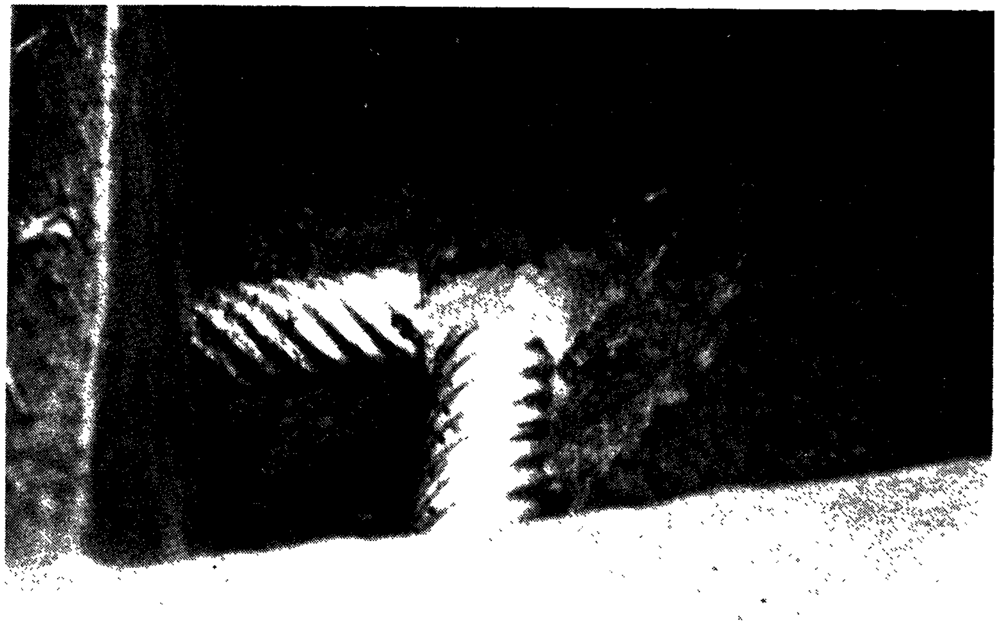

Figure 49. Flow through 6-in pipes during storm in September 1987.

Figure 50. Appearance of S-5 and S-8 during large, intense storm in September 1987.

Flow calculations indicate that each of these 6-in pipes should carry approximately 0.5 cfs at maximum flow, or a total of 1.0 cfs at any one time from the aggregate materials. After this early, unmonitored, heavy rain, at no time were the two pipes running close to full flow nor was the depth of the water in the reservoir that deep again. This researcher suspects that a blowout at the lower end of the lot occurred during this storm and the runoff was not being detained as designed because of the high hydrostatic head created due to the slope across the lot. Even if the blowout had not occurred, the hydrostatic head created at the lower end of the lot would have forced the water back up through the pavement. The flow curves generated during this study confirm this idea because the maximum flow never exceeded 0.6 cfs for any of the monitored storms. In addition, the flow curves would increase at a low rate (Figure 19, around 1900 hr) or decrease in flow (Figure 24, around 1300 hr) immediately after an intense period of rainfall. With the flow curves indicating these varied responses, it would suggest that all the water collected in the system is not being removed by the intended or designed ways and that the drainage is occurring by other means (i.e., the blowout location).

Structural Analysis

Within 4 days after placement, deflection readings were taken on the porous pavement and access roadway to ascertain the pavement's structural strength. These initial readings (August 28, 1987) indicate that the average deflection on the porous pavement is approximately twice that of the normal pavement design sections. The average equivalent asphalt thickness of the porous pavement was about one-half that of the normal design sections (Table 5).

To determine if there were any effects of winter weather and traffic on the porous pavement, deflection readings were taken in early spring through 1989 (Table 5). The results indicate that both mixtures have gained strength with time. This gain generally occurs with all pavements and should have been expected with these two asphaltic mixtures as well.

In the spring of 1989, VTRC replaced the dynaflect trailer with a newer model. Table 6 shows the results obtained for the parking lot pavement with the newer

Table 5
STRUCTURAL RESULTS FOR TWO ASPHALTIC MIXTURES (8/28/87)

	Average Deflection (in)		Average TI (in)	
Date	Normal	Porous	Normal	Porous
August 28, 1987	0.030	0.059	7.2	3.0
March 24, 1988	0.024	0.049	9.0	3.7
March 30, 1989	0.023	0.045	9.1	3.9

TI = average equivalent asphalt thickness.

Table 6
STRUCTURAL RESULTS: NEW TRAILER

Date	Average Deflection (in)		Average TI (in)	
	Normal	Porous	Normal	Porous
May 24, 1989	0.048	0.095	6.2	1.9
June 25, 1991	0.046	0.091	6.3	2.1

TI = average equivalent asphalt thickness.

trailer (May 24, 1989). These results were generated from data acquired approximately 6 weeks after the last data collected in Table 5, which was with the old trailer. The average deflection for the new trailer was approximately double that obtained with old trailer, and the average equivalent asphalt thickness was approximately one-half of the earlier results.

In June 1991, deflection readings were acquired with the new trailer. The results generated from that effort are shown in Table 6. It is evident that the structural capacity of the two asphaltic mixtures has not deteriorated over the last 2 years.

CONCLUSIONS

1. Use of the parking lot has steadily increased since its opening. In 1991, the lot averaged more than 50 percent occupancy on a fair weather day compared to the initial rate of 34 percent. On inclement days, the occupancy is slightly higher.

2. The soil at the site was not suitable for this type of stormwater management technique. The soil has an extremely low permeability and will not soak up the collected water. In addition, the content of clay and silt was high (greater than 40 percent –200 material), and the soil is susceptible to freezing. For a soil with less clay, it is recommended that auxiliary drainage be provided; but no auxiliary drainage was provided.

3. Results for pH, suspended solids, conductivity, and oil and grease were similar for storms monitored before and after the lot was opened for public use. Generally, the pH of the runoff was between 7 and 8, the suspended solids between 20 and 50 ppm, the conductivity between 150 and 200 µmohms/cm, and the oil and grease content less than 5 mg/l. Some of the fluctuations and variability are flow dependent and due to fallout accumulating in the monitoring box between storms.

4. No structural damage has been done to the system. Both asphaltic mixtures have gained in strength with age.

5. Discharge into the monitoring box generally occurred after 0.25 in of rainfall. The two discharge pipes never reached full flow after a blowout at the lower end of the lot during an early unmonitored storm. No water was stored in the aggregate reservoir 24 hr after a storm ceased and generally in less time than that.

6. Because of the slope on the parking lot, only 21 percent of the aggregate reservoir was available for storage.

RECOMMENDATIONS

In the design of porous asphaltic pavements, the following are recommended:

1. The permeability of the underlying soil layers should be determined and considered as the main criterion in the design.

2. The subgrade and riding surface of a porous asphaltic pavement should not have any slope.

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600